Tiaozini Land Reclamation
Preliminary Port Area Design
Tiaozini Land Reclamation – Preliminary Harbor Area Design

Course CT4061-09 – Multidisciplinary project

Multidisciplinary Project Group China 2012

D. Beemsterboer
J. Christophe
R. Scheepjens
M. Veldman
V. van der Wielen

Accompaniment Delft University of Technology

Prof. Ir. A.F. van Tol
Prof. Dr. Ir. C. van Rhee
Dr. J.C. van Ham

Accompaniment Hohai University

Prof. Dr. C. Zhang
Prof. Dr. H. Liu
Prof. Dr. Q. Jiang
Prof. Dr. Y. Chen
Asso. Prof. Dr. G. Kong
Asso. Prof. Dr. Y. Chen
Asso. Prof. Dr. Y. Shen
Asso. Prof. Dr. Z. Gong
Dr. X. Feng
Dr. Y. Zhuang
Delft Infrastructures & Mobility Initiative (DIMI)
Foreword

Commonly people think that most is learned from the lectures, the lecture notes and the examinations provided by the professors of TU Delft. This is however only the germ, that shoots roots when the students try to actively apply the theories and models. Then they discover that these models provide a fairly limited reflection of reality and that the data needed to run the models are not or partly available within the time frame given for the project.

The best way to discover this is a real project. A student project like the one reported in this note provides the opportunity to learn without grave consequences for others if mistakes are made. It is also learned that besides knowledge planning and organization of the activities is of primary importance.

The Tiaozini Land Reclamation project is of special interest because it shows that China undertakes gigantic projects in an area that is commonly described as the special expertise of the Dutch. And indeed on the coast in the city of Nantong the Dutch experts Johannes and Hendrik de Rijke, father and son advised on and designed the navigability of the Yangtze river and the flood safety of the city more than one century ago. The Chinese people have recognized this contribution by means of a bronze statue of "mr de Rec" as they say.

In 2012 the Chinese execute a gigantic land reclamation project in front of the Jiangsu coast out in sea with very simple means, taking advantage of the specific local conditions. It also shows that China strides forward where Europe hesitates to tread. They make great plans and execute them with speed and precision. This is visible not only in this land reclamation project but also in the gigantic metro networks, airports, railway stations, high speed train lines, high rise buildings, roads and ports that they have built in the last decades. Everything is under construction or completed and perfectly working. The future will show if this is the groundwork needed for a great economic development or an overinvestment. Currently is seems without doubt to be the first.

So besides providing the opportunity to apply civil engineering knowledge in an practical situation that differs from the Dutch environment and to organise a project in a different culture this project has given Dirk Beemsterboer, Jeroen Christophe, Robert Scheepjens, Michiel Veldman and Vincent van der Wielen the invaluable chance to see the incredible development of China with their own eyes. This aspect is not reported here and the reader would perhaps have difficulty to believe it when it was, but it is inerasably stored in these students memory like the practical engineering experience that is indeed reported.

J.K.Vrijling

22-11-12 Wuhan
Preface
As part of the master’s curriculum at Delft University of Technology it is possible to start a multidisciplinary project as an elective course. This can be done either in the Netherlands, or abroad. The ideal project group consists of 4 to 6 students from different disciplines. The goal of this project is to solve a current and/or recent civil engineering problem in a multi-disciplinary team. Integrate multiple studies and designs into a coherent whole, based on knowledge, understanding and skills acquired in previous years. Attention is paid to quality control and evaluation of the design. Part of the project will consist of creating a clear and solvable problem definition based on a wide starting point. Moreover the group of students will have to learn how to solve a problem using a multidisciplinary approach based upon different disciplines in civil engineering.

Planning the multidisciplinary project started in March 2012 when the project group contacted Prof. J.K. Vrijling about the possibility of starting a multidisciplinary project. This led to Ir. C. Timmers from TU Delft – Infrastructures and Mobility Initiative (DIMI). Timmers had close relations with Prof. C. Zhang from Hohai University based in Nanjing, China. Timmers went to China to contact Zhang and among other things explained what the multidisciplinary project would be about. Both parties saw great opportunities for the project group to work at Hohai University. Because Timmers and Zhang both where pre-occupied, further contact between Hohai University and Delft University of Technology would be established by Drs. J.J. de Boer and Prof. Y. Chen.

After successfully finding a project to work on, the group focused on the funding of the project. In order to be able to go to China and do the research a lot of contributors had to be found. After contacting approximate 120 companies the group was successful in finding sufficient contributors.

To get a grade for the multidisciplinary project, representative professors from Delft University of Technology were asked to revise the report. The professors will give one joint grade for the report but for the parts dealing with geotechnical engineering, Prof. A.F. van Tol will be the examiner. The Off-shore engineering parts will be revised by Prof. C. van Rhee. Dr. J.C. van Ham will grade the parts concerning transport, logistics and harbors.

After arrival at Hohai University on 9th of august 2012 the project group was coupled to several Chinese professors and students to assist them with their research. They provided the group with expertise, technical data and translated key documents from Chinese to English.

The result of ten weeks of research in China is the report in front of you. The project group believes this report is of use to Hohai University and the Jiangsu land reclamation project. In addition the project group believes this report meets the requirements set by the examiners of Delft University of Technology in order to pass their course in a good manner.
Abstract

In 2009 the Chinese State Council approved the developing plan for Jiangsu coastal zone that was part of the national developing strategy. The plan set goals to reclaim 21 areas in the Jiangsu coastal zone with main objective to achieve an economical impulse for the Jiangsu province. The Tiaozini reclamation acted as a pilot project and was assigned a priority area due to its size and strategic position between Lianyungang and Shanghai. The proposed occupation guidelines for Tiaozini hardly met the objective of giving an economical impulse and revision of the area was required. The northern part of Tiaozini was still under construction and was further assessed in this study.

The Dutch layer approach was explicitly used as an analyzing tool to make a top-down preliminary design for the Tiaozini port area using Dutch norms and standards. This tool allowed the study to be divided in three conditioning layers; occupation, network and base layer allowed for a multidisciplinary and iterative process. The most suitable occupation was determined using macro-economic, industry-specific and port analysis. Due to the favorable investment climate ensuring future potential, lack of competitors and presence of partners, an efficiently organized public-private petrochemical port area was determined as preliminary occupation for the northern part of Tiaozini. The supporting transport network should take the following criteria into account: modal split, flexibility, sustainability and capacity. These criteria were determined by the characteristics of the petrochemical industry. The connection between the network and base layer was made by the use of the main enabling road connection. The embankment settlement for the required six-lane highway was calculated using D-Settlement numerical program in combination with NEN-Bjerrum calculation method and checked using analytical NEN-Koppejan method. A 4.5m high surcharge preload in combination with vertical drains was required for six months in order to fulfill the residual settlement requirements. PLAXIS was used to evaluate the stability of the newly designed dike with a design return period of 1.000 years, allowing safety for the petrochemical port area. With a safety factor of 1.8 leading to multiple simultaneous slip surfaces, the new dike design is capable of ensuring safety for the petrochemical port area. All results support the presumption the Tiaozini area is suitable for petrochemical industry.

Keywords; China, Jiangsu, Tiaozini reclamation, petrochemical port area, multidisciplinary, Dutch layer approach, D-Settlement, NEN-Bjerrum, NEN-Koppejan, PLAXIS, transport network, vertical drains, dike stability
Acknowledgement

This work is carried out at the Geotechnical Research Institute of Hohai University (GeoHohai) as well as at the State Key Laboratory of Hydrology-Water Resources and Hydraulic Engineering of Hohai University in collaboration with the department Civil Engineering of Delft University of Technology.

First and foremost the authors would like to address their gratitude to the accompaniment of Hohai University; Prof. H Liu, Prof. Q. Jiang, Asso. Prof. Y. Shen, Asso Prof Z. Gong, Asso. Prof. G. Kong, Asso. Prof. Y Zhuang, Dr. Y. Chen, and Dr. X. Feng.

The authors would like to take this opportunity to gratefully acknowledge the accompaniment from Delft University of Technology, Prof. A.F. van Tol, Prof C. van Rhee and Dr. J.C. van Ham for their guidance and advice and Dr. Ir. R.B.J. Brinkgreve for his technical assistance.

For the establishment of the collaboration between Delft University of Technology and Hohai University, the authors would like to pay their respect to Ir. C. Timmers and Drs. J.J. de Boer from TU Delft – Infrastructures & Mobility Institute as well as Prof. C. Zhang and Prof. Y. Chen from Hohai University.

The authors are particularly grateful for the help of 刘嘉琦 (Steven); his help was of vital importance in the startup of this research. To all the other students from Hohai University who helped in the process, the authors would like to address their gratitude.

For the help with vital data acquisition and the invitations to different social events the authors would like to thank the Netherlands Business Support office (NBSO) and in particular Roeland Schuurman.

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List of Symbols

- $a$: Modified natural swelling index [-]
- $b$: Modified natural compression index [-]
- $c$: Modified natural secondary compression constant [-]
- $c_v$: Vertical consolidation coefficient [-]
- $c_h$: Horizontal consolidation coefficient [-]
- $c'$: Cohesion [kPa]
- $d$: Depth [m]
- $d_{max}$: Maximum effective depth [m]
- $e_0$: Initial void ratio [-]
- $e$: Void ratio [-]
- $g$: Gravitational constant [m/s²]
- $h$: Flow thickness [m]
- $h_0$: Vertical height of layer at the start of (un)loading [m]
- $h_b$: Water depth at berm [m]
- $h_{berm}$: Berm height [m]
- $h_t$: Vertical height of layer at time $t$ of (un)loading [m]
- $\Delta h$: Vertical settlement of layer of sample at time $t$ [m]
- $i_e$: Electric potential gradient [V/m]
- $k_o$: Electro osmotic permeability [m²/sV]
- $k_h$: Horizontal permeability [m²/s]
- $k_v$: Vertical permeability [m²/s]
- $k_x, k_y$: Darcy permeability [m²/s]
- $m$: Total mass [gram]
- $m_s$: Mass of solids [gram]
- $m_w$: Coefficient of compressibility [kPa⁻¹]
- $n_0$: Initial porosity [-]
- $p$: Pore pressure [kPa]
- $p_0$: Initial pore pressure [kPa]
- $q_A$: Specific flow [m³/s]
- $t$: Time [days] [s]
- $t_0$: Reference Time [days]
- $t_{age}$: Initial equivalent age [days]
- $t_{50\%}$: Time required for 50% consolidation [s]
- $t_{95\%}$: Time required for 95% consolidation [s]
- $t_{99\%}$: Time required for 99% consolidation [s]
- $t_d$: Drain thickness [m]
- $u_d$: Maximum water suction [kg/m³]
- $w$: Water content [-]
- $w_d$: Drain width [m]
- $A$: Surface area orthogonal on current [m²]
- $A_p$: Primary swelling coefficient [-]
- $A_s$: Secondary swelling coefficient [-]
- $B_{berm}$: Berm width [m]
- $C_{st}$: Coefficient of secondary compression [-]
- $C_c$: Primary compression index [-]
- $C_k$: The constant for strain dependent permeability [-]
- $C_p$: Primary compression coefficient below pre-consolidation [-]
- $C_{p'}$: Primary compression coefficient above pre-consolidation [-]
- $C_r$: Reloading/Swelling index [-]
$C_s$  Secular compression coefficient below pre-consolidation [-]
$C_s'$  Secular compression coefficient above pre-consolidation [-]
$C_{sw}$  Primary swelling index [-]
$CR$  Compression ratio [-]
$D$  Drain spacing [m]
$D_s$  Center-to-center drain spacing [m]
$E_{oed}$  Soil stiffness [kPa]
$E_s$  Soil stiffness [MPa]
$H_{ch,1}$  Transition wave height [m]
$H_{drop}$  Average drop height [m]
$H_d$  Design wave height [m]
$H_{fr}$  Freeboard [m]
$H_m$  Mean wave height [m]
$H_{m0}$  Significant wave height [m]
$H_s$  Significant wave height [m]
$H_{x%}$  Wave height exceeded by x% of the waves [m]
$K_w$  Bulk modulus of water [kg/m3]
$L$  Wave length [m]
$L_{berm}$  Berm length [m]
$M_{drop}$  Mass of drop weight [ton]
$N$  SPT blow count [-]
$OCR$  Overconsolidation ratio [-]
$POP$  Pre-overburden pressure [kPa]
$R$  Hydraulic permeability ratio [-]
$R_{102%}$  Wave run-up [m]
$RR$  Reloading/Swelling ratio [-]
$S$  Total settlement [mm]
$S_{int}$  Instant settlement [mm]
$S_{prim}$  Primary settlement [mm]
$S_{sec}$  Secondary settlement [mm]
$T$  Wave period [s]
$U$  Degree of consolidation [-]
$V$  Total volume [m3]
$V_s$  Volume of solids [m3]
$V_w$  Volume of water [m3]
$\alpha$  Angle of slope [°]
$\alpha_{comp}$  Compaction coefficient [-]
$\alpha_{slopecx}$  Angle at slope x [°]
$\beta$  Gumbel scale [-]
$\beta_{tr,1}$  Battjes & Groenendijk constant [-]
$\beta_{tr,2}$  Battjes & Groenendijk constant [-]
$\gamma$  Wet unit weight [kN/m3]
$\gamma_b$  Berm coefficient [-]
$\gamma_d$  Dry unit weight [kN/m3]
$\gamma_{sat}$  Saturated unit weight [kN/m3]
$\gamma_w$  Unit weight of water [kN/m3]
$e^c$  Engineering vertical strain [-]
$e^h$  Natural vertical strain [-]
$\dot{e}$  Strain rate [days⁻¹]
$\kappa$  Weibull parameter [-]
$\mu$  Gumbel mode [-]
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<tbody>
<tr>
<td>( v )</td>
<td>Volume</td>
<td>([m^3])</td>
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<tr>
<td>( v_0 )</td>
<td>Initial volume</td>
<td>([m^3])</td>
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<tr>
<td>( \xi_m^{1.0} )</td>
<td>Breaker parameter</td>
<td>[-]</td>
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<tr>
<td>( \rho )</td>
<td>Mass density</td>
<td>([kg/m^3])</td>
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<td>( \rho_d )</td>
<td>Dry density</td>
<td>([kg/m^3])</td>
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<tr>
<td>( \rho_s )</td>
<td>Soil particle density</td>
<td>([kg/m^3])</td>
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<tr>
<td>( \sigma )</td>
<td>Total pressure</td>
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<td>( \sigma_c )</td>
<td>Limit state pressure</td>
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<td>( \sigma' )</td>
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<td>( \sigma'_{0} )</td>
<td>Initial effective soil pressure</td>
<td>([kPa])</td>
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<tr>
<td>( \sigma_p )</td>
<td>Pre-consolidation pressure</td>
<td>([kPa])</td>
</tr>
<tr>
<td>( \tau_0 )</td>
<td>Creep rate reference time</td>
<td>([days])</td>
</tr>
<tr>
<td>( \varphi )</td>
<td>Friction angle</td>
<td>([^\circ])</td>
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1 Introduction
The Jiangsu province is an eastern coastal province, centrally located between Beijing and Shanghai (Figure 1.1). Because of its position, Jiangsu is holding an important strategic position of linking the South and the North, and bridging the East and the West of China (Planning Team, 2009). Its geographical position makes the province a hot spot for economic development. Results are shown in the economic numbers of the province. With a Gross Regional Product (GRP) per capita of ¥ 52840,- (¥ 1,- = € 0,12). Jiangsu is the fourth highest in China (National Bureau of Statistics, 2011). A significant trend that threatens the economic development is the movement of industrialization and manufacturing of goods to inland locations (van Yperen, 2012). This movement is facilitated by governmental policy with tax incentives (Portstrategy, 2012). To maintain and improve the current economic development of the Jiangsu province, provincial authorities encourage and initiate strategic large scale construction and development projects. The Jiangsu province needs land for advancing the fast development of marine economy, the adjustment of marine industrial structure and speeding up the balanced development of Jiangsu economy. Due to the characteristics of the Jiangsu coastal zone, which indicates rich tidal flats (approximately 5000 km\(^2\) in total), sand ridges and a coastline of 954 km, it is possible to reclaim land on a large scale from the sea (Planning Team, 2009). In 2009 the Chinese State Council approved The Developing Plan for Jiangsu Coastal Zone that is currently part of the national developing strategy. This makes the Development of the Jiangsu Coastal zone a project of national interest. The plan sets the goal to reclaim 21 new areas with a total area of 1800 km\(^2\). These land reclamations are constricted along the Jiangsu coast during the period from 2009 to 2020. At the first stage during 2009 to 2012, it is planned and realized to reclaim 510 km\(^2\) tidal flat. In the second stage during 2013 to 2020, it is planned to reclaim the remaining 1290 km\(^2\) (Planning Team, 2009).

![Figure 1.1 - Jiangsu province](image)

1.1 Problem description
The overall objective is to reclaim land in order to achieve an economic impulse for maintaining the economic development of the Jiangsu province. Subsidiary to this objective is to reclaim land according to elementary guidelines of the construction process. Examples of these guidelines are: the integrated development for coastal zone should be advanced with a high starting point, with objective first and reclamation later, planning first and construction later, pilot test first and extension of the experience later, so as to supply land sources for future sustainable social and economic development of Jiangsu (Planning Team, 2009). Important is that the guidelines are determined by five principles of development, which are stated in the Master plan. Briefly described, the principles emphasize to develop: scientifically, integrally, heterogeneously, efficiently and sustainably. In
practice the construction project is experiencing a few major problems each relating to the development principles. The major problems in the tidal flat development include: low diversity of occupation and uncertainty of economic potential occupation for the reclaimed area in order to guarantee development of the Jiangsu economy and a low scientific level of the development approaches, which lowers the efficiency of the development in general. Referred to this problem is stated; the integrated development for the coastal zone should be advanced with a high starting point, with objective first and reclamation later, planning first and construction later.

1.2 Approach and focus
In order to improve the construction and development process of the tidal flat area by redressing the major problem as stated before, the mechanism of development needs to be improved. A desired perspective is a multidisciplinary point of view addressing the issues in an iterative process. Taking into account that aspects of the development, such as the occupation of the reclamations and the dike stability, direct or indirectly influence each other. In this case an approach is needed that takes into account decisions, which are made in early stages of the development process, can have a big impact on the design of the reclamation and the actual reclamation process.

Characteristics of a model to be used are: recognizing and dealing with complexity, large scale systems and tolerate an iterative process of design and different aspect of the construction process. The Dutch so called layer approach meets these model characteristics. The focal point of the Dutch layer approach is the unbundling of the different layers. The mutual dependency and interaction between the layers is a crucial element in the approach.

The layer approach enables to demarcate the project geographically. This is desirable, because the initial Developing Plan for Jiangsu Coastal Zone contains 21 reclamations, which makes the scope of the project substantively complex. In this research the Tiaozini land reclamation will be used as a case study. Due to the sharp demarcation, aggregated technical analysis and area-related-economic analysis can be made. The aggregation level of analyzing will be in favor of using the layer approach. Chapter 1.3 describes in detail the scope of this research project.

1.3 Research objectives and questions
By implementing the Dutch layer approach an attempt is made to give a possible solution on the major development problems.

The central objective in this research is: To make a preliminary design for the Tiaozini port area by using the Dutch layer approach.

This objective will be achieved by using the following (sub) questions:

- What kind of occupation is suitable to give the area a significant economic impulse?
- Which criteria should be taken into account when designing a transportation network on Tiaozini for the chosen occupation?
  - Which characteristics of the occupation determine these criteria?
  - What transport network concepts does the ‘Maasvlakte 2’ contain with respect to these criteria?
- Which technical improvements are needed to facilitate the preferred occupation and network?
  - What is the ideal consolidation method for a road construction?
o What is the required design and significant wave height for the chosen occupation?
o Is it possible to construct a stable dike with this required height using the original Chinese construction method?

1.4 Scientific, methodological and Practical contribution

1.4.1 Scientific and methodological contribution
Although geotextile tubes have been used for different hydraulic and marine applications, a project with the method of construction and scale of Tiaozini is never used before. With this comes a different understanding of risks and technical complexity. In this study a few essential technical aspects are deepened and described. Due to a multidisciplinary approach and the use of the Dutch layer approach, the study contains a balanced yet technically detailed point of view on the development process.

1.4.2 Practical contribution
At this moment the development of the Tiaozini reclamation is in its first phase. According to the procedure of tidal flat reclamation the Tiaozini is a pilot reclamation, mainly focusing on conducting comprehensive development at pilot reclamation at development nodes, optimize the planning, discovering ways of efficient development, forming new mechanisms for tidal flat reclamation (Planning Team, 2009). Perhaps defining elements of the study as stated in this report can be used as an inspiration or can serve as a direct contribution to the development process.
2 The Dutch layer approach

In this chapter the Dutch layer approach will be introduced. Information is given about its history, purpose and the fundamental principles of layers on which the approach is based on.

![Figure 2.1 - Visualization of the Dutch layer approach](image)

2.1 Purpose and history of the approach

In 1998 a new approach, based on the principle of layers that distinguished spatial planning and spatial analyzing tasks, was introduced by De Hoog, Sijmons and Verschuuren (van Schaick & Klaasen, 2011). The basic principle can be compared with software families like Computer Aided Design (CAD) and graphically-oriented software such as Adobe Photoshop and Adobe Illustrator. These software families use the principle of layers as organizing structure of data. The Dutch layer approach hit a nerve in spatial planning practice in the Netherlands, in particular on a national level, but later also on the provincial and municipal level. However the layer approach is often called a model, it is designed to be used as an approach. Since 1998 the layer model has developed into an approach to spatial planning and design: the Dutch layer approach (van Schaick & Klaasen, 2011). In theory the approach is positioned as a framework to recognize conflicting sectorial implementations and developments as well as an analyzing and communication tool. In section 3.3, information is given about how the layer approach is positioned in this research.

2.2 Layers

The approach is based on the differing spatial dynamics of three layers: occupation, network and base layer. Each layer has its own specific contribution to the development in general. The strength of the layer approach lies in the interaction between the different layers (VROM, 2006). Due to this interaction, the process of planning and development gets dynamic. Because of the dynamic characteristic there is more interaction between disciplines which leads to a more efficient and more successful planning. The interaction is funded by the assumption that an underlying layer sets conditions to its parent layers. The approach does not only encounter matters within each layer, but also encounters matters between layers. For successful planning and development, the processes in and between the different layers should be associated (VROM, 2006). The content of the three layers will shortly be addressed.
2.2.1 Occupation layer
The occupation layer is the upper layer of the Dutch layer approach. The occupation describes the spatial patterns indicated by human use of the subsoil and the networks (VROM, 2006). This layer allows a broad perspective on the content within the layer. In detail the occupation layer fills in the accommodating spatial claims of an area and pays attention to the economical values and attractiveness of activities that can take place within a specific area.

2.2.2 Network layer
The network layer consists of both physical infrastructure and invisible connections. The physical infrastructure controls and steers the growth of mobility such as traffic and transport flows. The physical infrastructure is the set of roads, railways, waterways, ports, airports, transfer- and transshipment points. Invisible connections are mainly the underground pipes and cables used for ICT and utilities supply. Both types of networks are carriers of urban networks and are an important precondition for urban and economic dynamics (VROM, 2006). Major influences in the network layer find their origins in civil engineering studies (transport and planning engineering, port and waterways). Areas of expertise like logistics and urban planning also provide their point of view. These studies enable the use of the Complexes approach and the Corridor approach. These approaches are often used to address the major issues in the network layer (van Schaick & Klaasen, 2011). Main issues or points of attention consist of modal split, capacity, flexibility and the compatibility of networks.

2.2.3 Base layer
The base layer in this study focuses on the subsoil, its properties and the related site preparation. In addition the stability of the designed dike will be addressed. Dike constructions and soil consolidation for site preparation are examples which define the issues within this layer. The approach in this layer has its foundation in civil engineering studies (offshore engineering, geo-engineering and earth sciences). However in many studies the base layer (also called substrate layer) contains biological processes, whereas environmental engineering studies determine the approach. A characteristic of this layer is that the rate of change within the layer is low, 50 to 500 years. Activities within the base layer therefore have long term influence on the parent layer. The base layer sets strict boundaries and restrictions for the two upper layers.

2.3 The importance of the layer approach
International tunneling and underground space association committee (ITACUS) explains the importance of mapping and planning the occupation within an area with respect to the base layer and vice versa. Their focus pays more attention to underground transport facilities instead of site preparation, but their arguments are plausible and valid for any land reclamation project. ITACUS explains that due to the explosive growth of cities, shifting demographics and aging infrastructure in older cities, coupled with the demand for improved live ability and environmental protection, we are creating a strong demand for new underground infrastructure. As this happens, the impact of previously unplanned underground space use rapidly becomes clear, expensive relocations of existing facilities are required, access to favorable geological conditions may be blocked and underground transport facilities are forced progressively deeper to find suitable alignments. (ITACUS, 2010).

The biggest problem between the occupation layer and network and base layer is explained by ITACUS as following: the rule is typically first come, first served. The first come user takes the most
favorable place for his/her particular needs (location, geological conditions, easier construction, etc.), without any vision for the possible future uses of underground space at that location. Multi-functional structures underground are very infrequent (ITACUS, 2010). The Dutch layer approach can help to map the conflicting sectorial aspect of the occupation layer and other layers.

To give insights in the importance and the benefits of using a layer approach an example is given. The City of Shanghai provides such an example. The city would have endured problems if no planning existed. The use of underground space in Shanghai, as in many other Chinese cities, has been growing rapidly in the last two decades but conflicts with prior uses can cause major difficulties. (ITACUS, 2010). City planners were forced to reconstruct the alignments of planned metro lines because of recently constructed building foundations. The building foundations were extending deeper than the expected 16 meters below surface. This problem could have been foreseen by using the layer approach. The layer approach forces policy makers and engineers to think about future occupation activities and its consequence for the base layer. Recently Shanghai, Beijing and nearly 20 other cities in China, are coordinating spatial planning by using local regulations. Primarily attention is paid to the use of underground space in order to prevent spatial conflicts (ITACUS, 2010).

2.4 Positioning of the Dutch layer approach in this study
As stated above, the Dutch layer approach has many purposes and instrumental values. For that reason there is a broad variety of usage for this approach. In this study the Dutch layer approach is explicitly used as an analyzing tool for a preliminary port area design. By separating three layers, different construction processes and different construction phases can be distinguished. The base layer will mainly be addressed from the viewpoint of geo-engineering and offshore-engineering. The network and occupation layer will be addressed from the viewpoint of transport, infrastructure and logistics.

An interesting knowledge gap is how the demanded purpose of the area (occupation layer) affects the construction and reclamation methods (base layer). Another interesting knowledge gap is the requirements which the occupation sets for the transport network (network layer). The network layer sets requirements for the site preparation and its spatial planning. The base layer in turn sets limitations for both the network and occupation layer.

With respect to the research and its quality, a critical note must be made. In this study the layer approach is used as a top-down approach. This means that there are little consequences of underlying layers to their parent layers. In other words, the current situation of the soil does not give much restriction or boundaries to its occupation. The occupation is in these terms hierarchical with respect to the network and base layer. Of course this is not a realistic reproduction of the situation and its interdependencies of the layers. The assumption is made that there are little to none limitations to the technical changes that can be made to improve the soil properties for occupation and network related activities. This assumption can be afforded because the current situation of the land reclamation is still in its construction phase. This does not mean that possible solutions for improving the soil quality and dike stability are just made up without valid arguments. Every solution, method or implementation regarding the base layer is supported by technical analysis and calculations.
3 Demarcation

In this chapter the scope of the project is described. The scope sets research boundaries for the aspects that are covered within this study. Because of the variety of subjects within this study, sharp and detailed boundaries are needed. The following aspects are taken into account:

- Geographical demarcation
- Methodic demarcation
- Demarcation within the layers

3.1 Geographical demarcation

Figure 3.1 shows the coastline of Jiangsu, existing of the three coastal areas: Lianyungan, Yancheng and Nantong. The coastal zone of the three areas has a total length of 954 km. The Dutch layer approach enables to demarcate the project geographically. This is desirable, because the initial Development Plan for Jiangsu Coastal Zone contains 21 reclamations, which makes the scope of the project substantively complex. Figure 3.1 shows the 21 planned reclamation areas. In this research the northern part of the Tiaozini reclamation area will be used as a case study.

The Tiaozini reclamation area is positioned in the Yancheng area of the Jiangsu Province. It is planned to reclaim a total size of 266.7 km². A few reclamation areas were assigned as priority project within the project Master plan. These so called priority areas were constructed first and used as pilot areas. Together with the area’s Lianyungang Port, Dafeng Port, Xinchuangang, Yangkou Port and Lusi-Dongzhao Port, Tiaozini was the main area to reclaim (Planning Team, 2009). It was planned to finish the reclamation of Tiaozini in 2012. This means that within a time interval of three years a total of 400.000 mu (266.7 km²) should have been reclaimed on the Tiaozini sand ridge. The total area of
Tiaozini however, is currently not yet completed. Because the Tiaozini area is still under construction, a lot of influence on the construction process is possible. This makes the Tiaozini area a very interesting choice as the geographical area for this research. The layer approach is implemented only for the northern part of Tiaozini (see red marked area in Figure 3.2). The reason for this twofold:

- Firstly, the northern part is interesting to investigate regarding the occupation layer. The northern part of Tiaozini borders seashore that allows construction of a deep-sea harbor (Feng, 2012). This gives rich possibilities for an occupation which can give the area a significant economic impulse.
- Secondly, the southern part is being constructed at the moment. The northern part will be constructed in the near future. This means that outcomes of this study can contribute to the current construction process or planned developments. Especially for the network and base layer, outcomes might be interesting.

3.2 Methodic demarcation

The Dutch layer approach has a broad variety of use. As stated in section 2.4, the Dutch layer approach is explicitly used as an analyzing tool for a preliminary design of the Tiaozini port area in this study. The layer approach is positioned as a top-down method. This means that the occupation sets restrictions and calls for change in the base layer. Normally the base layer would have included a feedback link with the occupation layer. This feedback link for example contains economic feasibility. The reality is that occupation activities can set high standards for soil improvement. These soil improvements would probably have high economic impacts of the project. The economic impacts can lead to compromise of the occupation activities. Because of the time constraint for this study, the feedback loop from underlying layers to the parent layer is limitedly taken into account.

Another important point to mention is that the Dutch engineering (analyzing, planning, construction etc.) standards are preferred throughout this study. This does not mean that the Dutch engineering philosophy is better or worse than for example the Chinese engineering standards. However to maintain a consistency throughout the technical analysis and calculations, it is necessary to use one standard. Examples of Dutch standards that are used throughout this study are:

- NOA, Nieuwe Ontwerprichtlijnen Autosnelwegen, used as a manual by Rijkswaterstaat, executive body of the Ministry of Infrastructure and Environment.
- EC, EuroCodes, providing a common approach for the design of buildings and other civil engineering works and construction products.
• NEN, Netherlands Standardization Institute standards
• EurOtop, the overtopping-manual composed in a collaboration of the Dutch, German and British government

3.3 Demarcation within the layers
Each layer has its own issues and project boundaries. Within every chapter describing the analysis of one of the three layers, more detailed assumptions and project boundaries will be given. In this section the main assumptions and project boundaries of each layer are mentioned.

3.3.1 Occupation layer
The occupation layer exists partly of a market research, taking into account the port areas and industrial areas. The decision is made to demarcate the market research on two aspects:

• Marco-economic situation of the Jiangsu province and in particular the Yancheng province
• Port area’s along the coast line of the Jiangsu Province including the Shanghai port area district

Less attention is given to the financial construction and governance and administrative structures that can influence the occupation.

3.3.2 Network layer
Within the network layer four main criteria are determined that are important for designing a transport network. The criteria are based on the characteristics of the particular occupation. Thereafter a reference is made to a recently Dutch reclaimed area: the Second Maasvlakte that is part of the Port of Rotterdam. The case study will focus on how the criteria are obtained in the design of the transport network. The research covers areas such as modal split regarding the industrial area, flexibility of expanding the network, sustainability of the network and the capacity of the transport network. Because of the exploratory nature of the research in this layer, the analysis will be based on qualitative research rather than quantitative research.

3.3.3 Base layer
Activities in the base layer focus on the technical aspects. A minor role is given to the economical aspect such as project and development costs and planning. Attention is paid to consolidations methods for a highway and the stability of a dike constructed using geo-tubes. This is done by desk research, technical calculations and computer modeling. Computer calculations are applied using numerical modeling and finite elements methods. Activities in the base layer do not include the performance of laboratory and field research.

In many implementations of the Dutch layer approach the base layer also includes the environmental matters and issues such as wetland and biological resources and environment protection of reclaimed area. These environmental matters are not included in this study.

Due the composition of the project group with its specific expertise, emphasize in this study will be on both the occupation and base layer. This does not mean the network layer is left unattended; on the contrary, this layer is used to make additional connections between the three layers. This statement merely indicates the main focus of this study.
3.4 Data acquisition

Conventional sources of information are used, such as previously mentioned standards, libraries, scientific papers and interviews with experts.

Although a considerate amount of time is spent on acquiring adequate, reliable and consistent data, most of the information found is inconsistent, incomplete and/or contra dictionary. In addition, most information found is written in Chinese and has to be translated by local students, introducing an additional factor of uncertainty in the reliability of the data.

Even seemingly reliable sources, such as Chinese National Bureau of Statistics, are known to provide data that cannot blindly be assumed reliable. This is due to the fact that the statistics are provided by each province separately and these provinces want to appear as successful as possible. High growth rates will attract more investments from the government and the local business. Additionally all statistics are firstly checked by Chinese government before they are published, this greatly affects the reliability, since this causes the National Bureau of Statistics to lose its independency (Schuurman, 2012).

Common sense, engineering judgment and grounded assumptions are combined in order to acquire workable data sets. To be able to make valid assumptions and implement engineering judgment, the project group went on a two day site visit to the Tiaozini tidal flat reclamation area (appendix B).

To ensure reliability of the used data, all previous influences are considered during this study.
Occupation layer

In the occupation layer the human use of the area is described. The spatial claim for the northern part of Tiaozini is explained by the so-called occupation. The occupation analysis consists of a macro-economic analysis, industry analysis, and a port analysis. Figure 4.0 shows the research structure of the occupation analysis.

Figure 4.0 - Research structure occupation layer

The macro-economic analysis focuses on the Jiangsu Province. The macro-economic analysis shows the industry that is most suitable for Tiaozini with respect to the economic impulse objective. Thereafter an analysis of this industry is made to determine the feasibility and realization due to investments, growth competences, and local opportunities. In particular attention is paid to the Yancheng district. Because Jiangsu possesses a long coastline with ports that play a vital part in the economy, a geographical port analysis is important to determine Tiaozini’s future occupation (Planning Team, 2009). Due to limited availability of time for this study, the geographical port area analysis focuses on the port areas in the Jiangsu province and the Shanghai port area. Special attention is paid to the organization of the port area. The organization will be an important factor that supports and determines the competitiveness of the port area. The connection between the occupation layer and the network and base layer is made in terms of requirements that the occupation sets. These requirements can be interpreted as an introduction for the underlying network and base layer.
4 Preliminary occupation determination

4.1 Economic impulse as main objective

The strategy behind the Jiangsu coastal tidal reclamation project indicates a significant contribution to the development of the Jiangsu economy; the reclaimed areas should become a new spot for economic increase (Planning Team, 2009). Tiaozini is one of the first and biggest reclaimed areas being realized. The Master Plan sets guidelines for Tiaozini’s occupation. However these occupation guidelines hardly meet the objectives of giving a significant economic impulse in the region. This economic impulse is necessary in order to maintain and improve the current economic development of Jiangsu. The occupation guidelines are mainly focused on the development of towns, large scale seafood processing, large scale agricultural activities and eco-tourism (Planning Team, 2009). The distribution of the different occupation activities on Tiaozini is: 50% agriculture, 25% ecology and 25% construction (industry) (Planning Team, 2009). At this point, the Master Plan is critically rejected. The goal of the occupation on Tiaozini is to create long lasting significant economic impulse in the region. Agriculture is not considered to be a viable alternative. Revenues per square kilometer (¥/km.\(^2\)) are lower for agriculture than for industry and since reclaiming new land in sea is capital intensive, industry will pay back the expenses made for the project faster. Therefore agriculture as a possible occupation on Tiaozini will not be considered in this study.

4.2 Marco-economic analysis

The Jiangsu province is a thriving and well developed economic part of China. In 2010 the Jiangsu province had the second highest total GRP in China of all 23 provinces. With a total of 4143 billion RMB, 10.33% of the GDP is produced in Jiangsu. Another interesting indicator is the GRP per capita. With an GRP per capita of ¥ 52840,- Jiangsu is the fourth highest in China. The average income in Jiangsu is 1.76 times higher than the average of China (Netherlands Business Support Office, 2011). A wealth gap between the prosperous south and poorer north however, has led to unequal economic growth in Jiangsu. Cities like Nanjing, Suzhou and Wuxi have GRP per capita around twice the provincial average, making south Jiangsu one of the most prosperous regions in China (Netherlands Business Support Office, 2011). The Jiangsu province is literally and figuratively part of the heart of China’s economic motor.

4.2.1 Secondary sector as the main sector of Jiangsu

In order to understand the macro-economic situation of Jiangsu, a closer look is needed at the development and division of its GRP. Figure 4.1 shows this development of Jiangsu’s GRP.

![Figure 4.1 - Development of Jiangsu’s GRP over the last 30 years](image)
From Figure 4.1 it is derived that, 52.51% of the provinces GRP is produced in the secondary sector. This sector includes industry and construction. According to the National Bureau of Statistics of China, the secondary and tertiary sectors both show most development. Both sectors perform better in the province than in the nation’s average. With an average annual growth of 13.52% over the last five years, Jiangsu performs better than the nation in any sector or industry (National Bureau of Statistics, 2011).

4.2.2 Specification of the secondary sector into the main industry

The secondary sector contributes the most to Jiangsu’s economic situation. This sector has mainly focused on heavy industry (energy, steel and shipbuilding) and technological industries (electronics, automotive and petrochemical industry). Figure 4.2 shows which industries in the secondary sector of Jiangsu’s economy have the highest industrial output compared to China’s total production. The top 10 ten industries can be interpreted as dominant industries in the Jiangsu province.

Figure 4.2 - Industrial output of Jiangsu in 2010 in aspect to the total production of China [National Bureau of Statistics, 2011]

Figure 4.2 indicates that the IT, electronics and chemical/petrochemical industry are dominant industries in Jiangsu. In order to conclude which industry is efficient and can cause an economic impulse, a closer look is needed to the profits to industrial costs of the dominant industries. Figure 4.3 provides numbers of the 10 most profitable industries in Jiangsu related to its industrial costs. Note that only industries relevant for Tiaozini are taken into account, since the mining of resources for instance will not be applicable on the newly reclaimed land. The highlighted products in Figure 4.3 show that petrochemical industry is most profitable of all dominant industries in Jiangsu.
Figure 4.3 - Ratio of profits to industrial cost for Jiangsu’s industry (Statistical Bureau of Jiangsu Province, 2011)

According to both the economic factors: industrial output and profits to industrial costs, the petrochemical industry is a dominant and profitable industry.

4.2.3 Note of experts

Interviews with professors of the Hohai University reveal preferences for the petrochemical industry. According to information of the Hohai University, a lot of small petrochemical companies are present in the Yancheng district. These small petrochemical companies show economic potential growth. A bigger petrochemical industrial area with adjusted conditions and facilities could lead to a tremendous economic impulse. A possible adjacent port area mainly focused on the petrochemical industry would strengthen its industrial growth. The geographical location of the Tiaozini area for a port area is exceptionally suitable. The adjacent port area will be described later on in this chapter (Jiang Q., 2012).

4.2.4 Macro-economic analysis conclusion

Relying on above mentioned macro-economic statistics, the choice for petrochemical industry seems logical. Taking the comments of experts about the petrochemical industry and its local opportunities into account, the petrochemical industry is a valid occupation for Tiaozini. A closer look at this petrochemical industry is desired to determine its feasibility of realization due investments, growth competences and regional opportunities.

4.3 Petrochemical industry analysis

The literature defines the petrochemical industry in terms of its raw material basis. Petrochemicals are those chemicals that are manufactured from feedstock, and are obtained from oil or natural gas (Todeva, 2000). The boundaries between petrochemical and chemical industries are soft and often unclear. These boundaries are hard to define because of the complexity of the operations and the diversity of products. Often petrochemical industry is seen as a part of the chemical industry and/or process industry. In this study both chemical industry and petrochemical industry refer to the same industry.
4.3.1  China petrochemical market promises steady growth
The domestic industries performance during 2011 was encouraging, with total profit of the Chinese oil and chemical industry touching RMB 807 billion, an increase of 18.8% year-on-year. The gross output value of the Chinese oil and chemical industry rose to RMB 11.3 trillion, increasing by 31.5% year-on-year. Not only China plays a significant role, other emerging markets like India, Africa and Latin America will continue to expand. It is expected that the output of petrochemicals in emerging markets will outpace production in developed countries. According to mid-year projections by the American Chemistry Council, the global output is expected to grow 2.3% in 2012 and 4.3% in 2013 (KPMG, 2012). Overall the petrochemical industry grows and expects future growth.

4.3.2  Automotive gives regional business opportunities
A major consumer of petrochemical products is the automotive industry (KPMG, 2012). Shanghai together with the provinces Jiangsu and Anhui are responsible for around 20% of China’s total car output. Statistics of the National Bureau of Statistics of China show that in 2010 only 4.1% of China’s auto production is located in Jiangsu. This 4.1% doesn’t seem to favor the regional business opportunities for the petrochemical industry, but data shows that The Yangtze River Delta region, which is located closely to Tiaozini and is partly including Jiangsu, is the center for car part manufacturing. It is given that this region is accountable for 44% of the national production (Koninkrijk der Nederlanden, 2012). From these numbers the conclusion can be drawn that petrochemical industry on Tiaozini has a potential market for selling its products. Future developments of the automotive industry should be taken into account in order to give insights into the potency of regional business for the petrochemical industry.

As almost any other sector, the substantial growth of the automotive industry has declined. In 2011 the automotive sales grew 2.5% while in 2010 the growth was 32% (Koninkrijk der Nederlanden, 2012). On the other hand a trend is visible. Automakers continue to shift their production facilities from high-cost regions such as North America and Europe to lower-cost regions such as China, India and South America. China and South America together are projected to represent more than 50% of the growth in global light vehicle production from 2008 to 2015. (KPMG, 2012). It is concluded that the Yangtze Delta will maintain its position as a car part manufacturing output area. It is expected that this will benefit the future potential business for petrochemical enterprises.

4.3.3  Foreign investments and investment climate
The investment climate is an important factor that determines the attractiveness in order to measure the feasibility of a petrochemical industrial area on Tiaozini. In 2011, the paid-in foreign investments in Jiangsu were the highest compared to other Chinese provinces. With a percentage of 27.7 of the total national foreign investment, Jiangsu was ranked number one. Jiangsu had maintained the first place for nine consecutive years. In 2011, there were about 4500 foreign-funded projects being validated in Jiangsu. But Jiangsu enterprises are also operating internationally. By the end of 2011, 2454 overseas projects in 127 countries had been invested in by Jiangsu enterprises. Around 50% of the overseas projects are housing construction and manufacturing industry located in Asia and Africa (Department of Commerce of Jiangsu Province, 2012).

Focusing on the investment climate regarding the petrochemical industry, most recent statistics about 2012 reveal that the investment climate for the petrochemical industry is relatively strong and attractive. In the first quarter of 2012, 202 billion RMB was invested in the petrochemical industry in China, leading to 33.5% year-on-year investment rate (Xinhua, 2012).
The Chinese government recognizes the importance of the petrochemical industry for its national growth. This is shown by the government approving the significant expansion of upstream refining capacity, accelerating the growth of high-end materials and products. This is in favor for the petrochemical industry and its growth (KPMG, 2012).

Another promising development that improves the investment climate is the growing urbanization. This means that investments are made into fixed assets. These fixed assets can be new factories and new infrastructure (China Daily, 2012). In the first five months of 2012, fixed-asset investment, which is seen as the strongest force to drive the country’s economic growth, increased 20.1% compared with the same period last year. (KPMG, 2012). A growing urbanization means that investors are willing to participate in financing a new industrial area in terms of fixed assets. These developments are in favor of a new to build petrochemical industrial area.

4.3.4 Petrochemical industry analysis conclusion

The petrochemical industry shows lots of potential to accomplish the demand for an economic impulse on Tiaozini. The petrochemical industry has a history of stable year-on-year growth and promises future growth. This can be explained by a moderate yet positive growth of the petrochemical major market; the automotive market. The automotive market is mainly located in the Yangtze Delta and has a favorable investment climate determined by foreign investments in particular. The petrochemical industry partly explains the year-on-year growth of the industry. Due to the positive investment climate a new industrial area containing petrochemical enterprises and all its supporting facilities, seems to be financially feasible.

4.4 Port analysis

In section 4.3 the economic relevance of the petrochemical industry is described. The petrochemical industry seems to be a valid occupation to fulfill the economic impulse. According to interviews with experts from the Hohai University, the sea depth of the canal at the coastline of the northern part of Tiaozini is deep enough to be accessibly for big sized bulk carriers, general cargo ships and container ships. This makes the northern part of Tiaozini an interesting location for a port area. In this section a port analyses is made to determine the feasibility of a port area specialized in petrochemical industry. First the term port area will be described more specifically by means of port divisions. Thereafter the importance of port areas in the Jiangsu province will be described in terms of export and import numbers. A geographical analysis is made to give an overview on which port areas are responsible for the import and export. At last a port competition analysis is made to investigate if the Tiaozini port is located in a highly competitive area. The navigability of the port area, port design and port construction are not part of this study.

4.4.1 Port divisions: industrial area and harbor area

The port area can be divided into two different areas: the industrial area and the harbor area. The primary function of the harbor area can be divided into traffic and a transport function. Traffic functions include: nautical accessibility, safe handling of cargo and inland transport connection. The transport function covers the transshipment of cargo. The secondary harbor function can be divided into storage and distribution function, service function (shipping agents/forwarders, finance etc.) and an industrial function. The industrial function focuses on supporting the flow of cargo between the harbor area and industrial area. The harbor area will contain port infrastructure (port basins, quays and docks) and port superstructures (sheds, cranes, vehicles and other equipment).
The main function of the industrial area is facilitating the industries that are located within the industrial area. The area must be accessible, safe and facilitate all needed utilities. The industrial area contains petrochemical industries such as petrochemical plants and/or oil refineries. Clustering of the petrochemical enterprises can offer great business economics and sustainability advantages. The petrochemical industry in particular can benefit from being located close together. The factories can exchange semi-manufactures, residues and residual heat. In the network layer this will be discussed in more detail.

Since the petrochemical industries can be characterized as an industry that heavily relies on import and export of its feedstock and produced goods, it offers economic opportunities for the harbor area as well as for the industrial area when both areas are adjacent to each other. This would meet the initial objective of the occupation on Tiaozini. Both industrial area and harbor area have their own organization, facilities, stakeholders and financial structures. More information about the organization of the port area is given in section 4.5.

4.4.2 Import and export

With the second largest import and export value in China, ports are of great importance to the economy of Jiangsu (National Bureau of Statistics, 2011). The chemical import and export numbers for entire China were positive, with gross value of import and export of the Chinese oil and chemical trade growing in 2011. Exports to all regions expanded 29.5% and imports expanded 26.7% (Chemical & Engineering News, 2012). According to the Department of Commerce of Jiangsu Province, the import of chemical products ranks second with 13.2% of the total imported products in Jiangsu in 2012. The export of chemical products ranked fourth with a modest 5.9% of the overall product export (Department of Commerce of Jiangsu Province, 2012). From China’s overall statistics it is concluded that import and export of chemical and petrochemical products are positive.

4.4.3 Geographically location of port areas

In this section the geographically location of port areas will be given. This geographically distribution of ports will give insights in the competitiveness of the Tiaozini port that will be described in section 4.4.4. In general, port competition can be categorized into six categories (Robinson, 2002) comprising competition between:

- Port ranges or coast lines
- Ports in different countries
- Individual ports in the same country
- Operators or providers of facilities within the same port
- Different (access/egress) modes of transport
- Supply chain

In order to limit the amount of ports to be analyzed, both the competition analysis and the geographical distribution of ports will solely focus on ports located in the Jiangsu province. An exception is made for the Shanghai port, which is not located within the Jiangsu province. The Shanghai port area consists of two main operating areas: Shanghai and Chongming. With a joined capacity of 300MTEU and 563.20Mton, it is ranked number 1 as the world container port and therefore cannot be neglected in this study.
Jiangsu has a long coastal zone with thirteen noteworthy operating port areas (including Shanghai and Chongming). These port areas have different specifications in terms of capacity, function, hinterland connection, industry etc. A map of all sea ports is presented in Figure 4.4. Most of the operating ports in the area are relatively small. There however are several plans to expand the current capacity in these coastal regions. For instance Dafeng, Binhai and Dayang have presented plans to construct a port area. Appendix 0 provides a detailed overview of the specifications of each operating port area in Jiangsu province.

![Figure 4.4 - Port areas located along or near the Jiangsu coastline](image)

### 4.4.4 Port competition

This section indicates whether a port area might be a competitor or a potential partner for the Tiaozini port. In order to do a proper market research of ports and their competitiveness a lot of factors should be taken into account, such as; demand of port services, supply of port services, logistics systems, disaggregation of port traffic by cargo and traffic type, port handling productivity, port pricing, overall port performances and institutional and administrative issues. In this research an explorative competition analysis is made. This means that only a few criteria are taken into account such as; port size, petrochemical activities, maximum ship size and port strategy. The assumption is made that the port area on Tiaozini will be a future seaport that is navigable for 300,000 deadweight tonnage (DWT) vessels.

<table>
<thead>
<tr>
<th>Name port area</th>
<th>Name operating area</th>
<th>Port size</th>
<th>Petrochemical industry</th>
<th>Max. ship size [DWT]</th>
<th>Sea port/river port</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shanghai</td>
<td>Shanghai</td>
<td>Very large</td>
<td>Yes</td>
<td>400000</td>
<td>Sea port</td>
</tr>
<tr>
<td>Chongming</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.1 - Relevant port specifications
Almost every operating port area includes petrochemical activities (Table 4.1). The criterion petrochemical activity only filters out Sheyang port. When looking at the operating port area sizes, all the port areas regarding Yancheng and Lvsi are small. Excluding Binhai and Dafeng port, all the Yancheng and Lvsi operating port areas are river ports. Because of their geographical location, the operating port areas are only navigable for smaller bulk carriers. Bulk freighters like the Panamax, Capesize, Aframax and Handymax cannot navigate to these operating areas. This limits their ability to be a strong competitor against a seaport like the Tiaozini port. These small ports are not considered to be noteworthy competitors. Binhai and Dafeng are both small operating port areas that are currently under construction. These construction plans should not be neglected, because both Binhai and Dafeng are planning to function as a seaport for port bulk handling activities. Shanghai, Nantong and Lianyungang are large cargo port areas with perfect supporting facilities. Shanghai and Lianyungang mainly focus on containerized cargo, whereas Nantong is more focused on bulk cargo. The port area on Tiaozini should rather execute a port strategy that is based on a partnership with these port areas instead of conducting competition. So far the following ports are interesting: Shanghai, Lianyungang, Nantong, Binhai and Dafeng. In appendix 0 more specific information is given about each of these ports.

The Tiaozini port strategy should contain the following focus in order to be a successful port:

- Primarily focus on the import and export of bulk cargo for the petrochemical industry.
- Secondarily focus on inland transshipment and short sea shipping of containerized cargo. The transport movement should include feeder shipments to the large port like Shanghai, Nantong, Lianyungang and Busan.

Considering the above mentioned strategy for Tiaozini port, Dafeng, Binhai and Nantong must be regarded as competitive ports and Shanghai and Lianyungang can be seen as partners. According to the Jiangsu Coastal Development Strategic plan; Lianyungang, Dafeng and Nantong port are considered to be jointly constructed into three strategic supporting points of coastal development. This means that Dafeng and Nantong got recognition from the central government to develop into a competitive port areas. This might temper the possibility for Tiaozini to develop a successful port, but
according to interviews with professors of the Hohai University, Dafeng port suffers from severe sand sedimentation problems closely to Dafeng’s terminals. Dafeng is currently not accessible for huge bulk vessels. This undoubtedly is in favor of the Tiaozini port, because Tiaozini possesses of an advantageous deep sea channel that enables the navigability of 300,000 DWT vessels.

Binhai’s primary occupation consists of energy industry such as power plants for the aluminum industry. The planned bulk cargo will mainly be coal and oil. These are essential feedstock for the petrochemical industry. This port is currently under construction to pursue a port capacity and activities of an international orientated seaport. Like Dafeng, this port is currently not accessible for huge bulk carriers and this doesn’t seem to change within the near future.

Nantong port provides a relative high standard of bulk cargo handling facilities and has a high level of transport network for modal split transport activities. In addition, Nantong shares a significant hinterland area with the future Tiaozini port. Nantong’s strong position might hinder the economic potential of the Tiaozini port. Shanghai and Lianyungang are both ranked in the top 10 of China’s biggest throughput container ports (Cargonews Asia, 2011). Tiaozini port area is no match for Shanghai, which ranks number one in the world. Shanghai and Lianyungang should be seen as partner ports. The partnership could be described as feeder (containerized) cargo towards these port areas.

4.4.5 Port analysis conclusion
From previous sections it can be concluded that the major competitor ports with respect to the Tiaozini port are: Dafeng, Binhai and Nantong. Partner ports are Shanghai and Lianyungang. Shanghai and Lianyungang can be conceived as a springboard for Tiaozini to serve the world market. This can be done by providing feeder services due short sea shipping. However Dafeng and Binhai are currently under construction, these port areas are expected to be relative strong competitors. An advantage of the Tiaozini port is it navigability for large bulk carriers. Binhai’s focuses on different industry than Tiaozini’s. This might have consequences on the level of competition between both ports. Dafeng on the other hands has future plan to develop a petrochemical based industry. Dafeng has the disadvantage of its sedimentation problem near terminals and jetties. The conclusion can be drawn that a petrochemical based port has a reliable chance of being competitive.

4.5 Organization of the port area
The organization can be categorized due the separation of port responsibilities. According to internal publications of Delft University of Technology (Baggen, 2010), there are 4 responsibilities to be distinguished. These responsibilities are; basic infrastructure, port infrastructure, port superstructure and port services. Often a fifth responsibility category is distinguished: infrastructure-plus (e.g., surface hardening and tracks on the terminals) (Dekker, 2005). Each responsibility can be organized by public stakeholders or private stakeholders. The port organization form is mostly determined by the responsibilities; ports services, port superstructure and port infrastructure that are executed by public stakeholders. Figure 4.5 shows three different organization forms based on port responsibilities.
Table 4.2 gives an example of four port areas and its responsibilities organized by private or public stakeholders. The port of Singapore is an example of a port that is publically organized, a so-called service port. Felixstowe port is a good example of a private organized port.

<table>
<thead>
<tr>
<th></th>
<th>Basic infrastructure</th>
<th>Port infrastructure</th>
<th>Port superstructure</th>
<th>Port services</th>
</tr>
</thead>
<tbody>
<tr>
<td>Felixstowe</td>
<td>private</td>
<td>private</td>
<td>private</td>
<td>private</td>
</tr>
<tr>
<td>Hong Kong</td>
<td>public</td>
<td>private</td>
<td>private</td>
<td>private</td>
</tr>
<tr>
<td>Rotterdam</td>
<td>public</td>
<td>public</td>
<td>private</td>
<td>private</td>
</tr>
<tr>
<td>Singapore</td>
<td>public</td>
<td>public</td>
<td>public</td>
<td>public</td>
</tr>
</tbody>
</table>

It shows that private sector participation is often related to the construction of; container terminals, bulk/break-bulk cargo berths, warehouses, container freight stations, tank farms and dry docking facilities (Department of Shipping Government of India, 2006). This has to do with the commercial perspective of the investment. Private funding is related with port-commercial perspectives. These include:

- Maximization of profit
- Maximization of throughput
- Investment recovery (Dekker, 2005)

The petrochemical industry can be characterized as an industry that heavily relies on import and export facilities that are closely located to its plants and refineries. In other words petrochemical enterprises will encounter and demand for an efficient, well organized and competitive port area. This sketches opportunities for public-private partnerships. From section 4.3.3 the assumption can be made that there is a positive investment climate for funding an industrial area containing petrochemical industry. With this information one can conclude that there is a reasonable basis for a public-private partnership construction for a port area on Tiaozini.

For Tiaozini a landlord port structure or tool port structure would be favorable. The advantages of private interference are; the induction of additional resources, a higher efficiency level, a stronger global networking and new business opportunities (Department of Shipping Government of India, 2006). The interference of private enterprises like petrochemical enterprises could ensure a sustainable development.

Given the following facts:
The primarily port strategy is focused on import and export of bulk cargo, mainly for the petrochemical industry. Establishing that petrochemical enterprises share a high commercial interest in efficient operating port area.

The presence of a positive urbanization trend in the petrochemical industry, meaning that terminals, warehouses and port superstructures are fundable.

The petrochemical industry can be characterized as an industry that heavily relies on shipment transportation for import and export. The petrochemical industry shows a stable year-on-year growth and promising future growth drives import and export.

The conclusion can be made that there is potentially enough private funding in order to implement a landlord or tool port organization structure.

### 4.6 Preliminary occupation conclusion

The macro-economic analysis shows that the best suited industry to obtain the economic impulse objective is one of the following industries: electronic, energy and/or petrochemical industry. According to professors of the Hohai University, the petrochemical industry and its local opportunities is the most suited industry for Tiaozini. The analysis on petrochemical industry reveals that this industry has lots of future potential with a stable year-on-year growth. This can be explained by a moderate, yet positive, growth of its major market; the automotive industry. A favorable investment climate, determined by foreign investments, explains the year-on-year growth of the industry. Due to the positive investment climate, a new industrial area containing petrochemical enterprises and all its supporting facilities seems financially feasible. The port analysis gives insights in how competitive harbor areas, adjacent to petrochemical industry, can be in order to be a successful port. The major competitors are Dafeng, Binhai and Nantong. The field of competitors offers room for an additional port area that focuses on petrochemical industry. Due to the characteristic import and export of petrochemical industry together with the positive investment climate, a public-private partnership organization seems possible. This causes a higher level of port efficiency and makes realization of the port feasible. Overall an industrial area containing petrochemical industry, adjacent to a harbor area specialized in bulk-cargo and container feeding services, is a valid occupation for Tiaozini. This occupation meets the economic impulse objective.

### 4.7 Connection with the network and base layer

The preliminary occupation for Tiaozini is the petrochemical industry which can be categorized as heavy industry. Heavy industry projects can be generalized as more capital intensive or as requiring greater or more advanced resources, facilities and management. Due to the capital intensive character of the industry, plants, refineries should be located in safe area. This calls for sufficient measures in terms of flood risk and subsidence. A characteristic of the heavy industry is that often produced goods are sold to other industrial customers instead of the end consumer. This makes the heavy industry part of the supply chain of other products. This in turn makes a good infrastructure for exchanging feedstock and products highly relevant. Also the petrochemical industries can be characterized as an industry that relies heavily on import and export of its feedstock and produced goods (see section 4.4.1).

#### 4.7.1 Conditions for the network layer

A clear condition is given from the occupation to the network layer. Without a suited network regarding freight traffic, passenger traffic, utilities and waste management, the petrochemical area
will not be able to fulfill its business opportunities. This is made complex due to the nature of the industry: low transportability of goods and a high variety in different network infrastructure. In addition to a suited infrastructure with the demanded capacity, flexibility of the infrastructure is required. The industrial area and the infrastructure will constantly be in development, setting rigorous conditions to its flexibility. This development should go hand in hand with a sustainable approach.

4.7.2 Conditions for the base layer

Heavy industry sets condition for the base layer concerning safety and suitability of the site. As is stated in section 4.4.1 the petrochemical should be located in a safe area in terms of flood risks and have a flexible network. The origin of these matters can be traced back to the fact that the petrochemical industry is capital intensive in terms of fixed assets. The petrochemical industry contains industrial processes and products that lead to environmental pollution in case of flooding of the area. The probability of flooding is greatly dependent on the dike design and its design return period. For a flexible network, sufficient site preparation is needed in order to ensure residual settlements of the network are within limits.
Network layer
The proposed occupation for Tiaozini is a port area that consists of an industrial area and a harbor area. The industrial area focuses on petrochemical industry that can be categorized as heavy industry. The petrochemical industry sets specific criteria for the network layer. In order to understand its criteria and consequences, the term network layer should be described in more detail.

The network layer consists of both physical infrastructure and invisible connections. The physical infrastructure controls and steers the growth of mobility such as traffic and transport flows. The physical infrastructure is the set of roads, railways, waterways, ports, airports, transfer- and transshipment points. Invisible connections are mainly the underground pipes and cables used for ICT and utilities supply. In this study attention is only paid to the physical infrastructure.

Section 5.6, gives a brief introduction about what type of criteria the occupation sets for the network layer. Unquestionably the list of criteria is much bigger than that is initially given in section 5.6. Due to time limitations only a limited set of criteria will be described in this chapter. To pinpoint interesting noteworthy criteria a general understanding of the specific activity of the occupation and the effects for a transport system is needed. In the following section the coherence between the activities of the petrochemical industry and its infrastructure is described.
5 Tiaozini transport network

5.1 Coherence of the activity system & transport system

The activity system and the transport system always show coherence with each other. Short-term effects of this coherence can be coordinated at micro level and long-term effects can be coordinated at macro level. On micro level the coherence is determined by activity patterns like the productions of petrochemical products and transfer/transport patterns of feedstock or produced goods. On macro level the coherence is determined by the spatial structure of the industrial area like clustering of plants and the existing infrastructure (Schoemaker, 2002). To simplify the micro and macro level:

- Macro can be reduced to the spatial distribution of human activities
- Micro can be reduced to actions of individuals or in this case individual organizations / companies

On Tiaozini the spatial structure can be described as a clustering of petrochemical enterprises. This calls for focus on macro level.

5.2 Clustering

Clustering is one of the key concepts regarding the industrial area on Tiaozini. The benefits of clustering can be explained by so called economies of agglomeration. Economies of agglomeration describes that firms that cluster together may get benefits in terms of transport costs, transport time and overall production costs (Schoemaker, 2002). The petrochemical companies in particular can benefit from being located close together for exchanging semi-manufactures, residues and residual heat. Firms can also share the same infrastructure. This infrastructure can for example be roads, conveyers, railways etc. The petrochemical industry is an industry that needs infrastructure for another primary reason besides supplying feedstock and utilities or transporting their produced good. This third primary reason is infrastructure for its waste management. The eco-friendly approach of reducing emissions by clustering is being embraced by the Chinese government. Chemical parks are becoming more and more important to meet the targets of carbon emissions. The traditional parks in Shanghai, Nanjing and Tianjin have consistently been upgraded and serve as best practice examples for new environmental regulations (KPMG, 2012). Clustering also has some disadvantages and leads to some challenging issues. Petrochemical enterprises most of the time need to be located near deep water. In the case of Tiaozini the following question is highly relevant: do you locate the companies close together on relatively expensive land because this will benefit the exchange of semi-manufactures, residues and residual heat? Or is a greater distance between the enterprises acceptable and affordable to construct an environmentally friendly transport system instead? In order to answer such question, insights of future developments of the area and the industry as well are relevant. A disadvantage of clustering activities is that it can lead to traffic congestion and pollution due to a high ratio of polluting enterprises within an area. These factors eventually will decrease the pricing power of firms.

5.3 Four focal points for the network layer

Each occupation will set different criteria for its supporting network. These criteria depend on a variety of factors. Important factors for example are: the size of the occupation, frequency of movements, phase of development, located area of the occupation, availability of financial resources, laws and regulations etc. When looking at the occupation on Tiaozini the following factors are relevant because of the current construction phase:
The possible expansion of the occupation due to market growth and foreign investments
The characteristic of the petrochemical products and their supply chain, which enables different types of transportations (modal split)
Strict laws and regulations on emission
Clustering of petrochemical enterprises

The above mentioned factors determine the 4 following criteria that are interesting to describe in more detail:

- **Modal split:** a network containing different types of transportation
- **Flexibility:** a network that is able to respond to fast developments
- **Sustainability:** a network that minimizes the emissions and transport costs due to waste management
- **Capacity:** a network that meets the required traffic demand

In the following section each criteria will be described according to a case study on the recently constructed Second Maasvlakte.

### 5.4 Second Maasvlakte case study

This case study is relevant because the Second Maasvlakte is a recently reclaimed land that has many similarities with Tiaozini. The case study gives insights in how network related processes, issues and opportunities are embraced and implemented. Firstly an introduction is given of the reclaimed area of the Second Maasvlakte. After the introduction the four criteria from section 5.3 are described. This is done by referring how these criteria were conceived and carried out by policymakers from various administrative levels. With the help of this descriptive case study insights are given in how the network layer should interact with external influences like the growth of the industry and supplementary traffic demand.

#### 5.4.1 Brief introduction of the Second Maasvlakte

The port of Rotterdam is currently ranking number four on the international port list, making the port of Rotterdam the most important port of Europe. Every year 400 million tons of cargo enters the port. The port of Rotterdam is in urgent need of additional space. The existing port and its industrial area cannot meet up with the spatial demand for new large-scale activities. The second Maasvlakte provides the solution for this problem. By creating a new port area in the North Sea, the port of Rotterdam can continue to develop. It needs to continue developing, in order to maintain its strong competitive position with respect to its rival ports Hamburg and Antwerp (Port of Rotterdam, 2012). The construction of the second Maasvlakte started in 2008 and is planned to be operational in 2013. The construction area is located at the West of the existing Maasvlakte. The Maasvlakte is a large industrial area containing oil terminals, APM terminals, a coal and biomass power plant and refineries. The area is located at the estuary of the river Maas which flows into the North Sea. The Maasvlakte is marked green in Figure 5.1 and the newly constructed second Maasvlakte is marked yellow.
The second Maasvlakte has a total surface area of approximately two thousand acre (20 km²). There is room for three sectors: container industry, chemical industry and distribution. For the construction more than 325 million cubic meters of sand is needed. This cannot be realized instantly; in fact the new land does not have to be constructed at once. The second Maasvlakte is realized in a phased construction process. This makes the construction of the second Maasvlakte flexible and vital to react on growth and future developments. Figure 5.2 gives a visual impression of the second Maasvlakte.
5.4.2 Modal split
The distribution to and from the hinterland according to the type of transportation is called modal split. When a port area wants to optimize its benefits of a deep sea harbor that is 24/7 accessible, it needs to prioritize the quality of the hinterland connection. Container transport will have the largest share of the overall traffic flows on the second Maasvlakte. The transport flows of the other port sectors are very strongly linked to a modality. An example is dry bulk, of which more than 80% is transported by inland shipping. When referring to the second Maasvlakte and its container transport, almost 30% of the container cargo will be shipped over sea. The remaining 70% is transported by inland shipping, road- and railway. Objectives for the modal split of the second Maasvlakte from 2005 to 2033 are:

- Road from 47% to 35%
- Inland shipping from 40% to 45%
- Railway from 13% to 20%

As is shown the share of road transport is estimated to drop with 12%. The share of freight traffic will be reduced by the development of container transferiums and agreements with terminal operators regarding the reduction of freight by road (Port of Rotterdam, 2012). Furthermore the ratio of container per carrier will increase as well as the utility of each carrier is expected to increase. The percentage reduction of transport by road will be compensated by the transportation types: inland shipping and railway. Expected is that the chemical industry will be responsible for the inclining share of inland shipping transportation (TNO, 2008).

In terms of eco-friendly modal split, often is spoken of a low share of road transportation. This is not strange, because modal split is one of the most important parameter to determine the environmental impact of traffic. The second Maasvlakte pursues a reduction of road transportation in order to meets its sustainability criterion to reduce CO₂ emission. More about this sustainability criterion is described in section 5.4.3.

Tiaozini will have a different distribution of the modal split shares in comparison the second Maasvlakte, but policymakers can learn from its ambition to reduce road transportation. As is stated before, transport flows from specific port sectors are very strongly linked to a type of transportation. Tiaozini’s occupation exists mainly of petrochemical industry. This sector is primarily supported by the transportation types: (inland) shipping and railway. This offers opportunities to design a road infrastructure that does not have to apply to a high standard capacity level. It would benefit and support the modal split share of railway when a high level capacity railway would be constructed.

5.4.3 Flexibility
As is stated in section 3.1, Tiaozini has a planned total area of 266.7 km². Same as the second Maasvlakte, this reclamation is phased. At this moment only 30 to 50% of the total 266.7 km² is reclaimed. In contrast to the second Maasvlakte, the Tiaozini area is not yet suited for its occupation. The soil of the current reclaimed area is not ready for construction. Petrochemical industry will only make up for a small part of the total area on Tiaozini. The industrial area and harbor area, together described as the port area, will be positioned at the northern part of Tiaozini. This offers opportunities to design a road infrastructure that does not have to apply to a high standard capacity level. It would benefit and support the modal split share of railway when a high level capacity railway would be constructed.
the occupation demands a bigger activity area. The interaction between the activity area and the transport system is described in section 5.6.

The Master Plan, which describes the realization of the second Maasvlakte, is clear about flexibility; an important point of departure was maintaining flexibility: lay down what needs to be laid down, but where possible, keep your options open. For this reason, the sites along the outer contour will not be constructed in the first phase, so that future changes in current insights can be effectively addressed. The infrastructure can grow along with the capacity requirement and the client’s demands. The design for the second Maasvlakte needs to be sufficiently flexible to allow for an effective response to future developments in the market, such as deviations from the forecast growth in container transfer, the chemical sector and the distribution sector. Sites that have been assigned to container companies in the Master Plan but that have not yet been granted have been designed so that they can also accommodate clients working in the chemical industry, and vice versa (Port of Rotterdam, 2012).

An interesting concept that can be implemented when designing the transport network on Tiaozini, is the concept of leaving the outer industrial contours free of construction. When a higher capacity supply is demanded, the outer contours can be used to fulfill the capacity demand. A higher capacity supply is probably demanded by the modality road transport. This is not strange, because road transport is by nature the most flexible modality. Road haulage will always remain important for short distance transport due the flexibility of the mode of transport (Port of Rotterdam, 2012).

5.4.4 Sustainability

Government regulations play an important role in the implementation of a sustainable network. The intervention, laws and regulations of the Dutch government are different from those of the Chinese government. The differences can be found in the measures that are taken and their priority to strive for a sustainable network. In a general sense there are sustainable concepts that apply to both Tiaozini and the second Maasvlakte. In this section an attempt is made to give examples on how a more sustainable network can be achieved.

Regarding the second Maasvlakte, attention is paid to noise pollution, emission and toxic components. The past years the focus increasingly shifted to CO\text{2} emissions. For a sustainable second Maasvlakte, it will be important to develop visible initiatives for sustainable mobility, such as promotion of biofuels (TNO, 2008). Another development that contributes to a more sustainable network is the strict selection procedure of enterprises that want to establish themselves at the second Maasvlakte. Sustainability was an important selection criterion in the international tendering procedure for establishing stevedores and shipping companies at the second Maasvlakte. As a result, the second Maasvlakte has become a showcase for companies which demonstrate that sustainability and economic growth can go hand-in-hand (Port of Rotterdam, 2012). For the network layer this means that enterprises take own responsibility for cleaner transport and sustainable waste management.

When looking at Tiaozini and its petrochemical occupation, waste management takes on a huge priority. This is because manufacturing of many petrochemicals and other types of products results in most cases in the generation of substantial quantities of hazardous and toxic materials. The transportation cost of any waste management system is considered to be one of the major costs within the production of petrochemical products. (Abdulaziz, 1996). By supporting or facilitating infrastructure that is accessible for more than one enterprise and its waste management process, a
transportation flow can be sustainably designed in terms of advantages of economies of scale and efficiency.

Tiaozini can consider a sustainable layout like the second Maasvlakte did. A sustainable layout can be realized by encouraging transport of energy and other products through a network of pipelines. In this way, less transport by road and railway is needed. A criterion that goes hand in hand with encouraging another transport mode is the modal split which is described in section 5.4.2. By aiming to reduce road traffic, by providing good facilities for rail transport and inland shipping, a more sustainable network can be achieved.

5.4.5 Capacity

Accessibility is the ultimate goal of most transportation activities. This makes a well-functioning and accessible transport network highly relevant. When problems on the transport network arise such as congestion, the transport network fails and areas get less accessible (Litman, 2011). The capacity supply is most of the time at the center of attention when problems of congestion occur. Capacity can be defined as; the maximum number of items that can be squeezed through a system or its components per unit of time (Liang, 2009). In this section an illustration is given in how the second Maasvlakte manages sufficient capacity for its transport network. Also a reference is made to interesting elements for the transport network on Tiaozini.

A critical factor regarding the second Maasvlakte is the reliability of the road system. It is expected that the travel time will be more volatile after 2020 due to the higher utilization of the infrastructure in combination with a broader rush period. Consequently measures are taken to increase the capacity of the enabling traffic network. An important track, Beneluxplein – Vaanplein, will be constructed from a 2 x 3 lane track to 2 x 3 + 2 x 2 lane tracks. By unbundling this part of the network with an extra 2 x 2 track, it is possible to separate traffic flows. As a result a part of the congestion will be deducted (TNO, 2008).

The unbundling of traffic flow is a key element in the capacity control and capacity management of the second Maasvlakte. This unbundling could also serve as a key element for the transport network on Tiaozini. When troubles regarding capacity arise, the unbundling enables more targeted capacity enlargement specifically matched for a particular traffic flow. The unbundling of traffic flows starts with a main road and a secondary road. Parallel to the main road, plans outline a second road for recreational traffic and slow-moving traffic like tractors and shovels. This road will furthermore serve as an emergency route (Port of Rotterdam, 2012). The master plan of the second Maasvlakte reserved space along the entire outer rim for an internal route for container transport. This will relieve pressure on the main and parallel second roads. Because of such a ‘specialized’ container-road network, the containers that are transported from the container terminals to container depots, rail and inland shipping terminals will be kept off the public roads. Such a specialized container track might be interesting for Tiaozini.

An important element that can serve as inspiration for the transport network on Tiaozini is the internal transport network on the second Maasvlakte. The internal transport network facilitates local distributor roads on the large industrial sites allocated to the chemical and distribution sectors. This result in lower transport costs for the petrochemical industry and benefits the regional trade.
5.5 Tiaozini transport network conclusion

The following four criteria are interesting to take into account when a transport network is designed for Tiaozini; modal split, flexibility, sustainability and capacity. These criteria are determined by the characteristics of Tiaozini’s occupation; petrochemical industry. The first characteristic is the possible expansion of the industry due to market growth and foreign investments leading to the requirement of a flexible network. The second characteristic is that the petrochemical products and their supply chain enable different types of transportation. This sets a certain modal share in favor of rail and inland shipping. Thirdly, strict laws and regulations are set by the government on emissions. The regulations will demand a certain level of sustainability regarding the traffic network. Finally, the clustering of petrochemical enterprises requires a certain level of capacity control and capacity management.

Interesting network concepts that can be found in the development of the transport network of the second Maasvlakte are:

- A modal split that emphasizes a high share of rail and inland shipping transportation. This is possible because the petrochemical industry is mainly supported by those transportation types.
- By keeping the outer contour of the industrial area free of constructions, the transport network is able to effectively respond to future developments in the market, such as deviations from the forecast growth in container transfer, the chemical sector and the distribution sector.
- By supporting and/or facilitating a waste management infrastructure that is accessible for more than one petrochemical enterprise and its waste management process. The waste management process can attain a higher efficiency and eco-friendly level. This will overall benefit the sustainability of the transport network.
- Unbundling traffic flows from the main road. This causes a more targeted capacity enlargement, specifically matched for a particular traffic flow. Also a higher level of service due to less congestion can be obtained. An example of this is the unbundling of internal cargo to and from enterprises within the industrial area.

5.6 Connection network layer and base layer

A link between the network and base layer can be found when looking at the way the transport network is carried by the base layer. Being more specific, the transport network is in this case a six lane road (2 x 3) that functions as the main enabling road network of the Tiaozini area. A critical note has to be made regarding the decision of the capacity of the main road. There is no objective analysis performed to conclude of the six lane road will meet the desired capacity for the traffic demand. The virtual construction of a six lane road is based on interviews and firsthand information during the site visit (Appendix B). Also the assumption is made that the cargo, which is transported on the main road, can be categorized as heavy. In chapter 6 a preliminary design of the six lane road is given.
6 Preliminary road design

As stated in the previous section, an assumption concerning the required capacity of the enabling network is largely made based on firsthand experience. As a result a six lane road is designed to provide sufficient capacity to the transportation network. The proposed road design of a one-way section is given in Figure 6.1. Based on this drawing it is concluded that the total width of a one way section is 18.75m. Consequently the total width of the road is 37.50m; this excludes the extra space required for the slopes one both sides of the road. A detailed drawing of the road design including the pavement structure is given in the Appendix 0.

![Figure 6.1 - Cross section of a one-way road section](image)

In accordance with the New Design Guide Motorways (Rijkswaterstaat Adviesdienst Verkeer en Vervoer, 2007) the inclination of the road surface is 2.5%. This gradient is necessary to ensure adequate drainage of the motorway. There are two other dimensions which will have to be taken into account: Clearance and Obstacle free zone. The latter of the two is governing with a minimum width of 13.0m for a 120km/h highway. This zone includes the verge, the hard shoulder and the adjacent embankment slope. The obstacle free zone is installed in order to ensure the safety of both car passengers and traffic participants using the underlying and opposite roads. An obstacle free zone is preferred over a crash barrier, since a crash barrier itself can also cause damage to the car passengers in case of a crash. For the slopes to be part of the obstacle free zone, the maximum slope angle is set to be 1:6. The required buildup, thickness and densities of the pavement structure are given in Table 6.2.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Type</th>
<th>Density [kg/m³]</th>
<th>Thickness [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface layer</td>
<td>Stone Mastic Asphalt D= 0 – 11 mm</td>
<td>2450</td>
<td>35</td>
</tr>
<tr>
<td>Intermediate</td>
<td>Asphalt Concrete Dₘₐₓ=16 mm</td>
<td>2200</td>
<td>50</td>
</tr>
<tr>
<td>Bottom layers</td>
<td>2x Asphalt Concrete Dₘₐₓ=22 mm</td>
<td>2200</td>
<td>140</td>
</tr>
<tr>
<td>Base</td>
<td>Hydraulic mixed granulate</td>
<td>1800</td>
<td>400</td>
</tr>
<tr>
<td>Sandbed</td>
<td>Sand</td>
<td>1850</td>
<td>525</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1150</td>
<td></td>
</tr>
</tbody>
</table>

6.1 Design loads

In the Netherlands the maximum axle load is set to be 115kN. Loads of up to 200kN however occur regularly in European traffic. With exceptions even higher axle loads are measured. For the design
traffic load NEN6788 is used, according to this standard the heaviest class is a three axle truck with a total load of 600 kN divided over its axles, as presented in Figure 6.2.

However the axle forces are useful, wheel loads and surface contact pressures are more important. The wheel loading is represented as a point load. The governing load is 50kN per wheel, with a surface pressure of 850kPa. This pressure is generally assumed to be similar to the air pressure inside the tire. In order to make the design calculations, these point loads are modeled as a uniformly distributed load over the full width of the road. Taking a spread angle load of 45˚ for the point load, the loaded area under the pavement can be calculated using voorschriften voor het ontwerpen van stalen bruggen (VOSB-classes) (Infra, 2007).

Because the roads on Tiaozini will mainly be used by industrial traffic for the heavy industry, the assumption is made that the heaviest loading will apply. This means that conform VOSB class 600 and with a pavement depth of 1.15m the total uniformly distributed traffic load is 31kN/m².

6.2 Residual differential settlements
Construction of an embankment on soft soil may lead to severe settlements. Settlements of up to 40% of the height of the embankment have been recorded (Infra, 2007). These settlements are time dependent; first instant settlements occur due to rearrangement of grains, secondly consolidation takes place by the dissipation of excess pore water and thereafter creep takes place. For roads it is important that the residual settlements are within limits. For the purpose of this research CROW’s requirements for residual settlements for different kind of infrastructures will be used (see Table 6.2).

<table>
<thead>
<tr>
<th>Infrastructure</th>
<th>Residual settlements [mm]</th>
<th>Period [year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highway</td>
<td>&lt; 100</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>&lt; 300</td>
<td>30</td>
</tr>
<tr>
<td>Road</td>
<td>&lt; 400</td>
<td>30</td>
</tr>
<tr>
<td>Airport</td>
<td>&lt; 30</td>
<td>30</td>
</tr>
<tr>
<td>Train</td>
<td>&lt; 300</td>
<td>30</td>
</tr>
<tr>
<td>High-speed train</td>
<td>&lt; 30</td>
<td>100</td>
</tr>
</tbody>
</table>

The residual settlements should not exceed these requirements. Of greater importance than residual settlements are the residual differential settlements (see Figure 6.3). Differential settlements in roads may lead to inequalities in longitudinal direction that leads to a reduction of driving comfort and lower safety. The maximum residual differential settlements are 50mm over each 25m in longitudinal direction (Rooduijn, 2011).
For reliable predictions on settlements a model that describes the previously determined effects accurate enough is needed. For residual settlement creep is the most important for a good fit on measurements. D-Settlement is a software program which can describe consolidation on bases of permeability (Visschedijk, Onderhoud Door Restzetting: Meten is Weten, 2006).

6.3 Conclusion
In order to realize the petrochemical industry on Tiaozini, it is proposed to construct a 6-lane road network, taking into account the possible future developments and rapid expansion of the industrial occupation. These roads will be constructed in accordance with the New Design Guide Motorways (Rijkswaterstaat Adviesdienst Verkeer en Vervoer, 2007). The roads will be constructed on top of an embankment. This is done to ensure a high level of safety and a high level of comfort for the users of the road. Moreover the raised embankment causes the road to remain accessible during a flood.

The traffic induced loads are calculated using voorschriften voor het ontwerpen van stalen bruggen (VOSB-classes) (Infra, 2007). For VSOB class 600 and a pavement depth of 1.15m the total uniformly distributed traffic load is 31kN/m².

The residual settlements are determined using CROW’s requirements for residual settlements for different kind of infrastructures (see Table 6.2). Residual settlements should be less than 100mm over the first 10 years and less than 300mm over the first 30 years. In addition to the residual settlement, there should be no more than 50mm differential settlement over each 25m in longitudinal direction (Rooduijn, 2011).
**Base layer**

Based on the analysis made in the occupation layer, it turns out that petrochemical industry is the best suitable industry to settle on the newly reclaimed land of Tiaozini. Petrochemical industry is able to provide a long lasting significant economic impulse in the province. As described in the network layer, a flexible transportation system with high capacity is desired. A six lane road will allow subsequent growth. In order to prepare Tiaozini for this occupation and network, changes to the base layer have to be made.

The base layer section emphasizes the geotechnical and hydraulic engineering actions required to ensure a safe, accessible area which is suitable for petrochemical industry. In order to do so, all the current soil properties of Tiaozini will be defined. Secondly a suitable consolidation method for soft soil layers is determined. Finally, numerical models as well as analytical calculations will be used to verify the settlement criteria for the embankment design created in the previous section.

After having designed the desired consolidation method, the dikes are reconsidered. Since the original dikes are designed to protect low grade agricultural land (Planning Team, 2009) a drastic change is necessary in order to ensure the protection of the proposed high grade industry. The principles of geo-tube dikes and its construction will be discussed, after which the 1:1000 year wave heights will be calculated to determine the new dike height. Finally a preliminary dike will be designed. With the use of numerical models the stability of this new dike design is assessed.
7 Soil properties
The Jiangsu Delta is a tidal area which acts as a node for different tidal currents (Figure 7.1). Due to this environment a large amount of sediment is deposited. The location where the Tiaozini project is located has a natural land accretion of 200 m seawards every year (Gong, 2012). The tidal difference between high and low tide on average is 4.5 meters. With the use of dikes, the place of the deposited sediment is established in order to make one large peninsula.

![Figure 7.1 - Tide currents, (a) flood current, (b) ebb current](image)

Table 7.1 and Table 7.2 show the soil profile and properties presented in the top layers of the reclaimed land. All properties are verified using literature and match with reasonable values for such material descriptions. However to obtain and use these datasets, some assumption had to be made.

- Due to the fact that just one dataset is available and there is no time for additional ground investigation, it is assumed the whole Tiaozini area has the same soil layering, with the same soil properties. In reality material properties can vary greatly over small distances. This assumption leads to an overestimation of safety because failure often occurs through the adjacent weaker zones.
- The soil properties of the existing seabed are taken from the Tiaozini feasibility study created by Hohai University (Hohai University, 2010). The properties of the new deposited sediment layer, which is being deposited at the time of writing, are still unknown. Therefore the properties of a similar natural deposit of a reference land reclamation project north of Tiaozini are used.
- Assumptions on the thickness, groundwater level and speed of deposition of the new deposited layer are discussed below.

The authors are well aware of the fact that these properties might not occur in reality. Due to lack of sufficient ground investigation on the local soil properties, these assumptions however present the best acquirable data sets.

7.1 Thickness of new deposited sediment layer
The average seawater level is + 0.25m Chinese Standard Water Level (CSWL) and the average high tide is + 2.39m in the Tiaozini area (Hohai University, 2010). Based on correspondence with Professor F. van Tol it is assumed that the top elevation of the deposited layer is equal to the average high tide
elevation since sedimentation is strongest at high tide (Tol, 2012). In reality the top elevation of the sediment layer at the Tiaozini area is strongly dependent on the duration of the sedimentation process. It might not be economically feasible to let the sedimentation process last for as long as it is needed to reach + 2.39m. If the dike is closed quickly after construction of the sediment dikes, the top elevation of the sediment layer will be significantly lower than the average high tide elevation. For the sake of this study however, it is assumed the closure dike is kept open long enough to ensure a top elevation of + 2.39m for the deposition layer.

7.2 Groundwater level in new deposited sediment layer
Due to the fact that the newly deposited sediments are transported by sea, they are saturated at deposition. The groundwater table however is assumed to be equal to the average seawater level (+ 0.25m). This will result in a desaturation of the new sediment layer that is deposited above + 0.25m.

7.3 Speed of deposition new sediment layer
During the site visit (appendix B) it is found that for the already finished southern part of Tiaozini it took 2,5 years to deposit 5,5m of new sediment. Assuming the sediment rate for the northern part of Tiaozini is equal to the southern part (6mm/day). This assumption can be made due to the fact that the sedimentation process is the same for both parts.

Table 7.1 - Tiaozini soil profile

<table>
<thead>
<tr>
<th>#</th>
<th>Material</th>
<th>Condition</th>
<th>Grain type</th>
<th>Depth [m]</th>
<th>Thickness [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Clay</td>
<td>Silty clay</td>
<td>fine</td>
<td>2.93</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Sand</td>
<td>Light silt loam sand</td>
<td>fine</td>
<td>- 3.11</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>Sand</td>
<td>Light silt loam sand</td>
<td>fine</td>
<td>- 4.71</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>Sand</td>
<td>Silty sand</td>
<td>fine</td>
<td>- 6.50</td>
<td>&lt;</td>
</tr>
</tbody>
</table>

Table 7.2 - Tiaozini soil properties

<table>
<thead>
<tr>
<th>#</th>
<th>w  [%]</th>
<th>Y_sat [kN/m³]</th>
<th>e  [-]</th>
<th>E_s [MPa]</th>
<th>c' [kPa]</th>
<th>ϕ  [°]</th>
<th>k_v [m/s]</th>
<th>k_h [m/s]</th>
<th>N [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>41.7</td>
<td>18.0</td>
<td>1.17</td>
<td>1.91</td>
<td>10.0</td>
<td>25.0</td>
<td>1.80E-09</td>
<td>3.60E-09</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>32.5</td>
<td>18.5</td>
<td>0.88</td>
<td>4.49</td>
<td>11.0</td>
<td>15.2</td>
<td>2.02E-06</td>
<td>3.09E-06</td>
<td>5.30</td>
</tr>
<tr>
<td>3</td>
<td>30.2</td>
<td>18.8</td>
<td>0.82</td>
<td>6.98</td>
<td>11.0</td>
<td>23.5</td>
<td>2.13E-06</td>
<td>2.66E-06</td>
<td>9.80</td>
</tr>
<tr>
<td>4</td>
<td>28.6</td>
<td>19.0</td>
<td>0.78</td>
<td>9.45</td>
<td>9.0</td>
<td>31.0</td>
<td>7.26E-06</td>
<td>1.00E-05</td>
<td>22.30</td>
</tr>
</tbody>
</table>
8 Preliminary embankment design

8.1 Consolidation methods

The new reclaimed land will consist of very soft soil. This may lead to increased risk of foundation failure and/or high settlements due to the low strength, high compressibility, prolonged creep and low permeability of the soil. To fulfill the residual settlement requirements for the road as described in chapter 6 the soil characteristics need to be improved.

Since the majority of the settlements will be inflicted in the softer top layer, most compaction methods are not applicable in this case (see Table 8.1). Most emphasize will therefore be on consolidation methods for soft fine soils. The relevant methods will be discussed briefly in this chapter. More detailed information about the different compaction and consolidation methods can be found in appendix F.

Table 8.1 - Effectiveness of different compaction and consolidation methods

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Sand</th>
<th>Silty Sand</th>
<th>Silt</th>
<th>Silty Clay</th>
<th>Clay</th>
<th>Peat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic Compaction</td>
<td>++</td>
<td>+</td>
<td>-</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Vibro-Compaction</td>
<td>++</td>
<td>+</td>
<td>-</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Surcharge Preload</td>
<td>--</td>
<td>--</td>
<td>0</td>
<td>+</td>
<td>++</td>
<td>++</td>
</tr>
<tr>
<td>Vertical Drains</td>
<td>--</td>
<td>--</td>
<td>0</td>
<td>+</td>
<td>++</td>
<td>++</td>
</tr>
<tr>
<td>Electro Osmosis</td>
<td>N/A</td>
<td>N/A</td>
<td>-</td>
<td>++</td>
<td>++</td>
<td>++</td>
</tr>
<tr>
<td>Admixtures</td>
<td>Can be excellent for all soil types - dependent on the choice of additive</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Consolidation is a process which involves the decrease of water content of a saturated soil without replacement of water by air (Terzaghi, 1943). Adding additional stress or reducing the pore pressure will lead to a higher effective stress and hence a denser state of packing. For saturated soil this results in water being squeezed out of pores. The total settlement can be divided into three components;

\[ S_{tot} = S_{in} + S_{prim} + S_{cr} \]

In which \( S_{tot} \) is the total settlement and \( S_{in}, S_{prim}, \) and \( S_{cr} \) stand for the instantaneous, primary and creep settlement respectively.

Generally there are five separate methods of consolidation:

- Surcharge induced preloading
- Prefabricated vertical drains (PVD)
- Vacuum preloading
- Electro osmosis
- Admixtures

Often these methods are used in combination with each other, to achieve the best result. However this completely depends on the existing soil and project properties.
8.2 Multi-Criteria Analysis

In this section an advice will be given on what consolidation method will be most favorable for the Tiaozini reclamation. The most suitable method will be found using a multi criteria analysis (MCA). In order to determine the most suitable method, all methods are compared using the same set of criteria. These criteria are chosen in such a way that they cover all different demands required for a proper consolidation technique and do not overlap with each other. The different criteria are listed and explained below. For a more detailed description of the MCA see appendix I.

- Speed
- Costs
- Technological complexity
- Reliability
- Applicability on large scale
- Effectiveness
- Durability
- Sustainability

All methods might have a different impact on the natural environment. This criterion takes into account the different impacts. Not all criteria are of the same level of concern. They are unequally weighted with a weight factor. The result of the multi criteria analysis is given in Table 8.2. The total score is found by multiplying the separate scores given to the different criteria, with the corresponding weight factor.

Table 8.2 - Result of MCA for different consolidation methods

<table>
<thead>
<tr>
<th>Method</th>
<th>Total score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surcharge induced preloading</td>
<td>3.2</td>
</tr>
<tr>
<td>Vertical drains</td>
<td>4.0</td>
</tr>
<tr>
<td>Vacuum preloading</td>
<td>2.2</td>
</tr>
<tr>
<td>Electro osmosis</td>
<td>-3.4</td>
</tr>
<tr>
<td>Admixtures</td>
<td>-1.9</td>
</tr>
</tbody>
</table>

Based on these results, it is concluded that the vertical drain consolidation method is the most suitable method for the Tiaozini reclamation project. As stated before, the vertical drain method is often used in combination with surcharge induced preloading. The latter also has an attractive score when considering the MCA results. Both the surcharge preloading as well as the drain method will therefore further be assessed.

8.3 Numerical residual settlement calculation: D-settlement

For the residual settlement calculations for the embankment, D-Settlement software will be used. D-Settlement is a program designed by Deltares to determine settlements. It is especially designed for predicting embankment settlements. For a detailed description on D-Settlement, the consolidation models, the calculation models, the model parameters and the limitations of the software see appendix 0.
8.3.1 Consolidation model
D-Settlement allows for two different consolidation models, Darcy and Terzaghi respectively. The latest version of D-Settlement includes an improved version of the Darcy model with considerably less computation time, support of the same input as the Terzaghi model, improved submerging model and a significant increase in robustness (Visschedijk & Trompille, D-Settlement Version 9.3, 2012). The Terzaghi model has a few limitations compared to the Darcy model. For this reasons the Darcy model will be used.

8.3.2 Calculation model
A choice has to be made out of the following three calculation models; NEN-Koppejan, NEN-Bjerrum and Isotache. The NEN-Bjerrum parameters can be determined by using an oedometer test in combination with the void ratio. The model is applicable for staged loading and it allows unloading/reloading behavior. NEN-Bjerrum is an internationally known method for calculating settlements and is therefore chosen over NEN-Koppejan which is only accepted in the Netherlands. Due to the fact the Isotache parameters require both an oedometer as well as a CRS test, NEN-Bjerrum is chosen over the Isotache model.

8.3.3 Model Parameters
In order to use D-Settlement for settlement calculations, the model parameters need to be defined. A precise determination and good understanding of these parameters is essential for an accurate result of the software.

For NEN-Bjerrum the compression and swelling ratio is needed as well as the coefficient of secondary compression \((CR, RR \text{ and } C_a\) respectively) need to be determined. With the available laboratory test performed at Hohai University an accurate determination of \(CR, RR \text{ and } C_a\) is performed for the new deposited layer. These values are checked using Table H.1 by fitting the values and the material description. Due to the lack of sufficient laboratory tests the NEN-Bjerrum parameters of the other layers are solely based on engineering judgment and Table H.1.

Because the soil layers are all relatively young (lot of sediment is deposited every year) it is assumed that all soil layers are currently experiencing their highest stress, so all over-consolidation ratio’s (OCR) are equal to 1.0. An overview of the parameter used in D-Settlement is given in Table 8.3.

<table>
<thead>
<tr>
<th>#</th>
<th>(Y_{sat}) [kN/m³]</th>
<th>(Y_d) [kN/m³]</th>
<th>CR [-]</th>
<th>RR [-]</th>
<th>CA [-]</th>
<th>(k_{vp}) [m/s]</th>
<th>OCR [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18.0</td>
<td>12.7</td>
<td>0.1399</td>
<td>0.2594</td>
<td>0.0141</td>
<td>1.80E-09*</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>18.5*</td>
<td>16.5</td>
<td>0.0042*</td>
<td>0.0042*</td>
<td>0</td>
<td>2.02E-06*</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>18.8*</td>
<td>16.8</td>
<td>0.0077**</td>
<td>0.0077**</td>
<td>0</td>
<td>2.13E-06*</td>
<td>1.0</td>
</tr>
<tr>
<td>4</td>
<td>19.0*</td>
<td>17.0</td>
<td>0.0115**</td>
<td>0.0115**</td>
<td>0</td>
<td>7.26E-06*</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* Previously determined by Hohai University
** Based on Table H.1

8.3.4 The sedimentation process
D-Settlement is not primarily designed to model sedimentation processes. In order to successfully incorporate settlements due to own weight of the new sediment, a few “tricks” need to be performed in the model. The initial soil profile in D-settlement is modeled as the soil profile at the
end of the sedimentation process (the sediment layer is already completely present). To model the sedimentation process the new sediment layer is split up into 5 sections of 1.1m thickness. Each section is represented by a load equal to the own weight of that section. The loads are applied in sequence as they represent the gradually increasing own weight of the new sediment layer. To ensure that the initial stresses in the underlying silty sand layers are correctly modeled, an initial compensation load is applied just below the new sediment layer. This load has the same magnitude as the total own weight of the new sediment layer, but in opposite direction. This ensures that the initial stresses in the sand layers are realistically modeled as the initial weight of the new sediment layer is balanced with the compensation load. The initial stresses in the new sediment layer itself are neglected in this research.

Figure 8.1 - Initial soil profile

Running D-Settlement for this geometry results in the settlements of the new sediment layer, as well as the sand layers over time (see Figure 8.2). Settlements due to the own weight of the new sediment layer are calculated just before construction of the embankment commences (after 1080 days of sedimentation). The total settlement is 0.34m, whereas the sand layers experience a combined settlement of 0.05m and the new sediment layer settles 0.29m. For all further calculations the settlement due to own weight of the new sediment have the same value, because the sedimentation process is the same.

Figure 8.2 - Settlement due to own weight of new sediment layer
8.3.5 The embankment
The embankment is modeled as multiple loads with dimensions and weights as described in section 8.3.5. The total building time for the embankment is 4 weeks. In order to fulfill the requirements for residual settlement for the road section 6.2), construction of the asphalt could only start after 5.5 years after completion of the embankment (Figure 8.3). If construction of the asphalt top layer would commence earlier, the residual settlements would result in cracks and bumps in the road. Additional measures are required to speed up the construction time.

![Figure 8.3 - Settlements embankment](image)

8.3.6 Surcharge preload
One of the possible measures is the use of surcharge induced preloading as described in appendix F. Typical surcharge preloads are 3-8m in height and are removed after 3.5-12 months (Purushothama, 2011). Two different surcharge preloads are further assessed; an intermediate surcharge preload of 6m in height with a duration of 6 months and an ultimate surcharge preload of 8m in height with a duration of 12 months. The total settlements over time of both surcharge preloads are displayed in Figure 8.4.

![Figure 8.4 - Total settlements surcharge loads](image)

8.3.7 Vertical drains and surcharge preload
Another method for accelerating consolidation is by application of drains. They are inserted to a depth of -2.61m ensuring potential contamination cannot reach the permeable sand layer at -3.11m. The drains are placed with a center-to-center distance of 1.7m in a triangular grid as determined in
It is possible to combine drains with an additional surcharge preload in order to further accelerate the consolidation process. Using an iterative process, an optimization can be found between surcharge preload height and time. This iterative process results in a surcharge preload of 4.5m with a duration of 6 months. The settlements over time of both drains and drains in combination with surcharge preload are displayed in Figure 8.5.

![Figure 8.5 - Settlements drains and surcharge](image)

### 8.4 Analytical settlement validation calculation

In order to check the results of the numerical model, an analytical calculation is made. For the analytical calculation Koppejan formulas are used. To properly compare the numerical solution with the analytical solution, the D-settlement calculation for the embankment is repeated using the NEN-Koppejan model instead of the NEN-Bjerrum model. Koppejan considers the following formula to calculate the primary settlement as well as approximate the secondary consolidation effect (Verruit, 1999).

\[
\varepsilon = - \left[ \frac{1}{C'_p} + \frac{1}{C'_s} \log \left( \frac{t}{t_0} \right) \right] \ln \left( \frac{\sigma}{\sigma_1} \right), \sigma_1 < \sigma_p
\]

\[
C'_p = C'_s \geq \sigma_p
\]

This formula uses the relation between primary and secondary coefficients of compression, time, reference time, initial stress and total imposed stress, \(C_p, C_s, t, t_0, \sigma_1\) and \(\sigma\) respectively. For the Tiaozini reclamation, the soil is freshly deposited and has no history of loading. Therefore the stresses in the soil, including the initial stress, will always be higher or equal to the pre-consolidation stress \(\sigma_p\). Due to these circumstances and the fact that unloading or reloading will not occur, the following formula can be used throughout the analytical calculation.

\[
\varepsilon = - \left[ \frac{1}{C'_p} + \frac{1}{C'_s} \log \left( \frac{t}{t_0} \right) \right] \ln \left( \frac{\sigma}{\sigma_1} \right)
\]

An accurate analytical calculation of the present layering is a tedious and time-consuming job; hence a simplified version of reality is created. The analytical calculation is used to validate the order of
magnitude resulting from the D-Settlement numerical model. The key concern is to check the overall settlement of the clay layer. The required consolidation time and intermediate settlement are considered less important for the analytical check, since the analytical Koppejan model does not take dissipation of water overpressure into account. The strongly simplified one-layer-model is shown in Figure 8.6.

![Figure 8.6 - Analytical simplification](image)

In this model the deposited soft clay rests on a firm base. This firm base represents the original sand layers which will settle only marginally compared to the top layer. The loads on top are a summation of the embankment and traffic loads. The input parameters used for the calculation are shown in Table 8.4. The determination of the coefficients can be found in appendix 0.

### Table 8.4 - Koppejan analytical input parameters

<table>
<thead>
<tr>
<th>Koppejan parameters</th>
<th>$h$ [m]</th>
<th>$C_p$ [-]</th>
<th>$C'_p$ [-]</th>
<th>$C_s$ [-]</th>
<th>$C'_s$ [-]</th>
<th>$t$ [days]</th>
<th>$t_0$ [days]</th>
<th>$\sigma$ [kN/m$^3$]</th>
<th>$\sigma_1$ [kN/m$^3$]</th>
<th>$\gamma'_w$ [kN/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.5</td>
<td>16.5</td>
<td>8.88</td>
<td>110.0</td>
<td>110.0</td>
<td>10000</td>
<td>1.0</td>
<td>156.3</td>
<td>18.0</td>
<td>18.0</td>
</tr>
</tbody>
</table>

As previously described, the sedimentation process is a continue process. The soft layers will settle by consolidation during the deposition of the layers itself. The deposition of the sediment layer is modeled as five separate stages, to simulate the sedimentation process. In each stage 1.1m sediment is deposited and added to the existing geometry. The settlement is calculated after each stage using Koppejan’s formula. The initial stress then changes to the total imposed stress of the previous step. This ensures the soil becoming stiffer during the consolidation process. After the fifth step, the embankment is modeled as a uniform load equal to the sum of traffic and embankment loads. Again the initial stress is changed to the value of total imposed stress of the previous step. The complete table with all the calculation input and results is given in appendix K. The result of the staged analytical calculation is plotted in Figure 8.7 together with the numerical NEN-Koppejan and the numerical NEN-Bjerrum methods.
Figure 8.7 - Consolidation graph using the analytical calculation based on Koppejan’s formula and numerical methods Koppejan and Bjerrum

As shown in this figure the analytically calculated final settlement of the soft clay layer is 1.113m. This lies in between the two results of the numerical methods. When comparing the analytical method with the numerical methods, the order of magnitude is similar. In fact with a deviation of -5.90% and 3.75%, from the numerical Koppejan and numerical Bjerrum method respectively, the analytical Koppejan proves itself to be a reliable estimation method.

The deviation in the results is probably due to the different strain models used. With an original soft layer thickness of 5.5m, 1.113m settlement means a total strain of 0.295. It should be noted that the Koppejan method uses linear strains to calculate the final settlement. These linear strains are based on the initial layer thickness and calculated by $\varepsilon_{t+\Delta t} \approx \varepsilon_{t} + \Delta t \varepsilon_{t}$. For large strains ($\varepsilon > 0.35$) approximations using linear strains can cause deviations from reality causing in a larger settlement effects. This is one of the factors explaining the slight difference between the numerical and analytical Koppejan model, since the numerical model uses natural strains to calculate the settlement.

8.5 Preliminary embankment conclusion

The new reclaimed land on Tiaozini will consist of soft soil. In order to fulfill the residual settlement requirements for the road, the soil characteristics need to be improved. Not all soil improvement methods are applicable for the Tiaozini area, due to the presence of the soft top layer. In order to determine the most suitable soil improvement method for Tiaozini a multi criteria analysis is performed. For the Tiaozini area, the use of surcharge preloading with vertical drains is the most favorable.

Using the numerical modeling program D-Settlement, the settlements for the different soil improvement methods are determined. The calculations confirm the need for soil improvement, since the requirement for residual settlement is not met if no measures are taken. Application of an intermediate surcharge preload or merely drains are not sufficient to fulfill the requirements. Applying an ultimate surcharge preload of 8m with a duration of 12 months fulfills the requirements for residual settlement. This measure is however disregarded as feasible, due to the fact that a construction time of 12 months is considered too long and a surcharge height of 8m is assumed to cause instability and impracticability problems. The optimal combination is found when applying a
surcharge preload of 4.5m in height for duration of 6 months, in combination with drains. Together with previous described measures this result is plotted in Figure 8.8.

![Figure 8.8 - Settlements calculated by D-Settlement](image)

The shallow slope between 180 and 1080 days represents the settlement of the newly deposited layer due to its own weight. D-Settlement overestimates the relaxation after unloading which is made visible in Figure 8.8, by the sudden upward kink after removal of the surcharges. The ongoing increase of settlement after 4000 days is caused by creep. In Table 8.5 a summary of the residual settlements of all measures is given. It can be concluded that only the ultimate surcharge and the drains including surcharge fulfill the residual settlements requirements. Due to economical and practicability reasons the drains including surcharge is advised.

**Table 8.5 - Residual settlements**

<table>
<thead>
<tr>
<th>Requirement</th>
<th>After 10 year &lt; 100mm</th>
<th>After 30 year &lt; 300mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment</td>
<td>641mm</td>
<td>681mm</td>
</tr>
<tr>
<td>Intermediate surcharge</td>
<td>326mm</td>
<td>360mm</td>
</tr>
<tr>
<td>Ultimate surcharge</td>
<td>29mm</td>
<td>51mm</td>
</tr>
<tr>
<td>Drains</td>
<td>500mm</td>
<td>532mm</td>
</tr>
<tr>
<td>Drains and surcharge</td>
<td>100mm</td>
<td>130mm</td>
</tr>
</tbody>
</table>

The numerical calculations are checked using an analytical Koppejan calculation. When comparing the analytical method with the numerical methods, the order of magnitude is similar. In fact with a deviation of -5.90% and 3.75%, from the numerical Koppejan and numerical Bjerrum method respectively, D-Settlement proves an accurate method of predicting settlements. The deviation in the results is probably due to the different strain models used.
9 Preliminary water retaining structure

9.1 General dike design

The proposed preliminary occupation for Tiaozini sets requirements for the dikes at Tiaozini. The original dikes at Tiaozini were designed using a 50 year design return period, suitable for protecting rural environment. With petrochemical industry as occupation the design return period should be reassessed. According to the EurOtop manual (Pullen, 2007) a dike protecting industries must be able to retain a 1:1000 year wave. This value will be used as design return period for this study. The design life of the dike is 200 year.

In this chapter the dike building process using geo-tubes and the general lay-out of the original dike will be outlined. Based on datasets from the Tiaozine feasibility study (Hohai University, 2010) and the new design return period a preliminary dike is designed. The location of the wave characteristics dataset used is marked with a star in Figure 9.1.

![Figure 9.1 - Location of point wave characteristics dataset](image)

9.1.1 Dike lay-out

In this chapter the dike build-up will be discussed. Important fact is that the water retaining function has to be achieved without structural failure. Dikes are normally wide structures, with use of slopes to achieve the sufficient dike height. In fact the dike consists of three parts that are based on still water level (SWL). The SWL is the water level without wave fluctuations, and is dependent of the circumstances. The area around the SWL, were wave action will be concentrated, is called the center area. The areas above and below are called the upper and lower area respectively (Verhaeghe, 2005). Figure 9.2 gives a schematization of the three areas.

![Figure 9.2 - Overview dike areas](image)
9.1.1.1 Center area
The range of this area will be between \(1.5 \cdot H_{mo}\) below SWL and \(1.5 \cdot H_{mo}\) above SWL (Van der Meer, Tönjes, & De Waal, 1998). Where \(H_{mo}\) is the significant wave height in shallow water. In deep water the significant wave height \((H_s)\) is equal to \(H_{mo}\) but for shallow water \(H_{mo}\) will be 5-10% lower than \(H_s\).

In order to reduce the wave impact and wave heights, a berm is usually located near the SWL. A berm is a flat area (max. slope of 1:15) between slopes with a length of maximal \((TAW, 2002)\). Where \(L_0\) is the deepwater wavelength. During propagation over a berm waves will reduce their significant wave height due to the fact that the water depth is reduced. The significant wave height cannot be more than half the water depth (TU Delft, 2011).

9.1.1.2 Lower area
The lower area consists of another kind of berm, the toe. The toe mainly focuses on structural protection of the lower slope. The toe will act as a counter weight against slide circles. In addition to its structural function, the toe may force waves to break if the SWL is very shallow. For this situation the toe is considered part of the center area.

9.1.1.3 Upper area
The crest is located in the upper area and often has a strengthening function for the upper part of the dike. The armor on the slopes can be made out of different materials, all with different roughness. A rougher surface on the outer slope will cause less wave run-up. On top of the dike there can be an additional vertical retaining wall.

9.1.2 Dike height
The dike height depends on the design wave height, freeboard and wave run up. The required dike height can be calculated using the following formula (TU Delft, 2011):

\[
Z_p = H_d + R_{u2}\% + H_{fr}
\]

The total height of the dike is the sum of the design height \((H_d)\), wave run-up \((R_{u2}\%)\) and freeboard height \((H_{fr})\).

9.1.2.1 Design wave height
The design wave height is the maximum still water level and consists of multiple components; the wind set-up, the tidal influence, sieches, shower oscillations and shower gusts. Of all these components the tidal influence and the wind set up will be most responsible for water level rise. The sea level caused by those two components together is called the storm surge level. The maximum water level reached during every storm is collected, as well as the probability of occurrence of that particular storm. Out of the gathered data a prediction of the future storm surge level can be made by extrapolation.

The tidal influence near Tiaozini is of importance, since the tidal differences are large. In the Tiaozini area there is an average high tide of + 2.39m with a maximum of + 4.74m (Hohai University, 2010).
9.1.2.2 Freeboard

Freeboard on top of the dike is used as extra safety to take account of environmental changes. Research states that due to the changing climate, the water level will have risen 0.40m by the year 2100 (Nicholls & Wong, 2007). Settlements of the soil body of the dike and the underlying soil layers will lead to a decrease in crest height over time. In order to guarantee the required safety, freeboard is used to compensate for settlements and seawater level rise.

9.1.2.3 Wave run-up

The wave run-up can be found using the EurOtop manual (Pullen, 2007);

\[ R_{tu2\%} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1.0} \cdot H_m \]

In this formula \( \gamma_b \), \( \gamma_f \) and \( \gamma_\beta \) are coefficients concerning the influence of the berm, roughness of the slope and the angle of the incoming waves. The values of these coefficients will lie between 0.6 and 1.0. The significant wave height \( (H_{m0}) \) and the breaker parameter \( (\xi_{m-1.0}) \) have a great influence on the total wave run-up due to their wider range. Wave run-up is the only parameter influenced by the dike design and hence \( H_{m0} \) and \( \xi_{m-1.0} \) determine the dike design. The dike design will also be influenced by the settlement of the soil body and underlying layers, but these influences are small in comparison to the wave run-up changes.

Possible solutions to reduce wave run-up are:

- The use of a berm in order to lower \( H_{m0} \)
- Use of shallower slopes in order to lower \( \xi_{m-1.0} \)

Both solutions will result in a wider dike and therefore a higher material use. For an optimal result, the right balance between both solutions has to be found.
9.2 Original dike

9.2.1 Current situation

In order to obtain new land, an innovative and effective method is used. The area of Tiaozini is divided into different sections, which will be constructed in succession. For one single section a dike ring is constructed. A mouth in the dike ring ensures tidal flow bringing in sediment (displayed in Figure 9.3 as 3 parallel lines). Inside the dike ring several smaller dikes are build which act as thresholds. The height of these smaller dikes is less than high tide level, causing water to flow over the dikes during high tide. Since velocity of water decreases at the transition of tides, sediments will be deposited. The sediment is now trapped behind the lower dikes. Small gates are present in all small dikes allowing water to escape at low tide (displayed in Figure 9.3 as H). This process continues till the desired amount of sediment is accumulated. When enough sediment is present the gates in the smaller dikes are closed and the process to close the entire basin starts. At this point the small dikes are beneficial due to the decrease in flow velocity out of the basin at low tide. Decreasing this flow will make the closing procedure of the basin easier. The closing procedure and the used dikes will be discussed in section 9.2.1.1.

Figure 9.3 - Phased construction Tiaozini dikes

At the Tiaozine land reclamation several different types of dikes are present. Since all dikes have a different application, different characteristics are required. A description of dikes present at Tiaozini is given below.

- Outer dikes (purple line in Figure 9.3)
  The outer dikes will form the permanent barrier between the reclaimed land and the sea. These dikes have to withstand influences from the sea and should be designed for a long period.

- Inner dikes (red and green line in Figure 9.3)
  The land reclamation is performed in different stages. The dikes that are the primair sea barrier during the first stages will no longer face the sea once further phases are completed. For this reason the dikes are built using re-usable materials. The concrete bars and rubble stone (rip-rap) will be removed from the inner dikes once there is no more connection with the sea and are re-used for the new sea barrier.

- Sediment stimulating dikes
  The land reclamation project is based on natural deposition of sediment. In order to determine the position of the sediment, two long dikes are constructed. For several phases these dikes act as the sea barrier. After the whole project is finished, these dikes will no longer be connected with the
sea. For this reason the dikes can be considered as temporary. Afterwards the dike will be used for infrastructure and as extra safety measure.

- **Groynes**
  These small dikes are build perpendicular to the main seabARRIER. The groynes are constructed to minimise the erosion perpendicular to the shoreline. These dikes do not have a water-repelling function.

- **Inner threshold dikes**
  These dikes are built inside a larger dike ring in order to increase the sediment deposition and to simplify the closing of the tidal basin. They have no water-repelling function and are only temporary.

9.2.1.1 **Use of geo-tubes**
In the Tiaozini project geotextile of poor quality is used, only a lifespan of several years is guaranteed (Zhang, 2012). The characteristics of the geo-tubes are used during the construction process, and are not required during the operational phase of the dike (appendix 0). Several properties of the geotextile are important for the Tiaozini project:

- **Permeability**
  Due to the permeability of the geo-tubes it is possible to construct a dike from slurry. The slurry is pumped in the tubes, and the excess pore water will flow out of the geo-tube. The natural tidal flat material can be used as the source for the fill.

- **Shape**
  The strength of the geotextile allows for constructing steeper sloped dikes. It however should be taken into account that this effect is only temporary and measures should be taken to ensure dike stability in the operational phase.

- **Erosion**
  Geotextile prevents the soil body from eroding. This property is useful in the Tiaozini project, as it allows phased construction. The construction site is located in a tidal area, which causes the site to be submerged half of the time. The geotextile allows the dike to be constructed in phases and ensures no erosion will take place between the phases.

Due to the limited lifespan of the geotextile, these features are no longer guaranteed in the operational phase of the dike. This means that the dike has to be constructed in a way that it will be stable without the geotextile. The angle of the slopes has to be checked on stability. In order to prevent erosion of the dike after the decay of the geotextile, revetment has to be used.

9.2.1.2 **Construction method**
The construction process of the Tiaozini dike is explained in this section. The outer sea dike is investigated since this dike has the most critical and complex structure. Although different types of dikes are present, the same principles can be used for all dike types.
Toe construction
First a layer of geotextile is placed on the tidal surface (i); this layer is covered by rip-rap (ii). The area behind the rip-rap is used to construct a small dike of geo-tubes (iii). The geo-tubes are placed against the rip-rap. Soil from the surrounding tidal flats area is pumped into the geo-tubes filling them with slurry. During low tide the excess pore water can dissipate through the permeable geotextile. The top of this small geo-tube dike is covered by 100-200 kg rock protection.
The timespan of one ebb tide, roughly six hours, is too short to build the entire dike from geo-tubes at once. The occurrence of tidal currents with high velocity water flows, can cause erosion of the soil surrounding the newly placed geo-tubes. Therefore a layer of geotextile on the soil surface is necessary, to prevent erosion in the first phase of construction. The geotextile makes it possible to construct the dike in phases; one layer of tubes can be constructed during low tide, while the next layer is built in the next ebb tide. In the operational phase the same geotextile prevents erosion, caused by wave loads, underneath the rip-rap.

Berm construction
In order to create the outer berm another geotextile layer is placed behind the geo-tubes. This layer is then covered by a gravel layer of 0.80m thickness. On top of this gravel layer rip-rap with a height of 3.0m is placed (iv).

Center area construction
At the rear of the berm a dike of geo-tubes is constructed of 3.0m height. Approximately 15.0m behind the first, a new prism of geo-tubes with a height of 5.0m is constructed (v). The area between the two geo-tube prisms is filled with hydraulic fill using rainbow dredging (vi). The hydraulic fill will consolidate due to gravity loading and the dissipation of excess pore pressure through the permeable geotextile. Due to the ongoing consolidation, the soil properties of the hydraulic fill improve significantly. When the properties of the fill are sufficient, a new layer of geo-tubes is constructed on top of the fill to close of the intermediate gap. This principle, of geo-tubes on top of the hydraulic fill, is repeated several times. Two small dikes of geo-tubes are constructed and the area between is filled with hydraulic fill using rainbow dredging. After sufficient consolidation has taken place and this soil has enough strength a new layer is placed in top (vii).

To protect the outer side of the berm and center area, several protective layers are applied (viii). First of all, a layer of 0.3m gravel bags is applied covering the rip-rap of the berm. Resting on these gravel bags is a 0.5m thick stone cushion of 100-200 kg blocks. This cushion is covered with 1.3m thick concrete armor. The armor consists of prefab concrete plates; rectangular plates are pierced with horizontal bar shaped holes. The holes are applied to break the waves in an early stage. This
protection continues from the berm to the crest. To complete the dike design a vertical concrete wall is constructed as a final barrier against waves. A detailed technical drawing of the original Chinese dike is shown in Figure R.1.

9.2.2 Potential failure mechanisms
This section describes the Tiaozini specific fail mechanisms for both the geo-tubes as well as the general dike. For each fail mechanism, the possible measures are described. The section is concluded with an overview of the measures taken at Tiaozini to prevent these fail mechanisms.

9.2.2.1 Geo-tube failure mechanism
Geo-tubes with a good filling percentage are stable (appendix 0). If no protection is applied however, large displacement of the geo-tubes may occur due to wave loads. (Steeg & Vastenburg, 2010) For this reason a protective layer is advised to shield the geo-tubes from wave loads.

Scour of the subsurface is a significant risk, due to tidal hydraulic environment causing large velocities of water flow around the geo-tubes. Stacking of the geo-tubes leads to a large amount of excess filling water from the top geo-tubes. This can cause erosion or undermining underneath the bottom geo-tube. In addition, phased construction in combination with the tidal environment can lead to large flow velocities. In order to prevent scour of the subsurface a layer of geotextile can be placed on the subsurface. This layer will prevent erosion of the soil underneath the geo-tubes. With respect to the wave load in the operation phase, the geotextile should not only cover the area underneath the dike, but also advance towards the sea.

Rupture of the geotextile is a possible fail mechanism with a large chance, since a lifespan of only several years is guaranteed. The consequences however are not considered very large since the decay of the geotextile is incorporated in the design of the dike. For this reason precautions are taken to assure that when rupture occurs, the dike will maintain stable.

The loss of fill material through the geotextile depends on the marine environment and the time of exposure. At Tiaozini the geo-tubes are placed under a continual wave load for the entire lifespan. In order to prevent loss of fill material the apparent opening size of the geotextile skin should be smaller or equal to the 50% grain diameter of the fill and protection is required. However even with these measures, change in shape may occur (Lawson, 2008).

Deformation of the fill due to consolidation is not likely to happen at Tiaozini, since the fill consists of mainly sandy silt and silty sand. Failure due to deformation caused by too loose packing is also not likely to happen. In the design the decay of the geotextile is incorporated, which means no unnatural shapes or angles for the soil are addressed. Rip-rap is placed at the toe of the dike to prevent major displacement.

9.2.2.2 General dike failure mechanism
All general dike failure mechanisms are described in appendix L. In this section the failure mechanisms for the Tiaozini dike are described in more detail.

In order to reduce the chance of overflow and wave overtopping the right design height has to be determined. The inner slope has to be protected against erosion by a protective layer. In the Tiaozini hinterland capital intensive assets are located. For this reason overflow or significant wave overtopping will lead to significant damage, even if the dike remains stable.
Sliding of the inner or outer slope does not automatically lead to failure, since the dike is designed for a 1:1000 storm. The dike however should be checked for instability. It is expected that shearing will not be a problem since the dike is constructed of sand and silt. The mass of the dike will generate enough friction to support the water pressure. It should be checked whether this is also the case for the 1:1000 storm. With respect to micro-instability and piping a good protective layer should be applied on the inner slope. The present geotextile will also prevent solute transport. Because the geotextile will decade, it should be checked whether these mechanisms will occur without geotextile.

Erosion of the outer slope is a significant risk in Tiaozini, due to the tidal regime. For this reason a protective layer should be applied.

Erosion of the first bank can be neglected in Tiaozini, since a very gentle slope is present. Settlement will certainly occur as the dike is located on loose packed soil with high water content (chapter 7). For this reason the expected settlement has to be calculated and included in the design height of the dike.

Drifting ice will not occur in the Tiaozini area so this fail mechanism can be neglected. A collision with a large ship is very unlikely due to the shallow flat in front of the dike. However typhoons occur often at the Chinese coast so a collision with a small shipping vessel has a reasonable chance.

Due to the tidal regime at Tiaozini an additional fail mechanism may occur. As a result of the tidal flow after high tide, creeks will form in the tidal flat. These creeks can reach substantive depth over time. In the event of a typhoon large quantities of water are driven towards the shore, filling up existing creeks and forming new ones. This process leads to inversion of the sub surface. If the new creek is located near the toe of the dike this may load to global instability of the dike.

9.2.2.3 Measures to prevent failure

- A protective armor at the seaside is installed to decrease and distribute the wave loads. The protective layer will prevent erosion of the fill material in the geo-tube and erosion of the outer slope. This measure ensures that no large displacement of the geo-tubes take place.
- During construction a geotextile layer is placed on ground level to protect the subsurface from erosion and undermining. Due to this geotextile layer, scour will not take place during the filling of the geo-tubes.
- The inner slope is covered with a protective layer in order to prevent erosion of the inner slope and prevent micro instability. No extra measures have to be taken in order to prevent piping due to the fact that the seepage length is large. In addition, the geotextile will also prevent solute transport.
- To prevent overflow the present crest height is designed with a 50 year return period.

9.2.3 Geometry verification original dike

In this section the original dike geometry will be validated using wave run up and stability. All validations are based on European regulations.

9.2.3.1 Geometry and parameters

There are three horizontal planes in the dike lay-out; the toe, berm and crest located at a height of +1.0, +5.0 and +10.3 meter with a length of 15.0, 5.0 and 8.5 meter respectively. The toe is within the range of $1.5 \cdot H_{mf}$ and can therefore be seen as the center area of the dike. The slope in front of the toe has an inclination of 1:2.5 and the lower and upper slope both have an inclination of 1:2.
Previous dimension are shown in Figure R.1. Furthermore the freeboard used in the original dike design is 0.80m and the wave run-up is 5.13 meter. On top of the crest there is a wall of 1.20 meter.

The main parameters can be divided into two groups; parameters concerning seawater conditions and parameters concerning the dike. All data is gathered from the Tiaozini feasibility study (Hohai University, 2010). The parameters used for the wave conditions and water depth are; a significant wave height of 3.57 meter with a period of 7.73 seconds. The SWL is located at +5.56m and the sea bottom at -3.0m, resulting in a total depth in front of the dike of 8.56m. The incoming waves have an angle ($\beta$) of 16°, leading to $\gamma_B = 1 - 0.0022\beta = 0.96$.

### 9.2.3.2 Wave run-up using PC-overtopping

The verification of the dike height is performed using PC-Overtopping. PC-Overtopping is a program that calculates the wave run-up for a given dike lay-out, significant wave height, wave period, roughness and incoming wave angle.

The by PC-Overtopping calculated wave run up resulted in 6.58m opposite to the 5.13m noted in the feasibility study of Tiaozini (Hohai University, 2010). This is a big difference, which may lead to large consequence. A possible reason for this difference is that PC-overtopping is a preliminary design tool and not an exact calculation tool; the program makes simplifications if the dike lay-out is complex. The toe for example is modeled as flat, while the toe in reality has an inclination of 1:26.

### 9.2.3.3 Optimization of the berm

To save material and hence reduce costs, an optimization of the dike is required. Two factors influence the optimization of the material used; additional material required for the berm (green area in Figure 9.5) and saved material for the dike body due to the use of a berm (yellow area in Figure 9.5). For a profitable optimization, the saved material needs to bigger than the additional required material.

![Figure 9.5 - Berm material savings](image)

The amount of material required to build the berm can easily be calculated;

$$\text{additional material} = h_{\text{berm}} \cdot B_{\text{berm}}$$

The determination of the saved material is more complex, since the berm influences the wave run-up. Reduction of the wave run-up height will lead to material savings on top and on the inner slope of the dike. According to the EurOtop manual (Pullen, 2007), the total reduction factor caused by the berm is;

$$\gamma_B = 1 - \frac{B_{\text{berm}}}{h_{\text{berm}}} \left(0.5 + 0.5 \cos \left(\frac{\pi h_B}{x}\right)\right)$$

The berm coefficient is dependent on two components; a component that takes the length of the berm into account and a component coping with the influence of the berm height. $B_{\text{berm}}$ is the berm...
width $L_{berm}$, the berm length, $h_b$ is the depth of the berm below SWL and $x = 2 \cdot H_{mo}$ (see Figure 9.6).

![Figure 9.6 - Berm length](image)

For both components the following applies; they are both dependent on the significant wave height. The higher the value of the component, the lower the wave run-up will be. The highest values can be found at the longest berm for the first component and a height similar to the SWL ($h_b = 0$) for the second component. However those values are not leading to an optimal result since the amount of required material is high.

The amount of saved material is dependent on the height reduction. Taking the savings of the crest and inner slope into account, the total amount of saved material can be calculated by:

\[
\text{saved material} = \left( (1 - \gamma_b) \cdot R_{U2\%} \right) \cdot \left( 8.5 + 2.5 \cdot (1 - \gamma_b) \cdot R_{U2\%} + 3 \cdot \gamma_b \cdot R_{U2\%} \right)
\]

The optimized berm can be found by determining the highest value for the saved material minus the required additional material;

\[
\left( (1 - \gamma_b) \cdot R_{U2\%} \right) \cdot \left( 8.5 + 2.5 \cdot (1 - \gamma_b) \cdot R_{U2\%} + 3 \cdot \gamma_b \cdot R_{U2\%} \right) = (h_{berm} \cdot B_{berm})
\]

The result for different berm heights and lengths are shown in Figure 9.7.

![Figure 9.7 - Optimal berm length by different heights below SWL](image)

The most optimal berm dimensions are; a length of 5m and a height of 1m below SWL. This result differs from the original design by 0.5m in berm height. A reason for this difference could be that the toe structure was not taken into account.
9.2.3.4  **The toe**  
As mentioned in section 9.1.1 the toe structure has two functions. The first is structural protection of the lower slope. The second function is the breaking of waves and reducing the significant wave height. This will only have affect when the toe is located in the center area of the dike. The toe structure of the original dike is already located within the center area and will be of influence on the significant wave height.

As stated in section 9.2.3.3 the berm coefficient is dependent on the significant wave height. In the formulas from the EuROtop used in section 9.2.3 the wave run-up could be calculated for a dike with one berm. The change in significant wave height is already included in the coefficient $\gamma_b$. Since this particular dike can be seen as a dike with two berms, the first berm will change the significant wave height for the other berm; the second $\gamma_b$ is calculated with the significant wave height obtained after the first berm.

Tables in appendix 0 show that for the current situation, the dimensions of the toe are not optimized for material use; a smaller toe would be more efficient. Reasons for using a longer toe length are; the breaking of waves in low water conditions and the gain of stability. This results in the fact that the toe is there primary to protect the slope against low waves and could be left outside the optimization process. The berm will however still be dependent on the significant wave height, which in turn is influenced by the toe.

9.2.3.5  **Combination of toe and berm**  
To obtain the significant wave height at the end of the toe, the computer software Swan1 is used. With use of this 1D program, deep water waves can be converted to shallow water waves by input of the bathymetry, SWL, significant wave height and wave period. According to Swan1 calculations, the significant wave will decrease from a height of 3.57m in front of the toe towards 3.17m by application of a toe of 15.0m long at 4.56m below SWL (see Figure 9.8).

![Figure 9.8 - Swan1 significant wave height](image)

When 3.17m is used as the new significant wave height in the formula for the berm coefficient the optimal dimensions of the berm will alter; the new height of the berm is 0.56m below SWL. This exactly meets the dimensions given in the feasibility study of Tiaozini (Hohai University, 2010). The corresponding $\gamma_b$ is 0.72 and the total wave run-up is 6.04m as calculated using the EuROtop formula. This is still higher than the wave run-up of 5.13m as stated in the feasibility study of Tiaozini but less than the height calculated using PC-overtopping.
9.2.4 Geotechnical stability verification

The geotechnical stability of the original design is checked using PLAXIS, a numerical Finite-Element-Method program. This widely used program simultaneously solves numerical equations by approximating the stresses, strains and displacements in a soil body. It is controlled whether the already designed dike is stable and safe in comparison with the Dutch standards and requirements.

The calculation is also useful to verify the assumptions about the used parameters. For the designed dike the required safety factor (SF) is available. By calculating the original dike with the assumed parameters it can be confirmed that the assumptions are realistic. The assumptions might not be precise or exact, but this way it is checked whether they are in the right range. If the new dike were to be designed using parameters with large uncertainties, no reliable conclusion can be made.

9.2.4.1 Used method

The dike is modeled using linear elastic perfectly plastic mohr coulomb model. The situation is considered drained, since no quick loading or impermeable soils are present. In order to model the original dike in PLAXIS the design is simplified. The dike build-up is simplified to the concrete armor and three different material soil sets; rip-rap, geo-tube and hydraulic fill. With these materials for the dike and three different materials for the underlying layers, this results in the PLAXIS model displayed in Figure 9.9.

![Figure 9.9 - Input PLAXIS original dike](image_url)

The Mohr Coulomb model requires the least parameters of the different soil models, not all required parameters are present. For the soil layers extensive data is available, see Table 9.1. Data for the geo-tube, hydraulic fill and rip-rap however, is not present. For this reason a value of E has to be chosen that best approximates the stiffness during the actual loading. In Table 9.1 the Es_{1,2} is given; this is the stiffness with a stress from 100 to 200 kPa. In order to determine a value of E that represents the situation the stiffness is either; estimated, derived or based on literature and the SPT blow count for respectively soil layer 1, 2, and 3.

For the geo-tube and the hydraulic fill no soil data was present. Therefore rough assumptions have been made to allow geotechnical stability check using numerical modeling. The two materials are created using the nearby soil from soil layer 1. Parameters are determined by modifying the source layer, dependent on the method of deposition, circumstances and present situation. Since the original soil layer has a high void ratio of 0.884, the soil will be denser when located in the geo-tube or at the hydraulic fill. These two locations are above the groundwater table, and therefore the void ratio will decrease. This will lead to a denser material and higher stiffness in comparison with the original soil. In general a hydraulic fill will lead to a looser packing in comparison to the original soil (C.K. Lee, 1994). The geo-tubes will be filled to a certain percentage which leads to a large increase of stiffness. For this reason the increase in stiffness of the hydraulic fill will be smaller than the increase...
of stiffness of the geo-tubes. For the rip-rap no soil data is present. For this reason the general parameters are used.

The used parameters are shown in Table 9.1. More information about the used soil model, determination of the parameters, method of modeling and encountered problems in PLAXIS can be found in appendix O.

Table 9.1 - Soil properties PLAXIS model

<table>
<thead>
<tr>
<th></th>
<th>PLAXIS input parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Geo-tube</td>
</tr>
<tr>
<td>$E$ [kPa]</td>
<td>25000</td>
</tr>
<tr>
<td>$n$ [-]</td>
<td>0.37</td>
</tr>
<tr>
<td>$c$ [kPa]</td>
<td>5</td>
</tr>
<tr>
<td>$\varphi$ [°]</td>
<td>25.0</td>
</tr>
<tr>
<td>$\psi$ [°]</td>
<td>0.0</td>
</tr>
<tr>
<td>$\gamma_{\text{unsat}}$ [kN/m$^3$]</td>
<td>17.0</td>
</tr>
<tr>
<td>$\gamma_{\text{sat}}$ [kN/m$^3$]</td>
<td>20.0</td>
</tr>
</tbody>
</table>

9.2.4.2 Safety factor

The reliability of a dike with respect to slope stability can be expressed in terms of a safety factor. The safety factor is the ratio of strength (resisting force) and load (driving force). The resistance is a function of the shear strength of the soil. This factor can be expressed using two quantities, cohesion ($c$) and friction angle ($\varphi$). The load is a function of the weight of the soil and external loading. (Verruit, 1999). The determination of the safety factor is quite complex because of many uncertainties. The problem is statistically indeterminate and some simplifying assumptions are necessary in order to obtain a critical safety factor (Tan, 2006). Due to the differences in the assumptions numerous methods have been proposed, which in itself illustrates that none of them are exact. In general two kinds of trends in the methods can be separated:

- Analytical methods based on slices (Bishop, Fellenius)

Models based on the methods of slices assume circular failure planes and may therefore not always give accurate results. With FEM models the failure planes are not restricted, they may take any shape (de Rocquigny, Devictor, & Tarantola, 2008).

The safety factor can be found using the following formula (Verruit, 1999):

$$SF = \frac{\text{available strength}}{\text{strength at failure}} = \sum Msf \text{ at failure}$$

In PLAXIS this ratio is found by successfully reducing the input parameters $\varphi$ and $c$ until failure occurs. This is represented by the following formula (PLAXIS, 2011):

$$\sum Msf = \frac{\tan \varphi_{\text{input}}}{\tan \varphi_{\text{reduced}}} = \frac{c_{\text{input}}}{c_{\text{reduced}}}$$

Where the strength parameter with the subscript ‘input’ refer to the properties entered in the material set and parameter with the subscript ‘reduced’ to the reduced values used in analysis.
In the case that all properties are perfectly known, a safety factor higher than 1.0 means that the slope is stable and a safety factor lower than 1.0 means the slope will fail. However since soil is a highly heterogeneous and anisotropic material and uncertainty in the parameters is inherent, a safety factor higher than 1.0 is necessary for a safe design. The minimum safety factor, required to ensure sufficient reliability of the slope, strongly depends on the use and literature cannot provide one decisive value. Some sources state that values should at least range from 1.1 to 1.3 ((Rocquigny, Devictor, & Tarantola, 2008), (WSDOT, 2010), (Schweckendiek & Calle, 2010)). While others state that values should not be lower than 1.5 ( (Baker, 2006), (Samtani & Nowatzki, 2006)). Generally values should be higher if there is significant uncertainty in the input parameters of the analysis or if slope failure would cause greater damage.

A final remark on the safety factor in relation with the reliability of the slope is appropriate. Due to the fact that soil characteristics are ambiguous and can vary a greatly within small regions, soil parameters are not exact. In analyzing a slope stability problem, vagueness is involved in the determination of the shear strength parameters, the specific weight of the soil, the location of water table and the boundary of soil layers. Hence, the real factor of safety varies in a range that depends on the precision of the input data. This fact has encouraged many researchers have started to work with a probabilistic methods for the slope stability analysis instead of the conventional approach (Meidani, Habibagahi, & Katebi, 2004). Where the original (deterministic) approach uses mean property values for each layer, which leads to a single analysis and to a single factor of safety. The new probabilistic (stochastic) approach takes account of all properties within each layer. This requires multiple analyses, as part of a Monte Carlo simulation process, and leads to a more meaningful, definition of stability reliability. This new determination of reliability provides useful extra information about the probability that failure will not occur (Hicks & Samy, 2002).

### 9.2.4.3 Stability results

Using the finite element program PLAXIS, information about the settlement, deformation and stability of the original dike is obtained. The result of the final model is discussed in this section.

Figure 9.10 shows the settlement of the dike structure due to its own weight after construction. The settlement is magnified by a factor of 10 to improve the visibility. In reality the maximum settlement is 0.3996m. This settlement is caused by the densification of the particles in the underlying silty sand layers. The original design already incorporated this settlement by constructing a 1.2m high retaining wall on top of the crest.

![Deformed mesh of the original dike, showing settlements due to own weight (magnified 10x)](image)

The failure of the dike is shown in Figure 9.11 and Figure 9.12. From these figures it can derived that the governing failure occurs due to slip failure on the inner slope of the dike. As a result of this local...
circular failure plane, global failure of the dike occurs. Global failure means that the dike cannot guarantee the safety of the hinterland. If global failure of the dike occurs, the purpose of the dike is jeopardized.

Figure 9.11 shows the total displacements of the slip surface in its original position on the dike. The legend on the right side shows the magnitude of the displacements. It is noticeable that these displacements are unrealistically large; this is caused by the numerical method used to approximate the displacements during failure. Figure 9.12 shows the final position of the soil after failure has occurred.

![Figure 9.11 - Total displacements at failure](image1)

![Figure 9.12 - Deformed mesh at failure, showing the circular slip surface of the inner slope](image2)

The detailed Figure 9.13 shows that the concrete armor, on the outer slope of the dike, is damaged due to the slip failure. Moreover the height of the dike is reduced significantly due to this mechanism. The steep angle of the remaining soil on the left side of the failure is unrealistic. This soil will also collapse, causing additional loss of dike height. Both these consequences of the sliding of the inner slope can result in further failure due to overflow of water.

![Figure 9.13 - Detailed deformation of the dike during failure](image3)
Additional safety analyses in PLAXIS show that this failure occurs at a global safety factor of 1.951. This value is checked using several points along the failure surface of the dike shown in Figure 9.14.

This safety factor far exceeds the minimum safety factor required for a sea dike, which is 1.5. This would suggest that the original dike is significantly over-dimensioned and further optimization is possible. However due to the great uncertainty in the input parameters, assumptions and the strong simplification of the structure of the dike, a higher safety factor is desirable. More results of the PLAXIS model of the original dike can be found in appendix O.

9.2.5 Original dike conclusion
It can be concluded that the original dike designed for Tiaozini, does not meet the requirements set by the Planning team. With the original dike geometry the agricultural hinterland is not protected against a 1:50 year storm, since the dike is not sufficiently high. For the original dike, both the dimensions of the berm and toe have significant influence on the total wave run-up. When using EurOtop or PC-overtopping the wave run-up is 0.9m or 1.5m respectively; higher than the value supplied in the Chinese preliminary design. This substantial difference can lead to severe consequences. These differences might be caused by the different formulas applied or by the variation in available parameter. In addition to this surprising safety result, the berm dimensions are confirmed to be optimal. The toe is not designed optimally to influence the high waves, since the original toe is mainly built to retain lower waves, and not for crest lowering purposes.

Using PLAXIS the stability of the dike is checked and, with a safety factor of 1.9, the dike is determined stable. Additionally, the material properties and dike arrangement are tested and can reliably be used in the design of the new dike.

9.3 New dike design
9.3.1 Geometry determination
The new dike lay-out will be designed with the use of parameters which are identical to the parameters of the original dike and new parameters that suit the new requirements. The most important new parameters are the changed wave height and SWL, which come with a change in design return period from 1:50 to 1:1000 years.

9.3.1.1 New wave height and water level
The SWL and the significant wave height are of great importance in designing a new dike. The total dike height formula shows the importance of the wave characteristics, since $H_{m0}$ has the most influence on the wave run-up and the SWL in the design height.
As mentioned in section 9.1.2, the design height can be obtained by extrapolating measured storm surge levels. However there are no storm surge level measurements done for the entire Tiaozini area. This makes the computation of the required design height complicated. In order to calculate the design height the previous determined wave height distribution is used. The wave height appeared to be Weibull distributed (appendix Q) and the new design height is calculated. For a 1:1.000 design return period, the new design height is 7.53m. This value lies within the boundaries set by the extrapolation of the two available water levels and their probability.

When these measurements are used for shallow water, a conversion is required. This conversion is dependent on the bathymetry of the foreshore. The components of the conversion consist of shoaling, refraction and breaking, of which the later is the most important. Since the water in front of the dike is shallow the maximal significant wave height is limited by the depth of the sea. The relation between the maximal significant wave height and the water depth is \( (H_{\text{m0}})_{\text{max}} = 0.41 \cdot d_m \) (TAW, 2003). The significant wave height of the new dike will be the new depth (SWL + distance sea bottom to CSWL) multiplied by 0.41. The results are listed in Table 9.2.

<table>
<thead>
<tr>
<th>P [%]</th>
<th>0.1</th>
<th>1</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>50</th>
<th>Original design</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_d [m] )</td>
<td>7.53</td>
<td>7.14</td>
<td>7.02</td>
<td>6.85</td>
<td>6.71</td>
<td>6.32</td>
<td>5.56</td>
</tr>
<tr>
<td>( H_s [m] )</td>
<td>4.39</td>
<td>4.23</td>
<td>4.18</td>
<td>4.11</td>
<td>4.05</td>
<td>3.89</td>
<td>3.57</td>
</tr>
</tbody>
</table>

The dike will be designed on a once in thousand years probability of exceedence, therefore the P=0.1% will be used (Table 9.2). This will mean a design height of 7.53m and a significant wave height of 4.39m. For more details about the design height and significant wave height computations see appendix Q.

9.3.1.2 Parameters
In previous section the wave characteristics have been calculated for waves that occur once in thousand years. This is the most important changed parameter for the new dike design. Other parameters have remained the same; the slope steepness, the angel of incoming waves, the soil properties, roughness of the dike and the width of the crest.

The dimensions of the berms will be important for wave run-up since the slope steepness and usage of berm structures were most effective in reducing the wave run-up. With slope steepness remaining the same, the optimization of the length and height of the berm and toe will lead to the required dikes crest height.

9.3.1.3 Geometry determination of new dike
To design the new dike, some rough estimations are made first. These estimations will thereafter be optimized. Estimations are obtained out of the validation of the old dike (section 9.3.1). For a high \( \gamma_b \), the berm has to be in a range of +0.0m SWL to -2.0m SWL. The SWL is located at +7.53m, leading to a berm height within the range of +5.53m to +7.53m.

Another estimation is the enlargement of the Chinese dike by a factor \( \frac{H_{\text{m0}}_{\text{new}}}{H_{\text{m0}}_{\text{old}}} = \frac{4.39}{3.57} = 1.2 \). Together with the factor, which is near 1.2 as well, the total outline of the already designed dike is
multiplied by this factor. The results for the height and width for the crest, berm and toe are listed in Table 9.3 and Table 9.4. These heights and widths are used as guidelines for the optimization.

<table>
<thead>
<tr>
<th>Level</th>
<th>Old Height [m]</th>
<th>New Height [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toe</td>
<td>+ 1.0</td>
<td>+ 1.8</td>
</tr>
<tr>
<td>Berm</td>
<td>+ 5.0</td>
<td>+ 6.6</td>
</tr>
<tr>
<td>Crest</td>
<td>+ 10.3</td>
<td>+ 13.0</td>
</tr>
</tbody>
</table>

Table 9.3 - New obtained heights by enlargement dike

<table>
<thead>
<tr>
<th>Level</th>
<th>Old width [m]</th>
<th>New width [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toe</td>
<td>+ 15.0</td>
<td>+ 18.0</td>
</tr>
<tr>
<td>Berm</td>
<td>+ 5.0</td>
<td>+ 6.0</td>
</tr>
<tr>
<td>Crest</td>
<td>+ 8.5</td>
<td>+ 8.5</td>
</tr>
</tbody>
</table>

Table 9.4 - New obtained widths by enlargement dike

9.3.2 Optimization

As stated in the verification of the original dike, the optimal width and height of the berm and toe is reached when the most material is saved. In other words; the decrease of wave run-up height times the material saving per meter height loss has to be as large as possible while the required material to enable the decrease of wave run-up possible has to be as low as possible.

First the required dike height for a simple dike construction without use of a berm or toe is determined using PC-overtopping. The total wave run up with the right wave characteristics and slope steepness of 1:2 is $R_{U2\%} = 9.84\text{m}$, resulting in a dike height of $H_d + R_{U2\%} + H_{fr} = 10.53 + 9.84 + 0.80 = 21.17\text{m}$.

The toe structure for the original dike was not designed with a high wave protection purpose. The function of the toe structure is stability insurance and protection against low waves. No changes in the length of the toe are needed, since the low water waves stay the same. To encounter the possible slide circles, the toe height will be increased. Hereby the coefficient of the distance from SWL to the old dikes berm and $H_{m,o}$ will remain the same. With $H_{m,o} = 4.39\text{m}$, this leads to a toe height of +2m.

![Figure 9.15 - Significant wave height](image)

For a berm level at +2m, the decrease of $H_{m,o}$ can be calculated using Swan1 (Figure 9.15). It can be concluded that $H_{m,o}$ will drop from 4.33m to 3.89m if the toe length is 15m long. The dimensions of the berm which provide the maximum material saving by this significant wave height could be obtained by the use of the formula’s stated in section 9.3.2 (Figure 9.16).
From Figure 9.16 it can be concluded that the optimal berm for this new dike is 6m long, with a berm level 1m under SWL, resulting in a height of + 6.5m. Under these dimensions the berm coefficient becomes 0.73.

With the use of the formula stated in the EurOtop manual, the wave run-up is calculated to be 7.11m high. Together with the design height of 10.53m, this leads towards a total design height of 18.84m. In this height the wall on top of the crest of 1.2m and the depth below CSWL are included. Subtracting those will end up with a new crest height of + 14.64m. The new and original heights and widths of the dikes are listed in Table 9.5.

Table 9.5 - Comparison original and new dike

<table>
<thead>
<tr>
<th>Level</th>
<th>Original Height [m +CSWL]</th>
<th>New Height [m +CSWL]</th>
<th>New width[m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toe</td>
<td>1.0</td>
<td>2.0</td>
<td>15.0</td>
</tr>
<tr>
<td>Berm</td>
<td>5.0</td>
<td>6.5</td>
<td>6.0</td>
</tr>
<tr>
<td>Crest</td>
<td>10.3</td>
<td>14.6</td>
<td>8.5</td>
</tr>
</tbody>
</table>

**9.3.2.1 Conclusion on dimensions**

A new dike is designed for a design return period of 1:1000 years, required to guarantee safety for the new occupation of the Tiaozini area. For the dimensioning of the berm and toe an optimization of materials is made, in which the later of the two has most influence. Optimization of the berm and toe dimensions led to a decrease of 2.2m in dike height.

**9.3.3 Geotechnical stability verification**

In section 9.1.2 the required crest height is determined. The optimal berm height and width are determined from a material point of view. It will be investigated whether this optimal geometry is valid from a geotechnical point of view. In order to do this the safety factor of the dike is determined and evaluated. It will be checked whether the dike is suitable of protecting the hinterland for a 1:1000 year storm.
9.3.3.1 Used method
For this calculation the same material parameters are used as for the original dike. The geometry is different however. Although the width and height of the berm and the crest height are determined in section 9.1.2 not the entire geometry is specified. The geometry of the inner slope and the height of the berm located at this slope are not specified. For the design the slope angle and berm height remain the same with respect to the original dike. The length of this berm and the height of the toe are increased. This decision is based on engineering judgment, considering the location of the fail mechanism of the original dike.

The thickness of the concrete armor is not increased. A concrete plate with a thickness of 1.3m is extremely stiff in comparison with the surrounding soil. The concrete armor is therefore not assumed to induce a critical fail mechanism.

9.3.3.2 Stability results
Figure 9.17 shows the settlement of the dike due to its own weight after construction. The maximum settlement is 0.58m, located at the crest of the dike. This settlement is caused by the densification of the three soil layers underneath the dike due to the weight of the dike and the densification of the dike itself due to own weight. This settlement is incorporated in the design by the retaining wall on the crest.

The failure of the dike is show in Figure 9.18 and Figure 9.19. It can be observed global failure occurs due to sliding of the outer slope. Figure 9.18 shows the deformation with respect to the original location at the dike and Figure 9.19 shows the deformed mesh after failure. The numerical estimation of the displacement at failure assumes unrealistic large values. This implies a critical state is reached.
It can be observed that two failure mechanisms occur simultaneously. This is also observed in Figure 9.20, two slip surfaces can be distinguished clearly. The occurrence of multiple slip surfaces simultaneously is not uncommon, especially in non-associated plasticity problems (Brinkgreve, 2012). The concrete armor encounters little deformation, justifying the assumption of not increasing the thickness made in section 9.3.3.1.
The failure occurs at a safety factor of 1.795. This value is lower than the original dike, but still above the desired safety factor of 1.5. This safety factor is verified in appendix O.

9.3.4 New dike conclusion
A new dike is designed for retaining a 1:1000 year storm required to guarantee safety for the new occupation of the Tiaozini area. In the design the same construction properties as the original dike are used. For a 1:1000 year storm new wave characteristics are determined using a Weibull distribution; \( H_s = 4.39 \text{m} \) and \( H_d = 7.53 \text{m} + \text{CSWL} \). For the dimensioning of the berm an optimization of materials is used leading to a decrease of 2.2m in dike height compared to a dike with no use of berm. This decrease in volume reduces the settlement of the crest and decreases construction costs of the dike.

The geotechnical stability tests of the dikes result in two different failure mechanisms. The original dike fails due to inner slope sliding, with a safety factor of 1.951, whereas the new dike fails due to outer sliding, with a safety factor of 1.795. This means the new dike is stable from a geotechnical point of view and can perform its task of protecting the hinterland for a 1:1000 year storm. This validates the Chinese method of dike construction. The geometry of the new dike is optimized from a material point of view, explaining the difference in safety factor.
10 Conclusion

The movement of industrial activity from coastal areas in China to inland locations threatens the economic development of Jiangsu. To maintain the current economic situation a large scale construction project in the Jiangsu tidal flat area is initiated. Due to Jiangsu’s characteristic coastal zone, it is possible to reclaim land on a large scale. The goal is to reclaim 21 new areas with a total surface of 1800km². In practice the construction project experiences development problems, such as, low diversity of occupation; uncertainty of economic potential and low scientific level of the development approaches. By implementing the Dutch layer approach, an attempt was made to create a preliminary design for the Tiaozini port area.

During the occupation analysis it was found that the best suitable industry to obtain lasting economic impulse is the petrochemical industry. The promising future potential of this particular industry was demonstrated in a macro-economic analysis. This analysis also indicated a positive foreign investment climate; further qualifying the financial foundation necessary for the new industrial area. Port analyses revealed that the major competitor ports were Dafeng, Binhai and Nantong. However, this field of competitors’ offers opportunity for a port area which focuses on petrochemical industry. The characteristic import and export of petrochemical industry, together with the positive investment climate, create the possibility of a public-private partnership. This causes a higher level of efficiency and enhances the port feasible. An industrial area containing petrochemical industry, adjacent to a harbor area specialized in bulk-cargo and container feeding services, is proven to be a valid occupation for Tiaozini. The connection between the occupation layer and the network and base layer was made in terms of requirements. The petrochemical focus sets specific criteria for the network layer. Since petrochemical industry is part of a supply chain of other products and hence relies heavily on import and export of feedstock and produced goods, this occupation requires a flexible transportation system with sufficient capacity. Moreover the network must contain different modes of transportation, preferably rail and inland shipping, and needs to be sustainable in order to meet government requirements. The latter can be achieved by clustering petrochemical activities. Particularly petrochemical industry greatly benefits from bundled activities, with the exchange of semi-manufactured products, residues and residual heat. Moreover companies can share infrastructures, for instance with waste management purposes. The designed six lane road network can provide the required capacity and allows flexibility for sub-sequential growth. The proposed occupation consists of advanced, modern, capital intensive factories and thus demands appropriate measures in terms of reduction of flood risk and subsidence. Since the new layers of the reclaimed land consist of very soft clay, increased risk of foundation failures and high settlements are likely. Consolidation of these layers is necessary to improve the soil characteristics and fulfill the residual settlement design requirements. Not all soil improvement methods are applicable on the Tiaozini area. Calculations confirm that surcharge preloading in combination with drains is the best solution. The optimal combination is found when applying a surcharge preload of 4.5m high, for duration of 6 months, in combination with prefabricated vertical drains.

The original dike was constructed using geo-tubes, an innovative construction method with great advantages such as, enabling phased construction, usage of local slurry as building material and allowing for the creation of steeper slopes. The short lifespan of Chinese geotextile should be incorporated into the design, since the initial benefits cannot be guaranteed in the operational phase. This means that the dike has to be constructed in such a way that it will remain stable without
the geotextile. The original dike protected low grade agricultural land against 1:50 year storms. However, in accordance with European codes, the original dike geometry is not high enough to fulfill this requirement. The calculated run-up of waves is 0.9m higher than originally calculated in the Tiaozini feasibility study. This difference can have severe consequences. Even though the original dike is not sufficiently high, the berm and toe were optimized correctly and with a safety factor of 1.9 the dike is determined stable. Since a potential flooding of the proposed petrochemical industry causes severe damage to people, material and environment, additional safety must be guaranteed. Accordingly the dike must be able to withstand 1:1000 year storms; this required the determination of the governing design wave height and redesign of the dike. A design wave height of +7.53m CSWL and significant wave height of 4.39m were found as the governing storm. The new dike was optimized in such a way that material usage was reduced to a minimum, without reducing the safety of overtopping. The influence of the toe on the wave run-up was found to be negligible, resulting in a similar toe design as the original dike. On the other hand optimization of the berm led to a decrease of the crest height of 2.2m, significantly reducing the dike volume and thus the costs of the construction. With a safety factor of 1.8 the dike was stable from a geotechnical point of view, yet, for a more reliable determination of the safety factor additional research regarding soil input parameters is needed.
11 Future research
During the research a lot of possible additional research topics were encountered. Not all problems could be addressed; in our opinion the following topics deserve attention:

- Possible liquefaction of hydraulic fills and soil inside the geo-tubes
  As a result of the loose packing of the hydraulic fill and the soil in the geo-tube liquefaction could occur during an earthquake or a typhoon. This might lead to global failure of the dike.

- Research about possible slip surface caused by scattered geotextile.
  Shatters of ripped geotextile will remain in the dike after the deterioration of the geo-tubes. These surfaces are smooth in comparison with the surrounding soil and could lead to possible slip surfaces.

- Increase of stability of the road embankment on soft soil due to phased construction.
  Different methods of phased construction can be investigated to research the stability of the road embankment.

- Economical study dike and consolidation
  An economical study could be performed on the dike and the different consolidation methods. This way a better optimization of the dike and more accurate MCA is possible.

- Soil investigation.
  A more detailed and complete soil investigation should be performed. With a more accurate dataset the assumptions can be checked and the calculated results are more reliable.

- Investigate reliability instead of Safety Factor.
  With a larger dataset of the soil parameters, a probabilistic (stochastic) approach can be used leading to a more meaningful definition of stability reliability. This reliability provides extra information about the probability that failure will not occur.

- Further research on optimization of new dike geometry.
  The geometry of the new dike geometry could be investigated, since the geometry is currently only optimized from a material point of view. Moreover the inner geometry of the dike is not reconsidered in the new design.

- Research methods to increase of natural deposition of sediments.
  At the Tiaozini project a sediment-stimulating dike is present. More research can be performed on the possibilities to increase the speed of deposition of sediments at Tiaozini.

- Navigability of the harbor.
  Due to the deposition of sediments in the area, the navigation channel for ships leading into the harbor can get clogged. Research should be performed on methods to minimize the amount of sediments deposited in this trench.

- Impact of the project on adjacent existing port areas.
  The economic effect of the Tiaozini harbor area on the nearby ports can be investigated. Besides this, the morphologic changes due to the Tiaozini project affecting these ports can be researched.

- Side effects of loss of coastal habitat.
  Morphologic and environmental research can be performed on the side effect of the loss of the shallow coastal area located at Tiaozini.

- Environmental impact of Tiaozini land reclamation.
  The environmental effect of the reclaimed land on the surrounding terrestrial area.
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## A. Appendix – Definitions

<table>
<thead>
<tr>
<th>Indicator</th>
<th>Explanation</th>
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<tbody>
<tr>
<td>Economic impulse</td>
<td>Argument to maintain and improve social economic development on short and long term.</td>
</tr>
<tr>
<td>Seaport</td>
<td>Port in the vicinity to the sea with a direct connection to the sea.</td>
</tr>
<tr>
<td>Industry</td>
<td>Refers to the material production sector which is engaged in extraction of natural resources and processing and reprocessing of minerals and agricultural products, including:</td>
</tr>
<tr>
<td></td>
<td>1. Extraction of natural resources, such as mining, salt production, logging (but not including hunting and fishing);</td>
</tr>
<tr>
<td></td>
<td>2. Processing and reprocessing of farm and sideline produces, such as rice husking, flour milling, wine making, oil pressing, cotton ginning, silk reeling, spinning and weaving, and leather making;</td>
</tr>
<tr>
<td></td>
<td>3. Manufacture of industrial products, such as steel making, iron smelting, chemicals manufacturing, petroleum processing, machine building, timber processing; water and gas production and electricity generation and supply;</td>
</tr>
<tr>
<td></td>
<td>4. Repairing of industrial products such as the repairing of machinery and means of transport (including cars).</td>
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<td></td>
<td>Prior to 1984, the rural industry run by villages and cooperative organizations under village was classified into agriculture. Since 1984, it has been grouped into industry.</td>
</tr>
<tr>
<td>Light industry</td>
<td>Refers to the industry that produces consumer goods and hand tools. It consists of two categories, depending on the materials used:</td>
</tr>
<tr>
<td></td>
<td>1. Industries using farm products as raw materials. These are branches of light industry which directly or indirectly use farm products as basic raw materials, including the manufacture of food and beverages, tobacco processing, textile, clothing, fur and leather manufacturing, paper making, printing, etc.</td>
</tr>
<tr>
<td></td>
<td>2. Industries using non-farm products as raw materials. These are branches of light industry which use manufactured goods as raw materials, including the manufacture of cultural, educational articles and sports goods, chemicals, synthetic fiber, chemical products for daily use, glass products for daily use, metal products for daily use, hand tools, medical apparatus and instruments, and the manufacture of cultural and clerical machinery.</td>
</tr>
<tr>
<td>Heavy industry</td>
<td>Refers to the industry which produces capital goods, and provides various sectors of the national economy with necessary material and technical basis. It consists of the following three branches according to the purpose of production or the use of products: Mining, quarrying and logging industry refers to the industry that extracts natural resources, including extraction of petroleum, coal, metal and non-metal ores and logging.</td>
</tr>
<tr>
<td></td>
<td>1. Raw materials industry refers to the industry that provides various sectors of the national economy with raw materials, fuels and power. It includes smelting and processing of metals, coking and coke chemistry, chemical materials and building materials such as cement, plywood, and power, petroleum refining and coal dressing.</td>
</tr>
<tr>
<td></td>
<td>2. Manufacturing industry refers to the industry that processes raw materials. It includes machine building industry which equips sectors of the national economy, industries of metal structure and cement products, industries producing means of agricultural production, such as chemical fertilizers and pesticides. According to the above principle of classification, the repairing trades which are engaged primarily in repairing products of heavy industry are classified into heavy industry while these engaged in repairing products of light industry are classified into light industry.</td>
</tr>
<tr>
<td><strong>Gross Industrial Output Value</strong></td>
<td>Is the total volume of industrial products sold or available for sale in value terms which reflects the total achievements and overall scale of industrial production during a given period. It includes the value of the finished products, which are not to be further processed in the enterprises and have been inspected, packed and put in storage, the value of industrial services rendered to other units, and the changes in the value of the semi-finished products and products in process between the beginning and closing of the period. The gross industrial output value is calculated with factory method. No double calculations are to be made within the same enterprise. However, double counting does occur among different enterprises. Output value of light and heavy industries is also classified with the factory method. Under normal conditions, if the major products of an industrial enterprise belong to light industry products, the gross output value of that enterprise is classified wholly into light industry; the same principle applies to heavy industry.</td>
</tr>
<tr>
<td><strong>Value-added of Industry</strong></td>
<td>Refers to the final results of industrial production of the industrial trade in money terms during the reference period.</td>
</tr>
</tbody>
</table>
| **Ratio of Profits to Total Industrial Costs** | Refers to the ratio of profits realized in a given period to the total costs in the same period, which reflects the economic efficiency of input cost and is calculated as follows: 
\[
\text{Ratio of Profits to Total Industrial Cost} \times 100\% = \left( \frac{\text{Total Profits}}{\text{Total Costs}} \right) \times 100\%
\]
| **Number of Berths in Main Coastal Ports** | Number of Berths in Ports and Quays: the number of berths where there are installations for vessels to berth, including quay berths, buoy berths, anchor berths, platform berths for lightering, etc. A position for one vessel to berth is counted as a berth. Berths are divided by their usage into production berths and non-production berths, and by berthing capacity into 10,000-tonnage berths. |
| **Seaport** | Port in the vicinity to the sea with a direct connection to the sea (Van Dale, 2012) |
| **Clearance gauge** | The space within the cross section in which no fixed objects are allowed. This applies for the horizontal as well as the vertical direction (Rijkswaterstaat Adviesdienst Verkeer en Vervoer, 2007) |
| **Obstacle free zone** | The area within which no obstacles are allowed (Rijkswaterstaat Adviesdienst Verkeer en Vervoer, 2007) |
| **Multi Criteria Analysis (MCA)** | A decision making tool that has been developed for complex problems. The method includes qualitative as well as quantitative aspects of the problems in the decision making process |
| **Complexes approach** | Developing nodes for exchange of information and knowledge |
| **Corridor approach** | Developing main ports and hinterland connections |
| **Rip-Rap** | Heavy stone placed on the berm of a dike to provide protection against erosion. Rip-rap is a permanent, erosion-resistant layer intended to prevent soil erosion in areas of concentrated flow, turbulence or wave energy. |
B. Appendix - Site Visit

B.1 Site visit day 1; Tiaozini Reclamation

Monday 13th of August

We woke up early this day for a field trip towards the tidal flat project. The five of us were picked up by prof. Jiang, Asso. prof. Gong and mr. Lee, for a trip of four hours from Nanjing to Nantong. The four hours of travelling flew by while we were chatting with our professors about the differences between China and the Netherlands. During a very nice lunch we gathered a lot more information about Chinese traditions, making it all very educative as well.

After lunch we headed for the first of the three connected reclamation areas early in the afternoon. The road took us over an old sea dike, but we weren’t able to see the sea at all. According to dr. Gong the whole coastline of Nantong is moving 200 meters seawards per year, due to sedimentation. This is a natural process which has been going on for a very long time. Only at this moment the joint effort of Chinese government and researchers is interfering to control the sedimentation, so that the shape of the future land can be chosen.

Our first sight on the reclamation area made us clear how big the total project was, but reminded us as well of the Netherlands. All the dikes, windmills and sandy coasts did us feel home.

Arrived at the area map in the middle of the reclamation area, the local manager pointed out where we were and what was yet to build. This was a great opportunity to ask a lot of technical and general questions about the project.

Figure B.1 - Planned land reclamation

B.1.1 The construction and enclosure of the reclamation

The reclaimed land is constructed in stages by building dikes area by area. The several stages were also shown on the billboard. Out of the billboard it seems that the whole first reclamation, Tiaozini, is divided into 7 stages.

Around every area a dike is constructed, except for a distance of 300 meters, which is called the mouth. Through this mouth the reclaimed area will grow naturally due to sediment transport that occurs during the tidal changes. When the area is completely filled, the outer dike can’t be constructed yet, the volume of the water in the area will be too large and thereby leading to too high
velocities for enclosure. Within the phase that is surrounded by the dike and is in contact to the sea by the mouth several small dikes of geo-tubes are constructed. These dikes have two functions: They act as a threshold to increase the amount of deposited sediment during tidal flow. The second function is that the total capacity of the basin is decreased. Since the small dikes are half the height of the high tide, the total volume of tidal flow is decreased.

**B.1.1.1 Dikes**
The dikes were built with the dimension to withstand wave height with a 50 year return period, similar to a height of 7.5 meter +MSL. Quite low compared to the Netherlands, but according to the Chinese high enough because of the low quality of land behind the dikes. The berm is located at height as a wave height with a return period of 20 years.

The dike is constructed with the use of geo-tubes. These are sausages filled with slurry of granular soil and water that are permeable enough to let the water flow through the geotextile. After dissipation of the water a packed bag of soil is left behind, strong enough to build a dike.

All the dikes had a slope of 1:2 as well above as under the berm. This is quite high compared to the slopes most of the time used in the Netherlands (1:3-5). Actually don’t know if this is because the use of geo-tubes. At least the used sand has a different composition than in the Netherlands.

**B.1.1.2 Dike protection.**
The dike is protected underneath the berm with a prefab concrete plate. This plate has a rectangular form with horizontal bar-formed holes. The holes are created to break the waves in an early stage. All the water that enters one of the holes in the plate can return towards the sea underneath the plate, in space between the surface of the dike and the plate.

On the plates were hooks as well. They are on the plates because the plates are re-used and this makes transportation and construction very easy. The land reclamation is done in different phases. The dike we visited was at this moment at the frontier, but later another dike will be constructed in front of this dike. For that reason the wave protection has only to be temporary and can be reused. The inland dikes themselves will be maintained for extra safety.

**B.1.1.3 Water control**
Because it seemed to be that the most of the area will be used for agriculture, there must be kept an eye on the salinity of the water. In order to get no salt water within the dikes, the fresh water level will be kept higher than the sea level. The fresh water needed for this has to come from the mainland and will flow through canals nearby the dike. This all means that there is need for control systems on the Chinese mainland.

For the other reclamation areas more out of the coast this idea is not suited because there is no transport of fresh water available.

**B.1.1.4 Further use of the land**
Whether or not the other reclamation areas will be constructed is not sure. This depends on the construction of the first area and the development in this area. For the two other islands a feasibility study has been performed. A plan has been made for the method of construction and research has been performed on the hydraulic conditions. The three islands are not planned as one linked project.
Only if the first island, Tiaozini, provides enough economic boosts for the region a new island might be considered.

B.1.2 Diner
After the first part of the field trip, the reclamation area, we headed back to the hotel. When we arrived it was already time for the diner. Suddenly there appeared a special guest, the president of the tidal flat project Dr. Wong. For us it was a great opportunity to meet the president of the project and we were honored that he was willing to spend his time with us. It was a very nice dinner, with a lot of opportunities to ask questions.

B.2 Site visit day 2; Dafeng Port.
After a breakfast at 08.00 AM a long trip was organized to the Dafeng port area. This project not only included the visit to the harbor but also to the adjacent industrial and residential development zone. In the Dafeng Port Area exhibition center a good impression was given of the planned development:

Figure B.2 - Overview Dafeng harbor area

B.2.1 Dafeng Port Zone
During the day the planned development of the Dafeng Port Zone was explained. This zone does not only include the actual harbor, but also several Industrial and Residential zones.

B.2.1.1 Dafeng Harbor Zone.
The Dafeng harbor contains two different parts. An already build container terminal and a newly constructed part containing five long piers of 11km into the deeper sea. At the end of these piers bulk cranes and container lifters were placed. Due to the length of the piers and some required dredging Dafeng is the only port of Jiangsu accessible for deep sea ships. The port of Dafeng is mainly used to connect the broad hinterland with the nearby ports of Nagasakai (Japan), Busan (South Korea), Quinhungdaco (China), Lianyungchang (China) and Shanghai (China). The Dafeng port will not compete with the port of Shanghai and Busan, but merely use the ports as a springboard for the required connection to Europe and America. The port is planning on shipping 100 million tons per year in the future after all phases are completed. At this time however only 12 million tons is achieved. The development time from 12 million tons till 100 million tons is estimated on a period of roughly 5 years.
B.2.1.2  **Dafeng Port economic zone.**
The expansion of the port is believed to lead to a development of the economic zone situated nearby the port. The economic zone is planned to be 208 square km and will be divided in several different areas: ‘Special steel new material industry park’, ‘Paper Making Industry Park’, ‘Marine Biotechnology Park’, ‘Petrochemical Industry Park’ and ‘Stone Industrial Park’. These different parks will consist of clusters of identical industry which will aid each other in their development. However all the infrastructure and some nice buildings are present, but the industry still has to develop.

B.2.1.3  **Dafeng Residential Zone.**
With the planned port and economic zone, a large amount of extra residents are expected. For these residents a new residential area is constructed. In order to make the new zones attractive several like high tech buildings such as Nanyang middle school, Theme park for children, Shakespeare town and the Peninsula hot spring hotel.

B.2.2  **Dafeng competition strategy**
According to Nationwide Coastal Port Distribution Plan in 2006, Dafeng Port became an important constituent part of the port group in Yangtze River Delta. According to Jiangsu Costal Development Strategic plan in 2009, Lianyungang, Dafeng Port and Port of Nantong will be jointly constructed into three strategic supporting points of coastal development. In 2006, Dafeng Port was officially approved by the State Council to be a national class-one opening port for the coastal development of Jiangsu and Dafeng Port has become an important blue economic growth pole. In order for Dafeng to be an attractive port area, the port area has combined a port zone, economic zone and residential zone including attractive facilities to maintain a key competitor in the market.

The total costs for the new Dafeng Port Area consist of 1.2 trillion Yuan. This is equivalent to 150 billion Euro’s. Although this is an enormous investment, the R.O.I. is expected to lie between 5 and 10 years. However this promising return of investment range, there is a side note regarding the calculation construction of the R.O.I. Not only the harbor area is included as an input factor for the total income and the total assets, but also the total income and the total assets of the economic zone and economic zone is included.

Figure B.3 - Dafeng jetty

B.2.3  **The lunch and port visit**
After the exhibition an oceanographic museum was visited. This was located in the middle of the future ‘Marine Biotechnology Park’, which was more amusement. After this visit a lunch was planned
with the president of the Dafeng Port area construction project. Also during this lunch the Chinese traditions were not neglected.

This gave us the opportunity to ask all question related to the Dafeng Port area. Were the site visit of day 1 gave us the opportunity to ask all questions about the executional and technical aspects of the project, this visit gave us the opportunity to find out all about the planning and used strategy. In that way the two different visits were complementary.

After the lunch we went to visit the actual harbor of Dafeng. It was good to talk about the strategies and economical models, but as true Delft engineers we had to see the harbor or ourselves. The harbor contains of several long piers of 11 km. For us it was strange that only one small pier is present for a quay with multiple cranes. It seemed like a logistical nightmare to have only one pier and the place to store the goods located at the container terminal in the north. It however was explained that these harbors act as the harbors for the industries in the vicinity of Dafeng, and that the logistical areas are located near these industries. As Dutch students we compared this harbor to the port of Rotterdam, which is a transit harbor. This harbor however is not in the middle of a transport chain, but at the beginning.
C. Appendix – Occupation analysis

C.1 Market research

In order to objectively state what type of industry will be able to provide a significant economic impulse in the region, social economic and geographical market researches concerning Jiangsu province are performed.

However, first, one will have to establish general consent about the meaning of economic impulse. Oxford University Press describes an impulse in the following way: *Something that causes something to happen or happen more quickly* (Oxford University Press, 2010)

Adding the economic relevance to this definition, economic impulse is described in the following way: Economic impulse (driver) - argument to maintain and improve social economic development on short and long term.

The goal of a port on Tiaozini is to create long lasting significant economic impulse in the region, agriculture is not considered to be a viable option. Revenues per square kilometer (¥/km²) are lower for agriculture than for industry and since reclaiming new land in sea is capital intensive, industry will pay back the expenses made for the project faster. Moreover industrial products are more likely to be exported using an international port. Therefore agriculture as a possible application on Tiaozini will not be considered in this analysis.

C.2 Social economic research

In this section the social economic situation in Jiangsu province is described thoroughly.

C.2.1 Gross Regional Product

Figure C.1 shows that Jiangsu had the second highest total GRP in China in 2010, after Guangdong Province. With a total of 4143 billion RMB, 10.33% of the GDP is produced in Jiangsu.

![Figure C.1 - Top 10 GRP of Chinese provinces](National Bureau of Statistics, 2011)
Moreover with an GRP per capita of ¥ 52840, Jiangsu is the fourth highest in China. As Figure C.2 shows, the average income in Jiangsu is 1.76 times higher than the average of China. However a wealth gap between the prosperous south and poorer north has led to unequal economic growth in Jiangsu. Cities like Nanjing, Suzhou and Wuxi have GRP per capita around twice the provincial average, making south Jiangsu one of the most prosperous regions in China (Netherlands Business Support Office, 2011).

![Figure C.2 - GRP per capita per province, compared to China (National Bureau of Statistics, 2011)](image)

The past 30 years Jiangsu has developed itself to be one of the key industrial provinces in China. Figure C.3 shows the development and division of its GRP. From this figures one can derive that, 52.51% of the provinces GRP is produced in the secondary sector. This sector includes industry and construction. By far the biggest proportion is generated by local industry.

![Figure C.3 - Development of Jiangsu's GRP over the last 30 years](image)
With its ideal coastal location, suitable for international and domestic trade, Jiangsu’s economy has been thriving in the last years. With an average annual growth of 13.52% over the last five years, Jiangsu performs better than the nation.

Figure C.4 - Jiangsu’s GPR indices compared to China’s GDP indices (National Bureau of Statistics, 2011)

Figure C.5 shows the potential growth indices of Jiangsu and China in 2010. This figure shows that most development is concentrated in the secondary and tertiary sectors. Both sectors perform better in the province than the nation’s average.

Figure C.5 - GDP development of Jiangsu, split out over the three sectors of industry (National Bureau of Statistics, 2011)

C.2.2 Industry

Jiangsu has mainly focused on heavy industry (energy, steel and shipbuilding) and technological industries (electronics, automotive and petrochemical industry). Jiangsu’s geographical location, rich talent resources and perfect industrial base, provide powerful personnel and technological support for the development of its coastal industry. Additionally it owns multiple managerial and technical talents.

The top ten industrial product output for Jiangsu relative to Chinese total production, is shown in Figure C.6. This graph indicates that the electronic and petrochemical industry in Jiangsu is responsible for over 30% of the nation’s production.
Figure C.6 - Industrial output of Jiangsu in 2010 in aspect to the total production of China (National Bureau of Statistics, 2011)

Figure C.7 indicates the ten relevant industries with the highest profit to industrial cost ratio in Jiangsu. Only industries relevant for Tiaozini are shown, since the mining of resources will not be applicable on the newly reclaimed land. The industries which are present in the region are highlighted. This shows that most of the industries present in Jiangsu, have a high profitability. Figure C.6 and Figure C.7 show that most industrial activity in the region is capital-intensive.
C.2.3 Import & Export

With the second largest import and export value in China, ports are of great importance to the economy of Jiangsu. Figure C.8 shows most of Jiangsu’s economy is based on import and export.
C.2.4 Foreign investments

Trade, rapid annual growth, attractive geographical location and highly educated people, cause Jiangsu to have an attractive foreign investment climate. Figure C.9 shows Jiangsu is the leading province when it comes to foreign financiers. This is not a recent development; Jiangsu had been building international relations for a long time and has expertise in working with culture difference between Chinese and foreign countries.

C.3 Geographic research

As stated in section C.1, in order to be able to state which type of industry should settle on the new reclaimed land of Tiaozini, a thorough geographical analysis is made. In order to objectively limit the
amount of ports that will have to be analyzed; only the seaports in Jiangsu province are taken into account.

A map listing all the sea ports is shown in Figure C.10. Most of the harbors in the area are relatively small. However there are a lot of plans to expand the current capacity in these coastal regions. For instance Dafeng, Binhai and Dayang have presented plans to construct ports suitable for large sea vessels. The properties of all sea ports are given in Table D.1. The following ports will be described in more detail: Dafeng, Binhai, Shanghai, Lianyungang, Nantong and Tiaozini.

![Figure C.10 - Jiangsu province map, including all seaports](image)

**C.3.1 Dafeng port**

The Dafeng harbor consists of two different parts. The container terminal is already finished the part containing five long piers of 11km into the deeper sea is still under construction. At the end of these piers bulk cranes and container lifters are placed. Due to the length of the piers and some required dredging Dafeng is the only port of Jiangsu accessible for deep sea ships. The port of Dafeng is mainly used to connect the broad hinterland with the nearby ports of Nagasaki (Japan), Busan (South Korea), Quinhuongdao (China), Lianyungchang (China) and Shanghai (China). The Dafeng port will not compete with the port of Shanghai and Busan, but merely use the ports as a springboard for the required connection to Europe and America. The port is planning on shipping 100 million tons per year in the future after all phases are completed. At this time however only 12 million tons is achieved. The development time from 12 million tons till 100 million tons is estimated on a period of roughly 5 years.

According to Nationwide Coastal Port Distribution Plan in 2006, Dafeng Port became an important constituent part of the port group in Yangtze River Delta. According to Jiangsu Costal Development
Strategic plan in 2009, Lianyungang, Dafeng Port and Port of Nantong will be jointly constructed into three strategic supporting points of coastal development. In 2006, Dafeng Port was officially approved by the State Council to be a national class-one opening port for the coastal development of Jiangsu and Dafeng Port has become an important blue economic growth pole. In order for Dafeng to be an attractive port area, the port area has combined a port zone, economic zone and residential zone including attractive facilities to maintain a key competitor in the market.

C.3.2 Binhai port
Currently Binhai port is under construction. Unfortunately information about this port area and its future plans are scarce to obtain. According to interviews with professors of the Hohai University one of three China’s big power state company has planned to build an energy plant near Binhai port. Because of this energy plant there is a high demand of coal energy which is imported by means of Binhai port. The buildup of the harbor will be carried out by big investments. Followed by consolidating freight transportation, storage and energy production, with the full play of modern harbor equipped resources and regional organized economic functions, the Binhai harbor gradually transformed into a commercial industrial interfaced regional harbor. Clustered with Lianyungang, the Binhai harbor will be the new bridgehead to Eurasia continent, and it will turn into an important passage to the ocean for the regional area.

C.3.3 Shanghai
With a total trade of 30 million Twenty feet Equivalent Unit (TEU) and 563 million tons of bulk goods Shanghai is the biggest port in the world (National Bureau of Statistics, 2011). Shanghai port consists of five mayor areas, enabling it to trade all possible goods, from oil to car parts and from food to cement. However the main industry and trade in the area is related to the petrochemical and mechanical industry. Not only has Shanghai rail and road connections with the hinterland, it also provides air and ship connections, creating possibilities to transport goods using the necessary modality. Chongming Island, which is part of Shanghai province, is partly covered with port related industries.

C.3.4 Lianyungang
With a total capacity of 3.4 million TEU’s and 127 million tons of bulk goods Lianyungang is the biggest port in Jiangsu province. Lianyungang focuses on the energy market and consequently most of its goods are used in the local power plants and adjacent petrochemical industry. Just like Shanghai, it is the only port in the province that is able of receiving the biggest sea vessels currently existing, with a maximum capacity of 300.000 tons. Lianyungang has a ship, road and rail connection with the hinterland. Southeast of Lianyungang lies Guanyun port, the industrial development area of Lianyungang.

The development of the volume of freight handled in the two mayor ports in the region over the past sixty years is shown in Figure C.11. The past ten year China has been focusing on trade both nationally and internationally, influences the positions of the harbors in China.
C.3.5  Nantong
Nantong is the only seaport positioned in the estuary of the Yangtze River. This port mainly focuses on steel and shipbuilding industry. However due to the recent economic developments, the demand for new ships has decreased significantly and the market dried up. Therefore the port is more depended on its steel industry. With an industrial area with a direct connection with the sea, called Yangkou, Nantong tries to strengthen its position along the coast.

C.3.6  Tiaozini
Tiaozini is located strategically between Lianyungang and Shanghai (Figure C.10). With no adjacent competing ports, besides some of the smaller ports, Tiaozini has a favorable position. If Tiaozini is developed as a national hub which is able to collaborate with these bigger ports, the location can be of great benefit for the port. Instead of trying to compete with the other ports, it is better to work together and achieve synergy between them. When producing the same goods, scale advantages can be achieved.
## D. Appendix – List of port properties

Table D.1 - List of properties of sea ports Jiangsu

<table>
<thead>
<tr>
<th>#</th>
<th>Name area</th>
<th>Operating area</th>
<th>Port size</th>
<th>Function</th>
<th>Specification</th>
<th>Industry</th>
<th>Capacity [10^3 TEU]</th>
<th>Capacity [MTon]</th>
<th>Pier depth [m]</th>
<th>Number of Berths</th>
<th>Max. shipsize [KTon]</th>
<th>Hinterland connection</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Shanghai</td>
<td>Shanghai</td>
<td>Very large</td>
<td>Trade &amp; Transport</td>
<td>Containers</td>
<td>Petro chemistry, steel, minerals, construction material and mechanical equipment</td>
<td>30000</td>
<td>563.20</td>
<td>12.5</td>
<td>1160</td>
<td>400000</td>
<td>Road + rail + ship + air</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Lianyungang</td>
<td>Lianyungang</td>
<td>Large</td>
<td>Industry</td>
<td>Power plant</td>
<td>Containers, food (rice), coal, construction materials and petrochemical</td>
<td>3400</td>
<td>127.39</td>
<td>11.5</td>
<td>55</td>
<td>300000</td>
<td>Road + rail + ship</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Nantong</td>
<td>Nantong - Yangkou</td>
<td>Medium</td>
<td>Industry</td>
<td>Steel</td>
<td>Containers, electronic machinery, shipbuilding, metal</td>
<td>446</td>
<td>58.70</td>
<td>10.8</td>
<td>24</td>
<td>100000</td>
<td>Road + rail + ship</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Yancheng</td>
<td>Yancheng</td>
<td>Small</td>
<td>Industry</td>
<td>Building material</td>
<td>Energy, petro chemistry, construction materials and electronic machinery</td>
<td>37</td>
<td>21.01</td>
<td>15.0</td>
<td>66</td>
<td>30000</td>
<td>Road + rail + ship</td>
<td></td>
</tr>
</tbody>
</table>

Binhai * | Small | Industry | Power plant | Fishery, steel, petro chemistry and shipbuilding | - | - | - | - | - | Road + rail + ship | Under construction |

Sheyang | Medium | Industry | Building material | Energy, paper, electronic machinery, light industry and food | N/A | 35.00 | 10.0 | 5 | 50000 | Road + rail + ship |         |

Xiangshui | Small | Industry | Building material | Construction materials, coal, petro chemistry, minerals and food | N/A | 4.94 | 10.0 | 24 | N/A | Road + rail + ship |         |

Dafeng* | Small | Industry | Power plant | Energy, petro chemistry, steel, wood, medicine, electronics and wind energy | 28 | 12.37 | 15.0 | 8 | 50000 | Road + rail + ship + air | Under construction |

Lvsi | Small | Fishery | Fish landing | Energy, petro chemistry, steel and chemical industry | N/A | N/A | 16.0 | N/A | 50000 | Road + rail + ship |         |

Dayang * | Small | Industry | Power plant | Energy and petro chemistry | - | - | - | - | - | Road + rail + ship | Under construction |
E. Appendix – Detailed road design

Figure E.1 - Road design including pavement structure and dimensions
Table E.1 - Width of the road

<table>
<thead>
<tr>
<th>No.</th>
<th>Component</th>
<th>Amount</th>
<th>Dimensions [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Traffic lane</td>
<td>3</td>
<td>3.50</td>
</tr>
<tr>
<td>2</td>
<td>Hard shoulder</td>
<td>1</td>
<td>3.25</td>
</tr>
<tr>
<td>3</td>
<td>Verge</td>
<td>2</td>
<td>0.20</td>
</tr>
<tr>
<td>4</td>
<td>Hard strip</td>
<td>1</td>
<td>0.60</td>
</tr>
<tr>
<td>5</td>
<td>Berm</td>
<td>1</td>
<td>4.00</td>
</tr>
<tr>
<td></td>
<td>Total width for a one way section</td>
<td></td>
<td>18.75</td>
</tr>
</tbody>
</table>

Table E.2 - Pavement structure

<table>
<thead>
<tr>
<th>Layer</th>
<th>Type</th>
<th>Density [kg/m³]</th>
<th>Thickness [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface layer</td>
<td>Stone Mastic Asphalt D= 0 – 11 mm</td>
<td>2450</td>
<td>35</td>
</tr>
<tr>
<td>Intermediate layer</td>
<td>Asphalt Concrete D_{max}=16 mm</td>
<td>2200</td>
<td>50</td>
</tr>
<tr>
<td>Bottom layers</td>
<td>2x Asphalt Concrete D_{max}=22 mm</td>
<td>2200</td>
<td>140</td>
</tr>
<tr>
<td>Base</td>
<td>Hydraulic mixed granulate</td>
<td>1800</td>
<td>400</td>
</tr>
<tr>
<td>Sandbed</td>
<td>Sand</td>
<td>1850</td>
<td>525</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td>1150</td>
</tr>
</tbody>
</table>

Table E.3 - Traffic load after 45° spread (Nederlands Normalisatie-instituut, 2009)

<table>
<thead>
<tr>
<th>Traffic Class</th>
<th>Wheel Load [kN]</th>
<th>Pavement thickness [m]</th>
<th>Max. nr of wheels</th>
<th>Uniformly distributed traffic load [kN/m²]</th>
<th>Total traffic load [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>VOSB class 600 (axle load 200 kN)</td>
<td>50.0</td>
<td>0.25</td>
<td>2</td>
<td>98</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>50.0</td>
<td>0.50</td>
<td>4</td>
<td>46</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>50.0</td>
<td>0.75</td>
<td>8</td>
<td>35</td>
<td>4.0</td>
</tr>
<tr>
<td>OSB class 450 (axle load 150 kN)</td>
<td>37.5</td>
<td>0.25</td>
<td>2</td>
<td>73</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>37.5</td>
<td>0.50</td>
<td>4</td>
<td>34</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>37.5</td>
<td>0.75</td>
<td>8</td>
<td>27</td>
<td>3.0</td>
</tr>
<tr>
<td>VOSB class 300 (axle load 100 kN)</td>
<td>25.0</td>
<td>0.25</td>
<td>2</td>
<td>49</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>25.0</td>
<td>0.50</td>
<td>4</td>
<td>23</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>25.0</td>
<td>0.75</td>
<td>8</td>
<td>18</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>25.0</td>
<td>1.00</td>
<td>8</td>
<td>13</td>
<td>2.0</td>
</tr>
</tbody>
</table>
F. Appendix – Soil improvement methods
A short introduction will be given on the most commonly used soil improvement methods. It will be discussed what soil conditions these methods will be best for. These methods can roughly be divided into three classifications; remove and replace, densification and consolidation.

F.1 Remove and replace
The oldest, least complex and most reliable way to improve the characteristics of the soil is replacement. The soft soil is removed and replaced by a good quality foreign material that has the required parameters. This method will be rejected for the rest of this research because it is evident that this method will not be applicable due to the enormous volumes required for the improvement of the Tiaozini reclamation.

F.2 Densification
The purpose of densification is to improve the strength and to reduce the settlements of loose granular soils by ensure a tighter packing of the particles. It is possible to predict the density after densification, based on accumulated experience and engineering judgment. The prediction should later be confirmed by additional field tests.

F.2.1 Dynamic Compaction
Dynamic compaction is an in-situ soil improvement method in which dropping a heavy drop weight of steel or concrete, from a height of about 25m to densifies the soil. The drop weight (usually around 20 tons) is lifted by a crane and repeatedly dropped on the ground surface in a grid like pattern. The spacing of the grid and the number of passes is determined by the soil properties and the desired result. Typically five passes are required, in which the distance between the impact points decreases in the subsequent passes. The impact holes are backfilled with granular material after each pass. The last pass is used to densify the top layer without disturbing the already densified deeper layers. This pass is called the ironing-pass and is usually performed with lower compaction energy by using a lower drop height.

![Figure F.1 - Dynamic Compaction Grid Pattern](image_url)

After the by compaction induced primary settlements, secondary settlements may occur due to dissipation of excess pore water. Nevertheless dynamic compaction is a proven technology to increase bearing capacity, reduce liquefaction potential and reduce settlements.

Dynamic compaction is most effective for granular deposits with a low degree of saturation, high permeability and high soil mass. All compaction above groundwater table is immediate because the
soil particles are forced into a denser state of packing, without any impediment. For compaction below the groundwater table it is important that the permeability is sufficiently high to allow excess pore water pressures generated by the impact of the drop weight to dissipate.

Problems arise when the granular deposit is interlayered with silty or clayey soil layers. The impact energy from the drop weight is absorbed by these intermediate soft cohesive layers because the shear waves are damped. The compression waves are not damped, but are less important for compaction. Due to this damping effect of soft cohesive layers, compaction of the underlying layers will not occur.

To determine the depth to which the compaction is effective for a given soil deposit, we can use the following empirical relationship.

\[ d_{\text{max}} = \alpha_{\text{comp}} \sqrt{H_{\text{drop}} \cdot M_{\text{drop}}} \] (Massarsch, 1999)

In which \( \alpha_{\text{comp}} \) is the compaction coefficient, \( H_{\text{drop}} \) and \( M_{\text{drop}} \) are the drop height and the drop weight respectively.

F.2.2 Vibro-compaction

Vibro-compaction is an in-situ soil compaction method in which a vibratory probe is inserted into the ground. The probe can either vibrate vertically (Vibro-rod) or more commonly the probe vibrates horizontally (Vibro-flotation) to densify the granular material. Vibro-compaction improves strength, reduces total and differential settlements and drastically reduces the risk of liquefaction.

The soil must be permeable enough to allow rapid drainage of pore water and have a high enough friction angle to permit passage of the compaction shear waves. The grains should not easily be crushable. Therefore Vibro-compaction only works for non-cohesive granular soils. Typically depths between 5 and 15m can be compacted, but it is possible to use vibro-flotation up to 70m in depth. The top 5m usually loses strength during Vibro-compaction due to the lack of vertical confinement at the surface.

Construction process;
1. The probe is lowered to the required depth under its own weight and the controlled use of horizontal vibrations and the effect of jetted water.
2. When the required depth is reached the water jets at the tip of the probe close, leaving only the jets at the top of the probe open.
3. Excessive vibrating of the cone causes the grains to re-organize.
4. A granular backfill material can be inserted from the surface (top feed).
5. The probe is gradually lifted in successive passes of about 0,5m.
6. The obtained cavity should be backfilled with additional granular material.
Consolidation is any process which involves decrease in water content of a saturated soil without replacement of water by air (Terzaghi, 1943). Because $\sigma' = \sigma - p$, adding additional stress or reduce the pore pressure leads to a higher effective stress and hence a denser state of packing. For saturated soil this results in water being squeezed out of the soil.

The total settlement from consolidation can be divided into three components;

$$S_{tot} = S_{in} + S_{prim} + S_{cr}$$

In which $S_{tot}$ is the total settlement and $S_{in}$, $S_{prim}$, and $S_{cr}$ stand for the instantaneous, primary and creep settlement respectively.

### F.3.1 Surcharge induced preloading

Preloading is a simple way to accelerate consolidation in order to compensate or eliminate post-construction settlements. By applying an overburden pressure, the stresses and the pore water pressures in the soil increase. When the excess pore water slowly dissipates from the less permeable layer, the pore water pressures decrease but the increased stresses remain. Preloading increases the bearing capacity and reduces the compressibility of weak ground by forcing soft soils to consolidate (van Impe, 1989).

Applying a surcharge load will increase both vertical as well as shear stresses. This increase of shear stress may lead to stability problems. To prevent these problems the load can be placed in several stages. Loads that exceed the final construction load will be referred to as surcharge loads while other loads will be referred to as preloads as show in Figure F.5.
The temporary surcharge load may be removed after the desired part of the consolidation ($S_{\text{primary}}$) has taken place. The secondary settlements can be reduced by increasing the surcharge load or increase the time the surcharge load is applied. Increasing the surcharge load above the design work loads will ensure the soil being in an overconsolidated state after construction. Secondary settlements are smaller for overconsolidated soils than for normally consolidated soils. This will benefit greatly the subsequent geotechnical design (Chu, Bob, & Choa, 2004).

Surcharge induced pre-load has a long track record of success (over 50 years) for the accelerating of consolidation. Because it only requires conventional earthmoving equipment, every grading contractor can perform the work. The surcharge fill must extend at least 10 meter beyond the boundaries of the future planned construction. This might cause some difficulties in confined sites. Large quantities of soil are required for a long time but can later be re-used. Surcharge induced preloading works best for cohesive soil of small to intermediate thickness, however because consolidation time increases exponentially with depth, small thicknesses are desirable.

**F.3.2 Prefabricated vertical drains (PVD)**

With increasing thickness of the cohesive layer the time for consolidation expands exponentially (Shi, 2004). For most applications adding additional surcharge load isn’t economically efficient or technically feasible for thick impermeable layers. To accelerate consolidation, it is possible to shorten the drainage path to speed up the dissipation of pore water by the application of prefabricated vertical drains in combination with a surcharge load. Vertical drains are artificially created drainage paths in the cohesive layer. This method was first introduced in California, United States of America in the 1930’s. In Sweden Kjellman introduced the first prototype of a prefabricated vertical drain made entirely of cardboard (Jamiolkowski, Lancellotta, & Wolski, 1983)
With the use of PVD, the surcharge load can be decreased to still obtain the same consolidation time. A lower surcharge load leads to less increase in shear stress and therefore a lower chance on instability issues. An additional advantage is that PVD takes advantage of the higher horizontal permeability of the cohesive layer in relation to the vertical permeability. The typical h.t.h. distance of adjacent drains is about 3m. This allows a fast dissipation of pore water and hence low consolidation time.

**F.3.3 Vacuum preloading**

The method of vacuum preloading was introduced by Kjellman in 1952. Vacuum preloading is a way of accelerating the consolidation process by reducing pore water pressures in soft soils. Because total stresses maintain constant throughout the process, the effective stresses are increased (Terzaghi, 1943). This in turn leads to an acceleration of the consolidation process.

To reduce pore water pressures the soil site is covered with an airtight membrane, a system of vertical drains is installed and a drainage layer (sand) is applied on top. The ends of the membrane are placed in a trench filled with bentonite to maintain air tightness. Horizontal drains are installed in the drainage layer and subjected to vacuum pressures. This vacuum pressure can provide an equivalent surcharge induced preload of up to 4.5m (80kPa).

For very soft and instable soils, applying even the smallest fill can already lead to instability problems. Vacuum preloading can then be applied and has the additional advantage leading to isotropic consolidation. Isotropic consolidation eliminates the risk of failure under additional loading of the permanent construction, there is no risk of slope instability beyond boundaries and it allows a controlled rate and magnitude of loading and settlement (Masse, Spaulding, Varaksin, & Wong, 2001). Additional advantages of vacuum preloading are that no extra fill material is needed, generally shorter consolidation times, no chemical admixtures (eco-friendly) and no heavy machinery is needed. Until recently vacuum preloading was not seriously investigated as an alternative to conventional surcharge preloading, due to low costs of placing and removing surcharge fills and the difficulties involved in applying and maintaining the vacuum. The steadily increasing direct and

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**Figure F.7 - Surcharge induced preloading with PVD**

**Figure F.8 - Vacuum preloading**
indirect costs of placing and removing surcharge fill and the advent of technology for sealing landfills with impervious membranes for landfill gas extraction systems, have now made vacuum-consolidation an economically viable method as a replacement for or supplement to surcharge fill (Terashi & Juran, 2000).

Technical difficulties that might occur when applying vacuum preloading:

- Maintaining a leak proof system. The entire membrane is a problem area, but in particular the connection between the pumps and the membrane are very fragile.
- Maintaining an effective level of vacuum.
- Anchoring and sealing the membrane at the bentonite trench.
- Maintaining an effective drainage system that expels both water and air throughout the whole process.
- Reducing lateral seepage towards the vacuum preloaded area.

According to (Masse, Spaulding, Varaksin, & Wong, 2001), unsuccessful attempts have been recorded in the past for technological reasons. However, a major obstacle to development of vacuum based consolidation is the lack of understanding of its basic principles.

To increase the effectiveness of the vacuum preloading it can be combined with surcharge load. Field studies show that substantial cost and time can be saved by this technology, when compared to conventional surcharge load. The performance of the vacuum preloading is evaluated based on pore pressure as well as settlements.

Construction process (Cognon, Juran, & Thevanayagam, 1994)
1. Place a free drainage blanket (60-80 cm of sand) in order to provide a working platform.
2. Install prefabricated vertical drains (PVD’s) and relief wells.
3. Install horizontal drains at the base of the sand blanket.
4. Excavate trenches around the preload perimeter of about 50 cm and fill with bentonite.
5. Install impermeable membrane on ground surface and seal it to the bentonite trenches.
6. Connect vacuum pumps to the prefabricated surcharge module extending from the trenches.
7. Create vacuum pressure in the soft soil to allow an accelerated consolidation process.

F.3.4 Electro osmosis
This method’s first practical application was in 1939 when L. Casagrande used it for stabilizing a railroad cut in Germany. Water particles are transported by a potential difference on two electrodes. This potential difference causes positive charged ions to move to the cathode and negative charged ions to the anode. The moving ions provide viscous forces on the water in the soft soil. For clayey material there is a surplus of positive charged ions, causing an effective flux to the cathode. This theory is based on the Helmholtz-Smoluchowski model. This theory states the current is determined by positive electrical force and the opposite frictional force (Mitchell, Fundamentals of Soil Behaviour, 1993). The current is given by:

\[ q_A = k_e i_e A \]

In which \( k_e \), \( i_e \) and \( A \) are the electro osmotic permeability, electric potential gradient and surface area respectively.
The maximum water suction can be developed at the anode is defined by (Mitchell, In-place Treatment of Soil Behavior, 1970)

\[ u_a = \frac{k_e}{k_h} \gamma_w V \]

In which \( k_h \) is the horizontal permeability, \( \gamma_w \) is the unit weight of water and \( V \) is the total volume.

This model is independent on the size of the pores and therefore particularly suited for soils with small pores, like clay or peat. The movement of water to the cathode results in a reduction of pore water pressures which leads to higher effective pressures, higher shear strength and an accelerated consolidation process. Problems with electro osmosis may arise due to the fact of electrolysis. Redox reactions cause an acid zone to form at the anode and an alkaline zone to form at the cathode (Acar & Alshawabkey, 1993).

Anode: \( 2H_2O - 4e^- \rightarrow O_2 + 4H^+ \)
Cathode: \( 2H_2O + 2e^- \rightarrow H_2 + 2OH^- \)

To strengthen soft silty clay, it is beneficial to inject calcium chloride solution followed by a sodium silicate solution during the electro osmosis. Since \( Ca^{2+} \) in calcium chloride solution and \( SiO_3^{2-} \) in sodium silicate solution can freely migrate in the clayey soil under electric field, they improve cementation. The cementation effect is strongest near the anodes. Previous laboratory studies (Chien, 2003) have shown that the injection of a calcium chloride solution, followed by the injection of a sodium silicate solution, during electro osmosis is effective in strengthening soft silty clay (Ou, Chien, & Chang, 2009).

F.3.5 Admixtures

Admixture stabilization is a technique of mixing chemical additives with soil to improve the consistency, strength, deformation characteristics, and permeability of the soil. This improvement becomes possible by the ion exchange at the surface of clay minerals, bonding of soil particles and/or filling of void spaces by chemical reaction products. (Terashi & Juran, 2000). Most common chemical additives are lime and cement due to their high availability and low cost.

One of the ways to utilize admixtures is by deep mixing. In situ blades or augers manufacture a column of treated soil. This method is best suited for soils with moisture content of up to 60%. Mostly soft cohesive soils are treated with this technique as other soil types can be treated more
economically with other techniques. Benefits of this technique are the low vibrations and noise and reduction of project duration.
G. Appendix - Prefabricated vertical drains

Because the sub surface at Tiaozini consists of a thick layer of compressible soft material, the time it takes for the soil to consolidate, using just surcharge or a preloading will be years. Since most road constructions are on a tight schedule and additional surcharge load is not economically or technically efficient, additional methods will have to be implemented to speed up this process. Using drains in combination with a surcharge load can reduce the time from several years to a few months and therefore increase the economic attractiveness. With the following formula the time required for the layer to consolidate for 99% ($t_{99\%}$) of the expected final settlement can be approximated (Verruit, 1999):

$$t_{99\%} = \frac{2h^2}{c_v}$$

Installing vertical drainage material into the ground can shorten the drainage path ($h$) of soft clay deposits significantly, and, combined with surcharge improves the stiffness and strength of the ground substantially (Chai & Miura, 1999). With properly designed vertical drainage, pore water will flow laterally to the closest drain, rather than vertically to the underlying or overlying drainage layer, therefore the drainage distance is significantly reduced (Figure G.2).

With the use of vertical drains the surcharge load can be reduced to still achieve the same consolidation time. A lower surcharge load leads to a lower increase in shear stress and therefore a lower chance of instability issues. Moreover vertical drains use the higher horizontal permeability in naturally deposited layers, in relation to the vertical permeability. This allows a faster dissipation of pore water and hence reducing the consolidation time and magnitude of post-construction settlement (Taube, 2008).

In this project the chosen drain type is the widely used MebraDrain (Figure G.8). This drain type has proven its reliability with more than 400 million meters of installed drain in projects around the world. The filter of this drain type is developed to limit the passing of grains, which might cause clogging, while increasing the permeability for a higher drainage capacity.
Although it seems likely to install the drains from ground level up to the deep sand layers, this is often not allowed. By doing so a direct connection is created between the deeper groundwater and the surface. In case of a spill, pollutant materials can directly contaminate the deeper groundwater using the drainage system. This has to be avoided; hence the drains will be installed in the soft layers 0.5m above the sand layers, to a depth of - 2.61m.

**Drainage spacing**

Using the diagram shown in Figure G.4 the required drain spacing can be determined, based on site specific properties. In order to use this diagram the following input parameters and assumptions are used:

- The thickness of the compressible clay layer is 5.5m. However the water can drain two ways (top and bottom) and therefore the effective thickness is only 2.75m.
- The consolidation must achieve 95% (U=95%, \(t_{95}=3.15 \times 10^7\) s) within one year.
- During the construction of the pre- and surcharge load stability must be guaranteed. This means that within a period of four weeks the consolidation ratio must be 50% (U=50%, \(t_{50}=2.42 \times 10^6\)).
- The settlement criteria is 3.7 m² (\(c_v \cdot t_{95}\))
- The stability criteria is 0.3 m² (\(c_v \cdot t_{50}\))
- The ratio between horizontal and vertical permability (\(R = \frac{k_h}{k_v}\)) is set to be 2.0. (Rixner, Kraemer, & Smith, 1986).

Using part A of the diagram one can derive that, for a consolidation ratio of 95% and 50%, the necessary settlement and stability criteria are 9.5m² and 1.5m² respectively. These values are not reached and therefore show that vertical drainage is needed.

**Graphical determination**

Part A of the diagram shows that, with a settlement criterion of 3.7 m² and a thickness of 2.75m, the vertical consolidation ratio is 73%. Part B shows that, with the desired overall consolidation of 95%, the horizontal consolidation should be 82%. This is calculated using the following formula: \(U = 1 - (1 - U_v)(1 - U_h)\). Using part C, the determined \(c_h \cdot t_{95}\) and previously found \(U_h\) the input for part D is found. In order to determine the drain spacing, the equivalent diameter of the drains is needed, for the chosen MebraDrains this is \(d_s=0.052mm\) (Table G.2). This leads to a final spacing value of 3.0m.

The same procedure can be done for the stability criterion, also shown in Figure G.4. This results in a spacing of 1.8 m. In this situation the stability criterion is the governing criterion.
Since a triangular grid is used, the found spacing is not equal to the center-to-center distance required to complete the design. The method used to acquire the center-to-center distance is shown in Figure G.5.
This finally provides a center-to-center distance for the MebraDrain prefabricated vertical drain system of 1.7m.

G.1 Techniques

Over the years several techniques have been developed to achieve the optimal consolidation results, such as sand drains, sand compaction piles, gravel piles and prefabricated vertical drains (PVDs). Due to increasing cost of sand quarrying, strict environmental regulations and the fact that conventional sand drains are negatively affected by lateral soil movement, flexible PVD systems are currently the most common technique (Indraratna, 2008). Moreover PVDs can be installed quickly, without a lot of adjacent soil disturbance and are easy to fabricate, control and store. All these advantages make PVD systems a practical, low-cost, and effective technique.

Prefabricated vertical drains, also known as wick or band drains typically have dimensions of 100mmx4mm and are built up out of two separated parts, the core and filter. The flexible core is usually made of polyethylene and allows free flow of water along the drain. Different core shapes are shown in Table G.1. The filter is made of synthetic or natural fibrous material with a high resistance to clogging.

Table G.1 - Common core configurations (Tor & Sun, 2005)

<table>
<thead>
<tr>
<th>Description</th>
<th>Profile</th>
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<tr>
<td>Grooved</td>
<td>Corrugated</td>
</tr>
<tr>
<td></td>
<td>ribbed</td>
</tr>
<tr>
<td>Filament</td>
<td></td>
</tr>
<tr>
<td>Cusped</td>
<td></td>
</tr>
<tr>
<td>Studded</td>
<td>one side</td>
</tr>
<tr>
<td></td>
<td>two sides</td>
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</table>

The PVDs can be installed using dynamic and static procedures. With the dynamic method the drain is driven into the soil using a vibrating hammer. However the most common installation method is static. During the static installation the drain is pushed into the ground using a hollow, steel mandrel. The drain is fitted inside the mandrel, which protects it from damage as the mandrel is inserted into the ground to the final depth. At the base of the mandrel, the drain material is looped through an anchor which holds the drain in place as the mandrel is extracted. Once the mandrel has been
extracted from the ground, the drain is cut and the next drain is installed. Daily production rates can go up to 8000 m, depending on the site and soil properties (Taube, 2008). Normally the drains are put in a square, rectangular or triangular grid, spaced about 1.0 - 3.5m apart. At present PVD systems can be installed up to a depth of 65m.

**G.2  Factors affecting drainage**

In the past few decades prefabricated vertical drain improvement techniques have been widely applied in engineering projects, such as in berms, dikes, ramps and embankment constructions. However, field engineers often face a problem that the expected result of a PVD cannot be achieved in field. The main influencing factors are dealt with in this section to show that certain optimizations in the PV drains can be achieved.

For given soil conditions the effect of vertical drains depends on:

- Drain spacing
- Discharge capacity
- Equivalent drain diameter
- Smear effect
- Drainage boundary condition.

In quantifying the influence of these factors uncertainties exist, except for the drain spacing, this fully depends on the created optimal design. The drain spacing influence will be treated more thoroughly during the determination using a graphical method.

**G.2.1 Discharge Capacity of PVD**

The discharge capacity of a PVD is one of the main influencing factors on vertical drain behavior. The discharge capacity depends primarily on the following factors (Indraratna, 2008):

- The area of the drain
- The effect of lateral earth pressure
- Possible folding, bending and crimping of the drain
- Infiltration of fine particles into the drain filter

This capacity reduces significantly with time. The reduction is the result of the formation of a biofilm on the drainage channel side, together with the clogging of the drainage path because of fine particles (Chai & Miura, 1999).

**G.2.2 Equivalent Drain Diameter**

For the area or the equivalent diameter of a band-shaped drain (see Figure G.6), the following equation can be used \( d_w = 0.5w_d + 0.7t_d \) (Long & Covo, 1994). Where \( w_d \) and \( t_d \) represent the width and thickness of the drain, respectively. Clearly a bigger the equivalent diameter leads to a larger effect of the vertical drain.
Figure G.6 - Equivalent drain diameter (Indraratna, 2008)

G.2.3 Smear Effect
During the installation of the PVDs the soil surrounding the drains is completely disturbed. This zone is called the smear zone. The smear zone has a significant effect on the consolidation rate of the drainage system. The hydraulic conductivity in the smear zone is reduced significantly and consequently affects the effectiveness of the drain. The smear zone can be characterized using two parameters, namely, the diameter of the smear zone \((d_s)\) and the hydraulic conductivity ratio \((k_h/k_s)\). The smear zone is assumed to be circular with a diameter equivalent to 3 times the mandrel diameter \((d_s = 3d_m)\).

G.2.4 Drainage boundary condition.
Part of the water collected by the drains will flow to the ground surface and will then drain out through the outlet system, the sand embankment. Since the hydraulic conductivity of sand is significantly higher than that of clay, it is usually assumed that there is no hydraulic resistance in the embankment. If a thick layer (>0.5 m) of clean sand is used, this assumption is correct. However, in some cases, due to the unavailability of material as well as cost considerations, lower quality sand is used for the sand blanket. This in turn leads to a less successful result of the PVD system.

G.3 New developments in PV drains
Conventional prefabricated vertical drains consist of a core and a filter sleeve as two separable components, as shown in Figure G.8. There are two major shortcomings with this design:

1. The tensile strength of the core and the filter are not matched and sometimes differs a lot. As a result one component, either the core or the filter, will break first. Once one component breaks, the PVDs will no longer function properly.
2. As the filter is fitted loosely to the core, the filter will indent into the drainage channels of the core under earth pressure as shown in Figure G.7. As a result, the discharge capacity of the PVD will be reduced.
A new type of PVD is created by the Hohai University, in which the filter and the core are molten together forming an integrated body, as shown in Figure G.9. This new type of integrated PVD offers improved properties and performance compared to a conventional PVD system (Liu & Chu, 2008).

According to Liu & Chu (2008) the new design posses the following advantages:

1. The tensile strength of the integrated PVD will be higher than that of the separable type. This is because the core and filter will deform as one body and the combined tensile strength is higher than the weaker separate elements, due to interaction between the two components.
2. The discharge capacity of the integrated PVD will be higher. This is mainly because the indentation of filter into the drainage channels of the core is considerably reduced.
3. The integrated PVD is more resistant to clogging. This is due to fact that the drainage channels in the integrated type of PVD are not connected. Therefore, if one channel is blocked, it will not affect the rest.
4. Saving in the filter material. The width of the filter is exactly 100 mm in the integrated PVD, whereas in the separable PVD, the filter warps the core so it has to be at least 20–40 mm longer in circumference.
### G.4 Mebra-Drain properties

#### Table G.2 - Properties of Mebra-Drain® (Hayward Baker, 2012)

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<th>Mebra-Drain® MD-7407</th>
<th>Core properties</th>
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<td><strong>Width</strong></td>
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<td><strong>Thickness</strong></td>
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<td><strong>Mass</strong></td>
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<tr>
<td><strong>Tensile strength</strong></td>
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</table>

<table>
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<th>Filter properties</th>
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</tr>
<tr>
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</tr>
<tr>
<td><strong>Grab tensile strength</strong></td>
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<tr>
<td><strong>Elongation at break</strong></td>
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<tr>
<td><strong>Trapezoidal Tear</strong></td>
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<td><strong>Permittivity</strong></td>
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<td><strong>Apparent opening size</strong></td>
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<table>
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<th>Composite properties</th>
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<td><strong>Equivalent diameter</strong></td>
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<td><strong>Discharge capacity</strong></td>
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<tr>
<td><strong>Discharge capacity</strong></td>
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<td><strong>Roll length</strong></td>
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<td><strong>Roll weight</strong></td>
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#### Table G.3 - Input parameters drain spacing calculation

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<td></td>
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<td>t_50 = 2.42E+06 [s]</td>
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<td>k_v = 1.80E-09 [m/s]</td>
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<tr>
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<td>k_h = 3.60E-09 [m/s]</td>
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<td>c_v = 1.17E-07 [m²/s]</td>
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<td>c_h = 2.34E-07 [m²/s]</td>
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<td>E_s = 1170.0 [kN/m²]</td>
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<td>c_v*t_95 = 3.7 [m²]</td>
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## H. Appendix - Soil properties

Table H.1 - Representative values of soil properties (Nederlands Normalisatie-instituut, 2012)

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<th>Bijmengsel</th>
<th>Consistentie</th>
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<th>$C_m/(1 + e_0)'$</th>
<th>$E_{100}'$</th>
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<td>45</td>
<td>70</td>
<td>1300 2000</td>
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<td>15</td>
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<tr>
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</tr>
</tbody>
</table>
H.1  Stress - strain curves

Figure H.1 - Stress-strain curve

H.2  e-p curves

Figure H.3 - e-p curve
Multidisciplinary Project China 2012

Figure H.4 - e-log p curve including lognormal trendline

Figure H.5 - e-log p curve unloading/reloading

\[
m_w = m - m_s
\]

\[
w = \frac{m_w}{m_s} \times 100\%
\]

\[
\rho = \frac{m}{V}
\]

\[
\rho_d = \frac{\rho}{1 + w} = \frac{m_s}{V}
\]

\[
\rho_s = \frac{m_s}{V_s} = \frac{\rho_d V}{V - V_w} = \frac{\rho_d V}{V - w \rho_d V} = \frac{\rho_d}{1 - w \rho_d}
\]

\[
e_0 = \frac{\rho_s (1 + w)}{\rho} - 1 = \frac{\rho_s}{\rho_d} - 1 = \frac{n_0}{1 - n_0}
\]

\[
Y_{sat} = \frac{m g}{V} = \rho g
\]

\[
Y_d = \frac{m_s g}{V} = \rho_d g
\]

\[
C_p \approx \frac{(1 + \varepsilon_0) \ln 10}{c_{sw}} \text{ with } \sigma' < \sigma_p
\]

\[
C'_p \approx \frac{(1 + \varepsilon_0) \ln 10}{c_c} \text{ with } \sigma' > \sigma_p
\]
Table H.2 - Hohai laboratory tests

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Table H.3 - Aluminium case test

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Table H.4 - Cutting ring test

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<th>Cutting ring + soil [gram]</th>
<th>The weight of the wet soil [gram]</th>
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<td>111,76</td>
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<td>42,05</td>
<td>151,33</td>
<td>109,28</td>
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<td>486</td>
<td>43,01</td>
<td>153,30</td>
<td>110,29</td>
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<td>423</td>
<td>42,99</td>
<td>151,74</td>
<td>108,75</td>
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<td>446</td>
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<td>Unreadable</td>
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I. Appendix – Soil Improvement MCA

I.1 Multi criteria analysis – complete analysis

The different criteria are not all equally important. The relative importance of each criterion is displayed in Table I.1. When one criterion is more important than another, it will get mark 1 in the top right side. Equally the other criterion will get a mark 0 in the bottom left side of the matrix. When both criteria are equally important they will both get 1. The final weight factor is determined by summing all marks and dividing this number by 10.

Table I.1 - Criteria weight factor determination

<table>
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<th>Criterion</th>
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<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>Weight factor</th>
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<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.5</td>
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<tr>
<td>2</td>
<td>Costs</td>
<td>1</td>
<td>-</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
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</tr>
<tr>
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<td>-</td>
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<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.3</td>
</tr>
<tr>
<td>4</td>
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<td>0</td>
<td>1</td>
<td>-</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
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<tr>
<td>5</td>
<td>Applicability on large scale</td>
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<td>0</td>
<td>1</td>
<td>1</td>
<td>-</td>
<td>1</td>
<td>1</td>
<td>1</td>
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<tr>
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<td>0</td>
<td>1</td>
<td>0</td>
<td>-</td>
<td>1</td>
<td>1</td>
<td>0.4</td>
</tr>
<tr>
<td>7</td>
<td>Durability</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>-</td>
<td>0</td>
<td>0.1</td>
</tr>
<tr>
<td>8</td>
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<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>-</td>
<td>0.3</td>
</tr>
</tbody>
</table>

1. Speed
An important factor when looking at different methods of consolidation is the time in which a certain settlement is reached. This has everything to do with the speeds of the consolidation process and thus with the method that imposes settlement. The time to set up the equipment is also concerned.

2. Cost
Often money is the main incentive to make a decision. Certain methods might be fast, but require a lot of material or special machinery and are therefore very expensive. That is why costs of a technique must always be taken into account when objectively comparing methods.

3. Technologic complexity
This criterion not only takes into account the type of machinery and the amount required. It also looks at the availability of different machines and the complexity of the method. Some methods might be very appealing, but if the machinery is not easily available, this can cause mayor delays in the realization of the technique.

4. Reliability
This criterion takes into account whether or not the method has proven itself overtime. Certain methods might only be used in small pilot project, while others are widely used. This determines the amount of experience with a technique and tells something about the reliability of the method.

5. Applicability on large scale
Some methods might be very successful but are not suitable for a large scale as the Tiaozini reclamation project.

6. Effectiveness
Two methods might impose the same settlements over time. However not all techniques provide the same parameter improvement. This takes the effectiveness of the method into account.
7. Durability
This criterion states something about the long term benefits of a method. Some methods might have produced such an effect that further treatment is not necessary in the future, whereas others might only have short term benefits.

8. Sustainability
All methods might have a different impact on the natural environment. This criterion takes into account the different impacts. Not all criteria are of the same level of concern.

The final result of the MCA is given in Table I.3. For some of the distinctive scores an explanation is listed below. The total score is found by multiplying the separate scores given to the different criteria, with the corresponding weight factor.

I.1.1 Surcharge induced preloading
For this simple method the only thing required to accelerate consolidation is overburden pressure. This pressure can simply be generated by applying a large quantity of soil in top of the soft layers. The only machinery required are dump trucks and bulldozers, furthermore, the applied loading consists of natural material. Hence, the technological complexity and sustainability score the maximum value for this method. Costs are mostly related to the amount of soil needed and it transport; however the price for bulk sand material is relatively low in comparison with the other methods. The score for reliability is high because the method has been applied numerous times in different countries and circumstances and has proven itself to be reliable. The speed of this method is relatively slow and when applying it on a large scale the costs might become significantly high, therefore these criteria score lower.

I.1.2 Prefabricated vertical drains
With the use of PVD the consolidation time can be reduced significantly, this explains the maximum score for speed. However additional machines and material is required to successfully install the drains, thus there technological complexity and sustainability score a bit lower than the previous method. Since less soil is needed for this method the applicability on large scale increases.

I.1.3 Vacuum preloading
Because the complete site is covered with an airtight membrane, a system of vertical drains is installed and a drainage layer is applied on top, the costs, technological complexity and sustainability go down in comparison with the two previous methods. Additionally the method is well known and mostly applied on smaller scales, therefore the reliability is lower. Nonetheless, in terms of the speed the method scores maximum.

I.1.4 Electro osmosis
Due to the possible cementation of soft silty clay particles, the efficiency of the method scores higher than the previous methods. This cementation will greatly enhance the bearing capacity of the soil. However this cementation effect is strongest near the anodes and will not occur in the center of the soil mass, this explains why the score is not maximal. Since numerous anodes and cathodes are needed to consolidate a large area of soft soil, the technological complexity and applicability on large scale score low. Additionally the process requires much energy which influences the costs and sustainability. All in all this method is not suitable for the Tiaozini project.
I.1.5  Admixtures
Most admixtures are expensive and although they greatly enhance the soil properties the high costs and low applicability on large scale withhold this method from being suitable. If natural resins are applied to improve the soil properties this method can be sustainable, however in all other cases the usage and creation of artificial additives is an unsustainable solution.

Table I.2 - Weight factor

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<th>Weight factor</th>
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</tr>
<tr>
<td>2</td>
<td>Costs</td>
<td>0.7</td>
</tr>
<tr>
<td>3</td>
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</tr>
<tr>
<td>4</td>
<td>Reliability</td>
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<tr>
<td>5</td>
<td>Applicability on large scale</td>
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</tr>
<tr>
<td>6</td>
<td>Effectiveness</td>
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<tr>
<td>7</td>
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<tr>
<td>8</td>
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Table I.3 - Multi criteria analysis consolidation methods Tiaozini

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<th>4</th>
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<td>2</td>
<td>0</td>
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[Excellent 2, Good 1, Marginal 0, Poor -1, Extremely Poor -2]
J. Appendix – D-Settlement

D-settlement is a computer software program designed for calculating settlements of especially embankments in a semi-three-dimensional space. D-settlement takes account of spread of the load in the subsurface of existing as well as surcharge loads. The calculations show the settlements over time and the end settlements (settlements after 30 years). D-settlement is known to have a accuracy of at least 70% (Adviesgroep Geotechniek, 2007).

J.1 Darcy versus Terzaghi

D-Settlement is capable of working with two different consolidation models, the Darcy model and the Terzaghi model. Darcy’s model is more accurate in describing the influence of excess pore pressures on settlement. If this influence is limited, for instance if vertical drains are applied, Terzaghi’s model can be implemented. The two models differ in the fact that Darcy uses a step-wise numerical solution of effective stress and pore pressures at different points in time and space. The Terzaghi model on the other hand uses a time-dependent ‘degree of consolidation’ to adjust the drained settlement solution. The Darcy model consumes considerable less computation time in the newest version of D-Settlement, supports the same input as the Terzaghi model, features improved submerging modeling and a significant increase in robustness (Visschedijk & Trompille, D-Settlement Version 9.3, 2012).

J.2 NEN-Koppejan

The model that is used most in the Netherlands for modeling settlement is the NEN-Koppejan rule. It connects settlements and strresses from empirical relationships. The use however is limited as it has not been designed to model unloading/reloading behavior. Further Koppejan assumes a stress dependent slope of the creep tail after virgin loading (Visschedijk & Trompille, D-Settlement Version 9.3, 2012). Koppejan represents the traditional Dutch model, applicable for staged loading. For D-settlement with NEN-Koppejan the following parameters are used; \( C_p, C_s, C_p', C_s', A_p, A_s \).

The NEN-Koppejan model might be a logical choice if the model matches available historical parameters and user experience. Koppejan parameters are traditionally determined on a linear strain basis. The optional combination with natural strain theoretically requires that the parameters were also determined on the same basis (Visschedijk & Trompille, D-Settlement Version 9.3, 2012).

NEN-Koppejan;

\[
\Delta \varepsilon = -\left( \frac{1}{C_p} + \frac{1}{C_s} \log \left( \frac{t}{t_0} \right) \right) \ln \left( \frac{\sigma}{\sigma_0} \right)
\]

When using NEN-Koppejan in D-Settlement a choice has to be made if the model may use natural strains \( \varepsilon^h = -\ln(1 - \varepsilon^c) \) or has to use linear strains \( \varepsilon^c = \frac{\Delta h}{h_0} \). The linear strains (linear strain theory) are based on the initial layer thickness and calculated by \( \dot{\varepsilon}_{t+\Delta t} \approx \varepsilon_t + \Delta t \dot{\varepsilon}_t \). For large strains \( \varepsilon > 35\% \) the approximation by linear strains becomes too inaccurate. Natural strains should be implemented, since strain should not be defined as a change of the initial layer thickness, but as a change of the present layer thickness. The linear strains based on the present layer can be calculated by \( \varepsilon_{t+\Delta t} \approx \varepsilon_t + \Delta t \dot{\varepsilon}_t \cdot (1 - \varepsilon_t) \) (van Baars, 2003).
J.3 NEN-Bjerrum

The Dutch geotechnical design codes currently prescribe the use of the NEN-Bjerrum method or Isotache method (like other western countries do as well). This method directly relates overconsolidation with creep rate and equivalent age. The method implies increase of creep rate by loading and reduction of creep rate by unloading and time. The model and its parameters are based on a small strain assumption and therefore uses a linear strain approach. It assumes the slope is stress independent after virgin loading. Because of its underlying Isotache formulation this method is suited for staged loading, unloading and reloading (Visschedijk & Trompille, D-Settlement Version 9.3, 2012).

For the use of NEN-Bjerrum model, a choice between compression ratio and compression index has to be made. This choice will not influence the outcome of the calculation because the parameters can be back calculated from each other.

- Compression ratio ($RR, CR, C_a$)
- Compression index ($C_a, C_c, C_r, e_0$)

NEN-Bjerrum (with the use of compression index parameters);

$$
\varepsilon_p = \frac{C_c}{1+e} \log \left( \frac{\sigma}{\sigma_0} \right)
$$

$$
\varepsilon_s = \frac{C_a}{1+e} \log t
$$

According to the norm the void ratio should be estimated based on the difference between the wet and dry bulk density. Since the void ratio is not a constant but dependent on the strain, the settlement function is not explicit. NEN-Bjerrum uses an initial equivalent age for calculation which is divined as $t_{age} = \tau_0 OCR C_a$.

J.4 Isotache

The Isotache model is quite similar to the NEN-Bjerrum model. For instance, it also relates overconsolidation, creep rate and equivalent age. Differences lie in the fact that the Isotache model assumes natural strains and therefore is applicable for large strains. It makes parameters stress-objective and prevents prediction of unphysical large deformations. The model is more or less an extension to the NEN-Bjerrum model, but uses an initial equivalent age for calculation which is divined as $t_{age} = \tau_0 OCR C_a$.

Parameters $a$ and $b$ can be derived from an oedometer test. These parameters represent the angle of the natural strain with respect to the natural logarithmic stresses. The parameter $c$ on the other hand is more difficult to determine. The simple oedometer tests will not be able to provide this parameter; at least a constant rate of strain test (CRS) should be performed.

J.5 Common parameters

Independent on the choice of calculation model certain common parameters should be defined in D-settlement. $\gamma$ and $\gamma_f$ have to be provided or each layer. For compression a choice has to be made for the use of either;
- Pre-consolidation pressure ($\sigma_p$) is the maximum vertical overburden stress that a particular soil sample has sustained in the past (Solanki & Desai, 2008).
- Over consolidation ratio (OCR = $\sigma_p/\sigma'_0$) is defined as the highest stress experienced divided by the current stress.
- Pre-overburden pressure (POP = $\sigma_p - \sigma'_0$) is defined as the absolute difference between the highest stress experienced and the current stress.

Pre-consolidation pressure cannot be measured directly. It can be estimated using a number of different strategies. (Department of Defense, 2005). The pre-consolidation pressure can be derived from Casagrande’s method.

![Figure J.1 - e-log \(\sigma'\) curve](image)

The OCR method corresponds to a coefficient $K_0$ constant with depth, while POP method gives an infinite coefficient in the subsurface and that extend toward $K_{0,NC}$ coefficient of a normally consolidated soil, in depth, soil becomes normally consolidated. (Husein, Flavigny, & Boulon, 2002).

![Figure J.2 - OCR and POP](image)

For storage a choice has to be made between the use of;

- Vertical consolidation coefficient $(c_v = \frac{k_v}{Y_p m_v} = \frac{k_{p,Eed}}{Y_p} = \frac{k_h}{Y_p m_v} = \frac{k_{p,Eed}}{Y_p} = \frac{k_{h,Eed}}{Y_p})$ uses vertical and horizontal consolidation coefficients to model settlements.
- Constant permeability ($k_x, k_y$), takes a constant permeability throughout the modeling.
• Strain dependent permeability (Permeability strain modulus, \(k_{y,0}, k_x, k_y\)), assumes strain dependency of permeability.

**J.6 Model parameters**

Primary parameters are true soil parameters and therefore they are convertible from model to model. Secondary parameters on the other hand are still controversial and the strain dependency of the creep differs greatly per model. The creep parameter(s) are therefore different for each model and these parameters cannot be converted from model to model.

**J.6.1 NEN-Koppejan parameters**

Koppejan suggested that the formula of Keverling Buisman was combined with the logarithmic compression formula of Terzaghi because this took account of decreasing compressibility for increasing loads.

The NEN-Koppejan parameters can be derived by using the diagram displayed in Figure J.3 in which settlements are plotted against \(\log p\) for \(t = t_0\) and for \(t = 10t_0\). This results in two lines for \(t_0\) and \(10t_0\). If a strong bend in the line is noticeable, the point of that bend resembles the pre-consolidation pressure (see Figure J.4).

![Figure J.3 - Method of Koppejan](image)

![Figure J.4 - NEN-Koppejan model parameters](image)

Figure J.4 shows how the NEN-Koppejan parameters can be determined. The gradients of the trend lines describe the stiffness of the soil. For settlements after 1 day, all deformations are assigned to primary settlement whereas settlements between 1 and 10 days are caused by the secular effect. The calculated settlements after 10 days thus include the primary as well as the secular settlements. The NEN-Koppejan parameters are described as follows;
- Primary compression coefficient below pre-consolidation: \[ C_p \approx \frac{(1+\varepsilon_0)\ln 10}{C_{cv}} \] with \( \sigma' < \sigma_p \)
- Primary swelling coefficient: \[ A_p = \frac{(1+\varepsilon_0)\ln 10}{C_{cv}} \] with \( \sigma' < \sigma_p \)
- Primary compression coefficient above pre-consolidation: \[ C'_p \approx \frac{(1+\varepsilon_0)\ln 10}{C_{cv}} \] with \( \sigma' > \sigma_p \)
- Secondary compression coefficient below pre-consolidation: \[ C_s = \ln \left( \frac{\sigma'}{\sigma'_p} \right) \frac{d \log t}{d \varepsilon} \] with \( \sigma' < \sigma_p \)
- Secondary swelling coefficient: \[ A_s = \ln \left( \frac{\sigma'}{\sigma'_p} \right) \frac{d \log t}{d \varepsilon} \] with \( \sigma' < \sigma_p \)
- Secondary compression coefficient above pre-consolidation: \[ C'_s = \ln \left( \frac{\sigma'}{\sigma'_p} \right) \frac{d \log t}{d \varepsilon} \] with \( \sigma' > \sigma_p \)

### J.6.2 NEN-Bjerrum parameters

The Anglo-Saxon NEN-Bjerrum model parameters can be determined by plotting \( \Delta e \) against \( \log \sigma'_{vp} \) for the compression index parameters or by plotting \( \varepsilon \) against \( \log \sigma'_{vp} \) for the compression ratio parameters.

![NEN-Bjerrum model parameters](image)

**Figure J.5 - NEN-Bjerrum model parameters**

From Figure J.5 the NEN-Bjerrum model parameters can be determined. Because the void ratio doesn’t directly follow out of the oedometer test and because we are more interested in crest settlements we mostly use the \( \varepsilon \) against \( \log \sigma'_{vp} \) diagram. Due to the different scale the parameters that follow are now the compression ratio parameters. The compression index parameters and the compression ratio parameters can be back-calculated from each other.

![NEN-Bjerrum creep model parameter](image)

**Figure J.6 - NEN-Bjerrum creep model parameter**

The creep parameter \( C_\alpha \) is determined separate for a load that is bigger than the pre-consolidation pressure. The settlement is plotted against \( \log t \). When consolidation ends the graph displays a linear secular settlement. This secular effect is solely caused by creep. The gradient after 1 day is used to...
determine the creep parameter because after 1 day consolidation has always finished in an oedometer test. The model parameters that can be determined are displayed below;

- Coefficient of secondary compression
- Primary compression index
- Reloading/Swelling index
- Primary swelling index
- Compression ratio
- Reloading/Swelling ratio

\[ C_\alpha = \frac{d\varepsilon}{d \log t} \]

\[ C_c = (1 + \epsilon_0) \frac{d\varepsilon}{d \log \sigma} \text{ with } \sigma' > \sigma_p \]

\[ C_r = C_{sw} \frac{d\varepsilon}{d \log \sigma} \text{ with } \sigma' < \sigma_p \]

\[ C_{sw} = (1 + \epsilon_0) \frac{d\varepsilon}{d \log \sigma} \]

\[ CR = \frac{C_{cl} c}{1 + \epsilon_0} = \frac{\Delta e_p}{\log \left( \frac{\sigma_{tp} + \Delta \sigma_p}{\sigma_{tp}} \right)} \]

\[ RR = \frac{C_{rl} c}{1 + \epsilon_0} = \frac{\Delta e_p}{\log \left( \frac{\sigma_{tp} + \Delta \sigma_p}{\sigma_{tp}} \right)} \]

### J.6.3 Isotache parameters

The way the parameters for the Isotache model are determined is similar to those of NEN-Bjerrum method due to the fact that NEN-Bjerrum forms the basis of the Isotache model. Differences in parameters arise due the fact the Isotache model uses stress to determine parameter \( a \) and \( b \) as well as time for parameter \( c \) on logarithmic scale. For big deformations differences in parameters occur due the fact Isotache uses natural strains instead of linear strains. The way to determine the Isotache parameters is displayed in Figure J.7.

![Isotache model parameters](image)

**Figure J.7 - Isotache model parameters**

There are several methods for determining \( a \). It can be determined from the trend line from the first part of the oedometer test or from the reloading part of the oedometer test. The parameters are defined as follows;

- Modified natural swelling index
- Modified natural compression index
- Modified natural secondary compression constant

\[ a = \frac{C_{sw} c}{(1 + \epsilon_0) \ln 10} \]

\[ b = \frac{C_{cl} c}{(1 + \epsilon_0) \ln 10} \]

\[ c = \frac{C_{cl} c}{C_{sw} c} \ln 10 \]

Due to the fact the Isotache model is a derivation of the NEN-Bjerrum model, the model parameters can be back-calculated from each other. For this back-calculation the incremental approach between natural and linear strains is used so that;

- Modified natural swelling index

\[ a \approx \frac{RR}{(1 - \epsilon_0) \ln 10} \]
• Modified natural compression index $b \approx \frac{CR}{(1 - e_0) \ln 10}$
• Modified natural secondary compression constant $c \approx \frac{a_{\alpha}}{(1 - e_0) \ln 10}$

An overview of all model parameters used is given in Table J.1. By which tests the parameter are determined, where they are used, how they are used and the strain dependency is described in Table J.2.

Table J.1 - Model Parameters D-Settlement

<table>
<thead>
<tr>
<th>Model</th>
<th>Primary</th>
<th>Secondary (Creep)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma &lt; \sigma_c$</td>
<td>$\sigma &gt; \sigma_c$</td>
</tr>
<tr>
<td>NEN-Koppejan</td>
<td>$C_p$ or $A_p$</td>
<td>$C'_p$</td>
</tr>
<tr>
<td>NEN-Bjerrum</td>
<td>$C_r$</td>
<td>$C_c$</td>
</tr>
<tr>
<td>Isotache</td>
<td>$a$</td>
<td>$b$</td>
</tr>
</tbody>
</table>

Table J.2 - Overview model parameters

<table>
<thead>
<tr>
<th>Model</th>
<th>Parameters</th>
<th>Tests</th>
<th>Determination</th>
<th>Internationality</th>
<th>Linear/Natural</th>
</tr>
</thead>
<tbody>
<tr>
<td>NEN-Koppejan</td>
<td>$C_p$, $C_s$, $C'_p$, $C'_s$, $A_p$, $A_s$</td>
<td>Normal oedometer test</td>
<td>Directly out of oedometer test</td>
<td>Parameters used in The Netherlands</td>
<td>Both linear and natural strains</td>
</tr>
<tr>
<td>NEN-Bjerrum</td>
<td>$C_{\alpha}$, $C_c$, $C_r$, $e_0$ or $RR$, $CR$, $C_{\alpha}$</td>
<td>Normal oedometer test</td>
<td>Not directly, void ratio should be determined separately</td>
<td>Parameters used internationally</td>
<td>Linear strain</td>
</tr>
<tr>
<td>Isotache</td>
<td>$a$, $b$, $c$</td>
<td>Oedometer &amp; constant rate of strain test (CRS)</td>
<td>$a$, $b$ directly from oedometer test, $c$ from CRS test</td>
<td>Parameters used internationally in literature</td>
<td>Natural strain</td>
</tr>
</tbody>
</table>

J.7 Limitations

Some limitations when working with D-Settlement are (Visschedijk & Trompille, D-Settlement Version 9.3, 2012):

• When performing vertical displacement calculations, the program assumes no horizontal displacements. Influences from horizontal displacements are neglected in these calculations.
• The horizontal flow is modeled by a leakage term. The models do not explicitly describe horizontal flow to the drains.
• When using Darcy, the gradually changing submerged weight of only the non-uniform load is reduced, not the weight of uniform and soil loads.

When comparing the two models, the Terzaghi model has some more limitation compared to the Darcy model. When using the Terzaghi model;

• The submerged weight is determined on the basis of final settlements. Only the weight of non-uniform load is reduced, not the weight of uniform and soil loads.
• The model does not calculate the actual effective pressures, but approximates them based on adjustment of settlements from a drained solution.
• The settlements after completed consolidation will always be equal to the settlement from a drained solution.
• The updated pre-consolidation stress will be overestimated during reloading if unloading took place before the consolidation was finished.
• The influence of vertical drains and dewatering is averaged along a full layer. This is especially important for the layer in which the vertical drain ends.
• Less accurate results are obtained for combination of layers with different consolidation coefficients and/or vertical drains than with the Darcy model.
• Submerging is described by an initial load reduction while the Darcy model in combination with the NEN-Bjerum or Isotache model takes the gradual decrease into account.
• Terzaghi assumes equal consolidation time during un/reloading while Darcy shows faster consolidation during un/reloading.
K. Appendix – Analytical Koppejan calculation

K.1 Analytical Koppejan input parameters

Table K.1 - Koppejan analytical input parameters

<table>
<thead>
<tr>
<th>Soft layer properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$h$   = 5.3 [m]</td>
<td></td>
</tr>
<tr>
<td>$C_p$ = 16.5 [-]</td>
<td></td>
</tr>
<tr>
<td>$C'_p$ = 8.88 [-]</td>
<td></td>
</tr>
<tr>
<td>$C_s$ = 110.0 [-]</td>
<td></td>
</tr>
<tr>
<td>$C'_s$ = 110.0 [-]</td>
<td></td>
</tr>
<tr>
<td>$t$   = 10000.0 [days]</td>
<td></td>
</tr>
<tr>
<td>$t_0$ = 1.0 [days]</td>
<td></td>
</tr>
<tr>
<td>$\sigma$ = 156.3 [kN/m2]</td>
<td></td>
</tr>
<tr>
<td>$\sigma_1$ = 18.0 [kN/m2]</td>
<td></td>
</tr>
<tr>
<td>$y_w$ = 18.0 [kg/m3]</td>
<td></td>
</tr>
</tbody>
</table>

Table K.2 - Road fill properties

<table>
<thead>
<tr>
<th>Road fill properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer</td>
<td>Density [kg/m3]</td>
</tr>
<tr>
<td>Clay_1</td>
<td>1800</td>
</tr>
<tr>
<td>Clay_2</td>
<td>1800</td>
</tr>
<tr>
<td>Clay_3</td>
<td>1800</td>
</tr>
<tr>
<td>Clay_4</td>
<td>1800</td>
</tr>
<tr>
<td>Clay_5</td>
<td>1800</td>
</tr>
<tr>
<td>Surface layer</td>
<td>2450</td>
</tr>
<tr>
<td>Intermediate layer</td>
<td>2200</td>
</tr>
<tr>
<td>Bottom layers</td>
<td>2200</td>
</tr>
<tr>
<td>Base</td>
<td>1800</td>
</tr>
<tr>
<td>Sandbed</td>
<td>1850</td>
</tr>
<tr>
<td>Traffic</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>7026.0</td>
</tr>
</tbody>
</table>
**K.2 Analytical Koppejan results**

Figure K.1 - Consolidation graph using the analytical calculation based on Koppejan

Table K.3 - Koppejan analytical results

<table>
<thead>
<tr>
<th>Analytical results</th>
<th>Stage 1</th>
<th>Stage 2</th>
<th>Stage 3</th>
<th>Stage 4</th>
<th>Stage 5</th>
<th>Stage 6</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>+ 1.1m</td>
<td>+ 1.1m</td>
<td>+ 1.1m</td>
<td>+ 1.1m</td>
<td>+ 1.1m</td>
<td>Embankment</td>
</tr>
<tr>
<td>h</td>
<td>1.10</td>
<td>2.20</td>
<td>3.30</td>
<td>4.40</td>
<td>5.50</td>
<td>5.50</td>
</tr>
<tr>
<td>Δt</td>
<td>180</td>
<td>180</td>
<td>180</td>
<td>180</td>
<td>180</td>
<td>9100</td>
</tr>
<tr>
<td>t</td>
<td>180</td>
<td>360</td>
<td>540</td>
<td>720</td>
<td>900</td>
<td>10000</td>
</tr>
<tr>
<td>σ</td>
<td>19.4</td>
<td>38.8</td>
<td>58.3</td>
<td>77.7</td>
<td>97.1</td>
<td>156.3</td>
</tr>
<tr>
<td>σ_1</td>
<td>18.0</td>
<td>19.4</td>
<td>38.8</td>
<td>58.3</td>
<td>77.7</td>
<td>97.1</td>
</tr>
<tr>
<td>ε</td>
<td>0.01</td>
<td>0.09</td>
<td>0.05</td>
<td>0.04</td>
<td>0.03</td>
<td>0.07</td>
</tr>
<tr>
<td>Δh</td>
<td>0.01</td>
<td>0.20</td>
<td>0.18</td>
<td>0.17</td>
<td>0.16</td>
<td>0.39</td>
</tr>
<tr>
<td>h</td>
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<td>0.21</td>
<td>0.39</td>
<td>0.56</td>
<td>0.72</td>
<td>1.11</td>
</tr>
</tbody>
</table>
L. Appendix – Dike fail mechanisms

There are twelve general fail mechanisms for a dike.

- Overflow (A) is the most obvious failure, resulting from a water level higher than the height of the dike. This leads to damage caused to the hinterland by water. It could also lead to erosion of the inner slope when sufficient protection is absent. This erosion can lead to total failure of the dike.
- Wave overtopping (B) is also related to the water level, although the volume of water will be less in comparison to overflow.
- Sliding of the inner slope (C) occurs if the load is larger than the resistance in the sliding circle. It does not automatically have to lead to overall instability. The residual dike might be possible to retain the water.
- Shearing (D) occurs if the dike has a low own weight in comparison to the water pressure.
- Sliding of the outer slope (E) is similar to sliding of the inner slope.
- Micro instability (F) concerns the buildup of water pressure inside the dike. This could lead to erosion of the inner slope.
- Piping (G) is related to micro instability and can lead to erosion if flow velocity is high enough. In order for piping to occur a cohesive layer has to be present. Underneath this cohesive layer the seepage water can form a meandering cavity.
- Erosion of the outer slope (H) is causes by a wave load.
- Erosion of the first bank (I) can lead to instability of the outer slope. This process can start if there is a steep underwater slope in front of the dike.
- Settlement of the entire dike (J) will lead to a decrease in the height of the dike. This can lead to overflow or wave overtopping. It however is a slow process that can be monitored easily.
- Drifting ice (K) results in an extra load on the dike. For areas in the world where the appearance of ice occurs often this has to be taken into the design.
- A collision with a vessel (L) is a theoretical fail mechanism. It will result in an very high load and could possibly result is overall dike collapse. However due to the extremely small chance this is often not taken as a design criteria, since this would lead to unrealistic dimensions for the dike.
M. Appendix – Geo-tubes

Geo-tubes are large cylindrical shaped bags of geotextile (Figure M.1) filled with sand or other soil material, with a capacity of several hundred cubic meters. A significant part of the hydraulic fill has to consist of sand in order to prevent consolidation of the fill after construction.

The geotextile tube is filled in-situ with water-soil slurry in lengths varying from 25 to 100 meters. The diameter varies from 0.5 to 4 meter per tube. The geo-tube can be constructed under water. In order to do so the geo-tube has to be filled with water to inflate the tube. Due to the permeable property of the geotextile however, some sand has to be added to make it possible to inflate the geo-tube. When the right shape is reached, slurry of water and granular material is inserted into the geo-tube. The soil will precipitate inside the tube and will form a compacted stable mass. The filling water will drain through the permeable geotextile and through the drainage/filling ports.

![Figure M.1 - Filling of geo-tube](image1)

![Figure M.2 - Cross section geo-tube](image2)

The shape of the geo-tube is determined by the composition of the hydraulic fill. The tubes used in the past (Longard tubes) were impermeable and were inflated to perfect round form before they were filled with soil. Due to the permeable character of the geotextile a perfect circle is not possible. By obtaining a large fill percentage circular shape can be reached. A larger fill percentage however leads to larger stresses in the geotextile.

M.1 Use of geo-tubes

Geo-tubes are used in different aspects in hydraulic engineering, either as main construction or as segment of a construction. The geotextile membrane provides properties which enable applications which would not be impossible without the geo-tube. Due to the geotextile tube the soil body can adopt shapes that in natural circumstances are not possible. For this reason a geo-tube is often used for the core of a dike. Several tubes can be placed on top of each other. For this reason the sand can be placed at a steeper angle than the angle of internal friction (+30°). By using a core of geo-tubes, the width of the total dike can be reduced. This leads to a decrease of the amount of soil needed and area required.

The permeable geotextile will act as protective layer for the soil. Although water can flow through, the sand will not erode. The flow of sand through the textile is negligible if the right pore diameter of the textile is used. (CUR, 2006). Therefore the construction of a hydraulic structure in different phases with significant intervals is possible. The site specific circumstances however should be taken into account. For example during the closing of a tidal basin large increases in water velocity, can cause displacements of the complete geo-tube (Steeg & Vastenburg, 2010). During the construction
of a conventional sand dike fine material can dissolve in water and pollute the nearby environment. With a geo-tube construction, the soil is contained inside the geotextile causing less fine material to be released to the environment in comparison to a conventional sand dike.

M.2 Possible fail mechanism geo-tube

Additional to the conventional fail mechanisms of a dike (appendix L), the use of geo-tubes to construct a dike leads to a number of possible fail mechanisms. When looking at geo-tubes, nine different fail mechanisms can be distinguished (Lawson, 2008).

![Figure M.3 - Limit state modes for geotextile tubes: external](image1)

![Figure M.4 - Limit state modes for geotextile tubes: internal](image2)

These 9 failure modes can be divided in external mechanisms (Figure M.3), which affect the overall performance of the geo-tube structure, and internal mechanisms (Figure M.4), which affect the internal performance of the soil.

The external fail mechanisms consider a geo-tube as one solid unit. Due to the large base-contact width with respect to the height, the geo-tubes can be considered very stable. Sliding instability (i) and overturning instability (ii) have to be taken into account if the diameter of the geo-tube is small in comparison with the present waves. Global instability (iv) has to be checked when several geo-tubes are stacked on top of each other to construct a higher dike. Scour of the subsurface (v) can occur during construction phase and operational phase. During construction a large amount of filling water will be drained through the permeable geotextile, this can lead to erosion or undermining of the subsurface. When several geo-tubes are stacked, the excess filling water of the top geo-tubes can lead to scour of subsurface underneath the bottom geo-tube. During the operational phase scour can occur due to wave load.

The internal fail mechanisms consider the possible failure of the fill inside a geo-tube. The tearing of the geotextile (i) can be caused by external factors or continues strain of the geotextile. In order to determine the loss of fill through the geotextile (ii) the hydraulic environment has to be investigated. The severity of the hydraulic regime governs the properties of the required geotextile properties and
whether additional protection is required. The deformation of fill material (iii) can have several causes:

- **Incomplete filling of geo-tube**
  Although a geo-tube is never filled for 100%, a low filling percentage can cause deformation of the fill. When the filling percentage is too low, abundant space for deformation of the fill is available.

- **Liquefaction of fill**
  When a sudden rise of pore pressure occurs inside the geo-tube, liquefaction can take place. A deformation as a result of incomplete filling from dense to loose sand can also cause liquefaction.

- **Consolidation of fill after construction**
  For hydraulic applications a fill with a significant part of sand is advised. If clay is used as a fill or the sand has not achieved a dense packing during construction, consolidation can take place after construction. Especially if multiple geo-tubes are stacked this leads to instability.

- **Continuing deformation of geotextile skin over time (creep)**
  The geotextile should have enough strength and stiffness to maintain its shape. Relaxation of the geotextile could lead to loss of shape and height. An increase of volume could lead to liquefaction of the fill.

Extra attention has to be paid to the seams, as these are weak points of the geotextile. Geotextile has a maximum lifespan of around 50 years. After this period the advantages of the used geo-tube are no longer guaranteed and the hydraulic structure might become unstable. The chance of the internal failure mechanisms is directly related to the quality of the geotextile. Since the geotextile is used to construct the core of the dike, it is very difficult and costly to replace the geotextile.

It should be noted that Ten Cate™ has developed geotextile with a minimum lifespan of 200 years. Although it is difficult to determine the lifespan, this period is accepted by the Dutch government and is used in the Deltawerken (Zengerink, 2012).
### N. Appendix – Wave height

**Table N.1 - Measured wave height data with a 50 year return period**

<table>
<thead>
<tr>
<th>#</th>
<th>Water depth [-]</th>
<th>$H_{1/3}$ [m]</th>
<th>$H_{4/3}$ [m]</th>
<th>$H_{5/3}$ [m]</th>
<th>$H_{s}$ [m]</th>
<th>$H_{\text{mean}}$ [m]</th>
<th>$T$ [s]</th>
<th>$L$ [m]</th>
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<tbody>
<tr>
<td>1</td>
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<td>2.00*</td>
<td>1.62*</td>
<td>1.28*</td>
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**Table N.2 - Wave characteristics for different wind speeds with different return periods**

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<th>Wind direction</th>
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<th>Maximum value due wind</th>
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<td></td>
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<tr>
<td>E-ENE</td>
<td>$H$ [m]</td>
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<td></td>
<td>$H_{s}$ [m]</td>
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<tr>
<td></td>
<td>$T$ [s]</td>
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</tr>
<tr>
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<td>$T$ [s]</td>
<td>9.70</td>
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0. Appendix – PLAXIS results

0.1 Original dike

Figure 0.1 - Original dike structure in PLAXIS
Figure O.2 - Settlement original dike due to own weight in PLAXIS
Figure O.3 - Deformation at failure original dike in PLAXIS
Figure 0.4 - Total principal strain directions original dike, clearly showing the circular slip failure plane.
Figure O.5 - Total displacements original dike

Maximum value = 11.13 m (Element 1721 at Node 3467)
0.2 New Dike

Figure O.6 - Original mesh new dike in PLAXIS
Figure O.7 - Settlement new dike due to own weight in PLAXIS
Figure O.8 - Deformation at failure new dike in PLAXIS
Figure O.9 - Total principal strain directions new dike, clearly showing two circular slip failure planes.
Figure O.10 - Total displacement new dike PLAXIS

Maximum value = 4,259*10^3 m (Element 1526 at Node 17723)
Table O.1 - Soil properties PLAXIS

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Layer thickness (m)</th>
<th>Layer bottom elevation (m)</th>
<th>Gradation limit (%)</th>
<th>Specific gravity (G_s)</th>
<th>Water content (w)</th>
<th>Liquid limit (L_l)</th>
<th>Plastic limit (P_l)</th>
<th>Plasticity index (P_i)</th>
<th>Atterberg limits I_L (S_l)</th>
<th>COV (%)</th>
<th>Vertical permeability (m/s)</th>
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<tr>
<td>0.1</td>
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<td>3.10</td>
<td>3.20</td>
<td>3.30</td>
<td>3.40</td>
<td>3.50</td>
</tr>
</tbody>
</table>

* COV= Coefficient of Variation
0.3 Verification of Safety Factor.

The safety factor can be determined using PLAXIS. The program itself gives a value for $\Sigma$Msf in the ‘calculation information’ tab in PLAXIS Output. It is checked whether this value is reliable.

For points on the failure surface the total displacement is plotted against the $\Sigma$Msf. This way the determined factor of safety can be verified.

0.3.1 Original dike

For this dike a $\Sigma$Msf of 1.951 is listed in the ‘calculation information’ tab PLAXIS. From Figure O.11 it can be observed that failure occurs at a $\Sigma$Msf of approximately 1.95. For this reason it can be concluded the value obtained from the ‘calculation information’ tab is valid.

Figure O.11 - Plot of Displacement against Msf, original dike

Figure O.12 - points on failure surface used in Msf plot, original dike
0.3.2 New dike design
For this dike a ΣMsf of 1.795 is listed in the ‘calculation information’ tab PLAXIS. From Figure O.11 it can be observed that failure occurs at a ΣMsf of approximately 1.795. For this reason it can be concluded the value obtained from the ‘calculation information’ tab is valid.

Figure O.13 - Plot of Displacement against Msf, new dike

Figure O.14 - points on failure surface used in Msf plot, new dike
P. Appendix - PLAXIS

P.1 Used model
For the calculation the Linear Elastic Mohr Coulomb model for PLAXIS is used. The Mohr Coulomb (MC) model is a basic method and is often used in order to acquire a first estimation. Several properties of the MC-model will be discussed and their relation/importance for our research

- Linear, not accurate.
  The stiffness in the MC-Model is linear. This leads to inaccurate results. However if a good value of E (Young’s Modulus) is chosen that represents the actual stiffness during the loading a good estimation of the stresses and deformations can be derived. In this way the MC-model is a good method for a first rough estimate. A more accurate method is advised to get more reliable results. In our project the first estimate is given, and for this reason the MC-model is a good method.
- Small amount of parameters required.
  Since the MC-model is inaccurate and linear, the MC-model needs the least amount of parameters in comparison with the different methods that can be used in PLAXIS. If the available data is limited, the MC-model therefore is a good solution. Since the in-situ data in China is limited, the MC-model is advised.
- No loading/reloading
  When using the MC-model it is not possible to model loading/reloading in an accurate way. In our project however loading/reloading does not take place, so this is no restriction.
- No clay/soft soil
  It is advised not to use the MC-model for clay/soft Soil. Since the soil consist mainly of silty sand this is no restriction.

P.2 Determination of soil parameters
For the soil inside the geo-tube and the hydraulic fill no soil data is present. In order to model the dike, assumptions have to be made. In order to make an educated assumption the method of deposition, the situation and circumstances are considered. For the filling of the geo-tubes the same soil is used as for the hydraulic fill, the soil from the top soil layer present nearby the dike is used to construct the dike. This soil layer has a thickness of 0.5 to 3.5 meters.

In order to work with PLAXIS five parameters are needed: E, c, \( \phi \), \( \nu \) and \( \psi \). The cohesion(c) is given in Table O.1. The angle of internal friction(\( \phi \)) is used to determine the different parameters:

The neutral ground pressure, \( K'0 = 1 - \sin(\phi) \)

The poisson ratio, \( \nu = \frac{K'0}{1+K'0} \)

Dilatancy angle , \( \psi = \phi - 30 \)

The value of the poisson ratio is advised to remain between 0.3 and 0.4.

For the layers underneath the dike, almost all required parameters for a correct PLAXIS model are present; only the stiffness has to be determined. Since the MC-model uses a linear approach, an E-modulus has to be chosen that represents the stiffness during the whole project. The value \( E_{s12} \) is given in Table O.1, which represents the development of E from 200 to 100kPa.
P.2.1 Soil layer 1
This layer consists of deposited sediments. A high void ratio can be expected, this can also be found in Table O.1. Due to the weight of the new dike, the soil is expected to behave slightly stiffer than the \( E_{s_{1-2}} \) test result. For this layer an \( E \) of 5500kPa is used. The void ratio of this layer is lowered to 0.4 to stay within the advised boundaries. The dilatancy is restricted to 0°.

P.2.2 Soil layer 2
This layer also consists of deposited sediments and has a very high void ratio. Due to the weight of layer 1 on top of this layer, a higher stiffness can be expected. Since the combined load of the dike and layer 1 will be higher than 200 kPa, a stiffer value of \( E \) is chosen in comparison to \( E_{s_{1-2}} \): 8000kPa. The angle of dilatancy, -6.5° is not restricted to zero. A small negative value of the dilatancy angle for sandy soils is possible for very loose material. (R.B.J. Brinkgreve, 2011)

P.2.3 Soil layer 3
The bottom layer is described as dense in chapter 7. Also the blow count of 17.5 for the Standard Penetration Test predicts a medium dense cohesion-less soil or a very stiff clay (Cornell University, 1990). For this reason a higher value of \( E \) is can be expected in comparison with the determined \( E_{s_{1-2}} \). In order to obtain realistic results an \( E \) of 20.000kPa is used.

P.2.4 Soil inside geo-tube
The geo-tubes are filled with the nearby soil from layer 1. The soil has a large water content and this is pumped into the tubes. Due to this process the grain distribution is disturbed. The soil will however be pumped in the geo-tubes to a certain filling percentage. This will result in a denser packing. The geo-tubes will be placed above the groundwater level, so the water content will decrease. This, and the denser packing, will lead to a significant increase in the angle of internal friction and the stiffness.

P.2.5 Hydraulic fill
The hydraulic fill consists of the soil from layer 1, which is collected in the vicinity of the dike, identical to the soil inside the geo-tube. In general a hydraulic fill becomes looser packed due to the deposition method. The relative density of the deposited soil is 60% at best, but generally under 50% in comparison with the original soil. (C.K. Lee, 1994). This means the hydraulic fill will have a smaller angle of internal friction and stiffness in comparison with the original soil. Since the cohesion is negatively correlated with the angle of internal friction, the cohesion will increase. Since the hydraulic fill will be deposited above the water table the excess pore pressure can be drained through the geo-tubes. Because the original soil has a void ratio of 0.884, the hydraulic fill will lead to a denser soil in comparison with the original soil. Therefore even a hydraulically deposited soil will have a denser packing than the original soil.

Concluded this means that the hydraulic fill will have a small increase in stiffness with respect to the original soil, and the geo-tube has a large increase in stiffness with respect to the original soil. The applied values are assumptions based on the general properties of silty sand.

P.2.6 Rip-rap
In the construction of the dike rip-rap is applied at several locations. The rip-rap has a higher stiffness and angle of internal friction in comparison with the other soil. It is very hard to find information about the Young's modulus or angle of internal friction of rip-rap, since this is difficult to measure. The angle of internal friction is estimated as 40°. For the stiffness of the rip-rap, the stiffness of rock
is used: \( E = 105 \times 10^5 \) kPa. The other parameters are based on these parameters. The rip-rap has cohesion of 1kPa. This is not realistic, but the effective tensions start at zero, the angle of internal friction is high and the stiffness is an order higher in comparison with the other materials. This could lead to numerical errors.

P.2.7 Clay
As is stated in chapter 7, sediments will deposit after the construction of the dike. In chapter 8 it is stated that a layer of 5.4m clay will be deposited. This clay lies on the inner side of the dike, is deposited horizontal and therefore cannot fail. Only the weight of the clay is important for the model.

P.3 PLAXIS Modeling

P.3.1 Drainage type
When using the MC-model several drainage types can be applied: Drained, Undrained-A, Undrained-B, Undrained-C and impermeable. For the calculation the drainage type ‘Drained’ is selected. A calculation should be considered undrained if the soil has a low permeability or when rapid loading occurs. During our calculation no rapid loading occurs, the dike is constructed step by step. The soil has no (extremely) low permeability. For this reason the drainage type ‘Drained’ is valid.

P.3.2 Boundary conditions
In order to create a correct numerical model the right boundary conditions have to be determined. If the borders are too close to the dike, they will influence the dike. If the borders are too far away the calculation will take unnecessary long; efficiency has to be used to obtain the ideal mesh size and distribution.

P.3.3 Bottom layers
In order to model the dike in a correct manner the underlying layers have to be applied in the model. These are soil layer 1, 2 and 3. A problem arises when inserting these layers in the model: large settlements will occur due to the own weight of these layers. This is not realistic since the soil is present at the site at this time and is in rest. PLAXIS however calculates the stresses in the soil and following the constitutive equations this leads to strains, and thus settlements. Removing these underlying soils and modeling the dike on a stiff layer would result in unrealistic results. This problem might be solved by using the k0 procedure to calculate the initial stresses. The k0 procedure is not possible to calculate shear forces. Since our dike has sloped surfaces, and shear forces appear, it is not possible to use the k0 procedure. With help from PLAXIS a solution was found for this problem; in the first phase only the underlying layers are modeled and gravity load is applied. This way the correct stresses present in the soil are calculated. In the second phase the displacements are reset to zero. This way the correct stresses are present, and the correct displacement (none). After this procedure the dike is inserted in PLAXIS.

P.3.4 Concrete armor
The Chinese dike design has a concrete armor with a thickness of 1.3 meter located at the seaside. This armor is present to avoid erosion, but also increases the stability of the dike. The concrete can be implemented in the PLAXIS model in two different ways: Create a section of soil with the properties of concrete. Or make a plate in PLAXIS with the properties of PLAXIS. For a plate the
bending and rotation stiffness can be inserted. Since the concrete armor is a thick plate in reality, the second option is chosen.

In order to model the plate the stiffness, area and moment of inertia is needed. For the concrete a stiffness of \( E=30000 \) kPa is used. The armor is divided in different parts to calculate the right moment of inertia and the area. For each part the area is determined by multiplying the width with the height. The moment of inertia \( I = \frac{1}{12} w \cdot h^3 \) where \( w \) is the width of the part. The value \( h \) is the height of the concrete armor, 1.3 meter.

A small problem is that, although the thickness is inserted in PLAXIS to determine the bending and rotation stiffness it is not used in the geometry. This means the plate has no thickness and extra measures have to be taken to create a correct dike geometry. For this reason the area of the concrete is modeled in PLAXIS using geometry lines. The properties of the plate are assigned to the bottom line, which is in contact with the soil. In order to make a correct drawing the area where the concrete is located in reality, is assigned material properties. This material has no strength or stiffness, and is never used during the calculation. This material is grey, and the plate is the blue line in Figure P.1.

To model the interaction between the soil and the concrete structure an interface is inserted. This interface has a virtual thickness, which is an imaginary dimension used to define the material properties of the interface. Since the interface between the concrete armor and the soil is intermediate between smooth and fully rough, a virtual thickness can be used to model this behavior. The interface prevents numerical errors at the corners of structures, which is present at our situation.

Due to the large thickness of the plate, the concrete armor plays a significant role in the stability of the dike. Without the armor, or with a lower stiffness, the dike fails on the outer side. If the armor however is applied the dike fails on the inner side at a larger Factor of Safety.
Q. Appendix – Wave height

Q.1 New wave height
As mentioned in section 9.1.2 the design height can be obtained by extrapolating already measured storm surge levels of that specific point. However there are no storm surge level measurements done at the whole Tiaozini area. This makes the computation of the required design height complicated. In order to calculate the design height the wave height distribution is used. Since this is not the right way to calculate the wave height, there is made use of a upper boundary to verify our assumption.

Q.1.1 Upper Boundary
There are two water levels with their probability known; Those are the design height of the dike that has to retain a once in fifty year wave and of the dike that has to retain a once in hundred year wave, 9.1.2. With those two data points known, the upper limit of the design height can be obtained by linearization. Since the frequency distributions are often Gumbel or exponential distributions (TU Delft, 2011), the real design height will be lower than the linearized design height. The linearized design height with an occurrence of once in thousand years is 11.6 meter.

![Linearization of the design height](image)

Table Q.1 - Design height by different return period

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<th>Year</th>
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<tr>
<td>100</td>
<td>8.72</td>
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Q.1.2 Wave distributions
The wave height distribution must be known in order to calculate the height exceeded by waves that occur once in thousand years. Different wave distributions will be compared with the measurements done at point four of Tiaozini reclamation, to obtain the right distribution. The needed measurements done on Tiaozini reclamation are listed in Table Q.2

Table Q.2 - Measurements at point 4

<table>
<thead>
<tr>
<th>point</th>
<th>$H_m$ [m]</th>
<th>$H_s$ [m]</th>
<th>$H_{5%}$ [m]</th>
<th>$H_{4%}$ [m]</th>
<th>$H_{1%}$ [m]</th>
<th>Depth [m]</th>
<th>$T_s$ [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2.44</td>
<td>3.57</td>
<td>4.090</td>
<td>4.200</td>
<td>4.770</td>
<td>8.56</td>
<td>7.73</td>
</tr>
</tbody>
</table>
Q.1.3 Rayleigh Distribution
The first fitting was done using one of the best known wave distribution, the Rayleigh distribution. The Rayleigh distribution formula according to Holthuijsen (Holthuijsen, 2007):

\[ P(H) = e^{-2\left(\frac{H}{H_s}\right)^2} \]

With the probabilities and their corresponding wave height and the significant wave height of each area known, all the data can be verified.

This led to a remark that the \( H_m \) has not got a normal relation with the significant wave height, see Figure Q.2. The value of the \( H_m \) measured is larger than the relation of \( H_m = H_s \cdot 0.95 \cdot \sqrt{\pi/8} \) that is normal with a Rayleigh distribution (Holthuijsen, 2007).

When changing the measured \( H_m \) there is another problem with the Rayleigh distribution. The tail does not fit with the measured values, because the wave height calculated with the Rayleigh distribution is too high in comparison with the measured values. The reason for this difference is that Rayleigh is perfect for deep water but oversizes the wave height in shallower water conditions. With a wave length around 60 meter and a maximum depth of 5-8 meter, the water depth is between shallow and transitional.

Q.1.4 Composed Rayleigh and Weibull distribution.
Now the water depth is indicated to be shallow and transitional water, another distribution is needed. For a better wave height distribution in shallow water waves Battjes & Groenendijk (2000) proposed a distribution composed out of two separate distributions. This distribution was composed by the Rayleigh as well as a Weibull distribution. In here the Rayleigh distribution is used for all the waves under a certain transition height. Above this transition height the Weibull distribution is used to take the influence of the depth in account.

\[
P(H) = \begin{cases} 
  e^{-2\left(\frac{H}{H_s}\right)^2} & \text{for } H < H_{tr} \\
  e^{-2\left(\frac{H}{H_{ch.1}}\right)^{k1}} & \text{for } H > H_{tr}
\end{cases}
\]
The transition height is found using the formula:

$$H_{tr} = \left( \beta_{tr,1} + \beta_{tr,2} \cdot \tan(\alpha) \right) \cdot d$$

At the transition height the probability of exceedence given by the use of the Rayleigh distribution must be equal to the probability of exceedence given by the Weibull distribution. In this way the significant wave height of the Weibull distribution can be expressed in terms of $k$ and $H_s$.

$$P(H) = e^{-2\pi \left( \frac{H_{tr}}{H_{ch,1}} \right)^{\kappa_1}} = e^{-2\pi \left( \frac{H_{tr}}{H_s} \right)^2}$$

$$H_{ch,1} = \left( \frac{H_{tr}}{H_s} \right)^{\left( \frac{1}{\kappa_1} \right)}$$

The parameters for the composed distribution proposed by Battjes & Groenendijk based on their laboratory tests were $\kappa = 3.6, \beta_{tr,1} = 0.35$ and $\beta_{tr,2} = 5.8$.

These parameters were put to a test by Mai, Wilhelmi & Barjenbruch (2010). They carried out long-term in-situ measurements at several locations in front of the German coast. Those measurements were compared with the formula’s proposed Battjes & Groenendijk. The results were that the parameters proposed by Battjes and Groenendijk were higher than the optimal fit found for the in-situ measurements. So the parameters have to be checked for each specific location before using them for designing purposes.

For point 4 the approach is the same as the approach Mai, Wilhelmi & Barjenbruch used. All the data known was compared with the formulas and the result was that the parameters proposed by Battjes & Groenendijk did not fit the measurements of point 4. After adjusting the parameters, the best fit for the Rayleigh-Weibull distribution was found with the values $\kappa = 2.81, \beta_{tr,1} = 0.29$ and $\beta_{tr,2} = 6.7$. The corresponding transition height by those beta values is $H_{tr}$=3.48, a little lower than corresponding significant wave height for the Weibull part $H_{ch,1}$=3.54.

![Graph](image)

Figure Q.3 - The Rayleigh, Weibull and Rayleigh-Weibull distributions
With the Weibull distribution in the tail part of the total distribution, the predicted height has less difference with the measured heights for waves with a low probability, see Figure Q.3. Due to the Weibull tail, the error in prediction is 0.0, where the error with use of only the Rayleigh distribution was over 0.6 meter for the waves with a probability of 0.01.

### Q.1.5 The Weibull distribution

The predictions done by the composed Rayleigh-Weibull distribution fits the measurements done at point 4 perfectly. But one must keep in mind that the $H_m$ measurement was changed in a value more in line with the Rayleigh distribution. Now all other values of $H$ appear to be in line with the Weibull distribution, the measured $H_m$ value may be as well in line with the Weibull distribution, so that all values are Weibull distributed.

The value of $H_m$ appears to be a fit in the Weibull distribution, see Figure Q.4. The values for $k$, $\beta_{tr,1,2}$ and their corresponding significant wave heights will change due to this shift. The new values will become: $\kappa = 2.823$, $\beta_{tr,1} = 0.232$ and $\beta_{tr,2} = 6.718$, $H_{tr} = 3.494$ and $H_{ch,1} = 3.548$.

![Figure Q.4 - Weibull distribution](image)

### Q.1.6 Design height in storms

The obtained wave height distribution gives the probability for only one wave. If there are more waves to be taken into account the probability of exceeding a certain wave height will increase. Especially during storms a dikes will be encountered by a lot of waves. All those waves have their influence on the total design height and probability of exceedence according to the formula out of the Hydraulic structures manual (TU Delft, 2011):

$$P(H > H) = 1 - e^{-N\cdot e^{-2\left(\frac{H}{H_{tr}}\right)^2}}$$

This Formula is only dependent on the Rayleigh distribution, while the distribution that fits the measurements of point 4 was a Weibull distribution. This will lead to a change in the formula:

$$P(H > H) = 1 - e^{-N\cdot e^{-2\left(\frac{H}{H_{ch,1}}\right)^\kappa}}$$

Here N is the amount of waves during the storm. The storms will have a longer duration when the occurrence is less; a storm once in 1000 years will last longer than one storm that takes place every year. For the duration of a storm, typhoon in this case, the actual storm duration is 6 hours. (Jiang, 2012) But with the buildup and the calm down a storm will take up to 40 hours. The average wave period is 7.73 seconds. With those two durations known the total amount of waves can be
determent. Together with the data already gathered, the design height can be obtained. All the results are listed in Figure Q.5 and Table Q.3

![Figure Q.5 - Design height per probability](image)

### Table Q.3 - Design height per probability

<table>
<thead>
<tr>
<th>Probability [Probability]</th>
<th>0,001</th>
<th>0,01</th>
<th>0,02</th>
<th>0,05</th>
<th>0,1</th>
<th>0,5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design height [m]</td>
<td>7,53</td>
<td>7,14</td>
<td>7,02</td>
<td>6,85</td>
<td>6,71</td>
<td>6,32</td>
</tr>
</tbody>
</table>

The design height exceeded only once in thousand years will be 7.53m for a storm duration of 40 hours. This is almost 2 meter higher than the design height given to the original dike with a once in fifty year period.

### Q.2 Significant wave height

Out of the deep sea conditions the significant wave height is gathered. In order to use those characteristics there is a conversion towards shallow water needed. This all will be discussed in these sections.

#### Q.2.1 Wave characteristics in deep sea conditions

The wind speed and probabilities of those wind speed are measured for different directions. Those directions are the Northeast - East-northeast, East – East-southeast and Southeast- South-southeast.

Since the dike is placed at the northern part of Tiaozini, there will be looked towards the Northeast- East-northeast part of wind measurements only. The measured data of several wind speeds lead towards certain wave heights with a given return period.

### Table Q.4 - Wave heights for given return periods

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>100</th>
<th>50</th>
<th>25</th>
<th>10</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_m$ [m]</td>
<td>5,13</td>
<td>4,83</td>
<td>4,6</td>
<td>4,18</td>
<td>3,24</td>
</tr>
<tr>
<td>$H_s$ [m]</td>
<td>8,01</td>
<td>7,50</td>
<td>7,16</td>
<td>6,54</td>
<td>5,10</td>
</tr>
</tbody>
</table>

The wave heights given in Table Q.4, are the wave heights for deep water waves and not applicable for immediate use as the wave heights needed for computing the dikes. There is a conversion to shallow water needed. The wave heights are in line with a Gumbel distribution:
The best fit of the Gumbel distribution with the measurements is found with these values for the following parameters: \( \beta = 0.45 \) and \( \mu = 3.12 \) for the \( H_m \) curve and \( \beta = 0.68 \) and \( \mu = 4.91 \) for the \( H_s \) curve. In Table Q.5 the results of the measurements are compared with the results from the Gumbel distribution.

The wave heights corresponding with a return period of once in thousand year can obtained by extrapolation. By use of the same formula with the exact same parameters except for a probability corresponding with a return period of once in thousand year, the new wave height is computed. As can be seen in Figure Q.6.

This will give a maximum of 6.20m for the mean wave height in a period of thousand years. And 9.61m as the maximum of the significant wave height in thousand year (Table Q.5).

<table>
<thead>
<tr>
<th>Years</th>
<th>1000</th>
<th>100</th>
<th>50</th>
<th>25</th>
<th>10</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_m ) [m]</td>
<td>5,13</td>
<td>4,83</td>
<td>4,60</td>
<td>4,18</td>
<td>3,24</td>
<td></td>
</tr>
<tr>
<td>( H_m ) Height from gumble distr. [m]</td>
<td>6,20</td>
<td>5,17</td>
<td>4,86</td>
<td>4,55</td>
<td>4,12</td>
<td>3,29</td>
</tr>
<tr>
<td>( H_s ) significant [m]</td>
<td>8,01</td>
<td>7,50</td>
<td>7,16</td>
<td>6,54</td>
<td>5,10</td>
<td></td>
</tr>
<tr>
<td>( H_s ) from gumble distr. [m]</td>
<td>9,61</td>
<td>8,04</td>
<td>7,57</td>
<td>7,09</td>
<td>6,45</td>
<td>5,17</td>
</tr>
</tbody>
</table>

Q.2.2 Deep to shallow

As said earlier, there is a conversion needed from deep water waves towards shallow water waves. When converting deep-water waves into shallow-water waves, refraction, shoaling and breaking have to be taken into account. Those three phenomenon’s will have their influence on the wave height.

- Refraction: A phenomenon of change in propagation velocity due to different water depths. In shallower water the propagation velocity will be less than in deeper water. This will make wave fonts turn towards the coast.
Shoaling: A deduction with a constant period in propagation velocity and wavelength, when the water depth decreases.

Breaking: When the waves become too steep or if the water depth/wave height ratio becomes too small and the waves can break. The breaking of waves due to the wave steepness is dependent on the wave height and wave length.

For the water depth/wave height breaking occurs when $\frac{H_s}{d}$ is above $0.4 - 0.5$. With the shallow water in mind and the high water heights of the deep water waves, the breaking phenomenon will set the boundary for the $H_s$ (TU Delft, 2011).

A more specific value can be found in the guidelines of TAW (2003), where from a substitution of $H_d = 2.2 \times H_mo$ in the formula of $(H_d)_{max} = 0.9 \times d_m$ the relation between $H_mo$ and $d_m$ can be found. This results in:

$$(H_mo)_{max} = 0.41 \times d_m$$

In those formulas $d_m$ is the water depth at a distance of a half wave length in front of the dike. But because the foreshore is very shallow the water depth will remain the same as at the toe of the dike.

So the maximum significant wave height will be at $0.41 \times d_m$. This relation is used already in the designed dike, since $\frac{d_m}{H_mo} = \frac{3.57}{8.56} \approx 0.41$. For the new dike this relation will be used as well. So with the new design height known, the maximum new design wave height in front of the dike can be determined. See Figure Q.7

<table>
<thead>
<tr>
<th>P [-]</th>
<th>0.001</th>
<th>0.01</th>
<th>0.02</th>
<th>0.05</th>
<th>0.1</th>
<th>0.5</th>
<th>Chinese design</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_d[\text{m}]$</td>
<td>7.53</td>
<td>7.14</td>
<td>7.02</td>
<td>6.85</td>
<td>6.71</td>
<td>6.32</td>
<td>5.56</td>
</tr>
<tr>
<td>$d_{extra}[\text{m}]$</td>
<td>-3.00</td>
<td>-3.00</td>
<td>-3.00</td>
<td>-3.00</td>
<td>-3.00</td>
<td>-3.00</td>
<td>-3.00</td>
</tr>
<tr>
<td>$d[\text{m}]$</td>
<td>10.53</td>
<td>10.14</td>
<td>10.02</td>
<td>9.85</td>
<td>9.71</td>
<td>9.32</td>
<td>8.56</td>
</tr>
<tr>
<td>$H_s[\text{m}]$</td>
<td>4.39</td>
<td>4.23</td>
<td>4.18</td>
<td>4.11</td>
<td>4.05</td>
<td>3.89</td>
<td>3.57</td>
</tr>
</tbody>
</table>

Figure Q.7 - Significant wave height

Q.2.3 Conclusion

After use of multiple wave height distributions, the wave height distribution appeared to be a Weibull distribution. With this wave distribution the design height is calculated. The significant wave height appeared to be limited by the shallow water conditions and is now dependent on the total water depth. The following design height and significant wave height are calculated for the different probabilities.

<table>
<thead>
<tr>
<th>P [-]</th>
<th>0.001</th>
<th>0.01</th>
<th>0.02</th>
<th>0.05</th>
<th>0.1</th>
<th>0.5</th>
<th>Chinese design</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_d[\text{m}]$</td>
<td>7.53</td>
<td>7.14</td>
<td>7.02</td>
<td>6.85</td>
<td>6.71</td>
<td>6.32</td>
<td>5.56</td>
</tr>
<tr>
<td>$H_s[\text{m}]$</td>
<td>4.39</td>
<td>4.23</td>
<td>4.18</td>
<td>4.11</td>
<td>4.05</td>
<td>3.89</td>
<td>3.57</td>
</tr>
</tbody>
</table>

Figure Q.8 - Overview wave heights

The dike will be designed on a once in thousand year probability of exceedence, therefore the P=0.1% (P=0.001 in the tables) will be used. This will mean a design height of 7.53m and a significant wave height of 4.39m.
R. Appendix – Material saving

Table R.1 and Table R.2 show the situation as is the case with an 1:50 year wave. Every value in the table is the amount of material saving by use of both the berm and toe structure. Out those two tables can be concluded that the dimensions of the toe that came out the feasibility study Tiaozini (15m long, hight of 1m+CSWL) is not the most optimal with regards to material saving. A smaller toe would be more efficient. However this is not applied. Other two reasons for using a longer wave height are the breaking of waves in lower water conditions or gain stability. From Table R.2 until Table R.4 can be seen that a shift towards shorter toe lengths is more optimal by decrease of the significant wave height. Out of

Table R.5 and Table R.6 there is such a similar shift by increase of the SWL. Reason for both of the changes is that the toe is getting out of the influence area of the significant wave height. Then there will be no influence in the wave run-up and the total dike height. In this way the build of the toe would be material loss. Only for lower wave conditions (a decrease of the SWL or an increase in significant wave height), the length of the toe of the original dike will be reached, see Table R.7 and Table R.8.

There can be concluded that the long toes are constructed for protection against the lower wave conditions. They are needed for the stability and therefore not constructed material efficient.

For the new wave characteristics a new optimization can be performed. As can be seen from Table R.9 the toe length is the most optimal with a short length. But as concluded in the validation of the original dike is the purpose of the toe protection of the lower slope against the low waves. This will be material inefficient. Therefore the toe optimization can be left out the total optimization.
R.1 Original dike

Figure R.1 - Original dike cross section

Table R.1 - High water condition; colors overall

<table>
<thead>
<tr>
<th>Index</th>
<th>Heigth Toe [+m CSWL]</th>
<th>Toe Length [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SWL 5,56 m</td>
<td>186</td>
</tr>
<tr>
<td>1</td>
<td>1,56</td>
<td>2,56 23,41 32,73 39,09 44,19 48,28 51,40 53,70 55,28 56,24 56,67 56,63 56,17 55,33 54,16 52,68 50,94 48,95 46,74 44,32 41,72 38,96</td>
</tr>
<tr>
<td>2</td>
<td>2,06</td>
<td>2,06 23,41 30,88 35,69 39,40 42,31 44,45 45,92 46,80 47,17 47,09 46,61 45,78 44,65 43,25 41,99 39,71 37,63 35,37 32,93 30,14 27,42</td>
</tr>
<tr>
<td>3</td>
<td>1,56</td>
<td>1,56 23,41 29,01 32,25 34,63 34,63 37,39 38,00 38,15 37,90 37,29 36,38 35,18 33,74 32,08 30,23 28,19 26,60 23,65 21,18 18,79 15,88</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>1 23,41 27,03 28,60 29,56 30,02 30,03 29,67 28,99 28,09 26,93 25,54 23,97 22,21 20,30 18,25 16,06 13,77 11,37 8,87 6,28 3,63</td>
</tr>
<tr>
<td>5</td>
<td>0,56</td>
<td>0,56 23,41 25,66 26,08 26,06 25,68 25,00 24,05 22,86 21,48 19,92 18,20 16,34 14,37 12,31 10,15 7,89 5,56 3,15 0,67 -1,87 -4,47</td>
</tr>
<tr>
<td>6</td>
<td>0,06</td>
<td>0,06 23,41 24,40 23,75 22,84 21,72 20,41 18,94 17,33 15,60 13,76 11,82 9,80 7,71 5,53 3,33 1,05 -1,27 -3,64 -6,05 -8,50 -10,98</td>
</tr>
</tbody>
</table>
### Table R.2 - High water condition; colors per row

<table>
<thead>
<tr>
<th>SWL</th>
<th>Toe Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.57 m</td>
<td>0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20</td>
</tr>
<tr>
<td>5.56 m</td>
<td>2,56 3,27 3,90 4,48 4,94 5,39 5,82 6,20 6,56 6,89 7,19 7,48 7,73 8,00 8,23 8,43 8,60 8,74 8,86 8,96</td>
</tr>
</tbody>
</table>

### Table R.3 - High water condition; lower significant wave height leads to a little shift towards left for optimal design length

<table>
<thead>
<tr>
<th>SWL</th>
<th>Toe Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.57 m</td>
<td>0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20</td>
</tr>
<tr>
<td>5.56 m</td>
<td>2,56 3,27 3,90 4,48 4,94 5,39 5,82 6,20 6,56 6,89 7,19 7,48 7,73 8,00 8,23 8,43 8,60 8,74 8,86 8,96</td>
</tr>
</tbody>
</table>

### Table R.4 - High water condition; more shifting towards the lower toe lengths by lower significant wave height

<table>
<thead>
<tr>
<th>SWL</th>
<th>Toe Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.57 m</td>
<td>0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20</td>
</tr>
<tr>
<td>5.56 m</td>
<td>2,56 3,27 3,90 4,48 4,94 5,39 5,82 6,20 6,56 6,89 7,19 7,48 7,73 8,00 8,23 8,43 8,60 8,74 8,86 8,96</td>
</tr>
</tbody>
</table>

### Table R.5 - SWL of 5m; more material saving possible.

<table>
<thead>
<tr>
<th>SWL</th>
<th>Toe Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.57 m</td>
<td>0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20</td>
</tr>
<tr>
<td>5.56 m</td>
<td>2,56 3,27 3,90 4,48 4,94 5,39 5,82 6,20 6,56 6,89 7,19 7,48 7,73 8,00 8,23 8,43 8,60 8,74 8,86 8,96</td>
</tr>
</tbody>
</table>
### Table R.6 - SWL of 4.5m: shift to longer toe length

<table>
<thead>
<tr>
<th>SWL: 4.5 m</th>
<th>Toe Len (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hs: 3 m</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2,56</td>
</tr>
<tr>
<td>2</td>
<td>2,06</td>
</tr>
<tr>
<td>3</td>
<td>1,56</td>
</tr>
<tr>
<td>4</td>
<td>1,06</td>
</tr>
<tr>
<td>5</td>
<td>0,56</td>
</tr>
</tbody>
</table>

### Table R.7 - Lower waves conditions; with a lower significant wave height, the more benefit by longer toe lengths; color by row

<table>
<thead>
<tr>
<th>SWL: 7,5 m</th>
<th>Toe Len (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hs: 3 m</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2,56</td>
</tr>
<tr>
<td>2</td>
<td>2,06</td>
</tr>
<tr>
<td>3</td>
<td>1,56</td>
</tr>
<tr>
<td>4</td>
<td>1,06</td>
</tr>
<tr>
<td>5</td>
<td>0,56</td>
</tr>
</tbody>
</table>

### Table R.8 - Lower waves conditions; with a lower significant wave height, the more benefit by longer toe lengths; color overall

<table>
<thead>
<tr>
<th>SWL: 7,5 m</th>
<th>Toe Len (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hs: 3 m</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2,56</td>
</tr>
<tr>
<td>2</td>
<td>2,06</td>
</tr>
<tr>
<td>3</td>
<td>1,56</td>
</tr>
<tr>
<td>4</td>
<td>1,06</td>
</tr>
<tr>
<td>5</td>
<td>0,56</td>
</tr>
</tbody>
</table>

### R.2 New dike

Table R.9 - The material saving by different dimensions for the toe of the new dike

<table>
<thead>
<tr>
<th>SWL: 7,5 m</th>
<th>Toe Len (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hs: 3 m</td>
<td></td>
</tr>
<tr>
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