Polder terminal

By: Kasper Lendering
Polder terminal
A risk based design

Author:
Kasper Lendering
1304097

Date:
October 19, 2012

Course:
Master Hydraulic Engineering and Flood Risk
Civil Engineering Technical University Delft
Graduation work
CIE 5060

Graduation committee
Prof. drs. ir. J.K. Vrijling   TU Delft
Prof. dr.ir. S.N. Jonkman   TU Delft
Dr. ir. J.G. de Gijt       TU Delft
Dr. ir. O.A.C. Hoes       TU Delft
Ir. D.J. Peters           RHDHV

* Front page illustration source: Royal HaskoningDHV Maritime and Waterways Business Line
I. Preface

This report is the result of a graduation project of the Master Hydraulic Engineering; it contains a risk based design of a Polder terminal. The graduation project is made within the framework of the course CTS060-09 of the Technical University of Delft. This project forms the last phase of the specialization ‘Hydraulic structures and flood risk’ to obtain the title Master of Science in Hydraulic Engineering.

The thesis committee is responsible for the guidance and evaluation of the student during the graduation project. This committee is lead by Prof. drs. ir. J.K. Vrijling of the department of hydraulic engineering at the Technical University of Delft. Assisting him will be Prof. Dr. ir. S.N. Jonkman and dr. ir J.G. de Gijt of the same department of hydraulic engineering and finally dr. ir. O.A.C. Hoes of the department of water management.

Because the project is carried out outside of the university, at Royal HaskoningDHV, the committee is joined by a member of the RHDHV: ir. D.J. Peters, who works for the structural department of the Maritime and Waterways group.

<table>
<thead>
<tr>
<th>Member</th>
<th>Role</th>
<th>Specialization</th>
<th>Company</th>
</tr>
</thead>
<tbody>
<tr>
<td>J.K. Vrijling</td>
<td>Chairman</td>
<td>Hydraulic structures</td>
<td>TU Delft</td>
</tr>
<tr>
<td>S.N. Jonkman</td>
<td>Day to day supervisor</td>
<td>Flood risk analysis</td>
<td>TU Delft</td>
</tr>
<tr>
<td>J.G. de Gijt</td>
<td>Member</td>
<td>Quay wall structures</td>
<td>TU Delft</td>
</tr>
<tr>
<td>O.A.C. Hoes</td>
<td>Member</td>
<td>Water management</td>
<td>TU Delft</td>
</tr>
<tr>
<td>D.J. Peters</td>
<td>Supervisor</td>
<td>Hydraulic structures</td>
<td>Royal HaskoningDHV</td>
</tr>
</tbody>
</table>

Firstly I would like to thank the whole committee for their help, support and guidance during the full extent of the project. Special gratitude goes out to S.N. Jonkman for his support, feedback and advice on numerous occasions during the project. Furthermore I would like to thank P. van Gelder for taking the time to check all mathematics. Not in the least I would like to thank M. Smits, L. Mooyaart, J.P. Verschuure and other colleagues of the Maritime and Waterways group of Royal HaskoningDHV who were involved during the project for their feedback and advice, which is much appreciated.

Off course I would like to thank all other persons not mentioned above for making this project possible.

Kasper Lendering

Delft, October 2012
II. Summary

Container trade has been growing rapidly in the last decades resulting in large container port expansions around the world. New ports are mostly constructed on low lying coastal areas or in shallow coastal waters. The quay wall and terminal yard are then raised to a level well above mean sea level to insure safety against inundation. The resulting ‘conventional terminal’ requires large volumes of good quality fill material often dredged from the sea which is costly.

Royal HaskoningDHV developed the concept of a container terminal with a “polder yard”. The yard would lie below the outside water level and be surrounded by a combined quay wall flood defense structure. The ‘polder terminal’ could save a large amount of reclamation cost but have higher damage in case of flooding resulting in an increased risk of inundation. Further, a polder terminal requires a water storage and drainage system, against additional cost. The exact amount of reclamation saving, inundation risk and water drainage cost are investigated in this report which will focus on a risk based design of a polder terminal.

Important conditions for the feasibility of a polder terminal are low conductive (impermeable) subsoil, which limits the amount of seepage water entering the polder, and high reclamation cost. A conceptual design is made based on a case study of a new port construction at Tuas, Singapore, where these conditions are present.

The risk of inundation of the polder terminal is divided in two main events: ‘small scale flooding’ (water hindrance), which occurs when the inflow of water in the polder exceeds the drainage and storage capacity of the polder, and ‘large scale flooding’ which occurs after failure of the flood defense structure.
Overtopping is the dominant failure mechanism which determines the required quay wall flood defense level to assure safety against large scale flooding. The contribution of other failure mechanisms (seepage, instability and calamities together 10% of overtopping failure) to the risk of inundation is negligible. During overtopping the inundation depth of the polder terminal is larger than that of the conventional terminal. Accordingly, the damage (D) and risk of the polder terminal is higher than the conventional terminal.

So given a certain terminal level, the polder terminal will have a higher risk of inundation and lower reclamation cost compared to a conventional terminal because of the lower polder terminal yard. An optimum between both situations can be found by minimizing the total cost, which is the summation of the investments and risk.

![Figure 3: Comparison of total cost polder terminal vs conventional terminal](image)

The minimal total cost and corresponding quay wall and polder level are found through a risk framework approach. The maximum polder depth (the deepest polder) corresponds with minimal total cost. The optimal quay wall height for the polder terminal is higher than that of the conventional terminal. The reduction of the reclamation cost proved to be larger than the increased risk of inundation and required water drainage cost of the polder terminal. The resulting total cost of the polder terminal could be significantly lower than the total cost of the conventional terminal, proving that the polder terminal could be an attractive alternative.

At Tuas, Singapore, the optimal total cost of the polder terminal are significantly lower than that of the conventional terminal; 1.500 million euro (or 500 €/m²) compared to 2.200 million euro (or 700 €/m²). The savings vary between 10% (200 mln euro) for a terminal yard at Mean Sea Level and 30% (700 mln euro) for a terminal yard at the uplifting boundary (-6.5 meter MSL), which is the maximum depth of the terminal.

The present value of the risk of the polder terminal (3 mln euro) proved to be 10 times larger that of the conventional terminal (0.3 mln euro). The increased risk was insufficient to counteract the large reclamation saving, even if multiplied by 5 to include a flood risk possible flood risk premium.
II. Summary

These results are independent of the total polder area, because both the reclamation cost and the damage cost depend on the total polder area.

The quay wall of a polder terminal does not only ‘traditionally’ retain soil and water but also serves as a primary flood defense structure. There is not one particular structure which is best suitable as a quay wall flood defense; this depends on local boundary conditions. A gravity structure is further investigated. It is concluded that the resultant load on the quay wall food defense structure of the polder terminal is lower than that of the conventional terminal, due to the lower retaining height. This results in lower cost quay wall cost, which results in a small overestimation of the total cost of the polder terminal (5%).

The maximum polder depth depends on the failure mechanisms piping, uplifting and instability. The definitive boundary of the polder depth is set by the uplifting failure mechanism (at Tuas -6.5 meter MSL); piping and instability are dealt with in the structural design.

Small scale flooding occurs when the inflow of water due to rainfall (neglecting seepage and overtopping) exceeds the water storage and drainage capacity. The increase in cost of a water management system, about 10-40 mln €, is small compared to the reclamation cost saving of 700 mln €. Further, the investment required to minimize the risk of small scale flooding is small. The water management system should therefore be designed conservatively in order to minimize its risk rather than optimize its total cost.

When designing a new container terminal the chosen terminal levels should not only be based on minimal total cost but also take the return period of inundation and the risks involved in to account. The decision whether or not a design should be based on minimal total cost, a certain level of safety or an acceptable risk is one of a political nature.

The feasibility of the polder terminal is investigated for container terminals but could also be applied for other port functions. More research on the assumptions made in the risk framework model could lead to new insights, however through sensitivity analyses it is concluded that the influence on the total cost is negligible. Similarities in the approach with a dike and terp model could be further investigated.
### III. List of tables and figures

#### III.1 Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Thesis committee</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>Gravity quay walls</td>
<td>29</td>
</tr>
<tr>
<td>3</td>
<td>Sheet pile quay walls</td>
<td>31</td>
</tr>
<tr>
<td>4</td>
<td>Pile supported platforms</td>
<td>32</td>
</tr>
<tr>
<td>5</td>
<td>Special types of quay walls</td>
<td>33</td>
</tr>
<tr>
<td>6</td>
<td>Water balance parameters</td>
<td>35</td>
</tr>
<tr>
<td>7</td>
<td>Flood defense loads</td>
<td>37</td>
</tr>
<tr>
<td>8</td>
<td>Settlement estimations</td>
<td>38</td>
</tr>
<tr>
<td>9</td>
<td>Creep coefficients</td>
<td>41</td>
</tr>
<tr>
<td>10</td>
<td>Inundation levels</td>
<td>46</td>
</tr>
<tr>
<td>11</td>
<td>Damage cost 2nd Maasvlakte</td>
<td>47</td>
</tr>
<tr>
<td>12</td>
<td>Research in direct damage as a result of flooding</td>
<td>47</td>
</tr>
<tr>
<td>13</td>
<td>Soil assumptions [XVII]</td>
<td>59</td>
</tr>
<tr>
<td>14</td>
<td>Water levels, see Figure 30</td>
<td>60</td>
</tr>
<tr>
<td>15</td>
<td>Extreme wave conditions in the port of Tuas [XVII]</td>
<td>62</td>
</tr>
<tr>
<td>16</td>
<td>Singapore climate [ii]</td>
<td>63</td>
</tr>
<tr>
<td>17</td>
<td>Design vessel</td>
<td>65</td>
</tr>
<tr>
<td>18</td>
<td>Retaining levels, see Figure 30</td>
<td>66</td>
</tr>
<tr>
<td>19</td>
<td>Bearing requirements, see Figure 30</td>
<td>66</td>
</tr>
<tr>
<td>20</td>
<td>Soil assumptions [XVII]</td>
<td>71</td>
</tr>
<tr>
<td>21</td>
<td>Comparison quay wall structures</td>
<td>75</td>
</tr>
<tr>
<td>22</td>
<td>Failure probabilities SLS</td>
<td>82</td>
</tr>
<tr>
<td>23</td>
<td>Failure probabilities ULS (from ‘Handbook quay walls’ [VI])</td>
<td>85</td>
</tr>
<tr>
<td>24</td>
<td>Design values</td>
<td>99</td>
</tr>
<tr>
<td>25</td>
<td>Analytical determination of transitional quay wall height</td>
<td>100</td>
</tr>
<tr>
<td>26</td>
<td>Iterations optimal quay wall level</td>
<td>102</td>
</tr>
<tr>
<td>27</td>
<td>Numerical determination of minimal cost and corresponding quay wall height for ( h_p = 0 ) m.</td>
<td>102</td>
</tr>
<tr>
<td>28</td>
<td>Analytical optimization conventional terminal level</td>
<td>103</td>
</tr>
<tr>
<td>29</td>
<td>Numerical determination of minimal cost and corresponding quay wall height for ( h_p = 0 ) m.</td>
<td>104</td>
</tr>
<tr>
<td>30</td>
<td>Comparison Tuas case</td>
<td>105</td>
</tr>
<tr>
<td>31</td>
<td>Comparison for different reclamation cost</td>
<td>107</td>
</tr>
</tbody>
</table>
Table 32: Settlement estimate quay wall area ................................................................. 114
Table 33: Settlement estimate terminal area ................................................................. 115
Table 34: Uplifting calculation ...................................................................................... 124
Table 35: Loads on caisson gravity structure ............................................................... 127
Table 36: Water balance parameters .......................................................................... 136
Table 37: Permeability coefficients ........................................................................... 137
Table 38: Seepage through clay ................................................................................ 138
Table 39: Seepage through sand ................................................................................ 138
Table 40: Calculation storage/drainage capacity ......................................................... 142
Table 41: Storm events Tuas ...................................................................................... 142
Table 42: Unit rates water management ..................................................................... 143
Table 43: Total damage calculation for drainage capacity $Q_{\text{pump}} = 10 \text{m}^3/\text{s}$ ........ 143
Table 44: Total cost calculation for drainage capacity $Q_{\text{pump}} = 10 \text{m}^3/\text{s}$ .......... 143
Table 45: Total damage calculation for drainage capacity $Q_{\text{pump}} = 7 \text{m}^3/\text{s}$ .......... 144
Table 46: Total cost calculation for drainage capacity $Q_{\text{pump}} = 7 \text{m}^3/\text{s}$ .......... 144
Table 47: Risk minimization ....................................................................................... 145
Table 48: Total cost of basic case ................................................................................ 147
Table 49: Comparison for different quay wall cost .................................................... 149
Table 50: Comparison for non linear quay wall cost .................................................. 151
Table 51: Flooding time polder .................................................................................. 152
Table 52: Investment cost for different parameter A .................................................. 153
Table 53: Risk for different parameter A .................................................................... 153
Table 54: Investment cost for different parameter B ................................................... 154
Table 55: Risk for different parameter B .................................................................... 154
Table 56: Design values for sensitivity analysis of increased failure probability ......... 155
Table 57: Sensitivity analysis $P_{\text{f inundation}} = 1.1 \times P_{\text{f overtopping}}$ ..................... 155
Table 58: 'Extreme' design values ............................................................................. 157
Table 59: Total cost of extreme case with polder level at Mean Sea Level ................. 157
Table 60: Total cost of extreme case with polder level at uplifting boundary (-6.5m MSL) ......................................................................................................................... 158
Table 61: Total cost of extreme cases including insurance premium ....................... 160
Table 62: Total cost of container terminals at Tuas, Singapore .................................. 165
Table 63: Thesis committee ...................................................................................... 186
Table 64: Literature review ....................................................................................... 190
Table 65: Project planning ......................................................................................... 194
III. List of tables and figures

III.II Figures

Figure 1: Cross section conventional terminal (sketch) ................................................................. 5
Figure 2: Cross section polder terminal (sketch) ........................................................................... 5
Figure 3: Comparison of total cost polder terminal vs conventional terminal .............................. 6
Figure 4: Cross section Polder terminal [RHDHV] ......................................................................... 24
Figure 5: Failure mechanisms gravity type structures ................................................................... 30
Figure 6: Cost of quay walls [VI] .................................................................................................. 34
Figure 7: Failure mechanisms soil structures, [XXVII] ................................................................. 38
Figure 8: Crest height of a dike, [XXVII] ..................................................................................... 39
Figure 9: Slip circle computation with Fellenius and Bishop ......................................................... 39
Figure 10: Comparison Bligh and Sellmeijer [XVIII] .................................................................. 40
Figure 11: Failure mechanisms cofferdam [XXIV] ....................................................................... 42
Figure 12: Damage function (example) ....................................................................................... 46
Figure 13: Polder terminal DOW Chemical Terneuzen [Google earth] ........................................... 49
Figure 14: Polder airport [iv] ........................................................................................................ 50
Figure 15: Cross section dike / jetty Cai Mep Vietnam [RHDHV] .................................................. 52
Figure 16: Tuas polder terminal [RHDHV] .................................................................................. 52
Figure 17: Quay wall/apron polder terminal [RHDHV] ................................................................. 53
Figure 18: Comparison 2nd Maasvlakte as conventional and polder terminal [Google maps] ....... 54
Figure 19: Location Tuas container pier development [RHDHV] ................................................. 58
Figure 20: Polder terminal plan (possible) [RHDHV] ................................................................. 58
Figure 21: Polder terminal cross section (conceptual) ................................................................. 58
Figure 22: Extreme water levels .................................................................................................... 60
Figure 23: Extreme water level regression Tuas (A = 2.87 and B = 0.15) ....................................... 61
Figure 24: Extreme water level regression (logarithmic scale) ..................................................... 61
Figure 25: Dominant wave directions .......................................................................................... 61
Figure 26: Current speeds [RHDHV] ........................................................................................... 62
Figure 27: IDF Curve Singapore 2009 [v] .................................................................................... 63
Figure 28: Seismicity and return periods [XIV] .......................................................................... 64
Figure 29: Back reach crane (left) and portal crane (right) .............................................................. 67
Figure 30: Situation sketch, see 3.2.1 for details ........................................................................ 68
Figure 31: Polder terminal plan .................................................................................................... 70
Figure 32: Polder terminal cross section (half) ............................................................................. 70
Figure 33: Cross section polder terminal: soil layers ................................................................. 71
Figure 34: Total area required for quay wall vs dike ................................................................. 72
Figure 35: Gravity structure types ............................................................................................... 73
Figure 47: Terminal level for which
Figure 46: Relation inundation depth with outside water level (conceptual graph) ........................................ 92
Figure 45: Damage function container terminals [XIX] ..................................................................................... 92
Figure 44: Optimal level polder terminal .......................................................................................................... 90
Figure 43: Optimization quay wall height (conceptual graph) ................................................................. 89
Figure 42: Comparison flooding of conventional terminal and polder terminal .............................................. 88
Figure 41: Fault tree small scale flooding ......................................................................................................... 82
Figure 40: Fault tree large scale flooding ......................................................................................................... 84
Figure 39: Conventional terminal cross section (half) ...................................................................................... 77
Figure 38: Water channel inside quay wall structure [XIX] ............................................................................. 76
Figure 37: Pile supported platform ................................................................................................................... 74
Figure 36: Sheet pile structure types .................................................................................................................. 74
Figure 35: Comparison of both strategies for Ip = 40 €/m3 ........................................................................... 107
Figure 34: Comparison of both strategies for Ip = 1 €/m3 ............................................................................... 107
Figure 33: Behavior sheet pile wall ................................................................................................................ 119
Figure 32: Pile supported platform structures ................................................................................................ 120
Figure 31: Piping width quay wall ................................................................................................................... 123
Figure 30: Calculation uplifting ....................................................................................................................... 124
Figure 29: Loading combinations conventional terminal quay wall ............................................................... 125
Figure 28: Loading combinations polder terminal quay wall ........................................................................ 126
Figure 27: Loads on caisson gravity structure ................................................................................................ 127
Figure 26: Horizontal forces on quay wall flood defense ................................................................................ 128
Figure 25: Relation caisson weight / width with polder level during normal loading .................................... 129
Figure 24: Relation caisson weight / width with polder level during flood loading ....................................... 130
Figure 23: Design area gravity structure weight related to polder level ......................................................... 132
Figure 22: Seepage calculation ....................................................................................................................... 137
Figure 21: Optimization variables water management system .................................................................... 139
Figure 20: Graphs to approximate risk ........................................................................................................ 140
Figure 19: Cumulative rainfall intensity-duration-frequency curves ............................................................. 142

III. List of tables and figures
III. List of tables and figures

Figure 75: Optimization drainage capacity ................................................................. 144
Figure 76: Basic case sensitivity analysis ................................................................. 148
Figure 77: Retaining heights for conventional and polder terminal ............................ 149
Figure 78: Quay wall cost (linear and non linear) ...................................................... 150
Figure 79: Extreme case sensitivity analysis for polder level at Mean Sea Level .......... 158
Figure 80: Extreme case sensitivity analysis for polder level uplifting boundary .......... 159
Figure 81: Comparison of total cost polder terminal vs conventional terminal (Reclamation = 20 €/m³) .... 165
Figure 82: Requirements for quay wall flood defense structure related to polder depth .... 166
Figure 83: Cross section [IX] ................................................................................. 173
Figure 84: Traditional Dutch windmill ...................................................................... 173
Figure 85: Optimization variables water management system .................................... 175
Figure 86: Cross section Polder Terminal [Royal HaskoningDHV] .............................. 187
Figure 87: Combined quay wall / dike flood defense [Royal HaskoningDHV] .............. 192
Figure 88: Fault tree Polder Terminal ...................................................................... 193
Figure 88: Project planning ...................................................................................... 195
III. List of tables and figures
## IV. List of symbols

### IV.I Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Exponential distribution parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>$A_p$</td>
<td>Area of polder terminal design</td>
<td>$[m^2]$</td>
</tr>
<tr>
<td>$A_{storage}$</td>
<td>Area of water storage</td>
<td>$[m^2]$</td>
</tr>
<tr>
<td>$B$</td>
<td>Exponential distribution parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>$B_{quay}$</td>
<td>Width of the quay wall</td>
<td>[m]</td>
</tr>
<tr>
<td>$C_{ID}$</td>
<td>Settlement coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$C_{conventional}$</td>
<td>Total cost conventional terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$C_{creep}$</td>
<td>Creep coefficient piping calculation</td>
<td>[-]</td>
</tr>
<tr>
<td>$C_{n;tot;c}$</td>
<td>New total cost conventional terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$C_{n;tot;p}$</td>
<td>New total cost polder terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$C_{polder}$</td>
<td>Total cost polder terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$C_{pump}$</td>
<td>Total pump costs</td>
<td>[€]</td>
</tr>
<tr>
<td>$C_{storage}$</td>
<td>Total storage costs</td>
<td>[€]</td>
</tr>
<tr>
<td>$C_{tot;c}$</td>
<td>Total cost conventional terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$C_{tot;p}$</td>
<td>Total cost polder terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$c'$</td>
<td>Cohesion coefficient</td>
<td>[kPa]</td>
</tr>
<tr>
<td>$cu$</td>
<td>Cohesion coefficient</td>
<td>[kPa]</td>
</tr>
<tr>
<td>$D_0$</td>
<td>Constant level of damage during ‘small scale flooding’</td>
<td>[€]</td>
</tr>
<tr>
<td>$D_{breach}$</td>
<td>Damage cost of quay wall breach</td>
<td>[€/m]</td>
</tr>
<tr>
<td>$D_{conventional}$</td>
<td>Damage function conventional terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$D_{direct}$</td>
<td>Direct damage during inundation</td>
<td>[€]</td>
</tr>
<tr>
<td>$D_{indirect}$</td>
<td>Indirect (economical) damage during inundation</td>
<td>[€]</td>
</tr>
<tr>
<td>$D_i$</td>
<td>Inundation depth related damage</td>
<td>[€/m]</td>
</tr>
<tr>
<td>$D_{n;conventional}$</td>
<td>New damage function conventional terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$D_{n;polder}$</td>
<td>New damage function polder terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$D_{polder}$</td>
<td>Damage function polder terminal</td>
<td>[€]</td>
</tr>
<tr>
<td>$D_l$</td>
<td>Down time related damage</td>
<td>[€/yr]</td>
</tr>
<tr>
<td>$dc$</td>
<td>Direct damage to containers during inundation</td>
<td>[€/m$^2$]</td>
</tr>
<tr>
<td>$d_{clay}$</td>
<td>Thickness of clay layer in reclamation fill</td>
<td>[m]</td>
</tr>
<tr>
<td>$d_{i}$</td>
<td>Coefficient for damage dependant on inundation depth</td>
<td>[m$^{-1}$]</td>
</tr>
<tr>
<td>$d_{inundation}$</td>
<td>Inundation depth</td>
<td>[m]</td>
</tr>
<tr>
<td>$d_{sand}$</td>
<td>Thickness of sand layer in reclamation fill</td>
<td>[m]</td>
</tr>
<tr>
<td>$d_{storage}$</td>
<td>Inundation depth in water storage area in polder terminal</td>
<td>[m]</td>
</tr>
<tr>
<td>$ES$</td>
<td>Upward seepage</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$ET$</td>
<td>Evapo-transpiration</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$f$</td>
<td>Friction coefficient sliding failure caisson structure</td>
<td>[-]</td>
</tr>
<tr>
<td>$F_{nav}$</td>
<td>Probability distribution of the extreme water levels</td>
<td>[yr$^{-1}$]</td>
</tr>
<tr>
<td>$F_v$</td>
<td>Vertical crane load on quay wall</td>
<td>[kN/m]</td>
</tr>
</tbody>
</table>
IV. List of symbols

- \( g \): Gravitational force \([\text{m/s}^2]\)
- \( H \): Horizontal forces (subscript indicates force) \([\text{kN/m}]\)
- \( H_{\text{fill}} \): Height of the fill, for settlement calculation \([\text{m}]\)
- \( H_r \): Significant wave height \([\text{m}]\)
- \( H_t \): High water level \([\text{m MSL/CD}]\)
- \( H_{\text{retaining}} \): Retaining height quay wall \([\text{m}]\)
- \( h_b \): Port bottom level \([\text{m MSL/CD}]\)
- \( h_c \): Critical height difference for piping failure \([\text{m}]\)
- \( h_{\text{construction}} \): Construction height of quay wall \([\text{m MSL/CD}]\)
- \( h_{\text{gwl}} \): Groundwater level in polder \([\text{m MSL/CD}]\)
- \( h_{\text{HAT}} \): Highest Astronomical Tide \([\text{m MSL/CD}]\)
- \( h_p \): Polder terminal level \([\text{m MSL/CD}]\)
- \( h_{\text{uplifting}} \): Minimal polder level for uplifting \([\text{m MSL/CD}]\)
- \( h_r \): Quay wall height for polder terminal \([\text{m MSL/CD}]\)
- \( h_{\text{transition}} \): Transitional quay wall height for polder terminal \([\text{m MSL/CD}]\)
- \( h_{\text{optimal}} \): Optimal quay wall height for polder terminal \([\text{m MSL/CD}]\)
- \( h_{\text{aleveirise}} \): Level supplement for sea level rise in quay wall height \([\text{m}]\)
- \( h_{\text{seepage}} \): Seepage height polder terminal \([\text{m}]\)
- \( h_{\text{settlement}} \): Level supplement for settlement in quay wall height \([\text{m}]\)
- \( h_t \): Terminal height for conventional terminal \([\text{m MSL/CD}]\)
- \( h_w \): Water level seaside \([\text{m MSL/CD}]\)
- \( h_{\text{waves}} \): Level supplement for waves in quay wall height \([\text{m}]\)
- \( h_{\text{wind}} \): Level supplement for wind set up in quay wall height \([\text{m}]\)
- \( I \): Intake water for flushing of polder \([\text{mm/hr}]\)
- \( I_{\text{conventional}} \): Conventional terminal investment cost \([\text{€}]\)
- \( I_p \): Variable reclamation cost \([\text{€/m}]\)
- \( I_{\text{polder}} \): Polder terminal investment cost \([\text{€}]\)
- \( I_{\text{pump}} \): Cost of pumping stations \([\text{€}]\)
- \( I_q \): Variable quay wall cost \([\text{€/m}]\)
- \( I_{\text{soil,conventional}} \): Conventional soil reclamation cost \([\text{€/m}]\)
- \( I_{\text{soil,polder}} \): Polder soil reclamation cost \([\text{€/m}]\)
- \( I_{\text{storage}} \): Storage area cost \([\text{€/m}^3]\)
- \( i \): Hydraulic gradient for seepage calculation \([-]\)
- \( I_p \): Variable reclamation cost \([\text{€/m}^3]\)
- \( I_q \): Variable quay wall cost \([\text{€/m}^2]\)
- \( IS \): Downward seepage \([\text{mm/hr}]\)
- \( K_o \): Coefficient for horizontal soil pressure (neutral) \([-]\)
- \( k \): Permeability of soil for seepage calculation \([\text{m/s}]\)
- \( L \): Leakage water through locks \([\text{mm/hr}]\)
- \( L \): Length caisson \([\text{m}]\)
- \( L_{\text{Bligh}} \): Piping length by formula of Bligh \([\text{m}]\)
- \( L_{\text{breach}} \): Length of quay wall beach \([\text{m}]\)
- \( L_{\text{horizontal}} \): Horizontal piping length \([\text{m}]\)
- \( L_{\text{Lane}} \): Piping length by formula of Lane \([\text{m}]\)
- \( L_p \): Length of polder terminal design \([\text{m}^2]\)
- \( L_q \): Minimal quay wall length for one vessel \([\text{m}]\)
### IV. List of symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{\text{quay}}$</td>
<td>Total quay wall length</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_t$</td>
<td>Length of design vessel</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_{\text{seepage}}$</td>
<td>Seepage length</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_{\text{vertical}}$</td>
<td>Vertical piping length</td>
<td>[m]</td>
</tr>
<tr>
<td>$M$</td>
<td>Moment</td>
<td>[kNm/m]</td>
</tr>
<tr>
<td>$N$</td>
<td>Amount of blows for Standard Penetration Test</td>
<td>[-]</td>
</tr>
<tr>
<td>$N_{\text{storm}}$</td>
<td>Design rainfall intensity</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$P$</td>
<td>Production process water</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$P_{F;\text{subscript}}$</td>
<td>Probability of failure (<em>subscript indicates failure mechanism</em>)</td>
<td>[yr$^{-1}$]</td>
</tr>
<tr>
<td>$P_{F;\text{transition}}$</td>
<td>Probability of failure at transitional height</td>
<td>[yr$^{-1}$]</td>
</tr>
<tr>
<td>$P_{\text{max}}$</td>
<td>Maximum soil bearing capacity</td>
<td>[kN/m$^2$]</td>
</tr>
<tr>
<td>$Q_{\text{in}}$</td>
<td>Discharge during overtopping</td>
<td>[m$^3$/s]</td>
</tr>
<tr>
<td>$Q_{\text{pump}}$</td>
<td>Discharge through gravity flow or discharge pumps</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$q$</td>
<td>Seepage water in polder (<em>subscript indicates timeframe</em>)</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$q_{\text{surcharge}}$</td>
<td>Surcharge load</td>
<td>[kN/m$^2$]</td>
</tr>
<tr>
<td>$R_{\text{conventional}}$</td>
<td>Conventional terminal risk</td>
<td>[€]</td>
</tr>
<tr>
<td>$R_{\text{polder}}$</td>
<td>Polder terminal risk</td>
<td>[€]</td>
</tr>
<tr>
<td>$r'$</td>
<td>Reduced interest rate</td>
<td>[-]</td>
</tr>
<tr>
<td>$S$</td>
<td>Lock water in water balance</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$s$</td>
<td>Settlement (<em>subscript indicates area</em>)</td>
<td>[m]</td>
</tr>
<tr>
<td>$T$</td>
<td>Time required to flood the polder</td>
<td>[hrs]</td>
</tr>
<tr>
<td>$T_p$</td>
<td>Significant wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>$t_{\text{down}}$</td>
<td>Down time of polder terminal</td>
<td>[yr]</td>
</tr>
<tr>
<td>$t_{\text{flood}}$</td>
<td>Duration of a flood</td>
<td>[yr]</td>
</tr>
<tr>
<td>$t_{\text{pump}}$</td>
<td>Time required to pump the polder dry</td>
<td>[days]</td>
</tr>
<tr>
<td>$t_{\text{storm}}$</td>
<td>Duration of a storm</td>
<td>[hrs]</td>
</tr>
<tr>
<td>$V$</td>
<td>Vertical forces (<em>subscript indicates which force</em>)</td>
<td>[kN/m]</td>
</tr>
<tr>
<td>$V_{\text{concrete}}$</td>
<td>Concrete volume</td>
<td>[m$^3$]</td>
</tr>
<tr>
<td>$V_{\text{in}}$</td>
<td>Total volume rainfall during a design storm</td>
<td>[m$^3$]</td>
</tr>
<tr>
<td>$V_{\text{polder}}$</td>
<td>Total volume of the polder</td>
<td>[m$^3$]</td>
</tr>
<tr>
<td>$V_{\text{sand}}$</td>
<td>Sand volume</td>
<td>[m$^3$]</td>
</tr>
<tr>
<td>$V_{\text{soil}}$</td>
<td>Storage volume of water in channels</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$V_{\text{storage}}$</td>
<td>Storage volume of water in soil</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$V_{\text{terminal}}$</td>
<td>Storage volume of water in channels</td>
<td>[mm/hr]</td>
</tr>
<tr>
<td>$v$</td>
<td>Torricelli fluid speed</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$W$</td>
<td>Width of quay wall</td>
<td>[m]</td>
</tr>
<tr>
<td>$W_p$</td>
<td>Width of polder terminal design</td>
<td>[m$^2$]</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Reliability index</td>
<td>[-]</td>
</tr>
<tr>
<td>$\varphi'$</td>
<td>Angle of internal friction</td>
<td>[']</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td>Specific weight soil (dry)</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$\gamma_{\text{clay}}$</td>
<td>Specific weight clay</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$\gamma_{\text{concrete}}$</td>
<td>Specific weight concrete</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$\gamma_{\text{sand}}$</td>
<td>Specific weight sand</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Unit</td>
</tr>
<tr>
<td>----------</td>
<td>--------------------------------------------------</td>
<td>------------</td>
</tr>
<tr>
<td>$\gamma_s$</td>
<td>Safety factor hydraulic structures</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma_{sat}$</td>
<td>Specific weight soil (saturated)</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>Specific weight water</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$\gamma_{fill}$</td>
<td>Specific weight soil fill</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$\sigma'_{eff}$</td>
<td>Effective soil pressure</td>
<td>[kN/m$^2$]</td>
</tr>
<tr>
<td>$\sigma_a$</td>
<td>Soil pressure after reclamation fill</td>
<td>[kN/m$^2$]</td>
</tr>
<tr>
<td>$\sigma_b$</td>
<td>Soil pressure before reclamation fill</td>
<td>[kN/m$^2$]</td>
</tr>
<tr>
<td>$\sigma_{fill}$</td>
<td>Soil pressure of fill</td>
<td>[kN/m$^2$]</td>
</tr>
</tbody>
</table>
V. Table of contents

I. Preface ................................................................................................................................. 3
II. Summary ............................................................................................................................ 5
III. List of tables and figures .................................................................................................. 9
   III.I Tables ......................................................................................................................... 9
   III.II Figures ..................................................................................................................... 11
IV. List of symbols .................................................................................................................. 15
   IV.I Symbols ..................................................................................................................... 15
V. Table of contents ............................................................................................................... 19
1. Introduction ...................................................................................................................... 23
   1.1. Introduction ............................................................................................................... 23
   1.2. Project description .................................................................................................... 23
   1.3. Problem definition .................................................................................................... 25
   1.4. Report lay out .......................................................................................................... 26
2. Literature .......................................................................................................................... 27
   2.1. Introduction ............................................................................................................... 27
   2.2. Port infrastructure ..................................................................................................... 27
   2.3. Polders ....................................................................................................................... 34
   2.4. Flood defenses .......................................................................................................... 36
   2.5. Risk assessment for flood defenses ......................................................................... 44
   2.6. Reference projects .................................................................................................... 48
   2.7. Conclusions and recommendations ......................................................................... 55
3. Polder terminal; a conceptual design ................................................................................ 57
   3.1. Introduction ............................................................................................................... 57
   3.2. Tuas implementation ................................................................................................. 57
   3.3. Conceptual design polder terminal ............................................................................ 69
   3.4. Comparison conventional terminal ........................................................................... 77
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5.</td>
<td>Conclusions and recommendations</td>
<td>79</td>
</tr>
<tr>
<td>4.</td>
<td>Risk assessment</td>
<td>81</td>
</tr>
<tr>
<td>4.1.</td>
<td>Introduction</td>
<td>81</td>
</tr>
<tr>
<td>4.2.</td>
<td>Small scale flooding</td>
<td>81</td>
</tr>
<tr>
<td>4.3.</td>
<td>Large scale flooding</td>
<td>83</td>
</tr>
<tr>
<td>4.4.</td>
<td>Conclusions and recommendations</td>
<td>86</td>
</tr>
<tr>
<td>5.</td>
<td>Risk framework</td>
<td>87</td>
</tr>
<tr>
<td>5.1.</td>
<td>Introduction</td>
<td>87</td>
</tr>
<tr>
<td>5.2.</td>
<td>Risk framework</td>
<td>87</td>
</tr>
<tr>
<td>5.3.</td>
<td>Tuas implementation</td>
<td>98</td>
</tr>
<tr>
<td>5.4.</td>
<td>Discussion</td>
<td>108</td>
</tr>
<tr>
<td>5.5.</td>
<td>Conclusions and recommendations</td>
<td>111</td>
</tr>
<tr>
<td>6.</td>
<td>Flood defense system</td>
<td>113</td>
</tr>
<tr>
<td>6.1.</td>
<td>Introduction</td>
<td>113</td>
</tr>
<tr>
<td>6.2.</td>
<td>Design quay wall height</td>
<td>113</td>
</tr>
<tr>
<td>6.3.</td>
<td>Flood defense structure</td>
<td>115</td>
</tr>
<tr>
<td>6.4.</td>
<td>Polder level</td>
<td>121</td>
</tr>
<tr>
<td>6.5.</td>
<td>Conclusions and recommendations</td>
<td>131</td>
</tr>
<tr>
<td>7.</td>
<td>Polder water management</td>
<td>135</td>
</tr>
<tr>
<td>7.1.</td>
<td>Introduction</td>
<td>135</td>
</tr>
<tr>
<td>7.2.</td>
<td>Water balance</td>
<td>135</td>
</tr>
<tr>
<td>7.3.</td>
<td>Risk framework</td>
<td>138</td>
</tr>
<tr>
<td>7.4.</td>
<td>Tuas implementation</td>
<td>141</td>
</tr>
<tr>
<td>7.5.</td>
<td>Conclusions and recommendations</td>
<td>146</td>
</tr>
<tr>
<td>8.</td>
<td>Sensitivity analyses</td>
<td>147</td>
</tr>
<tr>
<td>8.1.</td>
<td>Introduction</td>
<td>147</td>
</tr>
<tr>
<td>8.2.</td>
<td>Basic case</td>
<td>147</td>
</tr>
<tr>
<td>8.3.</td>
<td>Quay wall cost analysis</td>
<td>148</td>
</tr>
<tr>
<td>8.4.</td>
<td>Probability of inundation analysis</td>
<td>151</td>
</tr>
<tr>
<td>8.5.</td>
<td>Feasibility analysis for ‘extreme’ parameters</td>
<td>156</td>
</tr>
</tbody>
</table>
V. Table of contents

8.6. Conclusions and recommendations ................................................................. 160

9. Conclusions and recommendations ....................................................................... 163
9.1. Introduction ........................................................................................................ 163
9.2. Conclusions ........................................................................................................ 163
9.3. Recommendations ............................................................................................ 167

10. Literature ............................................................................................................. 169
10.1. Literature ......................................................................................................... 169
10.2. Websites .......................................................................................................... 170
10.3. Programs .......................................................................................................... 170

Appendices ............................................................................................................... 171
A Personal evaluation ................................................................................................ 171
B Short history of polders .......................................................................................... 173
C Other polder terminal applications ....................................................................... 174
D Water management optimization .......................................................................... 175
E Visualization: drawings ......................................................................................... 178
F Thesis proposal ...................................................................................................... 185
1. **Introduction**

1.1. **Introduction**

Transportation over water has always been the easiest way to transport goods compared with transport by road, rail or air. This is why transport over water forms the largest means of transport in the world. Ports are still expanding and new ports are built to keep up with the demands due to increasing growth of sea trade.

New container ports are mostly constructed on low lying coastal areas or in shallow coastal waters. The quay wall and terminal yard are then raised to a level well above mean sea level to insure safety against inundation. The resulting ‘conventional terminal’ requires large volumes of good quality fill material often dredged from the sea which is costly. Furthermore, these dredging works usually have a large environmental impact.

Port operators generally demand terminals which are well above the extreme water levels, to assure flood safety. Is this justified or is there a way of constructing safe terminals with smaller volumes of dredged material and thus lower cost?

1.2. **Project description**

Royal HaskoningDHV developed the concept of a container terminal with a “polder yard”. The yard would lie below the outside water level and be surrounded by a combined quay wall flood defense structure. In addition to the facilities which are also present in a conventional terminal, a polder terminal requires storage capacity for rain and seepage water plus a system for discharging water (as in other polders).

A large part of the Netherlands consists of polders which lie below Mean Sea Level, these polders are generally used for agricultural purposes or housing. However there are certain locations where industrial activities take place inside polders near the dikes.

Figure 4 shows a cross section of the general idea of the polder terminal. It shows a terminal yard at Mean Sea Level surrounded by quay walls which provide the necessary safety against extreme water levels (to keep the water out).
1. Introduction

As shown in the figure a container port terminal is considered, because most of the port expansion and/or new port projects consist of container terminals. A short investigation on the application of other port functions is made in appendix C. Container trade has been growing rapidly during the last decades resulting in an increase of ship sizes and port operations. Container ships increasing in size require larger and especially deeper berths. To keep up with the demands of these mega ships and changes in port operations countries need to expand their existing ports or build new ports.

The main idea behind the polder terminal is to reduce the investment costs of the terminal by reducing the amount of fill required to construct the terminal. Royal HaskoningDHV made a preliminary study [XIX] on the feasibility of a polder terminal, which concluded that a polder terminal could be feasible. In comparison with a conventional terminal (with a terminal yard level equal to the quay wall height) the following was concluded:

- Normal operation of the terminal do not have to be affected by a lower polder yard;
- A polder terminal could have lower construction cost due to the reduction of reclamation material;
- The polder terminal is particularly feasible for larger terminals, at locations with low quality sub soils and high prices for reclamation material;
- The polder terminal could have a higher flood risk then the conventional terminal.

Based on a quick scan of the market of container ports around the world and considering the characteristics of making a polder terminal feasible (low quality sub soil and high prices for reclamation material), three area’s for possible business development were identified, i.e. South-East Asia, India and Brazil [XIX].
1. Introduction

1.3. Problem definition

As stated earlier port operators generally want a terminal yard to be designed above the extreme water levels to assure safety against flooding. The corresponding inundation risk of a ‘conventional terminal’ is expected to be lower than the inundation risk of a ‘polder terminal’, because of the lowered terminal yard. The exact savings in costs and the increased risk of inundation of the polder terminal require further investigation to prove the feasibility of the concept.

Project aim

This master thesis will therefore aim at making a risk based design of a polder terminal which is used to investigate the technical and economic feasibility of the polder terminal in comparison with the conventional terminal.

The following research questions are addressed:

- What are the consequences of making a polder terminal, in comparison to a conventional terminal?
- What are the risks involved when making a polder terminal?
- What levels / strength should be chosen for the quay wall and terminal yard of the polder terminal?
- What quay wall flood defense system is most suitable for a polder container terminal?
- How to deal with water storage and drainage inside the polder container terminal?
1.4. Report lay out

This report contains ten chapters and starts with the current chapter, giving an introduction of the subject which is investigated in this graduation project: the polder terminal. The second chapter contains a literature study; not much literature is available on the subject because this is a new concept. But still, on different aspects which finally form the polder terminal, a lot of literature can be found.

A conceptual design of a polder terminal is made in chapter three, which is based on a case study in Tuas, Singapore. A comparison is made of different aspects of a polder terminal compared to a conventional terminal which elaborates on the advantages and disadvantages of the polder terminal.

As stated before, a polder terminal is expected to have a larger inundation risk than a conventional terminal. The risk of inundation is investigated in chapter four, which makes a distinction between small scale flooding (water hindrance) and large scale flooding (inundation).

Chapter five investigates the feasibility of the polder terminal through a risk framework approach which is used to determine the economic optimal levels (height) of the combined quay wall flood defense and polder terminal yard. The corresponding total cost of the polder terminal is estimated and compared to the total cost of the conventional terminal. The chapter concludes with a discussion of the risk framework approach and gives recommendations on aspects which require further investigation.

One of the aspects requiring further investigation is the quay wall flood defense structure which is treated in chapter 6. Different structures are treated after which a conceptual calculation is made of a gravity type structure as a quay wall flood defense for the polder terminal. Different failure mechanisms are treated which from the boundaries of the maximum polder depth of the polder terminal yard.

Each polder requires an adequate water management system, which is treated in chapter seven. The required water storage and drainage capacity is investigated and corresponding investment cost and risk are estimated.

Chapter eight contains a sensitivity analysis of the risk framework approach made in chapter five. Different assumptions made in chapter 5 are investigated and conclusions are drawn regarding the feasibility of the polder terminal.

Finally chapter nine summarizes all conclusions and recommendations regarding the feasibility of the polder terminal. The research questions mentioned in paragraph 1.3 are answered and aspects requiring further investigation discussed.

Note: The symbols used in the different chapters are explained in the List of Symbols on page 15; design drawings are added in appendix E.
2. Literature

2.1. Introduction

The literature study will aim at identifying all relevant aspects regarding the research questions in the introduction. The polder terminal is a new concept; therefore not a lot of information is available on the subject itself, but when splitting the subject in to different categories relevant information can be found.

2.2. Port infrastructure

2.2.1. Port functions, lay out and equipment

Ports can have different functions depending on the ships and cargo which are handled through the port. Information on port infrastructure is obtained from the lecture notes Ports and Terminals of H. Ligteringen [XI] and the PhD thesis of J.G. de Gijt: “A history of quay walls” [VI]. Port functions are:

- General (multipurpose) cargo terminals;
- Container cargo terminals
- Dry (coal) or liquid (oil) bulk terminals;
- RoRo terminals;
- Specialized (passenger / food) terminals.

The layout of the corresponding terminals greatly depends on which type of cargo (and thus which port function) is handled, also of large influence is the type of handling equipment used to transfer cargo between the terminal yard and the apron.

Container ships are continuously growing in sizes, from the 1st generation container ships with a capacity of 1,100 TEU and a draught of 9 meters to the 7th generation ship with capacities over 15,000 TEU and maximum draughts of 25 meters. At this moment the largest expected container ships will have a maximum length of 500 meters, width of 70 meters and a draught of 25 meters. The design of port projects needs to take the expected ship dimensions in to account.
2.2.2. Quay walls in general

On the border of land and water quay walls are built to facilitate the transfer of goods between ships and terminals. The necessary retaining height of the quay wall depends on the draught of the ships; in ancient times ships without keels could easily be handled at quay walls made of natural stone or timber. Through century’s ship sizes have increased dramatically, so did the corresponding draught of the ships. This led to the introduction of steel and reinforced concrete as base material for quay walls.

Container port terminals require a quay wall along the whole length of the ships to be handled, cranes can then easily move along the quay to reach every container on board. Due to the development of ship sizes, the amount and size of container handling cranes have also increased resulting in larger loads on quay wall structures.

**Functional requirements**

Quay walls are multi functional constructions:

- *Retaining function*: The structure must safely retain water and soil. For a polder terminal, this poses extra requirements in comparison with conventional quay walls, because the quay wall will also act as a primary flood defense of the hinterland which has a lower surface level than the quay wall;
- *Bearing function*: The structure must safely bear all loads imposed by cranes, cargo and handling equipment on the apron and safely transfer these loads to the subsoil;
- *Mooring function*: The construction must enable ships to moor safely and transfer cargo between ship and shore efficiently;
- *Protection function*: When at berth ships need to be protected against possible damages. All forces including wind, waves and currents must be safely dealt with.

**Quay wall design**

Traditionally most quay walls are designed following a deterministic approach: by using an overall safety factor to account for uncertainties in loading and resistance. Recently semi or fully-probabilistic methods have been implemented in the design process. The probabilistic approach is based on the principle that a design is developed within a maximum probability of failure. Important factors to be taken in to account when designing a quay wall are summed up below.

- Expected ship sizes and corresponding loads;
- Soil and subsoil conditions;
- Water levels and wave heights/loads;
- Tidal variation and currents;
- Crane and terrain loads;
- Construction methods and materials;
- Re-usability of entire quay wall or solely of materials.
2. Literature

2.2.3. Quay wall classifications

In general quay wall structures can be divided in soil retaining structures, piled structures and structures on special foundations. An overview of different types of quay walls is summed up below. Information is obtained from the PhD thesis of J.G.de Gijt, “A history of quay walls”, made in 2010 [VI] and the lecture notes ‘Port Infrastructures’ of J.G. de Gijt, made in November 2004 [VIII].

Gravity-type quay walls

Gravity-type structures obtain stability through their own weight and friction between the structure and foundation. These structures cause high pressures on the subsoil compared to sheet pile structures, which require high bearing capacities. In areas with clay and peat such as the Netherlands these structures are not advised because of low bearing capacities of the subsoil.

<table>
<thead>
<tr>
<th>Type</th>
<th>Cross section</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast in place concrete or masonry wall</td>
<td></td>
<td>Concrete, natural stone</td>
</tr>
<tr>
<td>Angle type wall</td>
<td></td>
<td>Prefabricated or monolithic construction of reinforced concrete elements</td>
</tr>
<tr>
<td>Prefabricated from concrete blocks</td>
<td></td>
<td>Prefabricated heavy concrete blocks</td>
</tr>
<tr>
<td>Floated in caissons</td>
<td></td>
<td>Prefabricated or monolithic construction in reinforced concrete</td>
</tr>
<tr>
<td>Large diameter cylinders: silos</td>
<td></td>
<td>Prefabricated or monolithic construction in reinforced concrete or steel</td>
</tr>
</tbody>
</table>

Table 2: Gravity quay walls

Gravity structure calculation

The design of gravity type structures is carried out in a semi-probabilistic way, because probabilistic design methods have not been developed for these structures yet. In the design of a gravity wall system the following subjects need to be considered which are partly illustrated in Figure 5.

- Structural integrity: Internal bending and shear stresses as a result of external loading may not exceed the maximum values.
- Sliding: The sum of the horizontal forces may not exceed the frictional force between the structure and the foundation, caused by the weight of the structure.
➢ **Overturning:** The structure must be sufficiently safe against overturning: the resultant vertical force must lie within the middle third of the structure base.

➢ **Bearing capacity:** The occurring vertical load may not exceed the maximum bearing capacity.

➢ **Settlement and tilt:** Gravity walls are expected to have considerable settlement when built on non-rock foundations. Uneven settlements induce tilting of the structure and should be avoided.

---

**Figure 5: Failure mechanisms gravity type structures**

**Sheet pile-type quay walls**

Sheet-pile type quay walls consist of a wall which is driven or constructed in the subsoil below the port bottom level. The penetration of the wall into the subsoil generates a fixed-end moment that secures the stability of the wall. Sheet-pile walls can be made with or without anchorages which provide additional stability against soil pressures. When constructed with an anchorage a shorter penetration depth into the subsoil is required and smaller bending moments occur in the sheet pile wall.

Depending on the required retaining height of the quay wall these structures can be made of timber (low heights), prestressed concrete sheet piles (intermediate heights) or steel (large heights). Diaphragm walls are a special type of sheet pile quay wall; in this case the wall is not driven into the subsoil but constructed in the subsoil. After excavating a trench to the required depth, reinforcement is placed and concrete is poured in the trench bottom up. This will form a wall which functions as the soil retaining structure.

A relieving platform may be built on top of the wall structure to reduce the active earth pressure on the uppermost part of the sheet pile wall. Relieving platforms are built as small sheet-pile walls, diaphragm walls, L-shaped walls or any other type of soil retaining structure. On one side they are founded on the sheet pile wall and on the other side on bearing and tension piles. The stability of the relieving walls needs to be checked independent of the sheet pile wall itself.

<table>
<thead>
<tr>
<th>Type</th>
<th>Cross section</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cantilever bulkhead</td>
<td></td>
<td>Steel or reinforced concrete piles</td>
</tr>
</tbody>
</table>
2. Literature

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single anchor bulkhead</td>
<td>Steel or reinforced concrete piles with steel pile anchors, grout anchor, screw anchor, M.V. Pile</td>
</tr>
<tr>
<td></td>
<td>A combi wall consists of steel piles with steel sheet walls between them</td>
</tr>
<tr>
<td>Multi anchor bulkhead</td>
<td>Diaphragm wall with steel pile anchors, grout anchor, screw anchor, M.V. Pile</td>
</tr>
<tr>
<td>Slurry wall single / multi anchored</td>
<td>Steel wall, diaphragm wall or reinforced concrete piles with steel pile anchors, grout anchor, screw anchor, M.V. Pile</td>
</tr>
<tr>
<td>Relieving platform with front sheet pile wall</td>
<td>Prefabricated or monolithic reinforced concrete platform construction, steel or reinforced concrete piles and sheet piles</td>
</tr>
<tr>
<td>Relieving platform with rear sheet pile wall</td>
<td>Prefabricated or monolithic reinforced concrete platform construction, steel or reinforced concrete piles and sheet piles</td>
</tr>
</tbody>
</table>

Table 3: Sheet pile quay walls

Note: There is some hesitation against the use of steel in a primary sea defense because of the shorter expected lifespan, difficult supervision of the state of the steel, insufficient resistance against ship collision, difficult connection with hinterland and difficulty to extend the height of the defense system [IV].

Sheet pile calculation

To calculate the sheet pile wall the method of Blum is often used. This method assumes a failure situation in the ground in which the deformations are so large that maximum shear stresses can develop; minimum active and maximum passive earth pressures are used. The method assumes an embedded depth which is somewhat larger than the minimum required depth.

- The Blum method is a simple method used to develop preliminary designs for unanchored, single and multiple anchored sheet pile walls. It cannot be used for calculation of quay walls with high rigidity/stiffness such as diaphragm walls;
- A disadvantage of the Blum method is that the actual earth pressures on site may differ considerably from the minimum active and maximum passive pressures as used in the calculation resulting in different loading situations.

This method is very suitable for hand calculations of a sheet pile wall; if more detailed design calculations are required methods which describe the soil as an elasto-plastic spring model are usually applied. These calculate the actual occurring active and passive soil pressures which will produce better results. The main failure mechanisms are described below.
**Macro instability of the whole quay wall structure**

Macro instability occurs when structures slide along straight or curved slip planes. Different calculation methods are available; slip plane calculations and finite element methods (PLAXIS).

**Uplift, heave and piping**

The vertical equilibrium of the soil body on the passive side of the sheet piles needs to be verified to check if uplifting does not occur. The design value of the upward water flow pressure must be in equilibrium with the design values of the effective weight of the soil mass under consideration.

Heave and piping occur when seepage water induces sediment transport under a hydraulic structure. The phenomenon and calculation methods are explained in paragraph 2.4. The quay wall for a polder terminal should be water tight to prevent water from entering the polder; the water tightness of the structure depends on the materials used and the construction method.

**Geotechnical checks: bearing capacity**

When determining the bearing capacity of the subsoil the influence of the construction method need to be taken into account: excavation or dredging activities are able to reduce the maximum soil stresses, other installation techniques may also be of influence.

**Pile supported platforms (jetties)**

Pile supported platforms (jetties) have traditionally always been used as mooring facilities for ships in deeper waters. The difference in height between the port terminal and the port bottom is not overcome by a soil retaining structure as in previous examples, but by a slope which does not require a large soil retaining structure. A bank protection is required to protect the slope against currents and wave attack.

<table>
<thead>
<tr>
<th>Type</th>
<th>Cross section</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile supported platforms (jetty)</td>
<td></td>
<td>Monolithic or prefabricated concrete platform construction, steel or reinforced concrete</td>
</tr>
</tbody>
</table>

Table 4: Pile supported platforms

**Pile supported platform calculation**

These structures actually contain two structures; a piled deck to facilitate vessel berthing and provide a foundation for the container cranes and an embankment or soil retaining wall to overcome the height difference between port bottom and quay the terminal yard.

Regarding stability these structures need to be treated separately. The pile supported platform obtains stability through the piles which are driven in the subsoil to a level where sufficient bearing capacity is obtained. For the stability of the soil structure reference is made to section 2.4.1 where these are treated, if a soil retaining structure is used reference is made to the previous sections which treated sheet pile or gravity type quay walls.
2. Literature

**Special types of quay walls**

Special types of quay walls that have been applied in the past are illustrated in Table 5; these could be combinations of previous types of quay walls or new / innovative concepts.

<table>
<thead>
<tr>
<th>Type</th>
<th>Cross section</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large diameter cylinders with relieving platform</td>
<td></td>
<td>Cylinders and platform consist of prefabricated concrete or monolithic construction</td>
</tr>
<tr>
<td>Platform supported on deep caissons</td>
<td></td>
<td>Steel or concrete caissons, and monolithic or prefabricated reinforced concrete platform</td>
</tr>
<tr>
<td>Platform supported on piles with increases carrying capacity</td>
<td></td>
<td>Steel or concrete screw piles and monolithic or prefabricated reinforced concrete platform</td>
</tr>
<tr>
<td>Reinforced earth wall</td>
<td></td>
<td>Prefabricated concrete elements with metal anchor strips</td>
</tr>
</tbody>
</table>

**Table 5: Special types of quay walls**

**Quay wall of the future**

Quay walls are subject to developments in ship sizes and port industries. Because ship sizes have increased drastically during the last decades larger quay walls and more efficient cargo handling equipment are required. Due to these developments existing quay walls often lose their functionality before their technical lifetime has ended: the economical lifetime is shorter than the technical lifetime. Future quay walls need to be designed taking these developments in to account.

Currently new concepts for quay walls and quay wall design methods are under investigation which will better anticipate on future developments; flexibility is required so quay walls don’t lose functionality too early in their lifetime. In a master thesis performed by P.Bonte published in January 2007 [III], a study is made of possible new quay wall designs. The sandwich wall, two rows of tubex piles with a jet grout mass between them, proved to be the most promising concept for the future. Due to higher costs of a sandwich wall it is expected not to be applied in the near future. Other possibilities are summed up below:

- Sandwich wall, two rows of tubex piles with a jet grout mass between them;
- Frozen quay walls, creating a vertical wall by freezing the groundwater in the soil;
- Floating quay walls, a hollow concrete or steel structure which is able to move in vertical direction;
- Tunnel type quay walls: for very long quay walls a tunnel may be drilled to function as a quay wall.
2.2.4. Quay wall costs

An investigation in quay wall costs, made by J.G. de Gijt, provided the following results:

![Graph showing cost of quay walls vs. retaining height](image_url)

**Figure 6: Cost of quay walls [VI]**

This graph shows a relation between the retaining height and quay wall cost per running meter. These costs vary within a bandwidth of +/- 25% irrespective of the local soil conditions, tidal ranges and type of structure (for sheet piles the bandwidth is lower than for gravity structures) [VI]. Other comments on these prices:

- The initial cost of the quay walls is independent of the location;
- Large lengths of quay walls make improved logistics possible which result in cost savings;
- Maintenance cost have no large influence on the choice of the type of construction;
- The variation in cost for gravity structures is much larger than for sheet pile structures.

2.3. Polders

This paragraph deals with polders and the design of flood defense systems required to protect polders from flooding. Information is obtained from the lecture notes ‘Flood defenses’ of J. Weijers [XXVII] and M. Tonneijck and ‘Polders’ of Hoes, O.A.C., Leeuwen van, N.C. and Giesen van de, N.C [IX]. A short history on polders is given in appendix B.
2. Literature

2.3.1. Polder design

A polder is a low lying area protected by dikes which was originally subject to high ground- or surface waters; the water levels within the polder have no connection with outside water, these can be kept constant. Characteristics are mentioned below:

- The hydrological regime (ground water and surface water levels) of a polder is controlled artificially, independent of the water levels in the surrounding land;
- A polder has an outlet structure designed to discharge excess water.

Important parts of a polder are the flood defense system (usually a dike ring), the layout and dimensions of the water courses and the water discharge system. The flood defense system is treated in paragraph 2.4; the water discharge system is treated in the following sections; the layout and dimensions of the water courses will not be investigated. This is beyond the scope of this graduation project.

Water balance

The water balance inside a polder is influenced by factors that increase the amount of water (loading) and factors that decrease the amount of water (relieving). Water surplus occurs when the loading factors exceed the relieving factors; if the amount of water exceeds the storage capacity. Inundation of the hinterland can occur temporarily which is defined as the Serviceability Limit State, and permanently which is defined as the Ultimate Limit State.

\[
\begin{align*}
\text{Loading} - \text{Relieving} &= \text{Storage} \\
(N + ES + S + L + I + P) - (ET - IS - Q_{\text{pump}}) &= V_{\text{soil}} + V_{\text{storage}} 
\end{align*}
\]  

(2-1)

<table>
<thead>
<tr>
<th>Parameter [m/day]</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>Precipitation: rain, snow and hail (condensation is ignored).</td>
</tr>
<tr>
<td>ES</td>
<td>Upward seepage, groundwater discharge through dikes or from deeper layers.</td>
</tr>
<tr>
<td>S</td>
<td>Lock water enters the polder through locks.</td>
</tr>
<tr>
<td>L</td>
<td>Leakage water enters the polder at the locks due to leakage of the doors.</td>
</tr>
<tr>
<td>I</td>
<td>Intake water is used for flushing of the waterways in polders, to keep these clean.</td>
</tr>
<tr>
<td>P</td>
<td>Production process water is tap water discharged into the polder by the industry.</td>
</tr>
<tr>
<td>ET</td>
<td>Evapotranspiration is dependent on the type of vegetation present.</td>
</tr>
<tr>
<td>IS</td>
<td>Downward seepage, groundwater discharge through dikes or from deeper layers.</td>
</tr>
<tr>
<td>Q_{pump}</td>
<td>Discharge through gravity flow or discharge pumps.</td>
</tr>
<tr>
<td>V_{soil}</td>
<td>Storage of water in the soil</td>
</tr>
<tr>
<td>V_{storage}</td>
<td>Storage of water in the canals</td>
</tr>
</tbody>
</table>

Table 6: Water balance parameters

Storage areas

Traditional polders had a surface water area of 10% of the total polder area. Modern polders have much smaller surface water area, around 1 to 2% of the total polder area. Storage in soil is difficult to quantify,
when defining the water balance it is preferred to define a final situation comparable to the initial situation allowing storage in soil to be neglected.

**Discharge system**
Water can be discharged through gravity flow or through pumps. It should be noted that the discharge system of a polder is part of the flood defense system. Failure can be caused by direct failure of the discharge system itself or by structural failure: loss of strength or stability, insufficient retaining height, erosion or calamities.

**Gravity flow**
Gravity flow is the traditional way to discharge water from polders. To be able to discharge through gravity flow the outside water level should be a few centimeters below the polder level. This buffer of a few centimeters is required to overcome the larger pressure of outside salt water.

**Pumping station**
Most gravity flow discharge systems have been replaced by pumping stations. Wind mills were the first to mechanically discharge water from the polder to the adjacent water courses, currently electrical or diesel pumps are used for this purpose.

Most pumping stations are placed in the lowest part of the polder; if no slopes are present the preferred location is at the middle of the long side of a rectangle polder, to minimize the distance to each corner of the polder. Usually an economic balance between the location of the polder outlet and minimizing excavation costs by using existing water courses and natural flow paths determines the exact location.

### 2.4. Flood defenses

Flood defenses are designed to protect the hinterland from inundation. Information on failure mechanisms of flood defense systems is obtained from the TAW [XXI – XXIV] and the lecture notes ‘Flood Defenses’ of J. Weijers and M. Tonnejick [XXVII]. Inundation occurs more often because a failure mechanism is forgotten rather than poorly calculated, the calculations described below are often applied as semi probabilistic methods.

Flood defenses required to protect polders from inundation can be divided in two groups, structures which retain ‘outside water’: water which has a direct influence of high tides and storm surges or high river water levels. The second group contains flood defense structures which do not directly retain ‘outside water’; these are structures which provide secondary protection against flooding. A flood defense system for a polder terminal belongs to the first group of flood defense systems:

- Dunes which consist of large bodies of washed up sand by the sea; vegetation catches and holds sand which protects the hinterland during periods of high water (storms). A sufficiently large amount of sand needs to stay put during a storm for dunes to provide sufficient flood protection;
2. Literature

- Soil structures, dams and dikes, are manmade soil bodies which, contrary to dunes, may not erode. Stability against erosion is obtained through revetments of i.e. asphalt, stone or grass;
- Special water retaining structures are structures which are formed by other materials than soil; examples are a flood wall, cofferdam or a sheet pile. These structures obtain their strength through material properties such as reinforced concrete, steel or wood;

Water retaining hydraulic structures are usually made to fulfill multiple functions, not only do they retain water during a flood, they may also have a traffic function during normal situations (roadway or shipping lanes). These structures often are provided with a moveable means of closure, examples are an outlet sluice, tidal flood barrier or navigation lock.

In the case of a polder terminal a quay wall will act as a flood defense system, which is a special water retaining structure, possibly in combination with a dike which is a soil structure. The design of standard quay walls was treated in section 2.2.3; the next paragraphs covers the (design) aspects regarding soil structures and special water retaining structures. Special attention is paid to unusual loads: calamities.

### 2.4.1. Soil structures

Failure mechanisms of soil structures are illustrated in Figure 7, the loads are mentioned in Table 7.

*Note: The failure mechanisms regarding soil structures could very well also apply to quay walls when these have an additional flood defense function as in the case of polder terminals.*

<table>
<thead>
<tr>
<th>Category</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent loads</td>
<td>Dead weight</td>
</tr>
<tr>
<td></td>
<td>Soil pressures</td>
</tr>
<tr>
<td></td>
<td>Surcharges / terrain loads</td>
</tr>
<tr>
<td>Temporary loads</td>
<td>Water levels</td>
</tr>
<tr>
<td></td>
<td>Currents</td>
</tr>
<tr>
<td></td>
<td>Waves</td>
</tr>
<tr>
<td></td>
<td>Rain</td>
</tr>
<tr>
<td></td>
<td>Wind</td>
</tr>
<tr>
<td></td>
<td>Temperature (for structures)</td>
</tr>
<tr>
<td></td>
<td>Surchages / terrain loads</td>
</tr>
<tr>
<td>Calamities (unusual loads)</td>
<td>Ship collision</td>
</tr>
<tr>
<td></td>
<td>Drifting ice</td>
</tr>
<tr>
<td></td>
<td>Terrorism / Earthquakes</td>
</tr>
</tbody>
</table>

Table 7: Flood defense loads
Crest height

The crest height of a flood defense system is the most important design parameter, because it is directly related to the stability mechanisms of overflow and overtopping. During overflow or overtopping water enters the dike through the crest and inner slope, if the dike is saturated with water pore pressures increase reducing the effective stresses and thus the possible shear stress. The driving forces (soil weight) increase while the resisting (shear) forces decrease resulting in instability. Further, the amount of overflow and overtopped water may not exceed the water storage capacity inside the polder to prevent inundation.

In the 1953 flooding of the south west part of the Netherlands a large part of the dikes breached due to inner slope shearing as a result of overtopping. In current practice therefore a crest height is chosen such that very little or no overtopping occurs. It is clear that in the case of a quay wall structure as a flood defense system minimal overflow or overtopping should be allowed.

Settlement

The crest height is subject to settlements which are expected during the design lifetime. Failure of a dike is understood as something which will happen suddenly, settlement however is a time dependant process. Settlement is divided in the construction settlement and the residual settlement which includes time dependant creep. Estimates of settlement of freshly applied soil in percentages of the increase in height are given in Table 8.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Percentage</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay (ripe)</td>
<td>10 %</td>
<td>With careful implementation and densification 5% could be expected</td>
</tr>
<tr>
<td>Clay (unripe)</td>
<td>&gt;15%</td>
<td>The use of unripe clay is undesirable because serious cracking may develop</td>
</tr>
<tr>
<td>Sand</td>
<td>5 %</td>
<td>With good density this settling can be neglected</td>
</tr>
</tbody>
</table>

Table 8: Settlement estimations
2. Literature

Erosion

Erosion of the inner slope may occur due to overflow or overtopped waves which are avoided by increasing the crest height. Erosion of the outer slope may occur due to continuous loading by waves and currents in the outside water. The top layer of the outer slope provides protection against erosion through riprap, stone revetments, concrete block revetments or impervious asphalt layers. Stability is achieved through dead weight of the material and/or friction/coherence. Calculation methods are described in chapter 8 of ‘Introduction to bed, bank and shoreline protection’ [XVIII]. An elaboration of these methods in this section is considered redundant.

Macro stability

Macro instability occurs when large parts of the structure slide along straight or curved slip planes. As explained in the last paragraph sliding of the inner slope could occur after a long period with overflow and overtopping. Considering the outer slope, sliding could occur when the outside water levels drop fast. The water level inside the soil mass cannot follow resulting in an overpressure. This overpressure could cause the dike (or quay wall) to slide towards the water.

Micro stability

Micro instability concerns local instabilities in the outer layer of the structure which occur when individual soil particles erode from a slope due to groundwater flow. Calculations of micro instability are done through soil water analysis, equilibrium is obtained when the dead weight and/or friction between particles is higher than the water pressures, reference is made to the next section which treats heave and piping.
Heave and piping

A head difference over a hydraulic structure can induce seepage, because soil will never be completely impervious to water. Sediment transport can occur when the flow force in porous flow is larger than the weight of individual grains, a phenomenon called heave. The same happens with micro instability. However, micro instability concerns the stability of the outer grains whereas heave and piping occur in the inner layers.

If erosion due to this sediment transport continues a channel (pipe) can form with continuous sediment transport. This will eventually lead to progressive failure of the structure, it should be noted that a channel can only be formed when a ‘roof’ of cohesive material separates the channel from the structure, see Figure 10.

Traditionally this phenomenon is calculated with the simple formulas of Bligh and Lane: Bligh when considering a dike, or Lane when considering vertical structures such as sluices, locks and seawalls. A more detailed method is that of Sellmeijer. For small values of D, the thickness of the sand layer, Bligh gives a conservative value of the maximum permissible head difference when comparing with Sellmeijer, as illustrated in figure 5-11 of the lecture notes Bed Bank and Shoreline protection [XVIII].

\[
\Delta h_i \leq \frac{L_{\text{Bligh/Lane}}}{C_{\text{creep}}} \\
L_{\text{Bligh}} = L_{\text{vertical}} + L_{\text{horizontal}} \\
L_{\text{Lane}} = L_{\text{vertical}} + L_{\text{horizontal}} / 3
\]

The critical head difference \( \Delta h_i \) and creep coefficients are illustrated in Table 9.
2. Literature

Foreshore stability: erosion

Bottom erosion (scour) can make the outer slope of a dike unstable. A steep underwater slope consisting of loosely packed sand could move in to a more dense state resulting in higher soil water pressures. If this overpressure is not drained the sandy subsoil could change in to a thick fluid because the contact forces between the grains decreases, this is called ‘quicksand or liquefaction’. The sand will settle in a slope which is much gentler (and thus longer) possibly causing collapse or slides of the dike.

2.4.2. Special water retaining structures: quay walls

Quay walls as a flood defense will be subject to different loading situations than conventional quay walls. Besides design aspects as treated in section 2.2.3 additional aspects need to be considered, which are summed up below:

- When a body of soil is bound by two retaining walls (coffer dam) a drainage system is necessary to drain excess water out of the soil in the dike, frequent maintenance is required to prevent clogging of the system;
- Anchors used to increase the stability of a quay wall are subject to internal erosion; they have a short life span and create holes in the dike which could induce piping [VI];
- The structural stability of a quay wall as flood defense system needs to be investigated. Attention needs to be paid to the behavior during calamities like drifting ice, earth quakes and ship collision.

Cofferdam

A special type of quay wall as flood defense system is a coffer dam, which consists of a body of soil bound by a soil retaining structure on both sides. Both sheet piles are connected to each other with a tensile element. These structures are very sensitive to ship collision as failure of one or more sheet piles could endanger the stability of the whole structure (if no partitioning is made). It is possible that part of the flood defense system of a polder terminal will consist of a coffer dam, because a retaining wall is required on the portside and terminal side of the quay wall flood defense.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Lane</th>
<th>Bligh</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt</td>
<td>8.5</td>
<td>18</td>
</tr>
<tr>
<td>Fine sand (150 – 200 µm)</td>
<td>7</td>
<td>15</td>
</tr>
<tr>
<td>Coarse sand (300 - 1000 µm)</td>
<td>5</td>
<td>12</td>
</tr>
<tr>
<td>Fine gravel (2 – 6 mm)</td>
<td>4</td>
<td>9</td>
</tr>
<tr>
<td>Coarse gravel (&gt;16 mm)</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 9: Creep coefficients
An important aspect of cofferdams is to investigate whether or not the soil retaining structures influence each other, if this is the case the structure will behave as one structure: a cofferdam. If the walls are placed outside of the area of influence they will behave as two separate sheet piles.

Failure mechanisms [XXIV] of cofferdam walls are illustrated in Figure 11.

- Insufficient internal stability (a): failure due to excess shear pressures in the soil causing the structure to lose its shape;
- Soil failure on inner side (b1 b2): when the passive soil pressure on the inner side wall is insufficient it will move or rotate;
- Rotation of the whole structure (c d): If driving moments are larger than the resisting moments the structure will rotate around a point under or inside the soil between the cofferdam walls.;
- Failure due to stresses higher than the yield stress of steel (e);
- Failure of the anchors (f): when the forces on the anchor are too high they will fail removing the interaction between both retaining walls;
- Macro instability (g): when the structure fails along deep circular sliding planes;

![Figure 11: Failure mechanisms cofferdam [XXIV]](image-url)
2. Literature

The calculation of cofferdams is done according to the Method of Terzaghi, which models the subsoil as a series of vertical lamellae. The friction forces between different lamellae are checked, to assure that the structure retains its shape. Other failure mechanisms are calculated according to the methods described in the last paragraphs.

Cofferdams can be designed according to a deterministic method, a semi probabilistic method or a probabilistic method. For the latter two methods a finite element method is required. Because of the complex behavior of soil and different relations between the described failure mechanisms a calculation with a finite element method, such as PLAXIS, is advised for detailed design of these structures.

2.4.3. Unusual loads: calamities

Drifting ice
Drifting ice has not produced any damages to flood defenses in the Netherlands in the last century because of warmer surface waters due to increased industrial activities along the main rivers. Theoretically it could still happen; for dikes this loading situation is however not taken in to account. Considering polder terminals the local conditions determine whether or not drifting ice forms a threat to the flood defense.

Earthquakes
Earthquakes rarely occur in the Netherlands so calculations of earth quake loading in the Netherlands are unnecessary, for other locations it might be important however. Recent earthquakes have shown the vulnerability of port waterfront structures to earth quakes: in most cases limited deformations occurred.

Consequences of an earth quake on a quay wall
An earthquake can cause three major consequences for a quay wall structure. The driving forces on the structure will increase, the shear resistance of the structure may decrease due to excess pore pressures and finally resonance may occur if the vibrations reach the fundamental frequency of the structure. Loss of strength or stability and displacements belong to the consequences of failure of the structure [X]. For a quay wall as a flood defense system these failure mechanisms could induce leakage or breaching of the structure with inundation of the polder as a result.

Ship collision
Ship collision could theoretically produce large damages to dikes and other water retaining structures. For dikes this failure mechanism is not taken in to account in the design stage, because ships will run aground on the foreshore before they can breach a dike, except during extreme high water levels [XXIII].

For quay walls mooring forces are taken in to account, ship collision is not however because it is not economically sound to design a structure for full frontal collisions. Guiding works and breaking structures are usually constructed to prevent ship collision. The mooring / collision force is calculated from an energy balance, the total kinetic energy of the ship together with the water that moves along with the ship is absorbed by the structure, which can be modeled by a mass spring system [XIII].
The probability of flooding of the hinterland due to ship collision depends on the probability of a collision, the effect of a collision on the water retaining function of the structure and the probability of a flood due to loss of the water retaining function. To determine these probabilities the maximum kinetic energy which could be absorbed by the structure without losing the water retaining function needs to be calculated.

2.5. Risk assessment for flood defenses

This paragraph explains how risk assessments for flood defenses are made, what aspects are important and how these can best be illustrated through fault trees.

2.5.1. Development risk based design

After the flooding disaster in 1953 in the south west part of the Netherlands a statistical approach to the storm surge levels was applied and an extrapolated storm surge level would be the basis for dike design.

Dike overload approach
At first a dike overload approach was used, whereby overloading was perceived as an occurring discharge (due to overflow and overtopping) over a dike which was higher than a permissible discharge. This overload could induce failure mechanisms resulting from dike overflow / overtopping, omitting other failure mechanisms such as piping. The design water level was obtained through the determination of the permissible discharge over the dike. This approach was used for dike sections or whole dike rings [XXVII].

Flood risk approach
Since the 1980's it became possible, through the application of reliability theory, to assess the flooding risks taking all failure mechanisms into account. Risk is defined as the probability of an event times the consequences of the event (probability*consequence), when divided by the reduced interest rate the present value of the risk is obtained. Flooding is a typical high consequence, low probability event. In this case the actual risk is defined as a number of casualties and economical damage that could occur when a dike ring fails to protect an area against flooding. The new design water levels were determined in two steps: first, statistical data of water levels were extrapolated to levels never obtained before after which an economic optimization of the design water level was done (an optimization between expected damages and required investment costs) [XXVI]. The Delta Committee knew that other failure mechanisms than overtopping could be dangerous; therefore additional requirements were formulated in a classical way.
2. Literature

2.5.2. Probabilistic approach to dike design

A probabilistic approach evaluates the risk against flooding of a polder by determining the probabilities of failure of the dike ring and estimating the corresponding consequences. The consequences for a polder terminal will be expressed in monetary terms, focusing on the damages on the terminal and potential economic losses due to down time of the port.

A risk assessment is made following these steps:

1. System definition: simplifications and assumptions are used to model a system, in this case a polder;
2. Qualitative analysis: a description of the various failure mechanisms which induce flooding;
3. Quantitative analysis: to determine a quantitative value of risks all possible scenarios are simulated in four steps:
   a. Estimate the reliability of the system
   b. Simulate the flooding scenarios
   c. Determine the consequences (financial damage for a polder terminal)
   d. Quantify the risk
4. Risk evaluation: the quantified risk is evaluated by comparing with tolerable / acceptable risks;
5. Risk reduction and control: finally decisions regarding the acceptability of the determined risks are made. The effectiveness of proposed risk reduction measures can be assessed in the same manner as the first risk assessment making this a ‘circular’ method.

Fault tree

The relations between failure mechanisms and consequences can be depicted with a fault tree. The top event defines failure of the system, flooding in this case. The probability of occurrence of the individual failure mechanisms and the relations between them determine the probability of occurrence of the top event. Weak spots of the system can easily be identified through a fault tree analysis. Failure probabilities are calculated using methods of the modern reliability theory like level III Monte Carlo, Bayesian updating and Level II advanced first order second moment calculations.

2.5.3. Water depth during flooding

Inundation is related to an amount of water on the surface level of the terminal, to illustrate water depths in relation to inundation the following division is used [II].

<table>
<thead>
<tr>
<th>Depth</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 0.5 meter</td>
<td>The intrusion of water can be blocked by simple means.</td>
</tr>
<tr>
<td>0.5 – 2.0 meter</td>
<td>Serious damage is considered, but little danger to humans especially when there are flight opportunities to higher grounds / stories.</td>
</tr>
</tbody>
</table>
Material damage is unavoidable, danger to humans because even the first story of buildings is also flooded.

| Over 4.0 meter | High risk of total loss and high risks to humans. |

### Material damage

| Over 4.0 meter | High risk of total loss and high risks to humans. |

### Consequences of flooding

In order to quantify the risks involved in the design of a polder terminal the consequences need to be determined. Distinction is made between the direct consequences of flooding, which result from damage to the port, and indirect consequences of flooding, which are the result of down time of the port.

#### Direct damages to the port terminal

The direct damages will be expressed in costs per square meter. To determine the unit prices for direct damage due to inundation three sources were consulted; a study of the safety of the 2\textsuperscript{nd} Maasvlakte [II], a report of an internship studying the damages due to inundation in the Rotterdam port area [XIII] and an explanation of the standard method to determine the damage and casualties due to floods. The first two studies concern actual port areas whereas the latter option does not; this method [XIII] gives a more general approach (not specific for ports) to flood damage, which is why this method will not be discussed further.

#### ‘Safety 2\textsuperscript{nd} Maasvlakte’ [II]

In this report an assessment is made of the damage on a container terminal due to inundation. General unit prices used for inundation damage are given in Table 11; the unit price for container terminals was verified by making estimates of direct damages to containers and equipment. The results proved to have a
2. Literature

bandwidth of 15%. The damage costs shown in the table contain direct damage costs; costs which are made when the area is restored to the original state before the flood.

<table>
<thead>
<tr>
<th>Category</th>
<th>Inundation damage cost [€/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chemical</td>
<td>363</td>
</tr>
<tr>
<td>Container</td>
<td>500</td>
</tr>
<tr>
<td>General industry</td>
<td>358</td>
</tr>
</tbody>
</table>

Table 11: Damage cost 2nd Maasvlakte

‘Economical damage as a result of flooding’ [III]
This report focusses on the available Dutch literature from 1966 through 2005. The damage costs shown in the table only contain direct damage costs; costs which are made when the area is restored to the original state before the flood. Eight useful reports from both the Dienst Weg- en Waterbouwkunde (DWW) from Rijkswaterstaat and Tebodin were investigated which roughly determine the damages on the same way, the results of the study are given in Table 12.

<table>
<thead>
<tr>
<th>Source</th>
<th>Category</th>
<th>Price</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inundatie risicobepaling buitendijkse gebieden, kleine case MV1, DWW, 1996</td>
<td>General industry</td>
<td>85</td>
<td>€ / m²</td>
</tr>
<tr>
<td></td>
<td>Infrastructure</td>
<td>114</td>
<td>€ / km</td>
</tr>
<tr>
<td>Inundatierisico's van buitendijkse industrie, Tebodin, 1998</td>
<td>Chemical</td>
<td>425</td>
<td>€ / m²</td>
</tr>
<tr>
<td></td>
<td>Tank storage companies</td>
<td>700</td>
<td>€ / m²</td>
</tr>
<tr>
<td></td>
<td>Crude oil industry</td>
<td>1,300</td>
<td>€ / m²</td>
</tr>
<tr>
<td></td>
<td>Container</td>
<td>580</td>
<td>€ / m²</td>
</tr>
<tr>
<td></td>
<td>Distribution</td>
<td>740</td>
<td>€ / m²</td>
</tr>
<tr>
<td></td>
<td>Bulk terminals</td>
<td>380</td>
<td>€ / m²</td>
</tr>
<tr>
<td>Integrale ontwerpaanpak Maasvlakte 2, Economische optimalisatie, DWW, 1999</td>
<td>General industry</td>
<td>210,000</td>
<td>€ / laborer</td>
</tr>
<tr>
<td></td>
<td>Infrastructure</td>
<td>105</td>
<td>€ / km</td>
</tr>
<tr>
<td>Schadecurves industrie ten gevolge van overstroming, Tebodin, 2000</td>
<td>Crude oil industry</td>
<td>1,200,000</td>
<td>€ / laborer</td>
</tr>
<tr>
<td></td>
<td>Bulk chemical</td>
<td>750,000</td>
<td>€ / laborer</td>
</tr>
<tr>
<td></td>
<td>General industry</td>
<td>710,000</td>
<td>€ / laborer</td>
</tr>
<tr>
<td>Overstromingsrisico's buitendijkse gebieden, DWW, 2001</td>
<td>General industry</td>
<td>250</td>
<td>€ / m²</td>
</tr>
<tr>
<td></td>
<td>Infrastructure</td>
<td>21</td>
<td>€ / m²</td>
</tr>
<tr>
<td>Standaardmethode, Schade en Slachtoffers ten gevolge van overstromingen, DWW, 2002</td>
<td>General industry</td>
<td>300,000</td>
<td>€ / laborer</td>
</tr>
<tr>
<td></td>
<td>Infrastructure</td>
<td>1,500</td>
<td>€ / m</td>
</tr>
</tbody>
</table>

Table 12: Research in direct damage as a result of flooding

Note: These results are rather old and could require indexing to present time. However, in more recent damage cost calculations for the 2nd Maasvlakte also a cost of 500 € / m² is used which is of the same order as the cost in 1998 (Tebodin 1998). That is why it is assumed these cost are rather constant. The influence of this assumption will be investigated in a sensitivity analysis in chapter 8.
The conclusion of the report states that the report of Tebodin from 1998 contains the most reliable and detailed information, which can directly be concluded from the table as well. This report contains functions for inundation damages of 15 different industrial areas lying in flood risk areas, the results were used to predict inundation damages for the port of Rotterdam.

*Note: The results in the report of Tebodin apply to a terminal where the water is drained naturally; the water depths seldom reach a level higher than 0.5 meter because it can flow away under natural discharge. In a polder terminal a different situation occurs, one where the water is ‘trapped’ inside the polder and drainage is only provided by mechanical pumps. Flooding depths depend on the height difference of the polder level and quay wall, which could be much higher.*

**Direct damages to the flood defense structure**

Apart from the port damages, damage to the flood defense structure is also part of the total direct damage. These damages are related to the construction costs of quay walls which are shown in Figure 6 in relation to the required retaining height for quay walls around the world.

In the master thesis of J.W. Liang a short risk analysis is made of the cost of repair of quay walls after complete collapse [X]. It was assumed that the cost required for removal of the collapsed quay wall and repair of the cranes and other material amounted to 30% of the total construction cost of the quay wall. The repair of a part of the flood defense system which collapsed will then amount to 130% of the initial construction cost.

**Indirect damages**

The indirect damages are related to the consequences of down time of the port considered. Down time will have a significant impact on the total income of the port and on a larger scale on a country’s economy. A country's economy is largely influenced by large ports which not only produce income for the port operator but also produce income for second and third parties (spin-off effect) and the national government. This all depends on whether the port is an import/export port or a transshipment port.

The method to determine the loss of income of the port operator due to down time is explained below:

1. Determine port capacity and income per TEU’s in time;
2. Estimate port income in time by multiplying the port capacity with the income;
3. Determine the down time of the port due to flooding and estimate the loss of income.

**2.6. Reference projects**

This paragraph contains a short overview of reference projects. Arguments why polders were applied as land reclamation method are presented for other reference projects and finally possible locations for polder terminals are mentioned.
2. Literature

2.6.1. Industrial polders

**Vlissingen en Terneuzen, the Netherlands**

In Terneuzen (and Vlissingen) a refinery is situated directly behind a dike (+8m NAP) which acts as the primary sea defense, the polder lies between +1m and 2m NAP. High water levels could occur up to +5/6 m NAP. Two jetties provide berths for tankers so these can transfer cargo between the terminal and the ships. This figure directly shows that in the Netherlands polder terminals already exist.

![Google Earth Image](image)

Figure 13: Polder terminal DOW Chemical Terneuzen [Google earth]

**Suvarnabhumi airport, Thailand**

The Suvarnabhumi airport (see Figure 14), located in Thailand, is built on a flood plain close to mean sea level. It is part of the floodways of Bangkok’s eastern suburbs and therefore requires adequate flood protection. The area is part of a once low lying marsh which took about 5 years to clear through land reclamation. Instead of raising the surface level of the airport it was built inside a polder protected by dikes of 3.5 meters. This way large settlement of the surface level was prevented; cost effectiveness was further achieved by building the dikes with the soil dug out of the reservoirs and drainage canals required. Technical information of the polder system is summed up below [i]:

- The total area of the polder is 2800 hectares.
- A dike of 3.5 meters high, 3 meters wide on the top, and 23.5 kilometers in length protects the polder against flooding.
- Drainage canals of 35 meters wide and 2 meters deep collect rain water from the runways, taxiways, , roads and buildings.
- Six reservoirs of 2 meters deep are able to hold 3.2 million cubic meters of water which is the equivalent of five consecutive days of rainfall of each five-year cycle without the need for pumping.
- The total open water storage area is 160 hectares, which is about 6 % of the total polder area.
- Two pumping stations to the south of the airport, with a capacity of 6 cubic meters per second drain water from the polder. The discharge capacity amounts to 13.5 mm/hr or 324 mm/day.
This study proves that polders as a form of land reclamation are a good alternative when the subsoil consists of weak layers which are subject to large settlements when loaded. At the flood during the 2011 monsoon season in Thailand the Suvarnabhumi airport stayed dry proving the safety of its flood prevention system.

2.6.2. Possible polder terminal locations

Firstly a polder terminal for the expansion of the Schiphol airport of the Netherlands in the North Sea is treated, after which possible locations of polder container terminals are treated.

**Nort Sea airport, the Netherlands**

The airlift capacities of the airports in the Netherlands have almost reached their maximum, to assure growth of the sector studies are performed on possible airport expansions at various locations in the country. One of the options was to build a new airport in the North Sea, which would be connected to shore through a tunnel or bridge.

To build the airport in the North Sea land reclamation was necessary; B.Sommeling [XX] investigated three different forms of land reclamation: a polder, an artificial island and an island on piles. Considering expandability, costs and the amount of required fill material a polder model was advised, partly because in the Netherlands a lot of experience regarding polders is present. Conclusions relevant to this master thesis are summed up below:

- The polder will be constructed in deep water, which induces large amounts of seepage. The exact amount of seepage depends on the soil conditions, the design of the dike and possible seepage limiting constructions (only a closed dike and foreshore proved applicable). The required drainage capacity was economically feasible for the calculated amount of seepage;
- To reduce seepage the outer slope of the dike should be impermeable to water;
- The water drainage system consists of 4 pumps, 3 working regularly and one back up.
Note: this study was made in 1998, so the conclusions made should be handled with care. The cost estimates might be completely different nowadays due to economic growth and the effects of the economic crisis.

More recently the discussion of constructing a polder in the North Sea continued in the ‘Tweede Kamer’. A polder could be built to provide more space for housing near the big cities of the Netherlands. A press release [v], stated that hydraulic engineers think the applicability of a polder for this purpose in the North sea is impossible. Not in the sense of constructability but in the sense of costs; the required sea defense system would be too expensive compared to the benefits.

Following the above mentioned points one can conclude that constructing a polder in the North Sea is technically possible, provided the benefits are higher than the costs.

Cai Pei, Vietnam

Royal HaskoningDHV performed an assessment in possible options to significantly reduce the construction costs of the first phase development of the Gemalink Container Terminal [XVI] in Cai Pei Vietnam. The construction of a polder terminal, raising the surface to a level of 3 meters above mean sea level instead of 7.5 meters, belonged to these options. A polder terminal would reduce the reclamation costs, as well as the required slope stability measures. The expected settlement is reduced because less fill material is required to replace the soft soils with sand.

The terminal would be surrounded by a dike, 6 meters above mean sea level, consisting of a sandy core covered by a clay layer and grass. A small channel behind the dike would be necessary to collect seepage and rain water. Drainage would be done by natural discharge during low tide; the storage area would consist of channels between the container stacks and roads (partly covered). The quays of the port would be constructed as jetties, illustrated in Figure 15, the approach bridges between quay and terminal will require ramps due to the level differences. Advantages and disadvantages of the polder terminal are mentioned:

**Advantage(s)**

- A polder terminal would significantly reduce the quantity of fill material, the expected settlement and the required slope stability measures;
- If a jetty is built no vertical soil retaining structure is required;
- Because less filling and soil improvement is required time is saved during construction.

**Disadvantage(s):**

- An increase of the risk of flooding due to the lower lying terminal level which depends on the drainage capacity of the polder and the outside water levels;
- More space required for the same terminal capacity, due to storage areas of water;
- The detached quay wall is disadvantageous to the logistic efficiency of the port because of larger travelling distances and the impossibility of pre stacking near the quay;
- Larger influence of waves and currents due to position of jetty (further away from terminal).
Tuas, Singapore

Royal HaskoningDHV was involved in the development of a design for a new container terminal yard pier in the port of Tuas, Singapore, which is part of the MPA Coastal Development [XVII]. For many years already the need for new land in the area has been fulfilled through reclamation, resulting in a scarcity of good quality fill material. Apart from the cost of fill material there is a growing awareness of the environmental impact of dredging. To reduce reclamation costs and environmental impact a polder terminal was proposed. The polder terminal would also aid in minimizing the settlement expected and create sufficient bearing capacity.

The designed polder terminal consists of a pier with 8.6 kilometers of wharf structures, 320 ha of land reclamation, 420 ha of soil improvement, 6 km of off-shore containment bunds, dredging and other works.

The required terminal level, taking hydrological requirements in to account, is 6 meters above mean sea level. The subsoil consists of predominantly clayey and silt layers which would be subject to large settlements after reclamation. Good quality fill material in the area has become scarce. Apart from this additional problems to land reclamation include settlement, longer construction periods and lack of sufficient bearing capacity. For these reasons land reclamation in the area has become very expensive.
2. Literature

Because of the aforementioned reasons a polder terminal was proposed, which would require less fill material. Only the quay wall and apron area would need to be raised to a level of 6 meters above mean sea level whilst the terminal yard would only require a height of 2 meters below mean sea level. The quay wall and apron area would act as a flood defense system; an adequate water management system will be required to pump out water due to seepage, rainfall and/or overtopping. Costs will be saved on land reclamation and soil improvement; additional costs are made for dike construction and drainage systems.

In the area the costs per hectare of reclaimed land are relatively high so efficiency in spatial planning was required. In conventional polder water storage is provided in open water channels, because of limited space the water collecting channels were incorporated in the quay wall / dike structure, as illustrated in Figure 17. The combined storage capacity needs to be sufficient for coping with extreme rainfalls when the amount of inflow due to rainfall and seepage exceeds the pumping capacity.

![Figure 17: Quay wall/apron polder terminal](RHDHV)

In order to minimize seepage clayey material with low permeability would be used in the lower part of the reclamation and good permeable sand in the upper layer to affectively control the ground water levels. Agricultural type drains could be used in the upper layers to collect seepage water.

**Comparison with conventional terminal**

In comparison with conventional terminal design, the polder terminal is expected to be cost effective. The higher operational costs during the lifetime are compensated by the lower capital costs resulting in a lower life cycle cost than a conventional terminal. The advantages of the polder terminal concept in comparison with a conventional terminal are:

- Substantial cost savings in cost of fill material (about 25%) and thus lower investment cost;
- Overall lower life cycle costs;
- Less environmental impact due to dredging works (lower reclamation and less CO$_2$ emission).
- Less visual impact on the environment due to lower maximum container stacking heights;
- More adaptable to sea level rising, only the quay wall / dike structure would need to be raised.

*Note: This design considered a preliminary study based on qualitative assumptions.*
Maasvlakte 2, the Netherlands

The port of Rotterdam, being one of the most important ports in Europe, is expanding its capacity so it would not fall behind on other major ports in the area. A large land reclamation project is underway in the North Sea where new ports are built. The design was based on a design storm with a probability of 1/10,000 per year, which coincides with a design terminal surface level of NAP +6.10 m and a crest level of the sea defense breakwater of NAP +16 meters. The new port expansion consists for 60% of container terminals.

Polder terminals were not (or hardly) considered for the design of this port expansion, because the large dredging companies (which provided the design of the port) have all the knowledge available to provide a design by raising the entire surface level to the level of +6 meters above NAP. Further, the entire port area of Rotterdam is built outside of the dikes, with port levels between NAP +3.5m and NAP +5.0m, the subsoil conditions do not require large soil improvements as it consists of sand and reclamation costs aren’t expensive. If a polder terminal was considered very large amounts of seepage water is expected due to the sandy subsoil requiring enormous pumping stations to keep the polder dry.

![Figure 18: Comparison 2nd Maasvlakte as conventional and polder terminal](https://maps.google.com)

Nevertheless, due to the large size of the terminal, costs could have been reduced when the port expansion was made as a polder; raising only the sea defense system and quay walls to the required height (NAP +16m respectively +6.1 meters), leaving the terminal area at a level of 0 meter NAP or lower, as illustrated in Figure 18. This system differs quite a lot from the original polder terminal idea, because it would comprise an entire port, which is much larger than one terminal, making the design much more complex. It could be necessary to make compartments in such a polder to reduce flood risks.

An investigation whether or not costs could have been saved by constructing the 2nd Maasvlakte as a polder could be interesting. It is clear that the cost reduction would follow from not having to raise the whole terminal area (about 2,000 hectares) to +6.1 m NAP, but only to 0m NAP, saving an estimated amount of 120 million cubic meters which, with a unit price for reclamation of €3.50 per m$^3$, amounts to 400 million euro.
2.7. Conclusions and recommendations

Container terminals require a quay wall along the whole length of the ships to be handled, cranes can then easily move along the whole quay to reach every container on board. A quay wall for a polder terminal is an innovative quay wall design as the structure will not only ‘traditionally’ retain soil and water; it will also act as a primary flood defense protecting the lower lying terminal yard against inundation and possibly contain a water storage and/or drainage function.

The quay wall of a polder terminal could be made as a gravity type structure, sheet pile structure, or pile supported platform (in combination with a soil retaining wall). The lower surface level of the terminal yard could have significant influence on the behavior of the soil behind the quay wall. The top part of the quay wall could be constructed as a cofferdam. The maximum allowed depth of the terminal yard as well as its effects on the (structural) design of the quay wall as a primary flood defense structure need to be investigated to obtain the most practical type of quay wall.

Polder flood defense systems usually consist of soil structures and sometimes of special water retaining structures. Design aspects and failure mechanisms are treated in the corresponding sections. The flood defense system of a polder terminal could consist of a combination of both flood defense systems mentioned above.

A flood risk approach will be used to evaluate the risk against flooding of the polder terminal by determining the probabilities of failure of the dike ring and estimating the corresponding consequences, which will be expressed in monetary terms (due to damages and down time of the port). Flooding is defined as an unacceptable amount of water on top of the terminal; experts in the field of flood risk confirm that inundation corresponds with water depths over 50 cm.

Considering the polder terminal in general, an adequate water management system needs to be designed to control the water levels and keep the polder dry. The design of this water management system will be done using rules of thumb and assumptions, no detailed calculations will be done because this project will focus on the structural part and risks involved.

Examples of existing polders where industrial activities take place are the refinery in Terneuzen in the Netherlands and the Suvarnabhumi Airport in Thailand. The polder terminal is particularly feasible for larger terminals, at locations with low quality sub soils and high prices for reclamation material; possible cases are Tuas Singapore and Cai Pei Vietnam.
3. Polder terminal; a conceptual design

3.1. Introduction

To investigate the polder terminal a conceptual design is made which will be used to identify important aspects requiring further investigation. The conceptual design is based on the case study for Tuas, Singapore. Local boundary conditions and requirements are used, because these are considered generally applicable for ports where polder terminals could be a good alternative.

*Note: The case study is solely used to identify important aspects and make a design for a possible polder terminal implementation. The following chapters investigate the concept of the polder terminal; while also paying attention to the specific case study for Tuas, Singapore.*

3.2. Tuas implementation

Tuas is located in the west part of the port of Singapore and consists mainly of reclaimed land. The main arguments to propose a polder terminal at Tuas were a scarcity of good quality fill material making reclamation expensive and the presence of weak subsoil consisting of predominantly clay and silt material.

The original shape of the Tuas polder terminal is shown in Figure 19; the pier is not completely rectangular. The conceptual polder terminal designed in this section will have a rectangular plan to avoid difficulties in port planning as illustrated in Figure 20.

Further, at this stage of the project it is not yet clear whether the flood defense system of the polder terminal will consist of solely a quay wall on all boundaries between land and water or a combination of a quay wall and a dike, which is illustrated in Figure 20, both are possible.

*Note: Most of the data presented originates from the tender submission for the Tuas container port which is considered applicable in this conceptual design phase.*
3. Polder terminal; a conceptual design

Figure 19: Location Tuas container pier development [RHDHV]

Figure 20: Polder terminal plan (possible) [RHDHV]

Figure 21: Polder terminal cross section (conceptual)
3.2.1. Terms of reference: boundary conditions

The Tuas project site is located east of the Jurong Island in Singapore; local boundary conditions will be used in the design of the polder terminal. These are explained in the sections below. All levels are related to Chart Datum (CD), which is the Lowest Astronomical Tide (LAT; important for navigational purposes as it determines the minimum depth in a port).

Soil properties
The level of the sea bed is averaged at -20 meter CD with variations to -26 meter CD and -16 meter CD in the area of the future pier. Site Investigations (SI), obtained from 200 boreholes, laboratory tests and geotechnical cross sections along the proposed perimeter of the container terminal, indicate that the site is underlain by the Jurong Formation: a variety of sedimentary rocks which include beds of tuff, siltstone, mudstone, sandstone and limestone. The formation can be deeply weathered producing thick layers of residual soils consisting of mainly silt or clayey sand. Table 13 contains an overview of local soil characteristics [XVII].

Note: The layers present are indicated with a number ‘N’ which is related to the Standard Penetration Tests performed on site (blows/300mm).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Level [..m CD]</th>
<th>$\gamma_c$ [kN/m$^3$]</th>
<th>$\gamma_{sat}$ [kN/m$^3$]</th>
<th>$\phi'$ [degrees]</th>
<th>$c'$ [kPa]</th>
<th>$cu$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy silt</td>
<td>-20m CD</td>
<td>18</td>
<td>18</td>
<td>22.5</td>
<td>5</td>
<td>80</td>
</tr>
<tr>
<td>N=30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandy silt</td>
<td>-30m CD</td>
<td>20.5</td>
<td>20.5</td>
<td>25</td>
<td>14</td>
<td>145</td>
</tr>
<tr>
<td>N=70</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandy silt</td>
<td>-40m CD</td>
<td>20.5</td>
<td>20.5</td>
<td>25</td>
<td>14</td>
<td>145</td>
</tr>
<tr>
<td>N=100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 13: Soil assumptions [XVII]

Hydraulic properties
This section treats the hydraulic boundary conditions at the project site. Tuas is located in the port of Singapore where water has an annual average temperature of 29° Celsius and a weight of 1.025 kg/m$^3$. Information on sea water levels, waves and climate is given in the next sections.

Sea water levels
The variations in water levels are presented in Table 14 [XVII]. A sea level rise of 25 centimeters is taken in to account over an expected lifetime of 50 years. The quay wall height for the container terminal proposed in the tender submission has a crest level of +6 meter CD.
The probability of a certain high water level (overtopping) is expressed in equation 3-1, to determine the parameters A and B a statistical analysis of observed water levels is made. Parameter A represents a level more or less equal to the Highest Astronomical Tide (higher levels are considered extreme water levels); while parameter B represents the slope of the function when plotted on a logarithmic scale.

\[ P_f = e^{\frac{h_A - A}{B}} \]  (3-1)

Hourly sea level observations were used over a period of 30 years (1981 to 2011), the data was obtained at the University of Hawaii Sea Level Center [vi]. The Peak Over Threshold method [XXVIII] is applied to obtain statistically independent extreme water level data that consists of monthly maximum water levels higher than the POT level. The POT level is set at +3.2 meter CD, which coincides with the Highest Astronomical Tide level, after which a total of 73 extreme water levels obtained (approximately 2.4 high water levels per year), see Figure 22.
3. Polder terminal; a conceptual design

Figure 23: Extreme water level regression Tuas (A = 2.87 and B = 0.15)

Figure 24: Extreme water level regression (logarithmic scale)

Waves
Wave conditions are divided in wind waves, swell waves and ship waves. The wave climate in the area is dominated by wind and ship waves as the pier is sheltered from large storm waves (swell) and tsunamis. Table 15 shows the prevailing wave conditions offshore of the proposed port development, dominant waves originate from the West / South-West [XVII] which is illustrated in Figure 25. Tsunamis are not likely to impact the port of Singapore because of its location.

Figure 25: Dominant wave directions
Table 15: Extreme wave conditions in the port of Tuas [XVII]

The maximum wave height is 1.13 meter with a period of 4.5 seconds, originating from the West.

Currents and horizontal tide

In the tender proposal for the Tuas container terminal [XVII] the local current speeds, resulting form tidal and non tidal contributions, on site are estimated. The results are presented in Figure 26; a maximum current speed of 0.6 m/s is possible, which does not pose any large limitations for port infrastructure design.

Figure 26: Current speeds [RHDHV]

Climate properties

Singapore is located 1 degree north of the equator, which is why there is a tropical rainforest climate with no distinct seasons; a relatively uniform temperature is present, together with high humidity and rainfall. The only seasonal differences which occur are monsoons which happen twice a year. The Northeast Monsoon occurs from December to March with widespread heavy rains lasting from 1 to 3 days. The Southwest Monsoon occurs from June to September. Climate characteristics are summarized in Table 16.
3. Polder terminal: a conceptual design

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average temperature</td>
<td>27°C</td>
</tr>
<tr>
<td>Lowest temperature</td>
<td>20°C</td>
</tr>
<tr>
<td>Highest temperature</td>
<td>36°C</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rainfall</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Average annual rainfall</td>
<td>2,340mm</td>
</tr>
<tr>
<td>Normal rain day</td>
<td>13 – 22 mm</td>
</tr>
<tr>
<td>Rain days per year</td>
<td>137</td>
</tr>
<tr>
<td>Highest 24hr rainfalls</td>
<td>512mm (1978) / 467mm (1969) / 366mm (2006)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Northeast Monsoon</th>
<th>Wind direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind direction</td>
<td>Northeast</td>
</tr>
<tr>
<td>Average wind speeds</td>
<td>2.5 m/s</td>
</tr>
<tr>
<td>Wind gusts</td>
<td>14 m/s</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Southwest Monsoon</th>
<th>Wind direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind direction</td>
<td>Southwest</td>
</tr>
<tr>
<td>Average wind speeds</td>
<td>2.0 m/s</td>
</tr>
<tr>
<td>Wind gusts</td>
<td>15 m/s</td>
</tr>
</tbody>
</table>

Table 16: Singapore climate [ii]

For a risk assessment of the water management system a statistical analysis of local rainfall intensities is required; Intensity-Duration-Frequency curves were obtained from Singapore’s natural water agency [v]. Chapter 7 investigates the rainfall in relation to the required drainage capacity in the polder.

![Image](image.png)

*The IDF Curve has been revised to include till 2009 rainfall data.*

Figure 27: IDF Curve Singapore 2009 [v]
Seismic activity

During the last couple of years Southeast Asia has been struck by various earthquakes, sometimes resulting in large tsunamis. Fortunately for Singapore the country is relatively safe from seismic activity, because the nearest fault line lays hundreds of kilometers away in Indonesia. Tremors of earthquakes are not uncommon but do not pose a direct threat. The tsunami resulting from the earthquake in the Indian Ocean in 2004 did not strike Singapore because it is protected by the Sumatra landmass, some slight tremors were felt in high-rise buildings however.

Contrary to the statements above seismic loading is taken into consideration in the conceptual design, even though it is uncommon in Singapore. Polder terminals could be designed for other port areas where earthquakes do pose a threat so when investigating the technical feasibility this should be taken into account.

To quantify the risk of an earthquake the probability of occurrence of earthquakes on site needs to be determined. First the seismic climate of the area is determined, which consist of an analysis of the seismic loading on site and the corresponding probabilities of occurrence. A graph showing the relations between these parameters is shown in Figure 28.

In a risk assessment the horizontal acceleration (and corresponding magnitude) which produces failure of the flood defense system (instability or leakage due to deformations) is determined after which the return period of this earthquake for the specific location is looked up. The return period is directly related to the probability of failure of the structure. For a conceptual design a certain seismic loading is assumed.

![Seismicity and return periods](image)

Figure 28: Seismicity and return periods [XIV]

3.2.2. Terms of reference: program of requirements

The program of requirements contains the relevant requirements for the design of a new polder container terminal. The requirements are divided in different categories; navigational, retaining, protective, bearing and environmental requirements.
3. Polder terminal; a conceptual design

**Navigation requirements**
The navigational requirements are divivded in the identification of a design vessel and berthing requirements.

**Type of vessel**
The design vessel, which is the largest expected vessel in port, is a Malacca MAX vessel (Table 17). These are the largest vessels which can navigate through the Straits of Malacca, near Singapore. The largest container carriers to date are still smaller than the dimensions of a Malacca MAX vessel; the latest Maersk container vessels have a capacity of 18.000 TEU with maximum drafts of 16 meter. It is however advised to design for a Malacca MAX vessel, taking future developments in to account.

<table>
<thead>
<tr>
<th>Malacca MAX vessel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Weight Tonnage</td>
</tr>
<tr>
<td>Length Overall</td>
</tr>
<tr>
<td>Beam</td>
</tr>
<tr>
<td>Maximum draft</td>
</tr>
<tr>
<td>Container capacity</td>
</tr>
</tbody>
</table>

Table 17: Design vessel

Other vessels which would berth at the quay wall for the container terminal are:

- Feeder container vessels and general cargo carriers;
- Material supply and construction supporting barges;
- Dredgers (for construction and maintenance);
- Tugs.

**Number and length of berths**
The overall length of the design vessel determines the length required for each berth. For multiple berths in a straight continues quay the berth length is calculated with equation 3-2. This equation allows for a gap of 15 meters between vessels and an additional 15 meter for the outer berths. An extra margin of 10% of the vessel length is introduced to avoid additional waiting times due to repositioning of vessels at berth during unloading, this was the result of a study of UNCTAD in 1984 [XI]. As shown in the equation below the required length is 550 meter. Along the long sides of the polder 7 berths are possible; assuming only the largest ships make berth which in practice will not happen.

\[ L_q = 1.1 \times (L_s + 15) + 15 \]  
\[ L_s = 470m \]  
\[ L_q = 1.1 \times (470 + 15) + 15 = 548.5m \]

**Retaining requirements**
The retaining requirements are related to the top level of the quay which is determined by the expected water levels as shown in Table 14 and the draft of the design vessel. For a conventional terminal these two
parameters determine the retaining requirements, but for a polder terminal the polder level also has to be defined because another retaining structure is required between the quay wall and the terminal yard. For the Tuas case a polder level of -2 meter CD was estimated. Further investigation is done in chapter 5.

<table>
<thead>
<tr>
<th>Level</th>
<th>Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polder quay wall level</td>
<td>+6m CD</td>
</tr>
<tr>
<td>Polder terminal level</td>
<td>-2m CD</td>
</tr>
<tr>
<td>Conventional terminal level</td>
<td>+6m CD</td>
</tr>
<tr>
<td>Bottom level</td>
<td>-23m CD</td>
</tr>
</tbody>
</table>

Table 18: Retaining levels, see Figure 30

**Bearing requirements**

The bearing requirements are related to the loads imposed by cranes, vehicles and storage of containers or other cargo; these are given in Table 18 and illustrated in Figure 30.

<table>
<thead>
<tr>
<th>Load</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical crane load</td>
<td>750 kN/m</td>
</tr>
<tr>
<td>Horizontal crane load (5% of vertical)</td>
<td>37.5 kN/m</td>
</tr>
<tr>
<td>Distance crane from quay wall front</td>
<td>Minimal 3 m</td>
</tr>
<tr>
<td>Surcharge on quay wall and apron area (up to 65 m from quay wall)</td>
<td>43 kN/m²</td>
</tr>
<tr>
<td>Surcharge in storage area</td>
<td>143 kN/m²</td>
</tr>
</tbody>
</table>

Table 19: Bearing requirements, see Figure 30

**Settlement limitations**

One of the requirements for the Tuas container terminal yard was that the settlement of the subsoil was limited to 40 millimeter under a distributed load of 143 kPa, this requirement is not taken into account. It is however advised to keep the settlements as low as possible; recommendations regarding how to deal with settlement are made in chapter 5.

**Container handling equipment**

The loads of cranes placed on the quay wall to transfer cargo between vessels and terminal need to be transferred to the foundation; the exact loads depend on the type of crane used. The type of crane is dependant on the type of cargo to be transferred, in this case solely containers. Port logistics are greatly dependant on the type of container handling equipment applied. For a polder terminal, a solution must be sought for to overcome the level difference between the quay wall / apron area and the terminal yard.

In the report on the Creative Idea Polder terminal [XIX] a short investigation is made on possible cranes to use in a polder terminal. This report concluded that ‘crane efficiency is determined by ship trolley performance and not by the terminal level’.

An example of a crane that could be used for a polder terminal is a back reach crane with dual trolleys. The crane driver removes the container from a vessel and places it on a platform in the crane, from this platform automated trolleys take the container to the back reach of the crane and places it on an automatically
3. Polder terminal; a conceptual design

guided vehicle in the terminal yard on polder level, see Figure 29. In this case the area between the quay cranes is not used for container handling but will be used to store the latches of the container ships. This system is already applied in large automated terminals such as the port of Altenwerder in Hamburg; the minimum width of the quay wall structure required for such cranes is 40 meters.

Note: Naturally the crane efficiency will change when very large level differences are applied between the quay wall and polder level resulting in longer turnover times for each container. However, for small differences (polder level at Mean Sea Level) this effect is negligible.

Back reach operation with dual trolley

Figure 29: Back reach crane (left) and portal crane (right)

Back reach cranes are specifically made to efficiently transfer containers between vessel and terminal. In areas where not only containers but also other general types of cargo are handled portal cranes are often used. These cranes pose different loads on the structure.

Protective requirements
Protective requirements are related to berthing / mooring facilities and bottom / shore protections.

Berthing and mooring facilities
Along a quay wall berthing and mooring facilities are required for vessels at berth. These facilities mainly consist of bollards and fenders. Bollards and fenders are placed along the quay wall with a separation distance of 20 meters, the resulting bollard force is 1.000 kN (50kN/m’) and fender force is 3.500 kN (175kN/m’) [XVII].

Bottom and shore protection
The design bottom level of -23 meter CD was chosen such that no scour protection would be required, because an over depth of 2 meters is available. Detailed investigation on this assumption is however advised in the detailed design of the quay wall. Shore protections are required when part of the flood defense system is designed as a dike or slope.
Environmental requirements
There is a growing awareness of the environmental impact of dredging works. In Singapore dredging often takes place near environmentally sensitive areas, which can cause numerous impacts on coral reefs and marine life. Mitigation measures to control sediment suspension due to dredging are beyond the scope of this project, however it should be noted that the environmental impact due to dredging can be significantly reduced when a polder terminal is built. The reason is simple; less soil reclamation means less dredging works (shorter construction time) resulting in lower environmental impact.

3.2.3. Result: design framework

The boundary conditions and program of requirements are summarized in Figure 30 which is also shown in appendix E, which shows a situation sketch for the polder terminal. The hydraulic conditions, soil properties, design vessel, quay wall and terminal loading and bearing requirements are illustrated.

Figure 30: Situation sketch, see 3.2.1 for details

Port income estimate
Below an estimate of the annual income for the proposed conceptual design of the Tuas polder terminal is made, based on assumptions regarding container throughput and amount of vessels.

Container throughput
First an estimation of the container throughput is required, since Singapore is one of the largest container handling ports in the world it is assumed that the handling of containers is done efficiently. At the 2nd Maasvlakte 27.500 TEU’s are handled per hectare per year. This same amount is assumed for Tuas in Singapore [II]. With a total area of 320 ha 9 million TEU’s could be handled per year.

Port income
The port income is divided in the direct income of container handling and the income of port dues. It is assumed the largest part of the container throughput of the port of Singapore is transshipment (as opposed to import or export). The income of container transshipment in Singapore is $100 per TEU [I]. Considering
3. Polder terminal; a conceptual design

Port dues and other revenues, it is assumed that a percentage of 10% per TEU is added to the port income, resulting in an income of $110 per TEU. The total income of the proposed polder terminal in Tuas, Singapore, will then amount to $990 million per year.

3.3. Conceptual design polder terminal

The conceptual design of the polder terminal is made in this paragraph.

3.3.1. Polder terminal design

The original Tuas plan consisted of 8,600 m of wharf structure and 320 ha of land reclamation, which coincides with a rectangular plan with a length of 3900 m and a width of 800 m. A conceptual plan for the polder container terminal is made, taking the following required aspects into account:

- Dimensions of quay wall flood defense system;
- Dimensions and location of water storage and drainage system;
- Port infrastructure, including approach ramps for level difference container yard and apron area;
- Container storage areas;
- Other port facilities.

Plan

The plan for the concept container terminal is illustrated in a 3d model in Figure 20, the sketches below show a top view of the proposed polder container terminal plan and a cross section over the width of the terminal. On both sides of the polder terminal and in the center there is space available for out of gauge containers, facilities and a ramp to overcome the height difference between quay wall and terminal yard. The dimensions comply with standards for a fully automated container terminal with back reach cranes; reference is made to the port of Hamburg.
3. Polder terminal; a conceptual design

Figure 31: Polder terminal plan

In the polder terminal plan a water storage / drainage system is present. The proposed design of the channels serves as an illustration of a possible lay out in a preliminary design. An investigation in to the optimal lay out for these channels is required to minimize the amount of bridges required in the polder, which will minimize the cost. The required quay wall height and polder level are not defined, these will be determined in chapter 5.

Figure 32: Polder terminal cross section (half)

Note: The main difference with the conventional terminal plan is the water storage and drainage channels in a polder terminal, which are not present in a conventional terminal.

Reclamation fill

It will be clear that in both a conventional terminal design and a polder terminal design reclamation fill is required. For a polder terminal groundwater discharge and upward seepage can be reduced when using low conductive clayey materials (‘Clay fill’ in Table 20) in the lower parts of the reclamation fill. The upper layers will consist of high permeable granular material (‘Sand fill’ in Table 20) which allow for accurate controlling of the groundwater levels inside the polder, these layers also provide additional water storage. Table 20
3. Polder terminal; a conceptual design

contains an overview of the soil build up of the proposed polder terminal [XVII]. The fill consists of a clay layer on top of which a sand fill is made.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Level [m CD]</th>
<th>( \gamma_c ) [kN/m(^3)]</th>
<th>( \gamma_{sat} ) [kN/m(^3)]</th>
<th>( \phi' ) [graden]</th>
<th>( c' ) [kPa]</th>
<th>( c_u ) [kPa]</th>
<th>( C_{10} )</th>
<th>( k ) [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand fill</td>
<td>-20m CD</td>
<td>17</td>
<td>20</td>
<td>30</td>
<td>0</td>
<td>-</td>
<td>100</td>
<td>( 10^4 ) - ( 10^5 )</td>
</tr>
<tr>
<td>Clay fill</td>
<td>-23m CD</td>
<td>15</td>
<td>15</td>
<td>22.5</td>
<td>0</td>
<td>40</td>
<td>20</td>
<td>( 10^3 ) - ( 10^{11} )</td>
</tr>
<tr>
<td>Sandy silt</td>
<td>N=30</td>
<td>-25m CD</td>
<td>18</td>
<td>18</td>
<td>22.5</td>
<td>5</td>
<td>80</td>
<td>30</td>
</tr>
<tr>
<td>Sandy silt</td>
<td>N=70</td>
<td>-30m CD</td>
<td>20.5</td>
<td>20.5</td>
<td>25</td>
<td>14</td>
<td>145</td>
<td>40</td>
</tr>
<tr>
<td>Sandy silt</td>
<td>N=100</td>
<td>-40m CD</td>
<td>20.5</td>
<td>20.5</td>
<td>25</td>
<td>14</td>
<td>145</td>
<td>50</td>
</tr>
</tbody>
</table>

Table 20: Soil assumptions [XVII]

![Figure 33: Cross section polder terminal: soil layers](image)

Soil improvements

Soil improvements can be done by compaction of the sand fill and applying a temporary surcharge. Vertical drains will be required to accelerate consolidation of the clayey material, so construction can take place not long after the fill is complete. Details on soil improvement works are beyond the scope of this master thesis.

Costs

The reclamation costs at Tuas were estimated at 25 USD/m\(^3\) for sand and 10 USD/m\(^3\) for clayey material. A combined value of 20 €/m\(^3\) will be used. A small overestimation is the result, because the first layers consist of clay, which is considered acceptable. This price contains the cost of the fill material in combination with the soil improvement cost (vertical drainages), assuming both have a price of 10 €/m\(^3\).
3. Polder terminal; a conceptual design

3.3.2. Flood defense system

Traditional polders are constructed with a dike (soil structure) as flood defense system, which is the cheapest option. Container port terminals require a berth along the whole length of the ships to transfer containers between the terminal and the ship efficiently.

The flood defense system of the polder terminal will therefore consist of solely quay walls, both along the long sides of the pier and along the short side. Constructing a dike along the short side of the pier is also possible, as illustrated in Figure 20, but the following arguments explain why a quay wall is considered the better option:

- The transitions between dike and quay wall are vulnerable areas for flooding;
- The savings in cost when constructing a dike are expected to be marginal, because the length of the dike is relatively short. As the long sides are already designed as quay walls it is relatively easy to also make a quay wall along the short side;
- A longer quay wall increases the capacity of the port and possibly provides as good option to be used as a quay wall for barges or other type of ships requiring a slightly different berth. This way the long sides could be used completely for the large container vessels;
- A dike requires more space, with slopes of 1:3 and a retaining height of 30 meters on sea side and another 8 meters on the land side the total width of the dike would be about 130 meters (including the crest) which is disadvantage in areas where space is scarce, see Figure 34.

The remainder of this paragraph will focus on identifying the type of structures to serve as a quay wall flood defense for a polder terminal. The lower surface level of the terminal yard could have significant influence on the behavior of the quay wall. The flood defense system of the polder terminal actually has two functions. Firstly the flood defense system needs to provide sufficient safety against flooding while also facilitating vessel berthing. Possible structures to fulfill these functions are explained below.

- Gravity structures, in Table 2;
3. Polder terminal; a conceptual design

- Sheet pile structure, in Table 3;
- Pile supported platforms, in Table 4.

Aspects which will be addressed are water tightness, material, applicability in bad sub soils, combination of sea and landside structure and the crane foundation. Constructability and construction time are not treated in the concept phase of the polder terminal because these depend on local conditions.

**Gravity structures**

Gravity-type structures obtain stability through their own weight and friction between the structure and foundation. Aspects regarding the applicability as a quay wall flood defense structure are mentioned below:

![Figure 35: Gravity structure types](image)

- Possible structures: a block wall, an angle type wall, floated in caissons and silo's;
- The structure is water tight, so it is applicable as combined quay wall flood defense, attention should be paid to the connections between adjacent structures;
- The base material is concrete, prefabrication is possible. The very large elements (retaining height over 25 meters) make transportation over large distances difficult and costly;
- These structures are usually not applied in areas with bad sub soil because they require large bearing capacities, the weight often induces settlement and creep of the subsoil;
- The landside retaining structure, between quay wall and polder terminal, could be combined by making large caissons or silo’s;
- The seaside crane foundation is usually placed on the gravity structure; landside crane foundation is placed on a separate structure which could result in uneven settlements for the foundations.

*Note: A study of the evolution of container wharf structures in Singapore of the last 30 years [VI] show that most of the quay walls for larger container carriers in the area are built with caisson gravity structures, despite the bad sub soils. The soft clay layers where dredged away and replaced with good quality fill material followed by an expensive soil improvement (deep compaction and preloading) to limit the settlements.*

**Sheet pile structures**

Sheet-pile type quay walls consist of a wall which is driven or constructed in the subsoil below the port bottom level.
Three types of structures are possible: a sheet pile wall, a combi wall and a diaphragm wall (see Table 3), possibly in combination with a relieving platform;

- The structure is water tight, so it is applicable as combined quay wall flood defense;
- The base material is steel (or concrete for a diaphragm wall), which is subject to corrosion (especially in tropical areas) prefabrication of the steel sheets and piles is possible;
- These structures are often applied in areas with bad sub soils;
- Both the seaside and landside crane foundation can be made on separate sheet pile walls;
- These structures require anchors in the soil which provide additional stability against soil pressures, when designing both the seaside and landside retaining structure as a sheet pile wall both walls could be connected through the anchors;
- If both land and sea side retaining sheet piles lay in each others influence zones the structure could behave as a cofferdam, depending on the dimensions of the quay wall.

For the Tuas polder terminal a sheet pile structure (combi wall) was designed by Royal HaskoningDHV.

**Pile supported platforms**

With a pile supported platform the difference in height between the port terminal and the port bottom is not overcome by a soil retaining structure as in previous examples, but by a separate soil retaining structure (an embankment is often applied). The vessels are then moored along the jetty type structure.

The structure in itself is not water tight, so a separate water retaining structure is required which could make the total structure more expensive than the previous examples (see Table 4). When combined with an embankment to retain water a lot of space is required to overcome the large retaining height of over 25 meters (as with dikes);

- The base materials for the pile supported platform are steel and concrete, an embankment will consist of soil;
- This structure can be applied in areas with weak sub soils; uneven settlements could occur because the pile supported platform will generally not settle whereas the reclamation for the revetment will;
3. Polder terminal; a conceptual design

- A separate structure is required to overcome the height difference on the landside, between the apron area and the polder terminal yard;
- The cranes can be founded on top of the platform.

**Final conclusion**

The following table gives a summary of the aspects mentioned above; more information on this subject is treated in chapter 6. Costs aren’t added because the cost of quay walls related to the retaining height is fairly constant irrespective of the type of structure, as concluded in research performed by J.G. de Gijt [VII], more details on difference in cost between the different structures is investigated in chapter 6.

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Gravity structure</th>
<th>Sheet pile structure</th>
<th>Pile supported platform</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water tightness</td>
<td>+</td>
<td>+</td>
<td>+/-</td>
</tr>
<tr>
<td>Durability</td>
<td>+ concrete</td>
<td>- steel</td>
<td>- steel</td>
</tr>
<tr>
<td>Applicable in bad sub soil</td>
<td>- settlement &amp; creep</td>
<td>+</td>
<td>- uneven settlements</td>
</tr>
<tr>
<td>Space required</td>
<td>+</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>Possibility to combine with polder side retaining structure</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Provide foundation crane rails</td>
<td>- uneven settlements</td>
<td>+</td>
<td>+</td>
</tr>
</tbody>
</table>

Table 21: Comparison quay wall structures

There is not one structure which is most suitable for a polder terminal in general; this all depends on the local boundary conditions. More details on this subject are given in chapter 6.

**Tuas polder terminal quay wall**

Considering the aspects mentioned in the corresponding sections a sheet pile or gravity structure are considered the most suitable structures as a combined quay wall flood defense system for Tuas.

A pile supported platform is not practical because it is not water tight and when combined with a separate retaining structure (revetment) a lot of space (and soil) is required to overcome the height difference of 30 meters.

For sheet pile walls attention should be paid to the material, because steel is not very durable in warm salty environments. When applying gravity structures the expected differential settlements of the structure should be avoided through soil improvement works.

**3.3.3. Polder water management**

As explained in the literature study a polder requires a water collection, storage and drainage system to drain overtopped, seepage and rain water. Collection of water is mainly done through gullies and drain pipes
as on any other conventional terminal, drainage can be done through gravity flow or pumps. Storage is provided by water channels inside a polder.

A balance needs to be found between the drainage capacity and required storage capacity: more storage capacity requires less drainage capacity and vice versa. If the drainage capacity of the polder is large enough hardly any storage capacity is required.

**Water storage**

Two options are possible to deal with water storage: open water storage and water storage inside the quay wall flood defense system, a combination is also possible. Detailed calculations are made in chapter 7; a qualitative description of important aspects follows below.

**Water storage in the quay wall**

Efficient spatial planning in the design is required; in the original design for the Tuas container polder terminal the water channels were incorporated in the quay wall and dike structure, as shown in Figure 38. This way no space is lost inside the polder for water storage, however it introduces an additional risk because there is no experience with the design and construction of such quay walls. Further, it could be more costly than constructing open water channel in the terminal yard.

![Figure 38: Water channel inside quay wall structure](XIX)

**Open water storage**

For a conceptual design open water storage is proposed, as illustrated in Figure 31: a main channel in the middle of the polder with a width of 30 meters and 7 secondary channels or pipes with a width of 10 meters, together covering 5% of the total polder area. As stated before the proposed lay out serves as a preliminary design and an optimization of the lay out to minimize the amount of bridges is required.

A disadvantage of open water inside the polder terminal is the loss of space; this could be reduced by placing concrete plates over the smaller channels on top of which containers can be placed or by replacing the smaller channels by pipes with the same capacity. This is illustrated in Figure 31 and results in a design with only one main channel in the polder. Further, additional storage could be provided by using gravel beds in the container storage area.
3. Polder terminal; a conceptual design

Note: If the design storage area is exceeded flooding of the whole terminal can (temporarily) be avoided by designing the polder surface to have a slope, the lower parts of the polder could then (temporarily) provide extra storage during a storm

Drainage system
The drainage of the polder could be done through gravity flow, which is the traditional way, or with pumps. Gravity flow is only possible for polder levels higher than the Low Astronomical Tide, for lower levels pumping stations are required. For pumping stations usually a combination of electricity and diesel driven engines are used, to avoid problems during power failure.

Most pumping stations are placed in the lowest part of the polder. In very large polders the preferred location is usually along the long sides of the polder to limit the maximum distance the water has to travel to the pumping station. Polder terminals are relatively small, so a location along the shorter side is also possible, which is preferred so the long sides can be used completely for vessel berthing.

In areas where fresh drinking water is scarce (and expensive) which is the case at Tuas, the water from the polder could be pumped to the landside of the polder instead of to sea. Here, the water could be collected in a reservoir providing a new source for fresh water.

3.4. Comparison conventional terminal

For comparison purposes, the cross section of a conventional terminal is shown in Figure 39.

Figure 39: Conventional terminal cross section (half)
3.4.1. Advantages

- All design and construction methods for a polder are proven technology;
- The polder terminal concept can be applied in any low lying area in the world;
- The polder terminal requires less fill volume, resulting in shorter construction period, lower reclamation cost and less environmental impact due to dredging and construction works;
- Because the yard is not raised fully, less settlement of the subsoil is expected. This is especially attractive for soft soil types often found in river deltas. This way, expensive soil improvement can be avoided or reduced;
- The level transition just landside of the landside crane rail is fully compatible with layout requirements for modern dual-trolley ship to shore gantry cranes;
- If the sea level rises faster than forecasted, only the apron dike would need to be raised. There would be no need to raise the entire terminal which would be more economical and cause fewer disturbances to port operations;
- The visual impact and noise pollution of the container terminal yard are reduced because the terminal level and thereby the top level of the container stacks is lower.

3.4.2. Disadvantages

- Savings on reclamation cost could be partially offset by the extra operational cost of a water drainage system;
- The advantage of adaptation to sea level rise is also partially offset by a larger required drainage system to drain seepage water;
- If open yard surface is used for water storage, the total terminal area will be larger than without it;
- Additional space is required for ramps between terminal yard and apron area;
- The damage potential and corresponding risk in case of flooding could be larger than for a conventional terminal;
3. Polder terminal; a conceptual design

3.5. Conclusions and recommendations

A conceptual design of the polder terminal is made in the last sections, which is mainly based on the Tuas case. A comparison of a polder terminal and a conventional terminal is made in the last paragraph. Important conclusions regarding different aspects of the polder terminal are made below; detailed investigation on the different aspects will follow in the next chapters.

The flood defense system of the polder will consist of a quay wall on both the long and short side, the possible saving of costs when constructing a dike on the short side are expected to be low. Another disadvantage is the large space required for the slopes of a dike. The final quay wall and polder level are investigated in chapter 5.

Depending on the local conditions a choice can be made for a type of structure which is best applicable in that case. Possible structures are a gravity structure, a sheet pile wall and a pile supported platform. More investigation on this subject follows in chapter 6.

The water management system could contain open water storage or storage channels inside the quay wall flood defense. A conceptual design contains 5% open water storage; gravity flow is only possible for polder levels higher than Low Astronomical Tide, otherwise pumping stations are required. More investigation on this subject follows in chapter 7.

The reclamation fill of the polder terminal contains a layer of clayey material, to limit the amount of upward seepage, on top of which a sand fill is made till the final terminal level.
3. Polder terminal; a conceptual design
4. Risk assessment

4.1. Introduction

A general risk assessment is made to identify the flooding events of a polder terminal and the failure mechanisms which could induce flooding. The risk assessment is used to determine the detailed design approach of the polder terminal.

A distinction is made between temporary water hindrance, defined as ‘Small scale flooding’, and actual permanent inundation, defined as ‘Large scale flooding’. The following paragraphs will elaborate:

- ‘Small scale flooding’; (temporary) flooding with water depths up to 0.5 m;
- ‘Large scale flooding’; large scale (permanent) flooding, with water depths over of 0.5 m.

The failure probabilities presented in Table 22 and Table 23 are mainly based on assumptions or available literature. Its purpose is to identify the most important failure mechanisms requiring further investigation.

4.2. Small scale flooding

Small scale flooding is related to a temporary situation where water depths below 0.5 meter occur due to excess water inside the polder in relation to storage and drainage capacity. This is defined as the serviceability limit state. The consequences of some flooding are temporary down time of port operations and some damage to containers and other port facilities. Failure mechanisms, which could have small scale flooding as a result, are illustrated in the following fault tree.
4. Risk assessment

**Figure 40: Fault tree small scale flooding**

<table>
<thead>
<tr>
<th>Main mechanism</th>
<th>Sub mechanism</th>
<th>Argument</th>
<th>Failure probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seepage</td>
<td>Seepage</td>
<td>The seepage length can be determined using Lane (for vertical elements); the flood defense system can be designed such that the total amount of seepage is low, resulting in a low probability of flooding due to seepage water/piping, see chapter 6.</td>
<td>$\sim 10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>Heave / Piping</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overflow</td>
<td>Overflow / Overtopping</td>
<td>Overtopping and overflow can be neglected when designing a sufficiently high crest level of the flood defense. This level needs to be chosen based on the corresponding investments and risks involved, as is investigated in the next chapter.</td>
<td>$\sim 10^{-4}$</td>
</tr>
<tr>
<td>Rain</td>
<td>Excess rain</td>
<td>Some flooding due to rain occurs when during a short amount of time very large quantities of rain fall, see Table 16. A statistical analysis of the expected rainfall is required to determine the storage and drainage capacity, see chapter 7.</td>
<td>$10^2 \sim 10^3$</td>
</tr>
</tbody>
</table>

Table 22: Failure probabilities SLS

Notice that pump failure is not taken in to account in the main fault tree, because flooding due to pump failure occurs in combination with one of the 3 main events (seepage, overtopping or rain). If one would include pump failure in the fault tree it would be included one level below the three main events.

Pump failure could occur in two ways, the first being a breakdown of the pump itself (which could be solved when adding an auxiliary pump) and the second considers failure due to insufficient pumping capacity which naturally should be avoided.
4. Risk assessment

The drainage pumps (and storage capacity) need to be designed with enough capacity to cope with the maximum hourly rainfall possible, then small scale flooding could only occur if this design drainage capacity is insufficient (due to design errors);

For moderate rains small scale flooding could only occur if the storage capacity is exceeded and pumps fail due to a breakdown, to avoid this situation an auxiliary pump (or multiple pumps) is advised;

Further, because the drainage capacity of the pumps is based on the maximum hourly rainfall (which is assumed to be much higher than the inflow due to seepage and overtopping) the pumps will have no problem draining the inflow of seepage and overtopped water.

4.3. Large scale flooding

Large scale flooding is defined as inundation with water depths over 0.5 meter; this situation is related to failure of the flood defense system and is defined as the ultimate limit state. Apart from damage to flood defenses other consequences of large flooding are long down times of the port and damages to containers and port facilities. Failure mechanisms which could have large flooding as a result are shown in the fault tree illustrated in Figure 41.

Considering the structural stability of quay walls three safety categories are distinguished in the NEN-standards [V], each safety category defines a maximum probability of failure of the system during a certain reference period. To guarantee this probability of failure each failure mechanism is calculated by means of a probabilistic analysis in which the parameters are considered stochastic; in this case a structure is considered to have failed when it can no longer fulfill one or more of its main functions.

The combined quay wall flood defense system of a polder terminal is part of Safety Category 3 which is valid for quay walls serving as a flood defense with a high risk of danger to life and a high risk of economical damage. The corresponding failure probabilities are explainied in Table 23.
4. Risk assessment

Figure 41: Fault tree large scale flooding

<table>
<thead>
<tr>
<th>Main mechanism</th>
<th>Sub mechanism</th>
<th>Argument</th>
<th>Failure probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seepage</td>
<td>Uplift / Heave / Piping</td>
<td>Failure of the structure do to uplifting, heave or pipping can be avoided by increasing the seepage length as defined by Lane, resulting in a low probability of failure, see chapter 8.</td>
<td>$10^{-4}$</td>
</tr>
<tr>
<td>Overtopping</td>
<td>Overflow / Overtopping</td>
<td>Failure of flood defense structures due to overflow / overtopping is directly related to the total amount of overtopped water. This amount can be kept sufficiently low by increasing the crest height (against relatively low cost), which is investigated in the next chapter.</td>
<td>$10^{-4}$</td>
</tr>
<tr>
<td>Structural instability</td>
<td>Internal instability</td>
<td>As part of Safety category 3 in NEN6700/6702 [V] a $\beta$ value of 3.6 in ULS is used for the main mechanism, while Rijkswaterstaat uses a $\beta$ value of 4.5. The corresponding reference period is 50 years, which is the design lifetime of the structure. This value is applied for the main mechanism which is structural instability; the summation of failure probabilities of the sub mechanisms together may not exceed this value.</td>
<td>$10^{-3}$ to $10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>Anchor failure</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Material failure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Geotechnical instability</td>
<td>Foundation failure</td>
<td>See explanation above</td>
<td>$10^{-3}$ to $10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>Macro instability</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calamities</td>
<td>Earthquakes</td>
<td>Singapore is relatively safe from seismic activity, however because polder terminals could also be applied elsewhere in</td>
<td>$10^{-5}$</td>
</tr>
</tbody>
</table>
4. Risk assessment

<table>
<thead>
<tr>
<th>Event</th>
<th>Description</th>
<th>Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>South East Asia moderate seismicity</td>
<td>Assumed, at this stage a failure probability of $10^{-5}$ is assumed.</td>
<td></td>
</tr>
<tr>
<td>Ship collision</td>
<td>The port of Singapore is congested resulting in a significant number of ship collisions per year. A probability of $10^{-4}$ per year for ship – quay wall collision resulting in structural damage is assumed.</td>
<td>$\sim 10^{-4}$</td>
</tr>
<tr>
<td>Terrorism</td>
<td>No information available</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 23: Failure probabilities ULS (from ‘Handbook quay walls’ [V])

An investigation into failure mechanisms for quay walls was made in the handbook of quay walls [V] which provides more insight in the probabilities and interdependencies of the sub mechanisms of geotechnical and structural instability, the failure probabilities of the sub mechanisms were taken from the fault tree on page 137 of the handbook [V].

One important difference between the fault tree in the handbook and the fault tree in Figure 41 is that the top event of the latter fault tree is related to flooding which is a different top event than failure of the quay wall as defined in the handbook. Failure of the quay wall, whether resulting from structural or geotechnical failure mechanisms, will cause leakage of the flood defense structure; it is assumed that when there is a leak present it will grow resulting in flooding of the polder. Of course this depends on the level difference of the outside water and polder level.

Another difference between the behavior of the quay walls considered in the handbook and the quay wall for a polder terminal are the different types of loading in the ultimate limit state. In a polder terminal the top layer of the soil behind the quay wall is not present as in a conventional terminal. Also, during a flood of the polder terminal a large hydraulic load is present on the inside of the quay wall. Investigation into the effects the different loading situations have on the quay wall flood defense is made in chapter 6.
4.4. Conclusions and recommendations

Small scale flooding

‘Small scale flooding’ is related to a temporary inundation of the polder and is the result of an inflow of seepage water, overtopped and rain water.

The most important design factor to take in to account when looking at small scale flooding is the maximum hourly rainfall, the water drainage (and storage) system needs to be designed to drain this amount of water. To achieve this situation the design of the flood defense system needs to assure that a limited amount of seepage water and overtopping can be expected, which can easily be done by raising the crest height of the flood defense system and increasing the seepage length through seepage screens. Further investigation of ‘small scale flooding’ is made in chapter 7.

Large scale flooding

‘Large scale flooding’ is defined as permanent inundation with water depths over 0.5 meters; this situation is related to failure of the flood defense system. Not all failure mechanisms defined in the fault tree directly lead to large scale flooding.

The first step to assure safety against flooding is determining a sufficiently high crest height. A flood risk approach is made to determine this level, with overtopping as the dominant failure mechanism. This approach is elaborated in chapter 5.

Probabilistic design methods are then applied to verify the failure probabilities the instability failure mechanisms. Further investigation of these failure mechanisms and their influence on the design of the quay wall flood defense of the polder terminal is made in chapter 6.

Failure due to calamities requires further investigation, especially the behavior of quay walls subject to ship collision and earthquakes as these can result in large deformations of the structure resulting in leakage.
5. Risk framework

5.1. Introduction

This chapter will focus on the determination of the costs and risk of a polder terminal in order to optimize the levels of the quay wall flood defense and the polder level. The goal is to determine if a polder terminal is an economically attractive alternative for a conventional terminal. To do so a flood risk approach is made which is an optimization between the investments and risk of the terminal. This approach considers overtopping as the dominant failure mechanism of ‘large scale flooding’ which will determine the required quay wall flood defense height and polder level.

Economical scale of optimization

When making an economic assessment of a container terminal a large amount of factors and stakeholders can be taken in to account; not only the port operator itself will benefit but also second and third parties (local transport companies) in a country as well as the national government. This all depends on whether or not the port is used for transshipment or import/export.

The polder terminal design is proposed to save money in the construction phase of the terminal; to the benefit of the client who requires a new container terminal (port owner/operator). Naturally, a new container port also has influence on a larger scale (macro economical or political) because of the increase in employment, the total turnover and increase in port capacity which are all aspects from which second and third parties and the national government will benefit.

These aspects however do not directly influence the construction cost of the terminal and will thus be omitted at this stage. The investigation will take place on a micro economical scale where profit for the client (port operator) is the most important factor. This corresponds with a transshipment port where the influence on the local economy is smaller than in an import/export terminal.

5.2. Risk framework

If both a conventional and a polder terminal are built with the same crest height, the investment of the conventional terminal is higher than the investment for the polder terminal; because of the larger fill required for the conventional terminal.
However, the risk of the conventional terminal is expected to be lower than that of the polder terminal: the inundation depth is related to the extreme water level inducing the flood, as shown in Figure 42. The inundation depth of the conventional terminal is equal to the difference in height between the water level and the terminal level; a polder terminal will however ‘fill up’ completely with water during an extreme water level. Accordingly, the damage (and corresponding risk) of the polder terminal will be higher than that of the conventional terminal.

So, given a certain terminal level, a conventional terminal is expected to have higher investment cost and a lower risk whereas a polder terminal will have lower investment cost and a higher risk. An optimum between both situations is found by minimizing the total cost function, containing the investments and risks.

It is determined how the costs (investments) increase with a higher quay wall and polder elevation, these parameters determines the ‘strength’ of the system. The quay wall height related probability of inundation due to overtopping decreases and thereby the risk (probability x damage), this parameter is considered the ‘load’ of the system. The summation of the investments and risk is called the goal function:

\[
\text{Total cost} = \text{Investment} + \text{Risk} \quad \text{(5-1)}
\]

For a conventional terminal the decision variables (variables which determine the total costs) consist of the terminal level $h_t$, whereas for a polder terminal the decision variables consist of the quay wall height $h_q$ and the polder level $h_p$, as illustrated in Figure 42.

**Note:** For this approach only the investments which differ between both terminals are taken in to account; the reclamation and quay wall cost. Other investments required, such as costs of terminal infrastructure, are independent of the choice of a conventional or polder terminal and can therefore be omitted.

A conceptual graph containing the quay wall height related investments, risk and total costs for both a conventional and a polder terminal is illustrated in Figure 43.
Figure 43: Optimization quay wall height (conceptual graph)

Using this approach one could determine the quay wall elevation and polder level for which the total costs of a polder terminal are lower than that of a conventional terminal.

Note: In this method only flooding due to overtopping is considered. Other failure mechanisms are neglected when determining the required quay wall and terminal level, because it is assumed that mechanisms such as structural or geotechnical instability, seepage and calamities are independent of the overtopping mechanism. The design of the flood defense system should guarantee a sufficiently low probability of failure for each of these failure mechanisms, through modern probabilistic methods.

5.2.1. Risk framework approach polder terminal

The risk framework approach will provide a basis for determining the quay wall height and polder level for which it is economically feasible to design a polder terminal instead of a conventional terminal.
5. Risk framework

**Investments**

The investment costs of the terminal are divided in the quay wall cost and the reclamation costs. For a first estimate it is assumed the quay wall cost \( I_q \) is proportional to the retaining height \( h_q \) and that the reclamation cost \( I_p \) is proportional to the polder level \( h_p \). The construction cost can thus be expressed as is done in equation 5-2.

\[
I_{\text{polder}} = I_q \times h_q + I_p \times h_p
\]  

(5-2)

*Note: The variable quay wall cost, \( I_q \) is assumed to have a linear relation to the retaining height. In section 2.2.4 the relation between quay wall cost and retaining height is given, which is not linear. At this phase a linear relation is considered applicable.*

Equation 5-2 only contains the investments required for reclamation and quay wall construction. Part of the reclamation cost \( I_p \) contains soil improvement cost which is assumed equal for both the conventional terminal and the polder terminal; more details on the consequences of this assumption are given in paragraph 5.4.

**Risk**

The flood risk is equal to the probability of inundation \( P_f \) multiplied by its consequence \( D = \text{damage} \).

**Probability of inundation**

The required quay wall height is determined by the overtopping failure mechanism. During overtopping the inundation of the polder depends on the probability distribution of the extreme water levels which is expressed in equation 5-3. The probability that an extreme water level exceeds the quay wall height, which is not subjected to wave attack, \( [P_f] \) is given in equation 5-4; it depends on the parameters \( A \) and \( B \) of the extreme water distribution. The influence of these parameters on the risk framework is investigated in chapter 8.
5. Risk framework

\[
\begin{align*}
F_{\text{Hw}}(H_w) &= 1 - e^{-\frac{H_w - A}{B}} \quad (5-3) \\
F_a &= e^{-\frac{H_a - A}{B}} \quad (5-4)
\end{align*}
\]

Note: The probability distribution of the extreme water levels is assumed exponentially, while not all extreme water level distributions behave exponentially. This should be verified when a specific case is considered. Further, it is assumed polder terminals are built in port areas sheltered from large waves so no wave attack is taken in to account.

In practice not only overtopping failure but also other failure mechanisms (seepage, instability, calamities) could result in flooding of the polder, however when determining the required polder levels these are assumed to be of less importance than the overtopping mechanism (safety against these mechanisms should be guaranteed through a probabilistic design). These failure mechanisms could however be taken in to account in numerous ways, as mentioned below. When doing so a total probability of inundation \(P_{\text{inundation}}\) is obtained which will be in the order of 1.1 times the original probability of inundation (an increase of 10% which is commonly used for dikes [XXVIII]), as shown in equations 5-5 and 5-6.

- By designing with an additional failure budget of 10% of the overtopping failure probability;
- By designing with failure probabilities as defined in paragraph 4.3, obtained from the NEN-standards;

\[
\begin{align*}
P_f &= P_{f,\text{overtopping}} + P_{f,\text{seepage}} + P_{f,\text{instability}} + P_{f,\text{calamities}} \quad (5-5) \\
P_{f,\text{tot}} &= 1.1 * P_{f,\text{overtopping}} \quad (5-6)
\end{align*}
\]

**Damage**

In order to quantify the risks involved in the design of a terminal the consequences of a flood need to be determined. The damage due to a flood is hard to define; society often defines the total as the summation of material and economic damage, loss of life and immaterial damage such as trust in the water defense system.

For practical purposes only the material and economic damage is used (loss of life and trust is hard to quantify). Distinction is made between the direct consequences of flooding, which result from damage to the port facilities and quay walls, and indirect consequences (economic damage) of flooding, which are the result of long down time of the port. It should be noted that the damage function defined may not correctly describe society’s perception of the total loss during a flood [XXVI].

The damage to facilities depends on the inundation depth: the higher the water level above the ground surface (up to a certain maximum) in the flood defense area, the greater the damages. It was concluded in paragraph 2.5 that the study of Tebodin 1998 provides the most reliable method to determine the damages due to inundation for a container terminal; the corresponding damage function is shown in Figure 45.
5. Risk framework

Figure 45: Damage function container terminals [XIX]

Note: Even though this figure was obtained through a study of damage costs for container terminals it should be used very carefully. From the figure can be concluded that with an inundation depth of 5 meter a factor of 45% should be used, while most of these functions contain a factor of 100% for inundations depths of 5 meter.

In order to compare the damages of a conventional terminal and a polder terminal a damage function is assumed which is a summation of a constant level of damage during ‘limited flooding’ [XXVI] \( D_0 \) in equation 5-7) and damage which is dependant on the inundation depth \( D_i \) in equation 5-7). The inundation depth is in fact the level difference between the quay wall height and the polder level.

As a first approximation it is assumed that during overtopping the whole polder is flooded immediately (corresponding with an inundation depth of \( hq-hp \)) not taking a time factor in to account, as illustrated in Figure 46. In reality the inundation depth of the polder will increase in time, possibly inducing a relation more along the curved line in the graph: at the start of the flood the inundation depth will increase fast, due to the large head difference between the outside water level and inside polder level. While later the flood speed will reduce (as seen in the graph) due to the smaller head difference. The time required to fill up the polder during a flood is approximated in chapter 8.

Figure 46: Relation inundation depth with outside water level (conceptual graph)
5. Risk framework

\[ D_{\text{polder}} = D_0 + D_1 (h_q - h_p) + D_2 (t_{\text{flood}}) \] (5-7)

The indirect damage, damage due to down time of the port \([D_t]\), is determined by the duration of the flood, reconstruction time and the business losses per unit of time. When summing these functions one obtains the total damages due to a flood which is expressed in equation 5-7. For a given extreme water level the inundation depth and duration of the flood need to be determined.

*Note: This function assumes a linear relation between the material damage and inundation depth which in practice is not the case as can be seen in the graph of Figure 45. Further, the damage to quay walls is not taken in to account as it is assumed quay walls do not breach due to overtopping failure. This will be investigated further in chapter 8.*

**Risk**

Risk is equal to the probability of inundation (equation 5-4) multiplied by its consequence (equation 5-7). For comparison purposes the risk has to be discounted with the reduced interest rate \(r'\) (interest reduced by inflation and increased with the increasing economical growth in the area), to obtain the present value of the risk. The risk is expressed in equation 5-8 and 5-9.

\[
R_{\text{polder}} = \frac{P_f \cdot D_{\text{polder}}}{r'}, \quad (5-8)
\]

\[
R_{\text{polder}} = e^{\frac{h_q - A}{B}} \left[ D_0 + D_1 (h_q - h_p) + D_2 (t_{\text{flood}}) \right] \frac{r'}{r'}, \quad (5-9)
\]

**Total cost**

The goal function (total cost is investment plus risk) in relation to the decision variables (quay wall height and polder level) is expressed in equation 5-12.

\[
C_{\text{tot.p}} = I_{\text{polder}} + R_{\text{polder}} \quad (5-10)
\]

\[
C_{\text{tot.p}} = I_q * h_q + I_p * h_p + \frac{P_f \cdot D_{\text{polder}}}{r'}, \quad (5-11)
\]

\[
C_{\text{tot.p}} = I_q * h_q + I_p * h_p + e^{\frac{h_q - A}{B}} \left[ D_0 + D_1 (h_q - h_p) + D_2 (t_{\text{flood}}) \right] \frac{r'}{r'}, \quad (5-12)
\]

*Note: The cost of water drainage and maintenance (OPEX) is not included. The determined total cost are not the actual total cost of terminal, only the part necessary for the optimization (costs which differ between a conventional and a polder terminal).*
Optimization decision variables
To determine whether or not a polder terminal is economically attractive first a level can is found for which the additional risk of a polder terminal is equal to the saving of reclamation cost, as explained in the next section. After determining this level one can find the economic optimal quay wall height for which the total cost function (equation 5-12) is minimal.

Transition point conventional <-> polder terminal
To determine the level for which it is economically attractive to construct a polder terminal rather than a conventional terminal a terminal level can be found for which the additional risk of a polder terminal is equal to the saving of reclamation cost (illustrated in Figure 47).

\[
\Delta I_{\text{polder}} = \Delta R_{\text{polder}}
\]

When analyzing the total cost of a polder terminal (equation 5-12) one can conclude that the polder level has a linear contribution to the costs in the total cost function. To determine the level where the ‘additional investment to construct a conventional terminal \( I_p \)’ is equal to the ‘increased risk when constructing a polder terminal’, one should set the derivative of the goal function to the decision variable polder level to zero, see equation 5-13. This equation is independent of the polder level.

\[
\frac{\delta C_{\text{tot},i}}{\delta h_p} = I_p - \frac{h_q - h_p}{r'} D_i = 0 \quad (5-13)
\]

\[
I_p = e^{\frac{h_q - h_p}{r'}} D_i \quad (5-14)
\]

\[
P_{f,\text{transition}} = \frac{I_p r'}{D_i} \quad (5-15)
\]

\[
h_{q,\text{transition}} = A - B \ln(P_{f,\text{transition}}) \quad (5-16)
\]

The result is shown in the equations 5-14 to 5-16, these show that for a quay wall height equal to \( h_{q,\text{transition}} \) the additional risk (depending on \( h_q h_p \)) is equal to the investment cost (depending on \( h_q h_p \)), see equation 5-14.
5. Risk framework

For $h_q = h_{q;\text{transition}}$ the additional risk of constructing a polder terminal is equal to the additional investment required to construct a conventional terminal, making the total cost independent of the polder level;

For $h_q > h_{q;\text{transition}}$ the additional risk of constructing a polder terminal is lower than the additional investment required to construct a conventional terminal, lower polder levels produce lower total costs;

For $h_q < h_{q;\text{transition}}$ the additional risk of constructing a polder terminal is higher than the additional investment required to construct a conventional terminal, lower polder terminals produce higher total costs. In conclusion, a polder terminal is attractive from quay wall heights higher than $h_{q;\text{transition}}$, see equation 5-16.

**Economic optimal quay wall height (minimal total cost)**

A quay wall height higher than the ‘transitional quay wall height’ will result in lower total cost. In the function of the total cost the polder level ($h_p$) has a linear contribution; the total cost related to the polder level will therefore be minimal for the lowest possible polder level. To compare, the total cost as a function of the quay wall height are plotted for two polder levels in Figure 49:

- For a certain quay wall height, $h_q = h_{q;\text{transition}}$, the total cost for both polder levels are equal;
- For quay wall heights lower than the transitional quay wall height the total cost of the higher polder level (blue line $h_p = +1m$) are lower than that of the lower polder level (red line $h_p = 0m$) making a conventional terminal the better alternative;
- For quay wall heights higher than the transitional quay wall height the total cost of the higher polder level (blue line $h_p = +1m$) are higher than that of the lower polder level (red line $h_p = 0m$) making a polder terminal the better alternative. The lowest costs are therefore found for the lowest polder level.
5. Risk framework

Figure 49: Total quay wall costs for different polder levels (conceptual graph)

So, to determine which quay wall height corresponds with minimal total costs first a polder level is to be chosen. The choice of a polder level is influenced by the failure mechanisms of seepage, settlement, instability and logistic requirements. More investigation on this topic is done in chapter 6.

The minimal total cost can be determined by setting the derivative of the goal function (equation 5-12) to the decision variable quay wall height to zero (Figure 43), see equation 5-17 and 5-18.

\[
\frac{\delta C_{\text{tot}, p}}{\delta h_q} + e^{\frac{h_q - A}{B}} \frac{h_q - A}{B} D_i = 0 \quad (5-17)
\]

\[
e^{\frac{h_q - A}{B}} \left[ D_0 + D_i (h_q - h_p) + D_i (t_{\text{flood}}) \right] - D_i B = I_q \quad (5-18)
\]

To solve equation 5-18 one should derive an expression for the quay wall height \( h_q \), as is done in equation 5-19. The solution of this function, which is of the same type as equation 5-20, is a Lambert function: an infinite row. Such a function can only be solved numerically, through iterations.
5. Risk framework

In conclusion, after determining a polder level (depending on seepage, settlement, stability and logistic requirements) the economic optimal quay wall height can be found by solving equation 5-19 numerically.

5.2.2. Risk framework approach conventional terminal

For a conventional terminal, where \( h_p \) is equal to \( h_q \), a similar approach is made. The goal function, containing the investments and risks involved, is almost the same function as the total cost function of the polder terminal. The main difference is that the decision variables of the polder terminal consist of the quay wall height \( h_q \) and polder level \( h_p \) whereas for the conventional terminal only the terminal level \( h_t \) is considered.

**Investments**

The investments of the conventional terminal are expressed in equation 5-21, notice that the same parameters for the reclamation cost \( I_p \) and quay wall cost \( I_q \) are used which implies that these are assumed equal for both the conventional and polder terminals. In paragraph 5.4 this assumption is treated further.

\[
I_{conventional} = (I_q + I_p) h_t \tag{5-21}
\]

**Risk**

The risk is expressed in equation 5-22, the main difference is that for a conventional terminal only the constant level of direct damage \( (D_0) \), irrespective of the inundation depth, together with the indirect damage are taken in to account. This is assumed because large damages (and corresponding risks) due to overtopping are avoided by determining a sufficiently high crest height.

\[
R_{conventional} = e^{-\frac{h_t - \Delta}{a}} \left[ D_0 + D_i (t_{flood}) \right] \frac{1}{r} \tag{5-22}
\]
Total cost optimization

The resulting goal function of the total cost for the conventional terminal is expressed in equation 5-23. To find the economic optimal terminal level the minimum of the total cost is determined by setting the derivative of the total cost to the terminal level \( h_t = h_q \) to zero, see equations 5-24 through 5-26.

\[
C_{\text{tot},t} = (I_q + I_p) * h_t + \frac{h_t - A}{B} * e^{\frac{h_t - A}{B}} * \left[ D_0 + D_1(t_{\text{flood}}) \right] \tag{5-23}
\]

\[
\delta C_{\text{tot},t} = (I_q + I_p) - \frac{1}{B} e^{\frac{h_t - A}{B}} * \left[ D_0 + D_1(t_{\text{flood}}) \right] = 0 \tag{5-24}
\]

\[
P_{f,\text{optimal}} = \frac{(I_q + I_p) * B * r'}{\left[ D_0 + D_1(t_{\text{flood}}) \right]} \tag{5-25}
\]

\[
h_{t,\text{optimal}} = A - B \ln(P_{f,\text{optimal}}) \tag{5-26}
\]

In conclusion, equation 5-26 determines the terminal level for which the total costs of the conventional terminal are minimal. The investments, risk and total costs for a conventional terminal are plotted in Figure 43.

5.3. Tuas implementation

In the following paragraph the optimization is made for the Tuas case in Singapore, to validate the derived equations both the analytical approach is calculated as well as a numerical approach. In order to calculate the required levels the terms of reference defined in section 3.2.1 will be used. The important parameters are explained in Table 24.

<table>
<thead>
<tr>
<th>Design parameter</th>
<th>Explanation</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Port bottom level</td>
<td>Required bottom level -23m CD = -24.5m MSL</td>
<td>( h_b )</td>
<td>-24.5</td>
<td>m</td>
</tr>
<tr>
<td>Terminal length</td>
<td>Requirement from terminal developer</td>
<td>( L_p )</td>
<td>3,900</td>
<td>m</td>
</tr>
<tr>
<td>Terminal width</td>
<td>Requirement from terminal developer</td>
<td>( W_p )</td>
<td>800</td>
<td>m</td>
</tr>
<tr>
<td>Terminal area</td>
<td>( A_p = L_p * W_p )</td>
<td>( A_p )</td>
<td>3.12 e06</td>
<td>m³</td>
</tr>
<tr>
<td>Quay wall cost [per m retaining height per running m]</td>
<td>For quay walls with retaining heights of about 30 meters costs are 50.000 €/m, which coincides with 1.700 €/m/m</td>
<td>( i_q )</td>
<td>1,700</td>
<td>€/m/m</td>
</tr>
<tr>
<td>Reclamation cost</td>
<td>Good quality fill material costs in Singapore</td>
<td>( i_p )</td>
<td>20</td>
<td>€/m³</td>
</tr>
<tr>
<td>Exponential parameter</td>
<td>Determined through extreme water level statistics in Figure 23</td>
<td>( A )</td>
<td>2.87</td>
<td>-</td>
</tr>
<tr>
<td>Exponential parameter</td>
<td></td>
<td>( B )</td>
<td>0.15</td>
<td>-</td>
</tr>
</tbody>
</table>
5. Risk framework

### Failure probability
The failure probability for overtopping is expressed in equation 5-4

\[
P_f = e^{-\frac{\ln(2.87)}{0.15} q h_f}
\]

### Direct damage container terminal
The total potential damage due to inundation originates from the study of Tebodin 1998.

\[
dc = 580 \text{ €/m}^2
\]

### Linear coefficient damage function
The linear coefficient originates from the assumption that 100% damage potential occurs at an inundation depth of 5m.

\[
di = 0.2 \text{ m}^{-1}
\]

### Direct damage ‘some water on terminal’
The direct damage during ‘some flooding’ is related to the damage for an inundation depth of 0.5 m.

\[
D_0 = 1.81 \times 10^8 \text{ €}
\]

### Indirect damage due to down time
The indirect damage due to down time is determined in the section below.

\[
D_t = 7.70 \times 10^6 \text{ €/wk}
\]

### Down time during a flood
This depends on different parameters, storm duration and water drainage. A first assumption is set at 1 week, see chapter 8 for more research.

\[
t = 1 \text{ wk}
\]

### Reduced interest rate
The reduced interest rate is assumed at 5%.

\[
r' = 0.05
\]

<table>
<thead>
<tr>
<th>Design values</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Failure probability</strong></td>
</tr>
<tr>
<td><strong>Direct damage container terminal</strong></td>
</tr>
<tr>
<td><strong>Linear coefficient damage function</strong></td>
</tr>
<tr>
<td><strong>Direct damage ‘some water on terminal’</strong></td>
</tr>
<tr>
<td><strong>Indirect damage due to down time</strong></td>
</tr>
<tr>
<td><strong>Down time during a flood</strong></td>
</tr>
<tr>
<td><strong>Reduced interest rate</strong></td>
</tr>
</tbody>
</table>

*Note: The quay wall height and polder levels determined in the following case are relative to the Mean Sea level in Singapore, not to be confused with Chart Datum which lies 1.5 meters below MSL.*

#### Estimate of possible indirect damage in Tuas
In section 3.2.3 the total port income is estimated at $990 mln per year, which coincides with 800 mln euro. Possible indirect damages due to down time of the port can now be determined. During a flooding event in the polder, dependant on the inundation time, it is assumed only part of the total income of the polder terminal is lost (about 75%). Of this part the port operator could relocate the activities to a different terminal in the area saving another part of the total income (25%). The final result is that the port operator does not lose all income but only a certain percentage during the flood (50%). For Tuas, Singapore, this amounts to a loss of 400 mln euro per year or 7.7 mln euros per week.

*Note: This case considers a transshipment port which does not contribute to the local economy, as opposed to an import/export terminal which does. Further, it is assumed part of the transshipment losses can be relocated at another terminal of the port operator resulting in less damage.*

### 5.3.1. Transitional quay wall height
To determine, given the set of boundary conditions of Table 24, the transitional quay wall height for which the savings of reclamation cost is equal to the additional risk one could follow the analytical approach or compute this level numerically.
Analytical approach
For the analytical approach one should solve equations 5-13 through 5-16. The results are presented in Table 25 and Figure 51, conclusions:

<table>
<thead>
<tr>
<th>Analytical results</th>
<th>Parameter</th>
<th>Equation</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quay wall height</td>
<td>$h_q$</td>
<td>Equation 5-16</td>
<td>3.58</td>
<td>m MSL</td>
</tr>
<tr>
<td>Failure probability</td>
<td>$P_f$</td>
<td>Equation 5-15</td>
<td>0.00862</td>
<td>-</td>
</tr>
<tr>
<td>Design return period</td>
<td>$t_{p}$</td>
<td>$1/P_f$</td>
<td>116</td>
<td>yr</td>
</tr>
<tr>
<td>Total cost</td>
<td>TC</td>
<td>Equation 5-12</td>
<td>2.16 e9</td>
<td>€</td>
</tr>
</tbody>
</table>

Table 25: Analytical determination of transitional quay wall height

Figure 51: Total cost comparison for transitional quay wall height

- The green line indicates the total costs for a quay wall height equal to the transitional quay wall height $= 3.58$ meter MSL, which was determined with equation 5-16. For this point the reclamation cost are equal to the risk, therefore the total cost are independent of the polder level.
- The blue line indicates the total costs for a quay wall height of 3.25 meter MSL ($< h_{q;transition}$), it is clear that with decreasing polder level the costs increase indicating that the increased risk (in euro’s) of a lower polder level is higher than the reclamation saving (in euro’s) for that polder level.
- The red line indicates the total costs for a quay wall height of 4.5 meter MSL ($> h_{q;transition}$), it is clear that with decreasing polder level the costs decrease indicating that the risk of a lower polder level is lower than the reclamation saving for that polder level.

Numerical approach
The same transitional quay wall height could also be determined numerically by combining a polder level with a quay wall height and computing the total cost, which is done in the figure below. All lines, each
5. Risk framework

corresponding to a different polder level, intersect each other at one point, $h_q = 3.6$ meter MSL, which means that:

- The transitional quay wall height found analytically matches the quay wall height found numerically, 3.6 meter MSL;
- The corresponding total cost found numerically matches the total cost found analytically, 2.2 e9 euros.

![Numerical approach to determine transitional quay wall height](image)

Figure 52: Numerical approach to determine transitional quay wall height

Further, it can be seen that the total cost for quay wall heights higher than the transitional quay wall height keeps decreasing up to a certain economical optimal point. The quay wall height for which these costs are minimal will be determined in the next paragraph.

5.3.2. Two strategies compared: polder versus conventional

Minimal cost polder terminal
To determine the minimal cost of a polder terminal first a polder level needs to be chosen based on requirements of seepage, settlement, instability and logistics. For practical purposes a polder level equal to Mean Sea Level is chosen for a preliminary design. Seepage will be minimal because the level difference between the inside water level and outside will be minimal. An investigation of the influence of the polder level on the stability is made in chapter 6. Considering logistics no large alterations are expected because a back reach crane is used and the level difference will not be very large resulting in a slight increase of the turnover time. An illustration of a possible cross section is shown in Figure 33.
Analytical approach
Given a certain polder level the economic optimal quay wall height for a polder terminal can be determined by setting the derivative of the total cost function to zero. As stated before this function is to be solved through iterations (numerically) as the optimal quay wall height cannot be solved analytically. The iterations are shown in Table 26; the optimal level is 4.3 meter MSL.

<table>
<thead>
<tr>
<th>Quay level height [m MSL]</th>
<th>Left side equation 5-18</th>
<th>Right side equation 5-18</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.2</td>
<td>31,105,865</td>
<td>14,620,000</td>
<td>-16,485,865</td>
</tr>
<tr>
<td>4.25</td>
<td>22,532,114</td>
<td>14,620,000</td>
<td>-7,912,114</td>
</tr>
<tr>
<td>4.3</td>
<td>16,319,647</td>
<td>14,620,000</td>
<td>-1,719,647</td>
</tr>
<tr>
<td>4.35</td>
<td>11,818,703</td>
<td>14,620,000</td>
<td>2,801,297</td>
</tr>
<tr>
<td>4.4</td>
<td>8,558,155</td>
<td>14,620,000</td>
<td>6,061,845</td>
</tr>
<tr>
<td>4.45</td>
<td>6,196,448</td>
<td>14,620,000</td>
<td>8,423,552</td>
</tr>
</tbody>
</table>

Table 26: Iterations optimal quay wall level

Note: The difference between both sides of equation 5-18 found with a quay level height of 4.3 meter is still rather large. To find a smaller difference a smaller step size (> 0.05m) should be used. This is not necessary because the accuracy of the design quay wall height does not rely on centimeters; these levels are usually rounded of on 10 centimeters.

Numerical approach
A numerical approach is also done to determine the economic optimal quay wall level for which costs are minimal. Given a polder level of 0 meter MSL, the total costs are calculated for different quay wall levels, the results are shown in Table 27. The optimal quay wall height determined numerically is 4.3 meter MSL, which coincides with the level found analytically (through iterations).

<table>
<thead>
<tr>
<th>Quay level height [m MSL]</th>
<th>Investment cost [€]</th>
<th>Damage cost [€]</th>
<th>Probability of failure [-]</th>
<th>Return period [yr]</th>
<th>Risk [€]</th>
<th>Total cost [€]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>1.947 e09</td>
<td>1.67 e09</td>
<td>0.00027465</td>
<td>3,650</td>
<td>9.20 e06</td>
<td>1.956 e9</td>
</tr>
<tr>
<td>4.2</td>
<td>1.948 e09</td>
<td>1.71 e09</td>
<td>0.00014101</td>
<td>7,100</td>
<td>7.82 e06</td>
<td>1.953 e9</td>
</tr>
<tr>
<td>4.3</td>
<td>1.949 e09</td>
<td>1.74 e09</td>
<td>0.00007240</td>
<td>13,800</td>
<td>2.53 e06</td>
<td>1.952 e9</td>
</tr>
<tr>
<td>4.4</td>
<td>1.951 e09</td>
<td>1.78 e09</td>
<td>0.00003717</td>
<td>26,900</td>
<td>1.32 e06</td>
<td>1.953 e9</td>
</tr>
<tr>
<td>4.5</td>
<td>1.952 e09</td>
<td>1.82 e09</td>
<td>0.00001908</td>
<td>52,400</td>
<td>0.69 e06</td>
<td>1.954 e9</td>
</tr>
</tbody>
</table>

Table 27: Numerical determination of minimal cost and corresponding quay wall height for \( h_p=0 \) m.

The resulting graph of the investment cost, risk and total cost related to the quay wall height for a polder level at Mean Sea Level is shown in Figure 53.
5. Risk framework

**Minimal cost conventional terminal**

To compare the polder terminal with the conventional terminal one should determine the terminal level for which the total cost of a conventional terminal is minimal. This is also done analytically and numerically.

**Analytical approach**

This level is derived through equations 5-23 through 5-26. The results for the chosen parameters are given in Table 28, the terminal level is 3.74 meter MSL with a minimum of the total cost of 2.17 e09 €.

<table>
<thead>
<tr>
<th>Analytical optimization</th>
<th>Parameter</th>
<th>Equation</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimal terminal height</td>
<td>( h_{t;\text{opt}} )</td>
<td>( h_{t;\text{opt}} = A - B \ln(P_{f;\text{opt}}) )</td>
<td>3.74</td>
<td>m MSL</td>
</tr>
<tr>
<td>Inundation duration</td>
<td>( t_i )</td>
<td></td>
<td>1</td>
<td>wks</td>
</tr>
<tr>
<td>Cost quay walls</td>
<td>( C_q )</td>
<td>( I_q \times h_t )</td>
<td>4.13 e08</td>
<td>€</td>
</tr>
<tr>
<td>Cost reclamation</td>
<td>( C_p )</td>
<td>( I_p \times h_t )</td>
<td>1.76 e09</td>
<td>€</td>
</tr>
<tr>
<td>Total investment cost</td>
<td>( C_t )</td>
<td>( (I_q + I_p) \times h_t )</td>
<td>2.17 e09</td>
<td>€</td>
</tr>
<tr>
<td>Indirect damage cost</td>
<td>( D_t )</td>
<td></td>
<td>7.69 e07</td>
<td>€</td>
</tr>
<tr>
<td>Direct damage cost</td>
<td>( D_0 )</td>
<td></td>
<td>1.81 e08</td>
<td>€</td>
</tr>
<tr>
<td>Total damage</td>
<td>( D )</td>
<td>( D_0 + D_t )</td>
<td>1.89 e08</td>
<td>€</td>
</tr>
<tr>
<td>Optimal failure probability</td>
<td>( P_{f;\text{opt}} )</td>
<td>( P_{f;\text{opt}} = \frac{(I_q + I_p) \times B \times r'}{D} )</td>
<td>0.00306</td>
<td>-</td>
</tr>
<tr>
<td>Return period</td>
<td>( T_p )</td>
<td>( 1 / P_{f;\text{opt}} )</td>
<td>350</td>
<td>([\text{yr}^{-1}])</td>
</tr>
<tr>
<td>Risk</td>
<td>( C_{w;\text{risk}} )</td>
<td>( (P_{f;\text{opt}} D) / r' )</td>
<td>12 e06</td>
<td>€</td>
</tr>
<tr>
<td>Minimum total cost</td>
<td>( T_{C;\text{opt}} )</td>
<td>Equation 5-23</td>
<td>2.17 e09</td>
<td>€</td>
</tr>
</tbody>
</table>

Table 28: Analytical optimization conventional terminal level
**Numerical approach**

A numerical approach is also done to determine the economic optimal quay wall level for which costs are minimal. The optimal terminal level determined numerically is 3.75 meter MSL, which coincides with the level found analytically (through iterations).

<table>
<thead>
<tr>
<th>Quay level height [m MSL]</th>
<th>Investment cost [€]</th>
<th>Damage cost [€]</th>
<th>Probability of failure [-]</th>
<th>Return period [yr]</th>
<th>Risk [€]</th>
<th>Total cost [€]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5</td>
<td>2.16 e09</td>
<td>1.89 e08</td>
<td>0.0149</td>
<td>65</td>
<td>57 e06</td>
<td>2.21 e9</td>
</tr>
<tr>
<td>3.75</td>
<td>2.18 e09</td>
<td>1.89 e08</td>
<td>0.0028</td>
<td>350</td>
<td>11 e06</td>
<td>2.19 e9</td>
</tr>
<tr>
<td>4.0</td>
<td>2.20 e09</td>
<td>1.89 e08</td>
<td>0.0005</td>
<td>1900</td>
<td>2 e06</td>
<td>2.20 e9</td>
</tr>
</tbody>
</table>

Table 29: Numerical determination of minimal cost and corresponding quay wall height for \( h_p = 0 \) m.

The resulting graph of the investment cost, risk and total cost related to the terminal height is shown in Figure 54.

**Conclusion**

Firstly it can be concluded that the equations found are valid given the assumptions made, because the results found analytically correspond with those found numerically. The results are summarized in Table 30. In both economic optimal points a polder terminal scores better, the total costs are lower and the expected risk is lower. For comparison purposes the total cost, damages and risk of a conventional terminal with terminal height equal to the optimal quay wall height for a polder terminal are also shown.
5. Risk framework

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Optimal PT</strong></td>
<td>4.3</td>
<td>1,950</td>
<td>0.00007</td>
<td>13,800</td>
<td>1,737</td>
<td>2.5</td>
<td>1,953</td>
<td>625</td>
</tr>
<tr>
<td><strong>Optimal CT</strong></td>
<td>3.8</td>
<td>2,175</td>
<td>0.00310</td>
<td>350</td>
<td>189</td>
<td>12</td>
<td>2,187</td>
<td>700</td>
</tr>
<tr>
<td><strong>Comparison CT</strong></td>
<td>4.3</td>
<td>2,220</td>
<td>0.00007</td>
<td>13,800</td>
<td>189</td>
<td>0.3</td>
<td>2,220</td>
<td>710</td>
</tr>
</tbody>
</table>

Table 30: Comparison Tuas case

From the results can be concluded that a saving of about 12% in investment is possible, compared to the conventional terminal, with a quay wall height of 4.3 meter MSL. The risk for the polder terminal is however about a factor 10 larger than the risk of the conventional terminal, a cross section of both terminals is shown below.

These results are based on assumptions regarding quay wall cost, reclamation cost and damage potential, see Table 24, considering the reclamation saving a margin for error of 30% seems acceptable resulting in a total saving of 9%. It should be noted that no definitive conclusions can be made based on one case. The sensitivity of the model to different parameters and assumptions is treated in the next paragraph; finally a sensitivity analysis is made in chapter 8.

At this stage it can be concluded that the polder terminal is an economically attractive alternative to a conventional terminal.

![Figure 55: Cross section polder and conventional terminal for Tuas](image-url)
5. Risk framework

5.3.3. Comparison for different reclamation cost

This paragraph contains a comparison of the proposed polder terminal design for Tuas for different reclamation cost.

Expensive reclamation

In order to compare both the conventional terminal and the polder terminal both graphs are shown in one figure, for the chosen parameters presented in Table 24. These graphs correspond with an expensive reclamation cost of 40 €/m³.

![Graph showing comparison of both strategies for Ip = 40 €/m³](image)

Figure 56: Comparison of both strategies for Ip = 40 €/m³

The total cost of a polder terminal in the economic optimal point are lower than those of a conventional terminal in the economic optimal point, proving that for high reclamation cost a polder terminal is the better strategy.

‘Cheap’ reclamation

To compare the results for extremely low reclamation cost, where it is expected that the increase of risk will be higher than the savings in reclamation fill thus making a conventional terminal the better strategy the figure below shows a graph which corresponds with reclamation cost of 1 €/m³.
5. Risk framework

The total cost of a polder terminal in the economic optimal point are higher than those of a conventional terminal in the economic optimal point, proving that for low reclamation cost a conventional terminal is the better strategy.

**Conclusion**

The costs in the economic optimal point are summarized in Table 31, it is clear that for expensive soil reclamation cost the total cost of the polder terminal are lower than the total cost of a conventional terminal, while for cheaper reclamation cost a conventional terminal is the better option.

<table>
<thead>
<tr>
<th>Reclamation cost</th>
<th>Polder terminal cost</th>
<th>Conventional terminal cost</th>
<th>Polder terminal risk</th>
<th>Conventional terminal risk</th>
<th>Polder applicable?</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 €/m³</td>
<td>3.5 e9 €</td>
<td>4.0 e9 €</td>
<td>2.5 e6 €</td>
<td>0.3 e6 €</td>
<td>Yes</td>
</tr>
<tr>
<td>1 €/m³</td>
<td>445 e6 €</td>
<td>440 e6 €</td>
<td>3.5 e6 €</td>
<td>2 e6 €</td>
<td>No</td>
</tr>
</tbody>
</table>

Table 31: Comparison for different reclamation cost

The large influence of the reclamation cost on the total cost function can be explained by its quadratic influence. As the reclamation cost is dependant on the amount of cubic meters required for the fill, the value of $I_p$ in €/m³ needs to be multiplied by the terminal area after which it can be used in the goal function.
5.4. Discussion

For sake of simplicity certain assumptions were made regarding different aspects of the total cost function, this paragraph discusses the different assumptions made. It should be noted that the proposed new methods require further investigation.

5.4.1. Investments

Quay wall cost
The variable quay wall cost, $I_q$, are assumed to have a linear relation to the retaining height. In paragraph 2.3 the relation between quay wall cost and retaining height is given, which is not linear. Higher quay walls have higher cost than lower quay walls. The influence of non-linearity is expressed in equation 5-27 ($h_q$ and $h_t$ to the power $c$).

Another assumption made which required further investigation is that the quay wall height related costs are assumed equal for both the conventional as well as the polder terminal. This aspect is investigated further in chapter 6.

Reclamation cost
As stated earlier the reclamation cost (including soil improvement cost) for both the conventional and polder terminals are assumed equal, which can be argued because the soil improvement of the larger fill of a conventional terminal is different from the soil improvement required for the smaller fill of a polder terminal. This all depends on the amount of settlement expected in and under the reclamation fill.

Settlements, as part of the geotechnical instability, form a risk to hydraulic structures because of the loss of height. The expected amount of settlement depends on the quantity of the reclamation fill, which depends on the polder (or terminal) level; a conventional terminal will have larger settlements than a polder terminal. An estimate of the amount of settlement for Tuas is made in the next chapter. Due to the larger fill of the conventional terminal the consolidation time will be shorter than the polder terminal.

Correspondingly the soil improvement cost as well as maintenance cost (refilling) of a polder terminal is expected to be lower than a conventional terminal. When only looking at the total cost, after including soil reclamation cost, the final result will benefit the polder terminal rather than the conventional terminal.

A different conclusion could be made if construction time is very important, because the consolidation time of the conventional terminal is shorter than that of the polder terminal due to the larger fill and corresponding vertical load.

A simple way of including soil improvement cost is proposed in equations 5-27 and 5-28 containing a linear relation of the soil improvement cost (different cost for polder $I_s$;polder and conventional $I_s$;conventional) to the reclamation level. This requires further investigation.
5. Risk framework

**Water drainage cost**
The operational cost of the terminals is neglected in this phase. The operational cost of a conventional terminal and a polder terminal will be similar, apart from the pumping cost of the polder terminal: $I_{pump}$ in equation 5-27. These pumping costs will increase the total cost over the lifetime of the terminal, the contribution to the total cost is treated in chapter 7.

**‘New’ investment function**
Considering the proposed recommendations the following equations for the investment of a polder terminal and a conventional terminal will be the result (which require further investigation).

\[
I_{n,polder} = I_q \cdot h_q^c + I_p \cdot h_p + I_{n,polder} \cdot h_p + I_{pump} \\
I_{n,conventional} = I_q \cdot h_q^c + I_p \cdot h_p + I_{n,conventional} \cdot h_p
\]

(5-27) (5-28)

**5.4.2. Risk**

**Probability**
Only overtopping related inundation is treated in this approach, but other failure mechanisms could also produce inundation of the terminal. Different methods to include these failure mechanisms were described in section 5.2.1 which will result in an increase of the probability of inundation with 10%, which is common for dikes as a flood defense: $P_f = 1.1 \cdot P_{f,overtopping}$.

**Damages**
The damage is limited to direct damage to terminal facilities and containers and indirect damage due to down time of the port. When the additional failure mechanisms are added (due to seepage, instability and calamities) the corresponding damages will also be added: the replacement cost of breached quay walls. An example of the resulting damage functions are given in equations 5-28 and 5-29.

The inundation related damage function is assumed linear, whereas the study of Tebodin found a non linear relation which is shown in Figure 45. A non linear damage function will change the total cost function and the resulting derivates, see equation 5-28 where the non linearity is included by the power $d$.

\[
D_{n,polder} = D_0 + D_1 (h_q - h_y)^d + D_4 (t_{flood}) + D_{breach}(h_y) \\
D_{n,conventional} = D_0 + D_1 (t_{flood}) + D_{breach}(h_y)
\]

(5-28) (5-29)

Not only will the total damages increase, but also the method to determine the economic optimal quay wall levels. The damage due to a breach shows a dependency on the quay wall height (quay wall costs are related to quay wall height) resulting in a different goal function for the polder terminal and thus different derivatives. Further, during a possible quay wall breach the indirect damages due to down time will be higher due the long period of time required to replace the breached quay wall in which no port activities will take place.
Another aspect requiring further investigation is the assumption that during a situation of overtopping the whole polder will be flooded immediately, not taking time into account, as illustrated in Figure 46. When doing so a different relation will be found for the damage function of equation 5-28.

‘New’ risk function

The resulting damage functions for the polder and conventional terminal are shown in the equations below.

\[
R_{\text{polder}} = \frac{1.1 \star P_f \star D_{\text{n,polder}}}{r'}
\]

\[
R_{\text{conventional}} = \frac{1.1 \star P_f \star D_{\text{n,conventional}}}{r'}
\]

5.4.3. ‘New’ goal functions

The total cost function of the conventional and the polder terminal will change due to the proposed recommendations, each requiring further investigation. Examples of the resulting functions are presented in equations 5-32 and 5-33. The optimization will be different due to the non linear contributions and additions. A sensitivity analysis is made in chapter 8, which investigates the influence of different recommendations made in this paragraph.

\[
C_{\text{n,tor,p}} = I_q \star h_q^c + I_p \star h_p + I_{s,polder} \star h_p + I_{\text{pump}} + \frac{1.1 \star P_f \star D_{\text{n,polder}}}{r'}
\]

\[
C_{\text{n,tor,c}} = I_q \star h_q^c + I_p \star h_t + I_{s,\text{conventional}} \star h_t + \frac{1.1 \star P_f \star D_{\text{n,conventional}}}{r'}
\]
5. Risk framework

5.5. Conclusions and recommendations

This chapter contains a flood risk approach to determine the levels of a polder terminal and a conventional terminal for which the total costs are minimal. The goal was to determine whether or not the polder terminal is economically an attractive alternative of a conventional terminal.

It is determined how the cost (investment) increase with higher quay wall and polder elevation, the quay wall height related probability of inundation due to overtopping will decrease and thereby the risk. The summation of the investment cost and risk is called the goal function which is expressed in equation 5-1.

The decision variables of the polder terminal consist of the quay wall height and the polder level, whereas for the conventional terminal (where quay wall height = polder level) only the terminal level is a decision variable. Different assumptions where made, for sake of simplicity, to make this approach possible. The assumptions are discussed in paragraph 5.4. Finally the goal function was optimized, resulting in the following conclusions:

**Polder terminal**
- The transitional quay wall height of the polder terminal can be found analytically through equation 5-16, this represents a height for which the additional risk of constructing a polder terminal, independent of the chosen polder level, is equal to the additional investment required to build a conventional terminal (fill up the polder).
- For quay wall heights higher than the transitional height it is economically more attractive to build a polder terminal rather than a conventional terminal (saving of reclamation cost is larger then the increase of risk).
- To minimize the total cost the lowest possible polder level should be chosen, this level is however bounded by requirements of seepage, settlement, instability and logistics. Further investigation on this subject is made in chapter 6.
- After determining which polder level can be applied the economic optimal quay wall height, for which total costs are minimal, can be found numerically (solve equation 5-19).

**Conventional terminal**
- The economic optimal terminal level for the conventional terminal can be found by solving equation 5-26, this represents the level for which the total cost of the conventional terminal are minimal.

Both optimizations are applied for the Tuas case, where the subsoil is dominated by weak clayey layers and reclamation cost is expensive. The results are shown in Table 30. It is concluded that the polder terminal produced a saving of about 10% in investment, compared to the conventional terminal. The risk for the polder terminal is however a factor 10 larger then the risk of the conventional terminal. For this case the polder terminal proved to be an attractive alternative to save costs in comparison with the conventional terminal.
Finally a comparison is made for different reclamation cost ($I_p$), one expensive and one cheaper reclamation cost. It was clear that for expensive reclamation cost the polder terminal is the better strategy whereas for cheaper reclamation cost the conventional terminal is the better strategy.

*Note: These results only hold for the Tuas case, a sensitivity analysis is made in chapter 8 to further investigate the feasibility of the polder terminal for different parameters.*

### 5.5.1. Recommendations

The investigation is made on a micro economical scale where profit for the port operator is the most important factor, which is comparable to a transshipment port. When considering an import/export port it should be noted that on a larger scale (macro economical) the ‘extra’ investment required to build a conventional terminal rather than a polder terminal, could be acceptable because of the economical importance of the port for the country.

Risk mitigation measures are not taken into account, while these could increase the total cost of the port operator. The main risk mitigation measure is to insure the port against flooding which could be costly. When added the total cost of the polder terminal will increase possibly changing the final conclusion that for certain boundary conditions a polder terminal is economically more attractive than a conventional terminal.

When designing a new container terminal the chosen terminal levels should not only based on minimal total cost but also take the return period of inundation and the risks involved in to account. The result of a design based only on solely the total cost could be a terminal which is relatively cheap but has an unacceptable probability of inundation or risk. The decision whether or not a design should be based on minimal total cost, a certain level of safety or an acceptable risk is one of a political nature.

The levels determined through the optimization do not take sea level rise and settlement criteria in to account. Thus, when the required quay wall height is determined through the aforementioned analysis these contributions should be added which will determine the final design quay wall height, see chapter 6.
6. Flood defense system

6.1. Introduction

This chapter deals with a more detailed design of the combined quay wall flood defense system of the polder terminal. In chapter 3 it was concluded that all sides of the polder terminal will be constructed as a quay wall (no dikes) and that a sheet pile or gravity structure is best suitable for the Tuas polder terminal.

This chapter will determine the aspects and cost of each structure type suitable for a quay wall flood defense of a conceptual polder terminal. The boundaries for the polder level will be investigated in more detail looking at the requirements of the failure mechanisms seepage and instability, and port logistics.

6.2. Design quay wall height

The flood defense system of the polder terminal has two functions. Firstly the flood defense system needs to provide sufficient safety against flooding while also facilitating vessel berthing. The required crest height is determined in an economic approach in chapter 5; contributions of wind set up, waves and sea level rise, as illustrated in Figure 8, were not included. These will be determined in this paragraph as well as the contribution of possible settlement and subsidence during the design lifetime of the flood defense. Together these contributions form the construction height of the flood defense, see equation 6-1.

\[
h_{\text{construction}} = h_q + h_{\text{wind}} + h_{\text{waves}} + h_{\text{sealevelrise}} + h_{\text{settlement}}
\]  

Wind set up \([h_{\text{wind}}]\)

The shear force of local wind gusts and oscillations acting on a water surface will cause a local rise of the water level, which is called wind set-up. For the determination of the design crest level of a flood defense the most unfavorable situation should be taken in to account, which is onshore directed wind.

The amount of wind set up expected depends on the local depth in the port and fetch of the wind (length over which the wind acts on the water). Ports generally lie sheltered from open sea resulting in low fetches for the wind; in combination with relatively deep waters the expected local wind set-up can be neglected.

Waves \([h_{\text{waves}}]\)

Wave attack is minimal because ports lie in sheltered areas where local wave conditions are dominated by passing ships. In the economical approach made in chapter 5 it is assumed that the polder terminal is not
subject to wave attack or wave run up. At this stage no contribution is taken in to account for wave run up, however if the risk framework approach is applied in an area which is subject to wave attack a contribution should be taken in to account. The wave run up at vertical walls such as quay walls is twice the significant wave height which is the result of a standing wave pattern in front of the wall; this is also the contribution to be taken in to account for the design crest height of the flood defense.

**Sea level rise** \([h_{\text{sealevelrise}}]\)

It should be determined what sea level rise is expected during the design lifetime of the structure. For the Tuas case a sea level rise of 25 centimeters during a design lifetime of 50 years is determined in chapter 3.

**Settlement** \([h_{\text{settlement}}]\)

As a result of a large fill and/or construction of a flood defense settlement will occur, which is a vertical displacement of the soil. A certain extra margin on top of the design crest height (construction height) should be added to compensate for the expected settlement during construction and the design lifetime. This amount is largely dependant on the local subsoil and the soil material of the fill.

An estimate of local settlement for the Tuas case is made using the formula of Terzaghi (equation 6-2). The soil properties were determined in chapter 3; Figure 33 and Table 20.

\[
\Delta s = \frac{1}{C_{10}} \cdot H \cdot \log \left( \frac{\sigma_a}{\sigma_b} \right)
\]  

(6-2)

First the expected settlement of the quay wall flood defense area is calculated. The design crest height is determined with equation 6-1 resulting in a crest height of +4.55 meter MSL \((h_{c}=h_{q}+h_{\text{sealevelrise}})\).

<table>
<thead>
<tr>
<th>Settlement flood defense</th>
<th>Parameter</th>
<th>Function</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement constant sandy silt</td>
<td>(C_{10})</td>
<td>See Table 20</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Height fill</td>
<td>(H_{\text{fill}})</td>
<td>(h_{b} + h_{\text{construction}})</td>
<td>29.6</td>
<td>m</td>
</tr>
<tr>
<td>Soil pressure before fill</td>
<td>(\sigma_{b})</td>
<td>(h_{w} (\text{MSL}) \cdot \gamma_{w})</td>
<td>250</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Soil pressure after fill</td>
<td>(\sigma_{a})</td>
<td>(H_{\text{fill}} \cdot \gamma_{\text{fill}} + q_{\text{surcharge}})</td>
<td>720</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Settlement</td>
<td>(\Delta s 1)</td>
<td>Equation 6-2</td>
<td>0.45</td>
<td>m</td>
</tr>
</tbody>
</table>

**Table 32: Settlement estimate quay wall area**

*Note: The settlement of the quay wall flood defense area is equal to the total settlement of the conventional terminal, because the fill level is the same.*

Now the expected settlement of the terminal yard is calculated. A terminal yard at Mean Sea level is assumed, as was done in chapter 5. Lower terminal levels will result in lower settlement.

<table>
<thead>
<tr>
<th>Settlement flood defense</th>
<th>Parameter</th>
<th>Function</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement constant sandy silt</td>
<td>(C_{10})</td>
<td>See Table 20</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>
6. Flood defense system

<table>
<thead>
<tr>
<th></th>
<th>$H_{\text{fill}}$</th>
<th>$h_b$</th>
<th>25</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Height fill</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Soil pressure before fill</strong></td>
<td>$\sigma_v$</td>
<td>$h_w (\text{MSL}) \times \gamma_w$</td>
<td>250</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td><strong>Soil pressure after fill</strong></td>
<td>$\sigma_n$</td>
<td>$H_{\text{fill}} \times \gamma_{\text{fill}} + q_{\text{surcharge}}$</td>
<td>630</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td><strong>Settlement</strong></td>
<td>$\Delta s_2$</td>
<td>Equation 6-2</td>
<td>0.33</td>
<td>m</td>
</tr>
<tr>
<td><strong>Difference with quay wall</strong></td>
<td>$\Delta s$</td>
<td>$\Delta s_1 - \Delta s_2$</td>
<td>0.12</td>
<td>m</td>
</tr>
</tbody>
</table>

Table 33: Settlement estimate terminal area

In conclusion one can see that a polder terminal (terminal yard at MSL) has a differential settlement, between the quay wall flood defense and the terminal yard, which should be taken in to account in the design stage. An additional 0.45 meter should be added to the design height to obtain the construction height.

**Note:** The settlement calculated is an estimate assuming weak homogeneous subsoil; the actual settlement is different because the subsoil is inhomogeneous. Further, the potential settling of the fill material is not taken in to account, assuming this will be limited due to soil improvement works (vertical drains).

**Design crest level quay wall**

The crest level of the quay wall flood defense for Tuas can now be determined by adding the possible sea level rise (ignoring wave run up) to the level determined in the economic approach in chapter 5; +4.3 meter + 0.25 meter = +4.55 meter MSL. To obtain the construction height the expected settlement of the subsoil should be added, resulting in a construction height of +4.9 meter MSL which is +6.4 meter CD.

6.3. Flood defense structure

It was concluded that a gravity or sheet pile wall is most suitable for Tuas, Singapore; a pile supported platform was not suitable. However, in this section all structures will be treated to investigate the behavior of these structures with regard to different failure mechanisms of ‘large scale flooding’: seepage, instability and calamities. Regarding calamities, such as ship collision and earthquakes, the following conclusions are made:

Mooring forces are taken in to account in the design of quay walls, ship collision is not because it is not economically sound to design a structure for full frontal collisions; guiding works and breaking structures are usually constructed to prevent ship collision and/or reduce the damage during a collision.

A similar conclusion can be made regarding earthquake behavior. Up to a certain extent earthquake behavior is taken in to account when choosing a suitable quay wall structure type. However it is not clear up to what safety level it is economically sound to invest in a structure subject to earthquake loading, this subject requires further investigation.
6.3.1. Gravity structures

Examples of gravity structures are a block wall, an angle type wall, caissons and silos. Concrete block walls are relatively heavy, resulting in large settlements of the subsoil; this structure will not be treated further.

Angle type walls require two separate structures which are subject to differential settlements; this will cause problems for the crane foundations and water tightness of the wall. Caissons and silos are very practical as a quay wall flood defense for the polder terminal as they can retain soil on both sides requiring only one structure to overcome both height differences on seaside and polder side, see Figure 58.

![Diagram of gravity structures: angle type walls, caisson, and silo](image)

**Figure 58: Caisson and silo polder quay wall**

These structures are prefabricated in a dry dock in the area and transported (often floating) to the construction site where they are placed on a prepared bed. After placement the structure is filled with soil. The small number of construction operations results in a fast construction time, however the distance to a suitable dry dock or other type of construction facilities could have a large impact on the construction cost.

**Seepage**

Piping due to continuous erosion of sediment from seepage water can be avoided by increasing the seepage length under the structure. The seepage length for gravity structures can be increased by using vertical seepage screens under the structure such as sheet piles or horizontal seepage screens in front of the seaside wall.
6. Flood defense system

**Instability**
Gravity structures obtain stability through their own weight and friction between the structure and the foundation. The lowered soil pressure on polder side, due to the lowered terminal yard, will lower the horizontal soil pressure on the structure and therefore benefit the stability of the structure compared to a conventional terminal. Concrete is a durable material, attention should be paid to the amount of concrete cover. This material is favored for hydraulic structures (in comparison with steel), especially in tropical (hot) waters.

Differential settlements between adjacent caissons or silos should be avoided as these could cause leakage of the flood defense. Attention should be paid to the water tightness of the interlockages between caissons/silos. Soil improvement works are required to replace any weak subsoil with stronger soil.

**Calamities**
Caissons and silos actually contain two soil retaining walls, which provide additional safety for ship collisions. Ships colliding with the front wall of the caisson will damage this part whereas the back wall will only partly damage or not at all, providing a certain ‘residual’ strength.

Gravity structures do not behave well under earthquake loading. The biggest risk is liquefaction of the soil under the structure, which can be avoided by using a filter layer under the structure to drain water over pressure. This counteracts the polder principle because such layers will increase the amount of seepage water entering the polder and the risk of piping. Thus these structures are not advised in earthquake areas.

**Costs**
In comparison to other quay wall structure types for the polder terminal gravity structures are more expensive due to the prefabrication of caissons/silos, the necessity of a dry dock to construct the caissons and transportation to the construction site.

The investigation made by J.G. de Gijt on quay wall cost [VI] concluded that the variation in cost of gravity structures is the largest compared to sheet pile structures; the gravity structures for the polder terminal are assumed to be 20 – 25% more expensive than other structure types resulting in the following graph of the total cost related to the retaining height, see Figure 59.

The difference in cost between a gravity structure for a polder terminal and a gravity structure for a conventional terminal is determined in the next paragraph, depending on the polder level.
6. Flood defense system

Sheet pile structures which could be applied as a combined quay wall flood defense are sheet piles, a combi wall and a diaphragm wall. Diaphragm walls are built by digging a trench in the soil, stabilizing it with a suspension (temporarily) after which it is filled with reinforced concrete. The soil in which these walls are built requires a certain stiffness and strength to be able to dig a trench and construct the diaphragm wall, which is not the case in reclaimed soil where polder terminals are built. A diaphragm wall is therefore not investigated in more detail.

The remaining structures, a sheet pile or combi wall, are investigated further. Firstly it should be determined whether or not both retaining walls (sea and polder side) influence each other, if so the whole structure will behave as a cofferdam.

A rule of thumb to determine whether or not two retaining walls act as one, forming a cofferdam, was determined by Terzaghi and resulted in equation 6-3 [XXIV]; after filling in the parameters it is clear that the quay wall of the conceptual polder terminal for Tuas, with a retaining height of 29 meter and width of 40 meter, will behave as a cofferdam ($B=1.4^*H$).

$$B_{\text{quay}} < 1.5^* H_{\text{retaining}}$$  (6-3)

The walls of the cofferdam consist of steel sheet piles or combi walls, connected with anchors. The anchors provide the interaction between both walls; these are placed above the groundwater level. The horizontal soil pressure on a vertical wall is illustrated in Figure 60, which shows the minimal (active) soil pressure and the maximum (passive) soil pressure on the seaside wall of the cofferdam, assuming straight slip planes. One
can conclude that the maximum passive soil pressure on the sea side wall of a polder terminal (yellow area) is lower than that of a conventional terminal (red lines), due to the lowered terminal yard.

Figure 60: Behavior sheet pile wall

Note: A cofferdam actually is a form of gravity structure, as it obtains part of its stability from the soil between both retaining walls.

Seepage
The seepage length for cofferdam sheet piles can be increased by lengthening the sheet pile walls, if insufficient seepage length is present. An advantage of cofferdams in relation to gravity structures is that the walls penetrate the subsoil which already increases the seepage length compared to gravity structures.

Instability
Sheet piles obtain stability through penetration in the subsoil and the anchors. Compared to a conventional terminal, where the horizontal soil pressure is higher due to the higher terminal yard, the front sheet pile wall can be designed lighter. The use of steel in hot climates and salt water is not advised, but there are methods to sufficiently reduce the amount of corrosion expected i.e. by increasing the wall thickness and providing sufficient maintenance.

Compared to gravity structures the vertical load on the sub soil is almost the same, apart from the difference in weight between concrete and steel. The resulting settlement will however be very different; where gravity structures are subject to differential settlements at the interlockages of adjacent caissons/silos, sheet pile walls are able to redistribute the loads between the walls because these are all connected. This will result in lower differential settlement of the structure. That is partly why these structures are more often applied in areas with bad sub soils

Calamities
Cofferdams, similar to caissons and silos, also contain two soil retaining walls, but do not behave similarly during a ship collision. The stability of the front wall depends on the anchor wall and vice versa, if one is damaged due to a collision the overall stability of the quay wall could be endangered. Considering earthquake loading the behavior depends on the alignment of the sheet piles. A stiff structure, with sheet piles driven under an angle to provide horizontal stability, will behave worse than a structure with only vertical piles.
6. Flood defense system

Costs
As explained earlier, sheet pile walls are less costly than gravity structures. The retaining height related cost will lie around the general line of the relation found in Figure 59; in research on quay wall cost it was concluded that the spread of the costs of sheet piles is less than that of gravity structures.

6.3.3. Pile supported platforms

Traditionally one of the most common ways to construct quay walls is to make a pile supported platform in combination with either an embankment or a soil retaining wall, as illustrated in Figure 61. These structures actually consist of two structures; a piled deck to facilitate berthing of vessels and provide a foundation for the container cranes and an embankment or soil retaining wall to overcome the height difference between the terminal yard and the bottom level of the port.

Seepage
The seepage length of pile supported platforms depends on the type of retaining structure applied. Methods to increase the seepage length (if necessary) are similar to those of gravity structures; by using seepage screens. For an embankment this could mean placing horizontal or vertical seepage screens in front or below the embankment, for a retaining wall this can be done by increasing the length of the wall.

Instability
Pile supported platforms obtain stability through piles which are driven in the subsoil deep enough to obtain sufficient bearing capacity. The soil retaining wall obtains stability through the piles which are driven in the subsoil under an angle or possibly by using an anchor wall, whereas an embankment obtains stability though its own weight and a revetment on the top layer preventing erosion. Uplifting of the deck should be checked during heavy wave attack in the port. The platforms are usually constructed of concrete whereas the piles could also be made of steel. Concrete (precast) piles are preferred because of the higher durability.
A disadvantage of these structures is that they will undergo differential settlements; the pile supported platform is generally founded on deep soil layers which have large bearing capacities whereas an embankment or soil retaining wall is not. The embankment will generally undergo settlement while the pile supported platforms will not. Another disadvantage is the large space required for an embankment, which has a large influence on the cost.

**Calamities**

During a ship collision the polder terminal behind the pile supported platforms is better protected than with gravity or sheet pile structures. Ships will damage the pile supported platform considerably, but the risk of damage to the soil retaining structure is smaller (because the pile supported platform will act as a protective structure). Pile supported platforms with only vertical piles, such as those with an embankment, will behave better during earthquake loading than platforms with piles under an angle, as explained before.

**Costs**

Between both structures proposed the most significant cost related issue is the slope of the embankment which determines the length and width of the piled deck, a large width of the deck will be costly. Compared to gravity and sheet pile (cofferdam) structures the cost is expected to be in the order of the cost of gravity structures, because of the combination of a pile supported platform and retaining structure.

### 6.4. Polder level

As stated earlier the design polder level is bounded by requirements of seepage, stability and logistics, this will be further investigated in this paragraph. It is determined how deep the polder level can be made and what the influence is of the polder depth on the quay wall flood defense structure, compared to a conventional terminal.

For practical purposes a gravity type structure is used as a quay wall flood defense for the investigation of the stability. Sheet pile structures, when applied as a quay wall flood defense, will form a cofferdam which actually is a type of gravity structure.

#### 6.4.1. Seepage requirements

**Piping**

Safety against piping is assured by determining the required seepage length of the structure (with Lane, see section 2.2.3) and using either horizontal or vertical seepage screens, as described in the last paragraph. To determine the boundaries for different structures an example calculation is made for the Tuas case with a level 1 probabilistic method, using partial safety factors.
For vertical walls the method of Lane is often used, which is explained in chapter 2, equations 2-9 till 2-11. Using this formula the required quay wall width (given the properties of the subsoil and outside water levels) can be determined for all polder levels. The derivation of the relation between the required width of the quay wall and the polder level is shown in the equations below, following from the method of Lane. The parameters are shown in Figure 66 and explained in the list of symbols.

For polder terminals a creep coefficient of 8.5 is used, which is valid for silt material, and a partial safety factor $\gamma_s$ of 1.3. The creep coefficient of 8.5 is used because it is assumed these terminals are built in areas with low quality subsoil (silty material). The graph in Figure 62 shows the required quay wall width for each polder level relative to Mean Sea Level, for every meter reduction of the polder level an additional 29 meter of width (or seepage length) is required.

$$\Delta h_t \leq \frac{L}{C_{\text{creep}}}$$  \hspace{1cm} (6-4)

$$L_{\text{Lane}} = L_{\text{vertical}} + L_{\text{horizontal}} / 3$$  \hspace{1cm} (6-5)

$$\Delta h_t = h_w - h_p$$  \hspace{1cm} (6-6)

$$L = h_p + \frac{W}{3}$$  \hspace{1cm} (6-7)

$$h_{p,\text{uls}} = \frac{C_{\text{creep}} \cdot h_w - \frac{W}{3}}{1 + C_{\text{creep}}}$$  \hspace{1cm} (6-8)

$$h_{p,\text{uls}} = \gamma_s \cdot C_{\text{creep}} \cdot h_w - \frac{W}{3}$$  \hspace{1cm} (6-9)

From the figure can also be concluded that for polder levels at or above Mean Sea Level theoretically no width is required to provide safety against piping. This is explained by the fact that piping will only occur if there is a level difference between outside and inside water, which is not the case if the polder level is equal to Mean Sea Level.

For sheet pile structures, which have longer seepage lengths due to the penetration of the walls in the subsoil, the required width will be lower than for gravity (caisson type) structures.
6. Flood defense system

![Figure 62: Piping width quay wall](image)

**Tuas polder terminal**

For the Tuas conditions, where a quay wall width of 40 meters is required to facilitate the container cranes and additional space is scarce, a maximum polder level of -1.3 meter MSL is possible. If deeper polders are made seepage screens are required to provide safety against piping, which was explained in the last paragraph.

_Note: The results found for Tuas can be considered applicable in a conceptual case of a polder terminal, because the local boundary conditions of the Tuas case (a depth of 25 meter and low quality subsoil) are considered generally applicable for new port constructions_

**Uplifting**

Uplifting occurs when the water pressure below an impermeable layer in the subsoil exceeds the weight of the soil fill. Safety against uplifting is achieved by determining the required polder level which assures vertical equilibrium of the soil body in the polder. An example calculation is made for the conceptual design of the polder terminal in Tuas.

The soil pressure right under the fill \(\sigma_{\text{fill}}\) is expressed in equation 6-11 and the maximum water pressure under the fill \(p\) is expressed in equation 6-12. The effective soil pressure \(\sigma'_{\text{eff}}\) is the difference between the soil pressure and the water pressure; this should be higher than zero to obtain vertical equilibrium. This leads to a requirement for the thickness of the sand layer \(d_{\text{sand}}\) on top of the impermeable layer in equation 6-15. The calculation for Tuas is made in Table 34, for practical purposes again a level I probability approach (deterministic) with partial safety factors is made.
6. Flood defense system

Figure 63: Calculation uplifting

\[
D = d_{\text{clay}} + d_{\text{sand}} \quad (6-10)
\]

\[
\sigma_{\text{fill}} = d_{\text{clay}} \gamma_{\text{clay}} + d_{\text{sand}} \gamma_{\text{sand}} \quad (6-11)
\]

\[
p_{\text{max}} = h_{\text{HAT}} \gamma_{\text{w}} \quad (6-12)
\]

\[
\sigma'_{\text{eff}} = \sigma_{\text{fill}} - p_{\text{max}} \quad (6-13)
\]

\[
\sigma'_{\text{eff}} = (d_{\text{clay}} \gamma_{\text{clay}} + d_{\text{sand}} \gamma_{\text{sand}}) - h_{\text{HAT}} \gamma_{\text{w}} = 0 \quad (6-14)
\]

\[
d_{\text{sand,sls}} = \left(\frac{h_{\text{HAT}} \gamma_{\text{w}} - d_{\text{clay}} \gamma_{\text{clay}}}{\gamma_{\text{sand}}} \right) \quad (6-15)
\]

\[
d_{\text{sand,uls}} = \gamma_{\text{s}} d_{\text{sand,uls}} \quad (6-16)
\]

<table>
<thead>
<tr>
<th>Uplifting calculation</th>
<th>Parameter</th>
<th>Function</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial safety factor</td>
<td>(\gamma_{s})</td>
<td></td>
<td>1.3</td>
<td>-</td>
</tr>
<tr>
<td>Specific weight water</td>
<td>(\gamma_{w})</td>
<td></td>
<td>10.25</td>
<td>kN/m³</td>
</tr>
<tr>
<td>Specific weight clay</td>
<td>(\gamma_{\text{clay}})</td>
<td></td>
<td>15</td>
<td>kN/m³</td>
</tr>
<tr>
<td>Specific weight sand</td>
<td>(\gamma_{\text{sand}})</td>
<td></td>
<td>20</td>
<td>kN/m³</td>
</tr>
<tr>
<td>Maximum water level</td>
<td>(h_{\text{HAT}})</td>
<td></td>
<td>26.5</td>
<td>m</td>
</tr>
<tr>
<td>Thickness clay layer</td>
<td>(d_{\text{clay}})</td>
<td></td>
<td>2</td>
<td>m</td>
</tr>
<tr>
<td>Minimum thickness sand layer sls</td>
<td>(d_{\text{sand}})</td>
<td>Equation 6-15</td>
<td>12</td>
<td>m</td>
</tr>
<tr>
<td>Maximum polder depth sls</td>
<td>(h_{\text{puplifting}})</td>
<td></td>
<td>-9</td>
<td>m CD</td>
</tr>
<tr>
<td>Minimum thickness sand layer uls</td>
<td>(d_{\text{sand}})</td>
<td>Equation 6-16</td>
<td>16</td>
<td>m</td>
</tr>
<tr>
<td>Maximum polder depth uls</td>
<td>(h_{\text{puplifting}})</td>
<td></td>
<td>-5</td>
<td>m CD</td>
</tr>
</tbody>
</table>

Table 34: Uplifting calculation

In conclusion, to avoid uplifting the minimum thickness of the sand fill in the Ultimate Limit State is 16 m, resulting in a maximum polder depth of 11.4m below the crest level of the quay wall flood defense corresponding with a level of -6.5 meter MSL or -5 meter CD. This is the maximum depth of a polder terminal at Tuas. If deeper polders are preferred a solution needs to be found to avoid uplifting due to the water overpressures, such as a concrete floor with piles (which is expensive).
6. Flood defense system

6.4.2. Stability calculation

To investigate the stability of a polder terminal quay wall flood defense the loads and load combinations of the structures are determined. Following from the retaining function of the quay wall the pressures to be retained contain soil and water pressures. Other horizontal loads consist of fender and bollard forces as a result of vessel berthing. Bearing requirements consist of vertical loads posed by container cranes (founded on the quay wall structure) and a surcharge on top of the combined quay wall flood defense, see Figure 30.

**Conventional quay wall loading**

A schematization of the design load combinations on a conventional quay wall is shown in Figure 64. The design should be able to cope with the outside water pressure, which is variable, in combination with the horizontal loads of soil and vessels, vertical crane loads and possible surcharge. The load combinations are described and illustrated below:

- **During Low Astronomical Tide**: soil pressure, crane and surcharge load, bollard forces and minimum water pressure;
- **During Extreme Water Levels**: soil pressures, crane loads, surcharge and maximum water pressure.

*Note: It is assumed during an Extreme Water Level no vessels are moored to the quay wall, resulting in the absence of fender and bollard forces.*

**Polder terminal quay wall loading**

For a polder terminal different design loads are present, because of the lower terminal yard. The structure itself requires two retaining walls, one on the sea side to retain the outside water levels, and one on the polder side to overcome the height difference between the quay wall and the polder terminal.

The loading on the sea side retaining wall will be similar to that of a conventional terminal quay wall; loading on the polder side retaining wall is different. The resulting horizontal load due to soil pressure and groundwater pressure is lower than that of the conventional terminal, depending on the polder depth.
Also, when the terminal yard is lowered a different load on the quay wall structure should be taken into account, which is the horizontal water pressure on the polder side wall due to a flood in the polder terminal. A limit state occurs if the polder is flooded due to an extreme water level. After a while the outside water level will lower while inside the polder is still flooded.

The resulting load combinations of a polder terminal quay wall are illustrated in Figure 65, assuming the quay wall flood defense behaves as one structure as is the case for gravity structures as well as sheet pile wall cofferdams.

- ‘Normal’ loading: sea side retaining wall under same load combinations as conventional terminal quay together with landside wall loaded by soil pressure, groundwater pressure and crane loads.
- ‘Polder flood’ loading: sea side quay wall under loading due to Low Astronomical Tide, surcharge and polder side wall loaded by soil pressure and maximum water pressure due to a flood.

![Figure 65: Loading combinations polder terminal quay wall](image)

**Stability optimization**

This section will investigate the influence of the lowered terminal yard on the stability of the quay wall flood defense. The main function of a quay wall is to retain the horizontal loads due to soil and water pressure; there are three structural failure mechanisms to be considered for gravity structures. These are described in chapter 2: horizontal sliding, overturning (rotation) and bearing capacity, see equations 6-17 till 6-19. These equations are based on elastic stress distributions. The resulting loads on a gravity structure (caisson) are illustrated in Figure 66, an explanation of the parameters is found in Table 35.

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal sliding</td>
<td>( \frac{\sum H}{\sum V} &lt; f (= 0.5) )</td>
</tr>
<tr>
<td>Overturning</td>
<td>( \frac{\sum M}{\sum V} &lt; \frac{W}{6} )</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>( \frac{\sum V}{W \times L} + \frac{\sum M}{\frac{1}{6} W \times L^2} &lt; P_{\text{max}} )</td>
</tr>
</tbody>
</table>

\( f \): safety factor
\( P_{\text{max}} \): maximum allowable pressure
6. Flood defense system

![Diagram of caisson gravity structure with loads](image)

**Figure 66: Loads on caisson gravity structure**

*Note: Fender and bollard forces are omitted at this stage of the investigation of the stability. Further, loads due to calamities such as ship collision and earthquakes are also omitted; this is explained in paragraph 6.3.*

<table>
<thead>
<tr>
<th>Gravity loads</th>
<th>Parameter</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portside water pressure</td>
<td>$H_1 = 0.5 \gamma_w h_w^2$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Terminal side soil pressure</td>
<td>$H_2 = 0.5 K_0 \sigma_{eff} h_p^2$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Terminal side groundwater pressure</td>
<td>$H_3 = 0.5 \gamma_w h_{gw}^2$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Weight of quay wall structure</td>
<td>$V_1 = V_{concrete} \gamma_{concrete} + V_{sand} \gamma_{sand}$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Upward (constant) water pressure</td>
<td>$V_2 = \gamma_w h_{gw} W$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Upward (variable) water pressure</td>
<td>$V_3 = 0.5 \gamma_w (h_w - h_{gw}) W$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Surcharge on caisson</td>
<td>$Q = q_{surcharge}$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Summation horizontal forces</td>
<td>$\sum H = H_1 - H_2 - H_3$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Summation vertical forces</td>
<td>$\sum V = V_1 + Q - V_2 - V_3$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Summation moments</td>
<td>$\sum M$ moment around midpoint bottom caisson (R)</td>
<td>kNm/m</td>
</tr>
</tbody>
</table>

Table 35: Loads on caisson gravity structure

From the figure can be concluded that the terminal level has a large influence on the magnitude of the horizontal loads on the terminal side of the structure. This in turn has a large influence on the sliding, overturning and bearing capacity failure mechanisms. When the terminal yard level is variable three ‘extreme’ situations could occur with respect to the horizontal forces, see Figure 67.

- **A:** The first situation is a quay wall with a terminal yard equal to the outside bottom level of the sea.
  
  In this case the resultant horizontal force is towards the terminal side, due to the outside water pressure on port side of the structure.
B: The second situation is a quay wall with lowered terminal yard, up to a level where the resultant horizontal force is zero. This occurs when the water pressure on sea side is equal to the combined soil and water pressure on the terminal side. This favors the construction of the quay wall because the resultant load on the structure is minimal.

C: The third situation is a quay wall with terminal yard equal to the quay wall height, which is the case for a conventional terminal. In this case the resultant horizontal force is towards the sea side, because the soil pressure on terminal side is higher than the water pressure on sea side.

Figure 67: Horizontal forces on quay wall flood defense

Two load combinations were explained in the last section, the first being ‘normal loading’ and the second ‘flood loading’, the stability of the quay wall flood defense structure will be calculated for each load combination in relation to the polder level. The quay wall height is fixed as well as the wall thickness of the caisson (assumed \( t = 0.5 \) meter), resulting in a direct relation between the quay wall width and the required weight.

**Normal loading**

The loads during normal loading are shown in the left cross section of Figure 65. The graph in Figure 68 shows the relation between the required caisson weight versus polder levels relative to Mean Sea Level for each failure mechanism during ‘normal’ loading. Because of the direct relation between caisson weight and width the width is also shown in a secondary vertical axis. The polder levels of load situations A, B and C are shown in the graph, as well as the determined boundaries of piping and uplifting.

Regarding piping one could argue if this is an actual boundary because the seepage length could easily be increased by adding seepage screens to make deeper polders possible. The boundary of uplifting is however definitive.
6. Flood defense system

![Graph showing weight and width required for stability caisson during normal loading.]

**Figure 68: Relation caisson weight / width with polder level during normal loading**

The graph also shows for each failure mechanism (sliding, rotation and bearing capacity) the minimal weight required at every polder level. An ‘optimal’ point is found; a level where the required weight is minimal, for each failure mechanism.

The level where the resultant horizontal forces are zero (B) is found by looking at sliding failure: a level of -3 meter MSL is found. From polder levels lower than -3 meter MSL (A) the quay wall flood defense retains the outside water whereas for levels higher than -3 meter MSL (C) the quay wall flood defense retains soil pressure on terminal side (as in a conventional quay wall).

*Note: The weight of the caisson gravity structure can never be zero, as determined for sliding failure, because the other failure mechanisms also have influence on the required weight.*

The actual boundary of the required weight is determined by the rotation failure mechanism (blue line) and partly by the bearing capacity check (green line). Together with the uplifting boundary these mechanisms determine the ‘design area’ of the polder level with respect to the required weight of the caisson structure during normal loading (if it is assumed piping is dealt with by using seepage screens).

Using the graph the optimal quay wall weight / width can be found for a given polder depth. As the quay wall cost are directly related to the weight and width this optimal point also determines the polder depth at which costs of the quay wall are minimal.
Flood loading

The loads during normal loading are shown in the right cross section of Figure 65. The required caisson weight/width during ‘flood’ loading are shown in the graph in Figure 69, as well as the polder levels where situations A, B and C occur.

![Figure 69: Relation caisson weight / width with polder level during flood loading](image)

In this case the level where the resulting horizontal force is zero is found at maximum polder depth, which is obvious because in that case the sea side water level is equal to the terminal side water level due to a flood. Regarding other failure mechanisms the same conclusions as during normal loading can be made: the required weight is determined by the rotation failure mechanism (blue line) with a boundary at the maximum polder depth determined by uplifting failure.

Tuas polder terminal

For the functionality of the quay wall flood defense of a polder terminal a minimum width of the quay wall is required of 40 meters to provide a foundation for the container cranes. From the graphs can be concluded that with such a width the caisson will have sufficient weight to assure stability against sliding, rotation and bearing capacity failure. The only boundaries are those set by piping and finally uplifting. A maximum polder level of -6.5 meter MSL is therefore possible.
6. Flood defense system

6.4.3. Logistic requirements

Lower polder terminal levels will increase the turnover times of a container crane, decreasing the amount of cycles and thereby the efficiency of the cranes. This will have a large impact on the efficiency of the whole container terminal. To avoid such a situation part of the cost saving of the reclamation fill could be reinvested in a new container crane especially made to efficiently transfer containers between vessels and the polder terminal yard.

6.5. Conclusions and recommendations

This chapter focused on design aspects of the flood defense structure of the polder terminal which not only facilitates vessel berthing but also assures flooding safety. The design height of the quay wall is determined by taking sea level rise in to account, the construction height is determined by adding a supplement for possible settlement during the lifetime of the structure.

6.5.1. Quay wall flood defense structures

Possible gravity structures are caissons and silos, the main disadvantage of such structures are differential settlements between adjacent caissons and/or silos. These structures will be more costly than sheet pile structures due to the prefabrication in a dry dock and corresponding transportation to the construction site.

When sheet pile structures are used it should be checked whether or not both retaining walls influence each other making a cofferdam; this can be checked by a rule of thumb of Therzaghi. These structures will generally be less costly then gravity structures.

Piled deck structures are actually a combination of two structures, one piled deck to facilitate vessel berthing and an embankment or retaining wall to overcome the height difference between the port bottom and the polder yard. Regarding costs, these structures are expected to be more expensive than sheet piles because two structures are required.

At Tuas, Singapore, gravity structures or sheet pile walls are the most suitable quay wall flood defense structure. When sheet pile wall structures are used in a the structure will behave as a cofferdam, the resulting vertical load on the subsoil will not differ largely compared to a gravity structure.
6. Flood defense system

6.5.1. Polder level

For terminal yards below quay wall level, as is the case for polder terminals, another loading situation can occur which is a flood inside the terminal yard. This will result in an additional horizontal load on the quay wall structure which is to be taken in to account in the design stage of the quay wall flood defense.

Investigation in the requirements of the polder level for gravity structures resulted in the following conclusions regarding piping, uplifting, stability and logistics:

Piping failure determines a boundary for the polder level for a given width of the gravity structure. If deeper polders are made seepage screens are required to avoid piping failure. Uplifting failure determines a boundary for the polder level, to assure vertical equilibrium of the soil body in the polder. Deeper polders then the level determined for uplifting are not possible.

Considering stability of the polder terminal quay wall, compared to the conventional terminal quay wall, it can be concluded that the polder terminal quay wall flood defense can be made lighter than the conventional terminal quay wall due to the lower resultant horizontal load on the structure. The weight necessary for stability (for every polder level) is determined by the rotational instability mechanism.

The following graph shows the ‘design area’ for the required weight related to the polder level of a gravity structure as a quay wall flood defense for a polder terminal (when ‘piping boundary’ is neglected). Using the graph the optimal quay wall weight / width can be found for a given polder depth, which also corresponds with optimal quay wall cost.

![Design area gravity structure weight related to polder level](image)

Figure 70: Design area gravity structure weight related to polder level
6. Flood defense system

6.5.2. Cost comparison

The possibility of designing a ‘lighter’ structure for a polder terminal quay wall, compared to a conventional terminal quay wall, will result in lower cost for the polder terminal quay wall. Especially considering the fact that the relation between the quay wall cost and retaining height increases non linearly; lower retaining heights have much lower construction cost, see Figure 59 and chapter 8.

The savings in cost could be partially offset by the fact that the quay wall for a polder terminal has a minimum width determined by the container cranes which is wider than the width of the conventional terminal quay wall. These cranes are founded on top of the quay wall. The saving in cost due to a lighter structure could be offset by the requirement of a wider structure compared to a conventional quay wall.

Chapter 8 investigates whether or not the largest savings are made when constructing a terminal yard at a polder level where the horizontal load on the structure is minimal or when constructing the terminal yard at the boundary for uplifting stability. The largest savings are made at the uplifting boundary level, because the contribution of the reclamation cost (about 75%) to the total investment cost is much higher than the contribution of the quay wall cost (about 25%).

6.5.3. Sheet pile and pile supported platform structures

It is assumed that the same conclusions can be made for the sheet pile (cofferdam) structure. When comparing the sheet pile structure of a conventional terminal with a cofferdam for a polder terminal the anchors and the anchor wall of the cofferdam will be longer which could counteract the savings in cost due to a lighter structure.

For pile supported platforms different conclusions are made, depending on the type of soil retaining structure. When an embankment is designed the lowered polder yard will not have a large influence on the design loads of the structure. For retaining walls however, due to the lower horizontal soil pressure, the structure could be designed lighter in comparison to the conventional terminal quay wall.

Note: The conclusions drawn in this section are largely based on assumptions and require further investigation; this is however beyond the scope of this master thesis.
6. Flood defense system
7. Polder water management

7.1. Introduction

In chapter 3 a distinction is made between two top events: ‘Large scale flooding’ and ‘Small scale flooding’. Safety against ‘Large scale flooding’ is dealt with through the economic risk approach of chapter 6; this chapter will focus on how to deal with ‘Small scale flooding’.

It was concluded that the reclamation cost saving of the polder terminal could be partially offset by the cost of an adequate water management system to assure no ‘small scale flooding’ inside the polder. This is investigated in this chapter.

First the water balance of a polder terminal is determined, after which a simple approach, based on ‘rules of thumb’ is applied to determine the total cost of the water management system.

7.2. Water balance

To determine the parameters which are of influence on the polder water management the water balance needs to be determined. The general water balance function of a polder is given in equation 7-1, all parameters are explained in Table 36; certain parameters can be neglected in a polder terminal. The resulting functions will be used to determine, for a given rainfall intensity, the required storage and drainage capacity to avoid inundation of the polder terminal.

\[
\text{Loading} - \text{Relieving} = \text{Storage} \\
(N + ES + S + L + I + P) - (ET - IS - Q_{pump}) = \Delta V 
\]  

(7-1)

<table>
<thead>
<tr>
<th>Parameter [m/day]</th>
<th>Explanation</th>
<th>Negligible?</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>Precipitation: rain, snow and hail (condensation is ignored).</td>
<td>Most important parameter.</td>
</tr>
<tr>
<td>ES</td>
<td>Upward seepage, groundwater discharge through dikes and/or deeper layers.</td>
<td>Negligible, see section 7.2.1.</td>
</tr>
<tr>
<td>S</td>
<td>Lock water enters the polder through locks.</td>
<td>Neglected, because there are no locks.</td>
</tr>
<tr>
<td>L</td>
<td>Leakage water enters the polder at the locks due to leakage of the doors.</td>
<td>Neglected, because there are no locks.</td>
</tr>
<tr>
<td>I</td>
<td>Intake water is used for flushing of the waterways in polders, to keep these clean.</td>
<td>For a preliminary design this amount can be neglected.</td>
</tr>
</tbody>
</table>
7. Polder water management

<table>
<thead>
<tr>
<th>P</th>
<th>Production process water is tap water discharged into the polder by the industry, or in the case of a polder terminal, ship waste water.</th>
<th>For a preliminary design this amount can be neglected.</th>
</tr>
</thead>
<tbody>
<tr>
<td>ET</td>
<td>Evapo-transpiration is dependent on the type of vegetation present.</td>
<td>Neglected, there are no large areas with vegetation inside the polder so evapo-transpiration will be minimal. Evaporation of water on paved surfaces is also neglected.</td>
</tr>
<tr>
<td>IS</td>
<td>Downward seepage, groundwater discharge through dikes or from deeper layers.</td>
<td>Neglected, all outflow of water will be provided by pumping stations</td>
</tr>
<tr>
<td>Q&lt;sub&gt;pump&lt;/sub&gt;</td>
<td>Discharge through gravity flow or discharge pumps.</td>
<td>No discharge through gravity flow will be present, only discharge pumps.</td>
</tr>
<tr>
<td>V&lt;sub&gt;storage&lt;/sub&gt;</td>
<td>Storage of water in the canals</td>
<td>To be analyzed</td>
</tr>
<tr>
<td>V&lt;sub&gt;terminal&lt;/sub&gt;</td>
<td>Storage capacity inundated water on the terminal surface</td>
<td>To be analyzed</td>
</tr>
</tbody>
</table>

Table 36: Water balance parameters

The resulting water balance equations are presented in equations 7-2 till 7-5.

\[
(N - Q_{pump}) = \Delta V \tag{7-2}
\]

\[
V = V_{storage} + V_{terminal} \tag{7-3}
\]

\[
\Delta V_{storage} = A_{storage} * \Delta d_{storage} \tag{7-4}
\]

\[
\Delta V_{terminal} = A_{p} * \Delta d_{inundation} \tag{7-5}
\]

From the function can be understood that when the rainfall intensity (N) exceeds the drainage capacity (Q<sub>pump</sub>) storage is required. Storage can be provided by the storage areas inside the polder (V<sub>storage</sub>) or on the terminal surface (V<sub>terminal</sub>), the latter is defined as inundation or ‘small scale flooding’ and should be avoided.

7.2.1. Seepage

In the water balance equation seepage is considered negligible; the following section will provide a short explanation why. Seepage is the result of groundwater flow through different layers of soil in the reclamation fill. The amount of seepage water is calculated by the formula of Darcy which is shown in equations 7-6 and 7-7.

\[
i = \frac{dh}{ds} \tag{7-6}
\]

\[
q = -k * i \ (\text{m/s}) \tag{7-7}
\]
7. Polder water management

The total amount of seepage is defined as the product of the permeability of the subsoil ($k$) with the hydraulic gradient ($i$). The permeability ($k$) of the soil depends on subsoil conditions, as shown in Table 37.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$k$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>$10^{-10} - 10^{-8}$</td>
</tr>
<tr>
<td>Silt</td>
<td>$10^{-8} - 10^{-6}$</td>
</tr>
<tr>
<td>Sand</td>
<td>$10^{-6} - 10^{-4}$</td>
</tr>
<tr>
<td>Gravel</td>
<td>$10^{-3} - 10^{-1}$</td>
</tr>
</tbody>
</table>

*Table 37: Permeability coefficients*

The subsoil of successful polders usually consists of one or multiple layers of low conductive soil to prevent large amounts of upward seepage, an example of a fill for a polder terminal containing an impermeable layer in the lower part is shown in Figure 71. A short calculation is made of the expected upward seepage for the maximum polder depth determined in the last chapter by uplifting; for lower polder depths lower amounts of seepage are expected. The amount of seepage depends on the conductivity of the clay layer, see Table 38.

*Figure 71: Seepage calculation*
7. Polder water management

### 7.1. Clay seepage

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Function</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Head difference</td>
<td>$\Delta h$</td>
<td>8.5</td>
<td>m</td>
</tr>
<tr>
<td>Seepage length</td>
<td>$\Delta L$</td>
<td>40</td>
<td>m</td>
</tr>
<tr>
<td>Hydraulic gradient</td>
<td>$i$</td>
<td>0.21</td>
<td>-</td>
</tr>
<tr>
<td>Permeability clay</td>
<td>$k$</td>
<td>$10^{-8}$</td>
<td>m/s</td>
</tr>
<tr>
<td>Seepage (per second)</td>
<td>$q_s$</td>
<td>$2.1 * 10^{-9}$</td>
<td>m/s</td>
</tr>
<tr>
<td>Seepage (per day)</td>
<td>$q_d$</td>
<td>0.18</td>
<td>mm/d</td>
</tr>
</tbody>
</table>

Table 38: Seepage through clay

From the table can be concluded that, given a maximum head difference of 8.5 meters between the Highest Astronomical Tide and the polder water level, a maximum amount of 0.18 mm/ per day of seepage can be expected. To compare the same calculation is made for seepage through a sand layer.

### 7.2. Sand seepage

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Function</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic gradient</td>
<td>$i$</td>
<td>0.21</td>
<td>-</td>
</tr>
<tr>
<td>Permeability clay</td>
<td>$k$</td>
<td>$10^{-3}$</td>
<td>m/s</td>
</tr>
<tr>
<td>Seepage (per second)</td>
<td>$q_s$</td>
<td>$2.1 * 10^{-4}$</td>
<td>m/s</td>
</tr>
<tr>
<td>Seepage (per day)</td>
<td>$q_d$</td>
<td>18,000</td>
<td>mm/d</td>
</tr>
</tbody>
</table>

Table 39: Seepage through sand

Because the permeability of sand is much higher than that of clay a very large amount of seepage is expected which should be avoided in a polder. That is why it is very important that part of the fill for a polder terminal is impermeable, which will serve as a seepage screen limiting the amount of seepage.

#### 7.2.2. Rainfall intensities

When seepage is neglected rainfall is the only inflow of water in the polder, as can be seen in equation 7-2. The rainfall intensities and corresponding frequencies of occurrence are required to determine the risk of inundation. The probability of inundation is directly related to these maximum rainfall intensities, the storage capacity and possible pump failure.

#### 7.3. Risk framework

From the last paragraph can be concluded that inundation due to rainfall is prevented by two parameters, the drainage capacity and the storage capacity. To determine the optimal combination of both the total cost needs to be determined. The cost of drainage also depends on the polder level, which introduces a third variable to be optimized. If the polder level is chosen above the Lowest Astronomical Tide drainage through gravity flow is possible, whereas for a polder level below LAT pumps are required.
7. Polder water management

A similar economic approach, as made in chapter 5, to determine the optimal capacities of the water storage and drainage system which correspond with minimal total cost is explained in appendix D. This method is not investigated in detail. Instead a simple approach [XIX] is described which will be used to optimize the required storage and drainage capacity of the polder terminal.

### 7.3.1. Drainage / storage capacity

Simple steps to determine the required drainage and storage capacity are explained below:

1. First a certain storage area needs to be determined; a conceptual design of the water storage system in the polder terminal is made in chapter 3 which covers 5% of the total terminal area. This amount of storage is assumed sufficient at this stage.
2. The design rainfall intensity is determined based on an extreme storm event; the corresponding total amount of inflow can be found by multiplying with the terminal area, see equation 7-9.
3. Next, the inflow of rain is divided in a part which is drained immediately ($Q_{\text{pump}}$) and a part which will be stored ($\Delta V_{\text{storage}}$).
4. The required depth of the storage channels can be determined by dividing the required amount of storage volume by the storage area, equation 7-11.

\[
\begin{align*}
A_{\text{storage}} & = A_p \times 0.05 \\
V_{\text{in}} & = A_p \times N_{\text{storm}} \times t_{\text{storm}} \\
V_{\text{in}} & = \Delta V_{\text{storage}} + Q_{\text{pump}} \times t_{\text{storm}} \\
\Delta h & = \frac{\Delta V_{\text{storage}}}{A_{\text{storage}}} 
\end{align*}
\]
7.3.2. **Optimization approach**

To optimize the drainage and storage capacity the total cost of the water management system is to be determined, consisting of the investments and risk. The investments are determined through equation 7-12.

\[
I_{\text{tot}} = I_{\text{pump}} * Q_{\text{pump}} + I_{\text{storage}} * V_{\text{storage}}
\]  

(7-12)

To determine the risk the damage is determined for two storm events with rainfall intensities which exceed the design rainfall intensities; one with a probability of exceedance \( p_1 \) and damage \( d_1 \) and another with a probability of exceedance \( p_2 \) and damage \( d_2 \).

\[
P_f(\text{inundation}) = P_f(N > N_{\text{storm}})
\]

(7-13)

The risk is determined by approximating the area under the graph of probability versus damage; this area is approximated by the triangle shown in red, which is the linearised risk for both events. Using the angle between both events the surface of the triangle can be calculated through equation 7-18.

![Figure 73: Graphs to approximate risk](image)

Distance ‘a’ is approximated by multiplying the angle between both events with \( d_1 \); distance ‘b’ is approximated by multiplying the angle between both events with \( p_2 \), see equations 7-14 till 7-17.
7. Polder water management

\[
a = \frac{p_1 - p_2}{d_2 - d_1} \cdot d_1 \quad (7-14)
\]

\[
b = \frac{d_2 - d_1}{p_1 - p_2} \cdot p_2 \quad (7-15)
\]

*Basis* triangle: \(p_1 + a\)  
*Height* triangle: \(d_2 + b\)  

(7-16)

(7-17)

The present value of the risk is found by dividing the area of the triangle with the interest rate.

\[
R_{\text{surface}} = \frac{1}{2} \left( p_1 + d_1 \cdot \frac{p_1 - p_2}{d_2 - d_1} \right) \cdot \left( d_2 + p_2 \cdot \frac{d_2 - d_1}{p_1 - p_2} \right) / r' \quad (7-18)
\]

When the investment cost of equation 7-12 are added the total cost is found with equation 7-19. The economical solution is defined as the one with the minimum of total cost.

\[
C_{\text{tot}} = I_{\text{pump}} \cdot Q_{\text{pump}} + I_{\text{storage}} \cdot V_{\text{storage}} + \frac{1}{2} \left( p_1 + d_1 \cdot \frac{p_1 - p_2}{d_2 - d_1} \right) \cdot \left( d_2 + p_2 \cdot \frac{d_2 - d_1}{p_1 - p_2} \right) / r' \quad (7-19)
\]

*Note: The economical solution does not have to be the solution as implemented by the port operator. The final design is based on the investment costs, probabilities of exceedance and corresponding risks involved. This is a decision of political nature as was explained before.*

### 7.4. Tuas implementation

The described method in paragraph 7.3 will be implemented for the Tuas case; the boundary conditions are treated in chapter 3. The Intensity-Duration-Frequency curves for Singapore will be used to determine the required storage and drainage capacity; first the cumulative Intensity-Duration-Frequency is derived in Figure 74. These curves can easily be used to determine a combination of storage and drainage capacity which ensures no inundation for the storm events defined.
As is shown in Figure 74, a storage capacity of 128 mm is required in combination with a drainage capacity of 12 mm/hr. The required storage area and channel depth, as well as the pumping capacity, are determined in Table 40.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Parameter Origin</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage capacity</td>
<td>Q&lt;sub&gt;pump&lt;/sub&gt; To drain 12 mm/hr</td>
<td>10</td>
<td>m&lt;sup&gt;3&lt;/sup&gt;/s</td>
</tr>
<tr>
<td>Total polder area</td>
<td>A&lt;sub&gt;p&lt;/sub&gt; Paragraph 5.3, Table 24</td>
<td>312</td>
<td>ha</td>
</tr>
<tr>
<td>Storage volume</td>
<td>V&lt;sub&gt;storage&lt;/sub&gt; Equation 7-16 and 7-17</td>
<td>400,800</td>
<td>m&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>Total storage area</td>
<td>A&lt;sub&gt;storage&lt;/sub&gt; Equation 7-15</td>
<td>15.6</td>
<td>ha</td>
</tr>
<tr>
<td>Storage channel depth</td>
<td>h&lt;sub&gt;storage&lt;/sub&gt; Equation 7-18</td>
<td>2.57</td>
<td>m</td>
</tr>
</tbody>
</table>

Table 40: Calculation storage/drainage capacity

### 7.4.1. Storm events

The storm events which will be used to approximate the total risk are defined in Table 41, information is obtained from the IDF curves for Singapore in Figure 74.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Design storm</th>
<th>Event 1</th>
<th>Event 2</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability of exceedance</td>
<td>0.01</td>
<td>0.02</td>
<td>0.01</td>
<td>-</td>
</tr>
<tr>
<td>Rainfall intensity</td>
<td>0.14</td>
<td>0.04</td>
<td>0.08</td>
<td>m/hr</td>
</tr>
<tr>
<td>Storm duration</td>
<td>1</td>
<td>4</td>
<td>2</td>
<td>hr</td>
</tr>
</tbody>
</table>

Table 41: Storm events Tuas
7. Polder water management

As shown in the figure a critical situation occurs between 2 and 4 hrs, for cumulative rainfall intensities between 150 and 175 mm. Inundation of the terminal yard will occur if, during these events, insufficient storage capacity is available.

Note: This calculation is meant to approximate the total cost of the water management system to compare with the reclamation saving of the polder terminal. The storm events are largely based on assumptions to approximate the risk of the designed water management system.

7.4.2. Total cost

To determine the total cost of the designed water management system the unit rates for pumping and storage costs are required, as well as damage costs which are given in Table 42.

<table>
<thead>
<tr>
<th>Pumps</th>
<th>Origin</th>
<th>Rate</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>I_{pump}</td>
<td></td>
<td>15,000</td>
<td>€/(m³/min)</td>
</tr>
<tr>
<td>Storage</td>
<td>I_{storage}</td>
<td>5</td>
<td>€/m³</td>
</tr>
<tr>
<td>Direct damage</td>
<td>D_i</td>
<td>Paragraph 5.3, Table 24</td>
<td>360 mln</td>
</tr>
<tr>
<td>Indirect damage</td>
<td>D_t</td>
<td>Paragraph 5.3, Table 24</td>
<td>1,1 mln</td>
</tr>
</tbody>
</table>

Table 42: Unit rates water management

The total cost for the conceptual design is calculated in the following section. The risk is determined with the storm events defined in Table 41, assuming during these events only half of the storage capacity is available.

<table>
<thead>
<tr>
<th>Total inflow</th>
<th>Equation</th>
<th>Event 1</th>
<th>Event 2</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>ΔV_{in}</td>
<td>Equation 7-2</td>
<td>292,800</td>
<td>427,200</td>
<td>m³</td>
</tr>
<tr>
<td>Terminal storage</td>
<td>V_{terminal}</td>
<td>Equation 7-3</td>
<td>92,400</td>
<td>226,800</td>
</tr>
<tr>
<td>Inundation depth</td>
<td>d_{inundation}</td>
<td>Equation 7-5</td>
<td>0.03</td>
<td>0.07</td>
</tr>
<tr>
<td>Pumping time</td>
<td>t_{pump}</td>
<td>V_{terminal}/Q_{pump}</td>
<td>0.11</td>
<td>0.26</td>
</tr>
<tr>
<td>Direct damage</td>
<td>D_{direct}</td>
<td>D_t × d_{inundation}</td>
<td>11</td>
<td>26</td>
</tr>
<tr>
<td>Indirect damage</td>
<td>D_{indirect}</td>
<td>D_t × t_{pump}</td>
<td>0.1</td>
<td>0.3</td>
</tr>
<tr>
<td>Total damage</td>
<td>D</td>
<td>D_{direct} + D_{indirect}</td>
<td>11.1</td>
<td>26.3</td>
</tr>
</tbody>
</table>

Table 43: Total damage calculation for drainage capacity Q_{pump} = 10m³/s

<table>
<thead>
<tr>
<th>Pump cost</th>
<th>Equation</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_{pump}</td>
<td>I_{pump} × Q_{pump}</td>
<td>9</td>
<td>mln €</td>
</tr>
<tr>
<td>Storage cost</td>
<td>C_{storage}</td>
<td>I_{storage} × Q_{storage}</td>
<td>2</td>
</tr>
<tr>
<td>Total risk</td>
<td>R</td>
<td>Equation 7-20</td>
<td>11.4</td>
</tr>
<tr>
<td>Total cost</td>
<td>TC</td>
<td>C_{pump} + C_{storage} + R</td>
<td>22.4</td>
</tr>
</tbody>
</table>

Table 44: Total cost calculation for drainage capacity Q_{pump} = 10m³/s

A numerical approach is made to determine the total costs dependant on the drainage capacity, given the storage capacity and events found in the last sections. The investments, risk and total costs are plotted against the drainage capacity in Figure 75.
Optimization water drainage

![Optimization water drainage graph](image)

**Figure 75: Optimization drainage capacity**

The minimal total costs are found for an economic optimal drainage capacity of 7 m$^3$/s, see Table 45 and Table 46. Compared to a capacity of 10 m$^3$/s a saving of 500,000 euro is made, which is only marginal.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Equation</th>
<th>Event 1</th>
<th>Event 2</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total inflow</td>
<td>$\Delta V$</td>
<td>336,000</td>
<td>448,800</td>
<td>m$^3$</td>
</tr>
<tr>
<td>Terminal storage</td>
<td>$V_{\text{terminal}}$</td>
<td>135,600</td>
<td>248,400</td>
<td>m$^3$</td>
</tr>
<tr>
<td>Inundation depth</td>
<td>$d_{\text{inundation}}$</td>
<td>0.04</td>
<td>0.08</td>
<td>m</td>
</tr>
<tr>
<td>Pumping time</td>
<td>$t_{\text{pump}}$</td>
<td>0.22</td>
<td>0.41</td>
<td>day</td>
</tr>
<tr>
<td>Direct damage</td>
<td>$D_{\text{direct}}$</td>
<td>15.8</td>
<td>28.8</td>
<td>mln €</td>
</tr>
<tr>
<td>Indirect damage</td>
<td>$D_{\text{indirect}}$</td>
<td>0.25</td>
<td>0.45</td>
<td>mln €</td>
</tr>
<tr>
<td>Total damage</td>
<td>$D$</td>
<td>16</td>
<td>29</td>
<td>mln €</td>
</tr>
</tbody>
</table>

**Table 45: Total damage calculation for drainage capacity $Q_{\text{pump}} = 7 m^3/s$**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Equation</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pump cost</td>
<td>$C_{\text{pump}}$</td>
<td>6.3</td>
<td>mln €</td>
</tr>
<tr>
<td>Storage cost</td>
<td>$C_{\text{storage}}$</td>
<td>2</td>
<td>mln €</td>
</tr>
<tr>
<td>Total risk</td>
<td>$R$</td>
<td>13.6</td>
<td>mln €</td>
</tr>
<tr>
<td>Total cost</td>
<td>$TC = C_{\text{pump}} + C_{\text{storage}} + R$</td>
<td>21.9</td>
<td>mln €</td>
</tr>
</tbody>
</table>

**Table 46: Total cost calculation for drainage capacity $Q_{\text{pump}} = 7 m^3/s$**

The estimated risk for the optimal design of the water management system, 14 mln €, is rather high compared to the total risk of ‘large scale flooding’ as determined in chapter 5 which is 3 mln €. This is explained by the fact that the risk of ‘small scale flooding’ is determined by the multiplication of the probability of the extreme rainfall and the corresponding damages.

At the optimal drainage capacity the failure probability of small scale flooding is much higher (order 1/100) than the failure probability of large scale flooding (order 1/10,000); this results in a larger risk of small scale flooding. Due to the relative low amount of total cost for the water management system compared to the total investment of the polder terminal it is therefore advised to design a higher drainage capacity in order to minimize the risk.

K.Lendinger
7. Polder water management

### 7.4.3. Risk minimization

A drainage capacity is sought for which minimizes the risk of small scale flooding to a value of the same order as the risk of large scale flooding, which is 3 mln €. Further, a drainage capacity is sought for where the risk is minimal, see Table 47.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Equation</th>
<th>Drainage 32 m³/s</th>
<th>Drainage 40 m³/s</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pump cost</td>
<td>$C_{\text{pump}} = I_{\text{pump}} \cdot Q_{\text{pump}}$</td>
<td>29</td>
<td>36</td>
<td>mln €</td>
</tr>
<tr>
<td>Storage cost</td>
<td>$C_{\text{storage}} = I_{\text{storage}} \cdot Q_{\text{storage}}$</td>
<td>2</td>
<td>2</td>
<td>mln €</td>
</tr>
<tr>
<td>Damage cost (event 2)</td>
<td>$D = D_{\text{direct}} + D_{\text{indirect}}$</td>
<td>8</td>
<td>1</td>
<td>mln €</td>
</tr>
<tr>
<td>Total risk</td>
<td>$R = \text{Equation 7-20}$</td>
<td>3</td>
<td>0.5</td>
<td>mln €</td>
</tr>
<tr>
<td>Total cost</td>
<td>$TC = C_{\text{pump}} + C_{\text{storage}} + R$</td>
<td>34</td>
<td>39</td>
<td>mln €</td>
</tr>
</tbody>
</table>

Table 47: Risk minimization

*Note: For high drainage capacities the total damage is only determined by event 2, as the drainage capacities are higher than the rainfall intensities of event 1 so no inundation will occur.*

From the table can be concluded that the increase in investment (40 mln €) required to minimize the risk of small scale flooding does not exceed the reclamation saving of the polder terminal (200 – 600 mln €). This verifies the aforementioned conclusion that it is advised to invest in a drainage capacity to minimize the risk of small scale flooding, instead of using the optimal drainage capacity which minimizes the total cost, as determined in 7.4.2.
7. Polder water management

7.5. Conclusions and recommendations

This chapter described the water management aspects of the polder terminal through a simple approach which determines the total cost (investment and risk) of the water management system at Tuas. An optimal drainage capacity of 7 m$^3$/s is found with corresponding total cost of 22 mln euros. This is very low compared to the reclamation saving which is in the order of 200 to 700 mln euro (see chapter 8).

The saving made through the proposed optimization of the drainage capacity is only marginal, compared to the total cost of the water management system, especially when considering the unit rates have a certain margin for error. Therefore it can be concluded that an optimization at this stage is unnecessary if the capacities are based on the cumulative rainfall intensity-duration-frequency curves.

The risk at the optimal drainage capacity (14 mln euro) is rather large compared to the risk of large scale flooding (3 mln euro) as determined in paragraph 5.3. To minimize this risk a larger drainage capacity is required. The risk of small scale flooding at Tuas is minimal for a drainage capacity of 40 m$^3$/s; this results in total cost of the water management system in the order of 40 mln euros.

Therefore it is advised to conservatively design a water management system based on the extreme rainfall intensities, in order to minimize the risk (instead of optimizing the required drainage and storage capacities). Not only does this conclusion hold for the Tuas case, but also for other polder terminals due to the relatively low water management cost compared to the reclamation cost. The increased investment of the water management system is marginal compared to the reclamation saving of the polder terminal.

An important factor which is not taken into account in these approaches are the operational cost of the drainage system, which will increase the total cost over the lifetime of the polder terminal. However, due to the expected low contribution of the operational cost to the total cost of the polder terminal, this will not influence the final conclusion.

Note: In the Netherlands polder designs are usually made based on reference polders.
8. Sensitivity analyses

8.1. Introduction

Different assumptions lie on the basis of the economic risk framework approach made in chapter 5, these are discussed in paragraph 5.4 and further investigated in chapter 6 and 7. The influence of these assumptions on the risk framework approach of the polder terminal versus the conventional terminal is investigated in this chapter.

The influence of including the settlement cost in the risk framework approach is not investigated further, however it can be assumed, as explained earlier, that it will benefit the polder terminal. Mainly because a polder terminal requires less soil improvement works than the conventional terminal because of the lower amount of fill material.

Note: These analyses are made to investigate the sensitivity of the risk framework approach to different assumptions; the results are still estimates of the actual total cost.

8.2. Basic case

The ‘basic case’ which will be used for comparison purposes is based on the Tuas case in Singapore, see paragraph 5.3. The main difference with the estimates of chapter 5 is that the polder level will not be chosen at Mean Sea Level, but at the lower boundary posed by the uplifting failure mechanism which was determined in chapter 6: -6.5 meter MSL. Further, the water management cost as determined in chapter 7 is also added. The important design values are given in Table 24, the cost for the water drainage system is chosen for a design drainage capacity of 40 m$^3$/s, see Table 46.

The resulting economic optimal quay wall level (and corresponding cost) for the polder terminal and the conventional terminal are determined in Table 48.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimal CT</td>
<td>3.8</td>
<td>2,175</td>
<td>0.00310</td>
<td>350</td>
<td>189</td>
<td>12</td>
</tr>
<tr>
<td>Optimal PT</td>
<td>4.5</td>
<td>1,585</td>
<td>0.00002</td>
<td>50,000</td>
<td>4,170</td>
<td>2.1</td>
</tr>
<tr>
<td>Comparison CT</td>
<td>4.5</td>
<td>2,223</td>
<td>0.00002</td>
<td>50,000</td>
<td>189</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Table 48: Total cost of basic case
The economic optimal quay wall height for the polder terminal lies at +4.5 meter MSL, when taking sea level rise and settlement into account the construction height of the quay wall is +5.1 meter MSL which coincides with +6.6 meter CD. This is higher than the optimal height found for a polder level at Mean Sea Level, which was +6.4 meter CD.

From the results can be concluded that a saving of about 30% in investment (640 mln €) is possible compared to the conventional terminal. The risk for the polder terminal is about a factor 20 larger than the risk of the conventional terminal, partly due to the lower risk of the conventional terminal and the increased risk of the polder terminal. The results are plotted in Figure 76.

Figure 76: Basic case sensitivity analysis

The percentage of reclamation saving and the risk ratio between the polder and conventional terminal determined is irrespective of the total polder area; on one side the reclamation cost depends on the polder area but the damage cost also depends on the polder area resulting in the same amount of saving and risk irrespective of the polder area.

8.3. Quay wall cost analysis

Regarding quay wall cost two analyses will be made, firstly the influence of the conclusion that the polder terminal quay wall can be designed lighter and therefore less costly than the conventional terminal quay wall is investigated. Secondly the influence of the actual non-linear relation of the quay wall cost with the retaining height is investigated.
8. Sensitivity analysis

8.3.1. Linear quay wall cost

To determine the influence of lower quay wall cost of the polder terminal versus the conventional terminal, an assumption is made of the difference in cost between both quay walls. This assumption is based on the required retaining height of the quay wall.

In the optimization it is assumed that the retaining height of the polder terminal quay wall, and thus the quay wall cost of the polder terminal, was determined by the required crest height of the quay wall (as is the case for a conventional terminal). The actual retaining height of the polder terminal, as concluded in chapter 6, is determined by the polder level which is much lower than the quay wall level.

- The conventional terminal has a retaining height equal to the quay wall height; because the fill level (terminal yard) is equal to the quay wall height. The total retaining height is 29.6 meter.

- The polder terminal has a retaining height equal to the polder level; because the fill level (terminal yard) is equal to the polder level. The total retaining height is 18 meter, which is 60% of the retaining height of the conventional terminal quay wall.

Because the quay wall still requires a crest level of +5.1 meter MSL and a minimum width of 40 meters (which is wider than the minimum width of the conventional terminal quay wall) it is assumed that the polder terminal quay wall cost is 75% of the conventional terminal quay wall cost (instead of 60%).

This will result in a saving of 25% of the quay wall investment cost (100 mln €) which amounts to 5% of the total cost of the conventional terminal, see Table 49. This is explained by the fact that the reclamation cost form a large part of the total investment cost of the terminal. The total saving of the polder terminal
compared to the conventional terminal is now 34% (760 mln €), an increase of 5% compared to the basic case.

**Conclusion**
Concluding, looking solely at the quay wall cost the difference of 25% between the polder terminal and the conventional terminal is not negligible. But that same reduction of the quay wall cost with 25% results in a reduction of the total cost of only 5% which is not significantly large, especially considering that these total cost have a certain margin for error.

Thus the assumption made in the risk framework that the quay wall cost of the polder terminal is equal to the quay wall cost of the conventional terminal results in a small overestimation of the total cost, however this overestimation is within the margin for error so the end result will not change considerably.

**8.3.2. Non linear quay wall cost**

In the risk framework approach it is assumed that the retaining height has a linear relation with the quay wall cost, while actually it is non linear. A quay wall cost of 1,700 euro per meter retaining height per running meter quay wall was used to determine the quay wall costs, see Table 24, which coincides with a quay wall cost of 50,000 € per running meter quay wall for a retaining height of 30 meters.

![Figure 78: Quay wall cost (linear and non linear)](image)

The graph in Figure 78 shows the resulting linear relation and the actual non linear relation. For retaining heights around 30 meters the difference is marginal, as can be seen in the graph, while for lower retaining heights the difference can be larger.
8. Sensitivity analysis

In the last section it was determined that the actual retaining height of the polder terminal is 18 meter, which according to the linear relation results in a quay wall cost of 30,000 euro per running meter. The actual cost, following from the non linear relation, is 26,000 euro per running meter (or 1,500 euro per meter retaining height per running meter quay). The resulting non linear quay wall cost is determined in Table 50.

<table>
<thead>
<tr>
<th>Sensitivity analysis quay wall cost PT-CT</th>
<th>Retaining height [m]</th>
<th>Percentage basic case quay cost [%]</th>
<th>Quay wall cost [mln €]</th>
<th>Total investment [mln €]</th>
<th>Total cost [mln €]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic case, paragraph 8.2</td>
<td>30m</td>
<td>100</td>
<td>424</td>
<td>1,585</td>
<td>1,587</td>
</tr>
<tr>
<td>Non linear quay wall cost</td>
<td>18m</td>
<td>67</td>
<td>284</td>
<td>1,416</td>
<td>1,431</td>
</tr>
</tbody>
</table>

Table 50: Comparison for non linear quay wall cost

Conclusion

From the previous analysis can be concluded that for lower retaining heights the linear relation applied gives an overestimation of the quay wall cost of about 8%. In combination with the lowered polder quay wall cost from the last section it can be concluded that an overestimation of the actual quay wall cost of 33% is made, which when looking only at the quay wall cost is not negligible. The difference in total cost is still only 6% which is negligible.

Concluding it can be argued that the assumptions regarding linear quay wall cost in the risk framework method of chapter 5 are correct. However, the approximated investment cost and total cost result in small (negligible) overestimates of the investments and total cost of the polder terminal.

8.4. Probability of inundation analysis

Regarding the probability side of the risk of flooding three sensitivity analyses are made. First the assumption that during a flood the polder is filled immediately, not taking time in to account, is investigated after which the influence of the parameters A and B of the extreme water level distribution on the economic optimal levels and total cost are investigated.

Further, in chapter 5 it is explained that the actual probability of flooding is not only directly related to the overtopping failure mechanism, as was assumed, but also includes a failure budget (10%) for other failure mechanisms. The resulting probability of inundation is 10% higher than the probability of overtopping, the influence of this assumption is investigated in 8.4.2.
8.4.1. Flooding time calculation

In this paragraph the time required to fill up the polder is calculated to investigate the assumption that during a situation of overtopping the whole polder is flooded immediately. In the extreme water level distribution determined in chapter 3, parameter B represents the expected overtopping height. For this height the time required to flood the polder completely is calculated using equation 8-1, which determines the velocity at which water enters the polder given a certain head difference $h$.

\[
\text{Torricelli's law: } v = \sqrt{2gh} \tag{8-1}
\]

The time required to flood the polder is calculated in Table 51 for a polder terminal with polder depth of 11.5 meter (terminal yard at -6.5 meter MSL).

<table>
<thead>
<tr>
<th>Flooding time</th>
<th>Parameter</th>
<th>Function</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expected overtopping depth</td>
<td>$B$ ($h$ in Torricelli)</td>
<td></td>
<td>0.15</td>
<td>m</td>
</tr>
<tr>
<td>Total length quay walls</td>
<td>$L_{\text{quay}}$</td>
<td>-</td>
<td>8,600</td>
<td>m</td>
</tr>
<tr>
<td>Total area polder</td>
<td>$A_p$</td>
<td></td>
<td>3.1 e6</td>
<td>m$^2$</td>
</tr>
<tr>
<td>Depth polder</td>
<td>$d_{\text{inundation}}$</td>
<td></td>
<td>11.5</td>
<td>m</td>
</tr>
<tr>
<td>Total volume polder</td>
<td>$V_{\text{polder}}$</td>
<td>$A_p \cdot d_{\text{inundation}}$</td>
<td>36 e6</td>
<td>m$^3$</td>
</tr>
<tr>
<td>Water speed</td>
<td>$v$</td>
<td>Equation 8-1</td>
<td>1.7</td>
<td>m/s</td>
</tr>
<tr>
<td>Inflow of water in polder</td>
<td>$Q_{\text{in}}$</td>
<td>$Q_{\text{in}} = L_{\text{quay}} \cdot v$</td>
<td>2,210</td>
<td>m$^3$/s</td>
</tr>
<tr>
<td>Flooding time</td>
<td>$T$</td>
<td>$T = V_{\text{polder}} / Q_{\text{in}}$</td>
<td>4.5</td>
<td>hrs</td>
</tr>
</tbody>
</table>

Table 51: Flooding time polder

From the table can be concluded that for the given parameters the polder will be flooded in 4.5 hours. Assuming an average extreme water level (storm surge) lasts for a minimum of 6 hours the assumption that during an event of overtopping the polder will be flooded immediately is correct.

8.4.2. Extreme water level parameters

The extreme water level distribution for Tuas was determined in chapter 3. Parameter A represents a level more or less equal to the Highest Astronomical Tide; higher levels are considered extreme water levels; while parameter B represents expected overtopping height, see Figure 24.

**Parameter A**

The following tables show the influence of parameter A on the required quay wall level, investment and risk for a give polder terminal level (-6.5 meter MSL). Two cases are shown, one with a higher value for A and one lower.
8. Sensitivity analysis

<table>
<thead>
<tr>
<th>Basic case, A=2.9, B=0.15</th>
<th>4.5</th>
<th>1,585</th>
<th>2,223</th>
<th>29 % (638 mln €)</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘High’ case, A=4.0, B=0.15</td>
<td>5.5</td>
<td>1,600</td>
<td>2,311</td>
<td>26 % (611 mln €)</td>
</tr>
<tr>
<td>‘Low’ case, A=1.0, B=0.15</td>
<td>3.8</td>
<td>1,574</td>
<td>2,176</td>
<td>28 % (602 mln €)</td>
</tr>
</tbody>
</table>

Table 52: Investment cost for different parameter A

<table>
<thead>
<tr>
<th>Basic case, A=2.9, B=0.15</th>
<th>4.5</th>
<th>2.1</th>
<th>0.1</th>
<th>21</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘High’ case, A=4.0, B=0.15</td>
<td>5.5</td>
<td>4.6</td>
<td>0.2</td>
<td>23</td>
</tr>
<tr>
<td>‘Low’ case, A=1.0, B=0.15</td>
<td>3.8</td>
<td>0.5</td>
<td>0.1</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 53: Risk for different parameter A

From the tables can be concluded that the parameter A has a large influence on the economic optimal quay wall level, which is obvious because A represents the Highest Astronomical Tide. A high value results in a higher value of the quay wall height and vice versa for lower values of A.

Regarding investments and risk no real difference can be found. The reclamation saving of the polder terminal compared to the conventional terminal remains around 30% regardless of the parameter A. The same holds for the ratio of the risk between the polder terminal and the conventional terminal, which varies around 20. In conclusion the parameter A determines to a large extent the required quay wall height, but has no real influence on the relative amount of reclamation saving or risk.

Parameter B

The following tables show the influence of parameter B on the required quay wall level, investment and risk for a give polder terminal level (-6.5 meter MSL). Two cases are shown, one with a higher value for B and one lower.
8. Sensitivity analysis

<table>
<thead>
<tr>
<th>Basic case, A=2.9, B=0.15</th>
<th>Optimal quay wall level PT [m MSL]</th>
<th>Investment polder terminal [mln €]</th>
<th>Investment conventional terminal [mln €]</th>
<th>Saving in investment polder terminal [mln €]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.5</td>
<td>1,585</td>
<td>2,223</td>
<td>29 % (638 mln €)</td>
</tr>
<tr>
<td>‘High’ case, A=2.9, B=0.30</td>
<td>5.8</td>
<td>1,603</td>
<td>2,330</td>
<td>31% (727 €)</td>
</tr>
<tr>
<td>‘Low’ case, A=2.9, B=0.05</td>
<td>3.8</td>
<td>1,574</td>
<td>2,176</td>
<td>28% (602 mln €)</td>
</tr>
</tbody>
</table>

Table 54: Investment cost for different parameter B

<table>
<thead>
<tr>
<th>Basic case, A=2.9, B=0.15</th>
<th>Optimal quay wall level PT [m MSL]</th>
<th>Risk polder terminal [mln €]</th>
<th>Risk conventional terminal [mln €]</th>
<th>Ratio between risk PT and CT [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.5</td>
<td>2.1</td>
<td>0.1</td>
<td>21</td>
</tr>
<tr>
<td>‘High’ case, A=2.9, B=0.30</td>
<td>5.8</td>
<td>6.8</td>
<td>0.3</td>
<td>23</td>
</tr>
<tr>
<td>‘Low’ case, A=2.9, B=0.05</td>
<td>3.8</td>
<td>0.5</td>
<td>0.1</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 55: Risk for different parameter B

From the tables can be concluded that the parameter B has a similar influence on the economic optimal quay wall level as parameter A. A high value results in a higher value of the quay wall height and vice versa for lower values.

Regarding investments and risk no real difference for higher or lower values of B can be found. The reclamation saving of the polder terminal compared to the conventional terminal remains around 30% regardless of the parameter B. The same holds for the ratio of the risk between the polder terminal and the conventional terminal, which varies around 20.

For parameter B the same conclusion is found as for parameter A which is that this parameter determines to a large extent the required quay wall height, but has no real influence on the relative amount of reclamation saving or risk.

Note: Keep in mind that these results (saving percentage and risk ratio) hold for a polder terminal with a polder level at -6.5 meter MSL. Lower polder depths will result in lower saving percentages and lower risk, as is clear from Table 30 where a polder terminal is investigated with a level at Mean Sea Level.
8. Sensitivity analysis

8.4.3. **Actual probability of inundation**

The ‘actual’ probability of inundation not only depends on the probability of overtopping but also on the probabilities of other failure mechanisms such as seepage (piping, uplifting), instability and/or calamities, see Figure 41. To include these probabilities a failure budget of 10% is added to the overtopping failure probability.

These other failure mechanisms could not only lead to inundation, but also to quay wall breaching. The damage and repair cost of quay wall breaching should therefore also be added to the total damage potential of the polder terminal.

The cost of repair of quay walls after complete failure is related to the construction cost, as explained in section 2.5.4. The total cost of the quay wall breach would then amount to 130% of the construction cost of that same quay wall.

The length of a possible quay wall breach is assumed 200 meter. A quay wall breach will lead to longer down time of the terminal, resulting in higher economical damage. All these costs are taken in to account in the following sensitivity analysis, see Table 56.

<table>
<thead>
<tr>
<th>Design parameter</th>
<th>Explanation</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure probability</td>
<td>10% increase of overtopping failure probability, see equation 5-6</td>
<td>$P_f$</td>
<td>2.87</td>
<td>0.15</td>
</tr>
<tr>
<td>Breach cost</td>
<td>The breach cost is 130% of the initial quay wall cost: 1.3*1,700€/m/m=2,210€/m/m</td>
<td>$D_{breach}$</td>
<td>2,210</td>
<td>€/m/m</td>
</tr>
<tr>
<td>Breach length</td>
<td>The breach length is assumed at 200 meter</td>
<td>$L_{breach}$</td>
<td>200</td>
<td>m</td>
</tr>
<tr>
<td>Breach cost per meter</td>
<td>Multiplication of the breach length with the breach cost: 200*2,210= 440,000€/m</td>
<td>$D_{breach}$</td>
<td>440,000</td>
<td>€/m</td>
</tr>
<tr>
<td>retaining height</td>
<td>Duration of down time due to inundation after a quay wall breach</td>
<td>$t_{flood}$</td>
<td>20</td>
<td>wks</td>
</tr>
</tbody>
</table>

Table 56: Design values for sensitivity analysis of increased failure probability

After including the design values explained in the table the following investment cost, risk and total cost for the polder terminal are found.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{f,inundation} = 1.1*P_{f,overtopping}$</td>
<td>4.5</td>
<td>1,585</td>
<td>0.00002</td>
<td>4,170</td>
<td>2.1</td>
<td>1,587</td>
</tr>
<tr>
<td>$P_{f,inundation} = 1.1*P_{f,overtopping}$</td>
<td>4.5</td>
<td>1,585</td>
<td>0.00002</td>
<td>4,330</td>
<td>2.3</td>
<td>1,587</td>
</tr>
</tbody>
</table>

Table 57: Sensitivity analysis $P_{f,inundation} = 1.1*P_{f,overtopping}$
The increase of the probability of inundation and corresponding damage cost leads to an increase of the risk by 10%. Concluding, the assumption that the probability of inundation in the risk framework approach can be approximated by the probability of overtopping leads to an underestimation of the risk of 10% which is rather small.

8.5. **Feasibility analysis for ‘extreme’ parameters**

The feasibility of the polder terminal is investigated in this paragraph by choosing ‘extreme’ values for the different parameters which are of influence on the investments and risk of the terminal. All parameters used in the analysis have a certain bandwidth (or margin for error); the goal of this analysis is to see whether the polder terminal is still economically attractive if the parameters are chosen such that they will benefit the conventional terminal rather than the polder terminal. A similar analysis was already made in section 5.3.3 where the total cost of the polder terminal and the conventional terminal is compared for different values of the reclamation cost.

8.5.1. **Worst case**

The parameters chosen for the determination of the total cost in chapter 5 were based on prior investigations on the different subjects. In this analysis the values used will be chosen on the boundaries of the bandwidth in order to see whether or not a polder terminal is still an economically attractive alternative for a conventional terminal. The values used in this analysis are explained in Table 58.

<table>
<thead>
<tr>
<th>Design parameter</th>
<th>Explanation</th>
<th>Variable</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quay wall cost [per m retaining height per running meter]</td>
<td>The quay wall cost determined have a bandwidth of 25%, so the maximum quay wall cost is 1.25 * 1,700 = 2,125 €/m/m or 64,000 €/m for retaining height of 30 meter.</td>
<td>$i_q$</td>
<td>2,125</td>
<td>€/m/m</td>
</tr>
<tr>
<td>Reclamation cost</td>
<td>This parameter was already discussed in section 5.3.3; the average value is used.</td>
<td>$i_p$</td>
<td>20</td>
<td>€/m$^3$</td>
</tr>
<tr>
<td>Exponential parameter</td>
<td>The influence of these parameters was discussed in paragraph 8.4.</td>
<td>A</td>
<td>2.87</td>
<td>-</td>
</tr>
<tr>
<td>Exponential parameter</td>
<td></td>
<td>B</td>
<td>0.15</td>
<td>-</td>
</tr>
<tr>
<td>Failure probability</td>
<td>The failure probability for overtopping is expressed in equation 5-6</td>
<td>$P_f = 1.1 * e^{-0.15}$</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Direct damage container terminal</td>
<td>The total potential damage due to inundation originates from the study of Tebodin 1998 and calculations of the 2nd Maasvlakte, an extreme value of 1,000 €/m$^2$ is used for this analysis to see what the influence is.</td>
<td>$dc$</td>
<td>1,000</td>
<td>€/m$^2$</td>
</tr>
</tbody>
</table>
8. Sensitivity analysis

| Linear coefficient damage function | The linear coefficient originates from the assumption that 100% damage potential occurs at an inundation depth of 5m. | di | 0.2 | m³ |
| Direct damage ‘some water on terminal’ | The direct damage during ‘some flooding’ is related to the damage for an inundation depth of 0.5 m. | D₀ | 3.12 e08 | € |
| Indirect damage due to down time | The indirect damage due to down time is taken maximal, assuming the port operator has maximum economic losses due to a flood (no relocation of port operations possible as was assumed in chapter 5). | dt | 15.4 e06 | €/wk |
| Down time during a flood | The down time of the terminal is approximated at one year, resulting from a flood due to a quay wall breach. | t_down | 52 | wk |
| Reduced interest rate | The reduced interest rate is assumed at 5%. | r’ | 0.05 | - |

**Table 58: ‘Extreme’ design values**

*Note: The dimensions of the polder terminal are kept the same as in Table 24. The damage cost of a possible quay wall breach are also taken in to account, as well as the investment cost and risk of the water management system designed in chapter 7.*

The investigation will be made for two cases, one with a polder level equal to Mean Sea Level, which was used in chapter 5, and one with the maximum polder depth (corresponding with maximum reclamation savings), which is at the uplifting boundary: -6.5 meter Mean Sea Level

**Polder level at Mean Sea Level**

The results for a polder level at Mean Sea Level are summarized in Table 59 and Figure 79. For comparison purposes a conventional terminal is shown with a quay wall (terminal) height equal to the economic optimal quay wall height of the polder terminal.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional terminal</td>
<td>4.5</td>
<td>2,340</td>
<td>0.00002</td>
<td>1,140</td>
<td>0.2</td>
</tr>
<tr>
<td>Polder terminal</td>
<td>4.5</td>
<td>2,101</td>
<td>0.00002</td>
<td>3,950</td>
<td>2.1</td>
</tr>
</tbody>
</table>

**Table 59: Total cost of extreme case with polder level at Mean Sea Level**

The economic optimal quay wall height for the polder terminal lies at +4.5 meter MSL, when taking sea level rise and settlement in to account the construction height of the quay wall is +5.1 meter MSL which coincides with +6.6 meter CD as was already determined.
Figure 79: Extreme case sensitivity analysis for polder level at Mean Sea Level

From the results can be concluded that even with ‘extreme’ parameters a saving of about 14 % in investment (331 mln €) is possible given a polder level at MSL, compared to the conventional terminal. The risk for the polder terminal is about a factor 10 larger than the risk of the conventional terminal.

**Polder level at uplifting boundary: -6.5 meter MSL**

The results for a polder level at the uplifting boundary are summarized in Table 60 and Figure 80. The economic optimal quay wall height is the same as for a polder level at Mean Sea Level.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional terminal</td>
<td>4.5</td>
<td>2,340</td>
<td>0.00002</td>
<td>1,140</td>
<td>2,341</td>
</tr>
<tr>
<td>Polder terminal</td>
<td>4.5</td>
<td>1,695</td>
<td>0.00002</td>
<td>8,010</td>
<td>1,699</td>
</tr>
</tbody>
</table>

Table 60: Total cost of extreme case with polder level at uplifting boundary (-6.5m MSL)
8. Sensitivity analysis

Figure 80: Extreme case sensitivity analysis for polder level uplifting boundary

From the results can be concluded that now a larger saving of about 27 % in investment (640 mln €) is possible given a polder level of -6.5 meter MSL, compared to the conventional terminal. The risk for the polder terminal is however about a factor 20 larger than the risk of the conventional terminal. These results coincide with those found in paragraph 8.2 for average parameters.

Conclusion

The results, after including ‘extreme’ values for the parameters, do not have a large impact on the relative cost saving or risk of the polder terminal when compared to the results with average parameters (paragraph 5.3). The polder terminal is thus still an economically attractive alternative to a conventional terminal.

8.5.2. Worst case + mitigation

Port operators are hesitant when offered a polder terminal instead of a conventional terminal because of the flood risk, which as is determined in the previous analyses is actually not that high. However, the operators could still want to apply risk mitigating measures or insure the flood risk. The required insurance premium could amount to 2 to 5 times the present value of the risk, based on a rough estimate.

Risk mitigation measures could consist of an accurate system to predict whether or not inundation is expected given a certain storm, if so actions could be taken to limit the damage potential inside the polder terminal. Other risk mitigating measures could be structures to prevent ship collision, a flood wall to prevent a flood during extreme water levels etc. These methods will lower the probability of inundation and the risk; however it is hard to quantify the influence of these measures on the total cost.

If the port operator chooses to insure the risk, the total cost will increase with the risk premium. This will lower the amount of savings possible. In the most extreme case an insurer will ask for a premium of 5 times
the present value of the risk of inundation, the resulting total cost and possible saving compared to a conventional terminal is estimated in the following table (for a polder level equal to Mean Sea Level and the uplifting boundary).

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional terminal</td>
<td>4.75</td>
<td>2,360</td>
<td>0.00001</td>
<td>1,150</td>
<td>0.1</td>
<td>0.5</td>
</tr>
<tr>
<td>Polder level at 0m MSL</td>
<td>4.75</td>
<td>2,101</td>
<td>0.00001</td>
<td>3,310</td>
<td>0.8</td>
<td>4</td>
</tr>
<tr>
<td>Polder level at -6.5m MSL</td>
<td>4.75</td>
<td>1,696</td>
<td>0.00001</td>
<td>8,170</td>
<td>1.1</td>
<td>5.5</td>
</tr>
</tbody>
</table>

Table 61: Total cost of extreme cases including insurance premium.

Due to the contribution of the risk premium the optimal quay wall height for the polder terminal is higher than in the last case resulting in a low risk of inundation due to the lower probability of inundation. From the table can be concluded that, including a maximum risk premium of 5 times the present value of the risk, the saving of a polder terminal with a level at Mean Sea Level is still about 11% which is a small decrease compared to the situation where no risk premium is taken in to account.

For the deeper polder with a level at -6.5 meter MSL the saving is 27%, which is exactly the same. Thus it can be concluded that when choosing ‘extreme’ values for all parameters and including a maximum risk premium large savings are possible when constructing a polder terminal instead of a conventional terminal.

### 8.6. Conclusions and recommendations

This chapter investigated the influence of the assumptions discussed in paragraph 5.4, except for the influence of settlement. For settlement it is expected that the final result for the polder terminal will benefit from including soil improvement cost, because a polder terminal required less soil improvement works than a conventional terminal (lower amount of fill material).

#### 8.6.1. Polder depth

Chapter 6 investigated the maximum polder depth of a polder terminal and concluded that the polder depth was bounded by the uplifting failure mechanism. For the Tuas case this level lies at -6.5m MSL, the total cost for this level was estimated and resulted in a cost saving of about 30% compared to a conventional terminal, which is a significant increase compared to the saving of 10% for a polder with a terminal level at Mean Sea Level.
8. Sensitivity analysis

The ratio of the risk is higher though, for the maximum polder depth the risk is a factor 20 larger than the risk of the conventional terminal (compared to a factor 10 for a polder depth at Mean Sea Level). This is partly explained by the fact that the risk of the water management system is included in the sensitivity analyses and by the fact that deeper polders have larger damage potentials.

8.6.2. Quay wall cost

The influence of the quay wall cost on the total cost was also investigated, resulting from the conclusion of chapter 6 which stated that the quay wall cost of a polder terminal will be lower than that of a conventional terminal. The influence of a reduction of the quay wall cost of 25% resulted in a saving in the total cost of 5% which is not considered significant because the estimate has a certain margin for error.

Another assumption regarding quay wall cost stated that the cost was linearly related to the retaining height. After including the actual increasing nonlinear relation a final difference was found of 33%. This would result in an additional saving of 6% compared to the original case estimated in chapter 5, which is still rather insignificant.

Finally it can be concluded that the assumptions, made in chapter 5, regarding quay wall cost result in a small overestimation of the actual total cost, which benefits the polder terminal.

8.6.3. Probability of inundation

It is concluded that for the given parameters regarding the extreme water level distribution the polder will be flooded in 4.5 hours. Assuming an average extreme water level (storm surge) lasts for a minimum of 6 hours the assumption that during an event of overtopping the polder will be flooded immediately is correct.

The probability of inundation depends to a large extent on the parameters A and B of the extreme water level distribution. These have a large influence on the economic optimal quay wall heights (higher values result in higher quay wall heights). However, the influence on the amount of cost saving (30%) and risk ratio (20) proved to be small.

The assumption that the probability of inundation is determined by the probability of overtopping is considered correct. The influence of including a failure budget of 10% together with the cost of repair of a quay wall breach on the total risk of the polder terminal (and thus the total cost) results in an underestimation of the risk by 10% which was expected.
8. Sensitivity analysis

8.6.4. Feasibility in ‘extreme parameters’

Finally the investment, risk and total cost were re-calculated using ‘extreme’ values for the different parameters in the total cost function (Table 58), instead of the average values (Table 24).

The resulting investment cost, risk and total cost are determined for a terminal with a polder level at Mean Sea Level and a terminal with a polder level at the uplifting boundary (-6.5 meter MSL). The estimates proved that the polder terminal is still an attractive alternative for a conventional terminal. The increase in total cost proved to be small.

Port operators will still be hesitant when confronted with a polder terminal because of the larger flood risk compared to a conventional terminal. Risk mitigation measures are possible, which will lower the risk of flooding. Another option is to insure the flood risk against an insurance premium which could vary between 2 to 5 times the values of the risk.

This insurance premium will increase the total cost of the polder terminal (and decrease the reclamation saving), but even after including an insurance premium of 5 times the present value of the risk the polder terminal proved to save a significant amount of money compared to the conventional terminal.

8.6.5. Conclusion

Summarizing one can conclude that the polder terminal remains an attractive alternative for a conventional terminal, in the sensitivity analyses it is proved that the assumptions made in chapter 5 are acceptable and that the reclamation savings of the polder terminal remain rather large, especially for deeper polders.
9. Conclusions and recommendations

9.1. Introduction

This chapter discusses the main result found in the investigation of this master thesis. Each chapter of this report concluded with a paragraph explaining the different conclusions and recommendations regarding the subject treated. The goal is to provide feedback on the different research questions as defined in the introduction which together form the main conclusion of this report. The project aim is repeated below, as well as the research questions.

This master thesis will aim at making a risk based design of a polder terminal which is used to investigate the feasibility of the polder terminal in comparison with the conventional terminal.

The following research questions are addressed:

- What are the consequences of making a polder terminal, in comparison to a conventional terminal?
- What are the risks involved when making a polder terminal?
- What levels (height) should be chosen for the quay wall and terminal yard of the polder terminal?
- What quay wall flood defense system is most suitable for a polder container terminal?
- How to deal with water storage and drainage inside the polder container terminal?

9.2. Conclusions

First the main conclusion of this report is given after which the answers to the research questions will be addressed in the sub conclusions. The cost estimates are made based on a conceptual design of a polder terminal at Tuas Singapore, which is considered a generally applicable case study for a polder terminal.

9.2.1. Main conclusion

The polder terminal is particularly feasible at locations with high reclamation cost (including soil improvement cost); low conductive (impermeable) sub soil is required to limit the amount of seepage in the polder. These layers could already be present or made in the fill. A lower terminal yard of the polder terminal results in a reduction of the amount of fill material required and a reduction of the reclamation cost. However, the lower terminal yard also results in an increase of the risk of inundation, due to the higher damage potential during a flood. A water drainage system is required to drain excess water in the polder.
The reduction of the reclamation cost, due to the smaller fill, proved to be larger than the increased risk of inundation and required water drainage cost of the polder terminal. The resulting total cost (consisting of the summation of investments and risk) of the polder terminal is significantly lower than the total cost of the conventional terminal, proving that the polder terminal is an attractive alternative for a conventional terminal. The magnitude of the reclamation saving depends on the polder terminal depth; deeper polder result in larger savings. The polder depth is bounded by the uplifting failure mechanism.

It should be noted that these results largely depend on the reclamation cost, which at Tuas Singapore is 20 €/m$^3$. For low reclamation cost (i.e. 1 €/m$^3$) a conventional terminal proved to be the better solution. The percentage of reclamation saving as well as the risk ratio is independent of the total polder area, because both the reclamation cost and the damage cost depend on the total polder area.

9.2.2. Sub conclusions

Risks of the polder terminal
The risk of inundation of the polder terminal is divided in two main events; ‘small scale flooding’ and ‘large scale flooding’. Small scale flooding is dealt with by designing a water drainage system with sufficient capacity to drain the maximum rainfall intensities (neglecting the inflow of seepage and overtopping). Large scale flooding is dealt with by designing a quay wall flood defense with sufficiently high crest height, which limits the amount of overtopping. A failure budget of 10% is added to include the failure probabilities of seepage, instability and calamities, which are designed through a probabilistic approach.

Quay wall and terminal yard levels
A quay wall height is found where the increased risk of inundation of a polder terminal is equal to the reclamation cost required for a conventional terminal: the transitional quay wall height. Polder terminals are attractive from quay wall heights higher than the transitional level; in that case the reclamation saving is higher than the increased risk of inundation.

The economic optimal quay wall height for the polder terminal (which is higher than the transitional quay wall height) is higher than that of the conventional terminal. However, given the aforementioned conditions, the economic optimal total cost of the polder terminal are significantly lower than that of the conventional terminal, making the polder terminal the better solution, as shown in Figure 81.

Note: The cost estimated in Figure 81 was made for a reclamation cost of 20 €/m$^3$ and a polder level of -6.5 meter MSL which is the uplifting boundary condition (reference is made to paragraph 8.5).
9. Conclusions and recommendations

Figure 81: Comparison of total cost polder terminal vs conventional terminal (Reclamation = 20 €/m³)

The reclamation cost saving largely depends on the depth of the polder terminal yard; maximum savings are possible for the maximum depths of the polder terminal yard. The savings at Tuas, Singapore, vary between 10% (220 mln euro) for a terminal yard at Mean Sea Level and 30% (660 mln euro) for a terminal yard at the uplifting boundary (-6.5 meter MSL), which is the maximum possible polder depth.

The risk of the polder terminal (3 mln euro) proved to be 10 times larger than the risk of the conventional terminal (0.3 mln euro). This increase was insufficient to counteract the large reclamation saving, even if multiplied by 5 which coincides with a flood risk insurance premium (which is required when port operators want to insure the flood risk). The resulting total cost per square meter are summarized in Table 62.

<table>
<thead>
<tr>
<th>Quay wall level [m MSL]</th>
<th>Terminal level [m MSL]</th>
<th>Total cost [mln €]</th>
<th>Total cost [€/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional terminal</td>
<td>+5</td>
<td>+5</td>
<td>2,223</td>
</tr>
<tr>
<td>Polder terminal MSL</td>
<td>+5</td>
<td>0</td>
<td>1,950</td>
</tr>
<tr>
<td>Polder terminal -6.5 MSL</td>
<td>+5</td>
<td>-6.5</td>
<td>1,587</td>
</tr>
</tbody>
</table>

Table 62: Total cost of container terminals at Tuas, Singapore

Quay wall flood defense system

The quay wall of a polder terminal does not only ‘traditionally’ retain soil and water but also serves as a primary flood defense structure for the polder terminal yard. There is not one particular structure which is best suitable as a quay wall flood defense; this depends on local boundary conditions. Possible structures are a gravity structure (caissons or silos), sheet pile walls (cofferdam) or a pile supported platform in combination with a water retaining structure (retaining wall or embankment).
The quay wall flood defense structure of a polder terminal can be made lighter than that of the conventional terminal, which results in lower quay wall cost. This is mainly the result of the lower retaining height of the polder terminal quay wall due to the lower polder terminal yard.

Gravity structures are investigated in more detail. For gravity structures the depth of the polder terminal yard is bounded by failure mechanisms of piping, uplifting and structure instability. Uplifting produces the definitive boundary; piping and structural instability can be dealt with in the design of the structure. The design area is illustrated in Figure 82, which shows the relation between the required weight (width) of the caisson structure and the polder depth. It is assumed the same conclusions hold for a cofferdam sheet pile structure.

![Figure 82: Requirements for quay wall flood defense structure related to polder depth](image)

The difference in quay wall cost of the polder terminal and conventional terminal could amount to 25%. The influence of this reduction on the total cost of the terminal is relatively small (5%) due to the small contribution of the quay wall cost in the total investment cost of such a terminal.

**Water storage and drainage system**

Small scale flooding is related to a temporary situation of water hindrance in the terminal yard (water depths under 0.5 meter). It is avoided by designing a water drainage system with sufficient capacity to drain the maximum rainfall intensities expected.

The increase in cost of the polder terminal due to the water drainage system is negligible compared to the large reclamation saving possible. Further, the increase in investment required to minimize the risk of small
scale flooding is also small. It is therefore advised to design a water drainage system which minimizes the risk of small scale flooding rather than optimizes the total cost of the water drainage system.

9.3. Recommendations

The recommendations are divided in general recommendations, recommendations regarding the risk framework approach and those regarding the quay wall flood defense structure.

9.3.1. General recommendations

When designing a new container terminal the chosen terminal levels should not only based on minimal total cost but also take the return period of inundation and the risks involved in to account. The decision whether or not a design should be based on minimal total cost, a certain level of safety or an acceptable risk is one of a political nature.

The feasibility of the polder terminal is investigated for container terminals. Other types of terminals, such as dry or liquid bulk terminals and car terminals could also be designed as a polder terminal. More information is treated in appendix C. It is recommended to investigate the risk of a polder terminal for these functions to determine if the polder terminal is also an attractive alternative there.

All possible locations where polder terminals could be attractive could be shown on a map of the world, providing Royal HaskoningDHV with a fast way to determine whether or not a polder terminal could be attractive for a given project. A start was made in the report made by M. Smits [XIX], which was based on the requirement of new port constructions, local subsoil and expected reclamation cost. Using the conclusions of this report more detailed research on this subject could be made.

Further, the risk framework approach of the quay wall height and polder terminal yard shows similarities to a dike and terp level model. The risk framework approach could therefore also be used on a wider scale to determine the optimal levels and total cost of polders in general (not only polder container terminals).

9.3.2. Recommendations of risk framework approach

Regarding the risk framework approach different assumptions where made to simplify the calculation method, for detailed cost estimates the actual relations should be taken in to account. These are mentioned below:

- Include the actual non linear relation between the retaining height and the quay wall cost, it is expected this will not have a large influence on the final result (see chapter 8);
Include the actual soil improvement cost in the reclamation cost of the polder terminal, as these could influence the cost estimates of both the polder and conventional terminal;

Include the influence of possible settlements and consolidation time in the terminal yard, which has an influence on the required reclamation fill;

Investigate the influence of the duration of the flood. It was assumed that during overtopping the polder is flooded immediately, not taking time in to account (see Figure 46);

Include non monetary damage in the damage function such as loss of life or the reliability of the port terminal. In the current approach only direct damage cost to the terminal and financial damage was taken in to account. Another aspect which was not treated further are possible risk mitigation measures, which could to a large extent influence the actual risk of the polder terminal;

Include the influence of the level difference between quay wall and terminal yard on the port logistics. It was assumed that the turnover times of the port do not change considerably due to the lower terminal yard, this assumption requires verification.

Chapter 8 investigated the influence of these assumptions and concluded that the final cost estimates do not change significantly proving that the simplifications are correct. Every mathematical model has a certain margin for error, which depends on the margin of error of the parameters used in the model. The exact margin for error (bandwidth) of the results of the risk framework approach requires further investigation.

9.3.3. Recommendations of quay wall flood defense structure

The influence of a variable terminal yard level on the structural stability was determined for gravity structures, not for sheet pile walls (cofferdams) and pile supported platforms. The exact influence of the variable terminal depth on sheet pile walls (cofferdams) and pile supported platforms as a quay wall flood defense structure of a polder terminal requires further investigation.

The exact influence of the calamities ship collision and earthquakes on the quay wall flood defense structure of a polder terminal requires further investigation, especially regarding the displacement of the structure. Large deformations of the quay wall flood defense due to ship collision or earthquakes could cause leakage of the flood defense structure which result in flooding of the polder terminal.

Royal HaskoningDHV proposed a design of a quay wall flood defense where the water storage system was incorporated in the structure, which would save space in the terminal. Investigation on the consequences for the quay wall cost and risk of flooding is required when such a structure is designed for the polder terminal. Moreover, the water storage and drainage system is designed based on rules of thumb. Detailed calculations are required to verify the results found in chapter 7, see appendix D.
10. Literature

10.1. Literature

II. Boer, S, *Veiligheid tegen overstoring van Maasvlakte 2*, Rotterdam: Projectorganisatie Maasvlakte 2, 2005
III. Bonte, P., *Structural design of a sandwich wall as the quay wall for the future*, Delft: TU Delft, 2007
XVII. Royal HaskoningDHV, *Multi-disciplinary Civil & Structural Engineering, Quantity Surveying and Project Management Consultancy Services for Coastal Development at Tuas*, Rotterdam: Royal HaskoningDHV, 2011.
10. Literature

10.2. Websites


10.3. Programs

a) *AutoCAD 2010*, Autodesk, 2009

b) *Google Sketchup Pro 8.0.4811*, Google, 2010
Appendices

A Personal evaluation

A.1 Start project: goals

In addition to the graduation project related issues attention will be paid to issues on a more personal level. During the six month graduation internship I would like to have feedback on myself as an employee of Royal HaskoningDHV in order to keep learning and improving my personal competencies. I am good at prioritizing and planning the work which has to be done and keeping this planning. There are certain aspects of which I know I could improve on, which are summed up below:

1. When writing a report I tend to write in too much detail. I need to be more concise in writing a report.

2. I also tend to investigate certain subjects in too much detail when a more general approach is possible; these are things I could improve on.

3. Keeping everybody informed of my work / progress: I have the tendency to work very independent, without involving my colleagues, I am planning on giving lunch presentations of the work I have been doing in order to keep everyone informed and get feedback on aspects concerning this project;

4. Because I have the tendency to work very independent I often try to solve possible questions myself instead of simply asking for a solution. When actually asking for a solution often times I already provide an answer without giving those whom I asked the time to answer themselves, this is a result partially of a lack of patience and the drive to try to solve all questions independently.
A.2 Final evaluation

The graduation internship went very well in my opinion. I kept to my planning which resulted in no delays during the full extent of the graduation work. Of course this is also the result of the feedback and guidance of the graduation committee, who were always ready to answer whatever questions I had regarding the subject. About the aspects mentioned in the last section A.1:

1. The final report is rather long, but each chapter contains relevant new information in my opinion.

2. During the extent of the project there is a big difference between the conceptual design and the case specific design for Tuas. In the beginning I had some difficulties making a clear distinction between both designs; I had the tendency to go too much into details of the case specific design for Tuas rather than the conceptual design. Later on this proved to be less of a problem.

3. Keeping everybody informed went very well, I had regular meetings with members of the graduation committee and I also gave lunch presentations to the whole staff of the Maritime division of Royal HaskoningDHV in Rotterdam.

4. Again, in the beginning I had the tendency to go too much into details of the case specific design for Tuas, which was also the result of trying to solve possible questions independently. After a while I involved my graduation committee more often (S.N. Jonkman specifically) which proved to be very useful in the remainder of the whole project.

All in all I am very pleased with the end result and found the subject very interesting. That is also why I would like to continue doing research on this subject, possibly publishing a paper on the economic flood risk approach made in chapter 5.
B Short history of polders

The Netherlands have been fighting against water for centuries [I]. The first inhabitants of the country were fishermen and hunters living in the deltaic areas. During periods with high water they would move to higher grounds. The Romans were the first to build dikes along rivers around the 11th century; the main function was not to retain water but to build roads which would always stay dry. Naturally dikes are nowadays used to protect polders from flooding.

The first polders were formed in the Dutch provinces of Groningen, Friesland and Zeeland. These polders were protected from outside water through dikes. Around the 13th century the Dutch started damming of creeks, tidal channels and rivers through the country (the Rotterdam and the Amsterdam), this created large polders which were fundamentally different than the first ‘sea’ polders. They did not only provide adequate protection against flooding, they also provided a fresh water source. A new element within the polder was the ‘Boezem’ which is a belt canal system not connected to the outside (salt) water, nor was it connected to the polder water, as illustrated in Figure 83.

![Figure 83: Cross section [IX]](image)

The polder water was drained on to the Boezem and the Boezem water was drained to sea, thus keeping salt water out of the polder. The Boezems could supply the polders with fresh water during draughts. At the end of the 13th century windmills were introduced for drainage purposes of the polders.

![Figure 84: Traditional Dutch windmill](image)

Now even larger areas (whole lakes) could be pumped dry to create land. First a dike was built around the area after which it was pumped dry. A large part of the Netherlands still consists of polders (about 60%); nowadays pumping stations have taken over the drainage function of windmills.
0C Other polder terminal applications

It was explained that research made on polder terminals in this report considers container terminals. Polder terminals are also possible for other terminal functions, which is pointed out in the following:

**C.1 Car terminals**

Polder terminals are also possible for car terminals. In this case more ramps between the quay wall and terminal yard will be required to overcome the height difference between the quay wall and the polder terminal yard. These could easily be integrated in a typical terminal lay out for car terminals.

The damage potential of such a terminal is expected to be higher than that of a container terminal; the cars will be completely lost during a flood. Correspondingly the risk of the car terminal will be higher than that of the polder terminal.

**C.2 Dry bulk terminals**

Dry bulk terminals are also possible to be constructed as a polder terminal: conveyer belts are used to transport the bulk material between the vessels and the terminal yard. These conveyer belts can easily overcome the height difference between the quay wall and polder terminal yard, making a polder terminal very attractive for this type of terminal. In this case a dike could be used as a flood defense, with jetties to facilitate vessel berthing.

Dry bulk material will hardly damage or loose its value during a flood, resulting in a lower damage potential and lower risk compared to a container polder terminal.

**C.3 Liquid bulk terminals**

Liquid bulk terminals could also be made as polder terminals. In this case also a dike could be built as a flood defense with a jetty type structure to facilitate vessel berthing. The pipelines required transporting the liquid bulk cargo between the vessels and terminal yard can easily overcome the level difference between the flood defense and the polder terminal yard, which is the case in DOW Chemical in Terneuzen; see Figure 13.

Regarding damage potential the same conclusion can be made as with dry bulk terminals. The liquid bulk cargo is pumped in storage tanks which will not damage during a flood (if designed strong enough) resulting in little or no damage to the cargo. The risk of the liquid bulk terminal will therefore be lower than that of the container terminal.
D Water management optimization

Inundation due to rainfall is prevented by two parameters, the drainage capacity and the storage capacity. To determine the optimal combination of both the total cost needs to be determined. The cost of drainage also depends on the polder level, which introduces a third variable to be optimized. If the polder level is chosen above the Lowest Astronomical Tide drainage through gravity flow is possible, whereas for a polder level below LAT pumps are required.

![Figure 85: Optimization variables water management system](image)

This appendix describes an economic approach to determine the optimal capacities corresponding with minimal total cost, a similar approach as made in chapter 5. Simultaneously optimizing three variables is difficult, so a deterministic polder level is chosen based on the conclusions made in chapter 6.

It is determined how the costs (investments) increase with drainage capacity and storage capacity. The probability of inundation decreases with increased drainage and/or storage capacity and correspondingly the risk (probability x damage). This is called the goal function, and is expressed in equation 5-1. The decision variables (variables which determine the total costs) consist of the drainage capacity \( Q_{pump} \) and the storage capacity \( V_{storage} \).

D.1 Investment cost

The pump costs \( I_{pump} \) are dependant on the drainage capacity \( Q_{pump} \); the storage costs \( I_{storage} \) are dependant on the storage volume \( V_{storage} \), as shown in equation D-1.

\[
I_{tot} = I_{pump} * Q_{pump} + I_{storage} * V_{storage} \tag{D-1}
\]
D.2 Risk

The present value of the risk is defined as the multiplication of the probability of inundation with the damage cost, divided by the interest rate.

D.2.1 Probability of inundation

Storage capacity is required when the rainfall intensity exceeds the drainage capacity. Inundation will occur when the storage capacity of the polder is exceeded and water will be stored on the terminal. This could have two causes, the first being insufficient drainage capacity (for a rainfall intensity higher than the critical rainfall) and second during pump failure. The probability of inundation will then be the summation of the probability of a rainfall intensity exceeding the drainage capacity and the probability of pump failure when the storage capacity is exceeded, as expressed in equation D-2.

\[ P_{\text{i; inundation}} = P_f (N > Q_{\text{pump}}) + P_f (N > V_{\text{storage}}) * P_{f ; \text{ pump}} \]  \hspace{1cm} (D-2)

To determine the probability of inundation a statistical analysis is required of the rainfall intensities as well as the pumps.

D.2.2 Damage cost

Damage due to a flood is similar to the damage as defined in paragraph 5.2.1; it is expressed in equation D-3, implementing equations 7-2 till 7-5 one obtains equation D-4. The first term expresses the direct damages to containers and facilities and the second term expresses the indirect damage due to down time (which depends on the time required to drain the excess water from the terminal yard).

Direct damage to containers and facilities depends on inundation depth. The inundation depth is the difference of the total amount of inflow during a storm (N) with the total amount of drainage capacity \((Q_{\text{pump}})\) and the total amount of storage capacity \((V_{\text{storage}})\), divided by the polder area \((A_{\text{polder}})\).

\[ D = D_f \frac{V_{\text{terminal}}}{A_p} + D_f * t_{\text{down}} \hspace{1cm} (D-3) \]

\[ = D_f \frac{(t_{\text{storm}} (N * A_p - Q_{\text{pump}}) - V_{\text{storage}})}{A_{\text{polder}}} + D_f * t_{\text{down}} \hspace{1cm} (D-4) \]

D.2.3 Risk

The present value of the risk can now be found by the multiplication of the probability of inundation with the damage divided by the discounted value of the rent as expressed in equations D-4 and D-5.
Appendices

\[
R = \frac{P_{\text{inundation}} \cdot D_i \cdot V_{\text{min alt}} + D_i}{A_p} \quad (D-4)
\]

\[
R = \frac{\left[ P_f (N > Q_{\text{pump}}) + P_f (N > V_{\text{storage}}) \cdot P_{f, \text{pump}} \right] \cdot D_i \cdot \left( t_{\text{storm}} \cdot (N \cdot A_p - Q_{\text{pump}}) - V_{\text{storage}} \right) + D_i \cdot t_{\text{down}}}{A_p} \quad (D-5)
\]

**D.3 Total cost**

The total cost can now be expressed in the summation of the investment and the risk, see equation D-6.

\[
C_{\text{tot}} = I_{\text{pump}} + I_{\text{storage}} + \frac{\left[ P_f (N > Q_{\text{pump}}) + P_f (N > V_{\text{storage}}) \cdot P_{f, \text{pump}} \right] \cdot D_i \cdot \left( t_{\text{storm}} \cdot (N \cdot A_p - Q_{\text{pump}}) - V_{\text{storage}} \right) + D_i \cdot t_{\text{down}}}{A_p} \quad (D-6)
\]

*Note: A continuous formulation of the probability of inundation and corresponding damages is difficult to determine because the events are independent (pump failure, storm intensities and duration).*

**D.4 Optimization**

In order to determine the minimal total costs the derivatives of the goal function to the decision variables need to be determined. The optimal drainage and storage capacities are then found by setting these derivatives equal to zero.

Different functions in the economical approach require further investigation, but the method proposed serves as a start to determine the optimal drainage and storage capacity for a deterministic polder level. The optimization will not be treated further in this master thesis; instead an alternate method based on ‘rules of thumb’ to determine the drainage and storage capacity is used in chapter 7.
E Visualizations: drawings

This appendix contains two drawings:

1. Conceptual design 1 and 2, which contains the sketches of the conceptual design of a polder terminal.

2. Calculation drawings, which contains the drawings which serve as an explanation of the calculations made in this report.
soil profile of the terminal

situation sketch
F Thesis proposal

Thesis proposal

Kasper Lendering
April 25th, 2012
CIE 5060

Graduation committee
Prof. Drs. ir. J.K. Vrijling  TU Delft
Prof. Dr.ir. S.N. Jonkman  TU Delft
Dr. Ir. J.G. de Gijt  TU Delft
Dr. Ir. O.A.C. Hoes  TU Delft
Ir. D.J. Peters  RHDHV
F.1 Introduction

The following report contains the thesis proposal for a graduation project performed at Royal HaskoningDHV on the ‘Polder Terminal’. This master thesis is written within the Master Hydraulic Engineering, in the field of ‘Hydraulic structures and Flood risk’.

F.1.1 Thesis committee

The thesis committee is responsible for the guidance and evaluation of the student during the graduation project. This committee is lead by Prof. drs. ir. J.K. Vrijling of the department of hydraulic engineering at the Technical University of Delft. Assisting him will be Prof. Dr. Ir. S.N. Jonkman and dr. ir J.G. de Gijt of the same department of hydraulic engineering and finally dr. ir. O.A.C. Hoes of the department of water management.

Because the project is carried out outside of the university, at Royal HaskoningDHV, the committee is joined by a member of the RHDHV: ir. D.J. Peters, who works for the structural department of the Maritime and Waterways group.

<table>
<thead>
<tr>
<th>Member</th>
<th>Role</th>
<th>Specialization</th>
<th>Company</th>
</tr>
</thead>
<tbody>
<tr>
<td>J.K. Vrijling</td>
<td>Chairman</td>
<td>Hydraulic structures</td>
<td>TU Delft</td>
</tr>
<tr>
<td>S.N. Jonkman</td>
<td>Day to day supervisor</td>
<td>Flood risk analysis</td>
<td>TU Delft</td>
</tr>
<tr>
<td>J.G. de Gijt</td>
<td>Member</td>
<td>Quay wall structures</td>
<td>TU Delft</td>
</tr>
<tr>
<td>O.A.C. Hoes</td>
<td>Member</td>
<td>Water management</td>
<td>TU Delft</td>
</tr>
<tr>
<td>D.J. Peters</td>
<td>Supervisor</td>
<td>Hydraulic structures</td>
<td>Royal HaskoningDHV</td>
</tr>
</tbody>
</table>

Table 63: Thesis committee

F.1.2 Report structure

This report contains a project description in chapter 2, followed by a work plan in chapter 3. Chapter 4 focuses on the project planning and chapter 5 elaborates on aspects concerning the student’s personal evaluations. Finally a list of literature, tables and figures is shown in chapter 6.
F.2 Project description

F.2.1 Introduction

New ports are mostly constructed on low lying coastal areas or in shallow coastal waters. The surface level of the land for port facilities is then raised to a level well above the design water level. This requires large volumes of good quality fill material often dredged from the sea. Due to dredging, there is a significant environmental impact in the area.

Container terminal operators generally demand terminals which are well above the design water level. Is this justified or is there a way of constructing safe terminals with smaller volumes of dredged material thus lowering costs and the environmental impact?

Royal HaskoningDHV has developed the concept of a container terminal with a “polder yard”. The yard would lie below high water level and would be surrounded by quay walls and possibly embankments. These structures will keep the water out. The quay walls will have an apron for vessel berthing and crane operation. As with a conventional polder, a Polder Terminal requires storage capacity for rain and seepage water plus a system for discharging water.

Figure 86: Cross section Polder Terminal [Royal HaskoningDHV]
Figure 4 contains a cross section of the general idea of the Polder Terminal. It shows a terminal yard at Mean Sea Level surrounded by quay walls which provide the necessary safety against design water levels (to keep the water out).

**F.2.2 Preliminary study**

Royal HaskoningDHV made a preliminary study on the applicability of a Polder Terminal. This study proved that a Polder Terminal can be feasible: in comparison with a ‘conventional terminal’ (with port facilities at a level well above the design water level) the following can be concluded:

- Normal operation of the terminal do not have to be affected by the lower polder yard;
- A polder terminal can have lower construction cost due to the reduction of reclamation material;
- The Polder Terminal is particularly feasible for larger terminals, at locations with bad sub soils and high prices for reclamation material;
- The polder induces additional flood risks.

Based on a quick scan of the market of container ports around the world and considering the characteristics of making a Polder Terminal more feasible, three area’s for possible business development were identified; i.e. South-East Asia, India and Brazil. In these areas there is a large demand for new ports, subsoil conditions are bad and good quality fill material is scarce [I].

**Recommendations**

The study provided certain aspects which require further investigation with regard to the Polder Terminal:

- The risk of flooding;
- The application of the concept to other types of port terminals;
- The optimal polder level, related to flood risks;
- The structural design of the combined dike and quay wall;
- The water drainage system.

The final recommendation of the preliminary study stated that the following items could be further investigated by a graduation student:

- Optimize unit rates applied in the Polder Terminal CAPEX database;
- Develop graph(s) indicating the optimum polder level below a conventional terminal level;
- Study and solve possible drainage problems (water management system);
- Develop solutions to transfer all kinds of cargo (container, dry bulk, liquid bulk, general cargo, etc.);
- Assess the differences in construction time between polder and conventional terminal;
- Make an extensive risk assessment to provide a basis for estimating an appropriate insurance plan;
- Develop a concept for the most practical structural solution of a combined dike and quay wall;
F.2.3 Project goal

It is clear that not all topics presented above can be investigated thoroughly in one master thesis, so assumptions will be made on topics which will not be investigated. The focus will be made on the risks involved in the design and the structural part of the Polder Terminal.

Port authorities must be convinced of the Polder Terminal’s safety against possible flooding, because of the low lying surface levels. The most important part of the research on the design of the Polder Terminal therefore consists of an extensive risk assessment concerning the flood defenses.

This master thesis will therefore aim at making a risk based design of a polder terminal. The following research questions will be addressed:

- What are the consequences of making a polder terminal, in comparison to a conventional terminal?
- What are the risks involved when making a polder terminal?
- What levels (height) should be chosen for the quay wall and terminal yard of the polder terminal?
- What quay wall flood defense system is most suitable for a polder container terminal?
- How to deal with water storage and drainage inside the polder container terminal?

F.3 Work plan

This chapter explains the approach of the project. It should be noted that the approach as explained in the next paragraphs, including the planning in the next chapter are subject to changes depending on the progress of the project, the student’s interests and the opinion / evaluation of the graduation committee.

F.3.1 Literature investigation

The graduation project will start with an investigation in the available literature on the subject. The information is split up in different categories. When all relevant information is gathered a literature review is written in a short report (20 pages).
Appendices

F.3.2 Polder Terminal, a conceptual design

This phase consists of a detailed explanation of the conceptual design of a Polder Terminal. The basis of this research will be the preliminary study made by Royal HaskoningDHV, information obtained in the literature review will be added in order to identify all relevant aspects. All differences between a ‘conventional’ terminal and a Polder Terminal are investigated.

A container port terminal is considered, because most of the port expansion and/or new port projects consist of container terminals. Container ships are increasing in size which means larger and especially deeper berths are required to handle these ships. To keep up with the demands of these mega ships countries need to expand their existing ports or build new ports.

If a different type of terminal is required certain alterations can be done to the proposed designs made in this master thesis. In the final chapter “General conclusions and recommendations” a short overview of possible alterations to the design of the container Polder Terminal are presented for different port functions.

<table>
<thead>
<tr>
<th>Category</th>
<th>Argumentation</th>
<th>Aspects</th>
</tr>
</thead>
<tbody>
<tr>
<td>Port infrastructure</td>
<td>Information on port infrastructure design is required to make assumptions regarding terminal lay out.</td>
<td>Port functions</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Quay walls in general</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Quay wall classifications</td>
</tr>
<tr>
<td>Polders</td>
<td>Information on polder design is required because the port will be designed as a polder.</td>
<td>History of polders</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Polder design</td>
</tr>
<tr>
<td>Flood defenses</td>
<td>Possible flood defense systems need to be investigated</td>
<td>Soil structures</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Special water retaining structures</td>
</tr>
<tr>
<td>Risk assessment</td>
<td>The development of risk based design methods is investigated and relevant information regarding risk assessment for flood defenses is given.</td>
<td>Development risk based design</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Probabilistic approach to dike design</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fault tree</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Consequences of flooding to industrial areas</td>
</tr>
<tr>
<td>Polder reference projects</td>
<td>Possible reference projects where Polder Terminals could be applied or polders which are applied for industrial purposes are elaborated.</td>
<td>Industrial polders</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Polder land reclamation projects</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Container port reclamation projects</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Possible Polder Terminal cases</td>
</tr>
</tbody>
</table>

Table 64: Literature review
Polder Terminal case
In order to investigate the feasibility of the polder terminal a conceptual design is made of a Polder Terminal based on an actual case. When considering the polder attention should be paid to the surface levels, water levels, water storage, discharge system and potential wetlands. Decisions regarding the design of these aspects will be based on a general risk assessment of flooding of the polder terminal.

Water/soil retaining structure: flood defense
An important part of the Polder Terminal consists of the water retaining structure, in this case a combined dike and quay wall. Part of the conceptual design of the model thus will focus on the design of this combined dike and quay wall. Different types of quay walls are applicable, the most practical solution will be chosen, possibly incorporating multiple functions in one quay wall structure. The design of this structure will be made in detail in the next chapter.

Risk assessment
A general risk assessment is made in order to correctly identify possible failure mechanisms. It is stressed that all failure mechanisms need to be taken in to account: failure of hydraulic structures often occurs because certain failure mechanisms where forgotten rather than poorly computed.

Failure mechanisms in the case of a polder will result in flooding of the terminal. Flooding may occur temporarily or permanently. Temporary flooding in the Serviceability Limit State, due to seepage, excessive rainfall or overtopping, poses relatively small damages to the port terminal. Permanent flooding in the Ultimate Limit State due to dike breaching, earthquakes or possibly ship collision will result in much larger damages.

F.3.3 Flood defense design
An important part of the Polder Terminal consists of the flood defense system which consists of a quay wall possibly in combination with a dike, as illustrated in Figure 87. This chapter will focus on the detailed design of this structure. After determining the most important failure mechanisms in the conceptual design phase this chapter will focus on the design of the flood defense system in relation to these failure mechanisms.
The risks of the Polder Terminal identified in the last chapters will be investigated in detail in this chapter. All design parameters are investigated and corresponding conclusions and recommendations with regard to the feasibility of the Polder Terminal, its risks and possible influence on costs are made.

**Polder Terminal risks**
A detailed risk assessment is made of the Polder Terminals in order to correctly identify possible failure mechanisms and determine the actual failure probabilities.

When all possible failure mechanisms are identified the corresponding consequences need to be assessed (risk is defined as probability * consequence). Consequences need to cover financial damages due to closure / down time of the port terminal, damages to the terminal and possibly human casualties.

Whether or not risks are acceptable depends on the specific case: if an alternative terminal is available during flooding of the Polder Terminal the consequence of the terminal flooding will be lower resulting in a lower risk.
Appendices

Figure 88: Fault tree Polder Terminal

Costs
Costs are an important aspect of Polder Terminals, in fact lowering construction costs is the main reason to propose a Polder Terminal instead of a conventional terminal. Constructional (CAPEX) and operational (OPEX) costs for the construction of a Polder Terminal are to be estimated.

Comparison Polder Terminal versus ‘conventional’ terminal
In order to investigate the differences between the Polder Terminal and the ‘conventional’ terminal a design of a ‘conventional’ terminal is made for the same case. This design will however will not be very detailed. The aim is to make a design which is suitable for an analysis on the differences between the Polder Terminal and the ‘conventional’ terminal.

It is clear that the comparison between both types of terminals will be the end result of the feasibility study on Polder Terminals in general. The results will be presented in a table explaining similarities and differences between both.

F.3.5 Conclusions / recommendations

Finally conclusions and recommendations regarding the feasibility of a Polder Terminal in general are made. As stated before large assumptions are required on aspects which are not investigated in this thesis. This phase will focus on naming these aspects and making recommendations for further investigation.
In addition to recommendations concerning the assumptions made on these aspects other recommendations and conclusions will be given concerning the specific case design made in this thesis. Pitfalls, learning points, uncertainties etc need to be identified so research on this subject can proceed after this graduation project.

**F.4 Project planning**

During the extent of the graduation project different progress meetings will take place with the supervisors and graduation committee. During these meetings partial reports are to be handed in and evaluated by the committee in order to check the progress of the graduation work. In Table 65 an overview is given of these meetings, the partial reports to be handed in and their contents. An estimate of the time required to complete these reports is also given.

<table>
<thead>
<tr>
<th>Partial reports</th>
<th>Date</th>
<th>Partial report</th>
<th>Contents</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kick off meeting (committee)</td>
<td>1 May 2012</td>
<td>Report 1</td>
<td>Thesis proposal</td>
<td>1 wk</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Report 2</td>
<td>Ch 2. Literature</td>
<td>3 wks</td>
</tr>
<tr>
<td>Progress meeting 2 (committee)</td>
<td>June 2012</td>
<td>Report 3</td>
<td>Ch 3. Polder Terminal concept</td>
<td>4 wks</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Report 4</td>
<td>Ch4. Model simulation</td>
<td>5 wks</td>
</tr>
<tr>
<td>Progress meeting 3 (committee)</td>
<td>August 2012</td>
<td>Report 5</td>
<td>Ch5. Flood defense design</td>
<td>9 wks</td>
</tr>
<tr>
<td>GO / no GO meeting (committee)</td>
<td>Sept 2012</td>
<td>Report 6</td>
<td>Ch6. Conclusions / recommendations</td>
<td>1 wk</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concept thesis</td>
<td>Concept thesis / presentation</td>
<td>2 wks</td>
</tr>
<tr>
<td>Final meeting</td>
<td>October 2012</td>
<td>Final thesis</td>
<td>Final master thesis / presentation</td>
<td>3 wks</td>
</tr>
</tbody>
</table>

*Table 65: Project planning*

The information of Table 65 is shown in a planning in Figure 88. In this planning presentations for Royal HaskoningDHV are also added, which will be done to keep everyone informed of the progress of this graduation project.
Figure 89: Project planning