Upgrading of iron ore jetty

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Preface

This study forms the final thesis assignment for the Master's Degree in Civil Engineering at the Faculty of Civil Engineering of Delft University of Technology. The assignment was given by Witteveen + Bos Consulting engineers.

Without mentioning names I would like to thank those, who supported me in completing this assignment.

Delft, 28th May 1996
C.P. Labouchere
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Appendix IV : Models of load delivery in future situation
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Abstract

Background information

The jetty, discussed in this report, has to be adapted to meet new demands. The jetty is owned and operated by P.T. Krakatau Steel (P.T.K.S.). It is used for transferring iron ore. P.T.K.S. intends to increase the steel production in the coming years and therefore wants to upgrade a certain part of the jetty. The jetty exists of three smaller jetties, with an average length of 285 metres, which are in line with each other. The two oldest jetties were built between 1974 and 1975 and are designed for vessels up to 50,000 D.W.T.. The jetty built most recently was built between 1990 and 1992 and is originally designed for vessels up to 70,000 D.W.T. The latter jetty has to be modified to accommodate vessels up to 150,000 D.W.T.. The jetty is situated on the Northwest coast of the Indonesian island of Java, about 8 kilometres South-Southwest of Merak along the Sunda Strait.

Problem definition

The jetty which now accommodates ore carriers up to 70,000 D.W.T. should in the near future accommodate ore carriers up to 150,000 D.W.T.. This makes demands upon the jetty which cannot be met. The problem is to meet the new demands with as few and as simple adjustments as possible.

Objectives

The main objectives of this graduation report are enumerated below:

• to make a draft constructual design of how to upgrade the jetty;
• to make a plan of execution.

The elements adapted in this report are those which are of primary importance to the overall structural integrity and functionality of the jetty. These elements are:

• the strength and stiffness of supporting piles;
• the fendering system;
• the mooring system;
• the dimensions of the jetty.
Design procedure

The design procedure is subdivided into several steps. The first step is to make an inventory of the possible major upgrading problems. The problems distinguished at this point in the report are:

- dredging of sea bottom;
- intolerable reduction of design bearing capacity of piles as a result of dredging activities;
- insufficient fendering and mooring capacity;
- insufficient quay length.

The second step comprises a step-by-step solution of these problems. The solving procedure of each problem is discussed below.

**Problem 1**: The dredging of the sea bottom.

The problems related to the dredging activities itself are beyond the scope of this report. The result of the dredging activities, i.e. the course of the desired bottom level, is determined.

**Problem 2**: The upgrading of affected pile capacities in the future situation

A method for upgrading of the affected pile capacities in the future situation is designed according to the following procedure. First the pile loads in the future situation are determined, then the pile capacities in the future situation are determined, and finally 5 upgrading alternatives are presented from which one alternative is chosen.

**Problem 3 and 4**: Insufficient fendering and mooring capacity and quay length

The problems of insufficient fendering and mooring capacity and insufficient quay length are related to each other. The problems are twofold. On the one hand the individual fenders and bollards do not have the appropriate specifications, on the other hand the present spreading of the bollards and fenders will not meet the future demands. This is where the two problems are related. As the quay length is insufficient the bollards and fenders cannot be positioned correctly. An additional problem is the limited reach capacity of the unloader system.

The final step comprises a sublimation of the solutions of the forementioned problems to one overall structural upgrading design.

Conclusions and recommendations

The final overall design consists of the following elements:

- The piles of pile row 1 and 2 (see figure 1) are upgraded by means of compaction grouting directly below the pile points;
- The positions of the fenders remain unchanged. The present fenders themselves are replaced by fenders of the type "SUC 2000". These are provided by the same manufacturer as the present fenders;
• The positions of the bollards remain unchanged. The present capacity of the connection between the concrete deck and the bollard is upgraded from 1,000 kN to 1,500 kN. The detailed design of the connection between the bollard and the deckstructure is beyond the scope of this report;
• in order to lengthen the effective quay length 3 dolphins are constructed which provide the necessary extra mooring and fendering points.

The aforementioned adaptations are explained with the help of figure 1 to figure 3.

figure 1. cross section of upgraded jetty
Upgrading of iron ore jetty

Figure 2. Cross section of dolphin

Figure 3. Top view on upgraded jetty
1

Introduction

Commissioner

P.T. Krakatau steel (PTKS) assigned P.T. Krakatau Engineering Corporation (KEC) to undertake all necessary activities for the preparation of detailed designs and tender documents for the modification of an existing jetty. For the works mentioned above KEC participates with Witteveen + Bos Consulting engineers (W+B). The latter has given me the opportunity to make a design of the modification as a graduation assignment.

Evaluation of the situation

The jetty is owned and operated by PTKS. It is used for transferring iron ore. PTKS intends to increase the steel production in the coming years and therefore wants to upgrade a certain part of the jetty. The jetty exists of three smaller jetties, with an average length of 285 metres, which are in line with each other. The two eldest jetties were built between 1974 and 1975 and are designed for vessels up to 70,000 D.W.T.. The jetty built most recently was built between 1990 and 1992 and was originally designed for vessels up to 70,000 D.W.T.. The latter jetty has to be modified to accommodate vessels up to 150,000 D.W.T.. The jetty is situated on the Northwest coast of the Indonesian island of Java, about 8 kilometres South-Southwest of Merak along the Sunda Strait.

Problem definition

The jetty which now accommodates ore carriers up to 70,000 D.W.T. should in the near future accommodate ore carriers up to 150,000 D.W.T.. This makes demands upon the jetty which cannot be met. The problem is to meet the new demands with as few and as simple adjustments as possible.

Objectives

The following are the main objectives of this graduation assignment:

- to make a draft constructual design of measurements necessary to assure that the future structural and operational requirements are met;
- to make a plan of execution;
- to present the above in a final report.
Boundary conditions

The following are the main boundary conditions as given by KEC:
• the present state of the jetty;
• the other harbour activities must be disturbed as little as possible;
• the present equipment and infrastructure will not be changed.

Assumptions

The following are the main assumptions as made by the author:
• The only construction of interest is section B of the jetty.
• Elements of the jetty which are of no constructive significance will not be considered.

Working method

The working method used consists of the following steps:
• inventorise the potential problems, chapter 4;
• calculate the future pile loads, chapter 5;
• calculate the future pile capacities, chapter 6;
• design method for upgrading of piles, chapter 7;
• design of extra mooring- and fending points, chapter 8.

References

All references to literature are made by mentioning the author and the year of publication. The literature list is ordered alphabetically taking the initial letter of the author's name.
2

Collection and evaluation of existing data

2.1. Data of site

2.1.1. Source of data

The information in this paragraph about the existing situation has been obtained from data available in various reports received from W+B.

2.1.2. Location of the site

PTKS owns and operates a jetty at the Cigading Port which is situated in the vicinity of the city of Cilegon and the Krakatau Steel Plants. The port lays on the Northwest coast of the Indonesian Island of Java, about 8 kilometres South-Southwest of Merak along the Sunda Strait. The approximate position of the Cigading port is latitude 6°01' South and longitude 105°57' East. Figure 2.1., 2.2. and 2.3. will give a good impression of the location and configuration of the port. These figures can be found at the end of this report.

2.1.3. Reference level

All levels will be expressed in metres relative to "Klockner level" which is commonly used for all works in the Cilegon/Cigading area.

The relation between Klockner and Chart Datum (which is used as the reference level on the Admiralty Chart) is not known. This will probably not cause large problems as the tide range is not more than 1 metre. For this design it is assumed that Chart Datum and Klockner level are equal.

2.1.4. Tides and tidal levels

Tides in the Sunda Strait are semi-diurnal with the mean spring range being not more than 1.0 metre. Tidal conditions are indicated in table 2.1., and are expressed in accordance with Klockner's approach.
2.1.5. Wind

The predominant wind blows from the south-western and north-western directions in the period from October until May (Northwest monsoon). During the rest of the year winds blow mainly from the Southeast (Southeast monsoon).

Generally the wind velocities rarely go beyond 12.5 m/s (wind force 6 on Beaufort scale). Occasionally storms with wind velocities in the order of 20 m/s (wind force 8 to 9 on Beaufort scale) may occur, however these peak velocities are only of short duration. The wind directions are visualized in figure 2.4.

The design wind speeds are taken as follows:
* operational: 13 m/s; this results in a wind pressure of 125 N/m²
* storm: 23 m/s; this results in a wind pressure of 400 N/m²

### Table 2.1. Tidal Conditions

<table>
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<th>Abbreviation</th>
<th>Klockner [m]</th>
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<tr>
<td>High Water Spring</td>
<td>HWS</td>
<td>+1.39</td>
</tr>
<tr>
<td>Mean High Water</td>
<td>MHW</td>
<td>+0.98</td>
</tr>
<tr>
<td>Mean Sea Level</td>
<td>MSL</td>
<td>+0.75</td>
</tr>
<tr>
<td>Mean Low Water</td>
<td>MLW</td>
<td>+0.53</td>
</tr>
<tr>
<td>Low Water Spring</td>
<td>LWS</td>
<td>+0.17</td>
</tr>
</tbody>
</table>
Collection and evaluation of existing data


**Figure 2.4.** Current and wave directions
2.1.6. Waves

The predominant waves are from directions varying between Southwest and Northwest. During the east monsoon waves of limited heights from 0.5m to 1.0 m come from the Northeast. In April/May 1972 waves coming from the Southwest with heights of 1.5 m and with periods ranging between 5 s and 15 s were observed. Extreme heights that were observed are a wave height of 2.5 m in March 1975 and a wave height of 3.0 m in October 1975. Although such extreme heights occur only now and then it can be assumed that considerable wave action will have to be expected over longer periods not necessarily attributable to the rainy season.

From the wave measurement a significant wave can be deduced.

The characteristics of the significant wave are:
- wave height: 1.7 m;
- wave period: 2.5 s to 3.5 s;
- wave length: 18 m to 20 m.

Between October and April swell can be expected in an otherwise calm sea.

The wave actions are visualized in figure 2.4.

2.1.7. Currents

The currents along the coast and within the harbour area are determined by alternating tide currents in the Sunda Strait between Java and Sumatra and by the prevailing direction of winds. Tidal currents are largely parallel with the shoreline and the longitudinal axis of the existing jetty. The current directions are indicated on picture 2.4.

Current velocities of up to 0.6 m/s may occur in both directions. However, under normal circumstances a speed of 0.35 m/s will not be exceeded.

As wave data are not based on long term measurements the following is the design current:

design current velocity: 2.1 m/s.

2.1.8. Subsoil conditions

The information about the subsoil conditions can be found in various reports on the subsequent phases of the jetty design/construction. These data have been recapitulated in appendix I. The information is obtained from:
- SPT-tests;
- laboratory tests of material taken from the boreholes;
- pile loading tests;
- pile driving records;
- seismic survey.

The top layer exists of uncemented silty sand which is of medium density with SPT-values varying from 15 to over 40. The thickness of the layer varies from 6 to 10 metres. The subsequent layer consists of tuffaceous silty sand which is weakly cemented and very dense with SPT-values of over 100. The layer extends to about 25 to 30 metres below sea level.
2.1.9. Seismicity

The port is located in an earthquake prone area. The regulations by which the earthquake forces will be calculated are explained in appendix III.

2.1.10. Precipitation

Between the months of December and March there is a pronounced rainy season marked by intensive rain storms with a duration of up to 2 to 3 hours. During this period up to 75 percent of the annual rainfall of 2,000 mm occurs. An average of 156 days of rain per year has been observed. These strong rains can possibly affect visibility and consequently may hamper navigation as well as cargo handling.

2.1.11. Temperature

The average minimum temperature is about 22 °C and the average maximum temperature is about 31 °C. In the coastal region the temperature never falls below 19 °C and the absolute maximum temperature is circa 36 °C. For design purposes a fluctuation plus minus 15 ° will be adopted.
figure 2.5. top view on deck structure
2.2. Data Of Jetty

2.2.1. Port layout

Figure 1.1. shows the port plans taken from the latest issue of the Admiralty Chart No. 918. Figure 1.3. shows the existing port facilities in more detail. The existing facilities consist of an offshore 'T' head pier which comprises berths for:
- The importation of iron ore pellets and coal.
- The importation of scrap metal and import/export of semi-finished and/or finished steel products.
- The handling of general cargo.

There is also a small shore-based wharf for off-loading scrap metal from barges. The orientation of the frontal facade of the present jetty is approximately from Southwest to Northeast, i.e. broadly parallel with the original coastline. The extent of exposure to waves, currents, etc., is relatively high as the port of Caging does not comprise a natural harbour and is also not protected by breakwaters.

2.2.2. Jetty structure

General
The existing T-shaped jetty is projecting about 300 m from the shore and consists of a reinforced concrete deck supported by tubular steel foundation piles. Two sections shall be distinguished:
- Section A which was constructed in the years 1974 until 1978 and which is suitable to receive 50,000 D.W.T. vessels. The length and width of this section are about 570 m and 33 m.
- Section B which is constructed in the years 1990 to 1992 as an extension to section A. Section B is suitable for 70,000 D.W.T. vessels. The length of section B is 287.1 m and its width is 22.15 m.

Section B is the main subject of this study, it shall be modified so that vessels up to 150,000 D.W.T. can visit the jetty section. Section B of the jetty is primarily used to unload iron pellets by means of grab unloaders and is the latest port extension. It was built as a separate structure Northeast of the existing jetty structures.

Deck
As can be seen in the figure 2.5. every 5 metres in longitudinal direction the deck slab is supported by a reinforced concrete beam in transversal direction. Figure 2.6. presents a transverse cross-section of the jetty. The total number of these transverse beams is 58. The deck itself exists of precast prestressed concrete planks, with a thickness of 180 mm and a length of 4.1 m, on top of which a cast in situ reinforced concrete slab has been placed. Hence, the precast planks have served as form work during the construction and form permanent structural elements of the deck. The structural thickness of the in situ layer is 270 mm, although the actual thickness is slightly more due to the slope of the deck, In view of
figure 2.6. cross section of jetty
dewatering the deck slopes down from the central longitudinal axis of the jetty. The elevation of the jetty's longitudinal edges is Klockner +3.95 m.

**Foundation piles**
Each transverse beam is supported by four vertical tubular steel foundation piles with an outer diameter of 762 mm and a wall thickness of 14 mm. On the rear side of the jetty two raker piles are placed under the unloader track and centred between aforementioned transverse beams, except for the end spans where four raker piles are installed. All raker piles have an outer diameter of 1016 mm and a wall thickness of 14 mm.

**Fenders**
The frontal face of the existing jetty is provided with twelve Bridgestone fenders of type SUC 1600 H, rubber grade RE. This is an axially loaded cylindrical fender which is combined with a fender panel. Per fender six chains are connected to the steel frame of the panel. The centre-to-centre distance of the fenders is 25 m whereas both outer fenders are placed 7.5 m from the end beam axes. As an exception the distance between the two fenders nearest to the section A of the jetty is only 20 m. All fenders are centred between two transverse beams. The fender is mounted to a vertical concrete surface which is situated 600 mm behind the frontal face of the jetty. The other dimensions of the recess are 3800 mm horizontal and 2500 vertical. To protect the corners of the north-eastern end of the jetty tubular have been applied. Figure 2.7. shows the fender.

![Figure 2.7. A "Bridgestone" fender of type SUC 1600 H](image)
Bollards
Twelve bollards have been placed along the front side of the jetty at centre-to-centre distances of 25 metres, except for the distance between the bollards at the axes 54 and 58 which is 20 metres. At the other end the first bollard is placed 15 m from axes No. 1. Each bollard is situated on top of a transverse beam. The capacity of each bollard is 1000 kN. One additional bollard is placed at the rear side of axis 58.

Structural relation between section A and section B
There is a joint of 300 mm between jetty sections A and B so that the sections can move relative to each other. The rails for the unloaders is continuous over the joint in order to allow the unloaders to travel from one section to another. Yet the rail detail near the joint is designed in such a way that the jetty sections can still move independently from each other.

Labeling of structural elements of the jetty
The structural elements of the jetty are labeled in order to be able to accurately specify the different elements without the necessity of large expatiations. The labeling of the deck elements is indicated in figure 2.5. The labeling of piles is indicated in figure 2.6.

2.2.3. Pellet handling equipment

Gantry cranes
Unloading of the iron ore pellets or coal takes place by means of rail mounted gantry cranes equipped with a grab. The electrical powered unloaders move alongside the vessel during unloading so that warping of the vessel along the jetty is not required. The unloaders travel over rail tracks which are placed on the jetty deck with a centre-to-centre distance of 12.5 m. The grab is a clamshell bucket with a capacity of 20 tonnes (grab plus contents) at 28 metres outreach from the waterside rail. From the clamshells, the material is fed into the ship unloader bins and further, through vibrating feeders, onto the conveyor belts.

Presently four unloaders are operational in the port, however, as yet the two older ones cannot travel to the pallet jetty extension. As mentioned in the preceding, these two unloaders will be adjusted in such a way that in the future four machines will be able to service a vessel moored against the jetty extension. Figure 2.6. and 2.9. show the gantry crane.

Conveyor belts
The conveyor belts for horizontal transport of the iron ore pellets are situated at the rear side of the jetty. The conveyor system is elevated through its entire length and are covered by a canopy structure. The concrete foundation footings have been constructed every 15 metres centred on top of a transverse beam. Figure 2.6. shows the location of the conveyor belt on the deck of the jetty.

Nautical aspects and procedures
Vessels from Europe enter from the Sunda Strait from the south, vessels from Brazil come from the north. In the future traffic may be regulated as follows:
When sailing the Sunda Strait from north to south the vessels shall pass west of Sangian Island, when sailing from south to north the vessels shall pass east of Sangian Island. It is not yet known how the future approach to the port of Cigading will be.

Presently all vessels berth and de-berth with the assistance of tugboats. The permissible berthing angle is 10 degrees, however, in practice the vessels are almost parallel with the jetty
at the moment of contact. Berthing takes place against the current whenever possible whereas the current velocity shall be at least 0.5 m/s, but preferably higher.

The normal berthing procedure is that the vessel first stops in parallel with and about 20 m to 30 m from the jetty and then gradually approaches the jetty while being pushed by several tugboats. The ship unloaders will move away from the berthing point in order to avoid a possible collision with the berthing vessel. When a vessel is berthing the booms of the cranes will be in an upright position.

Normally a vessel will not turn in front of the berth. After unloading it will continue its course in the same direction as before berthing. An arriving vessel and a departing one will never meet.

2.3. Data of design vessel

The design vessel is an ore carrier of 150,000 D.W.T. According to Bridgestone it has the following numerical characteristics:

- full loaded displacement tonnage (FLD) = 200,000 tonnes
- length between perpendiculars (LBP) = 300 m
- width = 45.0 m
- depth = 25.0 m
- full draught (FD) = 16.0 m
- additional weight = 61,795 tonnes
- estimated weight = 261,795 tonnes
upgrading of iron ore jetty

**figure 2.8.** front view on gantry crane
3 Design requirements

3.1 Boundary conditions

3.1.1 Natural boundary conditions

Soil data
In front of the jetty the sea bottom is smooth with no signs of outcropping rocks. The sea bottom level lies at approximate 16 m below the sea level. The reference sea level has not yet been determined by W+B but is assumed to be the Klockner level.

The bottom material in the surroundings of the jetty has a strong layered structure. The layers of importance are the top three layers. They mainly consist of the following soil material:
- layer 1: soft sandy silt;
- layer 2: medium dense silty sand;
- layer 3: very dense tuffaceous silty sand, this layer extents further than 24 m below the sea bottom.

The sand is of fine to coarse grained type.

The available soil data has been gathered with the help of the following surveys:
- boring of test pits: 3 points with a total depth of 83 m;
- standard penetration tests: 40 tests;
- laboratory tests on the soil samples obtained from the boring;
- bathymetric, echo sounding and hydroseismic surveys;
- test pile loading: -1 test with a pile with a diameter of 762 mm;
  -1 test with a pile with a diameter of 1016 mm.

The principal results are presented in the figures 3.1. and 3.2. and table 3.1.

<table>
<thead>
<tr>
<th>table 3.1. soil properties.</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT-value</td>
</tr>
<tr>
<td>sandy silt</td>
</tr>
<tr>
<td>silty sand</td>
</tr>
<tr>
<td>silty sand</td>
</tr>
<tr>
<td>tuffaceous silty sand</td>
</tr>
<tr>
<td>tuffaceous clay</td>
</tr>
</tbody>
</table>

Where:
SPT-value: The blow count of the standard penetration test [blows/0,3 m].
γ : The density above water [kN/m³]
γ' : The density submerged [kN/m³]
ϕ : The theoretical value of the internal friction [°]

![Diagram of section B](image)

**Figure 3.1.** Plan of location of boreholes

![Soil profiles of boreholes](image)

**Figure 3.2.** Soil profiles of boreholes

**Environmental data**
The data concerning wind, waves, current, temperature and seismicity is given in chapter 2.
3.1.2. Technical-constructive boundary conditions

Material properties
The material properties of the jetty are as indicated by KEC. These can be found in appendix I. If the information given by KEC is incomplete or lacking, than common values as indicated in the British Standards will be used.

Loads
The load types, taken into account, are as indicated by KEC. A list is given in appendix II. The resulting pile loads are multiplied with a safety factor 2 to obtain sufficient design safety. This safety factor is obtained from W+B.

Design references
For the upgrading design of this jetty, more than 1 code of practice is used. The British Standards will be adopted as the general standard, the other codes of practice will be used when they are considered to be more appropriate or more accurate. The design references of PTKS are of principal importance. The codes of practice used are:
- the British Standards on Maritime Structures (BS 6349);
- the British Standards on Design of Foundations (BS 8004);
- the British Standards on Design of Concrete Structures (BS 8110);
- the Design Criteria for Ports in Indonesia;
- the Recommendations of the Committee for Waterfront Structures (EAU 1980);
- the Bridgestone Design Manual;
- the Indonesian Earthquake Code.

3.1.3. Boundary conditions in the operational phase

The boundary conditions in the operational state are:
- the vessels must be allowed to berth with the bow pointing to the Northeast as well as to the Southwest;
- the unloaders of section A must be able to operate on section B as well.
- section B of jetty must accommodate independently ore carriers with a capacity varying from 70,000 D.W.T. to 150,000 D.W.T.

3.1.4. Boundary conditions in the executive phase of upgrading

The boundary condition in the executive phase of upgrading is:
- The disturbance of the operations at the other berthing places in the port during upgrading activities must be negligible.
3.1.5. Design boundaries

The design comprises only the necessary reinforcement of the existing structure. Its basic shape is not to be changed. The upgrading design does not contain the following components of the jetty:

- the power supply;
- the lighting;
- the portable water supply;
- the fire fighting system;
- the fuel supply;
- the ladders and life saving equipment;
- the connection of the rails between section A and B.

3.2. Assumptions

3.2.1. Natural assumptions

Soil data
The soil profiles as shown in figure 3.2. are simplified. The soil profile is assumed to be similar for all locations. The top soil layer is neglected. This layer has a thickness of 1 metre. The sub layer, which extends in depth infinitely, is assumed to be homogeneous. The properties of this layer are determined in chapter 6.

3.2.2. Technical-constructive assumptions

Loads
Of all the load types indicated by KEC, the influence of 5 load types were taken into account. These 5 load types are:

1. the dead load of the concrete deck;
2. the live load on the concrete deck;
3. the unloader loads;
4. the vessel loads;
5. the seismic loads.
3.3. Programme of Requirements

3.3.1. General requirements

The general requirements are:
- the vessels can safely berth and moor in front of section B of the jetty;
- the unloading operations can be carried out smoothly;
- the unloading will be done with the existing cranes including those of section A of the jetty;
- all units shall be metric and in line with the SI standard. Where required for easy reference other units may be adopted but only in addition to the SI units;
- the present parking places for the unloaders will remain the same in the future situation.
  This means that only 2 unloaders will be parked on section B of the jetty.

3.3.2. Technical-constructive requirements

The technical-constructive requirements are:
- the jetty structure of section B and all related facilities can withstand the forces to which it may be subjected;
- the design loads for the jetty used for the upgrading design are presented in appendix I and III;
- the load combinations used for the upgrading design are as presented in appendix I;
- the pile loads are multiplied with a safety factor 2;
- the present material properties of the jetty are presented in appendix I.
- the length of the jetty is not to be altered;
- the frontal face of section B of the jetty is not to be shifted;
- the sea bottom level will be lowered from the present 16 m to 20 m below Klockner;
- the pile and deck deformations of section B of the jetty may not be significantly larger in the future situation than in the present situation.

3.3.3. Requirements for the constructive phase

The requirement for the constructive phase is:
- all activities in the constructive phase must be carried out without the help of other sections of the jetty apart from section B. No vessel or equipment, which is used for constructive activities, may obstruct operational activities of the jetty apart from section B.
Upgrading of iron ore jetty
4

Inventory of principal upgrading problems

4.1. Introduction

In order to be able to make a well-considered design of the necessary upgrading measurements it is necessary to accurately define the principal difficulties.

The principal difficulties, as selected by the author, are discussed in this chapter. The results will be used as a framework for the plan of action of the upgrading design further in this report.

Five problems were selected. These problems are of primary importance to the overall structural integrity and functionality of the jetty. In the following sections these problems will be discussed one by one.

The 5 principal problems are:
1. decrease of structural integrity of piles as a result of dredging activities;
2. insufficient quay length;
3. insufficient fending capacity;
4. insufficient mooring capacity;
5. insufficient outreach capacity of unloader.

4.2. Decrease of structural integrity of piles

The 150,000 D.W.T. iron ore carrier puts new demands to the water depth in front of the jetty. The average draft of this type of ship is 16.5 m. As a result of this the water depth must increase from approx. 16 m. to approx. 20 m. A reduction of the embedded depth of the piles from pile row 1 and 2 (figure 4.1.) is a result, assuming that a natural slope comes into existence beneath the jetty.

The axial and lateral bearing capacities are reduced by the decrease of embedded depth. To be able to estimate the extent of the problem, and eventually design structural measurements to eliminate the problem, it is necessary to quantify the problem. This is done according to the following plan of action: first an estimation of the future pile loads is made (chapter 5). Then an estimation of the future pile capacities is made (chapter 6). Finally measurements are thought of, in order to increase pile capacity (chapter 7).
4.3. Insufficient quay length

The 150,000 D.W.T. iron ore carrier puts new demands to the quay length of section B of the jetty. The overall length of a 150,000 D.W.T. vessel amounts up to 320 m. The necessary effective quay length is approx. 360 m. This number is obtained assuming a ratio between vessel length and quay length, that is independent of absolute values. The present jetty with its quay length of 285 m. falls approx. 75 m. short (figure 4.2.).

The jetty itself is not to be lengthened, as specifically stated in the programme of requirements. In order to increase the effective length of the jetty without lengthening the concrete deck structure, it is necessary to place dolphins. These dolphins must be able to withstand mooring and fending loads independently.
4.4. Insufficient fending capacity

The 150,000 D.W.T. iron ore carrier puts new demands to the fending system. These demands are twofold. On the one hand the spacing of the fenders is to be adapted. On the other hand the energy absorption capacity of the individual fender elements, and the strength of its connection to the deck are to be adapted.

The fender spacing problem is twofold also. On the one hand extra fenders must be placed, on the other hand the fender centres can be adapted. But as the maximum centres are determined by the smallest berthing vessel this will not be necessary.

4.5. Insufficient mooring capacity

The 150,000 D.W.T. iron ore carrier puts new demands to the mooring system. These demands are twofold. On the one hand the spacing of the bollards is to be adapted. On the other hand the strength of the individual bollards, and its connection to the deck are to be adapted.

The bollard spacing problem is twofold also. On the one hand extra bollards must be placed. On the other hand the bollard centres can be adapted. But as the maximum centres are determined by the smallest berthing vessel this will not be necessary.

4.6. Insufficient outreach capacity of unloader

The present unloader system is designed to cover a vessel with an overall length of approx. 260 metres. A vessel with an overall length of 320 metres will extend beyond the far end of the jetty. As a result of this the unloader cannot cover the entire vessel. (figure 4.3., configuration 2).
This problem can be solved in one of the following three ways:
1. lengthening of the jetty;
2. berthing of vessel in such a way that it is oriented with its bow towards section A of the jetty (figure 4.3., configuration 1);
3. moving of vessel parallel to jetty during unloading.

As lengthening of the jetty is not an option and moving of vessel during unloading takes too much time the second option is chosen as the best one. This means that no constructive measurements are required. The argument against this option is that the design requirements state that both mooring patterns must be possible. Because of the major cost reductions of this option, as no constructive measurements are necessary, the design requirement is dropped.
5

Estimation of future pile loads

5.1. Introduction

It is necessary to determine the pile loads in the future situation as a first step in the upgrading design process. This chapter deals with the calculation of the future pile loads. The jetty properties used are those as they exist in the present situation. This assumption is safe because it leads to an overestimation of the future pile loads. The pile loads are overestimated a result of an overestimation of the future axial pile stiffness, which in its turn is a result of the neglect of the reduced embedded depth. The pile loads are calculated with the computer program PC-Frame. The program and the calculations are explained in appendix IV.

KEC has given 5 load combinations. Each load combination consists of a series of load types. Of all these load types 5 are considered to be the most important. The locations where these loads act are marked in figure 5.1.

The 5 load combinations as defined by KEC:
1. the berth working under normal conditions;
2. a ship berthing at design speed;
3. a storm wind load;
4. a ship berthing at abnormal speed;
5. an earth quake.

The 5 principal loads from which the load combinations are composed:
1. dead load \([q1]\]
2. live load \([q2]\]
3. unloader load \([F2/F3]\]
4. vessel load \([F1]\]
5. seismic load \([F1]\]

All load combinations are examined in the following sections. The 5 principal loads of each load combination are determined after which they are implemented in PC-Frame. The loads are such that the maximum compression load in piles 1 and 2 is obtained. The resulting pile loads are given in tabular form. This chapter is concluded by a summary of the criterion pile loads.
Figure 5.1. Load types acting on cross section of jetty.
5.2.  The berth working under normal conditions

The results of calculation activities, performed to determine the design pile loads, are shown in this section.

The loads on the jetty
The loads as given by KEC and the loads as used as input in the PC-Frame program are both given in table 5.1. These two load groups are interrelated by three coefficients. The value of this coefficient is determined in appendix IV., and its function is to reduce the unavoidable distortion of reality. This distortion is caused by implementing the jetty structure properties into PC-Frame.

<table>
<thead>
<tr>
<th>load type</th>
<th>code</th>
<th>coefficient</th>
<th>value</th>
<th>resulting PC-Frame input</th>
</tr>
</thead>
<tbody>
<tr>
<td>dead load [kN/m]</td>
<td>q1</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>live load [kN/m]</td>
<td>q2</td>
<td>-</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>unloader load [kN]</td>
<td>F2</td>
<td>0.51</td>
<td>713</td>
<td>364</td>
</tr>
<tr>
<td></td>
<td>F3</td>
<td>0.57</td>
<td>309</td>
<td>176</td>
</tr>
<tr>
<td>vessel load [kN]</td>
<td>F1</td>
<td>0.14</td>
<td>53</td>
<td>7</td>
</tr>
</tbody>
</table>

The design pile loads
The pile loads as calculated by PC-Frame are multiplied with a safety factor "2" to obtain the design pile load.

<table>
<thead>
<tr>
<th>pile</th>
<th>axial pile load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>compression</td>
</tr>
<tr>
<td>1</td>
<td>938</td>
</tr>
<tr>
<td>2</td>
<td>2110</td>
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<td>3</td>
<td>2754</td>
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<tr>
<td>4</td>
<td>2266</td>
</tr>
<tr>
<td>5</td>
<td>2290</td>
</tr>
<tr>
<td>6</td>
<td>1144</td>
</tr>
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</table>
5.3. A ship berthing at design speed

The results of calculation activities, performed to determine the design pile loads, are shown in this section.

The loads on the jetty
The loads as given by KEC and the loads as used as input in the PC-Frame program are both given in table 5.3. These two load groups are interrelated by three coefficients. The value of this coefficient is determined in appendix IV., and its function is to reduce the unavoidable distortion of reality. This distortion is caused by implementing the jetty structure properties into PC-Frame.

<table>
<thead>
<tr>
<th>load type</th>
<th>code</th>
<th>coefficient</th>
<th>value</th>
<th>resulting PC-Frame input</th>
</tr>
</thead>
<tbody>
<tr>
<td>dead load [kN/m]</td>
<td>q1</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>live load [kN/m]</td>
<td>q2</td>
<td>-</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>unloader load [kN]</td>
<td>F2</td>
<td>0.51</td>
<td>1050</td>
<td>536</td>
</tr>
<tr>
<td>vessel load [kN]</td>
<td>F1</td>
<td>0.14</td>
<td>2460</td>
<td>344</td>
</tr>
</tbody>
</table>

The design pile loads
The pile loads as calculated by PC Frame are multiplied by a safety factor "2" to obtain the design pile load.

<table>
<thead>
<tr>
<th>pile</th>
<th>axial pile load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>compression</td>
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<tr>
<td>1</td>
<td>1100</td>
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<tr>
<td>2</td>
<td>2252</td>
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<td>3</td>
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<tr>
<td>4</td>
<td>1902</td>
</tr>
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<td>4072</td>
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<td>6</td>
<td>1056</td>
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</tbody>
</table>
5.4. A storm wind load

The results of calculation activities, performed to determine the design pile loads, are shown in this section.

The loads on the jetty

The loads as given by KEC and the loads as used as input in the PC-Frame program are both given in table 5.5. These two load groups are interrelated by three coefficients. The value of this coefficient is determined in appendix IV., and its function is to reduce the unavoidable distortion of reality. This distortion is caused by implementing the jetty structure properties into PC-Frame.

<table>
<thead>
<tr>
<th>load type</th>
<th>code</th>
<th>coefficient</th>
<th>value</th>
<th>resulting PC-Frame input</th>
</tr>
</thead>
<tbody>
<tr>
<td>dead load [kN/m]</td>
<td>q1</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>live load [kN/m]</td>
<td>q2</td>
<td>-</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>unloader load [kN]</td>
<td>F2</td>
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<td>790</td>
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<tr>
<td></td>
<td>F3</td>
<td>0.57</td>
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<td>vessel load [kN]</td>
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<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The design pile loads

The pile loads as calculated by PC-Frame have been multiplied with a safety factor "2" to obtain the design pile load.

<table>
<thead>
<tr>
<th>pile</th>
<th>compression</th>
<th>axial pile load</th>
<th>tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>972</td>
<td></td>
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<td>2128</td>
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<td>5</td>
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<td>-</td>
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<td>6</td>
<td>1180</td>
<td></td>
<td>-</td>
</tr>
</tbody>
</table>
5.5. A ship berthing at abnormal speed

The results of calculation activities, performed to determine the design pile loads, are shown in this section.

The loads on the jetty
The loads as given by KEC and the loads as used as input in the PC-Frame program are both given in table 5.7. These two load groups are interrelated by three coefficients. The value of this coefficient is determined in appendix IV., and its function is to reduce the unavoidable distortion of reality. This distortion is caused by implementing the jetty structure properties into PC-Frame.

<table>
<thead>
<tr>
<th>load type</th>
<th>code</th>
<th>coefficient</th>
<th>value</th>
<th>resulting PC-Frame input</th>
</tr>
</thead>
<tbody>
<tr>
<td>dead load [kN/m]</td>
<td>q1</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>live load [kN/m]</td>
<td>q2</td>
<td>-</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>unloader load [kN]</td>
<td>F2</td>
<td>0.51</td>
<td>1050</td>
<td>536</td>
</tr>
<tr>
<td>unloader load [kN]</td>
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<td>1550</td>
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<td>F1</td>
<td>0.14</td>
<td>2460</td>
<td>344</td>
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</tbody>
</table>

The design pile loads
The pile loads as calculated by PC-Frame have been multiplied with a safety factor "2" to obtain the design pile load.

<table>
<thead>
<tr>
<th>pile</th>
<th>axial pile load</th>
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</thead>
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<td></td>
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<td>3454</td>
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<td>6</td>
<td>722</td>
</tr>
</tbody>
</table>
5.6. An earthquake

The results of calculation activities, performed to determine the design pile loads, are shown in this section.

The loads on the jetty

The loads as given by KEC and the loads as used as input in the PC-Frame-program are both given in table 5.7. These two load groups are interrelated by three coefficients. The value of this coefficient is determined in appendix IV., and its function is to reduce the unavoidable distortion of reality. This distortion is caused by implementing the jetty structure properties into PC-Frame.

<table>
<thead>
<tr>
<th>load type</th>
<th>code</th>
<th>coefficient</th>
<th>value</th>
<th>resulting PC-Frame input</th>
</tr>
</thead>
<tbody>
<tr>
<td>dead load [kN/m]</td>
<td>q1</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>live load [kN/m]</td>
<td>q2</td>
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<td>75</td>
<td>75</td>
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<td>35700</td>
<td>615</td>
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<tr>
<td>vessel load [kN]</td>
<td>F1</td>
<td>0.14</td>
<td>2460</td>
<td>344</td>
</tr>
</tbody>
</table>

The design pile loads

The pile loads as calculated by PC-Frame have been multiplied with a safety factor "2" to obtain the design pile load.

<table>
<thead>
<tr>
<th>pile</th>
<th>axial pile load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>compression</td>
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<td>2264</td>
</tr>
<tr>
<td>4</td>
<td>70</td>
</tr>
<tr>
<td>5</td>
<td>3980</td>
</tr>
<tr>
<td>6</td>
<td>616</td>
</tr>
</tbody>
</table>
5.7. **Criterion design pile loads**

A maximum design load can be found for each pile. A list of maximum design loads and the accessory piles is given in table 5.11.

<table>
<thead>
<tr>
<th>pile</th>
<th>design load [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1382</td>
</tr>
<tr>
<td>2</td>
<td>2252</td>
</tr>
<tr>
<td>3</td>
<td>2952</td>
</tr>
<tr>
<td>4</td>
<td>3098</td>
</tr>
<tr>
<td>5</td>
<td>4072</td>
</tr>
<tr>
<td>6</td>
<td>1180</td>
</tr>
</tbody>
</table>
Estimation of future pile capacities

6.1. Introduction

This chapter comprises an estimation of the future bearing capacities of the plumb piles of pile row 1 and 2. Two formulas are designed. One formula which indicates the relation between embedded depth and the axial bearing capacity, and one formula which indicates the relation between the embedded depth and the lateral bearing capacity. Values for the pile capacities are given, assuming a reduction of the embedded depth of 4 metres.

**Determination of formula for axial bearing capacity**

The mechanism from which the piles derive their bearing capacity is determined. It turns out that the concerning piles derive their bearing capacity from shaft friction only. A formula is designed assuming a linear relation between depth and shaft friction at that depth (Tomlinson, 1994). The coefficients in the formula are determined with data obtained from field tests.

**Determination of the formula for lateral bearing capacity**

The lateral bearing capacity is determined with the help of a formula designed by "Broms" (Tomlinson, 1994). It is calibrated with data obtained from field tests.

6.2. Future axial bearing capacities

The axial bearing capacity is determined by one of the following two properties:
1. the bearing capacity derived from the soil in which it is driven;
2. the buckling strength of the pile.

The property which represents the smallest bearing capacity is the criterion property. Pile property 1 is examined in section 6.2.1. to section 6.2.4. Pile property 2 is examined in section 6.2.5.
6.2.1. Theory on pile response to axial load

A foundation pile can derive its bearing capacity from two different mechanisms. The point bearing capacity and the shaft friction capacity. This is explained in figure 6.1.

![Stress distribution in a foundation pile](image)

**figure 6.1.** Stress distribution in a foundation pile (v. Tol, 1995)

Where:
1. The stress distribution in case of shaft friction only.
2. The stress distribution in case of point bearing and shaft friction combined.

With the help of a load-settlement relation obtained from field tests one can determine what type of stress distribution occurs. The field tests are performed with the pile shown in figure 6.2. The test results are shown in appendix II. This test pile is assumed to be representative for all plumb piles supporting the jetty.

![Test pile diagram](image)

**figure 6.2.** Test pile, Ø 762 mm
The pile derives its bearing capacity from shaft friction only. This conclusion is drawn in consequence of the following three observations:
1. the load and its accessory settlement show a linear relation;
2. the load-settlement ratio of the pile loaded under compression and of the pile loaded under tension are approximately equal;
3. the displacement of the pile head, calculated assuming shaft friction only, show large resemblance to the displacement observed in the field test (see 6.2.2.).

6.2.2. Model of pile response to axial load

This section comprises the design and verification of a model of axial load delivery. The model is used to design a formula, which gives a relation between embedded depth and bearing capacity. The main assumption used in the model is that the pile derives its bearing capacity from shaft friction only. This shaft friction is assumed to have a linear relation with the embedded depth (Tomlinson, 1994).

The stress distribution model

Assumptions:
- homogeneous soil;
- shaft friction has linear relation with depth below ground surface level;
The above-mentioned assumptions lead to the following formula regarding pile capacity and pile stiffness.

\[ P = C \cdot l_2^2 \]

\[ \Delta z = \frac{P \cdot l_1}{EA} + \frac{P \cdot l_2}{2 \cdot EA} \]

Where:
- \( \Delta z \) : pile head settlement
- \( l_1 \) : length of part of pile above soil
- \( l_2 \) : length embedded part of pile

**Verification of model**

The model is verified with the data obtained from field tests.

\[
\Delta z = \frac{3400 \cdot 17.94}{6.6 \cdot 10^6} + \frac{3400 \cdot 12.06}{2 \cdot 6.6 \cdot 10^6} = (9.24 + 3.10) \cdot 10^{-3} = 12.34 \text{mm}
\]

The calculated settlement is 90% relative to the observed settlement. The assumed model can be accepted.

### 6.2.3. Future axial pile bearing capacity

The model described above gives the following formula to determine the pile bearing capacity. This is a simplified version of the formula designed by Prandl.

\[ P = C \cdot l_2^2 \]

The value of constant "C" is determined with the help of field test results. The embedded depth used is an average of the embedded depth of all piles of pile row 1 and 2 (appendix II, p23-26). The influence of the top 1 meter below ground surface level on bearing capacity is neglected.

\[ 3400 = C \cdot (9.5 - 1)^2 \Rightarrow C = \frac{3400}{(9.5 - 1)^2} = 47 \text{kN/m}^2 \]

The final formula is as follows:

\[ P = 47 \cdot l^2 \]

Where:
- \( P \) : the axial pile bearing capacity.
- \( l \) : the embedded depth. The top 1 meter soil is no longer neglected as it does not consist of soft mud but of dense sand.

**The future design bearing capacity:**

\[ P = 1421 \text{kN} \]
6.2.4. Future axial pile stiffness

To determine the future axial stiffness the situation as shown in figure 6.5. is examined.

\[ \Delta z = \frac{P \cdot l_1}{EA} + \frac{P \cdot l_2}{2 \cdot EA} \]

The above-mentioned formulas can be combined. This leads to the following formula:

\[ k = \frac{1}{\frac{l_1}{EA} + \frac{l_2}{2 \cdot EA}} \]

The future axial pile stiffness:

\[ k = 254 \, MN / m \]

The present axial pile stiffness:

\[ k = \frac{P}{\Delta z} = \frac{3400}{0.0137} = 248 \, MN / m \]

Conclusion:
The reduction in embedded depth does not influence the axial pile stiffness significantly.
6.2.5. Future buckling capacity

The plumb pile is assumed to be pinned at the deck and at the sea bottom. This assumption is not realistic but the result will be an underestimation of the real buckling strength. So this assumption leads to safe values.

The model

figure 6.6. model used to calculate axial buckling capacity

Calculations
(Hogeslag, 1995).

\[ l_{\text{buc}} = 24m \]
\[ i = \sqrt{\frac{I}{A}} = 0.26m \]
\[ \lambda = \frac{l_{\text{buc}}}{i} = 91 \]
\[ \lambda_e = 93.9 \]
\[ \lambda_{rel} = \frac{\lambda}{\lambda_e} = 0.97 \]
\[ \omega_{\text{buc}} = 0.6 \]
\[ P_{\text{buc}} = \omega_{\text{buc}} \cdot f_{yd} \cdot A = 4653kN \]

The value for \( \omega_{\text{buc}} \) is found in figure 6.7. regarding curve b.
The future axial buckling capacity

\[ P_{buc} = 4653kN \]
6.3. Future lateral pile capacities

The horizontal loads which act in the direction perpendicular to the longitudinal axis of the jetty will be absorbed by the raker piles. These piles will then be loaded axially. The horizontal loads which act in the direction parallel to the longitudinal axis of the jetty will be absorbed by all piles. The piles will then be loaded laterally.

Considering the abovementioned, it is necessary to assess the influence of reduced embedded depth on the lateral pile capacities.

6.3.1. Theory on pile response to lateral load

To determine the lateral pile capacities the analysis of Broms (Tomlinson, 1994) is adopted. The following assumptions are made in this analysis:

- the active earth-pressure acting on the back of the pile is neglected;
- the distribution of passive soil pressure along the front of the pile is equal to three times the Rankine passive soil pressure;
- the shape of the pile section has no influence on the distribution of ultimate soil pressure or the ultimate lateral resistance
- the full lateral resistance is mobilised at the movement considered.

Two failure mechanisms can be distinguished. For each of the two mechanisms the pile-soil interaction is modelled differently. The two models are:

1. short pile model, in which the soil fails (figure 6.8.a);
2. long pile model, in which the pile fails (figure 6.8.b).
Which model is most realistic is determined by comparing the pile yield moment with the moment generated by the pile-soil interaction. The equations for both models are shown below. The equations are adapted to the fact that the piles are restrained at the top, this is explained in figure 6.9. The model which presents the lowest value for the lateral bearing capacity is chosen the most realistic.
equations for short pile model.

\[ H_u = \frac{\gamma' d \cdot L^3 \cdot K_p}{e + L} \]

equations for long pile model.

\[ f = 0.82 \cdot \sqrt{\frac{H_u}{d \cdot K_p \cdot \gamma'}} \]

\[ H_u = \frac{2 \cdot M_y}{e + \frac{2}{3} \cdot f} \]

Where:
- \( H_u \): the ultimate lateral bearing capacity.
- \( M_y \): the yield moment of the concerning pile.

### 6.3.2. Calculation of lateral pile capacities

The ultimate resistance to lateral load of the pile as described in section 6.1. is calculated in this section. The calculations are performed with the spreadsheet program "Quattro Pro". First \( H_u \) of the raker pile is calculated. The relation between \( H_u \), according to Broms, and the embedded depth is shown in figure 6.10. In order to check the credibility of the Broms-model, the present embedded depth is also included. The calculated value is of the same order as the value obtained from field tests.

Second \( H_u \) of the plumb pile is calculated. The relation between \( H_u \) and the embedded depth is shown in figure 6.11. The credibility can not be checked as no field test data is available. The results are assumed to be reliable.

**Calculation of "**\( H_u \)** of the raker pile**

The systematics as explained in the preceding section are applied here. This leads to the following results as shown in figure 6.10.

The parameters:
- \( \gamma' \) = 11kN/m³
- overall pile length = 30 m
- \( L \) = varies
- \( M_y \) = 1946 kNm
- \( e \) = overall pile length - \( L \)
- \( d \) (diameter) = 1.016 m
Calculation of "$H_u$" of the plumb pile

The systematics as explained in the preceding section are applied here. This leads to the following results as shown in figure 6.11.

The parameters:
- $\gamma'$ = 11 kN/m$^3$
- overall pile length = 30 m
- $L$ = varies
- $M_y$ = 1081 kNm
- $e$ = overall pile length - $L$
- $d$ (diameter) = 0.762 m
Upgrading of iron ore jetty

lateral bearing capacity
pile diameter 762 mm.

figure 6.11. relation between embedded depth and ultimate lateral capacity plumb pile (762 mm)
6.4. Conclusions

The design axial bearing capacity of a plumb pile (762 mm)

\[ P = 1421 \text{ kN} \]

The design resistance to lateral load of a plumb pile (762 mm)

\[ H_u = \text{see figure 6.11} \]
Alternatives for pile upgrading

In the future situation the structural integrity of the piles can be affected in two ways. These are:
1. reduction of bearing capacity as a result of excavation activities;
2. increase of pile load.

This chapter comprises the design of methods to prevent loss of structural integrity. Five methods are explained. One of these methods is chosen and is elaborated more detailed.

Table 7.1. shows the future axial pile loads and the future axial pile capacities, in case no upgrading activities are performed. The future pile loads will not differ significantly from the present pile loads. This means that the principal problem concerning pile integrity is caused by reduction of pile bearing capacity. Bearing this in mind, it will be necessary to deal with the structural integrity of the front two pile rows, i.e. pile row 1 and 2, only.

<table>
<thead>
<tr>
<th>pile row</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>axial load [kN]</td>
<td>1390</td>
<td>2252</td>
<td>2952</td>
<td>3098</td>
<td>4072</td>
<td>1180</td>
</tr>
<tr>
<td>axial capacity [kN]</td>
<td>1421</td>
<td>1421</td>
<td>3400</td>
<td>5600</td>
<td>5600</td>
<td>3400</td>
</tr>
</tbody>
</table>

The pile capacities can be divided in two principal components. These are:
1. the axial bearing capacity;
2. the lateral bearing capacity.

The axial bearing capacity is the component that will be dealt with in this chapter. The reduction of the lateral bearing capacity in the future situation does not cause any problems. This is a result of the following reasons:
- The transverse lateral load is absorbed by the raker piles. The raker piles absorb lateral load by means of axial forces.
- The longitudinal lateral load principally exists of earthquake load. It is absorbed by all piles. As can be seen in figure 6.10, the reduction of embedded depth from 9 m. to 6 m. (the top 1 m. soil layer is neglected, see chapter 6) does not affect the lateral capacity significantly. The earthquake load does not change in the future situation. Given these 2 facts, one can conclude that the longitudinal lateral capacity is sufficient.

The design axial bearing capacity in the future situation can be upgraded according to two fundamentally different methods. These are: preserving the embedded depth with the help of a soil retaining structure or activating, and/or upgrading the pile point bearing capacity.
Preservation of axial pile bearing capacity by means of a soil retaining structure:
1. construction of a sheet pile wall;
2. construction of a soilcrete wall by means of jet grouting.

Preservation of axial pile bearing capacity by means of activating and/or upgrading of the pile point bearing capacity:
3. using the present soil plug in the pile tip;
4. compaction grouting at pile tip;
5. lengthening of the present piles.
7.1. **Alternative 1: a sheet pile wall**

7.1.1. **The concept**

**Preservation of present pile capacity by means of a sheet pile wall**

The sheet pile wall is to be constructed as shown in figure 7.2. The ground anchor can be attached to the raker piles instead of a grout body. This leads to an increase in stiffness of the anchor which will lead to a reduction of lateral movement of the sheet pile wall.

The present bottom level in the direct surroundings of the relevant piles is preserved, by constructing a sheet pile wall. As a consequence the embedded length of the pile and the horizontal soil pressure will be preserved thus preserving the present design pile capacity. There are, however, some restrictions to the extent to which the design capacity can be preserved. These restrictions are caused by the flexibility of the sheet pile wall. The horizontal soil pressures will be reduced at the right side of the sheet pile wall as a result of lateral deformations of the sheet pile wall. One can say that the final design pile capacity is related to the stiffness of the sheet pile wall.

![figure 7.1. geometries of the sheet pile wall](image)

7.1.2. **The sheet pile wall**

The sheet pile wall is often used to retain a soil body. It consists of a vertical steel elements or wooden or concrete planks, which are connected by means of a joint construction. Compared to the retaining wall, sheet pile walls are flexible constructions loaded by flexure. When designing a sheet pile wall one has to take account of its relative flexibility.

A standard sheet pile wall construction is shown in figure 7.2. Theory states that the sheet pile wall has the tendency to move to the left. As a result of this tendency to move to the left the soil pressure at the right side of the sheet pile wall will approach the level of active soil...
pressure, and the soil at the left side will approach the level of passive soil pressure. In order to obtain an equilibrium of forces without very large deformations a ground anchor must also be constructed.

![Cross section of a standard sheet pile wall](image)

**Figure 7.2.** Cross section of a standard sheet pile wall

### 7.1.3. Design of the sheet pile wall

The design of a sheet pile wall is a product of a process of iteration. This process is shown in figure 7.3.

![Design process of sheet pile wall](image)

**Figure 7.3.** Design process of sheet pile wall

**Estimation of geometries and material properties of sheet pile wall**

A first estimation of the geometries of the sheet pile wall is done with the help of a calculation by hand. The assumptions made when calculating forces and pressures by hand are explained in the undermentioned.

To determine the pressures, which are exercised on the wall by the soil, a plastic situation is assumed in which only the soil strengths play a part. The bending stiffness nor the deformation behaviour of the soil have any influence on the value of the soil pressures. It is assumed that the deformations are large enough to activate the full active and passive soil pressure. After having calculated the soil pressures, the system is modelled as a static determined model. One
can calculate the necessary embedment depth, the moments and the lateral displacements, with this model.

With the help of the above-mentioned assumptions a simple formula can be derived which uses as input the material properties of the surrounding soils. The output consists of the dimensions of the sheet pile wall necessary to obtain a state of equilibrium. An example of such a formula is given below.

\[
\left( \frac{d}{h} \right)^2 = \frac{2 \cdot K_a}{K_p} \left( 1 + \frac{d}{h} \right)^2 \cdot \frac{1}{1 + \frac{2}{3} \left( \frac{a}{h} - \frac{3}{3} \cdot \left( \frac{d}{h} \right) \right)}
\]

(Verruijt, 1994)

Where:
- \( d \) : the embedment depth;
- \( h \) : the excavation depth;
- \( a \) : the depth of the anchor.

**Calculation of resulting soil pressures, internal forces and deformations of sheet pile wall**

A numerical model can be used to make a realistic calculation of resulting soil pressures, anchor forces, internal sheet pile wall forces and deformations of the sheet pile wall designed in the preceding design step. A model which is commonly used is the "spring-model" (Heijnen, 1993).

The spring-model schematises the soil as a large amount of independent springs. The sheet pile wall is schematised as an elastic beam with a finite bending stiffness "EI". The springs have a bi-linear load-deformation curve, which exists of a elastic branch and a plastic branch. The orientation of the plastic branch depends on the strength characteristics of the soil, i.e. the active respectively passive soil pressure. The stiffness of the elastic branch is defined by a spring constant called the "bed constant".

A problem in the use of the spring-model is the determination of the bed constant. This constant is not a pure soil property like the internal angle of friction. It is partly determined by the distance over which the soil deforms. The soil constant has to be estimated on basis of experience. A commonly used bi-linear model of the soil behaviour is shown in figure 7.4.
figure 7.4. bi-linear model of soil behaviour (Heijnen, 1993)

Where:

the bed constant : \( \frac{\Delta \rho}{\Delta w} \)
7.1.4. Assessment of pile capacity

The relevant piles derive their bearing capacity from shaft friction, this is shown in chapter 6. Reduction of the horizontal soil pressure, will then cause reduction of bearing capacity. If it is possible to estimate the reduction of the horizontal soil pressure, it is also possible to estimate the reduction in design bearing capacity.

With the help of the spring-model the deformations and displacements of the sheet pile wall can be determined. With this information the reduction of the horizontal soil pressure can be determined with the help of the bi-linear soil response model as shown in figure 7.4. (Heijnen, 1993). The reduction of the design capacity can be estimated, when applying the aforementioned method. The influence of the horizontal soil pressure on the bearing capacity is explained with the help of the below-mentioned formula and figure 7.5.

\[ f_s(x) = K_s \cdot x \cdot \gamma' \cdot \tan \delta = C \cdot x \]  

(Tomlinson, 1994)

\[ Q_p = \int f_s(x) dx \]

Where:

- \( f_s(x) \): the shaft friction at embedment depth \( x \)
- \( K_s \): soil pressure coefficient
- \( \gamma' \): volumetric weight of submerged soil
- \( \delta \): angle of friction between pile and soil
- \( Q_p \): bearing capacity of pile

\[ f_s(x) \]

\[ Q_p \]

\[ x \]

\[ f_s(x) \]

\[ 1 \]

\[ 2 \]

Figure 7.5. shaft friction as a function of embedment depth along a plumb pile

Where:

1. the shaft friction in the present situation.
2. the shaft friction after construction of a sheet pile wall.
7.1.6. Conclusions and recommendations

A list of pros and cons of alternative 1 is given below. Overlooking this list, one can say that alternative 1 is an expensive and inefficient method.

Pros:
- all techniques used are well developed;
- no risk of endangering the overall integrity of the jetty while construction activities are busy;
- no disturbance of other harbour activities occur during upgrading.

Cons:
- to minimise lateral soil displacement it is necessary to use heavy sheet piles. This is costly.
- a ridge is created with which the ship's hull can collide;
- provisions preventing erosion have to be taken;
- heavy equipment is needed.
7.2. Alternative 2: jet grouting

7.2.1. The concept

A soilcrete body is realised by jet grouting with dimensions as shown in figure 7.6. The present bottom level, in the direct surroundings of the relevant piles, can be preserved as a result of jet grouting. This way the embedded depth of the relevant piles is not affected and the horizontal soil pressure is preserved.

Contrary to the sheet pile wall, the grouted soil retaining structure is a solid and extremely stiff structure. It can be modelled as a soil retaining wall. This means that the reduction of horizontal soil pressures will be minimal. Another positive contribution to the design bearing capacity is given by the cohesive properties of the soilcrete body by which the piles are surrounded. These two effects added together prevent reduction of the design bearing capacity, and may even cause an increase of the design bearing capacity.

7.2.2. The process of jet grouting

"Jet grouting" is a general term which can be applied to grouting methods which utilise high pressures (typically 300 to 500 N/mm²) to impart energy to a fluid which is injected into the soil at about 2.3 to 3.3 m/s. The high speed fluid is used to cut, replace, and then mix the native soil with a cementing material, often a water-cement grout. The resulting material is called soilcrete.

At present there are three general forms of jet grouting which involve injection as a single fluid (grout), two fluids (air/grout), or three fluids (air/water/grout). The single, double, and triple fluid systems of jet grouting all require 2 essential construction steps: first a vertical guide hole must be drilled accurately down to the required depth; then jet
grouting follows, usually from the bottom to the top. Panels, columns and other geometric shapes can be formed by varying the rotation of the drill rods. Columns are formed when the drill rods are rotated $360^\circ$ while lifting the drill string. Other geometric shapes can be formed depending on the angle of rotation of the drill string while lifting the rods. The aforementioned is illustrated in figure 7.8.

For this application the three fluid system is chosen because it has the most favourable ratio between column diameter and unconfined compression strength. The column diameters that can be realised in sands vary from 50 to 300 cm. (Kauschinger, 1992)

![Diagram of jet grouting procedure](image)

**figure 7.8.** jet grouting procedure (Ichihashi, 1992)

### 7.2.3. Design of soilcrete body

The dimensions of the soilcrete body can be determined with the same calculation techniques as those used when dimensioning a soil retaining wall. Theory states that the soil pressure at the right side equals the active soil pressure, the soil pressure at the left side equals the passive soil pressure and the friction force along the bottom of the soilcrete body is related to its weight by a friction coefficient. Common design criteria state that if the construction is not to fail it has to fulfil the below mentioned demands:

**Constructive demands commonly set to a soil retaining wall:**
1. resistance against horizontal and vertical displacement → equilibrium of forces;
2. resistance against rotation → equilibrium of moments.

**Provisional stability check**
In this section calculations are performed to check whether a soilcrete body with probable dimensions can meet the stability demands as indicated in the previous section. In order to obtain insight in the combined action of forces figure 7.9. can be studied.
Alternatives for pile upgrading

figure 7.9.  The combined action of forces on a soil retaining wall

Where:

\( \sigma_a \) : the active soil pressure
\( \sigma_p \) : the passive soil pressure
\( F_a \) : the active soil force
\( = \frac{1}{2} \cdot h_1^2 \cdot \gamma_s \cdot K_a = \frac{1}{2} \cdot 5^2 \cdot 11 \cdot 0.3 = 41.25 \text{kN} \)
\( F_p \) : passive soil force
\( = \frac{1}{2} \cdot h_2^2 \cdot \gamma_s \cdot K_p = \frac{1}{2} \cdot 1^2 \cdot 11 \cdot 2.95 = 16 \text{kN} \)
\( F_f \) : friction force between soilcrete and soil
\( = 0.9 \cdot \sigma_v \cdot d \cdot \tan \delta = 0.9 \cdot 55 \cdot 2.5 \cdot 0.7 = 86 \text{kN} \)

**Demand 1:** equilibrium of forces.

\[
\frac{F_a}{F_f + F_p} = \frac{41}{16 + 86} = 0.4 < 1
\]

Conclusion: The horizontal forces are balanced. The vertical forces can safely be regarded as balanced too.

**Demand 2:** equilibrium of moments.

The maximum moment generated by vertical soil pressures:

\[
M_v = \frac{1}{6} \cdot d^2 \cdot \sigma_v = \frac{1}{12} \cdot 2.5^3 \cdot 55 = 29 \text{kNm}
\]
The maximum moment generated horizontal soil pressures:

\[ M_h = \frac{1}{3} \cdot h_1 \cdot F_a = \frac{1}{3} \cdot 5.41 = 68 \, kNm \]

\[ \frac{M_s}{M_h} = \frac{68}{29} = 2.3 > 1 \]

Conclusion:
The moments are not balanced. The base of the soilcrete body has to be broadened. The soilcrete columns are pierced by the plumb piles, which provides an extra stabilising effect.

7.2.4. Execution of jet grouting

It is recommended to limit the amount of soilcrete columns constructed simultaneously. This precaution must be taken for 2 reasons: first to be able to monitor the influence of jet grouting activities on the bearing capacities of the relevant piles, second to prevent affecting the overall structural integrity of the jetty in case of sudden failure of piles during grouting.

The first step must be to drill holes at specific locations through the concrete deck. After this has been done proceedings are as explained in section 7.2.2.

7.2.5. Conclusions and recommendations

A list of pros and cons is given below. Overlooking this one can see that this alternative does not meet the upgrading demands in a satisfactory way. This alternative is therefore deleted.

Pros:
- All techniques used are well developed;
- No heavy equipment is necessary and the soilcrete itself is relatively cheap. This means that the major costs are those of labour;
- No risk of endangering the overall integrity of the jetty while construction activities are busy. This is the result of grouting only few columns at a time;
- No disturbance of other harbour activities occur during upgrading.

Cons:
- It is difficult to predict what will happen to the pile bearing capacities during grouting. It may occur that bearing capacity decreases dramatically;
- The deck has to be penetrated many times;
- It is a relatively expensive method;
- A ridge is created with which the ship's hull can collide;
- Provisions preventing erosion have to be taken.
7.3. **Alternative 3: a soil plug**

7.3.1. **The concept**

When a hollow pile is driven into dense soil a soil plug comes into existence at the lower end of the pile. This soil plug often is compressed in such a way that the pile point can be considered as closed instead of as open. As a consequence, the real ultimate bearing capacity of the pile will exceed the value obtained from field tests. According to Prandl the ultimate bearing capacity becomes (v. Tol, 1995):

\[ P_u = P_{soil} + P_{point} = 1421 + A_{point} \cdot \sigma'_{v} \cdot N_q = 1421 + 0.456 \cdot 55 \cdot 1000 = 26,501 \text{kN} \]

The resulting ultimate bearing capacity is considerable larger than its design load. This leads to the conclusion that non intervention should be sufficient to insure the structural integrity of the jetty!

It is possible that the soil plug has not developed in such a way that it can bear its loads. If this is the case measurements must be taken to increase the strength of the soil plug. Two methods to increase the strength of the soil plug will be discussed below.

By creating a grout/soil body in the pile core with high friction capacity, the ultimate bearing capacity of the concerning piles increases. This is a result of the increase of the pile point surface which leads to an increase of the pile point capacity.

The grout/soil body can be created in two ways. These are:
1. Creating a grout plug on top of the soil plug in the pile core. This is explained in figure 7.9.;
2. Strengthening of the existing soil plug by radial compression as a result of an expanding grout body in the soil plug. This is explained in figure 7.10.

The upgraded pile most probably shows a load-deformation curve which is comparable to that of a bored pile, as the influence of the shaft friction capacity in the future situation is reduced considerably. This phenomenon is explained with the help of figure 7.11. and 7.12.

The load-settlement behaviour of a bored pile compared to that of a driven pile is shown in figure 7.11. The differences in load-deformation behaviour between shaft friction capacity and pile point capacity is shown in figure 7.12.

Though the ultimate bearing capacity of the piles increases considerably after performing the discussed upgrading technique, the load-deformation behaviour will be such that settlements will occur, which are too large. The large settlements significantly diminish the effectiveness of this alternative.

The techniques used for grouting are similar to those used for ground anchors. They are explained in section 7.3.2.
Upgrading of iron ore jetty

**Figure 7.9.** Grout body on top of soil plug

**Figure 7.10.** Grout compression body in a soil plug
figure 7.11. load-deformation curve for 3 pile types (v. Tol, 1995)
7.3.2. The grouting techniques

The grouting technique used in variant 1.
The grout is simply poured into the pile core until the grout column has reached a predetermined level. This level must be such that the friction capacity of the grout plug together with the friction capacity of the soil plug is such that the soil beneath the pile point will fail before the grout/soil plug will fail.

The grouting technique used in variant 2.
A rod with a thick shaft is penetrated into the soil plug; the pipe is closed at its end by means of a loose plug.

The rod is penetrated into the soil by means of driving- or drilling equipment. After the pipe has reached the desired penetration depth the topside of the rod is closed temporarily and connected to a pump. Then a grout is injected into the rod under a pressure of 1 to 2 N/mm². After the rod has been completely filled with grout the rod is gradually retreated, while the continuously pressurised grout fills the thus created extra voids.

The extra radial soil pressures generated by the grout body should create a soil plug with such friction capacity that the soil beneath the pile point will fail before the soil plug will fail. This means that when determining the ultimate pile point capacity one can use the gross pile point surface. This will lead to a considerable increase in pile point bearing capacity.
7.3.3. Conclusions and recommendations

This alternative has few negative aspects. But the aspect mentioned below has such negative consequences that this alternative is deleted.

Pros:
- All techniques used are simple and well developed;
- No heavy equipment is necessary and the material used is relatively cheap. This means that the major costs are those of labour;
- No risk of endangering the overall integrity of the jetty while construction activities are busy;
- No disturbance of other harbour activities occur during upgrading.

Cons:
- The settlements are large. This leads to intolerable high deck deformations.
7.4. Alternative 4: compaction grouting

7.4.1. The concept

The bearing capacity of a foundation pile is composed of shaft friction and point bearing capacity. In the present situation the shaft friction only is activated. This is shown in chapter 6. This means that the piles have a reserve bearing capacity which is not used. The basic idea behind this alternative is to compensate the loss of shaft friction by gaining point bearing capacity.

When activating the point bearing capacity in the present situation, large pile point settlements, relative to the settlements necessary to activate full shaft friction, are necessary to activate sufficient pile point capacity. This is explained in figure 7.12. To overcome this problem compaction grouting can be used.

With compaction grouting an expanding body of grout is installed in the soil, which densifies the surrounding soils when expanding. If this expanding body of grout is installed directly beneath a pile point, one can increase the soil pressures beneath this pile point to the extent that the entire pile is lifted. One can imagine what effect this has on the axial stiffness of the pile. When loading the pile the pile point capacity is immediately being activated. This fact combined with the presence of a soil plug in the pile core creates a large bearing capacity at relative little settlements.

An estimation of the feasible bearing capacities and the matching settlements can be made by regarding the pile to be a driven closed pile. A typical load-settlement diagram for driven closed piles in cohesionless soils is given in figure 7.11. The settlement is given as a percentage of the pile diameter, and the matching capacity is given as a percentage of the ultimate capacity. To be able to make an estimation of where we stand it is necessary to know 3 things:

1. The pile diameter : 762 mm
2. The ultimate pile point capacity (v. Tol, 1995)
   \[ Q_p = N_q \cdot A_p \cdot \sigma'_v = 1000 \cdot 0.456 \cdot 55 = 25000 kN \]
3. The pile design load : 2500 kN

Knowing this the matching settlements can easily be estimated. The following calculations are performed with the help of figure 7.11.

\[ \frac{F_{N,point}}{F_{R,design,point}} = \frac{2500}{25000} \cdot 100\% = 10\% \Rightarrow \frac{z}{D_{pile}} < 0.5\% \Rightarrow z < 4 mm \]

Where:

z : the settlements of the pile point.

This results in an additional settlement compared to the future situation of less than 4 mm.

The above-mentioned situation occurs if the pile is unloaded during grouting, beneath the pile tip. It is possible to further reduce the absolute value of the settlements by loading the pile during grouting of the pile tip. This is explained below.

When the pile is loaded before grouting with, say 1000 kN, settlements occur. Now continued compaction grouting is performed lifting the pile until the pile cap reaches its
original level before it was loaded by 1000 kN. This situation differs essentially from the previously described situation.

When the pile is loaded by 2500 kN this is in fact an extra load of only 1500 kN compared to the original situation. This means that the additional settlements compared to the original situation will be only 60% of the settlements realised in the previous situation! When no load is applied the pile point will feel an upward load of 1000 kN and is pushed upward accordingly. This means that settlements will show a cyclic movement around a mean state. This means that the absolute settlements are reduced by app. 60%, as the settlements in this stage are linear with the loads. To determine the exact values and influence of the above-mentioned practice field tests must be performed. It is useless to try to quantify the effects exactly on the basis of calculations only.

A well designed balance between grouting pressure and preloading of the pile can manipulate the load-settlement characteristics such that they meet the demand made in the future situation. This demand is, an increase of bearing capacity with no significant increase of settlements as compared to the present situation.

The previously mentioned balance between grouting pressure and preloading must be determined empirically with the help of a test pile.

7.4.2. Theory on compaction grouting

Compaction grouting is a fairly new technique (Bowen, 1981), the basic concept is injecting an expanding bulb of highly viscous grout with high internal friction into compactible soil so that, acting as a radial hydraulic jack, it can physically displace the soil particles and move them into a closer spacing, thus achieving controlled densification (fig. 7.13.). Clearly, this process differs from conventional grout infilling of the interstitial openings within a soil mass.

Conventional penetration grouting involves injecting a very fluid grout into the ground in order to fill openings in soils and rocks, primarily to reduce permeability, but also in some cases to impart strength. Compaction grouting is different in that it does not depend upon grout entering such openings, but involves displacement and compaction of soils as a result of the intrusion of a mass of thick grout.

Compaction grouting can be applied in all sorts of sandy soils. It is not considered applicable to saturated clays. The primary uses of compaction grouting are:
1. compacting loose fills or natural loose soils;
2. lifting of structures;
3. support piers by injection of continuous grout columns.
Grout mix

The key to compaction is the silty sand grouting material. Clays add plasticity that may lead to hydrofracture; the clay content should be less than 1% to assure minimal plasticity. Silts are necessary to give the required water holding consistency that allows the grout to be pumped. The silt content is normally limited to 10% to 25% of the sand but can be as high as 35% if the silts are coarse. On occasion it is necessary to add small amounts of fire clay, fly ash, or similar material to make a locally available material pumpable but this must be done very carefully to avoid undesired plasticity. As cement most often three sacks of Portland cement per cubic metre of mix are used which is more than adequate for the purpose (Graf, 1992).

Radius of compaction

Rational analysis and observations indicate that the radius of compaction at each bulb is a function of (Graf, 1992):

1. The restraining pressure of the soil which is:
   - the weight of the inverted cone of soil above the bulb which is directly proportional to the cube of the depth but limited by the greater passive resistance of the soil with depth;
   - the shear strength along the shear surface of the restraining cone of soil which is directly proportional to the square of the depth.
2. The weight of the structure above the grout bulb;
3. The surface area of the gout bulb;
4. The grout pressure at the bulb which is, for practical purposes, the grout pressure developed at the collar at low pumping rates.
Field pumping criteria
Criteria for the completion of pumping at each grout injection is the first observed of the following (Graf, 1992):
1. A peak pressure drop at the collar when pumping at a constant rate, frequently of 350 to 500 kPa. This indicates overcoming of the shear strength of the soil. Observations have shown that surface heave will follow the sharp pressure drop within seconds of continued pumping.
2. Surface heave is a standard criterion.
3. Grout pressure refusal.
4. Maximum quantity of grout is occasionally established for each stage.

One other criteria has been developed that involves pressure relaxation in silty sands. After one of the primary criteria of compaction has been reached, pumping is stopped for at least 3 minutes. Then, that location is pumped again and this cycle repeated until the repumping results in less than 50% of the grout "take" at the first injection. Successful performance of many projects not using this relaxation criterion indicates the procedure is only necessary for extremely critical projects.

Lifting of structures
Controlled lifting of structures with compaction grout can be accomplished under almost all soil conditions. Structures have been lifted more than 0.5 m with compaction grout (Graf, 1992).

Equipment
Mixers are either pug mixers or screw mixers because of the high shear required for the stiff and harsh mix. Plaster mixers are sometimes used.

Pumps are, most often, hydraulic piston pumps modified to achieve grout pressures of 7 to 11 MPa. Many pumps can be controlled including pumping rate and reversing, by remote controls at the grout pipe.

Grout pipes are usually 38 to 76 mm. steel pipes, depending upon the contractor's method. The variance in size has no apparent effect on the results of the grouting. They may be driven, drilled or cemented in place, but they must be tight to the surrounding soil to prevent being pumped out of the soil (Graf, 1992).

Quality control
The following 8 precautions should be taken in mind when quality is demanded (Graf, 1992):
1. Sands used are usually unprocessed flood plain deposits subject to natural variations and the care of the loader operator at the borrow site. Each load should have representative samples at least qualitatively tested by an experienced hand squeeze test and/or a quick test such as a slurry settlement test.
2. Nearly continuous observation of the mix at the mixer with frequent slump tests. If a test indicates that the slump is too high, it must be taken again after the material has been circulated through the pump and hose because this will reduce the slump. The best work is done with the lowest slump possible.
3. Pressure build up and pumping rate must be closely observed and analysed during the pumping. The difference between the peak pressure and the relaxation pressure often indicates the tightening of the soil. Pumping rate should be reduced as the peak/relaxed pressure differences decreases and, especially, when the pressure approaches the "refusal pressure".

4. It is considered good practice to split space the injections. By working against compacted soil on each side, comparing the grout takes indicate the effectiveness of the work.

5. Pressure gauges must be protected by gauge protectors and they must be checked frequently during pumping. The gauge at the pump indicates the correctness of the mix and the pump operation. The gauge at the collar of the injection pipe indicates what the grout is actually doing in the soil.

6. Continuous monitoring for surface lift must accompany all grout pumping. Water levels are frequently used that can be read to 1.5 mm with the reference point being distant from any lifting effect.

7. Hoses, fittings, etc., must be checked because of the pressures used.

8. Cumulative totals of materials used, compared to pump stoke counts and mixer recorders, should be used to verify calibration.

The pros and cons of compaction grouting

Pros:
- during remedial work minimal disturbance of a structure and adjacent ground is required;
- although it is not a cheap method in itself, it is often the most economical method which is feasible for a particular job;
- compaction grouting is a rather flexible technique enabling unexpected conditions encountered during operations to be more easily taken care of than would be the case with almost any other method;
- it enables structures to be raised to grade by what may be called a jacking up process involving tolerances as small as 3 mm;
- it solves problems at the source, i.e. in the actual soil.

Cons:
- it cannot effectively stabilise near-surface soils because the overlying restraint is not large enough;
- it is not effective when utilised near to unsupported slopes;
- there is no easy way in which to measure the effectiveness of compaction grouting. The results are not so easy to interpret as is the case with the results of, say, underpinning;
- the possibility of inadvertently filling underground pipes with grout during grouting operations, e.g. where these may have been ruptured.

7.4.3. The realisation of compaction grout at pile tip

It is recommended to upgrade the piles one by one or few by few in order to be able to precisely monitor the influence of the upgrading and if necessary adjust or stop the treatment.
The upgrading of a pile can be divided in the following seven stages:

1. The pile core is made accessible by drilling a hole through the concrete deck at the centre of the pile. The hole must have a diameter of app. 60 mm. The drilling can be performed with a diamond core drill.

2. A drill rod is pierced through the drilled hole in the deck and is drilled downward until the pile point is reached. The exact location of the pile point can be obtained from available pile driving data, or can be obtained by echo soundings.

3. The drill rod is pulled upwards while simultaneously grout is injected in the drilled shaft with a pressure of 1 to 2 N/mm². This pressure is used for installing grout anchors. By doing this the density of the soil plug is increased thus increasing the ultimate pile point capacity, and a casing is created through which the grout pipe can be drilled.

4. A grouting pipe is drilled through the casing and the pumping of the grout is started.

5. Compaction grouting is continued until:
   - the deck is lifted until the same level is reached when unloaded;
   - grout pressure refusal occurs;
   - ground heave of the sea bottom is observed;
   - grout pressure as calculated further in this paragraph is reached.

   The critical event can be determined with the test pile.

6. The pipe is withdrawn while simultaneously the drilled shaft is filled with grout.

7. The hole in the deck is closed.

The sequence in which the piles are upgraded can best be determined by the contractor. A possible sequence is given in figure 7.14. The sequence is such that minimal bending moments occur in the deck during upgrading.

![Figure 7.14. Upgrading Sequence](image-url)
Problems during execution

**Problem**

The pile plug fails.

**Solution**

Excavate the pile shaft with the help of air lifting. Place the grout pipe in the centre of the pile. Pour concrete in the pile shaft. In this way a solid plug is realised and upgrading activities can be continued as mentioned in the previous section.

**Problem**

The pile tip is buckled such that the grout pipe cannot penetrate.

**Solution**

- 1- A hole can be drilled through the buckled pile wall.
- 2- Compaction grouting is performed around the pile tip as shown in figure 7.15.

![Cross section pile](image)

**figure 7.15.** compaction grouting around pile tip

**Calculation of maximum grout pressure**

A useful grout pressure prediction for compaction grouting is made as follows (Bowen, 1981). Consider a spherical grout mass of radius $a$, having a centre placed at a distance $h$ vertically below a horizontal soil surface. The grout pipe is assumed to be sealed to the adjacent material and then the upper limiting pressure coincides with the inception of a ground heave. In a limiting case a truncated soil cone above the grout source will be disturbed by injection. The potential conical shear failure surface will be inclined at an angle $\theta$ of about 65° to the horizontal ($\theta = 45^\circ + \theta/2$).

Assuming that the grouting pressure is essentially uniform throughout the mass, the radius of which increases with the increase in grouting pressure, the maximal permissible grouting pressure $p_g$ at a certain equivalent radius of the grouted mass can be derived as follows: the upward force exerted can be equated to the total weight of the truncated soil cone together with the downward shearing resistance of soil along the potential surface of failure.

**Calculation:**

When a spherical body expands beneath the soil surface a cone shaped volume of soil is disturbed. This is shown in figure 7.16. With the help of this figure the maximum grout pressure is calculated.
Upgrading of iron ore jetty

Where:
- \( W \): the weight of the soil cone.
- \( T \): the friction at the cone surface.
- \( V \): the volume of the cone.
- \( a \): the radius of the grout bulb.

The maximum grout pressure is:

\[
\rho_g = \frac{W + T}{\pi \cdot a^2}
\]

Determination of \( W \):

\[
W = I \cdot \gamma'
\]

\[
I = \int_{z=0,21}^{z=5,21} \frac{\pi \cdot z^2}{\tan^2 \theta} \, dz = \left[ \frac{\pi \cdot z^3}{3 \cdot \tan^2 \theta} \right]_{z=0,21}^{z=5,21} = 32,2 - 0,0021 = 32,2 \, m^3 \Rightarrow W = 354 \, kN
\]

\( \gamma' = 11 \, kN / m^3 \)

Determination of \( T \):

\[
T = \int_{z=0,21}^{z=5,21} \frac{2 \cdot \pi \cdot z}{\tan \theta} \cdot \tau(z) \, dz
\]

\[
\Rightarrow T = \frac{\pi}{\tan \theta} \cdot \int_{z=0,21}^{z=5,21} 114,6 \cdot z - 22 \cdot z^3 \, dz = \frac{\pi}{\tan \theta} \cdot \left[ 57,3 \cdot z^2 - 7,3 \cdot z^3 \right]_{z=0,21}^{z=5,21} \Rightarrow
\]

\[
\tau(z) = 11(5,21 - z)
\]

\[
\Rightarrow T = 1,46 \cdot (523 - 2,5) = 760 \, kN
\]
The maximum grout pressure $p_g$ thus becomes:

$$p_g = \frac{354 + 760}{\pi \cdot 0.1^2} = 35 N/mm^2$$

In this calculation "a" is chosen arbitrarily. $p_g$ does not vary linearly with the radius. It decreases with increasing radius. But given the extreme high value calculated this will not cause any problems when varying the radius of the grout bulb.

7.4.4. Conclusions and recommendations

A list of pros and cons of the concerning method is given below. Adding all the pros and cons this alternative comes out as a very good and feasible method to upgrade the jetty. As a consequence this method is chosen.

Pros:
- No heavy equipment is necessary and almost no material is added to the jetty. This means that the major costs are those of labour. This can cause a major cost reduction compared to other methods as labour is relatively cheap in Indonesia.
- No risk of endangering the overall integrity of the jetty while construction activities are busy. This is the result of handling only few piles at a time.
- A smooth bottom level course is obtained.
- No disturbance of other harbour activities during upgrading do occur.

Cons:
- It is difficult to predict what exactly will happen during compaction grout injections.
- Problems can arise when a lot of pile tips are heavily buckled. This is a major uncertainty. It is recommended to investigate the probability of buckling occurring previous to executing the upgrading activities.
7.5. Alternative 5: lengthening of piles

7.5.1. The concept

By driving piles, with a smaller diameter than the inner diameter of the relevant piles, through the shaft of these piles and next joining the inner pile with the outer pile, the design bearing capacity of the combined pile can be improved, compared to that of the single pile. The extent to which the present piles are lengthened determines the final bearing capacity. A draft of the principle is shown in figure 7.17.

![Draft of alternative 5](image)

7.5.2. Draft design of alternative 5

The design comes down to 2 important parts. These are:
1. determination of the type of pile used to drive through the shaft of the present piles;
2. determination of the driving depth of the inner pile.

1. Determination of type of pile used
The buckling strength is the main criterion of the pile used.

The provisional pile properties:
- \(A\) : net surface of cross section \(= 0.027\ m\)
- \(I\) : \(= 1.32 \times 10^{-3}\ m^4\)
- \(D\) : outer diameter \(= 635\ mm\)
- \(t\) : wall thickness \(= 14\ mm\)
- \(\lambda_e\) : \(= 93.9\)
The buckling capacity of the pile, "$F_{c;u;d}$":  
When starting with driving the pile is considered pinned at the ends. This means that the buckling length is equal to the pile length. An initial estimate is 28 metres. The coefficients with which one can determine the buckling force, $F_{c;u;d}$, are (Hogeslag, 1993):

$$\lambda = \frac{l_{buc}}{28} = 127$$

$$i = \frac{I}{A} = \frac{0.00132}{0.027} = 0.22m$$

$$\lambda_{rel} = \frac{127}{93.9} = 1.35$$

With the help of figure 6.7. (curve a,b) the following value for $\omega_{buc}$ can be found:

$$\omega_{buc} = 0.5$$

This leads to the following value of the ultimate design capacity, $F_{c;u;d}$:

$$F_{c;u;d} = \omega_{buc} \cdot f_{y;d} \cdot A = 0.5 \cdot 235000 \cdot 0.027 = 3173kN$$

The maximum axial design load, "$F_{c;u;d}$":  
As more accurate data is not available it will be assumed that the maximum axial force in the inner pile is equal to the maximum axial force in the outer pile during driving. This assumption leads to conservative results and is justified as the soil in the pile shaft of the present piles has been compacted during driving which has lead to higher N-values than given in the soil investigation report. The maximum axial force occurring during driving can be calculated with the following formula given by KEC.

$$P_u = \frac{e_f \cdot F \cdot W_h + e^2 \cdot W_p}{S + C/2}$$

Where:

- $P_u$ : the axial pile force during driving = tonnes
- $e_f$ : efficiency of hammer = 0.7
- $F$ : = 2 $W_h \cdot H$ ton-cm
- $W_h$ : weight of ram = 4.5 tonnes
- $W_p$ : weight of pile = 7.8 tonnes
- $H$ : drop height of weight = 300 cm
- $S$ : penetration of pile = 0.21 cm
- $C$ : rebound of pile = $\frac{L \cdot P_u}{E \cdot A}$ cm
- $e$ : reaction coefficient = 0.8

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As $P_U$ is present at both sides of the equation sign the formula cannot be solved directly. Iteration leads to the following result:

$$F_{C_s;d} = 7500 \text{ kN}$$

Conclusion:
Buckling of the pile during driving is very probable. In order to prevent buckling a very heavy pile must be used.

2. Determination of the driving depth
It is difficult to make a prediction of the achieved bearing capacity of the combined pile system after completion of driving activities as the soil strength parameters will have changed considerably. The performance of a pile test is thus recommended.

The recommended driving depth at which the combined pile must be tested is the present driving depth increased by 4 metres.

7.5.3. The execution procedure

The execution procedure exists of the following steps:
1. drilling holes through the concrete deck in line with the concerning piles. Diamond core drills can be used;
2. the soil in the pile shaft is excavated;
3. a piling frame is installed, i.e. a Menck tubular piling frame;
4. a steel pile is placed and driven to the desired depth;
5. grout is poured between the inner and outer pile shafts. This is done to assure a solid connection between the two piles;
6. the hole in the deck is closed.

Possible problems during execution:
1. the present pile points are heavily buckled, i.e. folded up;
2. the inner piles buckle during driving.

No simple solutions to these problems can be presented. The problems mentioned can obstruct the execution!

7.5.4. Conclusions and recommendations

A list of pros and cons is given below. Regarding this list, the concerning alternative is considered unfit. Therefore it is deleted.

Pros:
- The increase in design bearing capacity can be controlled easily by varying the driving depth of the inner piles;
- All techniques used are well developed;
- No risk of endangering the overall structural integrity of the jetty when only few piles are upgraded at the same time;
- No disturbance of other harbour activities occur during upgrading.
Cons:
- The aforementioned execution problems can prevent the completion of this alternative;
- This alternative is material intensive so it will be relatively expensive;
- The holes in the deck are relatively large compared to the pile diameter of the present piles. This can cause local structural integrity problems.
8

Additional mooring/fending points

8.1. Introduction

Problem definition
The future criterion vessel has an average overall length of 315 metres. The present length of section B of the jetty is 285 metres. The consequence of this discrepancy is that there are too few mooring/fending points.

Objectives
The objective of this chapter is to design a construction which solves the problem as mentioned in the problem definition and at the same time meets the design requirements.

Design requirements
A full list of design requirements is given in chapter 2. A summary of the most important requirements regarding this subject are reproduced below.

The principal design requirements:
• the jetty length remains the same;
• the constructive properties of the additional mooring/fending points are to be similar to those of the existing ones, i.e. the load-deformation properties and energy absorption characteristics;
• during construction the other harbour activities are not to be disturbed;
• the design is to be as simple as possible.

8.2. Design of additional mooring/fending points

The problem as well as its solution is visually explained in figure 8.2. Figure 8.1. shows mooring/fending patterns as recommended by the British Standards.

To answer to the future demands regarding mooring and fending, one can lengthen the jetty. This alternative is not considered as it is explicitly excluded in the design requirements. The most common alternative is the construction of dolphins.
8.2.1. Spatial design of dolphins

The British Standards (B.S. 6349) recommend optimum angles of mooring lines for an island berth as shown in figure 8.1. These recommendations lead the spatial design as shown in figure 8.2.

**Figure 8.1.** Mooring patterns conform British Standards

**Figure 8.2.** Spatial design of mooring/fending dolphins
8.2.2. Constructive design of dolphins

The principal constructive demand on the dolphins is that its strength and stiffness characteristics are similar to those of the jetty. The jetty derives its transverse stiffness from the raker piles. As can be seen in chapter 5, 14% of the load acting on one fender or bollard is absorbed by one set of raker piles. This means that the apparent stiffness of one point of fixation of a fender/bollard is 7 times the transverse stiffness of one set of raker piles, i.e. 500,000 kN/m. The strength of the dolphins must be such that it can absorb a load of 2460 kN.

Stiffness check
To obtain dolphins with a transverse stiffness of 500,000 kN/m, which are constructed with the same pile types as used in the jetty, 7 sets of raker piles will be necessary. This is not realistic. A solution can be found in 2 directions: firstly the use of heavier pile types, secondly accepting the lesser stiffness. When accepting the lesser stiffness an impression of the consequences has to be made. This is done in the following calculations.

- transverse displacement of jetty:
  \[ u = \frac{F}{k} = \frac{2460}{500,000} = 4.92 \cdot 10^{-3} \approx 5\text{mm} \]

- transverse displacement of dolphin consisting of two sets of raker piles:
  \[ u = \frac{F}{k} = \frac{2460}{160,000} = 15.38 \cdot 10^{-3} \approx 15\text{mm} \]

The difference in displacement is only 1 cm when loaded maximally. This is considered acceptable.

Strength check
When the dolphin is supported by 2 sets of raker piles the axial pile loads in tension and compression maximally become approx. 1900 kN. This is acceptable.

A dolphin design which is based on two sets of raker piles can answer all demands put to it.
Upgrading of iron ore jetty

The structure chosen is shown in figure 8.3. and figure 8.4.

Figure 8.3. Cross section of dolphin
**Stiffness and strength in longitudinal direction**

The dolphins derive their stiffness and strength in the longitudinal direction from a solid steel catwalk which connects the dolphins to the deck structure. As a consequence, the stiffness and strength characteristics in longitudinal direction are the same as those of the jetty.
Upgrading of iron ore jetty
Conclusions and recommendations

9.1. Conclusions

Section B of the iron ore jetty, presently suited to accommodate dry bulk carriers up to 70,000 D.W.T., can be upgraded, to accommodate dry bulk carriers up to 150,000 D.W.T., by simple and limited interventions. The interventions, as designed and discussed in this report, consist of the following two items:
1. construction of three dolphins;
2. upgrading of axial bearing capacity of piles of pile row 1 and 2 by compaction grouting at the pile tips.

9.2. Recommendations

It is recommended to do extensive research on the influence of compaction grouting on the bearing capacities of the concerning pile. The type of research is explained in section 7.4.

Also, it is recommended to adapt the berthing proceedings. This is a consequence of future limitations to the mooring alignments, as explained in chapter 4.
Literature


Frequency chart for wave heights and periods of Cigading Harbour 1977-1978.


Loading test data new 70,000 DWT pier, Axial Compression and Tension Load (Platform 1, pile 762 x 14 mm), obtained from KEC project files, januari 1990.

Loading test data new 70,000 DWT pier, Axial Compression, Axial Tension and Horizontal Load (Platform 2, pile 1016 x 14 mm), obtained from KEC project files, april 1990.


Pile driving records of new 70,000 DWT pier, obtained from KEC project files, 1991.


Spijkers, J. e.a., Dynamica van Constructies. Delft: Faculty of Civil Engineering, 1991.


figure 2.1.  location of jetty
Figure 2.3. Plan of jetty
Upgrading of iron ore jetty
Appendices I-V

Mei 1996

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Upgrading of iron ore jetty
I

General data

1. Design loads

1.1. Dead loads

- Self weight of jetty: determined in appendix V
- Conveyor: 7.5 kN/m'
- Unloader: see section 1.5.

1.2. Live loads

- Deck (UDL): 30 kN/m²
- Conveyor: 3.5 kN/m
- Unloader: see section 1.5.

1.3. Environmental loads

- Wind
  - operational: 125 N/m²
  - storm: 400 N/m²
- Wave: negligible
- Tsunami: 2.1 m/s
- Current: see section 1.6
- Earthquake: see section 1.6.

1.4. Vessel loads

The design vessel parameters are given in section 1.4.3. The loadings induced on the jetty by fenders and bollards are derived from the following criteria:
1.4.1. Fender loads

- Berthing energy
  - normal: determined from the ship characteristics according to the Bridgestone Design Manual.
  - abnormal: 2 times the normal berthing energy.
- Environmental
  - wind: see section 1.3.
  - wave: determined in appendix III.
  - current: 2,1 m/s with minimum angle of 5°.

Vertical and horizontal friction loads between the fender and ship are to be considered. Loading from a minimum of two fenders simultaneously is to be considered.

1.4.2. Bollard loads

- Environmental
  - wind: determined in appendix III.
  - current: determined in appendix III.

1.4.3. Design vessel characteristics

The design vessel is a 150.000 DWT ore carrier. According to the Bridgestone Design Manual it has the following characteristics.

- Type of ship: ore carrier
- Dead weight tonnage: 150.000 t
- Loaded displacement tonnage: 200.000 t
- Length between perpendiculars (LBP): 300 m
- Width: 45 m
- Depth: 25 m
- Full draught: 16 m
- Design berthing velocity: 0,15 m/s

1.5. Unloader loads

The unloaders are delivered by Babcock Contractors Limited. It is a gantry type grab unloader for iron ore and coal. The unloader is supported by four legs, of which two end on the seaside rail and two end on the backside rail. The vertical unloader loads given by Babcox are presented in table I.1. These loads are given under different circumstances. As the horizontal loads are not given by Babcox, they are derived from the vertical legloads, and calculated in appendix III.

Dead load of unloader: 5050 kN
Appendix 1

<table>
<thead>
<tr>
<th>Table I.1. Vertical unloader leg loads in kN.</th>
<th>Maximum outreach</th>
<th>Maximum backreach</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Waterside</td>
<td>Landside</td>
</tr>
<tr>
<td>Static working condition excluding wind</td>
<td>1780</td>
<td>1070</td>
</tr>
<tr>
<td>Static working condition including wind</td>
<td>1850</td>
<td>1000</td>
</tr>
<tr>
<td>Dynamic working condition excluding wind</td>
<td>1910</td>
<td>960</td>
</tr>
<tr>
<td>Dynamic working condition including wind</td>
<td>1980</td>
<td>1130</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wind on front</th>
<th>Wind on back</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static boom raised excluding wind</td>
<td>1050</td>
</tr>
<tr>
<td>Static boom raised including working wind</td>
<td>1390</td>
</tr>
<tr>
<td>Static boom raised including storm wind</td>
<td>790</td>
</tr>
</tbody>
</table>

### 1.6. Earthquake loads

The earthquake loads are calculated according to the Indonesian Earthquake Code. It is beyond the scope of this section to explain the principles on which this code is based. The essential results obtained when applying this code are explained below.

The equivalent statical horizontal force to which the jetty is to be subjected depends on its resonance frequencies. It can vary from 5.5% to 11% of the dynamic mass of the structure. The right percentage is determined with the help of figure I.1.

The following two loading cases can be distinguished:
1. \( x + \frac{2}{3}y + \frac{1}{2}z \)
2. \( \frac{2}{3}x + y + \frac{1}{2}z \)

Where:
- \( x \) : earthquake load in the direction of the longitudinal axis of the jetty.
- \( y \) : earthquake load in the direction of the transversal axis of the jetty.
- \( z \) : earthquake load in the vertical direction.
Upgrading of iron ore jetty

Design Spectra

Rej. 2.02 of Central a Indonesian Earthquake Code - 1996

Notes

1. Individual modal responses for any axis to be combined by the Complete Quadratic Combination Method (CQC)

2. Sufficient modes are to be considered to obtained a participating mass of at least 95%.

3. The results of the analysis about each axis are to be combined as follows

   and the worst member forces and deflections used as design loads.

   \[
   Z = a) \quad x + \frac{2}{3}y + \frac{4}{3}z \\
   b) \quad \frac{2}{3}x + y + \frac{4}{3}z
   \]

Figure I.1. Earthquake response graph according to the Indonesian Earthquake Code
2. Load Combinations

The following five load combinations given by KEC are considered.
1. the berth working under normal conditions;
2. a ship berthing at design speed;
3. a storm wind;
4. a ship berthing abnormally;
5. an earthquake.

Each load combination consists of a series of load types. The make-up of each load combination is shown below.

The berth working under normal conditions
- dead load
- live load
- unloader working
- working wind load
- current load
- temperature load
- environmental fender load

A ship berthing at design speed
- dead load
- live load
- unloader standing
- current
- temperature load
- normal fender load

A storm wind
- dead load
- live load
- unloader in storm wind
- storm wind load
- wave load
- current load
- temperature load

A ship berthing abnormally
- dead load
- live load (50%)
- unloader standing
- current load
- temperature load
- abnormal fender load
An earthquake.
• dead load
• live load (50%)
• unloader working
• current load
• temperature load
• earthquake load
• environmental fender load
• bollard load

3. Material Properties

3.1. Reinforced concrete

\[ F_{cu} = 40 \text{ N/mm}^2 \]
\[ E_c = 28 \text{ kN/mm}^2 \text{ instantaneous} \]
\[ \text{density} = 24 \text{ kN/m}^3 \]

3.2. Reinforcement

Plain round bars:
\[ f_y = 235 \text{ N/mm}^2 \]
\[ E_s = 200 \text{ kN/m}^2 \]

Type 2 deformed bars:
\[ f_y = 294 \text{ N/mm}^2 \]
\[ E_s = 200 \text{ kN/m}^2 \]

The material stresses must comply with BS 8110 Part 1. For the load case 4 the material factor \( \gamma_m \) may be multiplied by 0.87. The crack widths in the concrete must be limited to 0.3 mm. The cover to the reinforcement is to be 75 mm.

3.3. Piles

<table>
<thead>
<tr>
<th>table 1.2.</th>
<th>pile characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>pile diameter [mm]</td>
<td>762</td>
</tr>
<tr>
<td>wall thickness [mm]</td>
<td>14</td>
</tr>
<tr>
<td>( f_y ) [N/mm²]</td>
<td>145</td>
</tr>
<tr>
<td>( A_{net} ) [m²]</td>
<td>0.033</td>
</tr>
<tr>
<td>( A_{min} ) [m²]</td>
<td>0.46</td>
</tr>
<tr>
<td>( I ) [m⁴]</td>
<td>2.3·10⁻³</td>
</tr>
<tr>
<td>( W ) [m³]</td>
<td>4.6·10⁻³</td>
</tr>
<tr>
<td>( \gamma_s )</td>
<td>78.5</td>
</tr>
</tbody>
</table>
The material stresses are to be the lesser of:

- The requirements of BS 449. For the load case 4, stresses may be taken up to 90% of yield/buckling stresses.
- The limitations of the soil capacity.
- The driving stresses.

No allowance for corrosion is to be made as cathodic protection is present.
APPENDIX II
COLLECTION AND EVALUATION OF EXISTING SOIL DATA
# Contents

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    1.2.2. Laboratory tests ....................................................................................... 2
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Upgrading of iron ore jetty
Collection and evaluation of existing soil data

1.1. Introduction

The soil investigations which are being discussed in these chapters have been performed by PT. PETROSOL by order of the Krakatau Engineering Corporation. The purpose of these investigations is to obtain the necessary soil data for upgrading the existing jetty which is the subject of this essay.

The investigation program consists of the following elements:
1. standard penetration test;
2. laboratory tests;
3. bathymetric, echo sounding and hydroseismic survey;
4. pile loading tests

1.2. Description of the field work

1.2.1. Standard penetration test

The boring of the three test pits was done by means of the "Continuous Coring" method using 73 mm outer diameter single tube core barrel. Double steel casings of diameter 115 mm and 89 mm were used to case the borings. Drillings were done on fixed platform constructed from wooden and bamboo piles.

The Standard Penetration Tests were performed utilising a split spoon sampler with 2 inches (=50.8 mm) outer diameter times 3/8 inch (=9.525 mm) inner diameter and a 140 lbs (=87.18 kN) hammer with 30 inches (=762 mm) falling height. The results of the SPT-tests are presented in the boring logs. The soils are classified in accordance with the Unified Soil Classification System (USCS).

An explanation of the SPT-test.
The output of the SPT tests is used for the purposes mentioned below:

- To obtain disturbed samples for laboratory identification and index property tests.
- To determine the relative density, the internal angle of friction of the soil or estimate directly the bearing capacity of a foundation pile by blow count.

The blow counts "N" are measured for an 18 inch (= 450 mm) penetration of the samples. The blows required for the first 6 inches (= 150 mm) are neglected because this record may be in highly disturbed and slumped material. The blow counts for the last two 6 inch penetration are then added together to obtain the N-value in blows per foot (= 300 mm).
The results of the SPT-tests are questionable as is shown by several investigators. The actual delivered energy to the rods can vary between 30% and 80% of the theoretical value with an average of 55%.

A correction is required for depths in the SPT values because of the greater confinement caused by increasing overburden pressure. The thus increasing N-values due to confinement may indicate larger density than the actual. A correction factor leads to fairly consistent results.

\[ C_n = 0.77 \times 10^{\frac{20}{\sigma'_v}}; \sigma'_v \geq 0.25\text{tsf} \]

where: \( \sigma'_v \) : the effective overburden pressure in tons per square feet.

1.2.2. Laboratory tests

The disturbed samples taken from the bore holes were examined in a laboratory. The test results are mainly to be used for evaluating the value of the dredged soil as fill material. As such these results are of secondary importance to the subject of this thesis. The results will be reproduced as an appendix without commentary.

1.2.3. Bathymetric, echo sounding and hydroseismic report

The surveys which have been performed cover a large area around the jetty. The relevant information from this survey has been extracted and is reproduced in the following pages.

The seismic results have been correlated to the information obtained from the bore holes. The main conclusion that can be drawn from the seismic records is the fact that the soil has a very homogenous layered structure. So the soil properties of the different soil layers found in the bore holes can be extrapolated safely to the intermediate soil.

1.2.4. Pile loading tests

The pile loading tests have been performed at two locations which are shown in figure II.9. At one location a hollow circular steel pile with a diameter of 762 mm and a wall thickness of 14 mm has been tested. This type of pile is used in the jetty as a vertical foundation pile.

At the other location a hollow circular steel pile with a diameter of 1016 mm and a wall thickness of 14 mm has been tested. This type of pile is used as a raker pile.

The piles have been tested for axial pullout and compression loads. The test method performed for both types of loading is called the "Slow Maintained Load Test Method". This test method is considered as the ASTM Standard Test method and is generally used for site investigations prior to installing contract piles and writing specifications. The raker pile type has also been tested on lateral loads. No standard testing procedure has been used.

No pile type has been loaded in any way to failure. The pile load was restricted in such a way that the pile-soil response remained elastic for every loading type. As such it is impossible to determine the ultimate load for each loading type on the basis of the test results.

The test results and drawings of the testing equipment and instruments are reproduced in section 1.3.
1.3. Results

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- figure II.1. location map of boring holes.
- figure II.2. boring log of borehole 1.
- figure II.3. boring log of borehole 2.
- figure II.4. boring log of borehole 3.
PELLET HARBOR UPGRADING PROJECT
PT. KRAKATAU STEEL, CILEGON

DERMAGA PELLET LAMA

DERMAGA PELLET BARU

BH.1

BH.3

BH.2

285 m

Note:

- Boring Point

LOCATION MAP OF BORING POINTS

Not to scale
Appendix II

PROJECT/CLIENT: K. E. C.
LOCATION: Oemogo Pellet Cigoding.
BORE HOLE NO.: B. 1
DEEP: 22.00 m.
MATERIAL: 15.00 m.
DATE: Sept. 7 to Sept. 8, 1994.

BORING METHOD: Core Sampling
CORING METHOD: Thin-walled Geotube
SAMPLING METHOD: Thin-walled Geotube

ROCK/SOIL DESCRIPTION:

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>SPT - N value</th>
<th>Density (g/cm³)</th>
<th>Cohesion (kPa)</th>
<th>Friction Angle (°)</th>
<th>Recovery</th>
</tr>
</thead>
<tbody>
<tr>
<td>20.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

END OF THIS BORING.
CASING INSTALLED TO 22.00 METERS DEPTH.

BORING LOG - PT. PETROSUL
Upgrading of iron ore jetty

**BORING LOG**

- **SPT**: Automatic Hammer
- **DRILLER**: Yoson.
- **LOGGER**: Vahyu V.
- **RECORDED BY**: Tanny

**PROJECT/CLIENT**: K. E. C.

**LOCATION**: Dermago Pellet Cigading.

**BORE HOLE NO.**: B. 2

**DEPTH**: 30.45 m.

**WATER TABLE**: * 12.00 *

**DATE**: Sept, 14 to Sept, 18, 1994.

**COORDINATE**: X: -94532.057 Y: -655501.483 Z: -14.64 m LVS.

**BORING METHOD**: CORING & SAMPLING

**SAMPLING METHOD**: Thin Walled (Shelby) Tube

**DEPTH (m)** | **SPT - N value** | **BLOWS PER CM** | **N PER FOOT** | **RECORD**
--- | --- | --- | --- | ---
0.00 | | | | |
1.00 | | | | |
2.00 | | | | |
3.00 | | | | |
4.00 | | | | |
5.00 | | | | |
6.00 | | | | |
7.00 | | | | |
8.00 | | | | |
9.00 | | | | |
10.00 | | | | |
11.00 | | | | |
12.00 | | | | |
13.00 | | | | |
14.00 | | | | |
15.00 | | | | |
16.00 | | | | |
17.00 | | | | |
18.00 | | | | |
19.00 | | | | |
20.00 | | | | |
21.00 | | | | |
22.00 | | | | |
23.00 | | | | |
24.00 | | | | |
25.00 | | | | |
26.00 | | | | |
27.00 | | | | |
28.00 | | | | |
29.00 | | | | |
30.00 | | | | |

**ROCK/SOIL DESCRIPTION**

- **SANDY SILT**: yellowish grey, few shell and coral gravels, loose.

- **TUFFACEOUS SILTY SAND**: yellowish grey, fine to coarse grained sand, very dense, weakly cemented, core broken by drilling.

**END OF THIS BORING.**

*figure II.3.* boring log of bore hole 2

6
Appendix II

PROJECT/CLIENT  K. E. C.
LOCATION  Derawga Pellet Cigading.
BORE HOLE NO.  6.3
DEPTH  30.45 m.
WATER TABLE  +19.00 m.
COORDINATE  X: -94764.551 Y: -665500.922
Z: -14.76 m LWS.
BORING METHOD  CORING & SAMPLING
SAMPLING METHOD  Thin Walled (Shelby) Tube

BORING LOG  -  PT. PETROSOL

SPIT  Automatic hammer
DRILLER  Yogan.
LOGGER  Kohyu N.
RECORDED BY  Tanny

ROCK/SOIL DESCRIPTION

<table>
<thead>
<tr>
<th>DEPTH (m)</th>
<th>USERS CHART</th>
<th>SYMBOL</th>
<th>ROCK/SOIL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.00</td>
<td>MH</td>
<td></td>
<td>SANDY SILT, yellowish grey, few shell and coral gravels, loose.</td>
</tr>
<tr>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.00</td>
<td>SM</td>
<td></td>
<td>SILTY SAND, light brown, fine to coarse grained sand, tuffaceous, very dense.</td>
</tr>
<tr>
<td>22.00</td>
<td>SM</td>
<td></td>
<td>TUFFACEOUS Silt SAND, light grey, fine to coarse grained sand, tuffaceous, very dense, weakly cemented, core broken by drilling.</td>
</tr>
<tr>
<td>27.00</td>
<td>CH</td>
<td></td>
<td>TUFFACEOUS CLAY, light yellow, few silt, very stiff to dense.</td>
</tr>
<tr>
<td>30.45</td>
<td></td>
<td></td>
<td>END OF THIS BORING.</td>
</tr>
</tbody>
</table>

SPT - N value

<table>
<thead>
<tr>
<th>BLOW/SQ FT</th>
<th>N PER FOOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.15 50/30</td>
<td></td>
</tr>
<tr>
<td>4.15 50/30</td>
<td></td>
</tr>
<tr>
<td>6.15 50/30</td>
<td></td>
</tr>
<tr>
<td>8.15 49/30</td>
<td></td>
</tr>
<tr>
<td>10.15 77/30</td>
<td></td>
</tr>
<tr>
<td>12.15 90/25</td>
<td></td>
</tr>
<tr>
<td>14.15 90/26</td>
<td></td>
</tr>
<tr>
<td>16.00 50/06</td>
<td></td>
</tr>
<tr>
<td>16.00 50/15</td>
<td></td>
</tr>
<tr>
<td>20.00 50/12</td>
<td></td>
</tr>
<tr>
<td>22.00 50/13</td>
<td></td>
</tr>
<tr>
<td>24.00 50/15</td>
<td></td>
</tr>
<tr>
<td>26.00 50/14</td>
<td></td>
</tr>
<tr>
<td>28.15 29/30</td>
<td></td>
</tr>
<tr>
<td>29.15 43/30</td>
<td></td>
</tr>
</tbody>
</table>

figure II.4.  boring log of bore hole 3
1.3.3. Results of the hydroseismic survey

A short resume of the results is reproduced below. The hydroseismic records were related to the bore hole information. The depths are given in meters with respect to the seabed level.

- Bore hole no. 1

<table>
<thead>
<tr>
<th>depth</th>
<th>soil type</th>
<th>SPT-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00-1.00</td>
<td>silty clay, locally are found fine gravel and sand</td>
<td>13-18</td>
</tr>
<tr>
<td>1.00-8.00</td>
<td>medium dense silty sand</td>
<td>&gt;100</td>
</tr>
<tr>
<td>8.00-22.00</td>
<td>very dense tuffaceous sand</td>
<td></td>
</tr>
</tbody>
</table>

- Bore hole no. 2

<table>
<thead>
<tr>
<th>depth</th>
<th>soil type</th>
<th>SPT-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00-1.00</td>
<td>silty clay, locally are found fine gravel and sand</td>
<td></td>
</tr>
<tr>
<td>1.00-6.00</td>
<td>dense to very dense silty sand</td>
<td>48-80</td>
</tr>
<tr>
<td>6.00-24.00</td>
<td>very dense tuffaceous silty sand</td>
<td>&gt;100</td>
</tr>
</tbody>
</table>

- Bore hole no. 3

<table>
<thead>
<tr>
<th>depth</th>
<th>soil type</th>
<th>SPT-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00-1.00</td>
<td>silty clay, locally are found fine gravel and sand</td>
<td></td>
</tr>
<tr>
<td>1.00-8.00</td>
<td>dense silty sand</td>
<td>50</td>
</tr>
<tr>
<td>8.00-26.00</td>
<td>very dense tuffaceous sand</td>
<td>&gt;100</td>
</tr>
<tr>
<td>27.00-30.00</td>
<td>very stiff to dense tuffaceous clay</td>
<td>30-40</td>
</tr>
</tbody>
</table>

1.3.4. Laboratory test results

Contents:
- figure II.5. grain size analysis on bore hole 1 & 2.
- figure II.6. grain size analysis on bore hole 3.
- figure II.7. standard proctor compaction curves.
- figure II.8. standard proctor compaction curves.
Appendix II

PROJECT : Upgrading Dermaga Pellet
70,000 DWT ke 150,000 DWT
LOCATION : Pellet Harbor PT. K.S.
POINT NO : BH - 1 & BH - 2

DATE : Sept 22, 1994
TESTED BY : Shd
RECORDED BY : nick
CHECKED BY : NR

<table>
<thead>
<tr>
<th>NO</th>
<th>Point No.</th>
<th>Gravel</th>
<th>Sand</th>
<th>Silt</th>
<th>Clay</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>Cu</th>
<th>Cc</th>
<th>% weight &lt; #200</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1. (2.00 - 2.45 m)</td>
<td>5</td>
<td>54</td>
<td>41</td>
<td>0.32</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>####### 41</td>
</tr>
<tr>
<td>2</td>
<td>1. (4.00 - 4.45 m)</td>
<td>4</td>
<td>67</td>
<td>29</td>
<td>0.44</td>
<td>0.0075</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>####### 29</td>
</tr>
<tr>
<td>3</td>
<td>1. (6.00 - 6.45 m)</td>
<td>3</td>
<td>65</td>
<td>32</td>
<td>0.40</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>####### 32</td>
</tr>
</tbody>
</table>

**Grain Size Analysis of Borehole 1 & 2**

Figure II.5. Grain size analysis of boreholes 1 & 2
Upgrading of iron ore jetty

PROJECT: Upgrading Dermaga Pellet
LOCATION: Pellet Harbor PT. K.S.
POINT NO.: BH - 3

DATE: Sept 22, 1994
TESTED BY: Shd
RECORDED BY: nick
CHECKED BY: NR

<table>
<thead>
<tr>
<th>NO</th>
<th>Point No.</th>
<th>Gravel</th>
<th>Sand</th>
<th>Silt</th>
<th>Clay</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>Cu</th>
<th>Cc</th>
<th>% weight&lt;200</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3. (2.00 - 2.45 m)</td>
<td>0</td>
<td>52</td>
<td>48</td>
<td>0.16</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>3. (4.00 - 4.45 m)</td>
<td>4</td>
<td>62</td>
<td>34</td>
<td>0.30</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>3. (6.00 - 6.45 m)</td>
<td>3</td>
<td>61</td>
<td>36</td>
<td>0.30</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>3. (8.00 - 8.45 m)</td>
<td>2</td>
<td>64</td>
<td>34</td>
<td>0.2800</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>3. (10.00 - 10.45 m)</td>
<td>1</td>
<td>62</td>
<td>37</td>
<td>0.2600</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

figure II.6. grain size analysis of borehole 3
MODIFIED PROCTOR COMPACTION CURVES

PROJECT: Upgrading Dermaga Pellet
LOCATION: Pellet Harbor PT. K.S.
POINT: Mix Samples (1,2,3)
DATE: September 23, 1994
TESTED BY: Skm
RECORDED BY: Nick
CHECKED BY: NR

DEPTI IN METER:
SOIL DESCRIPTION: Silty Sand, grey.

<table>
<thead>
<tr>
<th>Volume of Cylinder (cc):</th>
<th>843</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of Rammer (lbs):</td>
<td>10</td>
</tr>
<tr>
<td>Height of Drop (in):</td>
<td>18</td>
</tr>
<tr>
<td>Blows per layer:</td>
<td>25</td>
</tr>
<tr>
<td>Number of Layers:</td>
<td>6</td>
</tr>
</tbody>
</table>

Natural Water Content (a): |
Amount Retained on 3/4" Sieve (a): 0
Specific Gravity: |
Liquid Limit (a): -
Plastic Limit (a): NON PLASTIC
Plasticity Index (a): |

MAXIMUM DRY DENSITY IS 1.62 g/cc AT WATER CONTENT 15.2%

figure II.7. standard proctor compaction curves
Upgrading of iron ore jetty

STANDARD PROCTOR COMPACTION CURVES

PROJECT: Upgrading Dermaga Pellet 70.000 DWT ke 150.000 DWT
LOCATION: Pellet Harbor PT. K.S.
POINT: Mix Samples (1, 2, 3)
DEPTH IN METER: 
SOIL DESCRIPTION: Silty Sand, grey.

<table>
<thead>
<tr>
<th>Date</th>
<th>Tested By</th>
<th>Recorded By</th>
<th>Checked By</th>
</tr>
</thead>
<tbody>
<tr>
<td>September 23, 1994</td>
<td>Skro</td>
<td>Nick</td>
<td>NR</td>
</tr>
</tbody>
</table>

Natural Water Content (%): 
Annual Retained on 3/4" Sieve (%): 0
Specific Gravity:
Liquid Limit (%): 
Plastic Limit (%): NON PLASTIC
Plasticity Index (%): 

MAXIMUM DRY DENSITY IS 1.463 g/cc AT WATER CONTENT 20.4%

![Compaction Curves Graph](image)

**Figure II.8.** Standard Proctor compaction curves
1.3.5. Pile loading test results

Contents:
- figure II.9. Map of location of pile tests.
- figure II.10. Pictures of the testing installation for piles with a diameter of 762 mm.
- figure II.11. & II.12. Pictures of the testing installation for piles with a diameter of 1016 mm.
- figure II.13. Results of the axial compression test for a pile with a diameter of 762 mm.
- figure II.14. Results of the axial compression test for a pile with a diameter of 1016 mm.
- figure II.15. Results of the pullout test for a pile with a diameter of 762 mm.
- figure II.16. Results of the pullout test for a pile with a diameter of 762 mm.
- figure II.17. Results of the lateral loading test for a pile with a diameter of 1016 mm.
- figure II.18. Blow counts on all piles of axis A.
- figure II.19. Blow counts of all piles of axis B.
- figure II.20. Blow counts of all piles of axis D.
- figure II.21. Blow counts of all piles of axis E.
Upgrading of iron ore jetty

figure II.9. map of location of pile tests
testing installation for pile with diameter of 792 mm
Upgrading of iron ore jetty

**Figure II.11.** Testing installation for pile with diameter of 1016 mm
Figure II.12. Testing installation for pile with diameter of 1016 mm.
Figure II.13. Results of axial compression test, 762 mm
Figure II.14. Results of axial compression test. 1016 mm
figure II.15. results of axial tension test, 762 mm
figure II.16. results of axial tension test, 1016 mm
Upgrading of iron ore jetty

Figure II.17. Results of lateral loading test, 1016 mm.
Figure II.18. Blow counts of all piles of axis A.
figure II.19. blow counts of all piles of axis B
Appendix II

Figure II.20. Blow counts of all piles of axis D
Upgrading of iron ore jetty

Pile and blowcount depth [m Klockner]

Axis E

Figure II.21. Blow counts of all piles of axis E
1.4. Conclusive Soil Parameters

The purpose of this section is to translate the results from the site survey into simple soil parameters with which provisional computations to determine the future bearing capacity of the foundation piles can be performed. For a cohesionless soil two parameters are of prime importance, these are:

1. The unit weight of submerged water.
2. The angle of internal friction, $\phi$.

1.4.1. The determination of design parameters

1. The unit weight of the soil above water.

General soil data:
- $e$: void ratio
- $n$: porosity
- $D_r$: relative density
- $\rho_s$: unit density of sand
- $\rho_w$: unit density of water
- $\gamma_{wet}$: unit weight of saturated soil $[kN/m^3]$
- $\gamma$: unit weight of submerged soil $[kN/m^3]$

$$\gamma = n \cdot \rho_w + (1-n) \cdot \rho_s$$

Based on the performed investigations one can say that the soil in the area surrounding the jetty can be divided in three layers.

- layer 1: Marine mud (silt, clay, sand and gravel). This layer extends to approximate 1 meter below the seabed.
- layer 2: Medium dense to dense silty sand with an SPT-value of 13 to 80. This layer extends from 1 meter to approximate 8 meters below the seabed.
- layer 3: Very dense tuffaceous silty sand with an SPT-value greater than 100. This layer extends from 8 meters to an undetermined depth below the seabed.

The soil properties for the above mentioned soil layers are given in the following tables. The depth given is the distance beneath the bottom level as indicated in the boring logs.
2. The angle of internal friction, φ.

The angle of internal friction can be deduced with the help of the above mentioned information and figure AI.... This leads to the following results.

- borehole 1
  - φ
  - layer 1: 30 - 40
  - layer 2: > 40

- borehole 2
  - φ
  - layer 1: > 40
  - layer 2: > 40

- borehole 3
  - φ
  - layer 1: > 40
1.4.2. The Design Soil Profile

The presented results from the survey performed by PT. PETROSOL can be summarised by the soil profile in figure II.22. This is quite a rude approximation of the real situation but sufficient to be used when making a first estimation of pile bearing capacities in the future situation. One remark however must be made and that concerns the fact that in borehole 3 a clay layer is discovered between 27 meters and 30 meters (the end of the bore hole). This has been confirmed by the seismic reports. The extent of the clay layer in depth is not known. Given the overall soil structure and the fact that the existing harbour structure did not encounter any bearing problems up until now, the importance of the clay layer can be considered as of secondary importance.

However problems can arise if it is thought to be necessary to lower the foundation pile points to such a depth that the pile point and shaft carries down its load in the clay layer. In this case it is important to know the extent of the clay layer and what is underneath.

![Modified reference soil profile](figure II.22.)

- borehole 3
  - layer 1: \( \phi > 40 \)
  - layer 2: \( \phi > 40 \)
APPENDIX III
FUTURE LOADS
Contents

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1.1. Introduction .................................................................................................................................. 1

1.2. Present dead load of section b of the jetty structure ................................................................. 1
  1.2.1. The weight of the deck structure ......................................................................................... 1
  1.2.2. The weight of the piles ........................................................................................................ 1

1.3. Future pellet handling equipment loads ..................................................................................... 2
  1.3.1. The unloader loads ............................................................................................................... 2
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Upgrading of iron ore jetty
1.1. Introduction

This appendix comprehends the discussion and determination of the characteristic loads on section B of the jetty in the future situation. It consists of the determination of a restricted set of loads in the future situation.

The following loads are dealt with in this chapter and are considered to be the most important:
1. the present dead load of section B of the jetty structure;
2. the future pellet handling equipment loads;
3. the future vessel loads;
4. the present earthquake loads.

1.2. Present dead load of section b of the jetty structure

1.2.1. The weight of the deck structure

The dead weight of the complete deck structure is determined in appendix V. The results are summarised below.

The dead weight of the complete deck structure: 135456 kN

The dead weight per square metre: 21 kN/m²

1.2.2. The weight of the piles

| pile Ø 762 mm | 78.5 | 30 | 0.033 | 78 |
| pile Ø 1016 mm | 78.5 | 32 | 0.044 | 111 |
| 1 transversal pile row | | | | 534 |
| total | | | | 31506 |
1.3. Future pellet handling equipment loads

1.3.1. The unloader loads

The characteristic vertical unloader loads, presented in appendix I, are given by Babcock Contractors Limited. The horizontal wind loads are not given explicitly but can be obtained from the differences in vertical corner load between similar unloader operating situations under different wind conditions. This is explained in the figures III.1. and III.2. Only one load case will be calculated here as an example. The other values are presented in table III.2.

![Diagram of wind load on unloader](image-url)
Appendix III

Figure III.2. Calculation wind load static working condition.

\[ M = 7 \cdot 12.5 = 87.5 kN/m \]
\[ M = H \cdot x \]
\[ \Rightarrow H_w = 6 kN \]

Where:
- \( M \): the moment generated by the wind load
- \( H_w \): the wind load
- \( x \): imaginary point of impact of wind load, which is chosen arbitrarily!

Table III.2. Horizontal unloader wind loads

<table>
<thead>
<tr>
<th>Condition</th>
<th>Wind Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static working condition</td>
<td>0</td>
</tr>
<tr>
<td>Excluding wind</td>
<td></td>
</tr>
<tr>
<td>Including wind</td>
<td>6</td>
</tr>
<tr>
<td>Dynamic working condition</td>
<td>0</td>
</tr>
<tr>
<td>Excluding wind</td>
<td></td>
</tr>
<tr>
<td>Including wind</td>
<td>6</td>
</tr>
<tr>
<td>Static boom raised</td>
<td>0</td>
</tr>
<tr>
<td>Excluding wind</td>
<td></td>
</tr>
<tr>
<td>Including working wind</td>
<td>6</td>
</tr>
<tr>
<td>Including storm wind</td>
<td>35</td>
</tr>
</tbody>
</table>
1.3.2. The conveyor belt loads

The conveyor belts for horizontal transport of the iron ore pellets are situated at the rear side of the jetty. The conveyor system is elevated through its entire length by steel H-profiles with a concrete footing. These footings have a centre-to-centre distance of 15 metres and are constructed every 15 metres centred on top of a transverse beam. The systematics by which the characteristic conveyor belt loads are calculated is explained in figure III.3.

![Diagram](image)

*figure III.3. Longitudinal cross section of jetty with conveyor load.*

The results are presented in table III.3.

<table>
<thead>
<tr>
<th>conveyor belt loads</th>
<th>q [kN/m']</th>
<th>load on 1 footing [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>conveyor belt dead load</td>
<td>7.5</td>
<td>112.5</td>
</tr>
<tr>
<td>conveyor belt live load</td>
<td>3.5</td>
<td>52.5</td>
</tr>
</tbody>
</table>
1.4. The Future Vessel Loads

Two types of vessel loads on the jetty can be distinguished:
1. the berthing loads;
2. the environmental loads.

These two types will be discussed in section 2.3.1 and 2.3.2.

1.4.1. The future berthing loads

The berthing and de-berthing will be performed with the help of tug-boats. The permissible berthing angle is 10°. In practice however, the vessels berth almost parallel with the jetty. It is assumed that a minimum of 2 fenders are loaded simultaneously during the first impact of the vessel with the jetty. The present fenders are installed by Bridgestone. For the future situation continuos use will be made of Bridgestone fenders. Bridgestone has developed its own design manual and for reasons of consistency the "Bridgestone design Standards" will be used here instead of the "British Standards".

The main functions of the fenders are to absorb and spread the berthing energy of the vessel, this in order to protect both the vessel and the jetty. The fender can be seen as a large rubber spring which transforms the energy of movement into energy of deformation. The larger the deformation of the fender the lower the resulting fender force on the jetty. For fender design it is important to find a optimal compromise between deformation capacity and strength of the fender.

The systematics used in the Bridgestone design Manual consists of determining the berthing energy of the vessel followed by choosing a suitable fender type with the help of fender performance tables. This will be done hereafter.

**Berthing energy.**
Following is the formula used by Bridgestone to calculate the berthing energy:

\[ E_{\text{berthing}} = \frac{(W_1 + W_2) \cdot V^2}{2 \cdot g} \cdot K \]

Where:
- \( E_{\text{berthing}} \): effective berthing energy [ton-m]
- \( W_1 \): displacement tonnage [ton]
- \( W_2 \): additional weight [ton]
- \( V \): berthing speed [m/s]
- \( g \): acceleration of gravity [m/s²]
- \( K \): eccentricity factor
\( W_1 \) : Displacement Tonnage.
This is the tonnage expressed by the total weight of the vessel body, engine, cargo and all other materials, loaded in it. There are two types of displacement tonnage:
1. Full Load Displacement : maximum cargo load
2. Light Displacement : no cargo loaded

To obtain the maximum berthing energy the Full Load Displacement is used here. This value is obtained from chapter 2.3. from the main report.

\( W_1 = 200,000 \) tons

\( W_2 \) : Additional Weight
The additional weight is generally defined as the weight of sea water of a cylindrical shape having a diameter equivalent to the draught and length of the vessel. Here it is calculated by the following "Stelson Mavils" formula:

\[
W_2 = \rho \cdot L \cdot H^2 \cdot \frac{\pi}{4}
\]

Where:
- \( \rho \) : specific weight of sea water = 1,025 ton/m\(^3\)
- \( L \) : length of vessel (L.O.A.) = 300 m
- \( H \) : full draught = 16 m

\( W_2 = 61,827 \) ton

\( V \) : Berthing Speed
The berthing speed is one of the most important criteria for designing a berthing system. The berthing speed of a vessel is determined from values measured or from data previously measured for the berthing speed in consideration of the size and loading conditions of the vessel, the location and structure of the berthing facilities, meteorological and marine conditions, presence of tugboats and their size, etc..

For large bulk carriers using tugboats which push or pull the vessel gradually parallel with the jetty to the jetty berthing speeds of 1.0 - 1.5 m/s can be adopted according to Bridgestone. As the berthing speed has a large influence on the berthing energy it will be looked at more carefully. Values recommended by the British Standards will be examined too.

\( V = 0.175 \) m/s
\( = 0.15 \) m/s
\( = 0.1 \) m/s

\( K \) : Eccentricity Factor
In most cases, a vessel berths with either the bow or stern at an angle of a certain degree (max. 10°) to the jetty. At the time of berthing, the vessel turns simultaneously. For this reason, the total kinetic energy held by the vessel is consumed partially as its turning energy, and the remaining energy is conveyed to the jetty. This remaining energy is obtained from the kinetic energy of a vessel by correction with the eccentricity factor. The following formula which is derived with the help of figure III.4. is used to calculate the factor \( K \).
Bridgestone takes for the value of the turning radius of the vessel 1/4 of the length of the vessel. This leads to the following:

\[ K = \frac{1}{1 + \left( \frac{L}{r} \right)^2} \]

where
- \( L \) : Vessel length
- \( CG \) : Center of gravity
- \( P \) : Berthing point
- \( \ell \) : Distance of line parallel to wharf measured from the contact point to the center of gravity of the ship (m)
- \( r \) : Radius of gyration about vertical axis passing through center of gravity on horizontal plane (m)
- \( m \) : Distance along a line joining the center of gravity and the berthing point
- \( \phi \) : Angle between "\( m \)" and the vessel speed vector "\( V \)"
- \( \theta \) : Berthing angle

Generally, two kinds of eccentricity factors, \( K \) or \( C_E \), are applied.

**figure III.4.** model used to derive K
With the data gathered above the berthing energies for 3 different berthing speeds can be calculated.

\[
E_{\text{berthing},0,1} = \frac{(200.000 + 61827) \cdot 0,1^2}{2 \cdot 9,81} \cdot 0,5 = 66,5 \text{ton} \cdot \text{m} = 665 \text{kNm}
\]

\[
E_{\text{berthing},0,15} = \frac{(200.000 + 61827) \cdot 0,15^2}{2 \cdot 9,81} \cdot 0,5 = 150 \text{ton} \cdot \text{m} = 1500 \text{kNm}
\]

\[
E_{\text{berthing},0,175} = \frac{(200.000 + 61827) \cdot 0,175^2}{2 \cdot 9,81} \cdot 0,5 = 204,5 \text{ton} \cdot \text{m} = 2045 \text{kNm}
\]

The berthing energy calculated above is considered to be the normal berthing energy, the abnormal energy is considered to be twice the normal energy. The criterion berthing speed is 0.15 m/s, as indicated in appendix I. The appropriate berthing energy is used further in this section.

**The provisional results are:**

\[
E_b;\text{normal} = 1500 \text{ kNm}
\]

\[
E_b;\text{abnormal} = 3000 \text{ kNm}
\]

**The provisional future fender system**

The fender system can be chosen with the aid of dimension and performance tables from a Bridgestone publication called "Cell Fender Series". To choose a fender system the maximum design energy to be absorbed by one fender must be known. The design criteria state that the berthing energy must be absorbed by a minimum of two fenders simultaneously. This results in the following design berthing energy:

\[
E_b;\text{design} = 150 \text{ ton}
\]

The fender system which fulfils this condition best is the cell fender series SUC2000H RE as can be seen in table III.4.

The performance capacities if this cell fender series is presented in figure III.5.

This results in a design abnormal fender force, as deduced from figure III.5., of

\[
F_{f;\text{design}} = 2460 \text{ kN}
\]
### SUC2000H

#### (3) Performance

<table>
<thead>
<tr>
<th>Rubber grade</th>
<th>Rated reaction force</th>
<th>Maximum reaction force</th>
<th>Rated energy absorption</th>
<th>Maximum energy absorption</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tons</td>
<td>Tons</td>
<td>Ton-M</td>
<td>Tons</td>
</tr>
<tr>
<td>RE</td>
<td>300.0</td>
<td>319.9</td>
<td>264.3</td>
<td>279.8</td>
</tr>
<tr>
<td></td>
<td>661.5</td>
<td>705.4</td>
<td>1912.2</td>
<td>2024.4</td>
</tr>
<tr>
<td>RS</td>
<td>267.1</td>
<td>283.9</td>
<td>234.6</td>
<td>248.4</td>
</tr>
<tr>
<td></td>
<td>689.0</td>
<td>762.0</td>
<td>1697.3</td>
<td>1797.2</td>
</tr>
<tr>
<td>RH</td>
<td>231.5</td>
<td>246.1</td>
<td>203.3</td>
<td>215.3</td>
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<td></td>
<td>510.5</td>
<td>542.7</td>
<td>1470.9</td>
<td>1557.7</td>
</tr>
<tr>
<td>R0</td>
<td>178.1</td>
<td>189.3</td>
<td>156.4</td>
<td>165.6</td>
</tr>
<tr>
<td></td>
<td>392.7</td>
<td>417.4</td>
<td>1131.6</td>
<td>1198.1</td>
</tr>
<tr>
<td>R1</td>
<td>142.5</td>
<td>151.4</td>
<td>125.1</td>
<td>132.5</td>
</tr>
<tr>
<td></td>
<td>314.2</td>
<td>333.8</td>
<td>906.1</td>
<td>958.6</td>
</tr>
</tbody>
</table>

Rated deflection: 52.5%
Maximum deflection: 55%

Tolerance: ± 10%

#### (4) Performance Curve

![Performance Curve](image)

**Figure III.5.** Bridgestone fender performance
Figure III.6. Sign convention and coordinate system.
2.3.2. The future environmental vessel loads

The environmental vessel loads are generated by wind forces and current forces on the vessel that in turn burdens the jetty because it is restrained by the fender and mooring system. To determine the resulting bolder and fender loads it is necessary to know the mooring patterns, the wind- and current speed and direction.

The mooring patterns.
Figure III.7. explains the mooring patterns.

Where:
- breast lines: to secure vessel in transversal direction
- spring lines: to secure vessel in longitudinal direction

The mooring loads.
The information needed to calculate the mooring loads is obtained from "Prediction of Wind and Current Loads on VLCC's" published by "Oil Companies International Marine Forum". Figure III.6. shows the sign convention and coordinate system.

The equations used are:
Wind load:
- Longitudinal wind force: \( F_{xw} = C_{xw} \cdot P_w \cdot A_T \) (1)
- Lateral wind force: \( F_{yw} = C_{yw} \cdot P_w \cdot A_L \) (2)
- Wind yaw moment: \( M_{xw} = C_{xrw} \cdot P_w \cdot A_L \cdot L_{BP} \) (3)

Current load:
- Longitudinal current force: \( F_{xc} = C_{xc} \cdot \left( \frac{\rho_c}{7600} \right) \cdot v^2 \cdot T \cdot L_{BP} \) (4)
- Lateral current force: \( F_{xc} = C_{yc} \cdot \left( \frac{\rho_c}{7600} \right) \cdot V_c^2 \cdot T \cdot L_{BP} \) (5)
- Current yaw moment: \( F_{yc} = C_{yc} \cdot \left( \frac{\rho_c}{7600} \right) \cdot V_c^2 \cdot T \cdot L_{BP}^2 \) (6)

The environmental characteristics are:
- \( V_c \): the current speed under normal circumstances \( = 0.35 \text{ m/s} \)
- \( V_c \): the current speed under extreme circumstances \( = 2.1 \text{ m/s} \)
- \( d \): water depth \( = 20 \text{ m} \)
$p_w$ : the wind pressure under normal circumstances = 125 N/m$^2$

$\rho_w$ : the wind pressure under extreme circumstances = 400 N/m$^2$

$\rho_c$ : the density in wind medium at 20° = 104,47 kg·s$^2$/m$^4$

$\rho_c$ : the density in current medium at 20° = 0,1248 kg·s$^2$/m$^4$

The vessel characteristics are presented in table III.4.

<table>
<thead>
<tr>
<th>vessel size [D.W.T.]</th>
<th>$L_{OA}$ [m]</th>
<th>$L_{BP}$ [m]</th>
<th>$B$ [m]</th>
<th>$MD$ [m]</th>
<th>$r$ [m]</th>
<th>$T$ [m]</th>
<th>$FB$ [m]</th>
<th>$A_t \times 10^3$ $m^2$</th>
<th>$A_T \times 10^3$ $m^2$</th>
<th>$d/T$</th>
</tr>
</thead>
<tbody>
<tr>
<td>150,000</td>
<td>280,0</td>
<td>268,0</td>
<td>53,5</td>
<td>20,0</td>
<td>3,1</td>
<td>full</td>
<td>14,7</td>
<td>5,3</td>
<td>2,5</td>
<td>1,2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ballast</td>
<td>3,8</td>
<td>16,2</td>
<td>5,5</td>
<td>1,8</td>
</tr>
</tbody>
</table>

The current and wind coefficients for three values of $\theta$ are presented in table III.5.

<table>
<thead>
<tr>
<th></th>
<th>$\theta = 5^\circ$</th>
<th>$\theta = 90^\circ$</th>
<th>$\theta = 175^\circ$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>fully loaded</td>
<td>ballasted</td>
<td>fully loaded</td>
</tr>
<tr>
<td>$C_{XYW}$</td>
<td>-0,94</td>
<td>-0,8</td>
<td>0,05</td>
</tr>
<tr>
<td>$C_{YWW}$</td>
<td>-0,05</td>
<td>-0,05</td>
<td>-0,72</td>
</tr>
<tr>
<td>$C_{XW}$</td>
<td>0</td>
<td>-0,01</td>
<td>0,04</td>
</tr>
<tr>
<td>$C_{XC^*}$</td>
<td>-0,06</td>
<td>-0,05</td>
<td>0</td>
</tr>
<tr>
<td>$C_{Y^<em>C^</em>}$</td>
<td>-0,02</td>
<td>0</td>
<td>-2,35</td>
</tr>
<tr>
<td>$C_{XYY^<em>C^</em>}$</td>
<td>-0,05</td>
<td>-0,025</td>
<td>0,05</td>
</tr>
</tbody>
</table>

*) No distinction is made between cylindrical bows and conventional bows. The values of these coefficients are determined with the following $d/T$ ratios:

vessel fully loaded : 1.20
vessel ballasted    : 3.00

The wind and current load patterns are:

**Pattern 1:**

![vessel](image)

Where:

$\theta_w = 5^\circ$

$\theta_c = 5^\circ$
Pattern 2:

\[ \text{current} \rightarrow \text{vessel} \]

\[ \text{wind} \rightarrow \text{vessel} \]

Where:
\[ \theta_w = 175^\circ \]
\[ \theta_c = 175^\circ \]

Pattern 3:

\[ \text{current} \rightarrow \text{vessel} \]

\[ \text{vessel} \rightarrow \text{current} \]

\[ \text{wind} \rightarrow \text{vessel} \]

Where:
\[ \theta_w = 90^\circ \]
\[ \theta_c = 5^\circ/175^\circ \]

The method by which the wind and current loads are calculated consists of the following three steps:
1. determine the vessel characteristics from table III.4.;
2. obtain the wind/current coefficients from table III.5.;
3. calculate the wind/current forces from the equations (1) to (6).

**Computation of wind and current load**

The final step in the determination of the environmental vessel loads consists of the following three elements:
1. calculate all wind forces as function of \( \theta \);
2. calculate all current forces as function of \( \theta \);
3. process the data into the three load cases.

**Computation 1: \( \theta = 5^\circ \)**

- Wind loads under normal conditions:
  \[ F_{xw} = -0.94 \cdot 125 \cdot 1.2 \cdot 10^3 = -141kN \]
  \[ F_{yw} = -0.05 \cdot 125 \cdot 1.2 \cdot 10^3 = -7.4kN \]
  \[ M_{xyw} = 0kN \]

  Vessel fully loaded:
  \[ F_{xw} = -0.8 \cdot 125 \cdot 1.8 = -178.5kN \]
  \[ F_{yw} = -0.05 \cdot 125 \cdot 1.8 = -11.1kN \]
  \[ M_{xyw} = 0.01 \cdot 125 \cdot 1.8 \cdot 268 = -603kN \]

Vessel ballasted:
- Wind loads under abnormal conditions:

\[ F_{xw} = -0,94 \cdot 400 \cdot 1,2 = -451 \text{kN} \]

Vessel fully loaded:

\[ F_{yw} = -0,05 \cdot 400 \cdot 1,2 = -24 \text{kN} \]

\[ M_{xw} = 0 \text{kN} \]

\[ F_{xw} = -0,8 \cdot 400 \cdot 1,8 = -576 \text{kN} \]

Vessel ballasted:

\[ F_{yw} = -0,05 \cdot 400 \cdot 1,8 = -36 \text{kN} \]

\[ M_{xw} = 0,01 \cdot 400 \cdot 1,8 \cdot 268 = -1929,6 \text{kN} \]

- Current loads under normal conditions:

Vessel fully loaded:

\[
F_{xc} = -0,06 \cdot \left(\frac{104,47}{7600}\right) \cdot 0,35^2 \cdot 14,7 \cdot 268 = -0,4 t = -4 \text{kN}
\]

\[
F_{yc} = -0,02 \cdot \left(\frac{104,47}{7600}\right) \cdot 0,35^2 \cdot 14,7 \cdot 268 = -0,13 t = -1,3 \text{kN}
\]

\[
M_{xyc} = -0,05 \cdot \left(\frac{104,47}{7600}\right) \cdot 0,35^2 \cdot 14,7 \cdot 268^2 = -88,9 t \cdot m = -889 \text{kNm}
\]

Vessel ballasted:

\[
F_{xc} = -0,05 \cdot \left(\frac{104,47}{7600}\right) \cdot 0,35^2 \cdot 3,8 \cdot 268 = 0,09 t = -0,9 \text{kN}
\]

\[
F_{yc} = -0,02 \cdot \left(\frac{104,47}{7600}\right) \cdot 0,35^2 \cdot 3,8 \cdot 268 = 0,13 t = 0 \text{kN}
\]

\[
M_{xyc} = -0,025 \cdot \left(\frac{104,47}{7600}\right) \cdot 0,35^2 \cdot 3,8 \cdot 268^2 = -11,5 t \cdot m = -115 \text{kNm}
\]

- Current loads under abnormal conditions:

Vessel fully loaded:

\[
F_{xc} = -0,06 \cdot \left(\frac{104,47}{7600}\right) \cdot 2,1^2 \cdot 14,7 \cdot 268 = 14,4 \text{tonne} = -144 \text{kN}
\]

\[
F_{yc} = -0,02 \cdot \left(\frac{104,47}{7600}\right) \cdot 2,1^2 \cdot 14,7 \cdot 268 = 4,68 \text{tonne} = -46,8 \text{kN}
\]

\[
M_{xyc} = -0,05 \cdot \left(\frac{104,47}{7600}\right) \cdot 2,1^2 \cdot 14,7 \cdot 268^2 = -3200,4 t \cdot m = -32004 \text{kNm}
\]

Vessel ballasted:

\[
F_{xc} = -0,05 \cdot \left(\frac{104,47}{7600}\right) \cdot 2,1^2 \cdot 3,8 \cdot 268 = -3,1 t = -31 \text{kN}
\]

\[
F_{yc} = -0 \cdot \left(\frac{104,47}{7600}\right) \cdot 2,1^2 \cdot 3,8 \cdot 268 = 0 t = 0 \text{kN}
\]

\[
M_{xyc} = -0,025 \cdot \left(\frac{104,47}{7600}\right) \cdot 2,1^2 \cdot 3,8 \cdot 268^2 = -413,6 t \cdot m = -4136 \text{kNm}
\]
Computation 2: \( \theta_w = 90^\circ \) 
\( \theta_c = 175^\circ \)

- Wind loads under normal conditions:
  
  \[ F_{xw} = 0,05 \cdot 125 \cdot 1,2 = 7,5kN \]
  
  Vessel fully loaded:  
  \[ F_{yw} = -0,72 \cdot 125 \cdot 1,2 = -108kN \]
  \[ M_{xtw} = 0,04 \cdot 125 \cdot 1,2 \cdot 268 = 1608kNm \]
  
  \[ F_{xw} = 0 \cdot 125 \cdot 1,8 = 0kN \]
  
  Vessel ballasted:  
  \[ F_{yw} = -0,99 \cdot 125 \cdot 1,8 = -225kN \]
  \[ M_{xtw} = 0,13 \cdot 125 \cdot 1,8 \cdot 268 = 7839kNm \]

- Wind loads under abnormal conditions:
  
  \[ F_{xw} = 0,05 \cdot 400 \cdot 1,2 = 24kN \]
  
  Vessel fully loaded:  
  \[ F_{yw} = -0,72 \cdot 400 \cdot 1,2 = -345,6kN \]
  \[ M_{xtw} = 0,04 \cdot 400 \cdot 1,2 \cdot 268 = 5145,6kNm \]
  
  \[ F_{xw} = 0 \cdot 400 \cdot 1,8 = 0kN \]
  
  Vessel ballasted:  
  \[ F_{yw} = -0,99 \cdot 400 \cdot 1,8 = -712,8kN \]
  \[ M_{xtw} = 0,13 \cdot 400 \cdot 1,8 \cdot 268 = 25084,8kNm \]

- Current loads under normal conditions:

  Vessel fully loaded:
  \[ F_{xc} = 0,1 \cdot \left( \frac{104,47}{7600} \right) \cdot 0,35^2 \cdot 14,7 \cdot 268 = 0,66t = 6,6kN \]
  
  \[ F_{yc} = -0,02 \cdot \left( \frac{104,47}{7600} \right) \cdot 0,35^2 \cdot 14,7 \cdot 268 = -0,13t = -1,3kN \]
  
  \[ M_{xtc} = 0,075 \cdot \left( \frac{104,47}{7600} \right) \cdot 0,35^2 \cdot 14,7 \cdot 268^2 = 133,3t \cdot m = 1333kNm \]

  Vessel ballasted:
  \[ F_{xc} = 0,1 \cdot \left( \frac{104,47}{7600} \right) \cdot 0,35^2 \cdot 3,8 \cdot 268 = 0,17t = 1,7kN \]
  
  \[ F_{yc} = 0 \cdot \left( \frac{104,47}{7600} \right) \cdot 0,35^2 \cdot 3,8 \cdot 268 = 0t = 0kN \]
  
  \[ M_{xtc} = 0,025 \cdot \left( \frac{104,47}{7600} \right) \cdot 0,35^2 \cdot 3,8 \cdot 268^2 = 11,5t \cdot m = 115kNm \]
• Current loads under abnormal conditions:
  Vessel fully loaded:
  \[ F_{xc} = 0,1 \cdot \left( \frac{104,47}{7600} \right) \cdot 2,1^2 \cdot 14,7 \cdot 268 = 23,9 t = 239 kN \]
  \[ F_{yc} = -0,02 \cdot \left( \frac{104,47}{7600} \right) \cdot 2,1^2 \cdot 14,7 \cdot 268 = -4,8 t = -48 kN \]
  \[ M_{xyc} = 0,075 \left( \frac{104,47}{7600} \right) \cdot 2,1^2 \cdot 14,7 \cdot 268^2 = 4800,3 t \cdot m = 48003 kNm \]
  Vessel ballasted:
  \[ F_{xc} = 0,1 \cdot \left( \frac{104,47}{7600} \right) \cdot 2,1^2 \cdot 3,8 \cdot 268 = 6,2 t = 62 kN \]
  \[ F_{yc} = 0 \left( \frac{104,47}{7600} \right) \cdot 2,1^2 \cdot 3,8 \cdot 268 = 0 t = 0 kN \]
  \[ M_{xyc} = 0,025 \left( \frac{104,47}{7600} \right) \cdot 2,1^2 \cdot 3,8 \cdot 268^2 = 413,6 t \cdot m = 4136 kNm \]

Computation 3: \( \theta = 175^\circ \)
• Wind loads under normal conditions:
  \[ F_{xw} = 0,78 \cdot 125 \cdot 1,2 = 117 kN \]
  Vessel fully loaded:
  \[ F_{yw} = -0,12 \cdot 125 \cdot 1,2 = -18 kN \]
  \[ M_{xw} = 0,05 \cdot 125 \cdot 1,2 \cdot 268 = 2010 kNm \]
  \[ F_{xw} = 0,62 \cdot 125 \cdot 1,8 = 139,5 kN \]
  Vessel ballasted:
  \[ F_{yw} = -0,15 \cdot 125 \cdot 1,8 = -33,8 kN \]
  \[ M_{xw} = 0,05 \cdot 125 \cdot 1,8 \cdot 268 = 3015 kNm \]

• Wind loads under abnormal conditions:
  \[ F_{xw} = 0,78 \cdot 400 \cdot 1,2 = 374,4 kN \]
  Vessel fully loaded:
  \[ F_{yw} = -0,12 \cdot 400 \cdot 1,2 = -57,6 kN \]
  \[ M_{xw} = 0,05 \cdot 400 \cdot 1,2 \cdot 268 = 6432 kNm \]
  \[ F_{xw} = 0,62 \cdot 400 \cdot 1,8 = 446,4 kN \]
  Vessel ballasted:
  \[ F_{yw} = -0,15 \cdot 400 \cdot 1,8 = -108 kN \]
  \[ M_{xw} = 0,05 \cdot 400 \cdot 1,8 \cdot 268 = 9648 kNm \]
• Current loads under normal conditions:
  Vessel fully loaded:
  \[ F_{xc} = 0.1 \left( \frac{104,47}{7600} \right) \cdot 0.35^2 \cdot 14.7 \cdot 268 = 0.66t = 6.6kN \]
  \[ F_{yc} = -0.02 \left( \frac{104,47}{7600} \right) \cdot 0.35^2 \cdot 14.7 \cdot 268 = 0.13t = 1.3kN \]
  \[ M_{xyc} = 0.075 \left( \frac{104,47}{7600} \right) \cdot 0.35^2 \cdot 14.7 \cdot 268^2 = 133.3t \cdot m = 1333kNm \]

  Vessel ballasted:
  \[ F_{xc} = 0.1 \left( \frac{104,47}{7600} \right) \cdot 0.35^2 \cdot 3.8 \cdot 268 = 0.17t = 1.7kN \]
  \[ F_{yc} = 0 \left( \frac{104,47}{7600} \right) \cdot 0.35^2 \cdot 3.8 \cdot 268 = 0t = 0kN \]
  \[ M_{xyc} = 0.025 \left( \frac{104,47}{7600} \right) \cdot 0.35^2 \cdot 3.8 \cdot 268^2 = 11.5t \cdot m = 115kNm \]

• Current loads under abnormal conditions:
  Vessel fully loaded:
  \[ F_{xc} = 0.1 \left( \frac{104,47}{7600} \right) \cdot 2.1^2 \cdot 14.7 \cdot 268 = 23.9t = 239kN \]
  \[ F_{yc} = -0.02 \left( \frac{104,47}{7600} \right) \cdot 2.1^2 \cdot 14.7 \cdot 268 = -4.8t = -48kN \]
  \[ M_{xyc} = 0.075 \left( \frac{104,47}{7600} \right) \cdot 2.1^2 \cdot 14.7 \cdot 268^2 = 4800.3t \cdot m = 48003kNm \]

  Vessel ballasted:
  \[ F_{xc} = 0.1 \left( \frac{104,47}{7600} \right) \cdot 2.1^2 \cdot 3.8 \cdot 268 = 6.2t = 62kN \]
  \[ F_{yc} = 0 \left( \frac{104,47}{7600} \right) \cdot 2.1^2 \cdot 3.8 \cdot 268 = 0t = 0kN \]
  \[ M_{xyc} = 0.025 \left( \frac{104,47}{7600} \right) \cdot 2.1^2 \cdot 3.8 \cdot 268^2 = 413.6t \cdot m = 4136kNm \]
The results
The results are presented below.

Table III.6. Wind and current loads under normal circumstances.

<table>
<thead>
<tr>
<th>Angle ⇒</th>
<th>θ_w = θ_e = 5°</th>
<th>θ_w = 90°, θ_e = 175°</th>
<th>θ_w = θ_e = 175°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load ↓</td>
<td>Full</td>
<td>Ballast</td>
<td>Full</td>
</tr>
<tr>
<td>F_xw</td>
<td>-141</td>
<td>-179</td>
<td>8</td>
</tr>
<tr>
<td>F_yw</td>
<td>-7</td>
<td>-11</td>
<td>-108</td>
</tr>
<tr>
<td>M_xw</td>
<td>0</td>
<td>-603</td>
<td>1608</td>
</tr>
<tr>
<td>F_xc</td>
<td>-4</td>
<td>-1</td>
<td>7</td>
</tr>
<tr>
<td>F_yc</td>
<td>-1</td>
<td>0</td>
<td>-1</td>
</tr>
<tr>
<td>M_xyc</td>
<td>-889</td>
<td>-115</td>
<td>1333</td>
</tr>
</tbody>
</table>

Table III.7. Wind and current loads under extreme circumstances.

<table>
<thead>
<tr>
<th>Angle ⇒</th>
<th>θ_w = θ_e = 5°</th>
<th>θ_w = 90°, θ_e = 175°</th>
<th>θ_w = θ_e = 175°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load ↓</td>
<td>Full</td>
<td>Ballast</td>
<td>Full</td>
</tr>
<tr>
<td>F_xw</td>
<td>-451</td>
<td>-567</td>
<td>24</td>
</tr>
<tr>
<td>F_yw</td>
<td>-24</td>
<td>-36</td>
<td>-346</td>
</tr>
<tr>
<td>M_xw</td>
<td>0</td>
<td>-1930</td>
<td>5146</td>
</tr>
<tr>
<td>F_xc</td>
<td>-144</td>
<td>-31</td>
<td>239</td>
</tr>
<tr>
<td>F_yc</td>
<td>-47</td>
<td>0</td>
<td>-48</td>
</tr>
<tr>
<td>M_xyc</td>
<td>-32004</td>
<td>-4136</td>
<td>48003</td>
</tr>
</tbody>
</table>

Wind and current pattern 1

![Wind and current pattern 1 diagram]

\[
\frac{L_{BP}}{2}
\]

<table>
<thead>
<tr>
<th>θ_w = 5°</th>
<th>Normal circumstances</th>
<th>Abnormal circumstances</th>
</tr>
</thead>
<tbody>
<tr>
<td>θ_e = 5°</td>
<td>full</td>
<td>ballast</td>
</tr>
<tr>
<td>F_xt</td>
<td>145</td>
<td>180</td>
</tr>
<tr>
<td>F_yt</td>
<td>8</td>
<td>11</td>
</tr>
<tr>
<td>M_xt</td>
<td>889</td>
<td>718</td>
</tr>
</tbody>
</table>
Wind and current pattern 2

- \( \theta_w = 90^\circ \)
- \( \theta_c = 175^\circ \)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Normal Circumstances</th>
<th>Abnormal Circumstances</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force ( F_{XT} )</td>
<td>15</td>
<td>2</td>
</tr>
<tr>
<td>Force ( F_{YT} )</td>
<td>112</td>
<td>225</td>
</tr>
<tr>
<td>Moment ( M_{XYT} )</td>
<td>2941</td>
<td>7954</td>
</tr>
</tbody>
</table>

Wind and current pattern 3:

- \( \theta_w = 175^\circ \)
- \( \theta_c = 175^\circ \)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Normal Circumstances</th>
<th>Abnormal Circumstances</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force ( F_{XT} )</td>
<td>124</td>
<td>142</td>
</tr>
<tr>
<td>Force ( F_{YT} )</td>
<td>19</td>
<td>34</td>
</tr>
<tr>
<td>Moment ( M_{XYT} )</td>
<td>3343</td>
<td>3130</td>
</tr>
</tbody>
</table>
2.4. Earthquake loads

An earthquake induces a periodical movement of the subsoil in which the jetty is founded. The way in which the jetty will react to this movement depends on the ratio of stiffness of the subsoil and the structure and the dynamic mass* of the structure. When consulting the Indonesian Earthquake Code the following system of determining the earthquake loading can be established.

The dynamic earthquake loading will be replaced by an equivalent static loading which equals a percentage of the dynamic mass* of the structure. This percentage depends on the lowest natural frequency \( N_x \) of the jetty. The relation between the natural frequency and the percentage of the dynamic mass which is to be applied as a static force is shown in figure I.1. (appendix I).

2.4.1. The lowest natural frequency of the jetty

As a first estimation of the lowest natural frequency one can model the structure by a SINGLE-DEGREE-OF-FREEDOM-SYSTEM. Here the total inertia of the structure is modelled as a single mass and the stiffness as a single spring. This is explained in figure III.8.

![Dynamic model of jetty](image)

**figure III.8.** dynamic model of jetty

The mathematical description of a single-degree-of-freedom-system vibrating freely is as follows:

\[
M^* \ddot{x} + C \dot{x} + K x = 0
\]

Where:
- \( M \) : the mass of the deck structure
- \( m \) : the mass of the substructure
- \( M^* \) : the dynamic mass, 0.405-M
- \( C \) : the damping
- \( K \) : the spring stiffness
- \( x \) : the displacement
The solution of this differential equation is determined with the formula presented below.

\[ N_1 = \frac{1}{2 \cdot \pi} \sqrt{\frac{K}{M}} \left( 1 - 2 \cdot \xi^2 \right) \]

\[ \xi = \frac{C}{2 \cdot \sqrt{M \cdot K}} \]

Where:
- \( N_1 \) : the lowest natural frequency
- \( \xi \) : the damping ratio

What is left to be done is estimating the single mass, the damping ratio and the spring-stiffness. This is done in the following sections.

**Estimation of the total inertia of the structure**

The total inertia of the structure consists of the following three elements:
1. The reinforced concrete deck
2. The superstructure and pellet handling equipment
3. The hollow steel piles

The above mentioned elements will be discussed element by element.

1. The reinforced concrete deck.
   \[ M_{\text{deck}} = 135,456 \text{ kN (appendix V)} \]

2. The superstructure and pellet handling equipment.
   No detailed information is available concerning the superstructure, the inertia of the conveyor belt will be neglected for the moment so this leaves to be considered the unloaders.
   \[ M_{\text{unloader}} = 4,505 = 20,200 \text{ kN (appendix I)} \]

3. The hollow steel piles.
   The dynamic mass of the piles is composed of the following elements:
   - the mass of the hollow member;
   - the mass of the water contained within the member;
   - the mass of the externally entrained water;
   - the mass of marine growth.

   The elements of the structure which contribute to the dynamic mass are those which can move freely. As the piles are partly embedded they will not contribute fully to the dynamic mass of structure. The part of the piles that will contribute is that stretching from the apparent level of fixation to the lowest part of the concrete deck. For the present situation this will be approximately 22 metres (chapter 6, main report). This results in \( \pm 75 \% \) of the pile inertia. This leads to the following values for the types of mass as indicated above.
The mass of a hollow member:

\[ M_{\text{pile;762}} = 78 \text{ kN} \]
\[ M_{\text{pile;1016}} = 111 \text{ kN} \]
\[ M_{\text{pile;row}} = 0.75 \cdot 534 = 401 \text{ kN} \] (section 2.1.2.)
\[ M_{\text{pile;total}} = 0.75 \cdot 31506 = 23630 \text{ kN} \] (section 2.1.2.)

The externally and internally entrained water.

The added mass of internally entrained water is easily determined. The added mass of externally entrained water can be determined mathematically by potential flow. This is thought to be beyond the scope of this thesis, so a simple estimation-formula will be used.

\[ M_{w;\text{external}} = \frac{1}{4} \cdot \pi \cdot D^2 \cdot L \cdot \rho_w \] (CRIA, 1977)

Where:
- \( D \) : pile diameter
- \( L \) : dynamic length of pile
- \( \rho_w \) : volumetric weight of water [kN/m³]

Pile \( \varnothing 762 \text{ mm} \):
\[ M_{\text{internal;762}} = 0.25 \cdot \pi \cdot (0.762 - 0.028)^2 \cdot 22 \cdot 10,25 = 95 \text{ kN} \]
\[ M_{\text{external;762}} = 0.25 \cdot \pi \cdot 0.762^2 \cdot 22 \cdot 10,25 = 103 \text{ kN} \]

Pile \( \varnothing 1016 \text{ mm} \):
\[ M_{\text{internal;1016}} = 0.25 \cdot \pi \cdot (1.016 - 0.028)^2 \cdot 22 \cdot 10,25 = 172 \text{ kN} \]
\[ M_{\text{external;1016}} = 0.25 \cdot \pi \cdot 1.016^2 \cdot 22 \cdot 10,25 = 183 \text{ kN} \]

Pile row
\[ M_{\text{internal;row}} = 4 \cdot 95 + 2 \cdot 172 = 724 \text{ kN} \]
\[ M_{\text{external;row}} = 4 \cdot 103 + 2 \cdot 183 = 778 \text{ kN} \]

Total
\[ M_{\text{internal;total}} = 59 \cdot 824 = 48616 \text{ kN} \]
\[ M_{\text{external;total}} = 59 \cdot 884 = 52156 \text{ kN} \]

The mass of the marine growth.

As no sufficient information is available the following table is used.
Table III.7. Mass of marine growth (CRIA, 1977)

<table>
<thead>
<tr>
<th>Depth below mean water level [m]</th>
<th>Mass per surface area [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 10</td>
<td>2.5</td>
</tr>
<tr>
<td>10 - 20</td>
<td>2.0</td>
</tr>
<tr>
<td>20 - 30</td>
<td>1.25</td>
</tr>
<tr>
<td>30 - 50</td>
<td>0.8</td>
</tr>
<tr>
<td>&gt; 50</td>
<td>&lt;0.02</td>
</tr>
</tbody>
</table>

The value used in this section is 2.5 kN/m². This leads to the following results:

Pile Ø 762 mm:

\[ M_{fouling:762} = \pi \cdot 0.762 \cdot 22 \cdot 2.5 = 132 \text{ kN} \]

Pile Ø 1016 mm:

\[ M_{fouling:1016} = \pi \cdot 1.016 \cdot 22 \cdot 2.5 = 176 \text{ kN} \]

Pile row

\[ M_{fouling:row} = 4 \cdot 132 + 2 \cdot 176 = 880 \text{ kN} \]

Total

\[ M_{fouling:total} = 59 \cdot 880 = 51,920 \text{ kN} \]

The total mass of the hollow steel piles is presented in the following tables.

Pile Ø 762 mm:

<table>
<thead>
<tr>
<th>M_{pile:762}</th>
<th>78</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{internal:762}</td>
<td>95</td>
</tr>
<tr>
<td>M_{external:762}</td>
<td>103</td>
</tr>
<tr>
<td>M_{fouling:762}</td>
<td>132</td>
</tr>
<tr>
<td>M_{total:762}</td>
<td>408</td>
</tr>
</tbody>
</table>

Pile Ø 1016 mm:

<table>
<thead>
<tr>
<th>M_{pile:1016}</th>
<th>111</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{internal:1016}</td>
<td>172</td>
</tr>
<tr>
<td>M_{external:1016}</td>
<td>183</td>
</tr>
<tr>
<td>M_{fouling:1016}</td>
<td>176</td>
</tr>
<tr>
<td>M_{total:1016}</td>
<td>642</td>
</tr>
</tbody>
</table>

Pile row

<table>
<thead>
<tr>
<th>M_{pile:row}</th>
<th>401</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{internal:row}</td>
<td>724</td>
</tr>
<tr>
<td>M_{external:row}</td>
<td>778</td>
</tr>
<tr>
<td>M_{fouling:row}</td>
<td>880</td>
</tr>
<tr>
<td>M_{total:row}</td>
<td>2,783</td>
</tr>
</tbody>
</table>
Upgrading of iron ore jetty

<table>
<thead>
<tr>
<th>Total</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{\text{nilletotal}}$</td>
<td>23,630</td>
</tr>
<tr>
<td>$M_{\text{internaltotal}}$</td>
<td>48,616</td>
</tr>
<tr>
<td>$M_{\text{externaltotal}}$</td>
<td>52,156</td>
</tr>
<tr>
<td>$M_{\text{finaltontotal}}$</td>
<td>51,920</td>
</tr>
<tr>
<td>$M_{\text{totalotal}}$</td>
<td>176,322</td>
</tr>
</tbody>
</table>

The lumping of the total divided mass.

As shown in figure III.8, the divided mass of the substructure is lumped in one discrete point with a representative dynamic mass $M^*$. The percentage of the total mass of the substructure which must be added to the total concentrated deck load depends on the mechanism of deformation of the substructure. Two mechanisms of deformation can be distinguished:

- deformation by bending
- deformation by shear

The adjoining constitutive equations are:

- bending : $M = L \cdot F_h = EI \cdot \kappa$
- shear : $F_h = GA \cdot \gamma$

From the results of calculations performed with the computer program PC-Frame, as explained in "Estimation of spring stiffness of the structure", the following values for the bending stiffness $EI$ and the shear stiffness $GA$ are determined:

- $F_h = 100 \text{kN}$
- $EI \approx 4 \times 10^9 \text{kNm}^2$
- $GA \approx 10 \times 10^6 \text{kNm}^2$

From the above one can conclude that deformation by shear is the principal deformation mechanism.

For this deformation mechanism one can lump 40.5% of the total divided mass of the substructure together with the total deck load. This is proven to be a realistic estimation of the lowest natural frequency (Spijkers e.a., 1992).

This leaves the following definition for the total dynamic mass:

$$M^* = M_{\text{deck}} + M_{\text{unload}} + 0.405 \cdot M_{\text{structure}} \Rightarrow$$

$$M^* = 135,456 + 94,691 + 20,200 + 0.405 \cdot 176,322 = 321,757 \text{kN}$$

**Estimation of the spring stiffness of the structure.**

The stiffness of the structure must be determined in the three main directions, which are:

1. the vertical direction
2. the horizontal transverse direction
3. the horizontal longitudinal direction

The stiffness in one particular direction can be deduced from the load-deformation curve for the concerning direction.

The stiffness in vertical direction will not be considered as the vertical earthquake force is neglected.
The stiffness in horizontal transverse direction is determined with the help of a computer program called PC-Frame. The specific file used is called FRAME. Its composition is explained in appendix IV. All loads but the earthquake loads are presented in table III.8. The relation between the horizontal transverse load and the concerning deformation is presented in table III.9. The relation is not linear. The horizontal load (earthquake load) depends on the stiffness of the jetty. This means that the stiffness must be determined iteratively.

**table III.8.** Load implemented in PC-Frame program FRAME

<table>
<thead>
<tr>
<th>loading type</th>
<th>calculation</th>
<th>devided deck load [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>dead load deck</td>
<td>135456/58/22,15</td>
<td>105</td>
</tr>
<tr>
<td>live load deck</td>
<td>0,5x30x5</td>
<td>75</td>
</tr>
<tr>
<td>pile load</td>
<td>0,405x2783/22,15</td>
<td>51</td>
</tr>
<tr>
<td>unloader load</td>
<td>4x5050/58/22,15</td>
<td>16</td>
</tr>
<tr>
<td>total load</td>
<td></td>
<td>247</td>
</tr>
</tbody>
</table>

**table III.9.** Relation between horizontal load and horizontal deformation of FRAME

<table>
<thead>
<tr>
<th>horizontal load [kN]</th>
<th>lateral deflection [m]</th>
<th>stiffness K [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.00001</td>
<td>1,000,000</td>
</tr>
<tr>
<td>200</td>
<td>0.00145</td>
<td>137,931</td>
</tr>
<tr>
<td>300</td>
<td>0.00292</td>
<td>102,740</td>
</tr>
<tr>
<td>500</td>
<td>0.00584</td>
<td>85,616</td>
</tr>
</tbody>
</table>

The stiffness in horizontal longitudinal direction is determined by hand calculation. The jetty is modelled by the statical redundant framework as shown in figure III.9. The stiffness properties of all supporting piles are implemented in the 2 vertical beams. The horizontal beam represents the deck structure and is regarded to have an infinite bending stiffness.

**figure III.9.** Model of jetty when loaded in horizontal longitudinal direction

In the future situation the lateral stiffness properties of the piles of pile row 1 and 2 are affected. To make a reasonable estimate of the future lateral stiffness is very difficult. To overcome this problem, two limiting situations are examined. These are:
- the lateral stiffness of the present situation is adapted;
- the lateral stiffness is neglected.

The piles of the substructure are considered to be fixed against rotation at a virtual point below the sea bottom level. This point is considered to be at the same level for both the raker and plumb piles. As a result of this, the lateral stiffness of the raker– and the plumb piles are directly related by their bending stiffness. The lateral stiffness of the raker piles can be derived from field test results. The determination of the lateral stiffness of the plumb piles reduces to a
comparison of bending stiffness, as performed below.

\[
\begin{align*}
    k &= \frac{E}{w} = \frac{3 \cdot EI}{l^3} \\
    k_{raker} &= \frac{120}{0.37} = 324 \text{kN/m} \\
    k_{plumb} &= \frac{EI_{plumb}}{EI_{raker}} = \frac{460,000}{1,106,000} = 0.4 \Rightarrow k_{plumb} = 0.4 \cdot 324 = 130 \text{kN/m}
\end{align*}
\]

The lateral stiffness, implemented in the model, is the summation of the values found in field tests and calculated above multiplied by two. This is explained in figure III.10.

\[\text{figure III.10. flexure of beam fixed against rotation at one end and of beam fixed against rotation at both ends}\]

The lateral stiffness in the present situation:

\[
k_{\text{present}} = 58 \cdot (4 \cdot 130 + 2 \cdot 324) = 67,744 \text{kN/m}
\]

The lateral stiffness in the future situation:

\[
k_{\text{present}} = 58 \cdot (2 \cdot 130 + 2 \cdot 324) = 52,664 \text{kN/m}
\]

**Damping**

The total damping can be divided in the following components:

- structural damping;
- hydrodynamic damping;
- damping by berthed vessel.

It is very difficult of not impossible to determine the damping analytically. As no experimental information is available no estimation of the damping can be made. In the formula of the natural frequency \(N_1\) it can be seen that the larger the damping coefficient \(\xi\) the lower the natural frequency \(N_1\). The determination of damping is considered beyond the scope of this assignment. The damping is neglected.
2.4.2. Estimation of the earthquake force

Estimation of earthquake force in transverse direction
As the stiffness of the jetty is related to the earthquake load, the earthquake load must be determined by iteration. The factors of importance are presented below.

\[
T_n = 2 \cdot \pi \cdot \sqrt{\frac{M^*}{K}} \cdot s
\]

\[
M^* = 227,066 \text{ kN}
\]

\[
K = \text{see table III.9.}
\]

\[
F_{\text{earthquake}} = 0.11 \cdot M^* \quad \text{as } T_n < 1.0 \text{ s}
\]

\[
F_{\text{earthquake}} = [0.11 - 0.055 \cdot (T_n - 1)] \cdot M^* \quad \text{as } 1.0 \text{ s} < T_n < 2.0 \text{ s}
\]

\[
F_{\text{earthquake}} = 0.055 \cdot M^* \quad \text{as } T_n > 2.0 \text{ s}
\]

Iteration leads to the following results:

\[
T_n = 1.5 \text{ s}
\]

\[
\text{acceleration factor} = 0.077
\]

\[
F_{\text{earthquake}} = 25000 \text{ kN}
\]

For safety reasons the maximum acceleration factor is taken (0.11). This leads to the following value of the earthquake load.

\[
F_{\text{earthquake}} = 35700 \text{ kN}
\]

Estimation of earthquake force in longitudinal direction
The factors of importance are presented below.

\[
T_n = 2 \cdot \pi \cdot \sqrt{\frac{M^*}{K}} \cdot s
\]

\[
M^* = 321,757 \text{ kN}
\]

\[
K = 67,774 ** \text{ kN/m}
\]

\[
F_{\text{earthquake}} = 0.11 \cdot M^* \quad \text{as } T_n < 1.0 \text{ s}
\]

\[
F_{\text{earthquake}} = [0.11 - 0.055 \cdot (T_n - 1)] \cdot M^* \quad \text{as } 1.0 \text{ s} < T_n < 2.0 \text{ s}
\]

\[
F_{\text{earthquake}} = 0.055 \cdot M^* \quad \text{as } T_n > 2.0 \text{ s}
\]

Implementation of the above-mentioned elements lead to the following results:

\[
T_n = 13.7 \text{ s}
\]

\[
\text{acceleration factor} = 0.055
\]

\[
F_{\text{earthquake}} = 17,697 \text{ kN}
\]

** this value generates the upper limit value of the acceleration factor.
4.2.3. Stability check on unloading during an earthquake

The overall stability of the unloader crane can be checked with the help of figure III.1. Stability is assured when the value of $x$, which indicates the imaginary application point of the horizontal earthquake load, is larger than app. 15 metres. The value of $x$ is determined by equalising the moments round turning point A. This leads to the following equation for $x$.

$$\sum_A M = 0 \Rightarrow 5050 \cdot \frac{12.5}{2} - 555 \cdot x = 0 \Rightarrow x = 56m \geq 15m$$

From the calculation performed above one can conclude that the unloaders remain stable during an earthquake!
APPENDIX IV
MODEL OF LOAD DELIVERY IN FUTURE SITUATION
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Model of load delivery in future situation

1.1. Introduction

The problem definition
The purpose of this appendix is to determine the maximum individual pile loads as a result of each of the 5 load combinations as given by KEC. The calculations will be performed with PC-frame. This is a computer program designed to calculate statical redundant frameworks.

The loads
The load combinations as given by K.E.C. consist of numerous loads. Of these loads 2 are considered in this appendix. These have the most influence on the pile loads.

The loads considered in this appendix are:
- dead load;
- live load;
- unloader load;
- berthing/mooring load;
- earthquake loads.

There are three main directions in which the loads can attack: the vertical direction, the transverse horizontal direction and the longitudinal direction. The directions considered in this appendix are: the vertical direction and the transverse horizontal direction. The load combinations are such that maximum compression loads occur in the piles 5 and 6 (see figure IV.2 for explanation of pile numbers).

The procedure
The jetty can be regarded as composed of a series of statical redundant frames, which are connected by the deck structure. This is visualised in figure IV.1. The jetty loads are spread over the frames by the deck structure. Each overall load combination results in a criterion load combination acting on one framework. In order to determine the criterion individual pile loads, the internal forces in the framework, resulting from the aforementioned load combination, have to be calculated.
In order to be able to determine the way in which the overall loads are spread over the frameworks it is necessary to know the stiffness of the different constructive elements. These elements are the deck structure and its supporting piles. The axial stiffnesses of the piles are obtained from the results of field tests (appendix II). The stiffnesses of the deck elements are determined in appendix V.

The stiffness of the deck structure and a transverse framework depend on the direction in which they are loaded. To deal with this problem three models are made. These are:

1. a model to absorb transverse horizontal loads;
2. a model to absorb vertical loads acting on beam A (see figure 2.5.);
3. a model to absorb vertical loads acting on beam B (see figure 2.5.).

This way the division of loads among the frames is determined for all load types. As the division of loads depends on its qualitative and not on its quantitative properties each load type can be coupled with a coefficient. This coefficient gives the criterion load on a framework as a result of an overall jetty load.

Thus calculating the criterion individual pile loads as a result of an overall jetty load combination comes down to:

- Design of models which represent the constructive properties of the jetty when subjected to specific loads.
- Implementing these models in the computer program PC-frame.
- Determining the way in which the loads are spread over the frameworks with the help of the aforementioned coefficients.
- Calculating the internal forces in the framework as a result of the loads as determined in the previous step.
1.2. Models

1.2.1 Introduction

The jetty is a complex composition of steel piles and concrete beams and slabs. It is impossible to create an exact picture of the action of forces in the jetty as a result of external loads. This design only requires a rough estimate of the action of forces. For this reason simple PC-frame models, 2-dimensional statical redundant frameworks, of the jetty will do.

1.2.2. Model 1

The model 1 represents a transverse pile section, as shown in figure IV.1. It consists of beam D (see figure 2.5.) supported by its relating piles. The function of the model is twofold. These are:
1. determine the stiffness of the supports of model 2, model 3, and model 4;
2. determine the individual pile loads as a result of unloader loads.

The properties of the plumb and raker piles are obtained from pile tests and the properties of beam D are derived in appendix IV.

1.2.3. Model 2

Model 2 represents the jetty structure when subjected to transverse horizontal load. It consists of a beam, which represents the deck loaded in plane, supported by the transverse frameworks as shown in figure IV.1. The function of this model is to determine the coefficient which indicates the distribution of berthing loads over the transverse frameworks (model 1). The properties of the supporting elements are derived from model 1. The properties of the horizontal beam are derived in appendix V.

1.2.4. Model 3

Model 3 represents the jetty structure when subjected to a vertical load acting on longitudinal beam A. It consists of beam A supported by beams, which properties are obtained from model 1. The function of this model is to determine the coefficient which indicates the distribution of unloader loads over the transverse frameworks (model 1). The properties of beam A are derived in appendix V.

1.2.5. Model 4

Model 4 represents the jetty structure when subjected to a vertical load acting on longitudinal beam B. It consists of beam B supported by beams, which properties are obtained from model 1. The function of this model is to determine the coefficient which indicates the distribution of unloader loads over the transverse frameworks (model 1). The properties of beam B are derived in appendix V.
1.3. Implementation of beams in PC-frame

1.3.1. Introduction

This section deals with the implementation of the aforementioned models in PC-Frame. The models are statical redundant frameworks and consist of beams which have pinned or fixed connections. PC-frame defines the beams by the following 4 properties:
1. \( E \), modulus of elasticity;
2. \( A \), surface area of cross section;
3. \( W \), moment of resistance;
4. \( I \), moment of inertia.

Firstly all beams used in the aforementioned models are implemented in PC-frame. Secondly the way in which the models are composed by these beams is explained.

1.3.2. Implementation of beams in PC-frame

This section deals with the implementation in PC-Frame of all the beams used in model 1 to model 4.

PC-frame only uses 1 constant value for "E". The properties of importance, however, are "EI" and "EA". As each beam or pile has a different value for "E", the values of "A" and "I" must be adapted in order to obtain the proper values of "EA" and "EI". When necessary this is indicated.

The different beam types used in these models are:
1. plumb pile;
2. raker pile;
3. deck loaded in plane;
4. transverse deck beam C;
5. longitudinal deck beam A;
6. longitudinal deck beam B;
7. transverse deck support.

1 Plumb pile
The profile name used in PC-frame is "PLUMB".
The pile properties:
\[
\begin{align*}
E &= 210,000,000 \text{kN/m}^2 \\
A &= 3.1 \cdot 10^{-2} \text{m}^2 \\
W &= 4.6 \cdot 10^{-3} \text{m}^3 \\
I &= 2.3 \cdot 10^{-3} \text{m}^4
\end{align*}
\]

2 Raker pile
The profile name used in PC-frame is "RAKER".
The pile properties:
\[
\begin{align*}
E &= 210,000,000 \text{kN/m}^2 \\
A &= 4.2 \cdot 10^{-2} \text{m}^2 \\
W &= 8.28 \cdot 10^{-3} \text{m}^3 \\
I &= 5.27 \cdot 10^{-3} \text{m}^4
\end{align*}
\]
3 Deck loaded in plane
The profile name used in PC-frame is "DEKTOT".
The original properties:
- \( A = 9.5 \text{ m}^2 \)
- \( I = 61.9 \text{ m}^4 \)
- \( W = 63.2 \text{ m}^3 \)

The PC-Frame properties:
- \( E = 210,000,000 \text{ kN/m}^2 \)
- \( A = 9.5 \text{ m}^2 \)
- \( W = 3.3 \cdot 10^{-2} \text{ m}^3 \)
- \( I = 2.4 \cdot 10^{-2} \text{ m}^4 \)

4 Transverse deck beam D
The profile name used in PC-frame is "BEAM4".
The original properties:
- \( EI = 1,680,000 \text{ kNm}^2 \)

This significant property of this beam is its bending stiffness. The other properties are taken unity.

The PC-frame properties:
- \( E = 210,000,000 \text{ kN/m}^2 \)
- \( A = 1.0 \text{ m}^2 \)
- \( W = 0.0 \text{ m}^3 \)
- \( I = 8.0 \cdot 10^{-3} \text{ m}^4 \)

5 Longitudinal deck beam A
The profile name used in PC-frame is "BEAM1".
The original properties:
- \( EI = 5,178,824 \text{ kNm}^2 \)

This significant property of this beam is its bending stiffness. The other properties are taken unity.

The PC-frame properties:
- \( E = 210,000,000 \text{ kN/m}^2 \)
- \( A = 1.0 \text{ m}^2 \)
- \( W = 0.0 \text{ m}^3 \)
- \( I = 2.4 \cdot 10^{-2} \text{ m}^4 \)

6 Longitudinal deck beam B
The profile name used in PC-frame is "BEAM2".
The original properties:
- \( EI = 3,531,901 \text{ kNm}^2 \)

This significant property of this beam is its bending stiffness. The other properties are taken unity.

The PC-frame properties:
- \( E = 210,000,000 \text{ kN/m}^2 \)
- \( A = 1.0 \text{ m}^2 \)
- \( W = 0.0 \text{ m}^3 \)
- \( I = 1.8 \cdot 10^{-2} \text{ m}^4 \)
7 Transverse deck support
The profile name used in PC-frame is HOR1. This beam represented model 1 loaded in horizontal direction. The property of importance is the lateral stiffness of model 1. The axial stiffness of HOR1 must equal the lateral stiffness of model 1. This leads to the following:

The lateral stiffness of model 1:

\[ F = u \Rightarrow k = \frac{F}{u} = \frac{1000}{0.014} = 71,429 \text{ kN/m} \]

\[ k = \frac{EA}{l} \Rightarrow A = \frac{k \cdot l}{E} = \frac{71,249 \cdot 10}{210,000,000} = 0.0034 \text{ m}^2 \]

The significant property of this beam is its axial stiffness. The properties which are not directly related to the axial stiffness are taken unity.

The PC-frame properties: 
- \( E = 210,000,000 \text{ kN/m}^2 \)
- \( A = 3.4 \cdot 10^{-3} \text{ m}^2 \)
- \( W = 1.0 \text{ m}^3 \)
- \( I = 1.0 \text{ m}^4 \)
1.4. Implementation of models in PC-frame

This section deals with the implementation of model 1 to model 4, as described in section 1.2., in PC-frame.

1.4.1. Implementation of model 1

Model 1 represents a transverse section of the jetty, as described in section 1.2.2. It consists of the framework shown in figure IV.2. The plumb and raker piles are connected to the supports and to the horizontal beam with hinges. This is not realistic but it has little influence on the results.
Upgrading of iron ore jetty

The numbers coincide with the beam types mentioned below.
1-5 : BEAM
6-8/10 : PLUMB
9 : RAKER, tension
11 : RAKER, compression

As the axial stiffness of the piles is determined by soil-pile interaction it cannot be derived from the material properties of the pile only. To adapt the axial pile stiffness to the values found in field tests one can vary the length of the beam.

**Adaptation of beam PLUMB to axial stiffness in compression**

\[ k = \frac{EA}{l} \Rightarrow l = \frac{EA}{k} = \frac{6,510,000}{248,180} = 26\text{m} \]

**Adaptation of beam RAKER to axial stiffness in compression**

\[ k = \frac{EA}{l} \Rightarrow l = \frac{EA}{k} = \frac{8,820,000}{564,440} = 16\text{m} \]

**Adaptation of beam RAKER to axial stiffness in tension**

\[ k = \frac{EA}{l} \Rightarrow l = \frac{EA}{k} = \frac{8820,000}{333,333} = 26\text{m} \]

1.4.2. Implementation of model 2

Model 2 consists of the framework shown in figure IV.3. It represents the deck loaded in plane. The vertical beams are connected to the supports and the horizontal beams with hinges. This is not realistic but it has little influence on the results.

![figure IV.3. model 2](image)

The beam types are specified below.
The horizontal beam : BEAM DEKTOT, \( l = 5 \) m.
The supporting beams: BEAM HOR 1/2, \( l = 10 \) m.
The length of the model is determined by the area which is influenced by an concentrated force acting on the horizontal beam. This area is called the characteristic length (Bouma, 1989). The characteristic length is calculated in the following.

\[ EI = 1.3 \cdot 10^{10} \, kNm^2 \]
\[ k = 71,000 \, kN/m \]
\[ \lambda = 2 \cdot \pi \cdot \sqrt{\frac{4 \cdot EI}{k}} = 260m \]

Where:
- \( EI \) = bending stiffness of BEAM DEKTOT
- \( k \) = axial stiffness of BEAM HORIZ
- \( \lambda \) = characteristic length of system

### 1.4.3. Implementation of model 3

Model 3 consists of the framework shown in figure IV.4. It represents the jetty loaded by the unloader on beam A. The vertical beams are connected to the supports and the horizontal beams with hinges. This is not realistic but it has little influence on the results.

The beam types are specified below.
The horizontal beam : BEAM A, \( l = 5 \) m.
The supporting beams: PLUMB2, \( l = 25 \) m.
The length of the model is determined by the area which is influenced by an concentrated force acting on the horizontal beam. This area is called the characteristic length (Bouma, 1989). The characteristic length is calculated in the following.

\[
EI = 5,178,824 \text{kNm}^2 \\
k = 220,000 \text{kN/m} \\
\lambda = 2 \cdot \pi \cdot \sqrt{\frac{4 \cdot EI}{k}} = 29 \text{m}
\]

Where:
- \(EI\) = bending stiffness of BEAMA
- \(k\) = axial stiffness of PLUMB2
- \(\lambda\) = characteristic length of system

1.4.4. Implementation of model 4

Model 4 consists of the framework shown in figure IV.5. It represents the jetty loaded by the unloader on beam B. The vertical beams are connected to the supports and the horizontal beams with hinges. This is not realistic but it has little influence on the results.

The beam types are specified below.
The horizontal beam : BEAMB, \(l = 5 \text{ m}\).
The supporting beams: SPRING2, \(l = 26 \text{ m}\).
The length of the model is determined by the area which is influenced by an concentrated force acting on the horizontal beam. This area is called the characteristic length (Bouma, 1989). The characteristic length is calculated in the following.

\[ EI = 3,800,000kNm^2 \]
\[ k = 883,000,000kN/m \]
\[ \lambda = 2\pi \sqrt[4]{\frac{4\cdot EI}{k}} = 27m \]

Where:
- \( EI \) = bending stiffness of BEAMB
- \( k \) = axial stiffness of SPRING2
- \( \lambda \) = characteristic length of system
1.5. **Coefficients**

This section deals with the calculation of the coefficients used to determine the maximum percentage of an overall jetty load that is absorbed by one transverse pile section (model 1).

1.5.1. **The coefficient concerning transverse horizontal load**

Model 2 is used.
- the overall load = 1000 kN
- the maximum load absorbed by one support = 136 kN
- the coefficient = 0.14

The result: $\alpha = 0.14$

1.5.2. **The coefficient concerning vertical load on beam A**

Model 3 is used.
- the overall load = 700
- the maximum load absorbed by one support = 355
- the coefficient = 0.51

The result: $\beta = 0.51$

1.5.3. **The coefficient concerning vertical load on beam B**

Model 4 is used
- the overall load = 700
- the maximum load absorbed by one support = 402
- the coefficient = 0.57

The result: $\chi = 0.57$
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The deck structure

1. Introduction

The concrete deck is a major part of the jetty. It is a composite structure. This means that the properties of the deck as a whole are determined by the properties of its components. This appendix examines the components and the resulting overall deck structure.

The components which can be distinguished are:
- longitudinal beam A;
- longitudinal beam B;
- longitudinal beam C;
- transverse beam D;
- deck slab which consists of precast concrete planks covered by in situ concrete.

The elements are indicated in figure 5.1. A detailed cross section of each element is given in the sections 1.2.1. to 1.2.4. The dead weight of the deck is determined in section 3. The bending stiffness of each element is determined in section 4. The bending stiffness of the deck loaded in plane is determined in section 5.
Upgrading of iron ore jetty

figure V.1. deck beam codes
2. Draft design drawings

2.1. Cross section of beam A

figure V.2. cross section beam A
2.2. Cross section of beam B
2.3. Cross section of beam D

Figure V.4. Cross section beam D
2.4. Side view on beam D

figure V.5. side view on beam D
3. **Dead weight of deck**

This section deals with the calculation of the dead weight of the deck as a whole. For this purpose the deck is divided into its components. The dead weight of each component is determined and added together.

**Beam A**
- \( \text{length} = 285 \, \text{m} \)
- \( \text{height} = 1.5 - 0.45 = 1.05 \, \text{m} \)
- \( \text{depth} = 2.50 \, \text{m} \)
- \( \text{volume} = 748 \, \text{m}^3 \)

**Beam B**
- \( \text{length} = 285 \, \text{m} \)
- \( \text{height} = 1.05 \, \text{m} \)
- \( \text{depth} = 3.00 \, \text{m} \)
- \( \text{volume} = 899 \, \text{m}^3 \)

**Beam C**
This beam is assumed to be incorporated in the deck slab.

**Beam D**
- \( \text{length} = 22.15 \, \text{m} \)
- \( \text{height} = 0.75 \, \text{m} \)
- \( \text{depth} = 1.2 \, \text{m} \)
- \( \text{number} = 58 \)
- \( \text{volume} = 1156 \, \text{m}^3 \)

**Slab**
- \( \text{length} = 285 \, \text{m} \)
- \( \text{height} = 0.45 \, \text{m} \)
- \( \text{depth} = 22.15 \, \text{m} \)
- \( \text{volume} = 2841 \, \text{m}^3 \)

\( \text{volume of whole deck} = 5644 \, \text{m}^3 \)

\( \text{average volumetric mass of reinforced concrete} = 24 \, \text{kN/m}^3 \)

**dead weight of whole deck** = \( 135456 \, \text{kN} \)
4. Bending stiffness of deck components

In this section the bending stiffness of beam A, B and D is determined. The material properties and the calculation method used are presented below.

The material properties
Concrete:
\( f_{cu} = 40 \, N/mm^2 \)
\[ f_{cd} = \frac{0.67 \cdot f_{cu}}{1.5} = 18 \, N/mm^2 \]

Reinforcement steel:
\( f_y = 400 \, N/mm^2 \)
\( f_{yd} = 350 \, N/mm^2 \)

Calculation method
Theory according to British Standards. The British Standards give the following design stress-strain curve for concrete.

Explanation:
\[ E = 5.5 \cdot \sqrt{\frac{f_{cu}}{\gamma_m}} = 28.4 \, kN / mm^2 \]
\[ \varepsilon_y = 2.4 \cdot 10^{-4} \cdot \sqrt{\frac{f_{cu}}{\gamma_m}} = 0.00124 \]
\[ \varepsilon_c = 0.00350 \]

If the concrete in the compression zone is maximally strained this will lead to a partly parabolic partly rectangular stress distribution. This realistic stress distribution is replaced by a rectangular stress distribution which has the same resulting force.
To perform the necessary calculations it is convenient to have a linear elastic stress-strain curve instead of a parabolic one. The primary demand which is made is that the ultimate compression force obtained from the stress-strain curve is equal to the compression force obtained with the help of the method explained above. The stress-strain curve will be as follows.

\[ f_{cd} - 0.9 \cdot x = f_{cd} - x \]

The unknown factor is the strain by which the concrete starts to deform plastically. The value can be determined by equalising the maximum concrete normal force obtained with the method of B.S. with the maximum concrete compression force obtained with the help of the curve shown above. This leads to the following.

\[ f_{cd} \cdot 0.9 \cdot x = f_{cd} \cdot x - \frac{1}{2} \cdot f_{cd} \cdot x \cdot \left( \frac{\varepsilon_y}{0.0035} \right) \]

\[ \Rightarrow \frac{\varepsilon_y}{0.0035} = 0.9 \cdot \left( f_{cd} \cdot x - f_{cd} \cdot x \right) = 0.1 \]

\[ \Rightarrow \varepsilon_y = 0.0007 \]

\[ \Rightarrow E = \frac{f_{cd}}{0.0007} = \frac{17.87}{0.0007} = 25524 \text{ N/mm}^2 \]
4.1. Bending stiffness of beam A

The picture below shows a simplified cross-section of the beam. The forces in the concrete slab are neglected.

- $A_{s1} = 26023 \text{ mm}^2$
- $A_{s2} = 8838 \text{ mm}^2$
- $A_{s3} = 19640 \text{ mm}^2$

The bending moment can act in two ways. Each way the beam has a different stiffness characteristic.

**Bending stiffness when loaded by moment 1**

Calculation of $M_y$:
The provisional assumptions:
- the reinforcement $A_{s1}$ starts to yield;
- the concrete compression zone is partially plastically deformed;
- the neutral axis lies between $A_{s3}$ and $A_{s2}$. 
The strains as a result of M1

\[
\begin{align*}
\varepsilon_c &= \frac{\varepsilon_{sl} \cdot x}{d-x} = \frac{0.00175 \cdot x}{1315-x} \\
\varepsilon_{sl} &= 0.00175 \\
\varepsilon_{s2} &= \frac{d-x-355}{d-x} \cdot \varepsilon_{sl} = \frac{0.00175 \cdot 960-x}{1315-x} \\
\varepsilon_{s3} &= \frac{x-128}{x} \cdot \varepsilon_c = \frac{x-128 \cdot 0.00175 \cdot x}{1315-x} = 0.00175 \cdot \frac{x-128}{1315-x}
\end{align*}
\]

The forces as a result of M1

\[
N_c = f_{ed} \cdot b \cdot x - \frac{1}{2} \cdot f_{ed} \cdot b \cdot x \cdot \frac{0.0007}{\varepsilon_c} = f_{ed} \cdot b \cdot x - \frac{1}{2} \cdot f_{ed} \cdot b \cdot x \cdot \frac{0.0007 \cdot (1315-x)}{0.00175 \cdot x} = 54000 \cdot x - 11836 \cdot 10^6
\]

\[
N_{s1} = E_s \cdot A_{sl} \cdot \varepsilon_{s1} = 9108050 N
\]

\[
N_{s2} = E_s \cdot A_{s2} \cdot \varepsilon_{s2} = \frac{378.97626 \cdot 10^6 - 0.39476694 \cdot 10^6 \cdot x}{1315-x} N
\]

\[
N_{s3} = E_s \cdot A_{s3} \cdot \varepsilon_{s3} = \frac{0.87725988 \cdot 10^6 \cdot x - 112.28926 \cdot 10^6}{1315-x} N
\]

The summation of the forces mentioned above must be zero. The expressions are first all multiplied by the factor \((1120-x) \cdot 10^{-6}\) then they are added up to each other and the result is equalised to zero.

\[
\begin{align*}
N_c &= 0.054 \cdot x^2 - 82.846 \cdot x + 15564.34 \\
N_{s1} &= - 910805 \cdot x + 11977.08575 \\
N_{s2} &= - 0.39476694 \cdot x + 378.97626 \\
N_{s3} &= - 0.87725988 \cdot x + 112.28926 + 0.054 \cdot x^2 - 93.22607682 \cdot x + 28032.69127
\end{align*}
\]

The results are:

\[
\begin{align*}
x &= 388 \text{ mm} \\
\varepsilon_c &= 0.00073 (>0.0007)!
\end{align*}
\]

The assumptions prove to be right!
The determination of the yielding moment when loaded by M1.

The forces are:

\[ N_c = 54000 \cdot x - 11,836 \cdot 10^6 = 9105848N \]

\[ N_{s1} = 9108050N \]

\[ N_{s2} = \frac{378,97626 \cdot 10^6 - 0,39476694 \cdot 10^6 \cdot x}{1315 - x} = 243619N \]

\[ N_{s3} = \frac{0,87725988 \cdot 10^6 \cdot x - 112,28926 \cdot 10^6}{1315 - x} = 245821N \]

The geometry of the forces.

![Diagram showing the forces and geometry](image)

Explanation:

\[ N_{c1} = \frac{1}{2} \cdot f_{ed} \cdot b \cdot x_1 = 8370000N \]

\[ N_{c2} = f_{ed} \cdot b \cdot x_2 = 720000N \]

The resulting moment becomes:

\[ M_y = N_{c1} \cdot 248 + N_{c2} \cdot 380 + N_{s1} \cdot 927 + N_{s2} \cdot 572 + N_{s3} \cdot 260 = 10995kNm \]

\[ \kappa = \frac{\varepsilon_s + \varepsilon_c}{1,315} = 0,0011m^{-1} \]
The calculation of $M_u$.

The provisional assumptions are:
- the concrete in compression starts to crack;
- the neutral axis lies in-between $A_{s3}$ and $A_{s2}$;
- all reinforcement deforms plastically.

The forces are:

- $N_c = 0.9 \cdot f_{cd} \cdot b \cdot x = 40500 \cdot x$
- $N_{s1} = A_{s1} \cdot f_{yd} = 9108050 \, N$
- $N_{s2} = A_{s2} \cdot f_{yd} = 309330 \, N$
- $N_{s3} = \frac{-13748000 \cdot x + 1759744000}{x} \, N$

The summation of the forces must be zero, this leads to the following equation.

$$-40500 \cdot x^2 - 1546650 \cdot x + 1759744000 = 0$$

The results are:
- $x = 190 \, mm$
- $\varepsilon_{s1} = 0.0207$
- $\varepsilon_{s2} = 0.0142$
- $\varepsilon_{s3} = 0.0015$

The assumptions prove to be right.

The determination of the ultimate moment:

The forces are:

- $N_c = 50400 \cdot x = 7695000 \, N$
- $N_{s1} = 9108050 \, N$
- $N_{s2} = 3093300 \, N$
- $N_{s3} = 4486189 \, N$
The geometry is:

![Geometry Diagram](image)

The resulting moment becomes:

\[ M_y = N_c \cdot 106 + N_{s1} \cdot 1125 + N_{s2} \cdot 770 + N_{s3} \cdot 106 = 13722 \text{ kNm} \]

\[ \kappa = \frac{\varepsilon_s + \varepsilon_c}{1315} = 0.0184 \text{ m}^{-1} \]

The calculations lead to the following moment-deflection curve:

![Moment-Deflection Curve](image)

The bending stiffness \((EI_r)\) of the cracked profile can be established with the help of the curve.

\[ EI_r = \frac{10995}{0.0011} = 9995455 \text{ kNm}^2 \]
Bending stiffness when loaded by M2

The calculation of $M_{y2}$.
The provisional assumptions are:
- the reinforcement $A_{s3}$ starts to yield;
- the concrete compression zone is elastically deformed;
- the neutral axis lies between $A_{s2}$ and $A_{s3}$.

The strains as a result of $M_{y2}$:
\[
e_c = \varepsilon_{3} \cdot \frac{x}{d - x} = \frac{0.00175 \cdot x}{1372 - x}
\]
\[
\varepsilon_{3} = 0.00175
\]
\[
\varepsilon_{2} = \frac{d - x - 832}{d - x} \cdot \varepsilon_{3} = 0.00175 \cdot \frac{483 - x}{1372 - x}
\]
\[
\varepsilon_{1} = \frac{x - 185}{x} \cdot \varepsilon_{c} = \frac{0.00175 \cdot x - 185}{1372 - x}
\]

The forces as a result of $M_{y2}$:
\[
N_c = \frac{1}{2} \cdot E_s \cdot \varepsilon_c \cdot b \cdot x = \frac{55833.75 \cdot x^2}{1315 - x}
\]
\[
N_{s1} = E_s \cdot A_{s1} \cdot \varepsilon_{s1} = \frac{910805 \cdot x - 168498925}{1315 - x}
\]
\[
N_{s2} = E_s \cdot A_{s2} \cdot \varepsilon_{s2} = \frac{1494063900 - 3093300 \cdot x}{1315 - x}
\]
\[
N_{s3} = E_s \cdot A_{s3} \cdot \varepsilon_{s3} = 6874000N
\]

The summation of the forces mentioned above must be zero. The expressions are first all multiplied by the factor $(1315-x) \cdot 10^{-6}$ then they are added up and the result is equalised to zero.

\[
N_c = -0.05583375 \cdot x
\]
\[
N_{s1} = -0.910805 \cdot x + 168,49825
\]
\[
N_{s2} = -3.0933 \cdot x + 1494,0635
\]
\[
N_{s3} = -6.874 \cdot x + 9039,31
\]
\[
0 = -0.05583375 \cdot x^2 - 10,878105 \cdot x + 10701,87282
\]

The results are:
\[
x = 351 \text{ mm}
\]
\[
\varepsilon_c = 0.00042 (< 0.0007)!
\]

The assumptions prove to be right!
The determination of $M_{y2}$:

The forces are:

\[ N_c = 7140346 N \]
\[ N_{s1} = 156948 N \]
\[ N_{s2} = 423294 N \]
\[ N_{s3} = 6874000 N \]

The geometry is:

The resulting yielding moment becomes:

\[ M_y = N_c \cdot 234 + N_{s1} \cdot 1021 + N_{s2} \cdot 189 + N_{s3} \cdot 223 = 8804 kNm \]

\[ \kappa = \frac{e_s + e_c}{1.372} = 0.00171 m^{-1} \]

The calculation of $M_{u2}$ as a result of $M2$

The provisional assumptions are:
- the concrete in compression starts to crack;
- the neutral axis lies in-between $A_{s1}$ and $A_{s2}$;
- all reinforcement deforms plastically.
The forces as a result of $M_{u2}$ are:

\begin{align*}
N_c &= 0.9 \cdot f_{cd} \cdot b \cdot x = 40500 \cdot x \\
N_{s1} &= A_{s1} \cdot E_s \cdot \varepsilon_s = \frac{18216100 \cdot x - 3369978500}{x} \\
N_{s2} &= A_{s2} \cdot f_{yd} = 3093300 N \\
N_{s3} &= A_{s3} \cdot f_{yd} = 6874000 N
\end{align*}

The summation of the forces must be zero. After multiplying all forces with $x \cdot 10^{-6}$, this leads to the following equation.

\[-0.040500 \cdot x^2 - 8.2488 \cdot x + 3369,978500 = 0\]

The results are:

- \(x = 204 \text{ mm}\)
- \(\varepsilon_{s1} = 0.0200\)
- \(\varepsilon_{s2} = 0.0142\)
- \(\varepsilon_{s3} = 0.0003\)

The assumptions prove to be right!

The determination of the ultimate moment:

The forces are:

\begin{align*}
N_c &= 40500 \cdot x = 8262000 N \\
N_{s1} &= 1696598 N \\
N_{s2} &= 3093300 N \\
N_{s3} &= 6874000 N
\end{align*}
The geometry is:

\[ M_u = N_c \cdot 112 + N_{s1} \cdot 19 + N_{s2} \cdot 80 + N_{s3} \cdot 1168 = 9234 \text{kNm} \]

\[ \kappa = \frac{\varepsilon_s + \varepsilon_c}{1.372} = 0.0172 \text{m}^{-1} \]

The resulting ultimate moment becomes:

\[ M_u = N_c \cdot 112 + N_{s1} \cdot 19 + N_{s2} \cdot 80 + N_{s3} \cdot 1168 = 9234 \text{kNm} \]

The calculations lead to the following moment-deflection curve:

The bending stiffness (EI_r) of the cracked profile can be established with the help of the curve.

\[ EI_r = \frac{8804}{0.0017} = 5178824 \text{kNm}^2 \]

**Summary**

<table>
<thead>
<tr>
<th>moment</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>bending stiffness, EI [kNm^2]</td>
<td>9,995,455</td>
<td>5,178,824</td>
</tr>
</tbody>
</table>
4.2. Bending stiffness of beam B

A simplified cross section of beam B is shown below. The forces in the concrete slab are neglected.

- \( A_{s1} = 22586 \text{ mm}^2 \)
- \( A_{s2} = 23077 \text{ mm}^2 \)

The bending moment can act in two ways. Each way the beam has a different stiffness characteristic. For this beam only one stiffness characteristic is determined. This is explained below.

Bending stiffness when loaded by moment 1

Calculation of \( M_y \):

The provisional assumptions are:
- the reinforcement \( A_{s1} \) starts to yield;
- the concrete loaded in compression deforms elastically;
- the neutral axis lies in-between \( A_{s1} \) and \( A_{s2} \).

The strains as a result of \( M_y \):

\[
\varepsilon_c = \frac{0.00175 \cdot x}{1400 - x}
\]

\[
\varepsilon_{s2} = 0.00175 \cdot \frac{x - 150}{1400 - x}
\]
The forces as a result of $M_y$.

\[ N_c = \left( \frac{\sqrt{2}}{2} \cdot E_c \cdot \varepsilon_c \cdot b \cdot x \right) = -\left( \frac{\sqrt{2}}{2} \cdot 25524 \cdot 0.00175 \cdot 3000 \cdot x \right) = \frac{-67000.5 \cdot x^2}{1400 - x} N \]

\[ N_{s1} = +(f_{yd} \cdot A_{s1}) = +(350 \cdot 22586) = +7905100 N \]

\[ N_{s2} = -(E_s \cdot \varepsilon_{s2} \cdot A_{s2}) = -(200,000 \cdot 0.00175 \cdot \frac{x - 150}{1400 - x} \cdot 23074) = -8076950 \cdot \frac{x - 150}{1400 - x} N \]

The summation of the aforementioned forces must be zero. The expressions are first all multiplied by the factor "1400-x" and then they are added to each other and the resulting equation is equalised to zero.

\[
\begin{align*}
N_c &= -67000.5 \cdot x^2 \\
N_{s1} &= -7905100 \cdot x + 1.1067 \cdot 10^{10} \\
N_{s2} &= -8076950 \cdot x + 1.2115 \cdot 10^9 + \cdots \\
0 &= -67000.5 \cdot x^2 - 15982050 \cdot x + 1.2279 \cdot 10^{10}
\end{align*}
\]

The results are:

- $x = 325$ mm
- $\varepsilon_{s1} = 0.00175$
- $\varepsilon_{s2} = 0.00028$
- $\varepsilon_c = 0.00053$

The assumptions prove to be right!

The resulting forces are:

\[
N_c = \frac{-67000.5 \cdot x^2}{1400 - x} = -6583189 N
\]

\[ N_{s1} = +7905100 N \]

\[ N_{s2} = -8076950 \cdot \frac{x - 150}{1400 - x} = -1314852 N \]

The resulting moment becomes:

\[
M_y = 325 \cdot N_c + (325 - 50) \cdot N_{s2} + (1400 - 325) \cdot N_{s1} =
\]

\[ = 2,139,536,400 + 2,173,902,500 + 1,413,465,900 = 5,726,904,800 = 5,727 kNm \]

\[ M = 5,727 kNm \]

\[ \kappa = 1.63 \cdot 10^{-3} \, m^{-1} \]

The resulting bending stiffness becomes:

\[ EI = 3,531,901 kNm^2 \]
Summary

The design bending stiffness of beam B:

$$EI = 3,531,901 \text{kNm}^2$$
4.3. Bending stiffness of beam D

A simplified cross section of beam D is shown below. A part of the slab is incorporated in the beam.

- $A_{s1} = 12950 \text{ mm}^2$
- $A_{s2} = 3928 \text{ mm}^2$
- $A_{s3} = 6872 \text{ mm}^2$

The bending moment can act in two ways. Each way the beam has a different stiffness characteristic.
Bending stiffness when loaded by moment 1

Calculation of $M_y$:

The provisional assumptions are:
- the reinforcement $A_{s1}$ starts to yield;
- the concrete compression zone is plastically deformed;
- the neutral axis lies between $A_{s2}$ and $A_{s3}$

The strains as a result of $M_y$:

$$
e_c = \frac{E_s \cdot A_{s1} \cdot \varepsilon_{s1} - \frac{1}{2} \cdot f_{cd} \cdot b \cdot x \cdot \varepsilon_c}{d-x} = 0,00175 \cdot \frac{x}{1080-x}$$

$$
\varepsilon_{s1} = 0,00175
$$

$$
\varepsilon_{s2} = \frac{d-x-410}{d-x} \cdot \varepsilon_{s1} = 0,00175 \cdot \frac{670-x}{1080-x}
$$

$$
\varepsilon_{s3} = \frac{x-80}{x} \cdot \varepsilon_c = \frac{x-80}{1080-x} = 0,00175 \cdot \frac{x-80}{1080-x}
$$

The forces as a result of $M_y$:

$$
N_c = f_{cd} \cdot b \cdot x - \frac{1}{2} \cdot f_{cd} \cdot b \cdot x \cdot \frac{0,0007 \cdot (1080-x)}{E_s} = f_{cd} \cdot b \cdot x - \frac{1}{2} \cdot f_{cd} \cdot b \cdot x \cdot \frac{0,0007 \cdot (1080-x)}{0,00175 \cdot x} =
$$

$$
= 25920 \cdot x - 4665600
$$

$$
N_{s1} = E_s \cdot A_{s1} \cdot \varepsilon_{s1} = 4532500 N
$$

$$
N_{s2} = E_s \cdot A_{s2} \cdot \varepsilon_{s2} = \frac{921,116 \cdot 10^6 - 1,3748 \cdot 10^6 \cdot x}{1080-x} N
$$

$$
N_{s3} = E_s \cdot A_{s3} \cdot \varepsilon_{s1} = \frac{2,4052 \cdot 10^6 \cdot x - 192,416 \cdot 10^6}{1080-x} N
$$

The summation of the forces mentioned above must be zero. The expressions are first all multiplied by the factor $(1120-x) \cdot 10^{-6}$ then they are added up and the result is equalised to zero.

$$
N_c = 0,025920 \cdot x^2 - 32,6592 \cdot x + 5038,848
$$

$$
N_{s1} = - 4,5325 \cdot x + 4895,1
$$

$$
N_{s2} = - 1,3748 \cdot x + 921,116
$$

$$
N_{s3} = - 2,4052 \cdot x + 192,416
$$

$$
0 = 0,025920 \cdot x^2 - 40,9717 \cdot x + 11047,48
$$
The results are:
\[ x = 345 \text{ mm} \]
\[ \varepsilon_c = 0,0008 (> 0,0007)! \]

The assumptions prove to be right!

The determination of the yielding moment when loaded by \( M_y \).

The forces are:

\[ N_c = 25920 \cdot x - 4838400 = 4276800 \, N \]
\[ N_{s1} = 4532500 \, N \]
\[ N_{s2} = \frac{811,132 \cdot 10^6 - 1,3748 \cdot 10^6 \cdot x}{1120 - x} = 607905 \, N \]
\[ N_{s3} = \frac{2,4052 \cdot 10^6 \cdot x - 192,416 \cdot 10^6}{1120 - x} = 867181 \, N \]

The geometry of the forces.

Explanation:
\[ N_{c1} = \frac{1}{2} \cdot f_{ed} \cdot b \cdot x_1 = 3261600 \, N \]
\[ N_{c2} = f_{ed} \cdot b \cdot x_2 = 993600 \, N \]

The resulting moment thus becomes:
\[ M_y = N_{c1} \cdot 201 + N_{c2} \cdot 325 + N_{s1} \cdot 735 + N_{s2} \cdot 325 + N_{s3} \cdot 265 = 4737 \, kNm \]
\[ \kappa = \frac{\varepsilon_c + \varepsilon_e}{1.2} = 0,0021 \, m^{-1} \]
The calculation of $M_{u2}$.

The provisional assumptions are:
- The concrete in the compression zone starts to crack.
- The neutral axis lies in-between $A_{S2}$ and $A_{S3}$
- All reinforcement deforms plastically.

The strains as a result of $M_{u1}$.

\[ \varepsilon_c = 0.0035 \]
\[ \varepsilon_{s1} = 0.0035 \cdot \frac{1080 - x}{x} \]
\[ \varepsilon_{s2} = \frac{670 - x}{1080 - x} \cdot \varepsilon_{s1} = 0.0035 \cdot \frac{670 - x}{x} \]
\[ \varepsilon_{s3} = \frac{x - 80}{x} \cdot \varepsilon_c = 0.0035 \cdot \frac{x - 80}{x} \]

The forces as a result of $M_{u1}$.

\[ N_c = 0.9 \cdot f_{cd} \cdot b \cdot x = 19440 \cdot x \]
\[ N_{s1} = E_s \cdot A_{s1} \cdot \varepsilon_{s1} = 4532500 N \]
\[ N_{s2} = E_s \cdot A_{s2} \cdot \varepsilon_{s2} = 1374800 N \]
\[ N_{s3} = E_s \cdot A_{s3} \cdot \varepsilon_{s3} = 2405200 N \]

The summation of the forces must be zero. This leads to the following equation.

\[ 3502100 - 19440 \cdot x = 0 \]

The results are:
- $x = 180 \text{ mm}$
- $\varepsilon_{s1} = 0.0175$
- $\varepsilon_{s2} = 0.0095$
- $\varepsilon_{s3} = 0.0019$

The previous assumptions prove to be correct.

The determination of the ultimate moment when loaded by $M_{u1}$.
The forces are:

\[ N_c = 39499200 \, N \]
\[ N_{s1} = 4532500 \, N \]
\[ N_{s2} = 1374800 \, N \]
\[ N_{s3} = 2405200 \, N \]

The geometry is:

![Diagram of forces and geometry](image)

The resulting moment is:

\[ M_y = N_c \cdot 100 + N_{s1} \cdot 900 + N_{s2} \cdot 490 + N_{s3} \cdot 100 = 4997 \, kNm \]

\[ \kappa = \frac{\varepsilon_s + \varepsilon_c}{1.2} = 0.0175 \, m^{-1} \]

The calculations lead to the following moment-deflection curve:

![Moment-deflection curve](image)

The bending stiffness \( (EI_r) \) of the cracked profile can be established with the help of the curve.

\[ EI_r = \frac{4737}{0.0021} = 2255714 \, kNm^2 \]
Bending stiffness when loaded by M2

Calculation of $M_{y2}$.

The provisional assumptions:
• the concrete compression zone is elastically deformed;

The strains as a result of $M_{y2}$.

$$\varepsilon_e = \frac{\varepsilon_{s3} \cdot x}{d - x} = 0,00175 \cdot \frac{x}{1120 - x}$$

$$\varepsilon_{s1} = \frac{x - 120}{x} \cdot \frac{\varepsilon_e}{x} = \frac{- 0,00175 \cdot x}{1120 - x}$$

$$\varepsilon_{s2} = \frac{d - x - 590}{d - x} \cdot \frac{\varepsilon_{s3}}{1120 - x} = 0,00175 \cdot \frac{530 - x}{1120 - x}$$

$$\varepsilon_{s3} = 0,00175$$

The forces as a result of $M_{y2}$.

$$N_e = \frac{1}{2} \cdot E_e \cdot \varepsilon_e \cdot b \cdot x = \frac{1}{2} \cdot E_e \cdot b \cdot x \cdot \frac{0,00175 \cdot x}{1120 - x} =$$

$$= \frac{41093,64 \cdot x^2}{1120 - x}$$

$$N_{s1} = E_s \cdot A_{s1} \cdot \varepsilon_{s1} = 1,78232 \cdot 10^6 \cdot 0,00175 \cdot \frac{x - 120}{1120 - x} = \frac{31,1906 \cdot 10^6 \cdot x - 3742,872 \cdot 10^6}{1120 - x}$$

$$N_{s2} = E_s \cdot A_{s2} \cdot \varepsilon_{s2} = 2405900 \cdot \frac{530 - x}{1120 - x} = \frac{1,275127 \cdot 10^9 - 2405900 \cdot x}{1120 - x}$$

$$N_{s3} = E_s \cdot A_{s1} \cdot \varepsilon_{s3} = 2405200 N$$

The summation of the forces mentioned above must be zero. The expressions are first all multiplied by the factor (1120-x)-10^6 then they are added up and the result is equalised to zero.

$$N_e = - 0,04109364 \cdot x^2$$

$$N_{s1} = \quad - 31,1906 \cdot x \quad + 3742,872$$

$$N_{s2} = \quad - 2,405900 \cdot x \quad + 1275,127$$

$$N_{s3} = \quad - 2,405200 \cdot x \quad + 2693,824$$

$$0 = - 0,04109364 \cdot x^2 \quad - 36,0017 \cdot x \quad + 7711,823$$

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Upgrading of iron ore jetty

The results are
\( x = 178 \text{ mm} \)
\( \varepsilon_c = 0,00031 (<0,0007) \)

The assumptions prove to be right!

The determination of the yielding moment when loaded by \( M_{y2} \).

The forces are:

\[
N_c = \frac{41093,6 \cdot x^2}{1120 - x} = -1382177 N
\]

\[
N_{s1} = \frac{31,1906 \cdot 10^6 \cdot x - 3742,872 \cdot 10^6}{1120 - x} = -1920440 N
\]

\[
N_{s2} = \frac{1,275127 \cdot 10^9 - 2405900 \cdot x}{1120 - x} = 899020 N
\]

\[N_{s3} = 2405200 N\]

The geometry of the forces is:

The resulting moment becomes:

\[
M_y = N_c \cdot 119 + N_{s1} \cdot 58 + N_{s2} \cdot 352 + N_{s3} \cdot 942 = 2858 kN m
\]

\[
\kappa = \frac{\varepsilon_s + \varepsilon_c}{h} = \frac{0,00175 + 0,00033}{1,2} = 0,0017 m^{-1}
\]
The calculation of $M_{u2}$.

The provisional assumptions are:
- the maximum strain of the concrete compression zone is $3.5 \%$;
- the concrete compression force is determined according to the B.S. 8110;
- the neutral axis lies in the concrete slab;
- the reinforcement $A_{x2}$ will yield also.

The strains as a result of $M_{u2}$:

$$\varepsilon_c = 0.0035$$

$$\varepsilon_{s1} = \frac{x - 120}{x} \cdot \varepsilon_c = 0.0035 \cdot \frac{x - 120}{x}$$

$$\varepsilon_{s2} = \frac{d - x - 590}{d - x} \cdot \varepsilon_{s3} = 0.0035 \cdot \frac{1120 - x}{x} \cdot \frac{530 - x}{x} = 1.855 - 0.0035 \cdot \frac{x}{x}$$

$$\varepsilon_{s3} = \frac{1120 - x}{x} \cdot \varepsilon_c = 0.0035 \cdot \frac{1120 - x}{x}$$

The forces as a result of $M_{u2}$:

$$N_c = 0.9 \cdot f_{cd} \cdot b \cdot x = 29808 \cdot x N$$

$$\varepsilon_{s1} = E_s \cdot A_{s1} \cdot \varepsilon_{s1} = 1.7832 \cdot 10^9 \cdot 0.0035 \cdot \frac{x - 120}{x} = 6241200 \cdot x - 0.748944 \cdot 10^6 \frac{N}{x}$$

$$N_{s2} = E_s \cdot A_{s2} \cdot \varepsilon_{s2} = 2.4059 \cdot 10^6 \frac{N}{x}$$

$$N_{s3} = E_s \cdot A_{s3} \cdot \varepsilon_{s3} = 2.4059 \cdot 10^6 \frac{N}{x}$$

The summation of the forces mentioned above must be zero. The expressions are first all multiplied by the factor $x \cdot 10^{-6}$ then they are added up and the result is equalised to zero.

$$N_c = -0.029808 \cdot x^2$$

$$N_{s1} = -6.2412 \cdot x + 748,944$$

$$N_{s2} = +2.4059 \cdot x$$

$$N_{s3} = +2.4059 \cdot x$$

$$0 = -0.029808 \cdot x^2 - 1.4294 \cdot x + 748,944$$

The results are:
- $x = 136 \text{ mm} (< 270 \text{ mm})$
- $\varepsilon_{s3} = 0.02 (< 0.04)$
- $\varepsilon_{s2} = 0.01 (> 0.00175)$

The assumptions made can be considered correct.
The determination of the ultimate moment when loaded by $M_2$.

The forces are:

$$N_c = -29808 \cdot x = -4053888 \, N$$

$$N_{s1} = \left[ \frac{6241200 \cdot x - 0,748944 \cdot 10^9}{x} \right] = -734259 \, N$$

$$N_{s2} = 2405900 \, N$$

$$N_{s3} = 2405900 \, N$$

The geometry of the forces is:

The resulting moment becomes:

$$M_y = N_c \cdot 75 + N_{s1} \cdot 16 + N_{s2} \cdot 394 + N_{s3} \cdot 984 = 3638 \, kNm$$

$$\kappa = \frac{e_s + e_c}{h} = \frac{0,0253 + 0,0035}{1,2} = 0,024 \, m^{-1}$$
The calculations result in the following moment-deformation curve.

The bending stiffness \( (EI_r) \) of the cracked profile can be established with the help of the curve.

\[
EI_r = \frac{2858}{0.0017} = 1681176 \text{kNm}^2
\]

**Summary**

<table>
<thead>
<tr>
<th>moment</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>bending stiffness, EI [kNm(^2)]</td>
<td>2,255,714</td>
<td>1,681,176</td>
</tr>
</tbody>
</table>

**4.4. Summary**

<table>
<thead>
<tr>
<th>beam</th>
<th>A</th>
<th>B</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>bending stiffness, EI [kNm(^2)]</td>
<td>5,178,824</td>
<td>3,531,901</td>
<td>1,681,176</td>
</tr>
</tbody>
</table>
5. Bending stiffness of deck loaded in plane

The transverse cross section of the deck can be modelled as shown in figure V.6.

![Transverse cross section of deck](image)

The blocks A1 to A3 represent the beams A to C as indicated in the introduction.

**Area**

\[
A1 = 2.5 \times 1.5 = 3.75 m^2 \\
A2 = 3.0 \times 1.5 = 4.5m^2 \\
A3 = 1.2 \times 0.8 = 1.0m^2
\]

**Elasticity modulus, E**

\[ E_b = 28 \times 10^6 \text{kN/m}^2 \]

**Centre of mass, z**

\[
z = \frac{12.5 \cdot A2 + 18.8 \cdot A3}{A1 + A2 + A3} = 7.9m
\]

**Moment of resistance, W**

\[ W = 7.9 \cdot A1 + 4.6 \cdot A2 + 10.9 \cdot A3 = 63.2m^3 \]

**Moment of inertia, I and EI**

\[
I = 7.9^2 \cdot A1 + 4.6^2 \cdot A2 + 10.9^2 \cdot A3 = 464m^4 \\
EI = 1.3 \times 10^{10} \text{ kNm}^2
\]