Design of an

Offshore LNG Import Terminal

MSc. thesis

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Design of an

Offshore LNG Import Terminal

Master thesis of

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Preface

This report is the Master thesis of Hein Oomen, student at Delft University of Technology, Faculty of Civil Engineering & Geosciences, Sub faculty Civil Engineering, Section Ports and Waterways. It consists of three main parts (i.e. Preliminary Study, Conceptual design and Technical Feasibility) preceded by an introduction and complemented by conclusions and recommendations.

This thesis has been carried out under guidance of Shell Global Solutions, The Hague at the business group Civil, Storage and Marine Engineering (OGEC) and in co-operation with Delta Marine Consultants, Gouda.

I would like to thank the members of my graduation committee for their valuable comments and support. Furthermore I would like to thank Mike Said, Harke Jan Meek, Joram Meijerink, Chris van den Berg, Dave Thomson, Clemens van der Valk, Mike Persaud and John van Beek (Shell), as well as Leon Groenewegen, Peter Bos (DMC), Bas Buchner and Adri van Dijk (MARIN) for sharing their experience and providing me with valuable input. Finally I would like to thank my friend Klaas for creating an impressive 3D model of my terminal design, which is shown on the front page of this report.

Hein Oomen

Delft, October 2002.

Executive summary

General

Natural gas (NG) is a hydrocarbon mixture consisting primarily of methane and ethane, both of which are gaseous under atmospheric conditions. After processing, the gas is cooled and converted to liquid at –162° C (LNG) to be transported by insulated tankers. At its destination, the LNG import terminal, the LNG is transferred from the moored ship to shore through pipelines connected to the ship's manifold. On shore the LNG is stored in insulated storage tanks, after which it can be 'regasified' and supplied to the customer.

At the present time the construction of conventional onshore LNG import terminals encounters difficulties due to perception of unacceptable risk to public safety, long permitting processes and/or local environmental issues. An offshore terminal may offer a solution to overcome these problems.

Already several studies on offshore LNG terminals have been carried out. In this thesis the feasibility of alternative, more cost effective concepts for offshore LNG import terminals will be discussed. The terminal should be based on a throughput of 5 million tonnes per annum and comprise of a fixed offshore LNG storage facility of 200,000 m^3 with regasification equipment for high-pressure gas send-out and a mooring system that provides maximised terminal operability without constructing breakwaters.

Site location

A site location in the vicinity of Boston, Massachusetts will be used as a base case for the design of the terminal. A preliminary site selection study has determined that a location near Gurnet Point, situated at the north-west side of Cape Cod Bay, is most suitable for the offshore terminal.

Environmental data has been collected from several sources such as wave buoys and satellite data. The offshore wave climate has been translated to local conditions using the two-dimensional wave model SWAN. The most important environmental design criteria for the selected site are given below.

Water depth (min – max)	15.0 – 19.2 m
Significant wave (100 year return period)	H_s = 7.9 m, T_p = 16 s
Wind (100 year return period)	24.9 m/s
Max. current velocity	0.5 m/s
Soil conditions	Loose/medium dense sand, ϕ = 30°, ρ = 18 – 20 kN/m^3

Environmental design criteria for site location

LNG Storage facility

For the design of the storage facility a number of conceptual design choices have to be made:

- The foundation of the offshore storage tank will be gravity based, because other options require either large quantities of dredging or complex offshore operations. Ballast compartments in the structure will provide sufficient on-bottom stability and can serve as a safety buffer against ship collision at the same time.

- One single LNG storage tank is preferred above several smaller tanks with the same total volume because of rapidly increasing costs for LNG containment material.
- The storage tank will have a prismatic shape because, due to the available draft, the height of the tank is limited. A cylindrical tank of 200,000 m^3 is therefore not feasible due to the large span length required for the roof. With a rectangular shaped tank this required roof span can be reduced.
- The caisson will be constructed in pre-stressed, reinforced concrete. The membrane-type containment system will be used to form the actual storage tank.

The design parameters of the storage tank have been optimised. By defining net storage volume, minimum height for overtopping, maximum span width, maximum caisson length and maximum draft afloat as boundary conditions, the optimal design values have been determined while checking the on-bottom and marine stability as well as the local and global structural strength during decisive combinations of external loads.

After checking the sensitivity of the design parameters the optimal solution, with minimised material costs (concrete and membrane), has been identified:

Inner length x width x height [m]	148.6 x 57.7 x 26.6
Outer length x width x height [m]	162.3 x 71.5 x 38.1
Draft of floating caisson [m]	14.5
Concrete volume [m^3]	60,447
Ballast volume (sand) [m^3]	134,099
Estimated material costs [million USD]	84
Costs per cubic meter storage [USD]	420

Optimal values for caisson design parameters

The storage tank will be built in a purpose built graving dock onshore. After completion the caisson will be floated out and towed to the site location, where it will be ballasted with wet sand to provide sufficient on-bottom stability.

Process equipment

One of the major problems offshore is the lack of space for safe placing of equipment for regasification compared to onshore terminals. It is not feasible to apply the conventional philosophy of safety distances between hazardous components.

By analysing the operational process on the terminal, dividing components into low and high risk areas, applying active and passive safety measures and providing sufficient access for construction and maintenance, a preliminary layout for the process equipment deck has been defined for an area of 150 x 70 meters.

Mooring configuration

The purpose of mooring systems is to hold the ship accurately in position while the unloading process takes place via the loading arms. Depending on the local environmental conditions and the required operability, the optimal mooring concept with appropriate loading arm can be selected. Due to the exposed location, conventional berths with a fixed vessel heading do not provide sufficient operability. The operability of a soft-yoke mooring with limited weathervaning capability has been analysed.

The following downtime criteria have been assumed:

Max. mooring force	200 ton
Max. yaw	45°
Max. roll	1.5°
Max. pitch	1.5°
Min. distance vessel – caisson	82 m

Downtime criteria for soft-yoke system

The moored ship behaviour has been modelled for the local environmental conditions using the simulation program TERMSIM. From the results can be concluded that in this case the mean heading of the vessel is mainly determined by the current. The optimal orientation for the terminal is at 105 degrees (0 being north).

For the time series considered, the number of occurrences where the limitations for maximum mooring force or maximum vessel motions are exceeded is negligible. However, due to the wide spreading in directionality of the wind and waves, the criterion for minimum distance between vessel and caisson is frequently exceeded (5.4% of the time).

Because of the persistency of the service time a window of 18 hours of up-time is required for the unloading process. The resulting operability of the terminal then becomes 62%.

When either flexible dolphins, a DP system or tug assistance are incorporated in the mooring configuration, the number of downtime events, caused by the distance criterion, will be reduced significantly. Because the other downtime criteria hardly generate additional downtime, it is expected that in that case terminal will have a much higher operability.

Conclusions and recommendations

The proposed offshore LNG import terminal with a throughput of 5 mtpa comprising of:

- a reinforced concrete GBS with 200,000 m^3 storage capacity,
- regasification equipment for 1650 m3/hour peak send-out installed on top,
- connected with a 180 m long jetty to a jacket-based soft-yoke with limited weathervaning capability,

has a terminal operability of 62%, an average waiting time of 11.1 hours and 26% chance of interruption of gas send-out (buffer under-run) when regarding the environmental conditions at the selected site location near Boston.

Compared to other offshore LNG import terminal concepts, there is a potential saving on material costs with respect to the LNG storage facility. However these savings may be nullified, or even changed into additional expenses, by the increased costs for the jetty and the complex soft-yoke mooring system.

An expensive mooring system may be justified when it results in high terminal operability. For the considered site location this is not the case, unless additional improvements, such as flexible fender dolphins, DP systems or tug assistance, are incorporated. Therefore, without such improvements, the suggested terminal concept is considered not to be a cost-effective solution for Boston.

It is recommended to investigate the operability of an offshore LNG import terminal featuring a fixed storage/regasification facility and an improved soft-yoke mooring system for other site locations with different environmental conditions.

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List of abbreviations

BOG	Boil-Off Gas
CAPEX	Capital Expenditure
CBM	Conventional Buoy Mooring
CD	Chart Datum
DP	Dynamic Positioning
DWT	Dead Weight Tonnage
ESD	Emergency Shut Down
ESHA	Environmental, Social and Health Impact Assessment
FSRU	Floating Storage and Regasification Unit
GBS	Gravity Based Structure
IFV	Intermediate Fluid Vaporiser
LAT	Lowest Astronomical Tide
LNG	Liquefied Natural Gas
LNGC	Liquefied Natural Gas Carrier
LOA	Length Over All
MBM	Multiple Buoy Mooring
mtpa	Million Tonnes Per Annum
NOL	Normal Operating Level
OPEX	Operational Expenditure
ORV	Open Rack Vaporiser
QRA	Quantitative Risk Assessment
RQD	Rock Quality Designation
SCV	Submerged Combustion Vaporiser
SPM	Single Point Mooring
ULS	Ultimate Limit State

1 Introduction

Although natural gas has been used since ancient times, it did not become an important source of energy until the 1930s, when improved pipeline technology allowed natural gas to compete with town gas produced from coal. Since that time, natural gas has been exploited increasingly as a residential and industrial fuel, particularly since the oil crises of the 1970s. Discoveries of major natural-gas fields in Western Europe, Russia, North Africa, and the Middle East have contributed to this trend.

Natural gas continued to make inroads into energy markets previously dominated by oil. The fuel received a big boost in 1997 when countries attending the international climate-change conference in Kyoto, Japan, voted to impose legally binding targets for the reduction of greenhouse gases. One of the main ways to reduce such emissions was to replace coal-fired electricity-generation plants with those that used natural gas.

Natural gas is a hydrocarbon mixture consisting primarily of methane and ethane, both of which are gaseous under atmospheric conditions. Producers obtain natural gas by extracting it through wells drilled into the earth. After processing, the remaining gas, consisting almost entirely of methane, is transported either by pipeline or tankers.

For shipment, the gas is cooled and converted to liquid at –162° C and is then pumped aboard a tanker for transit in stainless steel tanks that are surrounded by heavy insulation to prevent absorption of heat and to keep the liquid from evaporating during the voyage. The liquefied natural gas (LNG) then occupies only about 1/600 of the volume of the gas.

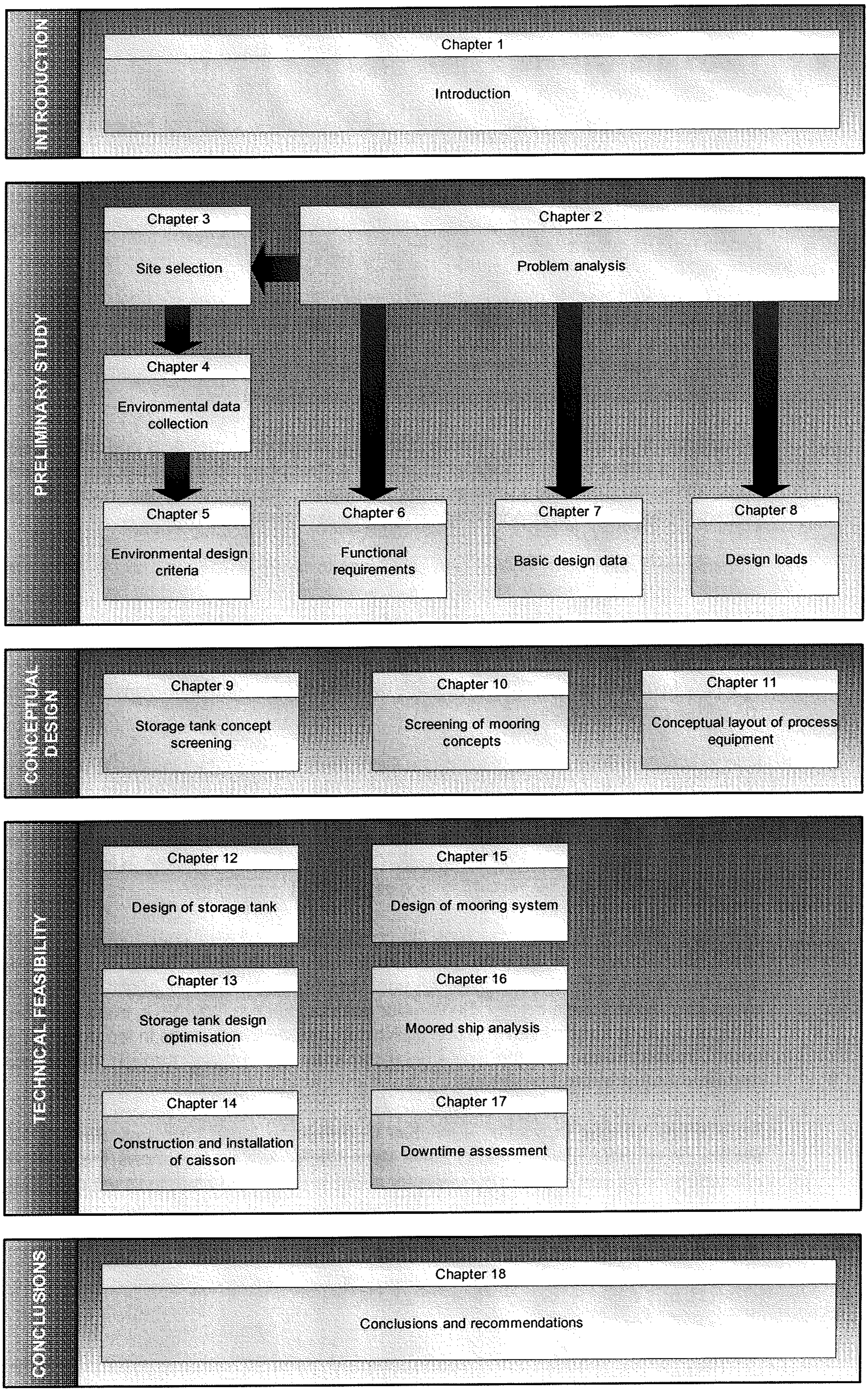
The country that imports the LNG requires an LNG import terminal to transfer the LNG from the carrier (LNGC) to shore.

Due to the hazardous nature of the cargo these terminals are either located in a remote area of an existing port, or at a separate, dedicated location. Depending on the site conditions such as wind, waves and currents the terminal may require additional breakwaters to protect the moored ships. Generally the LNG is transferred from the moored ship to shore through pipelines connected to the ship's manifold. On shore the LNG is stored in insulated storage tanks, after which it can be 'regasified' and supplied to the customer.

At the present time in some countries the construction of conventional onshore LNG import terminals encounters a growing public resistance to terminals being located close to populated areas with a perception of unacceptably high risks to public safety. This is particularly evident since the 11th September terrorist attack in New York. In addition the land required to site a terminal can often be prohibitively expensive for sites in the vicinity of major urban developments and the implementation may be delayed through the associated environmental permitting processes.

An offshore terminal may offer a solution to overcome these problems. Already several studies on offshore LNG terminals have been carried out. In this thesis the feasibility of alternative, more cost effective concepts for offshore LNG import terminals will be discussed. A site location in the vicinity of Boston, Massachusetts will be used as a base case.

The structure of this report is explained in the document map on the next page.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2 Problem description

2.1 Problem analysis

2.1.1 Business drivers for offshore terminals

To date, LNG Import terminals have always been built onshore. In some cases however, an offshore terminal seems an attractive alternative. There are a number of reasons to choose for an offshore solution rather than the conventional terminal with an exposed jetty:

- When a terminal is planned in a densely populated area, problems might occur applying the risk contours regarding the safety of the population. Even if all safety requirements are met, the public opinion could hinder the development of the terminal.
- Due to a possibly shorter overall implementation time the offshore alternatives can be interesting if the time span between the 'Final Investment Decision' and the "First Gas to Customer' is limited, as is the case for "First Gas to Customer Wins".
- Local environmental issues could slow down or prevent the development of an onshore terminal.
- If the terminal will be built in a country with an unstable political climate, it is attractive to have the possibility of withdrawal with minimal capital loss.

2.1.2 Downtime

The offshore conditions (wind, wave and currents) in the open sea without the protection of a main breakwater will be harsh. Therefore the construction of an offshore terminal as well as the moored ships will have to deal with much higher loads and motions. When these conditions exceed certain limits (e.g. a storm) the ship cannot make a safe approach to the berth, continue unloading if in the berth or even remain in berth depending on the conditions. This is defined as "downtime". Too much downtime means that the operability of the terminal is too low, hence that it is economically unattractive. The downtime of an offshore terminal can be reduced by the design of the moorings, allowing the ship to weathervane, providing some protection on the leeside of the offshore construction or by providing a main breakwater. Because breakwaters are very expensive, this study will focus on possible offshore concepts with minimised downtime without the use of breakwaters.

2.1.3 Regasification

The natural gas is transported by the ship as liquid (LNG) at -162° C, which has to be converted to gas (regasification) before it can be transported to the customer. This is usually done at a regasification plant onshore comprising LNG storage tanks, offloading system, regasification equipment and send-out pipelines. The possibility to regasify onboard an LNGC has potential but requires significant ship modifications. Moreover, the capacity will be limited to the storage capacity of an existing LNGC and interruptions in the send-out will occur more often. Transferring LNG from the offshore terminal to shore and regasifying it in a conventional onshore regasification plant is not considered a feasible option, because economically attractive concepts for subsea cryogenic (insulated) pipelines have not been developed yet.

However, there is no technical reason why the regasification facilities cannot be integrated into an offshore design. This study will therefore focus on solutions based on an offshore regasification plant.

2.1.4 Continuity of supply

When the LNG import terminal lacks a storage facility, the gas scheme will be interrupted from the moment that a ship is disconnected and leaves the terminal until next ship has moored and starts to unload. The gas market needs to be able to cope with these interruptions in the form of for example alternative fuel. An LNG storage tank at the offshore terminal will function as a buffer for the interruptions, thus providing a more continuous gas send-out and more attractive sales terms for the customers. This study will focus on concepts including an offshore LNG storage facility.

2.1.5 Floating or fixed

Already several studies on offshore LNG terminals have been carried out, although none of them have actually been constructed yet. Mainly these studies can be divided into floating (anchored barges or SPM's) and non-floating (platform on piles or gravity based structures) concepts. This study will be limited to non-floating storage and regasification solutions.

2.2 Problem definition

In a typical situation an offshore solution is unlikely to be competitive on cost alone compared to a conventional onshore terminal. However in projects that are politically, environmentally or time-constrained, an offshore LNG import terminal could provide a better solution. The problem is the lack of a fixed offshore concept including storage and regasification facilities which is an acceptable alternative for onshore terminals.

2.3 Objective

The objective of this study is to present the most economical design for a non-floating offshore LNG receiving terminal including an LNG storage tank and a regasification facility. To be able to compete with conventional onshore terminals, the downtime of this design concept should be kept minimal, without the use of breakwaters.

2.4 Starting points

2.4.1 Functional requirements

- The yearly throughput should be 5 million ton LNG (5 mtpa).
- The terminal should provide a fixed offshore storage facility for 200,000 m^3 of LNG.
- The terminal berth should be capable of handling 125,000 - 145,000 m^3 LNGC's.
- LNG should be vaporised at an offshore regasification plant.
- A single high-pressure gas send-out capability is required.
- The operability of the terminal should be maximised.
- Future expansion of the terminal must be possible.

2.4.2 Technical boundary conditions

- The construction schedule has a preferred maximum of 3 years.
- The design life of the total construction is 30 years.
- The construction of the terminal should be modular.

- No floating storage concepts and no use of breakwaters.

2.4.3 Natural boundary conditions

As a case study, the concept will be designed for the environmental conditions along the coast of Boston, USA. However this study will be focused on the innovative offshore design and its feasibility, rather than providing the optimal solution for the Boston case.

3 Site selection

3.1 General

The purpose of the new offshore import terminal in the vicinity of Boston is to receive LNG and convert it to natural gas, which then can be transferred via an offshore pipeline to the gas transportation network surrounding Boston. When looking at the coastline of the greater Boston area, bearing in mind the depth requirements for fixed offshore terminals (approximately 15 meters minimal is required for the draught of the LNGC), four possibly favourable locations can be selected, which are shown in Figure 3-1 below:

- Ipswich Bay, 42°41' N, 70°42' W
- Gurnet Point, 42°00' N, 70°36' W
- Vineyard Sound, 41°21' N, 70°54' W
- Block Island, 41°17' N, 71°35' W

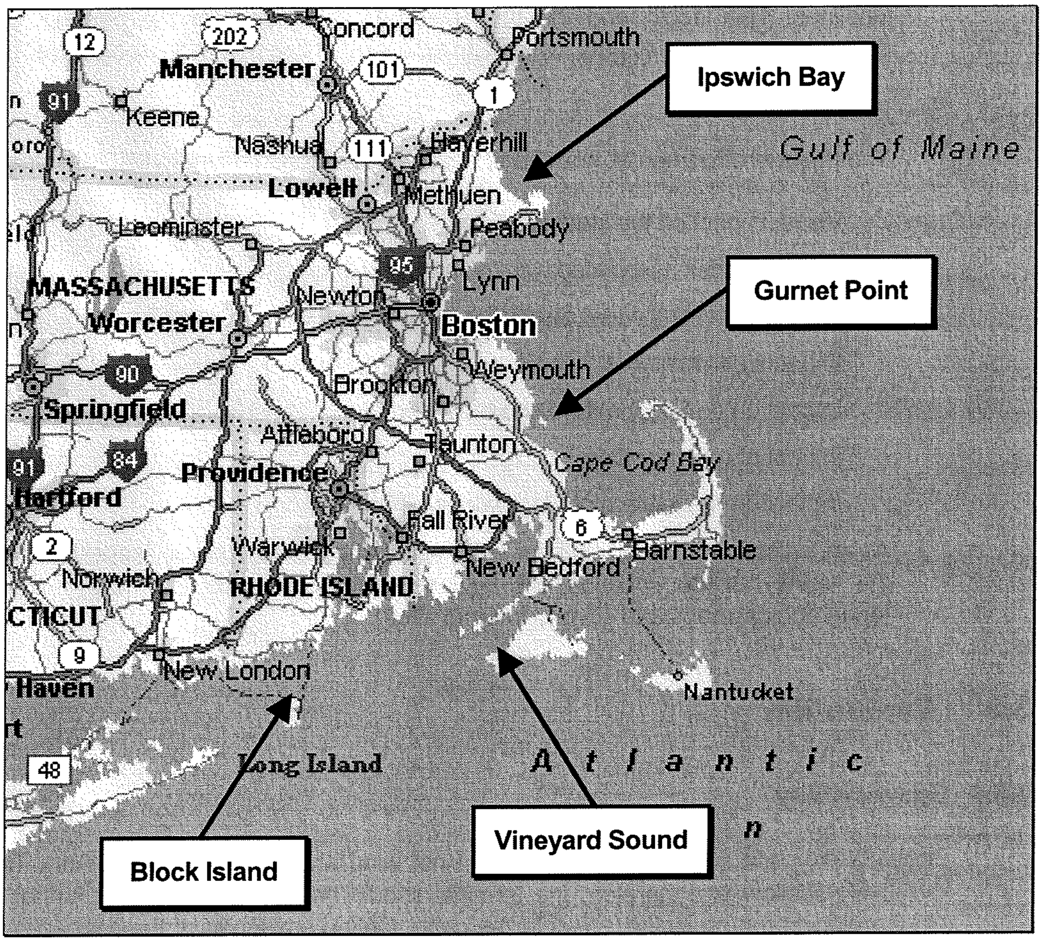


Figure 3-1 Possible site locations [Ref 27]

3.2 Criteria

Because detailed information about local site conditions, government regulations, political preferences and commercial interests is lacking, these four options will be roughly evaluated based on the following main criteria:

- Shelter for metocean conditions

  The prevailing wind direction along the coast is west to south-west. Therefore the greater part of the wind-generated waves can be expected from the south-west direction. Also most of the hurricanes originated from the south-west. However the swell is more coming from a south-east direction. The locations will be evaluated on the amount of protection against these conditions provided by land or shallow waters.

- Pipeline construction costs

  It will be assumed that a gas pipeline will have to be constructed from the offshore terminal location to Boston City, for it is unlikely that the existing transporting pipelines can accommodate the extra throughput. Also it is unclear at this stage if the supplied regasified LNG will be of the same specification as is transported via the existing pipelines. The length and the type (submarine or land) of the pipeline will mainly determine the pipeline costs, which can be an important part of the total project costs.

- Environmental impact

  The major environmental, social and health impacts of an offshore LNG terminal are:

  - Visual impact on the landscape
  - Perceived risk associated with the handling of LNG
  - Impact of heating water discharge (used by the vaporisers)
  - Impacts associated with increased shipping activity
  - Impact on sea-bottom and shore caused by construction of terminal and pipeline
  - Impact of air emissions
  - Impact on marine ecology e.g. whale population

  As an indication of the impact, for each option the environmentally sensitive areas within the vicinity of the location will be assessed, based on information provided by topographical maps and several Internet sites.

3.3 Evaluation

3.3.1 Ipswich Bay

Ipswich Bay is a small bay protected against wind and waves by Cape Ann. The bay is relatively shallow and therefore the terminal should be located somewhat further offshore, thus reducing the protective effect of the cape. Only a small part of the connecting pipeline will have to be constructed submarine, the rest will lead through some marshes directly into Boston. The total pipeline length amounts to approximately 60 km. The proposed terminal will be located near (< 5 km) the Parker River National Wildlife Refuge, which features about 300 species of birds, wildlife, dunes and marshes.

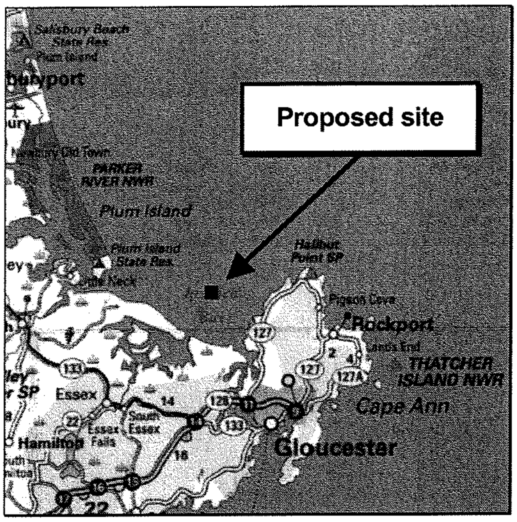


Figure 3-2 Ipswich Bay [Ref 42]

3.3.2 Gurnet Point

Gurnet Point is located near the town of Plymouth on the west coast of Cape Cod Bay. Wind and wave action is reduced considerably by the protection of Cape Cod. After a short submarine pipeline to the shore, about 75 km of pipeline through some marshes will connect the terminal with Boston. The nearby town of Plymouth is known for its whale watching activities.

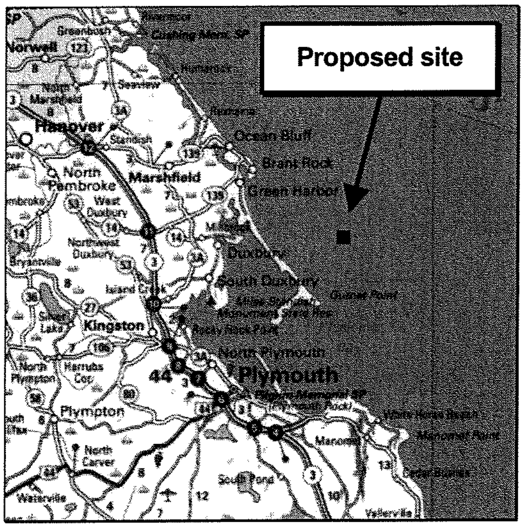


Figure 3-3 Gurnet Point [Ref 42]

3.3.3 Vineyard Sound

To the south-east of the city of Providence, at the south coast of Rhode Island, lies an island called Martha's Vineyard. The Vineyard Sound, north of the island, provides a relatively sheltered location. A long submarine pipeline (about 25 km) is required to connect the terminal to the shore. The total length of the pipeline connection to the Boston area amounts to 125 km. There are no significant parks or nature reserves in the vicinity of the site. However a very prosperous residential area is located nearby.

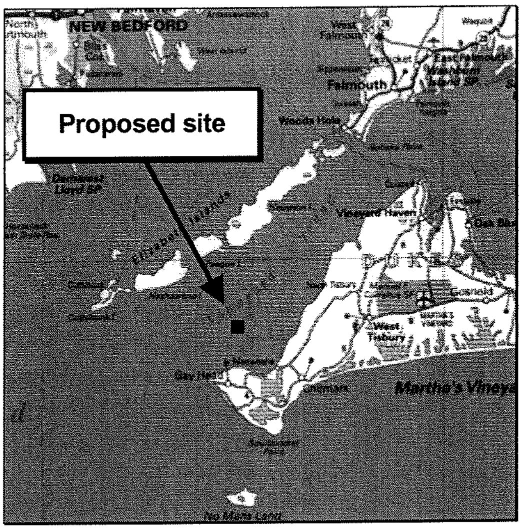


Figure 3-4 Vineyard Sound [Ref 42]

3.3.4 Block Island

Block Island is located some 60 km west from Martha's Vineyard. North of the island a more or less protected area might be an option for the location of the terminal, although there still might be some persistent wave action originating from the south-west. Again some 15 km of submarine pipeline is required to connect to the shore, where another 140 km is needed to deliver the gas in Boston. The island itself is a National Wildlife Refuge, and the pipeline track should be selected such that it will not cross the nearby National Parks that are located at the hinterland of the nearby coastline. Local tourism is mainly based on the beautiful scenery.

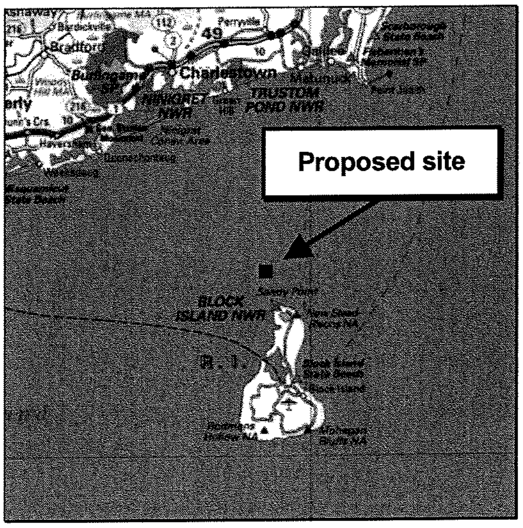


Figure 3-5 Block Island [Ref 42]

3.4 Site selection

Regarding the scope of this study and the amount of information available, the site selection is mainly based operability and financial feasibility. For each criterion a ranking from 1 (best) to 4 (worst) for the site options has been determined.

When looking at the amount of shelter available for each location, Block Island has the worst conditions, exposed to the daily waves coming from the south-west. Vineyard Sound is slightly better because the island provides more shelter. The two sites located north from Cape Cod have better conditions. Ipswich Bay is less favourable than Gurnet Point because the bay is relatively shallow, which means the terminal has to be located to a further seaward, more exposed site.

The pipeline costs for Ipswich Bay will be the lowest for it is located at close distance from Boston and requires very little submarine parts. This can also be said for Gurnet Point, but it is located slightly further away. Vineyard Sound and Block Island are both located at a greater distance from Boston, and they both require a significantly long submarine pipeline.

The environmental impact at each site of constructing an offshore terminal is very difficult to compare to one another. Ipswich Bay seems the worst location because of its vulnerable refuge nearby. Block Island itself is also called a refuge, making the location less favourable. Vineyard Sound has no protected nature reserves in the vicinity, but it is likely that the local community will protest against the construction of the terminal. Because Gurnet Point is located just outside the boundaries of the Cape Cod Bay reserve, this location seems to be slightly favourable to the other options.

The result of the evaluation has been summarised in Table 3-1 below.

Criteria	Ipswich Bay	Gurnet Point	Vineyard Sound	Block Island
Shelter for metocean conditions	2	1	3	4
Pipeline construction costs	1	2	3	4
Environmental impact	4	1	2	3
Total score (ranking)	**2**	**1**	**3**	**4**

Table 3-1 Evaluation of site locations

From the result of the evaluation table can be concluded that Gurnet Point has the best potential for location of the proposed terminal.

A more comprehensive assessment, including a thorough analysis of environmental consequences of the terminal, will be required to back up this conclusion. The proper way to do that is to start an ESHA (Environmental, Social and Health Impact Assessment) early in the project development, which optimises the project design and implementation to the local needs and sensitivities. It also provides input to the environmental permitting process. Part of the process is a structured engagement of stakeholders (those organisations and individuals that have an interest in the project and/or are affected by it).

An ESHA is beyond the scope of this study. Therefore the local conditions at Gurnet Point will be further analysed in the following chapters.

4 Environmental data collection

4.1 General

In this chapter all currently available environmental data for the site location has been collected. In paragraph 4.2 the bathymetry of the site is shown. Then the available metocean data has been put together in paragraph 4.3. Finally a description is given about the local seismicity (4.4) and the geotechnical conditions (4.5).

4.2 Bathymetry

The local bathymetry at the site location can be derived from the Admiralty Chart of the area. A close-up of the selected site is shown in Figure 4-1 below. Note that the hydrographic survey data of this chart dates from 1971, so current depth contours might differ from the present situation. A more detailed view of the local bathymetry has been enclosed in Appendix G.

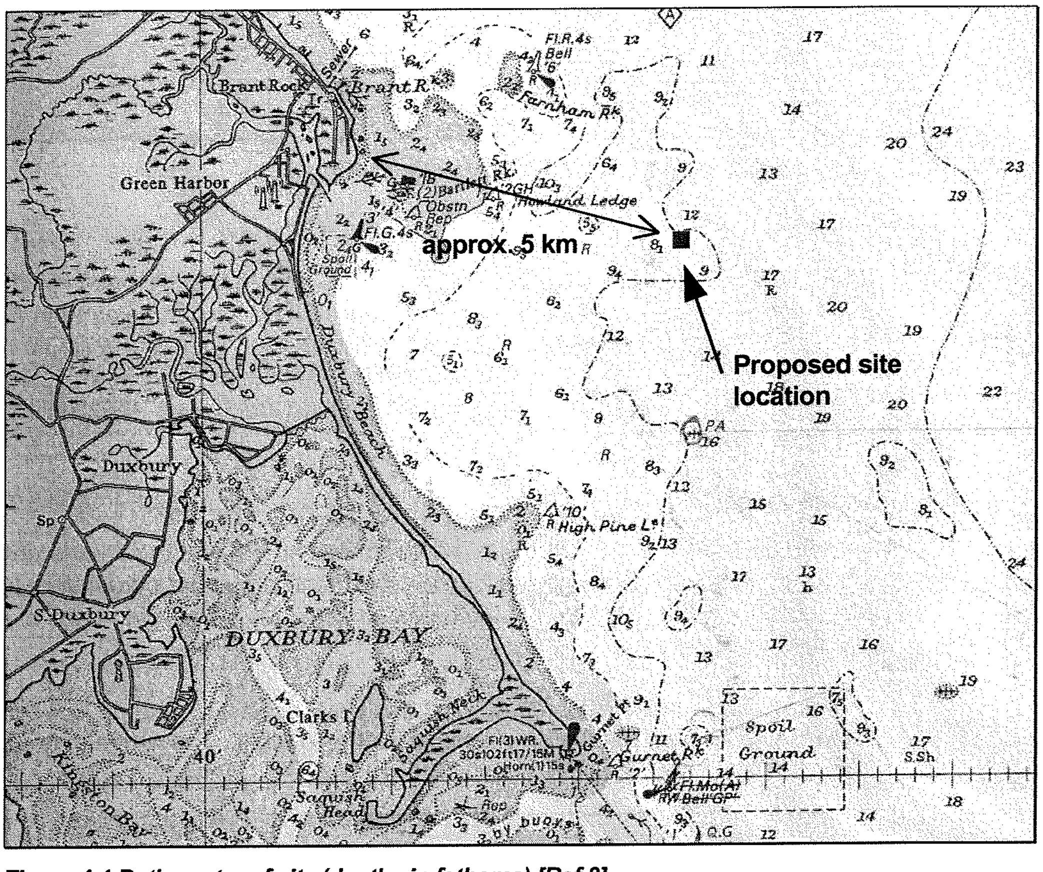


Figure 4-1 Bathymetry of site (depths in fathoms) [Ref 8]

4.3 Metocean conditions

4.3.1 Data sources

Two main sources of metocean data have been used, which are described below.

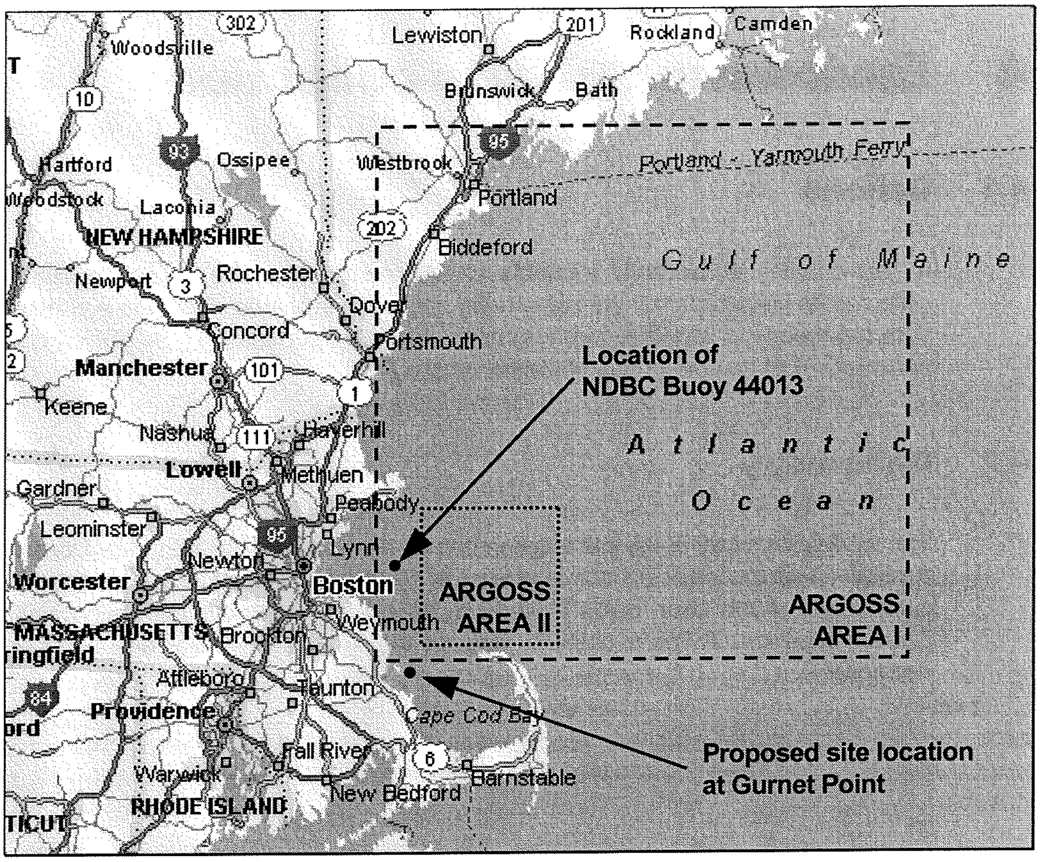


Figure 4-2 Location of metocean data sources [Ref 27]

- Buoy data

  The National Data Buoy Center [Ref 44] owns and maintains buoy 44013 located 42°21'14"N, 70°41'29"W, right in front of Boston Harbor, as can be seen in Figure 4-2. The local water depth is 55.0 m. The buoy measures wind, (non-directional) waves, air and water conditions at fixed intervals. To obtain an impression of the local metocean conditions, the statistics of historical data registered from June 1983 until December 2001 have been used.

- Satellite image data

  The ARGOSS Internet site [Ref 5] provides metocean data for most parts of the world. With the use of satellite images, offshore wave, wind and other parameters have been collected since 1993 until now. In the metocean analysis below, two areas of ARGOSS data have been used:

  - Area I: Some of the parameters (wind and wave direction, wave period) can only be produced for a selected area of at least 200 by 200 km^2. The centre of this area is located at 43°0 N, 69°30 W.
  - Area II: To take into account the nearshore reduction in waves, an area as small as possible (50 x 50 km^2) has been selected located as near as possible to the proposed site where still data were available. The centre of this area is 42°19 N, 70°10 W.

The results of the different data sources will be compared and evaluated in the metocean analysis below. The following remarks should be envisaged:

- Directional wave and wind data is only available for the ARGOSS Area I.

- The ARGOSS Internet site mentions that tropical storms are known to occur in the selected areas. These storms are not properly represented in the wind and wave climate data and should be analysed separately (see paragraph 4.3.12).
- The locations of both the buoy and the selected ARGOSS areas are more exposed than the proposed site location. Therefore the metocean conditions resulting from the analysis below should be considered as conservative.

4.3.2 Wind

According to the NDBC Buoy Data the average wind speed (average of eight-minute interval) is varying from 4.2 m/s in July to 7.5 m/s in December. Peak wind gust speeds (measured during five seconds within an interval of eight minutes) lie between 4.9 m/s in July to 9.4 m/s in December.

The ARGOSS data shows that the prevailing wind direction is south-west (25% of the time), but also winds originating from the north-west quadrant are quite common (see wind rose enclosed in Appendix G). Within the ARGOSS area II average wind speeds of 6.8 m/s are measured, varying between 4 m/s in July and 10 m/s in January. A histogram of the wind speed distribution is shown in Figure 4-3 below.

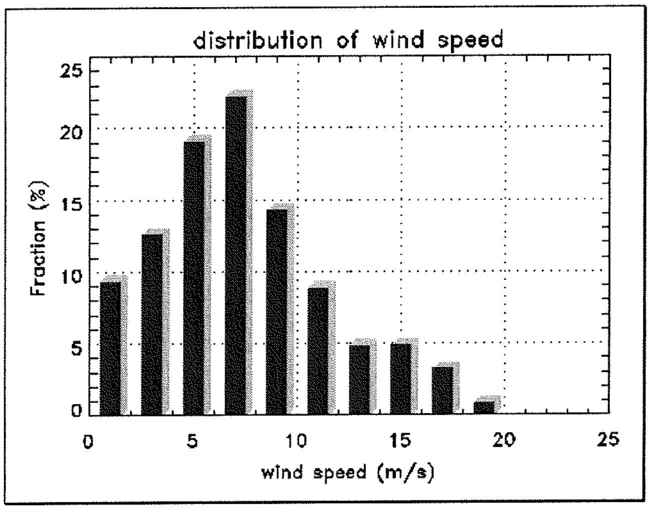


Figure 4-3 Wind speed fraction of time [Ref 5]

4.3.3 Waves

The NDBC Buoy has measured the significant wave height from 1986 to 2001, resulting in an average of 0.7 m. From May to September this average is reduced to 0.4 – 0.6 m, while from October until April it increases to 0.8 – 0.9 m. The dominant wave period amounts to approximately 8.0 s. The average wave period is measured at 5.7 seconds.

According to the data measured in the larger ARGOSS area I, the wave field originates from south to south-east directions (about 40% of the time, see wave rose in Appendix G), with an average significant wave height of 1.2 m and a mean wave period of 7 s. In the smaller area II, the significant wave height reduces to 1.1 m. A histogram of the significant wave height distribution is shown in Figure 4-4 below.

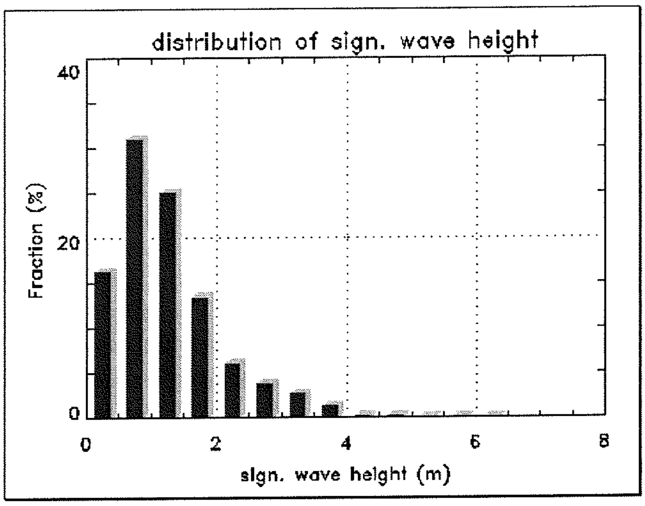


Figure 4-4 Significant wave height fraction of time [Ref 5]

The wave field consists of swell combined with wind-generated waves. When analysing these components separately, approximately 63% of the swell comes from south to south-east directions with an average height of 0.9 m and a mean period of 8.7 s. Most of the wind-generated waves originate from south to south-west directions. These waves have an average height of 1.0 m with a 4.1 s mean wave period.

4.3.4 Tide

The occurring tidal levels, measured exactly at Gurnet Point, have been summarised in the table below.

Tide	Water level above CD
MHWS	3.0 m
MHWN	2.6 m
MLWN	0.4 m
MLWS	0.0 m

Table 4-1 Tidal levels referring to Chart Datum [Ref 8]

The Admiralty Chart shows that at within the vicinity of the selected location the tidal streams occurring do not exceed 0.5 m/s.

4.3.5 Extreme water levels

The highest and lowest water levels observed in Boston from 1922 until 1970 according to the Shore Protection Manual [Ref 59], are given in Table 4-2 below.

Boston MA, observation period 1922-1970	
Mean range	2.90 m
Average yearly highest above MHWL	0.91 m
Extreme highest above MHWL	1.34 m
Average yearly lowest below MLWL	0.94 m
Extreme lowest below MLWL	1.16 m

Table 4-2 Highest and lowest water levels in Boston

4.3.6 Currents

According to USGS [Ref 70] the mean current typically flows southerly through Massachusetts Bay and turns offshore into the Gulf of Maine. During much of the year, this weak circulation persists in Massachusetts and Cape Cod Bays, typically at a strength of about 5 to 10 cm/s. This flow pattern may reverse in the fall. In Figure 4-5 the observed mean flow (small blue arrows) and the variability (shown as a green ellipse) for near-surface currents has been drawn. In general, the fluctuations are larger than the mean. The bold grey arrows indicate the overall direction of the residual drift.

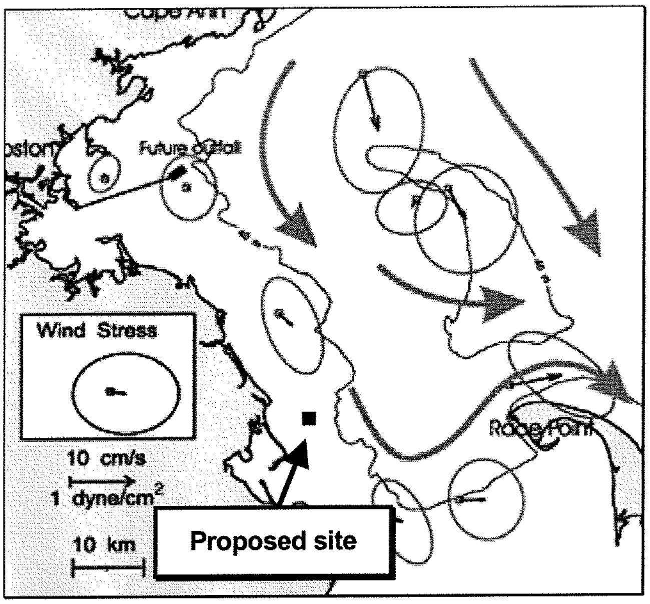


Figure 4-5 Mean current flow pattern [Ref 70]

4.3.7 Sediment transport

According to a study carried out by USGS it appears that winds from the northeast drive near-surface currents to the south toward Cape Cod Bay and near-bottom current to the southeast offshore toward Stellwagen Basin. Sediments that reach the sea floor in Cape Cod Bay are likely to remain there. In this coastal system, currents caused by surface waves are the principal cause of sediment re-suspension. Cape Cod Bay is sheltered from large waves by the arm of Cape Cod. Thus, once sediments reach Cape Cod Bay, carried either by the mean flow or transported by storms, it is unlikely that they will be re-suspended and transported again by waves.

Figure 4-6 shows wind-induced currents as well as near-bottom wave current velocity driven by a 14 m/s wind from a north-east direction. Near-bottom wave speeds in excess of about 10 cm/s are sufficient to re-suspend fine-grained sediments.

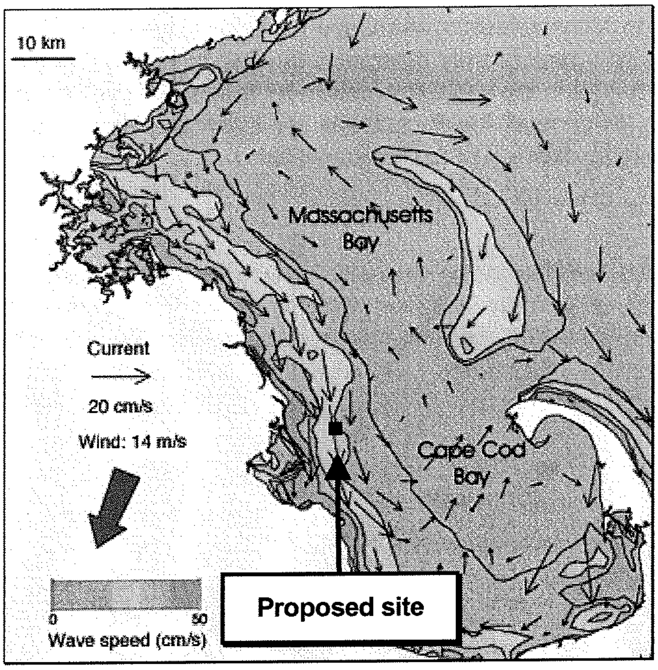


Figure 4-6 Near-bottom wave and wind-induced currents [Ref 70]

4.3.8 Air and sea temperature

Continuous measurements of the NDBC buoy show that the air temperature is varying from –0.6° C in February to +19.8° C in August. The average sea temperature is following with 2.8° C in February to 18.0° C in August (see Figure 4-7).

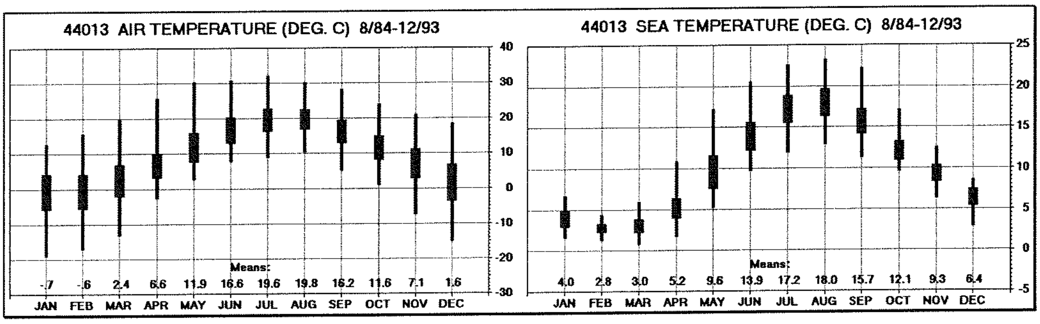


Figure 4-7 Seasonality of average air and sea temperatures [Ref 44]

4.3.9 Atmospheric pressure

The NDBC Buoy provides data on the atmospheric pressure measured at sea level. Its average fluctuates from 1013 to 1018 mbar, but extreme values show very low values around 965 mbar, as can be seen in Figure 4-8.

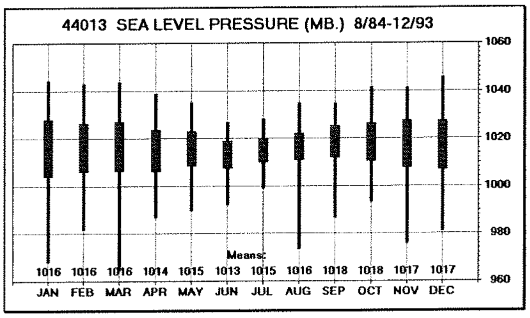


Figure 4-8 Atmospheric pressure at sea level [Ref 44]

4.3.10 Ice

In wintertime harbours may be closed because of ice, the average temperature in January and February being below zero. Shipping however normally keeps the main channels open [Ref 3].

4.3.11 Precipitation and fog

The average precipitation is 980 mm per year, rather constant throughout the year. In winter much of the precipitation falls as snow. An appreciable snow cover (over 25 mm) is occurring on 40 to 80 days per year. During storms blizzards may develop. The combined effect of high winds and low temperature often produces very poor visibility for shipping during the winter period [Ref 3].

Fog develops mainly when sub-tropical air is cooled by the Labrador Current and occurs on average on 20 days per year. In the period from May to September fog may persist for several days [Ref 3].

4.3.12 Tropical storms and hurricanes

About 40 tropical storms and hurricanes have affected the region of southern New England since 1900, 12 of which made landfall with significant impact (see Figure 4-9). August and September were the most active months for tropical cyclone activity with 10 occurrences. The remaining two occurred in July. Each of these systems, with varying degrees of impact, brought high winds, coastal flooding and heavy precipitation to the region.

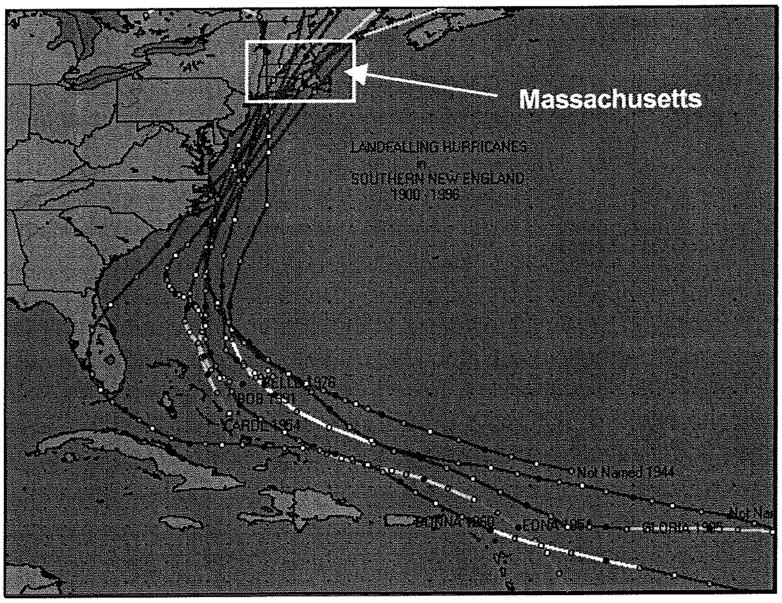


Figure 4-9 Land falling hurricanes in southern New England 1900-1996 [Ref 1]

The average duration of tropical storm force winds ranged from 12 to 15 hours, while hurricane force winds were generally produced for 3 to 6 hours. The angle at which the systems made landfall was south-west, close to perpendicular to the southern coastline of New England, resulting in storm surges of 0.5 to 1 meter for tropical storms to 4 meters for Category 2 and 3 storms.

4.4 Seismicity

Nineteen earthquakes, intensity 5 or greater (Richter scale), have centred in Massachusetts (see Figure 4-10). A number of other earthquakes were centred off the coast of Massachusetts and affected the eastern portion of the state. A shock in 1755, located at Cape Ann, reached intensity 8 at Boston. In addition, Massachusetts was affected by some of the more severe Canadian shocks plus the earthquake of 1929 that centred on Grand Banks of Newfoundland [Ref 45].

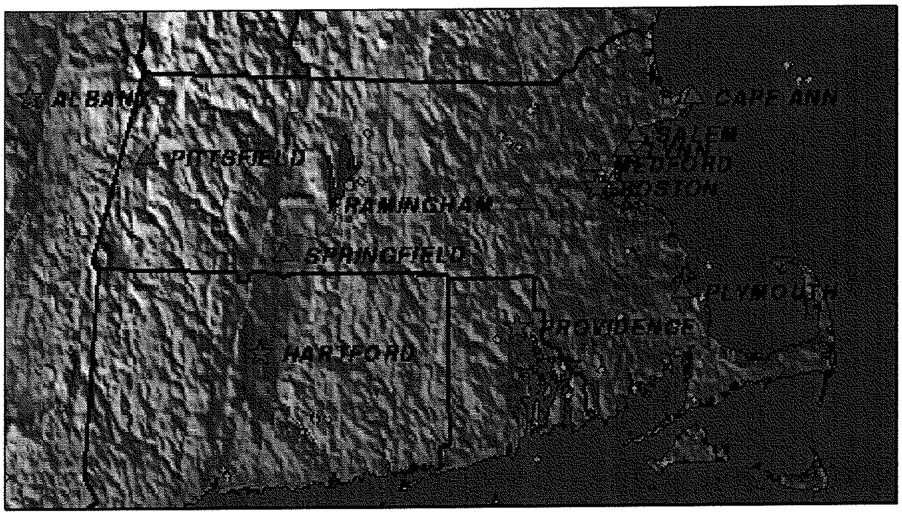


Figure 4-10 Seismicity of Massachusetts from 1977 until 1996 [Ref 45]

According to the 1997 Uniform Building Code Volume 2 [Ref 69] the proposed site is located within Seismic Zone 2A (moderate seismic activity), which means seismic zone factor Z = 0.15.

4.5 Geotechnical conditions

According to a presentation on outfall diffusers in the Boston Harbour by HBG (see Figure 4-11), the principal rock formation is Cambridge Argillite, a type of siltstone. The rock is known to have numerous discontinuities, folded bedding and faulting. The rock typically appears to be uniform and has high Rock Quality Designation (RQD) values. Several layers of sediments, clay and till varying in thickness between 6.0 and 21.0 meters are situated above the bedrock.

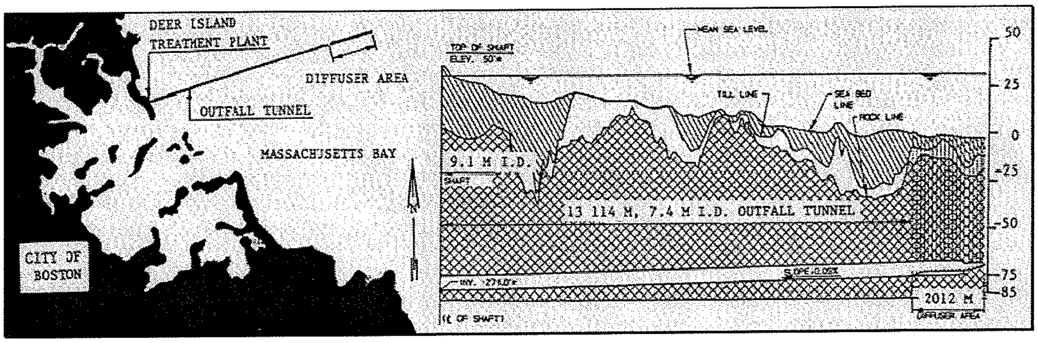


Figure 4-11 Boston outfall diffuser project with geological profile [Ref 15]

5 Environmental design criteria

5.1 General

The general environmental data that has been collected must be converted to environmental design criteria. First the design winds will be calculated in 5.2, after which the design water levels can be determined in 5.3. Then a two-dimensional wave model is applied to translate the offshore waves to design wave conditions at the site location in section 5.4. Furthermore the design criteria regarding the local soil conditions are given in 5.5. Finally a summary of the environmental design criteria is included in section 5.6.

5.2 Design winds

The wind measurement data provided by the NDBC Buoy 44013, located approximately 60 km to the north of the site, will be used as basis for the design wind calculations. The anemometer (instrument to measure wind speed and direction) registers wind velocities elevated 5 meters above the water surface. However in formulas for wind calculation the wind speed at 10 meters height is used. Therefore the following transitional formula is applied (Ref 50):

$$U_w(z)/U_{10} = (z/10)^{0.11} \qquad z = 5\text{ m},\ U_{10} = 1.08\ U_5$$

Occurring winds can be divided into short period winds (gusts) and longer period winds (sustained winds), which will be described below.

5.2.1 Sustained wind

The buoy has measured 105862 times the 8-minutes sustained wind speed during the period from 1987 until 2000. This data has been retrieved from the internet. For 26 wind speed intervals the number of occurrences has been counted. From the result a probability of exceedance for each interval has been calculated.

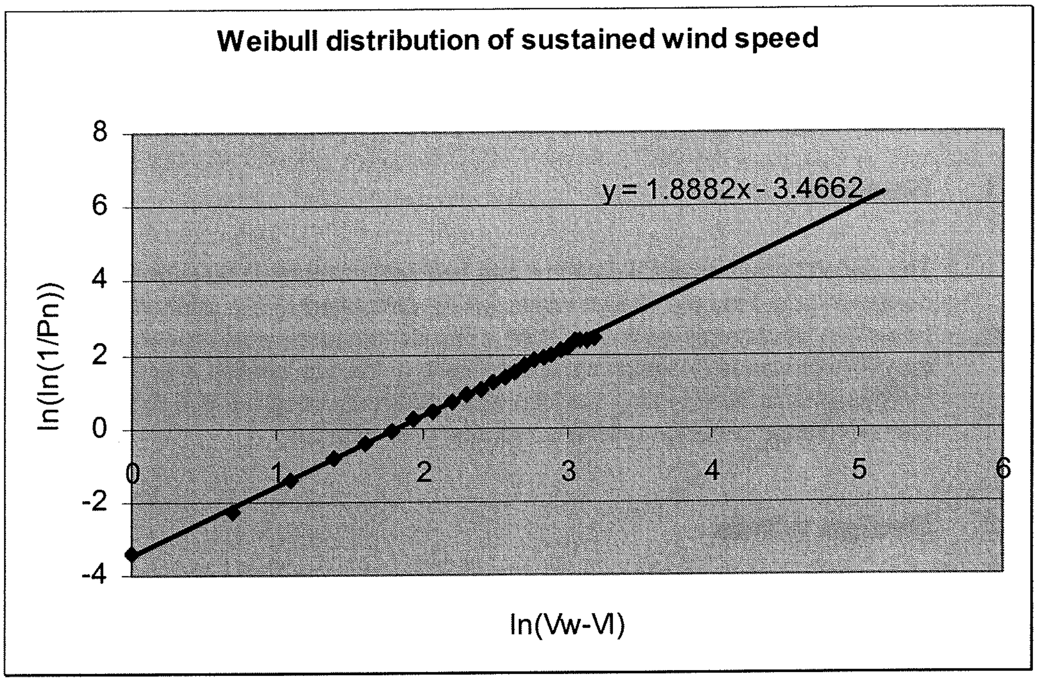


Figure 5-1 Weibull distribution of sustained wind speed

To extrapolate the measurements a curve should be fitted to the plotted probability points. In this calculation the Weibull distribution has been selected to fit the data (see Figure 5-1). When plotting ln (ln (1/probability)) against ln (Hs), a spreadsheet can be used to determine the linear function matching with the data set. With this function the return periods for the wind velocity can be calculated (Table 5-1).

Return period	Sustained wind velocity (m/s)
1 year	20.0
10 years	22.6
50 years	24.2
100 years	24.9

Table 5-1 Sustained wind velocity return periods

5.2.2 Wind gusts

For the determination of the 100-year wind gust exactly the same calculation as for the sustained wind has been made. As a result the wind gust that can be expected once in hundred years amounts to 31.8 m/s. The buoy data defines a wind gust as wind sustained during for 3 seconds. To determine the 30-second and 60-second gust, which are often used as design conditions, the following formula applies (Ref 19):

$$\frac{U_t}{U_{3600}} = 1.277 + 0.296 \tanh\left\{0.9 \log \frac{45}{t}\right\}$$

30-second gust: U_{30} = 27.9 m/s

60-second gust: U_{60} = 26.2 m/s

5.2.3 Wind direction

The wind directions measured by the NDBC Buoy have been analysed. The originating direction of the wind is very fluctuating: one direction where most of the winds come from cannot be given, as can be seen from the wind rose shown in Figure 5-2.

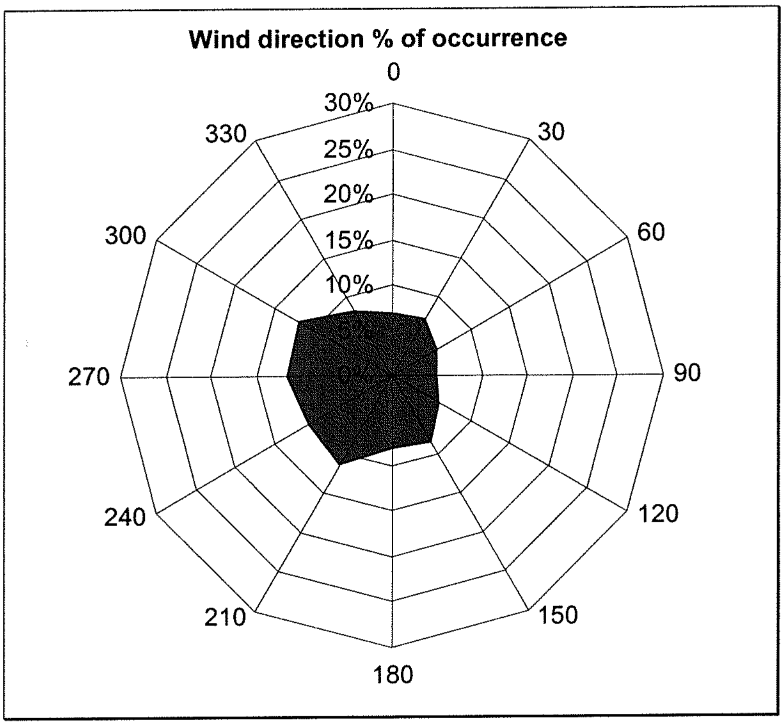


Figure 5-2 Wind direction % of occurrence

The occurrence of wind directions has been analysed for higher velocities to still get an impression. Using a threshold of either 5, 10 or 15 m/s as a minimum wind speed, the occurrence of wind directions has been assessed. As can be seen in Figure 5-3, the directional spreading decreases when measuring only higher wind speeds: West and North-Northeast become the prevailing directions.

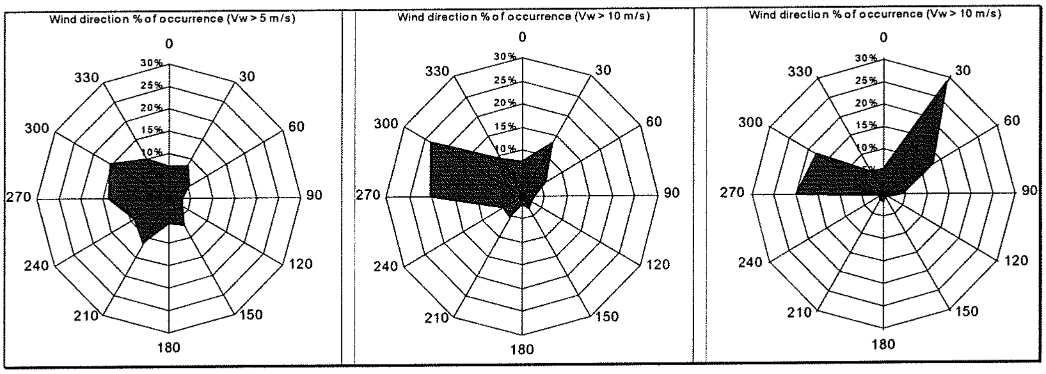


Figure 5-3 Wind speed greater than 5, 10 or 15 m/s direction % of occurrence

5.2.4 Seasonality of winds

Wind speeds and directions can vary in different seasons. When defining winter from October until March and summer from April until August, differences can be noted. The seasonal directional spreading is shown in Figure 5-4.

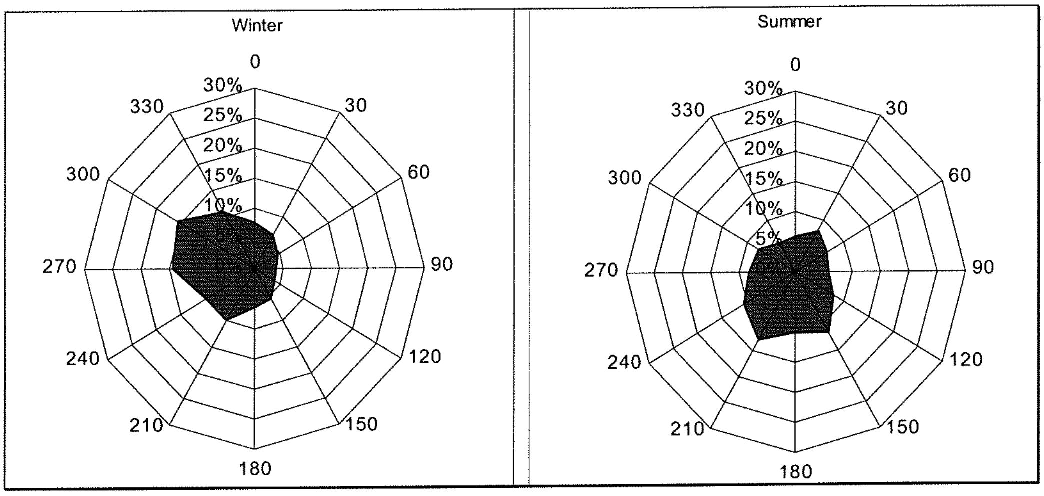


Figure 5-4 Directional spreading of wind during winter and summer

Seasonal variations in average wind speed have also been assessed, which are shown in Table 5-2 below.

	Summer	Winter
Average wind speed (m/s)	8.9	12.4
Average wind gust (m/s)	10.5	15.3

Table 5-2 Season variation in average wind speed

5.3 Design water level

Chart Datum has been determined at mean low water spring (MLWS). The extreme water level comprises of tide, storm surge, wind set-up, wave set-up and seiches.

5.3.1 Storm surge

Static rise of mean water level can be induced by local depressions during storms. A first estimate of the storm surge can be calculated with the following formula (Ref 19):

$$z_a = 0.01(1013 - p_a)$$

When applying the lowest atmospheric pressure level measured by the NDBC buoy (over 19 years), the corresponding static rise amounts to 0.5 m.

5.3.2 Wind set-up

Wind set-up is caused by shear stress exerted by wind on the water surface. It is most pronounced along relatively shallow waters. The formula for wind set-up (Ref 19):

$$\eta_w = \frac{c_w\left(\rho_{air} / \rho_{water}\right)U_w^2 F}{gh}$$

Using friction coefficient c_w = 0.003, densities ρ_{air} = 1.21 kg/m^3 and ρ_{water} = 1030 kg/m^3, maximum wind speed U_w = 24.9 m/s (100-year wind speed at deep water), effective fetch length F = 100 km and average depth h = 100 m, the resulting wind set-up amounts to 0.3 meters.

5.3.3 Wave set-up

Shoaling of the incoming waves after the breaker line causes a wave set-up at the shoreline. However the proposed terminal will most probably be located outside the breaker zone. The maximum water level will not be affected by wave set-up.

5.3.4 Seiches

Seiches are oscillations caused by external forces and trapped by local bathymetry of the coast. Although the proposed site is located in a bay, it is assumed that the entrance of the bay compared to the area/volume of the bay does not induce the occurrence of seiches. Therefore the local water level will not be affected by this mechanism.

5.3.5 Extreme water level

The extreme water level is the sum of mean high water level, a storm surge and the wind set-up, 3.0 + 0.5 + 0.3 = 3.8 m above CD.

An alternative estimate can be made by applying a Weibull distribution to the water level data. The resulting probability function is shown in Figure 5-5. According to this distribution, the high water level with a 100-year return period is determined at 4.6 m above CD. This level incorporates all water level increasing effects (tide, storm surge, wind and wave set-up and seiches). However the location where the data for this calculation has been collected might differ from the site location. If for instance the data is measured in Boston Harbour, seiches may be of influence.

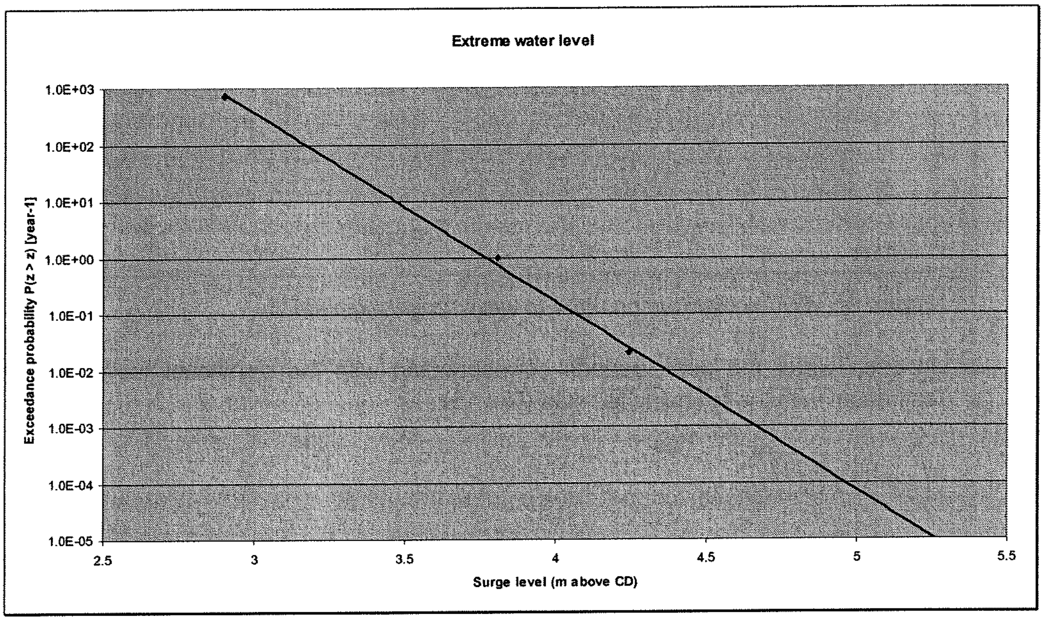


Figure 5-5 Probability of exceedance of extreme water levels

As an approximation, the average of these two calculations, CD + 4.2 m, will be chosen as the extreme water level value.

5.4 Design wave height

5.4.1 Offshore waves

The design wave for the Ultimate Limit State (ULS) should have a return period of 100 years. Such a high wave can only reach the selected site location originating from the north to east quadrant due to the form of the coastline and protection provided by Cape Cod.

The return period for offshore waves has been determined using 16 years of satellite observations gathered in the ARGOSS database, resulting in 36360 wave height measurements. From this data a table can be calculated that shows the cumulative probability of occurrence of a certain wave height. To extrapolate the measurements again a Weibull distribution has been fitted to the data (only the purple dots have been used for the curve fitting). Then the wave height return periods can be calculated.

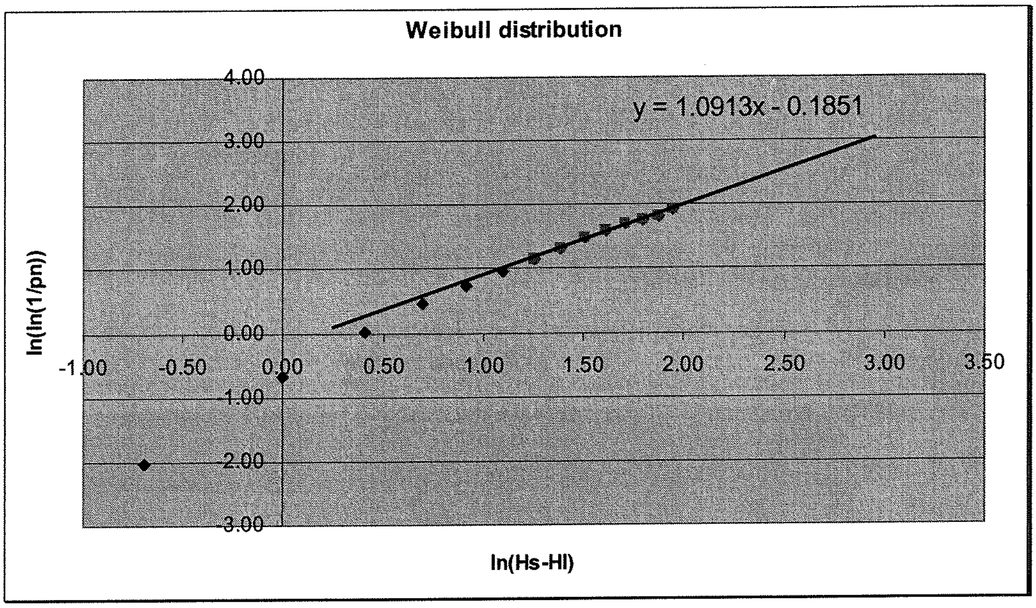


Figure 5-6 Weibull distribution fitted to wave height exceedance

In deep water the peak periods corresponding with the significant wave heights can be estimated with the following formula (Ref 19):

$$T_{p;0} = 4.4\sqrt{H_{s;0}}$$

The resulting significant waves with their return periods are shown in Table 5-3 below.

Return period	H_s (m)	T_p (s)
1 year	7.7	12.2
10 year	9.8	13.8
50 year	11.2	14.7
100 year	11.8	15.1

Table 5-3 Significant wave height return periods for ARGOSS data

5.4.2 SWAN Calculation

For an estimation of the nearshore wave climate the computer program SWAN (Simulating Waves Nearshore) has been used, developed by Delft University of Technology. The program is a numerical 2D wave model designed to obtain realistic estimates of wave parameters in coastal areas from given wind-, bottom- and current conditions. The model is based on the energy balance equation. The physical phenomena which are accounted for in the calculation are given below.

- Recti-linear propagation
- Refraction
- Shoaling
- Wind to wave energy transfer
- White capping
- Depth-induced wave breaking
- Bottom friction
- Wave – wave interaction

Diffraction is not modelled in SWAN. Because the proposed site is located approximately 40 km from Cape Cod and the prevailing wind/wave direction is NNE, it has been assumed that the conditions at the site will not be affected by any diffraction effects caused by Cape Cod.

The following parameters have been used for the SWAN model:

- Type of computation

  The model has been processed in a static (stationary conditions, no input variations over time) 2-dimensional (including refraction due to curved bottom contours and directional spreading) mode.

- Bottom grid

  The Unites States Geological Survey Internet site (Ref 70) provides a bathymetric profile of the Gulf of Maine. The depths are given with a resolution of 15" (15 seconds), which results in grid meshes of approximately 350 by 460 meters. A rectangular area of about 140 by 165 km has been cropped from the huge amount of data, with an origin (the south-west corner) located at 41.5° N, 71.2° W (see Figure 5-7).

- Computational grid

  The computational grid should be selected within the boundaries of the bottom grid, and determines for which part of the bottom grid the model should perform the calculation. As can be seen in Figure 5-7, the computational grid has been selected as a rectangular area of 80 km wide and 120 km long. This area has been rotated by 27.5 degrees to cover all important characteristics of the coastline.

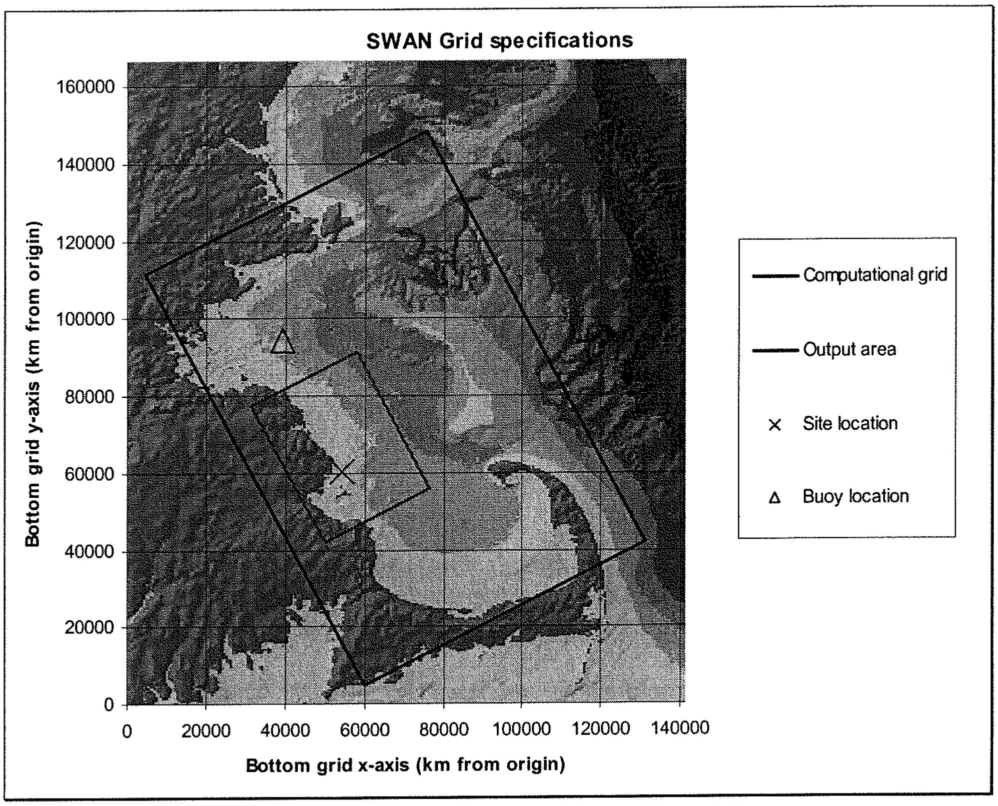


Figure 5-7 Definition of SWAN grids

- Boundary conditions

  To determine the significant wave height at the site with a return period of 100 years, the 100-year water level, offshore waves and wind speed (calculated in the previous paragraphs) have been used as boundary conditions for the model (see Table 5-4).

Significant wave height	11.8 m
Peak period	15.1 s
Angle of incidence (coming from)	NNE
Wind velocity	25 m/s
Wind direction (coming from)	NNE
Water level	CD + 4.2 m

Table 5-4 Boundary conditions as input for SWAN model

  The wind speed has been assumed constant over the entire area. The offshore waves have been simulated entering the computational grid at the upper x (east) and upper y (north) borders. Because the current velocities occurring in the area are relatively low, the model has been used without any current velocity input.

- Wave spectrum

  The spectrum of the waves at the boundaries is more specified by the characteristics given in Table 5-5 below.

Type	JONSWAP
Peak enhancement parameter α	3.3
Characteristic peak period	15.1 s
Peak wave direction	NNE
One-sided directional spread of waves	10° (wind-sea, little swell)
Significant wave height	11.8 m

Table 5-5 Wave spectrum characteristics

- Resolution and accuracy

  The amount of computation time is highly dependent on the resolution and ranges of parameters and the required accuracy of the output, which are given in Table 5-6 below.

Spectral directional resolution	10°
Wave period range	3 – 20 s
Frequency grid resolution	0.008
Wave height iteration limit accuracy	2%
Wave period iteration limit accuracy	2%

Table 5-6 SWAN resolution and accuracy

The input parameters mentioned above have been used to generate the input file for the SWAN model. A listing of this input file has been enclosed in Appendix I.

5.4.3 Model output

An output area has been selected to cover the site location and its surroundings (see Figure 5-7). The area has a width of 30 km and a length of 40 km, with a grid resolution of 500 x 500 meters. For each grid point the model has calculated wave height, period and direction. A map of significant wave height isolines is shown in Figure 5-8.

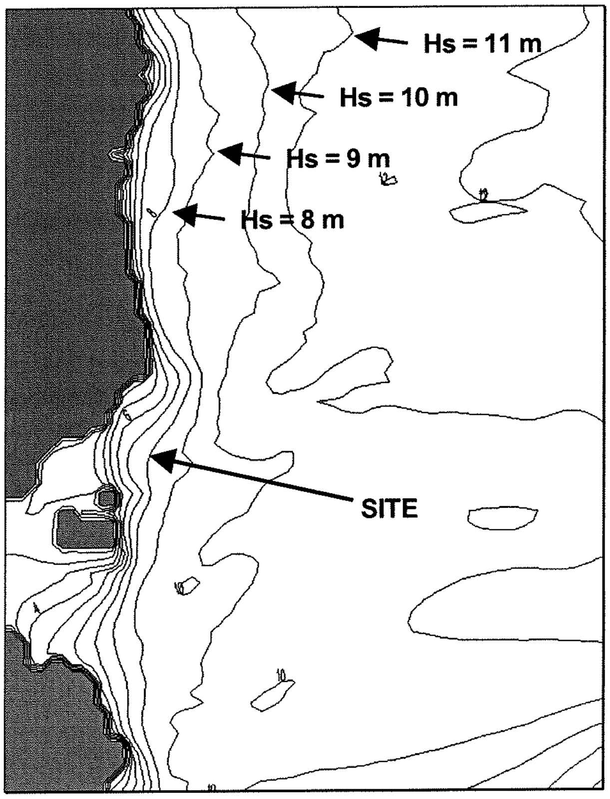


Figure 5-8 SWAN Output wave height isolines

Also output is generated for the exact location of the site, summarised in Table 5-7.

Water level	CD+4.2 m	CD+0.0 m
Local water depth	19.47 m	15.27 m
Significant wave height	7.94 m	6.63 m
Mean wave period	11.4 s	11.1 s
Peak wave period	16.2 s	16.2 s
Peak wave direction	225°	225°

Table 5-7 SWAN output results for site location

5.4.4 Validation of results

- Sensitivity

To check the sensitivity of the used model, several input parameters have been varied and their influence on the model output has been analysed.

Changing the input water level from 4.2 m to 3.5 m has little effect on wave heights (-2%) and almost no effect on wave periods. Altering the frequency resolution, range or accuracy within reasonable limits does not have any significant impact on the output of the model.

When the direction of the wind and waves is changed to North, wave height at site increases only slightly (+2%) while wave period and angle of incidence remain the same. However due to the limited fetch length, the 100-year conditions are not likely to originate from this direction. Changing the direction –30 degrees (ENE) results in significantly reduced wave action (-10%) at the site, proving that the original NNE direction should be used to determine the Ultimate Limit State conditions.

The amount of directional spread in the waves determines whether the incoming wave field consists mainly of swell (2 – 5°) or wind-generated waves (10 – 15°). In SWAN increased directional spread also results in some diffraction effects. Variation of this input parameter shows little impact on the output of the model, as can be concluded from Table 5-8.

	Hs	Tp
Less wind-gen, more swell (3°)	7.78 m	16.23 s
Used in model (10°)	7.94 m	16.23 s
More wind-gen, less swell (23°)	7.92 m	16.23 s

Table 5-8 Model sensitivity due to directional spread

- Comparison with other sources

The return period for offshore waves has also been determined using the continuous observations of NDBC Buoy 44013 from 1987 until 2000, resulting in 114682 wave height measurements. Applying a fitted Weibull distribution the return periods for the wave height have been calculated. Because the buoy is located within the area processed by the SWAN model, its 100-year wave height can be compared with the wave height produced by the model at the buoy's location.

	SWAN	NDBC Buoy
Northing	Y = 93352 m	42°21'14" N
Easting	X = 42350 m	70°41'29" W
Water depth	58.9 m	59.2 m
100 year wave height	10.55 m	10.6 m

Table 5-9 Comparison SWAN results versus buoy data

As can be seen from Table 5-9 the wave heights match perfectly. Such a small difference should be considered as a coincidence, but at least it shows that the output of the model has sufficient resemblance with other independent sources of wave data.

5.4.5 Daily wave conditions

Due to continuously varying winds, wave directions and water levels, the daily wave conditions cannot easily be determined using a model. As a first approach, the continuous buoy measurements will be used as if the buoy was located at the site. Using this simplification, the following remarks should be made:

- The buoy is located approximately 60 km from the site, surrounded by completely different bathymetry and nearby coastlines. Due to its location the buoy will measure more wave action coming from east and south-east, but less coming from north and north-east.
- The local water depth at the buoy location is much higher (55 m) than at the site (15 m). Large waves measured by the buoy will therefore not be the same at the site because they (partially) break. However when looking at daily wave action this will be less an issue.

To get an impression of the daily wave heights that can be expected, the 114682 waves measured by the buoy have been analysed to produce a curve for probability of wave height exceedance. The result is shown in Figure 5-9.

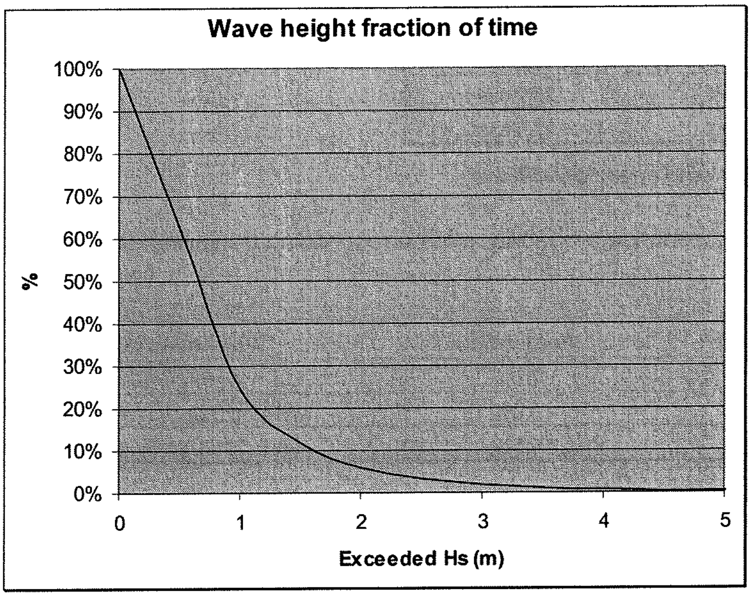


Figure 5-9 Probability of wave height exceedance

A trend line has been fitted to the exceedance data to calculate the chances of occurrence for different wave heights (see Table 5-10).

Chance of occurrence	Significant wave height (m)
5%	2.4
10%	1.8
20%	1.3
30%	1.0
40%	0.7
50%	0.6

Table 5-10 Chance of significant wave height occurrence

5.5 Soil conditions

The soil data that has been collected (see paragraph 4.5) holds for a site located approximately 60 km north to the selected site location. The validity of this data is therefore questionable. A sound soil investigation must be carried out to determine the actual soil parameters. If the measurements do not suffice the requirements, soil improvement may be conducted. At this stage, the soil parameters are assumed to have the values that are given in Table 5-11.

Type	Loose to medium dense sand
Angle of internal friction	30°
Dry density	18 kN/m^3
Wet density	20 kN/m^3

Table 5-11 Assumed soil parameters

5.6 Summary

The wind conditions with a return period of 100 years are used as design criteria and are summarised in Table 5-12.

100 Year condition	Wind speed
Sustained wind	24.9 m/s
60-seconds gust	26.2 m/s
30-seconds gust	27.9 m/s

Table 5-12 100 Year wind speed conditions

The wind speed exceedance probability with corresponding prevailing directions is shown in Table 5-13 below.

Wind speed	Probability	Prevailing directions
> 5 m/s	51%	WNW (16%), W (14%)
> 10 m/s	9.0%	WNW (23%), W (20%), NNE (13%)
> 15 m/s	0.5%	NNE (29%), WNW (20%), W (18%)

Table 5-13 Wind speed exceedance

The seasonal variations in wind speed and directional spreading are given in Table 5-14.

Seasonal average	Summer (April – September)	Winter (October – March)
Wind speed	8.9 m/s	12.4 m/s
Wind gust	10.5 m/s	15.3 m/s
Prevailing directions	WSW – SSE (45%)	WNW – W (30%)

Table 5-14 Seasonal wind speed variations

Chart Datum has been established at Mean Low Water Spring level (MLWS), which in this case equals Lowest Astronomical Tide (LAT). The astronomical tide at the site location is shown in Table 5-15.

Tide	Water level above CD
MHWS	3.0 m
MHWN	2.6 m
MLWN	0.4 m
MLWS	0.0 m

Table 5-15 Tidal levels

The water level with a return period of 100 years is determined at 4.2 m above CD.

The significant design wave characteristics with a return period of 100 years are given in Table 5-16.

Significant wave height	7.9 m
Mean wave period	11 s
Peak wave period	16 s
Peak wave direction	225°

Table 5-16 100 Year design wave conditions

The daily chances of occurrence of wave heights are shown in Table 5-17.

Chance of occurrence	Significant wave height (m)
5%	2.4
10%	1.8
20%	1.3
30%	1.0
40%	0.7
50%	0.6

Table 5-17 Chance of significant wave height occurrence

Tidal currents as well as wind or wave-induced currents never exceed 0.5 m/s.

Soil parameters are described in Table 5-11.

Type	Loose to medium dense sand
Angle of internal friction	30°
Dry density	18 kN/m^3
Wet density	20 kN/m^3

Table 5-18 Soil parameters

Miscellaneous metocean parameters are given in Table 5-19 below.

Parameter	Yearly minimum	Yearly maximum
Air temperature	-0.7° C.	19.8° C.
Sea water temperature	2.8° C.	18.0° C.
Atmospheric pressure	1013 mbar	1018 mbar

Table 5-19 Yearly maxima and minima

6 Functional requirements

6.1 Terminal components

The following functional components should be incorporated in the offshore terminal. A description of the most important components and their layout on a conventional onshore terminal has been enclosed in Appendix C.

6.1.1 Berth

- Approach channel
- Marine support craft (tugs, pilots)
- Navigational aids (buoys, marker lights)
- Breasting and mooring dolphins or similar berthing/mooring facilities
- Fendering system
- Speed of Approach measurement system

6.1.2 Unloading equipment

- Marine loading arms
- Unloading lines and manifolding
- Vapour return lines
- Fire monitors
- Emergency Shut Down (ESD) system

6.1.3 Storage facility

- LNG storage tank with cryogenic containment system
- In-tank low-pressure LNG pumps
- Connecting pipelines

6.1.4 Process equipment

- Vaporisers
- Boil-Off Gas (BOG) compressor and recondenser
- LNG High pressure pumps
- Knock-Out (KO) Drums
- Metering and odorising equipment
- Vent or flare

6.1.5 Utilities and general facilities

- Electric power generator
- Fresh water system
- Fire fighting facilities
- Nitrogen supply

- Control room / office buildings
- Workshop / warehouse
- Accommodation

6.2 LNG Carriers

The terminal's berth should be able to accommodate LNG carriers with capacities ranging from 125,000 m^3 to 145,000 m^3. The average cargo load amounts to 135,000 m^3 of LNG.

6.3 Capacity

Calculations to determine the capacity requirements of the terminal are enclosed in Appendix H. The most important figures are given in Table 6-1.

Annual throughput	5.0 mtpa
LNG Storage capacity	200,000 m^3
Average ship's cargo volume	135,000 m^3
Average send-out rate (LNG)	1268 m^3/hr
Swing factor	1.3
Peak send-out (LNG)	1650 m^3/hr
Turn around time	24 hours
Acceptable ship downtime	1.6 days
Ship inter-arrival time	4.4 days
Berth occupancy	23%

Table 6-1 Terminal capacity figures

6.4 Construction

Preferably the construction time schedule should have a maximum duration of 3 years. The terminal will have to consist of several modules with individual dimensions that do not exceed the limitations set by the construction dock and sailing route to the site.

6.5 Design life

In line with industry practice, the design life of the import terminal will be 30 years, although it is common that this type of facilities operate safely well beyond their design lifetime.

6.6 Future expansion

The design of the terminal should be based on minimum life time cost (CAPEX + OPEX). Provisions for future expansion should be kept to a minimum, although the initial terminal should be capable to accommodate expansions of the send-out capacity without major interruption of the gas supply, and where feasible additional storage.

7 Basic design data

7.1 LNG characteristics

The basic properties of the transported LNG are described in Table 7-1.

Parameter	Minimum	Maximum
Temperature	-157° C.	–166° C.
Density	430 kg/m^3	480 kg/m^3

Table 7-1 LNG characteristics [Ref 21]

7.2 LNG Carrier characteristics

The main design parameters of the minimum and maximum sized ships that should be able to access the terminal are given in Table 7-2 below.

Parameter	Minimum	Maximum
Gross cargo capacity	125,000 m^3	145,000 m^3
Net cargo capacity	120,000 m^3	140,000 m^3
Length (LOA)	272 m	295 m
Beam	47.2 m	48 m
Height (distance keel – main deck)	26.5 m	27 m
Draft (ballasted)	9 m	10 – 10.5 m
Draft (laden)	11.0 m	11.5 m
Water displacement	93,240 m^3	110,000 m^3

Table 7-2 LNG Carrier characteristics [Ref 46]

7.3 Design water depth

According to the British Standard the design water depth has been calculated in Table 7-3.

Maximum draft of laden design LNGC	11.5 m
Minimum keel clearance	1.0 m
Allowance for siltation	1.0 m
Allowance for ship movements	1.5 m
Minimum water depth required	15 m

Table 7-3 Determination of minimum required water depth [Ref 9]

7.4 Limiting environmental conditions

The following design limitations have been assumed for defining the operability of the LNG terminal.

7.4.1 Ship approaching terminal

The limiting environmental conditions for a ship approaching the terminal are given in Table 7-4 below.

Normal tug boat assistance (hook up)	Hs < 1.8 – 2.0 m, depending on heading and sheltering by LNGC
Tug boats at lee-side of vessel (tug operations)	Hs < 2.5 m (when sailing)
Cross-currents	< 0.5 m/s

Table 7-4 Limiting conditions for ship approaching terminal [Ref 33]

7.4.2 Ship berthing / at berth

The limiting conditions for a ship at a conventional berth have been determined by a study done by Marin (see Table 7-5).

Significant wave height	For moored LNGC	Max. 30° offset Hs = 2.4 m at Tp < 9 s Hs = 1.8 m at Tp = 10 s Hs = 1.5 m at Tp = 11 s Hs < 1 m at Tp > 12 s
	For mooring launches	Hs < 1.5 m
Wind	To remain moored	20 m/s (30 seconds gust)
	To continue cargo operations	16 m/s (30 seconds gust)
	Berthing/manoeuvring	12.5 m/s (60 seconds gust)
Currents	During berthing	Cross current 0.2 – 0.3 m/s
	For moored LNGC	Cross current 0.4 m/s In direction of berthing line 2.5 m/s

Table 7-5 Limiting environmental conditions according to Marin [Ref 33]

7.5 Ship movements

PIANC has set motion requirements for safe working conditions for a conventionally moored gas tanker, which are given in Table 7-6.

Surge	2.0 m
Sway	2.0 m
Yaw	2°
Pitch	2°
Roll	2°

Table 7-6 Recommended motion criteria according to PIANC [Ref 18]

8 Design loads

8.1 General

In this chapter all loads that may influence the dimensioning of the terminal or parts of the structure will be assessed qualitatively. According to Det Norske Veritas [Ref 55] the loads will be categorised in permanent, live, deformation, environmental and accidental loads.

8.2 Permanent

- Dead weight

  The dead weight consists of the weight of the base structure (concrete, steel and ballast material).

- Superimposed loads

  Superimposed loads comprise all fixed equipment and buildings situated on top of the terminal.

- Hydrostatic loads

  The hydrostatic loads can be considered semi-permanent because they depend on the actual water level. The hydrostatic water pressure will induce horizontal loads on the submarine parts of the structure as well as vertical upward loads as result of buoyancy.

8.3 Live

- Equipment

  Movable operational equipment such as cranes, helicopters and people induce live loads on the structure.

- Cargo

  The fluctuating level in the LNG storage tank causes varying hydrostatic loads.

- Berthing

  Berthing loads are the result of the forces by a berthing ship on the fenders depending on the approach velocity of the ship and the fender characteristics.

- Mooring

  Mooring loads are caused by the forces in the mooring lines which are hooked-up on the structure induced by the movements of the moored ship.

8.4 Deformation

Loads that are associated with imposed deformation:

- Temperature gradients
- Creep and shrinkage of structural material

- Settlement of the structure

8.5 Environmental

Environmental loads are loads due to wind, waves, and currents and should be calculated for a 100 year return period.

- Wind

  Wind causes pressure on the exposed surfaces of ship and terminal. Depending on the most unfavourable situation either the maximum gust or the maximum sustained wind should be considered.

- Waves

  Wave loads cause hydrodynamic forces on the structure. Maximum waves as well as operational waves should be considered.

- Currents

  Currents induce drag loads on the immersed parts of the structure.

- Earthquakes

  The horizontal forces on the structure caused by an earthquake should be taken into account.

8.6 Accidental

- Collision

  The structure should provide sufficient strength to resist the impact of a colliding supply or service boat.

- LNG spilling

  A leakage in the LNG containment system could cause a temperature gradient from cryogenic to normal outside temperature.

- Explosions and fire

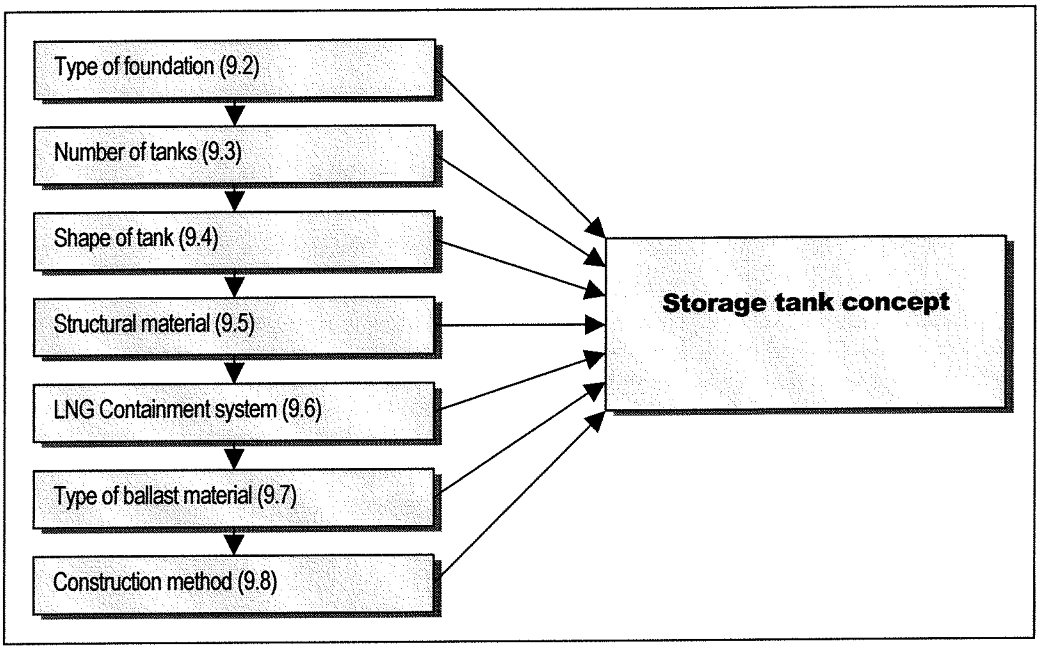
  Construction and material should be able to resist explosions and fire to a certain level.

9 Storage tank concept screening

9.1 General

No offshore LNG storage facilities have ever been built. The cryogenic characteristics of LNG, the expensive containment system and the large required volume are some of the issues that request for an inventive concept. In this chapter the main conceptual design choices will be made based on a merely qualitative selection.

An overview of the conceptual design choices discussed in this chapter is given below.



9.2 Foundation

One of the most important issues of the storage facility is the foundation of the tank(s). The enormous volume of the tank requires an innovative solution for the foundation to overcome the buoyancy.

9.2.1 Land reclamation

An artificial island reclaimed by dredging will be used to situate the terminal. Process equipment will be located next to the tank, and a jetty is required to be able to unload the LNG.

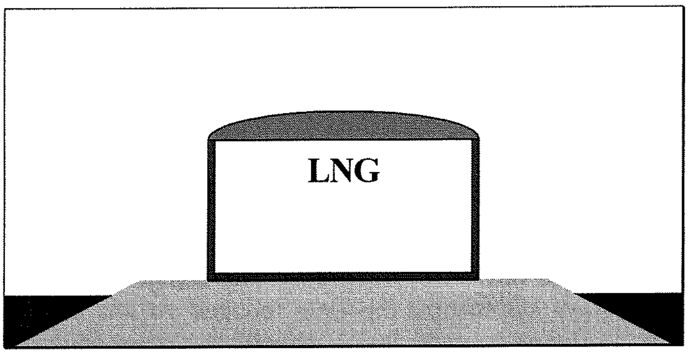


Figure 9-1 Tank founded on sand reclamation

Actually this concept is just an offshore version of a land-based terminal facility. For storage an ordinary cylindrical Full Containment tank can be used. Except for its remote location it has no advantages to the conventional onshore solution.

- For the terminal 15 to 20 ha is required. With a local water depth of 15 meters this will result in huge quantities of sand required.
- The equipment located next to the tank should be sufficiently protected from the ocean, resulting in expensive dikes and/or revetments.
- The jetty should be constructed to reach sufficient draft for the LNGC, thus only allowing a side-by-side unloading facility.
- The entire terminal has to be constructed in situ, on the land reclamation. The slopes of the reclamation will prevent easy access for ships with construction materials. Some sort of work harbour should be built.

9.2.2 Quay wall with land-fill

Prefab concrete combi-wall caissons will be ballasted onto the seabed. The enclosed area will be filled with dredged material. The LNG tank and the process equipment will be located on the land-fill, similar to the previous concept, but a jetty might not be necessary, because the LNGC can be moored alongside the quay wall.

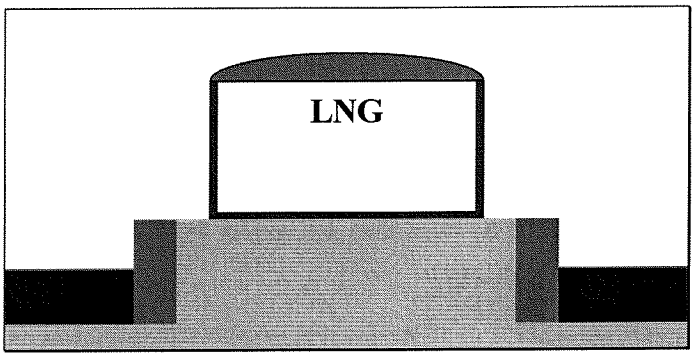


Figure 9-2 Tank founded on land-fill surrounded by quay walls

The advantage of this concept against the previous one is that a jetty is not required for the LNG offloading, and during construction ships can easily access to provide materials for the offshore construction.

- To protect the land-fill against waves the quay wall elements should have large dimensions to prevent instability, with high construction costs.
- Although construction of slopes has been avoided, still a large amount of dredged volume is required.

9.2.3 Gravity Based Structure (GBS)

A double-walled storage tank with ballast compartments in the walls and bottom will be sunk onto the seabed. Ballast has to provide sufficient stability. Process equipment can be located on top of the tank. The LNG carrier can either be offloaded in tandem or side-by-side.

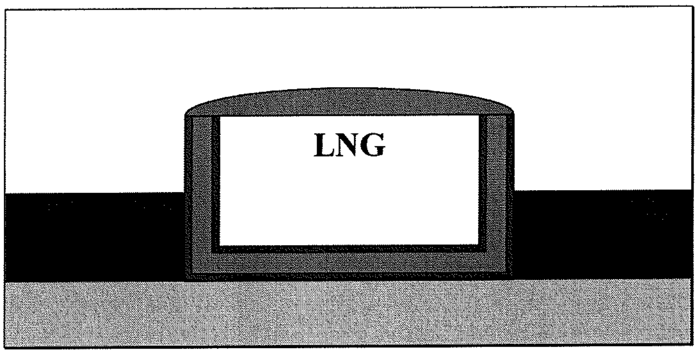


Figure 9-3 Tank founded as Gravity Based Structure

The ballast compartments can also act as a safety buffer zone in case of ship collision.

- When the entire structure will be built in a dock, the height of the caisson is limited due to available draft at the site (the draft of a floating caisson is approximately 50% of its height). Hence to reach the required storage volume the length and width of the structure will increase, resulting in more buoyancy. To resolve the buoyancy, larger ballast compartments are required, which again causes more buoyancy.
- If the structure will be partially built in a dock and then finished offshore, the height of the caisson can be increased. Because of a larger "dry" part, the ballast requirements will decrease. Then however, it is required to partially construct the tank at a deep water location which involves complex construction and installation procedures.

9.2.4 Anchored structure

Instead of adding ballast to the tank structure, tension piles can also be used to prevent the tank from floating. A reduction of construction material should then be possible. During installation the tank must be flooded in order to sink it down, where it can be connected to the tension piles.

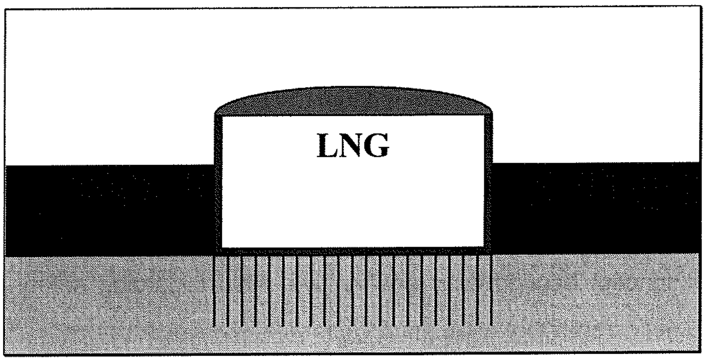


Figure 9-4 Tank founded with tension piles

- The subsea connection of the tank with the anchors will be a delicate and complicated offshore operation.
- The single walled tank is vulnerable to ship collisions.
- The submerged volume of the tank causes at least 85,000 tonnes of upward pressure due to buoyancy. A structure that can deal with these high-tension forces probably will be very uneconomical.

9.2.5 Semi-floating

Another option is allowing part of the structure to float. A floating, single walled tank will be surrounded by a protecting ballasted caisson wall (similar to the Ekofisk Tank Protective Barrier). The tank, with equipment on top, is allowed to follow the water level fluctuations, while heavy wave attacks are kept outside by the caisson barrier.

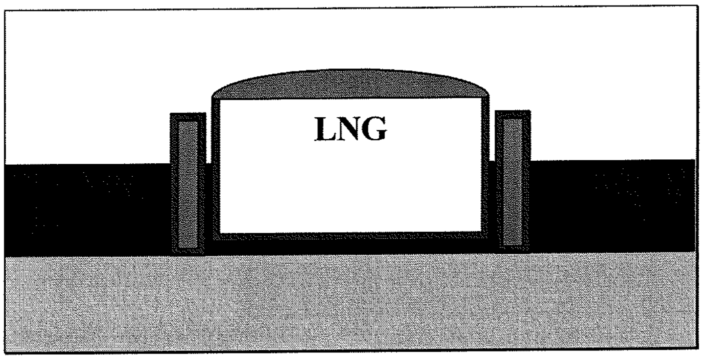


Figure 9-5 Semi-floating tank concept

- Although the tank can be constructed without any additional ballast compartments because buoyancy is allowed, the barrier caissons still have to be sunk to the bottom. It should be checked whether the dimensions required for the barrier might nullify the material savings on construction of the tank.
- Problems can be expected at the interface between the moving floating tank and the fixed outer walls. Because the ship is unloaded outside the barrier, there will be piping required to connect the loading arms on the fixed structure to the process equipment located on top of the floating tank. This piping should be able to withstand the movements due to water level fluctuations.

9.2.6 Conclusion

The first two concepts can be defined as land-based. Because equipment will be located next to, instead of on top of the tank, there is no synergy between components. Furthermore the concepts feature a conventional jetty or quay, which limits the mooring configuration options to side-by-side offloading. Use of breakwaters might be required. These concepts will be considered as uneconomical, unattractive alternatives for an onshore terminal.

The Gravity Based Structure concept seems to be a logical solution for the foundation of the tank. Process equipment and unloading installation can be located on top of the tank, resulting in a compact terminal. The compartments in the walls have a double function, providing space for ballast material and acting as a safety buffer for ship collision.

The major disadvantage of the GBS is the large amount of ballast required to provide stability, resulting in increased dimensions. The “anchored structure” concept tries to solve this problem, but the resulting extremely high tension forces and its complex offshore installation make this concept economically unattractive. Also regarding safety it is less favourable.

The “semi-floating” concept is another way to deal with the buoyancy of the tank. However the protective outer walls will again cause buoyancy versus stability problems. Furthermore the fixed-floating interface is likely to pose technical difficulties regarding the installation of the process equipment and (cryogenic) piping.

Of course fully floating storage facilities can also be an interesting option. Either a modified LNG carrier or a dedicated barge constructed in steel or concrete can be used to store the LNG. The storage tank can be single point moored so it is able to weathervane. Potential problems are the possibility of sloshing of the LNG within the floating storage facility as well as the LNG transfer between two floating (moving) bodies. However the floating option will not be worked out further here because floating concepts are beyond the scope of this study (as defined in chapter 2).

Taking the above into account, it can be concluded that a Gravity Based Structure is the only viable, fixed solution for the foundation of the LNG storage tank. Its dimensions will have to be optimised to minimise the ballast requirements.

9.3 Number of tanks

The total net storage volume that is required for the terminal amounts to 200,000 m3. This volume can be provided by one large tank or several smaller ones.

9.3.1 Single tank

The main advantage of building a single tank is the effect of economies of scale: for the same amount of storage less structural and LNG containment material is required than for multiple smaller tanks.

However, while increasing the dimensions of the tank, problems can arise concerning the span width of the bottom and roof slabs. Large differences in settlement of the subsoil have to be compensated. Also the handling of such a large structure during construction and installation (towage to the site) can become uneconomical.

9.3.2 Multiple tanks

An advantage of constructing multiple tanks is redundancy: if during operation one tank (or its in-tank pumping system) fails, another tank can still fulfil its function and thus it is not required to shut down the entire terminal. Furthermore the evidently smaller dimensions of the tanks will result in less excessive roof and bottom span lengths.

Although the smaller caissons will be easier to handle during installation, the costs for towage operations will increase with the number of caissons. After the caissons have been installed, an increased length of cryogenic piping is required to interconnect the tanks. Moreover the required amount of cryogenic insulation material (related to the total area of inner surface) is much higher than for one single tank, hence increasing costs significantly.

9.3.3 Conclusion

The pros and cons of the number of tanks have been summarised in Table 9-1 below.

Single tank		Multiple tanks	
-	No redundancy	+	Redundancy
+	Less structural material	-	More structural material
+	Less cryogenic insulation	-	More cryogenic insulation
+	No interconnection required	-	Cryogenic piping for interconnection
-	Increased span length	+	Decreased span length
+	One towage operation	-	Multiple towage operations

Table 9-1 Evaluation of number of tanks

Because the costs of the LNG containment system (cryogenic insulation) have great influence on the total costs of the storage tank, the relation between the number of tanks and these costs has been analysed. The unit rate of the membrane containment system is assumed 960 USD per square meter.

Due to fixed costs for each extra tank, and the increased wall area that has to be insulated, the costs of the containment system increase rapidly when building more, smaller tanks to obtain the same storage volume. From Figure 9-6 can be seen that the containment system costs, being approximately two third of the total costs, increase with more than 15% when building two tanks of 100,000 m^3 instead of one tank of 200,000 m^3. This does not even include increased costs for structural material.

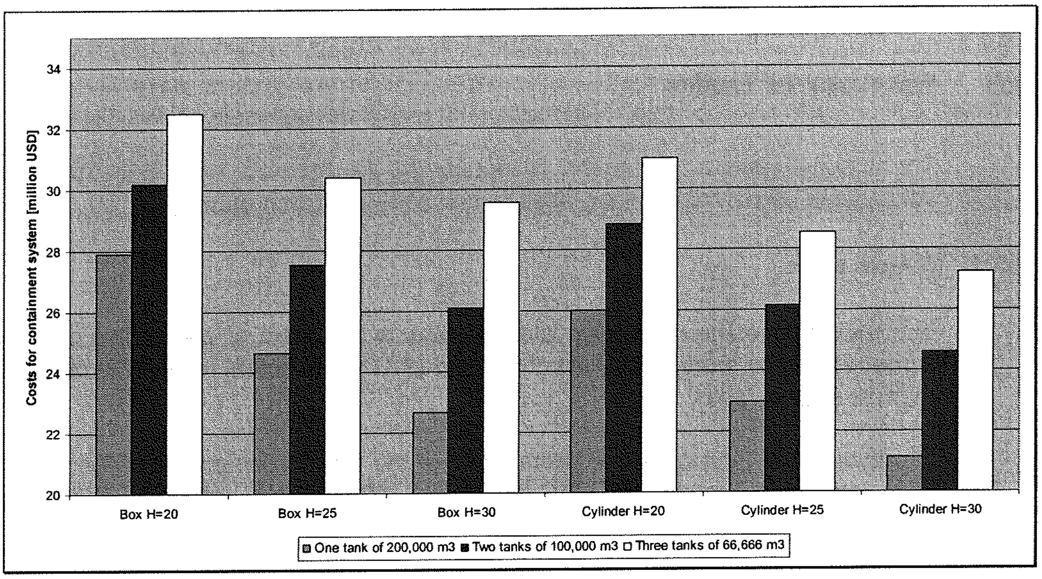


Figure 9-6 Relation between containment system costs and number of tanks

Because of this large cost difference, one single LNG tank with a volume of 200,000 m^3 will be constructed. The lack of redundancy can be compensated with increased safety measures. The span length should be minimised by selecting an optimal shape for the caisson (see paragraph 9.4). Placing bottom slab ribs or skirts should allow for some settlements of the subsoil. The costs for the complex towage operation of such a large structure are assumed to be approximately equal to the costs for towage of several smaller structures.

9.4 Shape

The shape of the caisson will be determined by constructive as well as operational requirements. In this paragraph only the choice between prismatic or cylindrical shaped storage tanks will be made. Preliminary dimensions of the caisson will be discussed later on.

9.4.1 Prismatic

The shape of a prismatic, rectangular box caisson has several advantages to other shapes. When constructing in concrete, the rectangular shape is convenient for shuttering and reinforcement. To achieve the optimal deck span length, the footprint of the caisson should be designed rectangular rather than square. Inner dimensions of such a storage tank would be in the order of 140 m long by 60 m wide and 25 m high. The rectangular deck area can be used efficiently for installation of the process equipment. There is extensive offshore experience available on constructing box-shaped caissons.

To maximise the use of the available space within the caisson for the storage of LNG, rectangular shaped containment tanks are required. This requirement limits the number of containment systems that can be applied. Compared to a cylinder, the more angular shape of a prismatic tank will be more vulnerable to ship collisions and can cause significant scouring of the seabed.

9.4.2 Cylindrical

Practically all existing land-based storage tanks have a cylindrical shape. The (least expensive) full containment system (see paragraph 9.6) then can be applied. A cylinder is the optimal shape to deal with the loads of the cargo, resulting in reduced thickness of the walls, hence an economical design. The cylindrical shape is also more favourable when looking at the reflection of waves and the scouring of the seabed.

However the available deck space in the form of a circle is not convenient for the installation of the process equipment. Furthermore, when constructing a single tank, the required diameter to achieve sufficient storage volume will be large, depending on the height of the tank. For instance with an inner height of 25 m, the required diameter lies in the order of 100 m to achieve 200,000 m^3 storage volume. This will lead to uneconomical deck span lengths.

9.4.3 Conclusion

The main advantages and disadvantages have been summarised in Table 9-2.

	Prismatic tank		Cylindrical tank
+	Easy to shutter and to reinforce	-	More complex to shutter and to reinforce
+	Efficient shape of deck area	-	Inconvenient shape of deck area
-	No full containment system possible	+	Full containment system possible
-	Increased scour at edges of caisson	+	Less scour
-	Increased wave reflection	+	Less wave reflection
-	Corners vulnerable to collision	+	Less vulnerable to collision
+	Reduced span length possible	-	Huge span length of deck and base slab

Table 9-2 Evaluation of shape of storage tank

From this table it seems that the cylindrical tank has more advantages than the prismatic one. However when looking at the preliminary dimensions of the tank, the span length of the deck and base slab are an important aspect. For a volume of 200,000 m^3 and a specific height, the diameter of the cylinder, and therefore the span length, is automatically determined. With a prismatic tank, the length of the caisson can be increased with decreasing height, while the width (maximum span length) remains constant. Some possible inner dimensions of cylindrical and box-shaped caissons are given in Table 9-3.

Height (m)	25	30	35	40	45	50	55
Diameter of cylinder (m)	101	92	85	80	75	71	68
Dimensions of box (L x W in m)	145x55	121x55	104x55	91x55	81x55	73x55	66x55

Table 9-3 Comparison of possible inner tank dimensions (cylindrical and prismatic)

Depending on the construction and installation method (see paragraph 9.8), the maximum height of the caisson is likely to be determined by its draft during towage. Taking into account that to the inner height mentioned in the table at least another 10 meters should be added for construction and ballast compartments, it will be unlikely that the inner height will exceed 35 meters. Therefore it can be concluded that the internal diameter of the cylinder will easily exceed 85 meters. Such a span length might be technically feasible, but it will be a highly uneconomical solution.

Furthermore it is likely that ballast compartments will be required. In the case of a cylinder, the ballast cells will have to be separated by constructing radial walls in the bottom and walls of the structure. Previous studies (Ref 21) have shown that in that case the required thickness of the walls is determined by the local shear stresses in these walls rather than the global strength of the entire cylinder. One of the large benefits of a cylinder, the efficient load distribution, is then nullified.

Considering the above, the prismatic shaped caisson with inner dimensions of approximately 145 x 55 x 25 m will be used for accommodation of the LNG storage. Outer dimensions will be larger because of the presence of ballast compartments in walls and bottom. The vulnerability of the corners can be reduced by chamfering the edges of the caisson. This will also have a positive effect on the scouring of the seabed, although additional bottom protection might be required.

9.5 Structural material

Although it is common practice to select concrete as the construction material for offshore structures, a qualitative assessment of the advantages and disadvantages of possible materials will be made in this paragraph.

9.5.1 Concrete

Concrete as structural material has been successfully used in a large number of offshore projects. Especially combined with reinforcements the material is fatigue resistant and durable, even in an offshore environment. Pre-stressed concrete can withstand very high loads such as large dropped objects or ship collision. Concrete has proven to have good fire resistance, and its behaviour at cryogenic temperatures (in case of an LNG spillage or tank rupture) is favourable to other structural materials.

Constructions of concrete are known to be relatively heavy. An advantage of the weight is that less ballast is required to achieve sufficient stability, but on the other hand it means that the caisson has a large draft when afloat, thus limiting the maximum height of the caisson that can be constructed in a dock.

9.5.2 Steel

Generally a steel structure will be more expensive than a concrete structure, although this is quite dependent on factors such as local labour availability, cost and skills, shipyard order books and schedules. When using steel a lighter construction is possible because less material is required to provide sufficient strength. A reduced weight will be an advantage during the towage of the structure to the site location, for the draft is decreased. However it will be a significant disadvantage regarding the buoyancy of structure when installed, because the weight is still required for stability of the tank. A large volume of additional ballast would then be required.

The characteristics of the material are less favourable than for concrete: Steel is sensitive to corrosion, it has a low fire resistance and it can become brittle when exposed to cryogenic temperatures.

The use of steel allows for a reduced construction time and the possibility of using an existing shipyard (if large enough) instead of a dry dock for the construction of the tank. This increases the number of available construction sites.

9.5.3 Composite

Bi-Steel consists of steel and concrete, designed to make the most efficient use of their strongest assets - the tensile strength of steel with the compressive strength of concrete.

Pairs of steel plates are permanently connected by an array of transverse bars to form panels that are rapidly assembled and filled with concrete (see Figure 9-7). The panels are made by friction welding both ends of the bars to the faceplates simultaneously.

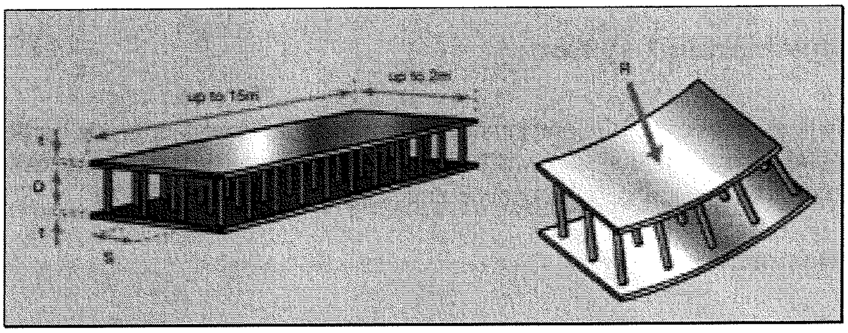


Figure 9-7 Bi-Steel concept (Ref 7)

Due to the modular elements with low weight (unfilled) and the automated construction process, rapid assembly of large structures is possible. It can be assembled in high tech production facilities or in more remote and less sophisticated yards offshore. Once assembled, the steel caisson can be towed to the site where the concrete fill can be poured or pumped into the panels.

The material has a high strength to weight ratio and good impact and ductility characteristics. However, due to use of steel on the outside its behaviour in cryogenic and corrosive environment is less favourable than concrete.

9.5.4 Conclusion

The different structural material characteristics are stated in Table 9-4 below.

Concrete		Steel		Composite	
+	Non-corrosive	-	Corrosive	-	Corrosive
+	Fire resistant	-	Less fire resistant	-	Less fire resistant
+	Good cryogenic behaviour	-	Brittle when cryogenic	-	Brittle when cryogenic
+	Withstand high impact loads	-	Problem with high impact loads	+	Withstand high impact loads
-	Heavy (large draft)	+	Less heavy (medium draft)	+	Light when unfilled (low draft)
+	Heavy (less ballast)	-	Less heavy (more ballast)	+	Heavy when filled (less ballast)
-	Slower but easy construction	+	Fast construction	+	Fast but complex construction
-	Purpose built graving dock	+	Shipyard	-	Specialised facility

Table 9-4 Evaluation of structural material for storage tank

Concrete has obvious advantages when looking at its primary material characteristics. Although the concrete caisson has a larger draft when afloat, it also requires much less ballast when installed, which seems to be a more decisive criterion.

Steel does not seem a logical choice for an offshore environment. The main advantage is that the steel caisson can be constructed in a shipyard within a relatively short period. However the availability of a shipyard with sufficient dimensions is doubtful.

The composite Bi-Steel has some interesting advantages, especially the possibility to tow the steel-only caisson to the site and then fill it with concrete. However specialised knowledge and equipment is required for construction and installation, hence increasing costs. Future potential for the use of this material should be investigated, but for now it is considered to be outside the scope of this study.

The caisson for the LNG storage will be constructed in reinforced concrete. Depending on the final outer dimensions of the design, the structure can either be built in an existing dry dock or in a purpose-built graving dock. The draft of the structure should be optimised and might be decreased by the use of air cushions during towage (see paragraph 9.8).

9.6 LNG containment system

Generally three possible LNG containment systems can be identified. In this paragraph only the pros and cons of each system will be discussed. A more comprehensive description of these systems can be found in Appendix B and Appendix D.

9.6.1 Full containment system

The Full Containment system is widely used in land-based storage tanks for cylindrical tanks. The steel inner tank is then only loaded by the ring tensile forces exerted by the LNG, resulting in efficient use of material. The FC system used to be the cheapest system available, but in some cases the membrane system is competitive.

9.6.2 Membrane system

In the membrane containment system the structural and tightness functions are separated. This results in flexibility of shape, being capable of using the space in a rectangular shaped caisson efficiently. The system is not affected by thermal stress and accepts rapid cool-down.

9.6.3 SPB-IHI system

The SPB-IHI containment system (see Figure 9-8) has been applied in a few LNG carriers. It has a good service record but it is known to be expensive. Because of the prismatic shape of the inner tanks it can be used in a rectangular caisson. The tanks being self-supporting structures can easily be installed in the caisson.

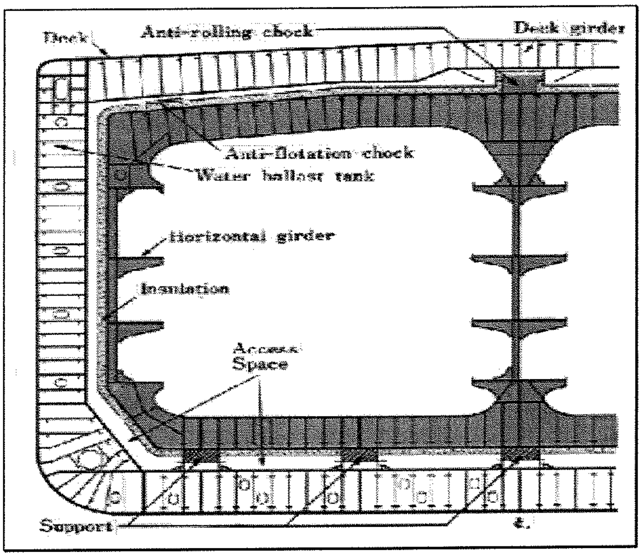


Figure 9-8 Cross section of SPB tank in LNGC (Ref 35)

9.6.4 Conclusion

For a prismatic caisson the full containment system is not applicable. The SPB-IHI containment system has been used rarely and is expensive. In a more detailed analysis of the containment designs the benefits of being able to inspect the tank from inside the GBS should be investigated. On cost alone industry experience shows little benefit for this system. Therefore it has been chosen to use the membrane containment system to contain the liquid natural gas.

9.7 Ballast material

The choice of ballast material mainly depends on weight and cost. Three different ballast materials will be discussed below.

9.7.1 Water

Using plain seawater as ballast material is very cost effective. The water ballast can be given a secondary function: instead of installing a complex heating system in the concrete bottom slab of the tank (required to prevent freezing of the water), the water can be circulated with a pumping system. However the relatively low density might result in a very large volume of ballast that is required for the stability of the caisson.

9.7.2 Wet sand

Wet sand has a density of approximately 2000 kg/m^3, which means that for equal ballast weight only half the volume compared to water ballast is needed. A suction hopper dredger can easily collect the sand and pump it into the compartments. A heating system installed in the bottom slab is required to prevent the wet sand from freezing. Although seawater will be less expensive, sand as ballast material is still relatively cheap.

9.7.3 Wet iron ore

Iron ore can also be used as ballast material, which has a wet density of approximately 3000 kg/m^3. Although its weight could be an advantage because less ballast volume is required, it can also have a negative impact on the thickness of compartment walls due to the increased loads of the ballast material. Furthermore iron ore is expected to be almost twice as expensive as sand.

9.7.4 Conclusion

The choice for ballast material will be made during the determination of the optimal dimensions of the caisson in relation to the total construction costs. This optimisation process will be carried out in chapter 13.

9.8 Construction and installation

The construction and installation of the Gravity Based Structure will be a complex operation. The choice of the construction method will have great influence on the optimal dimensions of the structure. Three significantly different construction and installation procedures have been identified in this paragraph.

9.8.1 Single phase

The procedure of construction and installation of the LNG storage tank in one single phase has been illustrated in Figure 9-9 below.

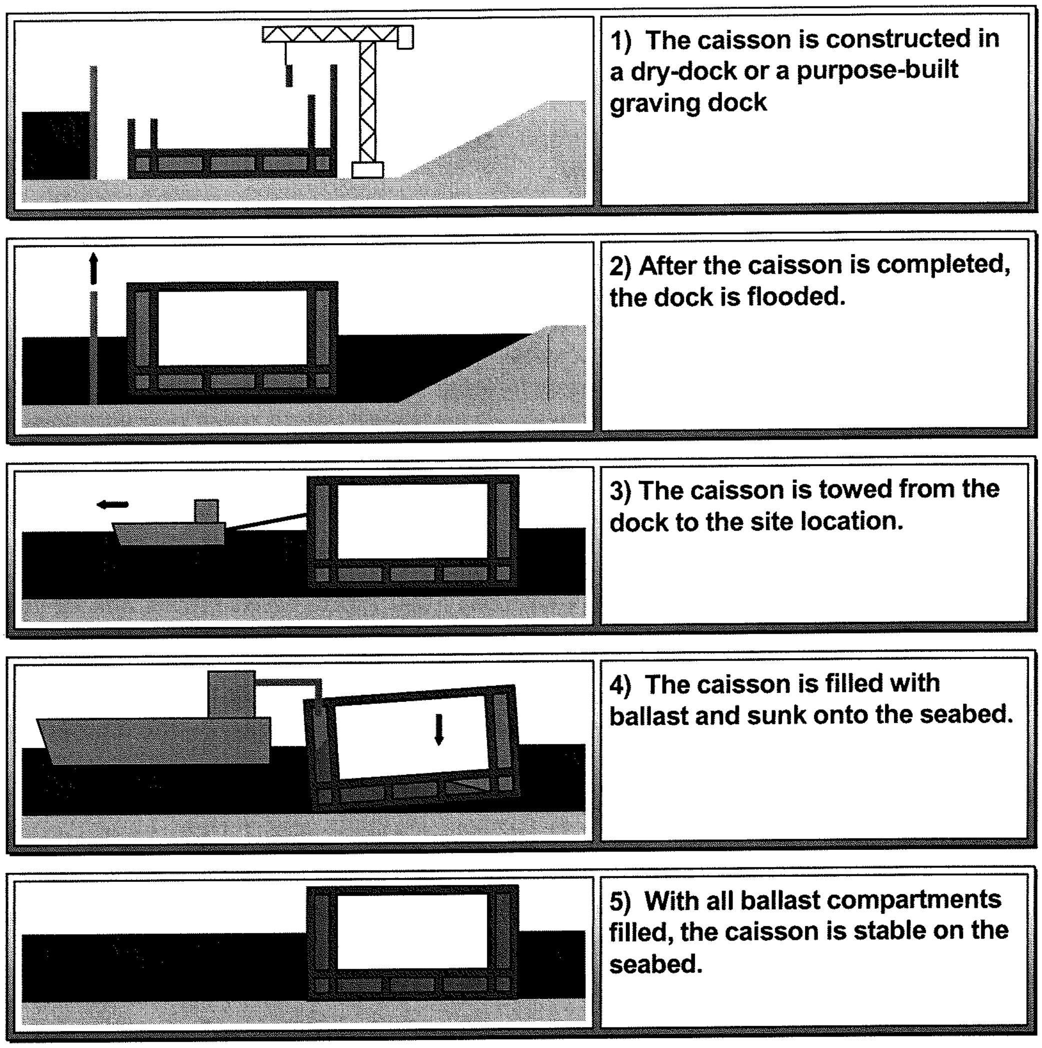


Figure 9-9 One-phase construction and installation sequence

When an existing dry-dock is used for the construction of the caisson, the dimensions of the structure are limited by the maximum available space in the dry-dock. Moreover, existing dry-docks provide limited depth (or draft, when flooded), which restricts the maximum height of the caisson. As a rule of thumb, the draft of a concrete caisson when afloat (without ballast) amounts to 40 – 50 % of its height. Whether this maximum allowable height is sufficient for installation on site, should be checked by buoyancy / stability calculations.

Due to the massive dimensions of the structure, the number of existing dry-docks that are able to accommodate the storage tank will be very limited. This number will decrease even more if the distance from the dock location to the site is considered.

If a new purpose-built graving dock is constructed, its dimensions can be adjusted to be able to accommodate the structure. Still, a suitable location for such a dock has to be found, preferably close to the site location. When sufficient depth is not available at the location, an expensive access channel has to be dredged. Also a significant drainage system is required to prevent the dock from flooding.

A major advantage of a single phased construction is that the containment system and the process equipment can already be installed when the structure is still in the dock. This will reduce costs significantly (compared to offshore installation).

9.8.2 Two phases

The construction and installation procedure can also be separated into a dock-phase and an offshore phase, as is shown in Figure 9-10 below.

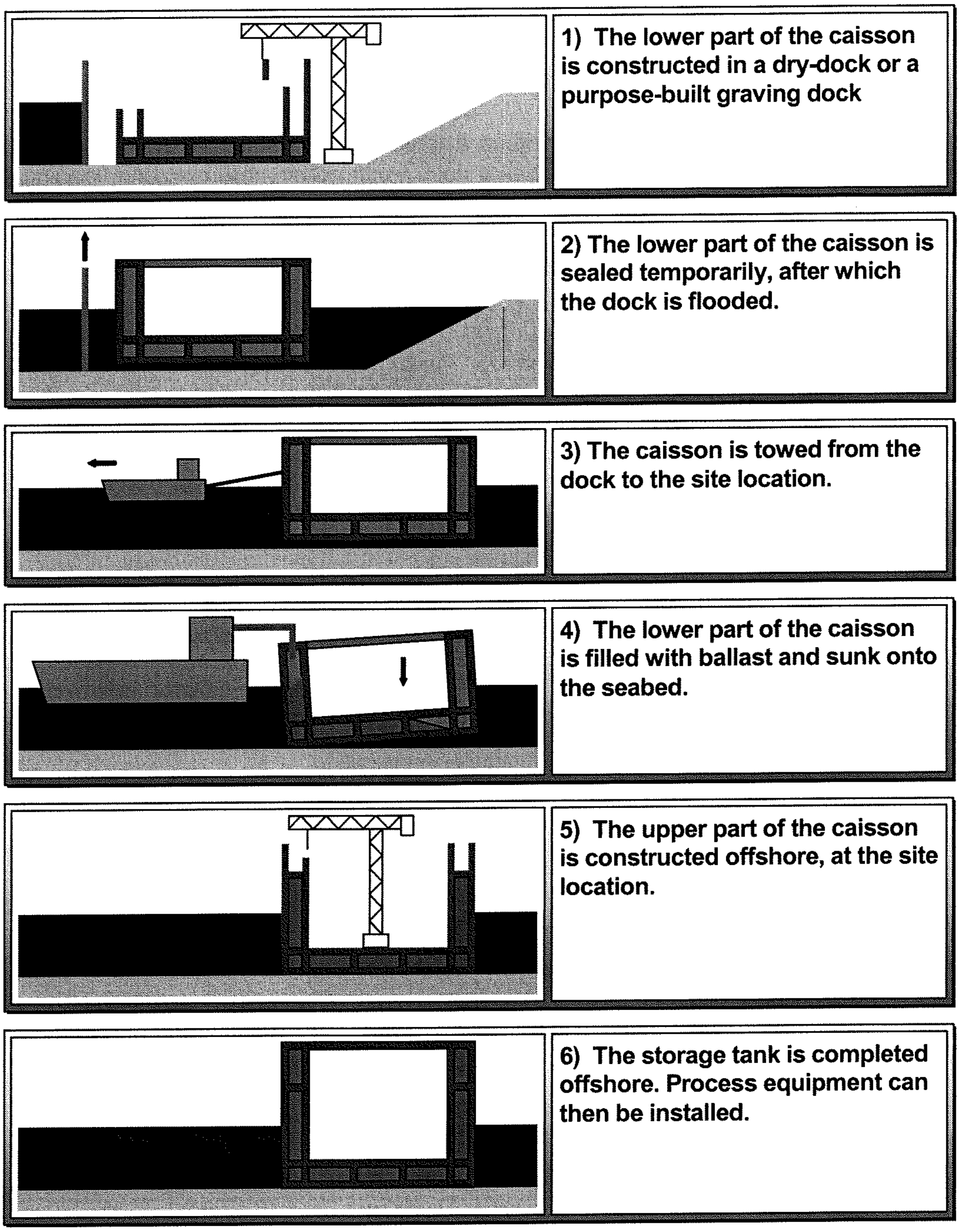


Figure 9-10 Two-phase construction and installation sequence

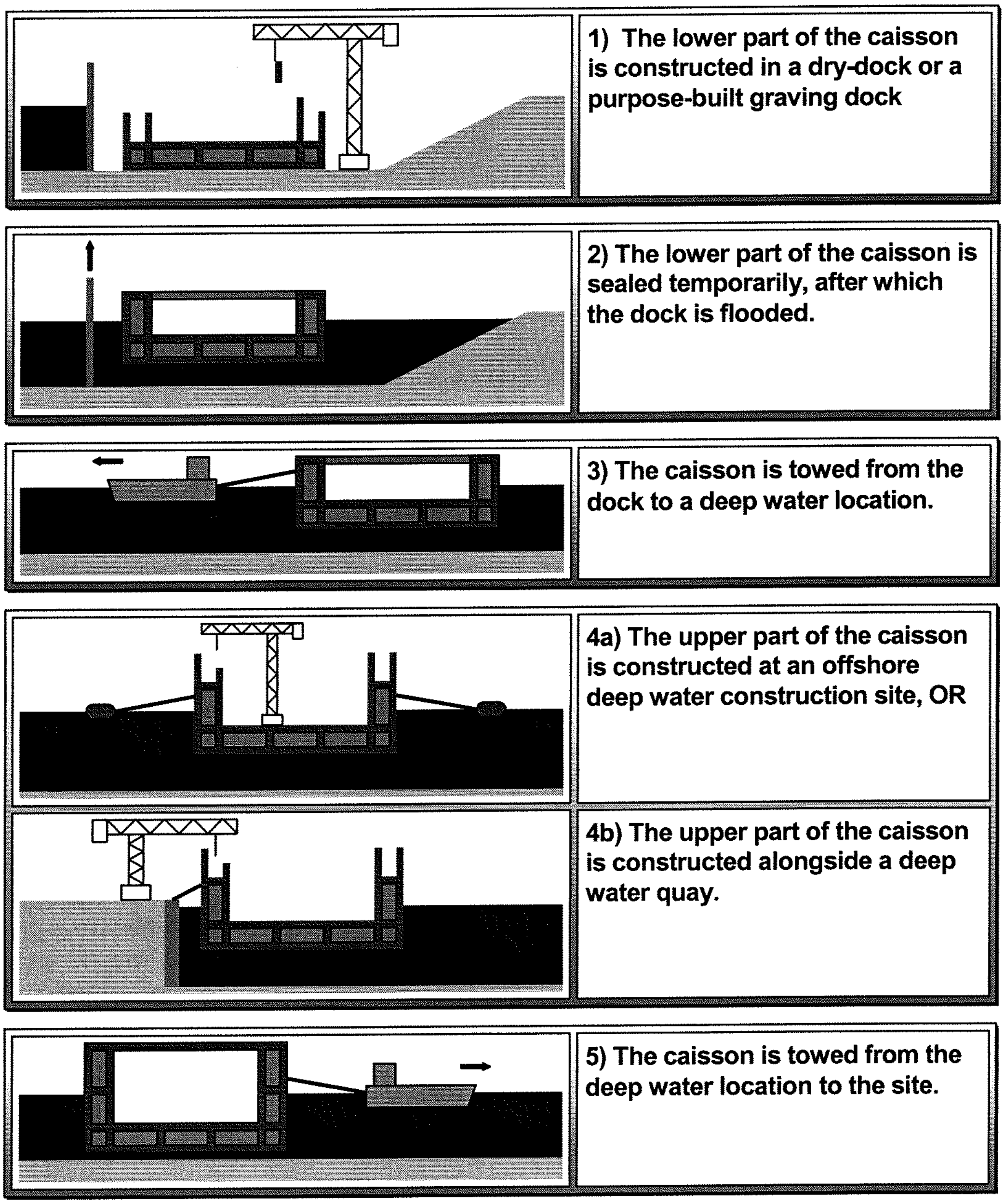
The main reason for constructing the caisson in two phases is that a larger overall height of the structure can be achieved. First, the lower part is constructed with the maximum height allowed by the dimensions of the dock. This part is then ballasted at the site location. It should be checked that the freeboard after the ballast operation is sufficient to protect the structure from daily wave action.

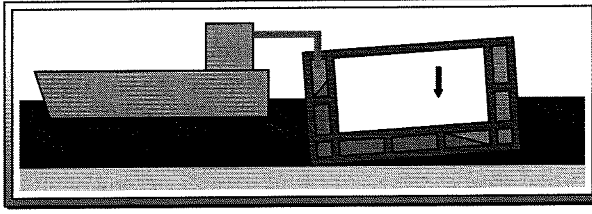
The upper part of the caisson will be constructed offshore at the site location. This means that also the installation of the LNG containment system has to take place in an offshore environment, which is relatively expensive. Moreover the offshore installation of the equipment deck (deck mating) will be a complex operation, for the deck must be elevated before it can be placed on top of the caisson.

However because of the increased maximum height of the total structure (compared to a single phased construction) the amount of ballast that is required for stability is much less. As a consequence, dimensions of ballast compartments can be reduced.

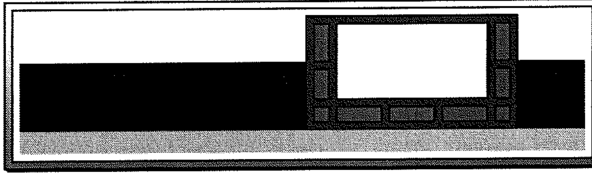
9.8.3 Three phases

In Figure 9-11 has been described a three-phased construction and installation: starting onshore in a dock, progressing at a deep water construction site, and finishing at the site location.





6) The caisson is filled with ballast and sunk onto the seabed.



7) With all ballast compartments filled, the caisson is stable on the seabed.

Figure 9-11 Three-phase construction and installation sequence

Compared to a construction in two phases, the major advantage of this alternative is the fact that building the upper part of the caisson and installing all equipment can be done at a location which is relatively protected from harsh metocean conditions. Compared to the previous method, the deck mating operation is less complex, because the caisson can be partially ballasted to reduce the freeboard while the deck is floated on top of the caisson. The final height of the caisson is only limited by the draft available at the site location (or at the deep water construction site).

Constructing alongside a quay provides easy access for construction equipment and material. But probably the only place where a deep water quay can be found, will be in a nearby deep water port. It is likely that such a quay is already being used by for example the unloading of large container vessels. If instead, the quay will be occupied by the construction of the storage tank for a long period, this will probably result in high costs.

Instead of a quay a deep water construction site can be used, if available. Hiring such a site will be less expensive, but it lacks the easy access of a quay. Compared to a two-phased construction, it should be checked if the additional protection provided by such a site weighs up against the additional costs of the extra towage operation.

9.8.4 Conclusions

The different aspects of the construction methods have been summarised in Table 9-5 below.

Single phase		Two phases		Three phases	
-	Draft limited by dock and site	+	No draft limitations	o	Draft limited only by site
+	Membrane installation onshore	-	Membrane installation offshore	-	Membrane installation offshore
+	Deck mating onshore	-	Complex deck mating offshore	o	Deck mating offshore
+	Protected construction site	-	Exposed construction site	o	Medium exposed construction
o	One large towage operation	+	One small towage operation	-	Two towage operations
-	High dock requirements	+	Low dock requirements	+	Low dock requirements
+	Good access	-	Poor access	o	Medium access
-	High material costs	+	Low material costs	o	Medium material costs
+	Low construction costs	-	High construction costs	o	Medium construction costs

Table 9-5 Evaluation of construction methods

As a conclusion can be said that the allowable draft of the caisson in different phases will be the governing factor. In principle the single phased construction will be the cheapest solution, looking at construction aspects only. However, the dimension analysis (see chapter 12) might result in a caisson with optimal dimensions that do not allow for a single phased construction method. In that case the three-phased method is favourable to the two-phased method due to a less exposed construction site and a less complex deck mating operation.

Then it would be preferable to choose a different site location providing a water depth that matches the higher draft requirements of the completed caisson.

Recapitulating, the selection of the construction method will be postponed until the optimal outer dimensions of the caisson are known. These dimensions will be determined in chapter 12.

9.9 Conclusions

The conceptual design choices that have been made in this chapter are summarised in Table 9-6.

Aspect	Choice
Foundation	Gravity based structure
Number of tanks	One
Shape of tank	Rectangular
Structural material	Concrete
LNG containment system	Membrane
Ballast material	Depending on dimension analysis
Construction and installation	Depending on dimension analysis

Table 9-6 Conceptual design choices

The inner dimensions of the tank will lie in the order of 145 by 55 by 25 meters to achieve a storage volume of 200,000 m^3. This concept will be worked out further in chapter 12.

10 Screening of mooring concepts

10.1 General

The purpose of mooring systems is to hold the ship accurately in position while the unloading process takes place via the loading arms. In most existing LNG terminals, the ship is moored alongside a quay or jetty berth, while the LNG is transferred using conventional loading arms. At an exposed offshore location however such a system will result in a low terminal throughput due to the high downtime. Alternative concepts will be discussed in this chapter.

The characteristics of the loading arm have great influence on the possibilities for mooring concepts. Therefore first the two main categories of loading arms, side-by-side and tandem, are discussed in section 10.2. Then some existing as well as some new mooring concept are proposed, divided into fixed heading concepts (10.3) and weathervaning concepts (10.4). Finally a qualitative evaluation of all concepts is given in section 10.5.

10.2 Loading arms

The basic requirements for the loading LNG system are to transfer LNG in cryogenic condition from ship to terminal and at the same time to compensate for the motions of the LNG shuttle tanker. The type of unloading arm heavily depends on the choice of the mooring configuration. Basically two types of offloading exist: Side-by-side or tandem offloading. These two options will be discussed below.

10.2.1 Side-by-side offloading

The conventional way to transfer LNG from ship to shore is called side-by-side offloading. The LNG carrier is moored alongside an unloading platform fitted with conventional loading arms, which can be connected to the standard mid-ship manifold. Such a configuration consists of four 16" loading arms with a design capacity of 4,000 m^3/h each: two for liquid transfer, one for vapour transfer and one hybrid arm for liquid or vapour transfer. The extra arm is considered required to increase the reliability of the loading system.

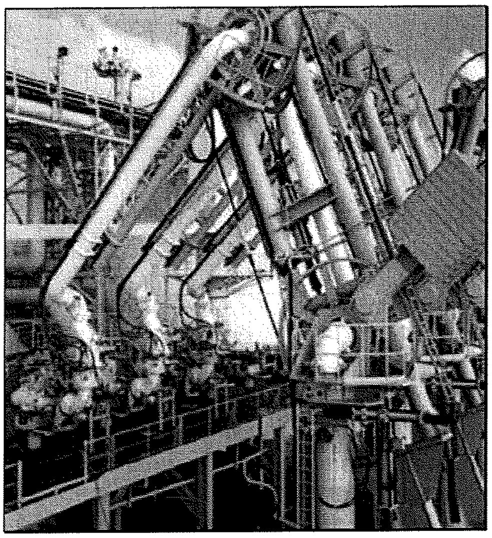


Figure 10-1 Conventional loading arms (Ref 29)

A loading arm consists of a fixed vertical riser and two mobile sections. Swivel joints enable movements in six degrees of freedom. The length of the arms depends on the size of the LNG carriers, their manifold elevations during laden and ballast conditions and their movements due to waves and/or change of water level. An example of a set of conventional loading arms is shown in Figure 10-1.

10.2.2 Tandem offloading

When the LNG is offloaded via the bow of the LNG carrier, the process is called tandem offloading. The loading arm will be connected to a bow manifold. Because a bow manifold is not a standard feature of an LNG carrier, the ship has to be modified. Although the LNG tandem-offloading concept is not applied in practice at the moment, there are several manufacturers presenting designs for such a system (see also Appendix E).

The BTT (Boom To Tanker) system has been designed by FMC to carry out loading in tandem condition. The BTT features a boom that swings around a kingpost to compensate for angular motions in the horizontal plane (70°) and a double pantograph system, which compensates for relative wave frequency motions.

A 24" diameter line ensures the nominal LNG flow rate of a 10,000 cubic meter/hour, while the vapour return line is 16" diameter. The BTT system can operate in significant wave heights of up to 5.5m while following the weathervaning of the LNG carrier for up to 360 degrees. An artist impression of the loading arm is shown in Figure 10-2.

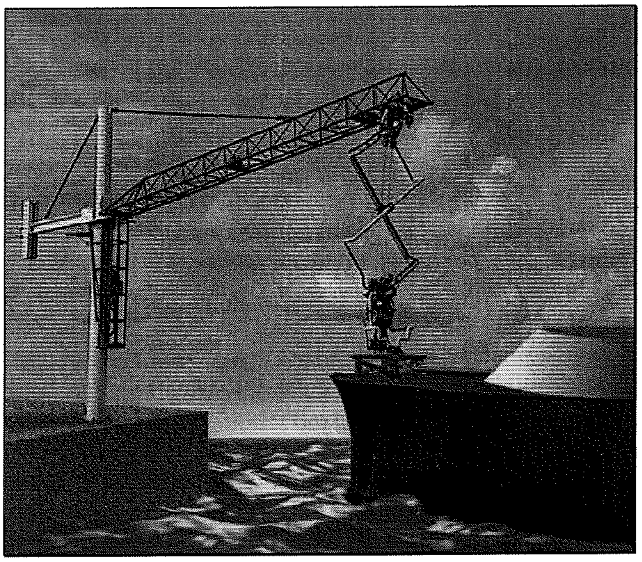


Figure 10-2 Boom-To-Tanker loading arm (Ref 29)

10.3 Fixed heading concepts

10.3.1 Conventional island jetty

The jetty consists of a platform with conventional loading arms, a service area, fire fighting facilities and gangways. Two breasting dolphins with fenders serve to absorb the kinetic energy of the berthing ship. The breast and stern lines can be made fast to the mooring dolphins. The mooring lines fore and aft should have a maximum angle of 15° in the horizontal pane with the normal to the ship, because these lines restrain the lateral movements of the ship and have thus optimum effect. The spring lines should have a maximum angle of 10° with longitudinal axis in order to function most effectively in restraining the surge motion. Likewise the angle of all mooring lines in the vertical plane is limited to 25° with the horizontal.

Tugs are required to assist the ship during its berthing manoeuvre, and mooring launches are needed to connect the bow and stern lines with the mooring dolphins.

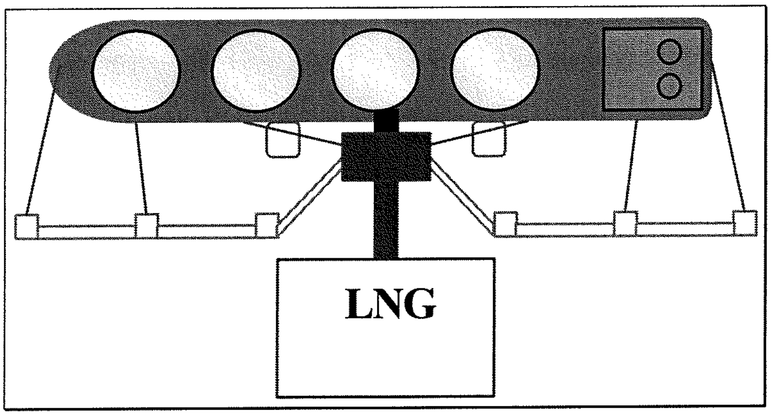


Figure 10-3 Island jetty configuration

An advantage of this conventional mooring configuration is that it has a proven record of service. In relatively benign conditions the ship is easily secured and can be kept relatively stable during unloading. The availability of an unloading platform provides easy access for fire fighting equipment and bunkering operations. Also the maintenance costs for mooring dolphins are lower compared to for instance mooring buoys.

However the costs for construction and installation of the jetty and the dolphins are relatively high. Due to its fixed heading and the "hard" mooring lines, this configuration allows for only little wave, wind and current action resulting in relatively high downtime.

10.3.2 Integrated jetty

This alternative is just an integrated version of the island jetty concept in the previous paragraph. The breasting dolphins can be omitted and the fenders can be constructed onto the outer tank walls. Spring lines can also be fastened to the structure, while bow and stern lines will be attached to ordinary mooring dolphins, interconnected by a gangway attached to the platform. Unloading can take place using conventional loading arms.

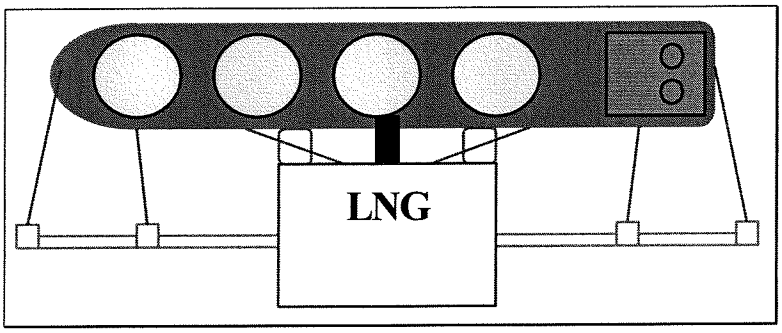


Figure 10-4 Integrated jetty

Compared to the island jetty, the advantage of this concept is the reduction in construction costs achieved by omitting the breasting dolphins. Also the jetty which connects the loading platform with the terminal is not required anymore.

However there are also some disadvantages. Problems can arise when designing the interface between the ship and the structure: The height of the terminal has to be adjusted to the optimal level for the unloading process. Moreover the position where the spring lines are fastened to the tank structure, and the requirements for the angle of the lines, can have great influence on the layout of the deck. Finally this concept probably will have a low score regarding safety aspects, because the distance between the moving ship and the rigid tank structure is very small. In case of an accidental collision the fenders are the only element that can protect the vulnerable terminal.

10.3.3 Conventional Buoy Mooring

In the past the Conventional Buoy Mooring (CBM) system has been applied for medium sized oil carriers. The ship is moored between typically four mooring buoys using “soft” mooring lines. The increased flexibility of the lines allows for larger ship movements and therefore the unloading process can be continued during more harsh weather conditions. A floating hose is used to discharge the oil from the vessel.

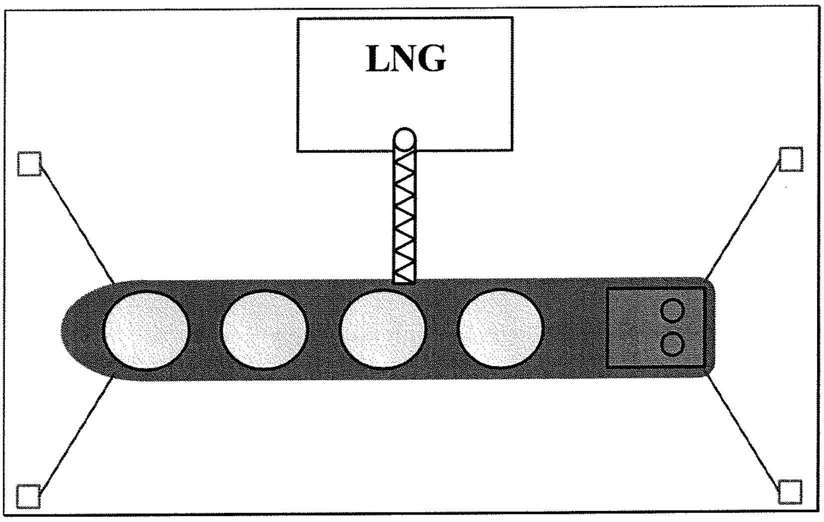


Figure 10-5 Conventional Buoy Mooring alongside terminal

For LNG transfer this hose should be replaced by an advanced type of LNG loading arm (see section 10.2.2) that can accommodate the increased envelope of ship movements. The ship can either be moored alongside (see Figure 10-5) or perpendicular to the terminal (see Figure 10-6). The latter will require a ship modification in the form of a bow manifold.

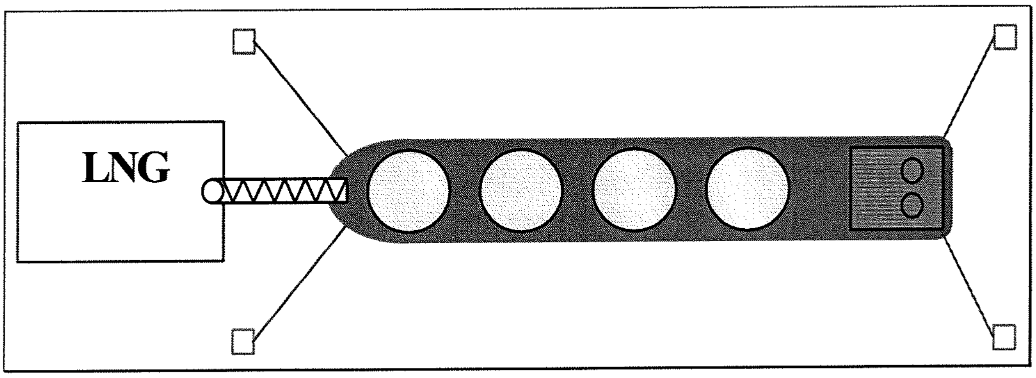


Figure 10-6 Conventional Buoy Mooring perpendicular to terminal

A tug may be required to assist the carrier in the berthing manoeuvre plus a launch to run the moorings.

A benefit of the CBM configuration is that it is safer to operate in sea conditions where it would be difficult to safely operate a fixed berth on a jetty. Furthermore, depending on the prevailing weather conditions, berthing and un-berthing can be carried out without the assistance of tugs. Finally the configuration only requires buoys, reducing the construction and installation costs (CAPEX) of this option.

On the other hand, inspection and maintenance of the buoy system requires a diving operation, resulting in high maintenance costs (OPEX). Also the mooring procedure requires greater seamanship to berth the ship than for a conventional jetty berth. Additionally, the buoy moorings require the use of an ancillary craft (mooring launch) and mooring gangs to run the moorings. These gangs are not allowed to access the buoys during harsh weather conditions, which reduces the advantage of the soft mooring lines.

Another limitation of the CBM system is that it has only been applied for relatively small tankers (maximum 100,000 DWT) causing low mooring forces. A 145,000 m^3 LNG carrier with 300 meters length and a large freeboard is likely to generate very high stresses in the mooring lines.

10.4 Weathervaning concepts

10.4.1 Multiple buoy mooring

Similar to the old stern-loading system of the Brunei LNG terminal, the CBM configuration could be extended with additional buoys in a circular pattern around the stern of the ship, resulting in a system that can moor the ship in several orientations. Depending on the prevailing direction of the wind, wave and current conditions, the orientation with the least resistance to these external actions can be selected for the mooring procedure. Changing orientation during the unloading process is not considered to be possible.

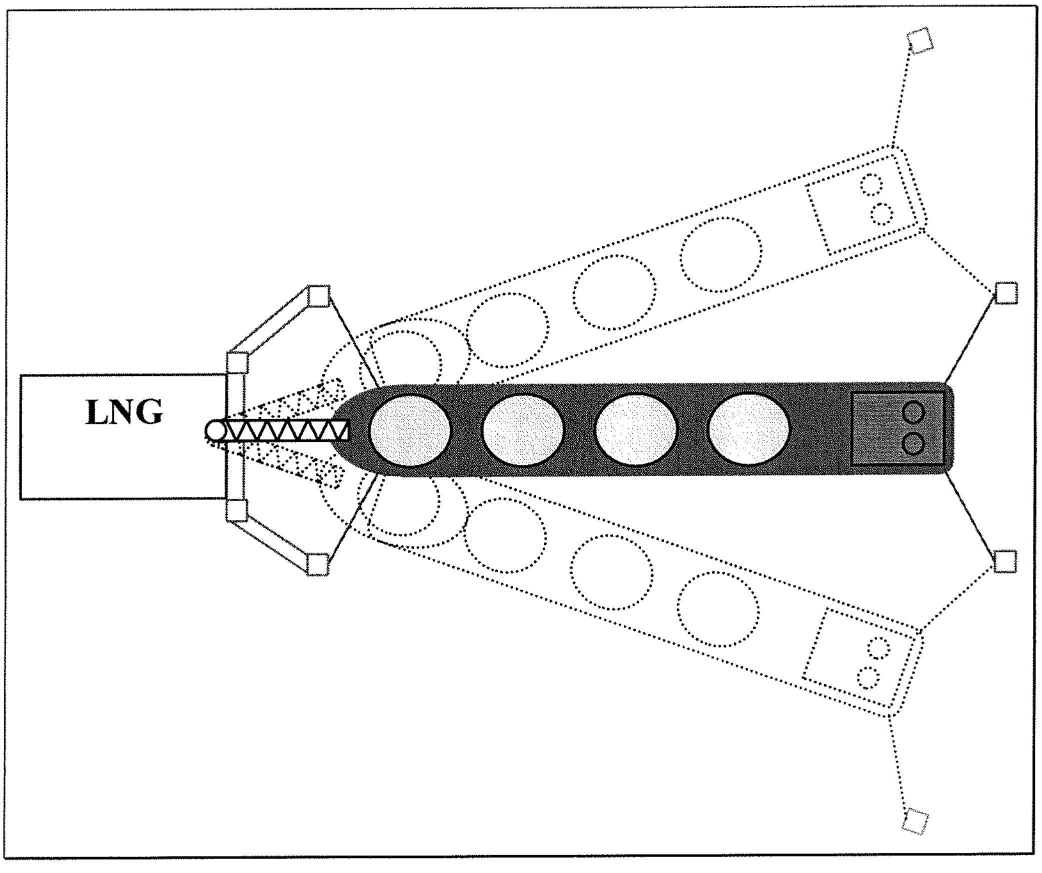


Figure 10-7 Multiple Buoy Mooring

In fact the MBM system is just an extended version of the CBM configuration with the same benefits and limitations. An assessment of the reduction in downtime due to the multi-oriented berth could provide an answer to the question whether the additional costs for mooring buoys and dolphins are justified.

10.4.2 Single Point Mooring

When looking at oil industry practice, the best way to deal with harsh weather conditions is to moor the ship at one point only (SPM). By letting it freely rotate around a fixed point, the ship can continually assume the position of least resistance to wind, current and waves, resulting in lower downtime.

For the transfer of LNG the tandem offloading arm should be installed on a rotating part of the terminal, equipped with cryogenic turning swivels. A yoke or similar construction should be applied to prevent the ship from drifting towards the structure, but apart from that the ship is allowed to move freely.

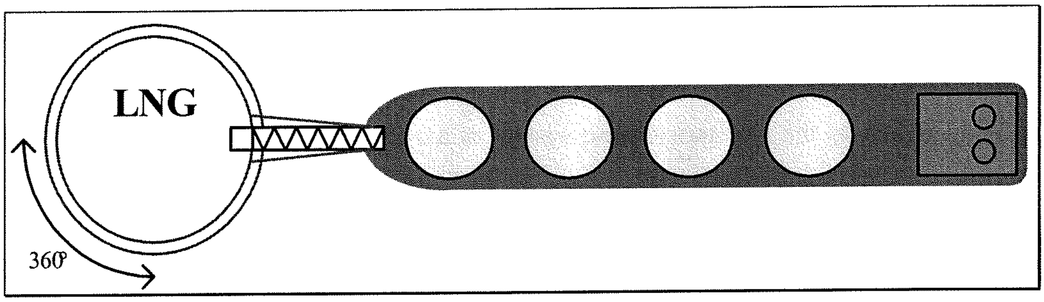


Figure 10-8 Full weathervaning SPM

The major advantage of this system is the reduction of downtime, because high winds and waves up to 5 meters can be accepted before the ship has to leave the berth.

In practice, however, it seems very difficult to combine the full-weathervaning principle with the presence of a large storage tank and the location of the process equipment. The dimensions of the rotating part of the structure will become so large that probably inertia will hinder the weathervaning capability of the system.

10.4.3 Single point mooring with limited weathervaning capability

To combine the fixed storage tank and the weathervaning capability, an alternate SPM concept has been developed. The yoke system on a rotating swivel is located on a separate jacket extended in front of the storage tank. Within a certain envelope, depending on the size of the storage tank and the length of the extended jetty, the LNG carrier is allowed to weathervane. When the vessel drifts across the critical limits of the envelope due to the prevailing conditions, the LNGC must either disconnect or deploy an active system (e.g. thrusters) to compensate for this movement.

Given a certain directionality, this system can be operational under relatively harsh weather conditions. A downtime analysis should be carried out to check whether the expensive loading arm (including jetty and platform) as well as the modifications to the carrier are justified.

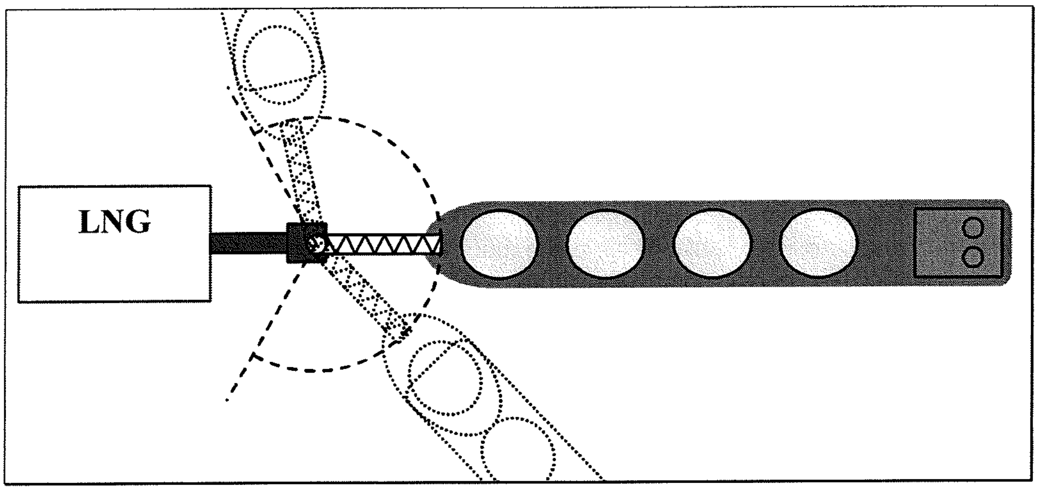


Figure 10-9 Single Point Mooring with limited weathervaning capability

10.5 Evaluation and selection

When taking into account the design of the LNG storage tank (see chapter 9), it can be concluded that the full weathervaning concept presented in section 10.4.2 above is not feasible. For the remaining concepts their typical characteristics have been summarised in Table 10-1 below.

Aspect	Jetty	Int. jetty	CBM	MBM	SPM (limited w.v.)
Nautical access	Reasonable		Poor		Good
Expected downtime	Very high		High	Medium	Low
Maximum wave height	< 1.5 - 2.0 m		< 2.5 – 3.0 m		< 4.0 - 5.0 m
Mooring line forces	High		Low		Medium
CAPEX	High	Medium	Low	Medium	High
OPEX	Low		High		Medium
Assistance during berthing	Mooring launches and tugs		Mooring launches and tugs preferable		None
Assistance during departure	Mooring launches and tugs		Mooring launches		None
Modifications to LNGC	None		Bow manifold		Bow manifold
Damage sensitive parts	Breasting dolphins and fenders		Buoy chains and loading arm		Loading arm

Table 10-1 Characteristics of mooring systems

The jetty concepts suggest a conventional mooring at an exposed location. Due to the fixed heading of the moored ship combined with the stiff mooring system, these concepts can only function during benign weather conditions and are therefore likely to generate a large number of downtime events during operation.

The CBM concept features a "softer" mooring system, resulting in lower mooring forces and higher limits for operational metocean conditions. The MBM concept adds the possibility of selecting the optimal orientation for the moored ship, although the variation in angles is limited. Moreover the vessel's orientation cannot be changed during the unloading process (approximately 18 hours), while wind, wave and current can change direction. However the main problem of these buoy concepts is that assistance from mooring launches is required to attach the moorings lines to the buoys. Because such ancillary vessels cannot operate during harsh weather conditions (H_s > 1.5 – 2.0 m), the operability of the entire terminal is reduced.

The limited weathervaning SPM concept does not have this disadvantage because the ship is able to connect or disconnect without additional help. Also a wide variation in vessel orientations is possible. Therefore a much higher operability can be expected, although the limiting weathervaning envelope could reduce this figure.

Simulations to determine the moored vessel behaviour (see chapter 16) as well as an assessment of the operability of this SPM concept (see chapter 17) will be carried out to decide whether these operability advantages weigh up against the expected high CAPEX as well as the required modifications to the LNG carrier.

11 Conceptual layout of process equipment

11.1 General

An offshore LNG import terminal has never been built offshore. One of the major problems offshore is the lack of space for safe placing of equipment for regasification compared to onshore terminals. In this chapter a conceptual layout for the required process equipment will be presented.

First the conventional terminal layout philosophy will be explained. Then, in paragraph 11.3, a proposal for offshore terminal layout philosophy will be given. Additional safety measures may be required due to the lack of safety distances, which are described in 11.4. Then an assessment is made of the necessary process equipment to be able to achieve the required throughput in section 11.5; a summary of all equipment with their specifications and dimensions is given in 11.6. Subsequently some other requirements regarding the optimal layout are discussed in 11.7. Finally a preliminary design of the process deck layout is presented in paragraph 11.8.

11.2 Conventional separation distances

Obviously a major factor affecting the safety levels achievable on an offshore terminal relative to an equivalent onshore installation is the difficulty of inherent safety by means of generous equipment spacing. Normally the spacing is determined by taking into account the radiation levels impinging on neighbouring process equipment in the event of an ignited release. The potential hazards which are intrinsic to the operation of an LNG import facility are mainly the refrigerated LNG and its gaseous form NG. Functional blocks where these hazards are present or could have an effect on, consist of the ship-unloading facility, LNG storage tanks, regasification equipment, power generation and buildings.

The consequences from a release of hydrocarbons can be flammable clouds, fires, explosions or the brittle fracture of steel structures exposed to LNG. At a specific location the radiation level caused by a release is a function of the pipe diameter, flow, pressure, wind direction and of course the distance from the release.

The vulnerability of the functional blocks affected by the release (generally classified as a jet fire or a pool fire) determines the maximum accepted radiation level. International standards recommend a fire radiation limit of 15 kW/m^2 for accidental releases.

Using the philosophy described above a safety distances matrix for all terminal components can be defined to use as a tool to come to the initial terminal layout.

11.3 Offshore terminal layout philosophy

If equipment separation were to be based on incidents involving a typical release as described above, this would imply that all other process equipment should be located at least 40 meters from any high pressure LNG equipment equipped with flanges or instrumentation fittings, as recommended by the safety distances matrix. This is clearly not feasible on an offshore terminal, so an alternative approach should therefore be applied.

The procedure that should be followed to come to the layout for offshore is described below.

- Following the operational requirements and the process flow, functional blocks of equipment and their relation should be identified, resulting in a preliminary layout.
- Define high-risk and (relatively) low-risk areas and classify all equipment as hazardous or non-hazardous. In practice this means that the high-risk equipment (high-pressure) is separated from the accommodation by the relatively low-risk equipment (power generation and utilities). Other low-risk equipment can be installed below deck.
- Allow safety distances between modules. A detailed QRA (Quantitative Risk Assessment) should determine the minimum distance based on the 3D layout. Additional active or passive safety measures should be taken where necessary (see section 11.4).
- Provide adequate access for construction and maintenance operations. Access for maintenance while adjacent process equipment is live is a vital aspect of the operability of the terminal.
- When looking at construction and installation of the terminal, preferably the different equipment components should be grouped as convenient modules with equal dimensions. The (pre-) commissioning prior to installation of modules on deck should be maximised by minimising the split of (pre-) commissioning systems over more than one module.

Other aspects that should be taken into consideration:

- Ergonomics
- Operational efficiency
- Unobstructed emergency escape routes
- Strategic positioning of fire-fighting equipment
- Future expansion flexibility

11.4 Additional safety measures

As an alternative for the safety distances some possibilities for additional safety measures are described below.

- The equipment must be positioned in a non-congested manner to reduce the potential explosion over-pressures. If this is not feasible blast walls should be provided. The implications of the equipment spacing should be investigated by an Explosion Modelling Analysis.
- A Fire Risk Analysis should be conducted to determine the required passive thermal radiation protection for the process equipment. Occurrence of pool fires should be minimised by taking measures to redirect a spillage overboard. Flanges and instrument tapping orientations shall be considered during the detailed design to minimise the risk of directly impinging jet fires affecting neighbouring equipment.
- Suitable passive and active fire protection systems should be installed in combination with an Emergency Shut Down system, fire/gas detectors, dry powder extinguishers and a fire water monitor system.

11.5 Sizing of equipment

Before a layout of the terminal can be designed, the required number and capacity of the different pieces of equipment should be determined. The sizing of the equipment will be based on an average send-out rate of 1270 m^3/hour, with a peak send-out rate of 1650 m^3/hour and without any minimum send-out requirements.

11.5.1 In-tank pumps

The LNG is pumped from the storage tanks by the low-pressure in-tank pumps and sent at medium pressure (approximately 10 bar) to a recondenser.

Nowadays the maximum conventional pump capacity is 450 m^3/hour. When the terminal is working at maximum capacity, four of these pumps are required, plus one spare. The average flow rate can be achieved with three pumps.

These so-called submerged motor tanks have very good operating records and have high safety and reliability levels. The pumps will be installed in pump wells, which allow their removal for maintenance and installation without disrupting the process. The in-tank pumps do not require any space on the main deck.

11.5.2 Boil-off gas compressor

The principal function of the BOG compressor is to recover tank pressure which continuously increases because of the heat leaks. During holding operation (no ship unloading) the boil-off gas is cooled to –100° C in a desuperheater and sent via a knock-out drum to the boil-off gas compressor which discharges at varying medium pressure levels. The compressed boil-off gas then flows into the recondenser.

The estimated BOG rate is 6,000 kg/hour while in holding mode and 10,000 kg/hour during ship unloading. These values are taken from a comparable project and should be confirmed in detailed design. The actual capacity of the compressor depends on the overpressures in the storage tanks. Normally reciprocating compressors are used in import terminals because the relatively limited quantity of BOG and the relative variation between normal send-out and unloading operation does not allow for the use of a centrifugal compressor. One compressor is active while a second one is standing by.

However, the BOG quantity is in this case large enough to consider a centrifugal compressor. The reliability of such compressors is higher than the reliability of reciprocating compressors, making a spare (stand-by) compressor unnecessary. To save space, this type of compressor will be used. As for preliminary design a similar centrifugal compressor as for comparable onshore terminal designs will be applied, which has a capacity of 11360 kg/h at –150° C. and a weight of 50 tons. Typical dimensions for such a compressor skid including driver are 9 x 4 x 4 m plus an additional lube oil skid of 5 x 4 x 3 m. Because only one compressor is installed the boil off gas will be sent through the vent in case of maintenance.

11.5.3 Recondenser

The recondenser is used to condense the compressed boil off of the storage tanks and is used as a suction drum for the high-pressure pumps. The liquid and the gas enter at the top; the liquid flows through a perforated plate while the gas flows through a second plate with gas chimneys and holes for liquid. The gas-liquid mixture then flows through a packing of stainless steel rings where the LNG is dispersed to create a direct heat exchange surface. The gaseous phase is then completely condensed, and the liquid is transported to the high-pressure pumps.

The BOG recondenser should have the same capacity as the compressor, i.e. 11360 kg/h, with approximate dimensions of 3.5 m diameter and 6 m height.

11.5.4 High-pressure pumps

The LNG from the recondenser is pumped by the high-pressure pumps (also called LNG booster pumps or send-out pumps) and sent at a pressure of approximately 90 bar to the vaporisers.

The approximate maximum capacity for high-pressure pumps will be used, also matching the capacity of the in-tank pumps. It is common practice to select the same capacity for all the pumps, except if a low start-up or minimum send-out rate is expected, which is not the case. Therefore five (4 + 1 spare) of 450 m^3/hour HP pumps should be installed to deal with the peak send-out rate.

The HP pumps are of a vertical can-mounted type, with either an internal submerged motor or an external motor. Typical dimensions of these pumps are 6.2 x 2.0 x 2.0 m.

11.5.5 Vaporisers

Generally there are three types of vaporisers available:

- Open rack vaporisers (ORV)

  Open rack vaporisers utilise seawater as heat source. Seawater flows down at the outside surface of the aluminium heat exchanger panel and vaporises the LNG inside the panel. The ORV features low OPEX, high reliability and safety and easy construction and maintenance. Therefore it is commonly used for base load operation.

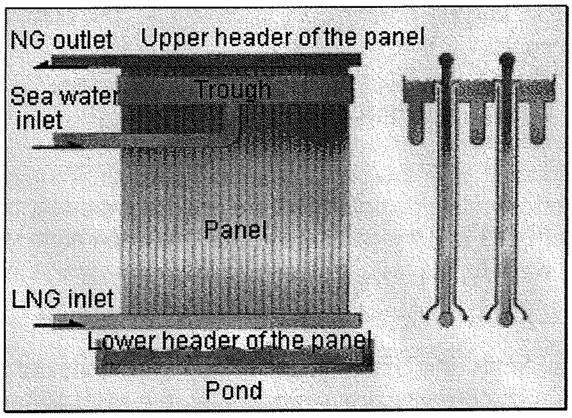


Figure 11-1 Flow scheme for ORV (Ref 64)

The capacity and dimensions of an ORV are varying significantly for different manufacturers. Some examples have been given in Table 11-1. The relatively new "high-performance" ORV's achieve higher capacities with smaller dimensions due to improved panel tube design.

Capacity	Dimensions	Sea water flow	Source
100 ton/hour	14 m x 7 m	3500 m^3/h	[Ref 64]
170 ton/hour (HP)	8.4 m x 8.4 x 9.9 m	3000 m^3/h	[Ref 66]
180 ton/hour	23 m x 7 m	7200 m^3/h	[Ref 64]

Table 11-1 Open Rack Vaporiser design data

- Submerged combustion vaporisers (SCV)

  LNG is vaporised by passing through a stainless steel bundle submerged in a warm water basin which is heated by one or several submerged combustion burners using gas as fuel. About 1.2% of the vaporised LNG is used for heating. Fuel gas comes from the fuel gas system at 3.5 bar which is normally fed by the compressed boil-off gas. The SCV features low initial CAPEX, quick start-ups and shut-downs and has a wide allowable load fluctuation. Therefore the SCV is mainly applied for emergency or peak shaving operation, but in inland areas it can also be used as a base load vaporiser.

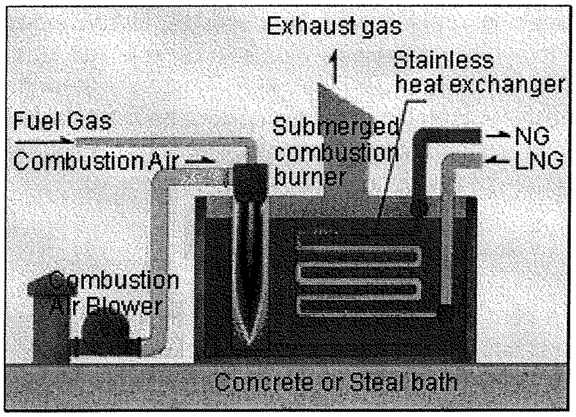


Figure 11-2 Flow scheme for SCV (Ref 64)

Some examples of SCV capacities and dimension are given in Table 11-2.

Capacity	Dimensions	Source
100 ton/hour	8 m x 7 m	[Ref 64]
120 ton/hour	17 m x 9.5 m x 10 m	[Ref 66]
180 ton/hour	11 m x 10 m	[Ref 64]

Table 11-2 Submerged Combustion Vaporiser design data

- Intermediate fluid vaporisers (IFV)

  IFV's transfer heat to the LNG via an intermediate fluid, typically propane or freon. These vaporisers offer an alternative method for using seawater to vaporise LNG without the risk of freezing with direct seawater-LNG exchange. A secondary heat exchanger is vaporising the intermediate fluid which then recondenses while after heating up the LNG. Hence the intermediate fluid is in constant circulation. Some examples of IFV with their capacities and dimensions are shown in Table 11-3.

Capacity	Dimensions	Source
138 ton/hour	17 m x 2.1 m	[Ref 34]
150 ton/hour	17 m x 3 m	[Ref 66]

Table 11-3 Intermediate Fluid Vaporiser design data

Although SCV's are relatively compact, they consume a lot of gas resulting in high OPEX. The IFV's are more efficient but require a propane installation (as intermediate fluid) which is known to be hazardous. Because the High-Performance ORV's are reliable and cost-effective in use they will be chosen for this offshore terminal. However, due to the low sea water temperature in Boston winters, which is a few degrees above zero, the ORV's sea water intake may require an additional heating system to prevent the water from freezing.

Considering the process flow and flexibility of equipment it is convenient that the number of vaporisers equals the number of HP pumps. Therefore they should have approximately the same capacity, i.e. 450 m^3/hour (200 ton/hour). Capacity of ORV's can easily be upgraded by installing additional panels. The relation between the capacity and the dimensions of an ORV is known to be approximately linear. Therefore (4+1) x 200 ton/hour ORV's have been selected with assumed dimensions of 6.1 m by 11.2 m.

11.5.6 LNG Unloading

The fundamental requirements for the loading LNG system are to transfer LNG in cryogenic condition from ship to terminal and at the same time to compensate for the motions of the

LNG shuttle tanker. The unloading arms should be designed for an industrial standard throughput of 10,000 m^3/hour to avoid unnecessary high ship service time.

The type of unloading arm and its location (on the main deck or on a trestle) heavily depends on the choice of the mooring configuration. Possible mooring configurations with the appropriate LNG offloading systems have been discussed in Chapter 10.

11.5.7 Vent

Terminals are provided with a vent or flare stack to dispose of boil off should there be an equipment failure or if the rate of boil off exceeds the capacity of the BOG compressors.

When using a flare the boil-off gas will be burned at the top of the flare stack, while with a vent the gas will just be emitted into the air. From an environmentally point of view, the burning of gas causes less damage than venting it. However, the flare must burn continuously, while the vent will only be used when there is an acute surplus of boil off. Industry practice shows that incidental venting has a smaller impact on environment than a continuous flare.

When looking at the safety aspects, the difference between a flare and a vent is minimal. Therefore a vent is most suitable for the offshore terminal. The capacity should be 25 ton/hour of natural gas. A vent stack with a height of 40 m is included to provide sufficient distance between the flare and other equipment.

11.5.8 Power generation

A linear relation between the peak send-out rate of a terminal and its power requirements has been assumed. Based on data for a similar terminal project (17.5 MW, 2400 m^3/hour peak send-out using open rack vaporisers), a power requirement of 12 MW for the offshore terminal will be taken as a first estimate.

Following the redundancy requirements, three (2 + 1) generators should be installed on the platform. Three Solar Mars 90 gas turbine generators (or similar) will provide 9.5 MW each (ISO continuous duty output). Dimensions are 14.5 x 2.8 x 3.6 m each with a weight of 64.7 ton. Two high-pressure fuel pumps will supply the fuel gas for the turbines.

However, because of the relatively modest power requirements, the possibility of obtaining this power from the power grid onshore should also be taken into consideration. A submarine power cable of approximately 5 km would be required for connection with the power grid. Obviously construction of this connection could be combined with the construction of the gas pipeline to shore. Taking this into account, the costs for the construction of the power line are estimated at 5000 m x 150 USD/m = 750,000 USD. Comparing this figure with the costs of the generators (one Mars 90 costs approximately USD 3.5 million) this option would result in large cost savings.

As a consequence the gas turbine generators would not be required anymore, saving space on the platform. On the other hand the reliability of such a configuration will become an issue, because if the power line connection to the shore fails, terminal operation will be down. Installing a set of back-up generators is unattractive, for it will decrease the space saving advantage.

It is unsure whether it is possible to make a connection to the power grid that can provide the required power. Therefore, as a base case, the preliminary plot plan layout will include the set of gas turbine generators as described above.

In case of an emergency a diesel generator should be provided for the vital components of the terminal. Its capacity will not be sufficient for normal terminal operation. The 768 kW generator measures 12 x 2.8 x 3.8 m and is accompanied by one 50 m^3 diesel storage tank.

11.5.9 Utilities

The following utility equipment will be required for the LNG import terminal:

- Sea water intake/outfall installation

  A sea water intake installation is required to feed the heating water for the ORV's. It is assumed that the vaporisers require 4 vaporisers x 4,725 m^3/h = 19,000 m^3/h of seawater. The figure has been determined based on the assumption that the maximum allowed seawater temperature decrease of 7° C is not exceeded. The allowable temperature drop is subject to local environmental considerations. The low seawater temperature in the Boston winters could be a potential problem. A more detailed analysis is recommended looking at the possibility to increase the throughput and reduce the temperature drop in the water and possibly supplement this with submerged combustion vaporisers.

  This amount will be provided by 3 (2 + 1) sea water intake pumps with a capacity of 10,000 m^3/h each. They should be located at the edge of the platform, where vertical pipes connect the pumps with the intake basin located in the concrete substructure. The intake basin is fenced and equipped with band screens and filters.

  Seawater from the ORV's is collected in a concrete channel and sent by gravity to the water outfall located far from the intake to prevent recirculation problems.

- Fire water system

  Submerged pumps supplying a seawater-based firewater system are proposed. However the required seawater for extinguishing fires can easily be tapped from the seawater intake installation described above. One plus one spare additional firewater pumps with a capacity of 1500 m^3/h each are foreseen. The materials selection for process equipment should be consistent with the use of seawater.

- Fresh and potable water system

  The terminal requires a 300 m^3 water storage tank to supply fresh and clean industrial water. Part of this water is fed to the potable water treatment package which includes a 180 m^3 storage tank.

- Instrument air system

  The compressed air will be necessary for the instrument air system (pneumatic valves) and service air (maintenance). The standard air plant package measures 8 by 8 meters.

- Nitrogen gas system

  The purpose of nitrogen is to maintain a positive pressure in the insulation spaces and for other safety devices used on the ship. Furthermore it will be necessary for purging the process plant during maintenance and it shall be supplied from a N2 storage tank. The nitrogen package consists of two nitrogen vaporisers with dimensions 2 x 4 meters and liquid nitrogen storage tank of 10 m^3. Depending on the final quantities of N2 required, it may be necessary to consider an air separation unit, instead of storing liquid N2.

- Fuel gas

  Fuel gas heaters supply the gas turbines, which require approximately 10,000 kg/h. One heater and one spare are foreseen to heat the gas coming from the vaporisers. The dimensions are 3.5 x 2.5 meters for the heaters, and 0.5 x 1.8 meters for a knock-out drum.

- Metering

Before leaving the terminal the gas passes through a pressure regulating and metering station, where it also can be odorised. The metering package consists of 2 x 100% parallel meter runs. Because the dimensions of such a package are relatively large, it has been assumed that the metering can also be done in a small separated building on land, located where the gas pipeline reaches the shore.

11.6 Equipment list

The main components discussed above have been summarised in the equipment given in Table 11-1.

Equipment	#	Specification	Dimensions (m)	Weight (ton)
In-tank LNG pumps	4+1	450 m^3/h – 11.3 bar	Ø 0.9 x H 3.0	1.2
HP pumps	4+1	450 m^3/h – 104 bar	L 6.2 x W 2 x H 2	15.1
Open Rack Vaporisers	4+1	200 ton/h	L 6.1 x W 11.2 x H 8	60
Sea water intake pumps	2+1	10,000 m^3/h @ 40 m	L 2.5 x W 2.5 x H 15	50
BOG Compressor (centrifugal)	1	11360 kg/h @ -150° C	L 9 x W 4 x H 4	50
BOG Compressor lube oil skid	1		L 5 x W 4 x H 4	
BOG Recondenser	1	11360 kg/h – 10 barg	Ø 3.5 x H 6	27
BOG Compressor Scrubber	1	3 barg @ -165° C	Ø 0.9 x H 1.75	0.3
BOG Compressor Scr. drain drum	1	4.5 barg @ -160° C	L 3.4 x Ø 0.6	0.5
Vent + stack structure	1	25 ton/h – 7 barg	L 8 x W 8 x H 40	100
Knock-out drum	1	7 barg @ -200 ° C	Ø 1.9 x H 5.5	4
Solar Mars 90 Gas turbine	2+1	9.5 MW ISO	L 14.5 x W 2.8 x H 3.6	65
Emergency diesel generator	1	768 kW	L 12 x W 2.8 x H 3.8	30
Diesel storage tank	1+1	50 m^3	Ø 2 x H 4	3
Fuel gas heaters	1+1	143 kW – 15.5 barg	L 3.5 x W 2.5 x H 1.5	1.2
HP Fuel gas knock-out drum	1	30 barg @ 65/-29° C	Ø 0.5 x H 1.8	0.3
Fire water pumps	1+1	1500 m^3/h @ 130 m	L 2 x W 2 x H 15	30
Nitrogen vaporisers	1+1	500 Nm^3/h	L 2 x W 4 x H 5	3
Liquid nitrogen storage tank	1	10 m^3	Ø 1.5 x H 5	10
Instrument air plant	1	400 Nm^3/h	L 8 x W 8	40
Fresh/potable water package	1	2 m^3/h	L 8 x W 8	40
Helideck	1		L 30 x W 22	

Table 11-4 Equipment list

The total empty weight of the process equipment is estimated at 2500 – 3000 ton. As a rule of thumb can be said that the total weight of the topsides including structural steel is three times this figure, resulting in 7500 – 9000 ton.

11.7 Other layout requirements

- Buildings

One superstructure will contain accommodation, public rooms, offices and operation centres for the terminal. Access to the warehouse and workshop should be on the main deck level. Accommodation is to be arranged for approximately 30 persons (three gangs of 10 people).

The required area for buildings has been assessed using data from a similar terminal. Of course, offshore space is limited, so the required area has been compressed to smaller estimates, as can be seen in Table 11-5. This assumes removal for process and utility equipment to shore for major maintenance.

Building	Onshore terminal	Estimated for offshore terminal
Administration	600 m2	300 m^2
Control room	350 m2	200 m^2
Warehouse / workshop	900 m2	600 m^2
Accommodation	0 m2	1800 m^2
Total	**1850 m2**	**2900 m^2**

Table 11-5 Building area requirements

The accommodation block will be a three-storey building to provide sufficient area.

- Pipe rack

All process and utility lines are to be routed via the central pipe-rack. The area underneath the pipe-rack is open and will have a floor which is flush with the process deck level of the platform. By having a minimum height between this floor and the first level in the pipe-rack, an unrestricted passage is created to transport material from the process equipment areas to the workshop and vice versa. The area below deck under the pipe rack can be used for electrical and instrumentation cables. The pipe rack is assumed to be approximately 8 meters wide.

- Tank dome

The area of the roof of the LNG storage tank where the piping to the tank enters and exists is called the tank dome. From a safety point of view this area is very vulnerable, which means that it is not allowed for cranes to reach over the dome. The main pipe rack should be connected to the dome. Furthermore the dome should be as close to the loading arm as possible, for the amount of expensive cryogenic piping should be minimised. The tank dome occupies an area of approximately 4 x 4 m.

- Send-out risers

The high-pressure gas is leaving the terminal through the send-out risers. The vertical pipes from the deck to the sea bottom are a hazardous piece of equipment which should be protected at all time.

- Maintenance

Maintenance access is provided underneath the central pipe rack. Spare parts will be delivered by supply vessels mooring alongside a cantilevered landing platform. Lay down areas are needed to provide sufficient space for replacing equipment. Cranes should be provided for vertical transport, while horizontal transport can be carried out with trolleys.

- Heli-deck

An offshore industry standard heli-deck (25 meter diameter) shall be installed on top of superstructure. One complete package for helicopter refuelling shall be provided.

- Lifeboats

At both ends of the platform lifeboats should be available in case of an emergency.

11.8 Design of deck layout

Applying the design philosophy mentioned in the beginning of this chapter, the preliminary deck layout has been determined for a platform of 150 by 70 meters (see chapter 9). Fitting the equipment on a slightly smaller deck should be possible, for the drawing shows significant free areas (lay-down areas).

Furthermore it has been assumed that the loading arm installation is situated on an extended platform, as proposed in one of the mooring concepts in chapter 10. Evidently the choice of mooring concept has great influence on the deck layout.

A drawing of the deck layout has is shown in Figure 11-3 below. It should be noted that the given layout is still in a very preliminary phase. This layout proposal should be envisaged only as a proof of the feasibility of an offshore import terminal from a process equipment point of view. Further analysis regarding process flow, maintenance and especially safety should be conducted in the next stage.

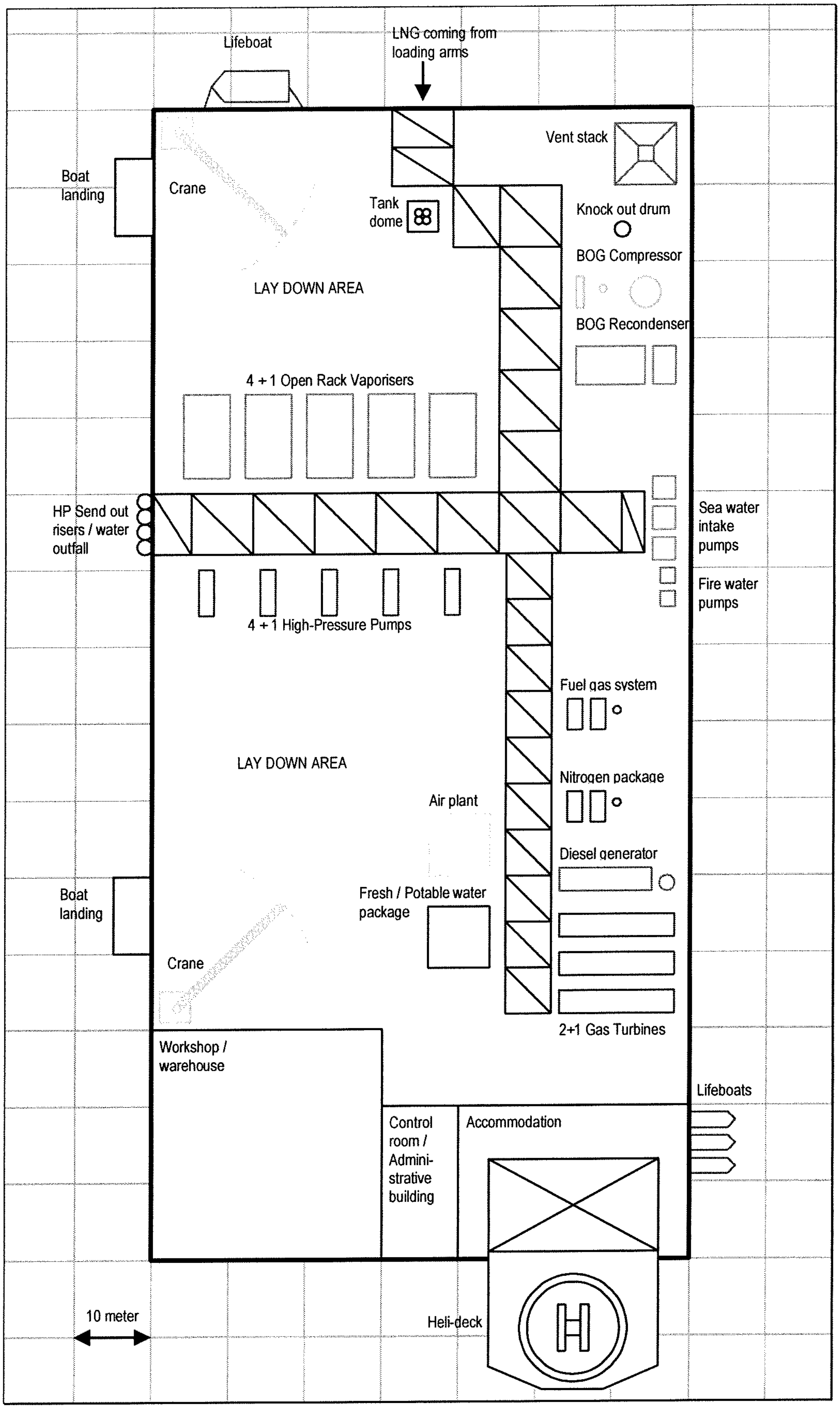


Figure 11-3 Preliminary process equipment deck layout

12 Design of storage tank

12.1 General

The gravity based storage tank concept that has been selected in the previous chapter will be worked out further below. First the structural concept with general dimensions will be presented in paragraph 12.2. In section 12.3 the boundary conditions that pose a limitation to the design will be discussed. Subsequently, an assessment of all loads on the structure will be made in paragraph 12.4. Afterwards, the calculation procedure for checking the on-bottom stability of the caisson (12.5) as well as the marine stability when afloat (12.6) will be described. The methodology of dimensioning the concrete structural elements of the caisson is explained in paragraph 12.7. Unit prices for structural material will be defined in section 12.8.

Finally, all these calculations and limitations will be linked to one optimisation model to find the optimal design for the storage tank. This model with its input and output will be in explained in section 12.9.

12.2 Structural concept

The structural concept of the Gravity Based LNG storage tank consists of a single pre-stressed reinforced concrete caisson. Depending on the selected method of construction and installation (see section 9.8), the structure will be (partially) built in a dock, towed to site in floating condition and then grounded to the seabed with ballast. The GBS houses an LNG storage tank, insulated with the membrane containment system, which can accommodate 200,000 m^3 of LNG.

Between storage tanks and the outer wall of the GBS, a grid of cells is implemented, which will assure sufficient draft during towage, and will be filled with ballast to ground and secure the structure after it has been towed to the site location. In addition, the peripheral belt of wall compartments provides a paramount protection against an accidental ship impact, while the bottom compartments separate the tank bottom from the foundation slab, thus preventing local deformations of the storage tank base. The outer dimensions are in the order of 160 by 70 by 40 meters, see Figure 12-1.

The inner tank is covered by a concrete roof, which is supported by prefab girders. The roof of the structure provides a working platform area of approximately 10,000 m^2 for accommodating all the process equipment and facilities. The platform is elevated sufficiently to be out of reach of the largest waves.

A grid of concrete skirts will be constructed underneath the GBS to achieve sufficient on-bottom stability. Thinner at the bottom than at the top, they will penetrate the soil until the bottom slab ribs arranged below the walls are in contact with the seabed. This system significantly improves the load bearing capacity of the foundation of the structure and has only limited influence of the draft during towage, because an air cushion will be used between the skirts.

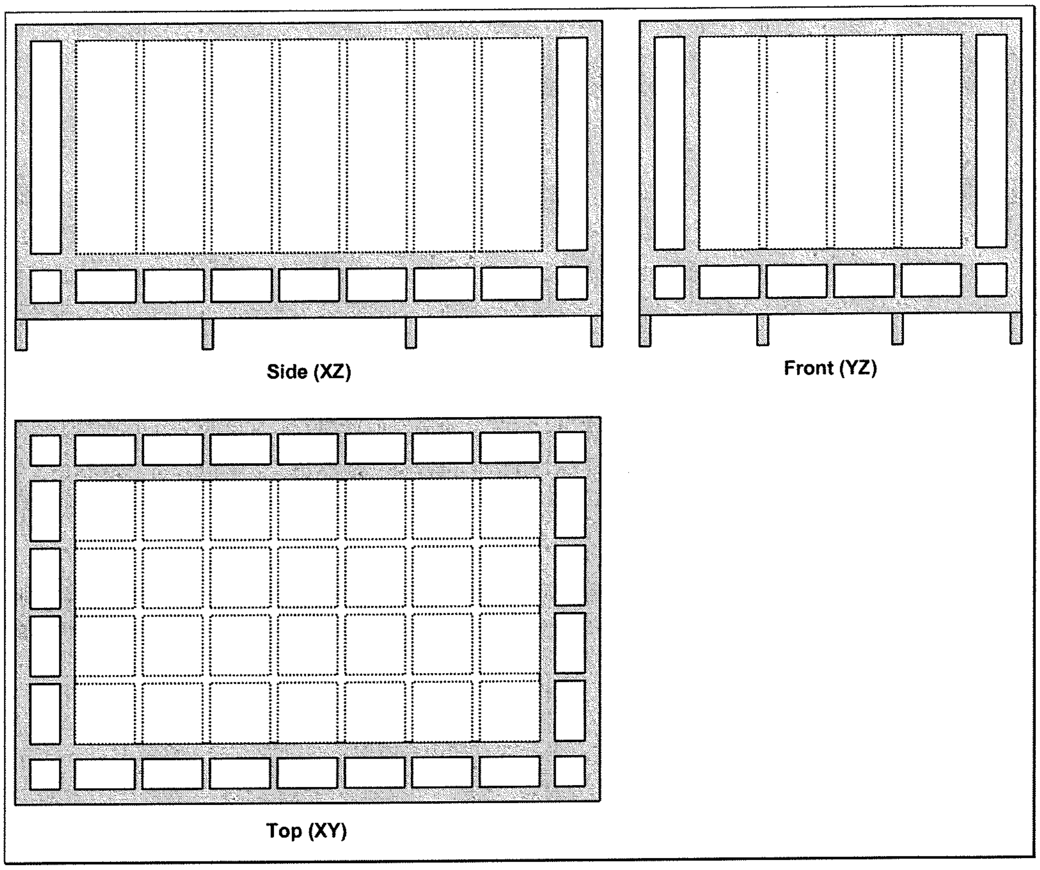


Figure 12-1 Sketch of structural concept of caisson (not on scale)

12.3 Design limitations

12.3.1 Net volume

The net volume of the tank should provide storage for 200,000 m^3 of LNG. However this is not the same as the actual volume of the tank (from floor to the underside of the roof beams), because there are lower and upper limits for the level of the LNG inside the tank. The net capacity is the volume of the tank between the Minimum Normal Operating Level (NOL_{min}) and the Maximum Normal Operating Level (NOL_{max}), which is determined by a tank rim allowance (typically 500 mm).The MinNOL is set by the location of the impeller eye (typically 400 mm above the bottom) and the required head above the impeller eye to prevent cavitation (typically 800 mm).

The average thickness of the containment insulation (t_{ins}) is assumed to be 500 mm. The formula for the gross volume of the inner tank then becomes:

$$V_{gross} = (B_{net} + 2 \cdot t_{ins}) \cdot (L_{net} + 2 \cdot t_{ins}) \cdot (NOL_{\max} - NOL_{\min} + 2 \cdot t_{ins})$$

12.3.2 Minimum height of structure

Because the process facilities will be situated on top of the tank, preferably the storage tank should be high enough to prevent overtopping of water during extreme conditions. Instead a wave retaining wall could be constructed, but that is not recommended because it will have a negative impact on the safety aspects (overpressures in case of a blast, no quick escape routes) and the logistical flow on the platform. Therefore the height of the tank should be determined by the extreme water level increased with the maximum wave crest height and freeboard, which has been shown in Figure 12-2 below.

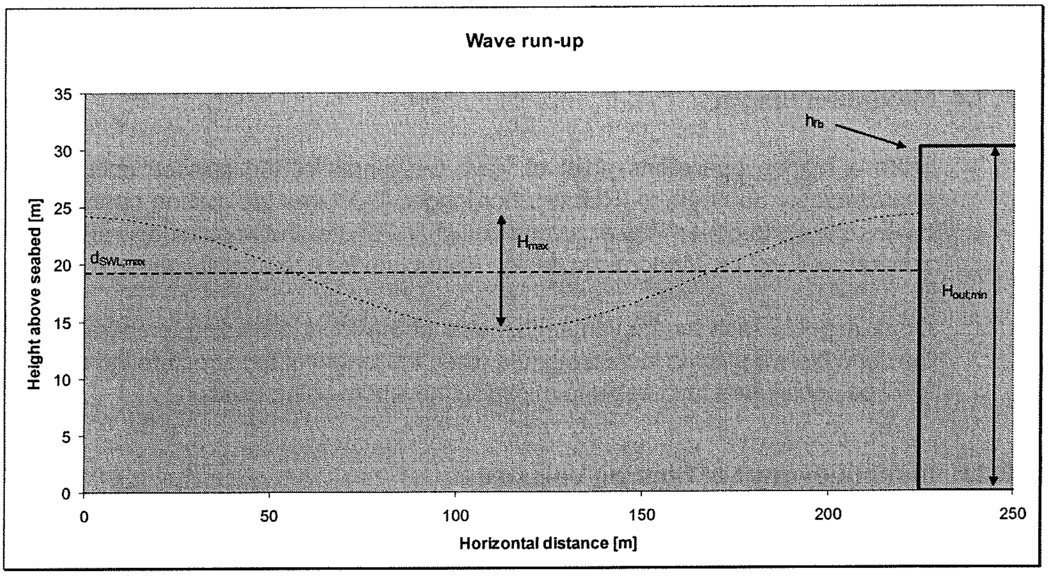


Figure 12-2 Calculation of minimum height of caisson

The maximum wave crest height is calculated by multiplying the maximum wave height with a reflection coefficient (a vertical impermeable wall results in a standing wave in front of the caisson). The minimum height of the caisson to prevent overtopping is then calculated with the formula:

$$H_{out;\min} = d_{SWL} + K_r \cdot \tfrac{1}{2} \cdot H_D + h_{fb}$$

in which

$$H_D = 1.8 \cdot \min\{H_s, 0.5 \cdot d_{SWL}\}$$

$H_{out;min}$	Minimum caisson height to prevent overtopping [m]
D_{SWL}	Water depth at maximum SWL [m]
K_r	Reflection coefficient (for vertical impermeable walls K_r = 2.0)
H_D	Design wave height [m]
h_{fb}	Required freeboard (typically 1.0 m) [m]

The reflection coefficient K_r with a value of 2 holds for a vertical impermeable wall of infinite length. The dimensions of the caisson, however, lie below the length of the maximum occurring waves. A reduced value for K_r = 1.75 will therefore be assumed.

12.3.3 Maximum span width

From the viewpoint of on-bottom stability it is more favourable to increase the width instead of the length of the caisson. However the maximum width is limited by the available prefab concrete solutions to span the width of the inner tank. The prefab technology used in bridges can be applied for the girders in the roof. Their maximum span length will be approximately 60 meters (see section 12.7.2). Larger spans might be achievable by extrapolating the existing prefab girder dimensions, but then the roof construction is likely to be less economical.

A second limitation for the maximum width can arise when taking the construction and marine operations (towage) into account. Existing docks, as well as their access channels, will not allow for outer widths larger than 60 meters. However the caisson is not considered to be constructed in an existing dock. Instead a purpose-built graving dock will be built. The width of the access channel to such a dock should be large enough to accommodate the transportation of the caisson (approximately 3 times the width of the floating body).

12.3.4 Maximum length

From a marine operations point of view, the length of the caisson should be limited to approximately 350 meters (Ref 49). For longer structures the marine operations (open sea tow) are likely to govern the structural design instead of the in-place operating conditions. In general it will not be economic to design a structure for a temporary phase.

Other aspects such as uneven settlements of the foundation and temperature gradients in the concrete will play an increasing role when the length of the structure increases. Therefore it will be considered favourable to minimise the length of the caisson.

12.3.5 Maximum draft of floating caisson

Before the caisson is installed at the site, it will have to be towed to the location in floating condition. Therefore the draft of the floating caisson should be less than the actual water depth at the site decreased with the required under-keel clearance.

Probably skirts will be attached below the bottom slab of the caisson to increase on-bottom stability. The compartments between the skirts can be filled with compressed air, acting as air cushions, thus reducing the draft of the caisson. As a rule of thumb one can assume that, independently from their length, skirts will increase the draft only by 0.5 meters when using air cushions. The formula for the draft of the caisson then becomes:

$$\frac{W_{concrete} + W_{topsides} + W_{trimwater}}{\rho_{sw} \cdot B_{out} \cdot L_{out}} + D_{skirts} \leq d_{site} - d_{ukc}$$

in which

$W_{concrete}$	Weight of concrete structure [ton]
$W_{topsides}$	Weight of topsides (process equipment deck) [ton]
$W_{trimwater}$	Weight of trim water ballast (to adjust draft/angle of caisson during towage) [ton]
ρ_{sw}	Density of sea water [ton/m^3]
B_{out}	Outer width of caisson [m]
L_{out}	Outer length of caisson [m]
D_{skirts}	Additional draft generated by installed skirts [m]
d_{site}	Water depth available at site or during transport [m]
d_{ukc}	Required under keel clearance [m]

The effectiveness of air cushions in open sea conditions should be checked during the next stage.

12.4 Loads on caisson

12.4.1 General

In this paragraph the loads exerted on the structure during the design life of the structure will be assessed. Figure 12-3 provides an overview of these loads.

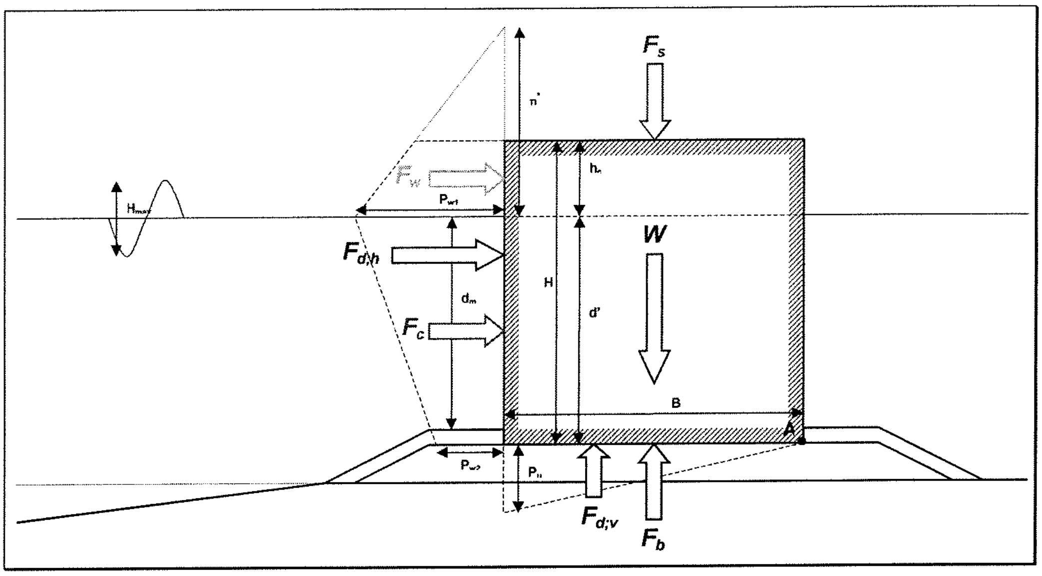


Figure 12-3 Environmental forces on caisson

The most important loads are caused by self-weight (W), deck loads (F_s), buoyancy (F_b), hydrodynamic wave loads (F_d), current loads (F_c) and wind loads (F_w).

12.4.2 Wave loads

The magnitude of wave forces depends not only on the wave height, wave period and dimensions of the structure but also on the resulting hydrodynamic regime. This is determined by the relationship between the width of the structure and the wavelength (Ref 9):

- for B / L > 1, reflection applies
- for 0.2 < B / L < 1, diffraction theory applies
- for B / L < 0.2, Morison's equation applies

In this case, with a W_s around 150 m and a L around 200 m, the ratio lies around the 0.7 - 0.8, which means that the hydrodynamic effects are in the diffraction regime. Sarpkaya (Ref 41) provides formulas for the wave forces in the diffraction regime on a vertical circular cylinder extending from the seabed and piercing the free surface.

$$F_{\max} = \frac{\pi^2 \cdot \rho \cdot H_{\max} \cdot L \cdot D^2}{4 \cdot T_p^{\ 2}} \cdot C_m$$

$$M_{\max} = \rho \cdot g \cdot H_{\max} \cdot L \cdot D^2 \cdot C_m \cdot f$$

$$f = \frac{k \cdot d \cdot \tanh(k \cdot d) + \cosh^{-1}(k \cdot d) - 1}{16}$$

in which

F_{max}	Maximum hydrodynamic force in diffractive conditions [kN]
ρ	Density of sea water [kN/m^3]
H_{max}	Maximum occurring wave height [m]
L	Wave length at site [m]
D	Diameter of cylinder [m]
T_p	Peak period [s]
C_m	Effective inertia coefficient, assumed 0.6, taken from Figure 6.3 in Ref 41 [-]
M_{max}	Maximum hydrodynamic moment in diffractive conditions [kNm]
K	Wave number [m^{-1}]
D	Water depth at site location [m]

However, these formulas not apply for a rectangular caisson, in which case a numerical approach is required, involving developing a surface integral equation and solving this by a discretisation procedure. This is out of the scope of this project at this stage.

Using the Sarpkaya formulas the wave forces have been calculated. The results turned out to be substantially lower than using the reflective theory discussed below. However, it should be noted that for a rectangular caisson with sharp edges, instead of the vertical cylinder, the forces will be much higher than calculated. Furthermore, the diffractive regime changes into the reflective regime when W_s / L > 1, which, in this case, means with wave periods shorter than 12 seconds. Hence, the effects in reflective regime should also be taken into account.

Considering the above, as a first, conservative estimate, the wave forces will be calculated in reflective conditions. A numerical wave model should be used to determine the actual wave forces during detailed design later on.

Goda has provided a method to estimate the hydrodynamic wave forces on a vertical wall in reflective conditions.

$$p_1 = \tfrac{1}{2} \cdot (1+\cos\beta) \cdot (\alpha_1 + \alpha_2 \cdot \cos^2\beta) \cdot \rho_w \cdot g \cdot H_{\max}$$

$$p_2 = \frac{p_1}{\cosh(k \cdot h)}, \quad p_3 = \alpha_3 \cdot p_1$$

$$\eta^* = 0.75 \cdot (1+\cos\beta) \cdot H_{\max}$$

in which

$$\alpha_1 = 0.6 + \tfrac{1}{2}\left[\frac{2 \cdot k \cdot h}{\sinh(2 \cdot k \cdot h)}\right], \quad \alpha_2 = \min\left\{\frac{h_b - d}{3 \cdot h_b}\left(\frac{H_{\max}}{d}\right)^2 ; \frac{2 \cdot d}{H_{\max}}\right\},$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1 - \frac{1}{\cosh(k \cdot h)}\right]$$

in which

p_1, p_2, p_3	Hydrodynamic pressures on structure caused by waves [kN/m^2]
β	Angle of incidence of waves [deg]
ρ_w	Density of sea water [kN/m^3]
H_{max}	Maximum occurring wave height [m]
k	Wave number [m^{-1}]
h	Water depth [m]
h_b	Water depth at a distance of five times the wave height from the wall [m]
d	Water depth above armour units at the toe of the wall [m]
h'	Water depth from toe of structure to water line [m]

12.4.3 Wind loads

The wind force on a structure is given by the following formula, provided by DNV 30.5 (Ref 24):

$$F_w = \tfrac{1}{2} \cdot \rho_a \cdot C_w \cdot A \cdot V_w^{\,2}$$

in which

F_w	Forces on structure exerted by wind [kN]
ρ_a	Density of air [kN/m^3]
C_w	Wind coefficient [-]
A	Area exposed to wind, perpendicular to wind direction [m^2]
V_w	Wind velocity [m/s]

For three-dimensional bodies placed on a horizontal surface, with a length/width ratio of approximately 2 and a height – width ratio smaller than 1, the wind coefficient should be in the range of 1.0 – 1.1 (Table 5.5 in DNV 30.5).

12.4.4 Current loads

For uniform prismatic structures immersed in a uniform current, the steady drag force, which acts at the centre of the area perpendicular to the flow, can be calculated by the formula stated below (Ref 9).

$$F_c = \tfrac{1}{2} \cdot \rho_w \cdot C_d \cdot A \cdot u_c^{\,2}$$

in which

F_c	Forces on structure exerted by current [kN]
ρ_w	Density of sea water [kN/m^3]
C_d	Drag coefficient, 0.6 (round corners) - 2.0 (sharp corners) (Ref 9, Table 7) [-]
A	Area exposed to current, perpendicular to current direction [m^2]
u_c	Current velocity [m/s]

The maximum current load is calculated with a drag coefficient of 2.0 and exposed area A taken as the length of the structure multiplied with the maximum still water level height.

12.4.5 Self-weight

The weight of the caisson consists of several components. The following formula will be applied:

$$W_{caisson} = V_{concrete} \cdot \rho_{concrete} + \alpha \cdot V_{ballast} \cdot \rho_{ballast} + V_{LNG} \cdot \rho_{LNG} + W_{topsides}$$

in which

$W_{caisson}$	Total weight of caisson [kN]
$V_{concrete}$	Total volume of concrete [m^3]
$\rho_{concrete}$	Density of pre-stressed, reinforced concrete [kN/m^3]
α	Ballast compartment fill coefficient (estimated at 90%) [-]
$V_{ballast}$	Total volume of ballast compartments [m^3]
$\rho_{ballast}$	Wet density of ballast material [kN/m^3]
V_{LNG}	Total volume of LNG cargo load in tank [m^3]
ρ_{LNG}	Density of LNG [kN/m^3]
$W_{topsides}$	Weight of topsides installation [kN]

Which components are taken into account depends on the load combination that is considered (see section 12.4.7).

12.4.6 Seismic loads

An earthquake is characterised by rapid horizontal and vertical ground acceleration which will tend to excite any structure supported by either deep or shallow foundations. If the frequency of the ground motions (typically 1 – 2 Hz) coincides with, or is close to, the relevant natural frequencies of the structure, then the structure will be subject to dynamic amplifications. The large weight of a GBS will cause relatively large inertia forces during an earthquake.

The other danger posed by earthquakes is soil liquefaction. A GBS has a shallow foundation, so the possibility of foundation liquefaction should be carefully checked in detailed design.

A GBS is an internally rigid, large mass structure on spring support (soil/structure interaction) with a natural frequency of typically 0.5 – 2 Hz. Consequently it is relatively sensitive to earthquakes. However, the GBS will be designed to withstand large horizontal and vertical loads (on-bottom stability, see section 12.5) and is capable of absorbing a large amount of energy.

Therefore the earthquake condition is, in general, not a governing design condition for the structure (Ref 31).

To verify this assumption, a rough calculation for a non-building, rigid structure has been made according to the method provided by the Uniform Building Code (Ref 69).

$$V = 0.7 \cdot C_a \cdot I \cdot W$$

in which

V	Total design lateral shear at the base of the structure [kN]
ρ_w	Density of sea water [kN/m^3]
C_a	Seismic coefficient (Seismic zone 2A; Very dense soil and soft rock S_C; C_a = 0.18)
I	Importance factor (Hazardous facility I = 1.25) [-]
W	Total seismic dead load (entire tank weight and its contents) [kN]

This calculation results in a shear force of approximately 15% of the on-bottom weight. Calculations below will show that the total horizontal forces caused by waves, wind and current are in the range of 50% of the on-bottom weight.

Therefore it will be assumed that the earthquake shear forces are not a governing design condition. A more comprehensive earthquake analysis should be carried out during detailed design.

12.4.7 Load combinations

The load combinations that will be considered for calculating the dimensions of the caisson are given in Table 12-1 below.

LC	Description
I	Maximum waves, wind and current during the highest water level, full ballast compartments, topsides installed, empty LNG tank.
II	Minimum waves, wind and current during highest water level, full ballast compartments, topsides installed, empty LNG tank.
III	Maximum waves, wind and current during the lowest water level, full ballast compartments, topsides installed, full LNG tank.
IV	Minimum waves, wind and current during lowest water level, full ballast compartments, topsides installed, full LNG tank.

Table 12-1 Description of load combinations

Depending on the considered failure mode the worst case load combination will be applied to calculate the required strength (resistance) during the Ultimate Limit State.

12.5 On-bottom stability

12.5.1 Sliding

When the horizontal forces on the caisson exerted by the wave, current and wind loads exceed the maximum friction force that can be reached between the caisson bottom and the subsoil, sliding will occur.

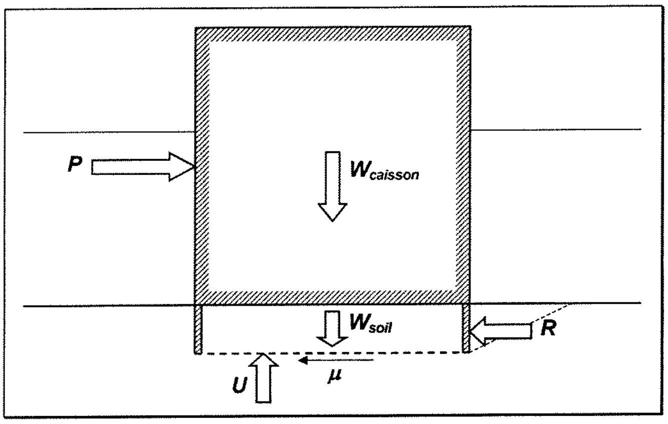


Figure 12-4 Sliding criterion

When skirts are used, the submerged weight of the soil enclosed by the skirts can be added to the weight of the caisson, thus improving stability. Moreover, for the sliding criterion the skirts initiate a passive soil reaction force that also increases stability. The formula for the sliding criterion becomes:

$$\frac{\mu \cdot (W_{soil} + W_{caisson} - U)}{P - R} \geq \gamma_s$$

$$\mu = \tan(\varphi)$$

in which

μ	Friction coefficient [-]
ϕ	Angle of internal friction of subsoil [deg]
$W_{caisson}$	Submerged weight of structure [kN]
W_{soil}	Submerged weight of soil enclosed by skirts [kN]
U	Hydrodynamic uplift force [kN]
P	Sum of horizontal forces on structure (hydrodynamic waves, wind, current) [kN]
R	Passive soil reaction force [kN]
γ_s	Safety factor for sliding criterion (1.5) [-]

12.5.2 Overturning

If the destabilising moment caused by the environmental loads on the structure exceeds the compensating moment caused by the weight of the structure, the subsoil can slip causing the construction to overturn around the heel of the caisson, as can be seen in Figure 12-5.

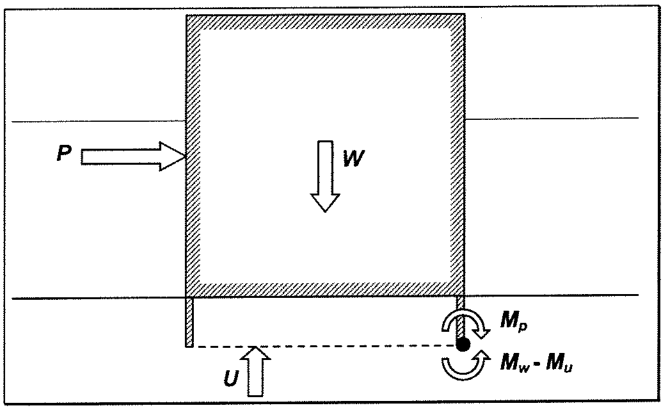


Figure 12-5 Overturning criterion

To prevent this type of overturning, the following criterion must be used:

$$\frac{M_w - M_u}{M_p} \geq \gamma_o$$

in which

M_w	Compensating moment caused by submerged weight of structure [kNm]
M_u	Overturning moment caused by dynamic wave uplift [kNm]
M_p	Overturning moment caused by sum of horizontal forces on structure [kNm]
γ_o	Safety factor for overturning (1.5) [-]

Also has to be checked that the resultant of the ground reaction force lies within the middle third of the cross-sectional width of the caisson, to assure that the caisson remains in contact with the foundation

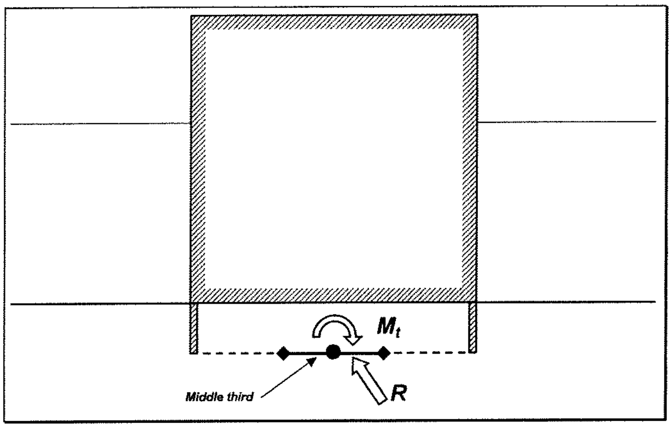


Figure 12-6 Middle third criterion

This is called the middle-third criterion, calculated by

$$\frac{6 \cdot M_t}{B \cdot V} \geq \gamma_m$$

M_t	Total remaining moment on structure [kNm]
B	Width of structure [m]
V	Sum of vertical forces on structure [kN]
γ_m	Safety factor for the middle third criterion (1.0) [-]

The safety factor is 1.0 because this condition is not yet an overturning situation, but merely a middle-third criterion with significant hidden capacity to actual overturning.

12.5.3 Bearing capacity of subsoil

Finally, the bearing capacity of the subsoil has to be checked applying the Brinch Hansen formula for a strip foundation in drained conditions (Ref 36):

$$q_d = 0.5\gamma' B' N_\gamma K_\gamma + (p_0' + a) N_q K_q - a$$
$$N_\gamma = 1.5 \cdot (N_q - 1) \cdot \tan\phi$$
$$K_\gamma = s_\gamma d_\gamma i_\gamma$$
$$a = c' \cdot \cot(\phi')$$
$$N_q = \left\{\tan\left[\pi/4 + 0.5\arctan(\tan\phi)\right]\right\}^2 \cdot \left\{\exp\left[\pi(\tan\phi)\right]\right\}$$
$$K_q = s_q d_q i_q$$

where

$$i_q = \left\{1-0.5\cdot\left[H_{bd}/\left(V_{bd}+A'a\right)\right]\right\}^5$$
$$s_q = 1+i_q\left(B'/L\right)\cdot\sin\left[\arctan\left(\tan\phi\right)\right]$$
$$d_q = 1+2\left(D_b/B'\right)\cdot\left(\tan\phi\right)\cdot\left\{1-\sin\left[\arctan\left(\tan\phi\right)\right]\right\}^2$$
$$i_\gamma = \left\{1-0.7\cdot\left[H_{bd}/\left(V_{bd}+A'a\right)\right]\right\}^5$$
$$s_\gamma = 1-0.4\cdot i_\gamma\cdot\left(B'/L\right)$$
$$d_\gamma = 1$$

in which

q_d	Design unit bearing capacity of subsoil [kN/m^2]
γ'	Effective unit weight of subsoil [kN/m^3]
B', L, A'	Effective width, length and area of foundation [m]
N_c, N_q, N_γ	Internal friction correction factors [-]
s_c, s_q, s_γ	Shape correction factors [-]
i_c, i_q, i_γ	Inclination correction factors [-]
d_q, d_γ	Depth correction factors [-]
p_0'	Effective overburden pressure [kN/m^2]
a	Soil attraction (intercept between Mohr-Coulomb failure line and horizontal stress axis) [kN/m^2]
ϕ, ϕ'	Angle of internal friction of subsoil [deg]
c'	Soil cohesion intercept [kN/m^2]
D_B	Depth to base level [m]
V_{bd}, H_{bd}	Sum of vertical / horizontal forces on structure [kN]
γ_b	Safety factor for bearing capacity of subsoil (2.0) [-]

12.5.4 Summary of failure modes

A summary of the four on-bottom stability failure mechanisms with the appropriate safety factors and the governing load combinations in the Ultimate Limit State has been given in Table 12-2 below.

Failure mechanism	Safety factor	Minimum value	ULS Load combination
Sliding	γ_s	1.5	I, III
Overturning	γ_o	1.5	Not governing
Overturning (middle third)	γ_m	1.0	I, II, III, IV
Bearing capacity of subsoil	γ_b	2.0	I, II, III, IV

Table 12-2 Failure mechanisms and their safety factors

12.6 Marine stability

Aside from the on-bottom stability when the caisson has been installed, also the floating stability has to be checked. The floating stability is secured when a rotation of the caisson, initiated by external forces such as waves or wind, is compensated by a stabilising moment returning to the equilibrium.

The method supplied by (Ref 14) has been used to check whether the design of the caisson meets the floating stability criterion. The procedure, illustrated by Figure 12-7, has been elaborated below.

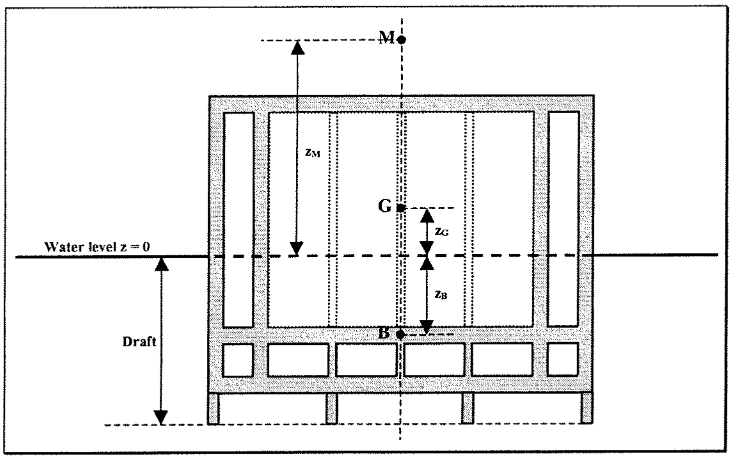


Figure 12-7 Floating stability of caisson

- The total weight of the floating structure divided by the density of seawater and the area of the bottom of the caisson provides the draft (Archimedes).
- The "pressure point" z_B is positioned at half of the draft of the caisson.
- The position of the centre of gravity of the caisson (z_G) is determined by the weight of the individual concrete elements and their position relative to the water level.
- The position of the "meta-centre" (z_M) is calculated applying the following formula:

$$z_B + z_M = \frac{I}{V}$$

$$I = \tfrac{1}{12} \cdot L \cdot B^3$$

in which

I	Moment of inertia of surface area at water level [m^4]
L	Length of caisson at water level [m]
B	Width of caisson at water level [m]
V	Volume of caisson below the water level [m^3]

- Now to achieve static floating stability the following criterion has to be met

$$z_M - z_G \geq 1.0$$

12.7 Concrete dimensioning

12.7.1 Methodology

The method for dimensioning the concrete elements has been simplified in order to roughly calculate the required material quantities. The procedure used in a comparable GBS design study (Ref 21) has been followed. It should be noted that the procedure described below only serves in the conceptual design phase, and that detailed design later on has to confirm the preliminary results of this calculation.

- The calculation is based on reinforced concrete.
- Although pre-stressing will be applied in detailed design, the effects have not been taken into account in this calculation. Because pre-stressing increases the shear resistance

and the prevention of cracks, the results of the calculation will be on the conservative side.

- Based on comparable offshore projects in the past, the maximum acceptable amount of reinforcement is estimated at 300 kg/m^3. The average amount of pre-stressing has been set at 20 kg/m^3. These figures result in an approximate concrete density of 2650 kg/m^3.
- Fifty percent of the 300 kg/m^3 reinforcement is considered as longitudinal reinforcement, which results in a value for the maximum reinforcement percentage for a cross-section:

$$\omega_{0;\max} = 50\% \cdot \frac{\rho_\omega}{\rho_{steel}} = 0.5 \cdot \frac{300}{7850} = 1.91\%$$

- Based on the environmental loads on the element in the Ultimate Limit State (ULS), described in the pages hereafter, the maximum bending moment for each element is calculated.
- The tensile stresses generated by the maximum bending moment (M_d) must be absorbed by the reinforcement (Ref 48)

$$M_d \leq M_u$$

$$M_u = A_s \cdot f_s \cdot z$$

$$A_s = \omega_{0;\max} \cdot bd$$

in which

M_d	Design value of bending moment [Nmm]
M_u	Ultimate value of bending moment [Nmm]
A_s	Cross-sectional area of reinforcement [mm^2]
f_s	Design yield strength of reinforcement steel (typically 435 N/mm^2)
z	Arm (distance from location of resultant of compressive forces to heart of reinforcement)
$\omega_{0;max}$	Maximum reinforcement percentage [-]
b	Effective width of cross-section [mm]
d	Effective height of cross-section [mm]

- The maximum acceptable amount of shear stirrups in plates is estimated at Φ12-200 mm in two dimensions.
- The maximum shear stress (τ_d) must be absorbed by the shear strength of the concrete plus the shear stirrups (Ref 48)

$$\tau_d \leq \tau_u$$

$$\tau_d = \frac{V_d}{bd}$$

$$\tau_u = \tau_1 + \tau_s \leq \tau_2$$

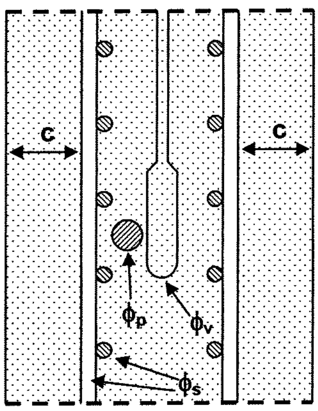
$$\tau_s = \frac{A_{sv} \cdot z \cdot f_s}{bd}$$

in which

τ_d	Maximum shear stress [N/mm^2]
τ_u	Maximum acceptable shear stress [N/mm^2]

V_d	Maximum design shear force in cross-section [N/mm^2]
b	Effective width of cross-section [mm]
d	Effective height of cross-section [mm]
τ_1	Maximum shear stress absorbed by concrete [N/mm^2]
τ_s	Maximum shear stress absorbed by shear stirrups [N/mm^2]
τ_2	Maximum total shear stress in concrete limit [N/mm^2]
A_{sv}	Area of shear stirrups per length unit [mm^2]

- The requirements for bending moment and shear stress determine whether the chosen combination of the dimensions span length, width (b) and thickness (d and z) are sufficient.
- The minimum thickness of the elements is determined by the constructability, as can be seen from Figure 12-8 below. The cross-section should allow not only for cover (c), mild steel reinforcement on both areas in both directions (ϕ_s) and for pre-stressing ducts (ϕ_p), but for placing and compacting concrete (ϕ_v) as well (Ref 13).



$$t_{min} = 2 \cdot c + 4 \cdot f_s + 2 \cdot f_p + f_v$$
$$= 2 \cdot 75 + 4 \cdot 20 + 2 \cdot 80 + 60$$
$$= 450 \text{ mm}$$

Figure 12-8 Minimum thickness of concrete elements

- The following load factors have been applied for the required strength of the elements during the Ultimate Limit State.

Type of load	Min	Max
Environmental load	1.00	1.35
Dead load	0.90	1.10
Live load	1.00	1.50

Table 12-3 Load factors according to API RP-2A LRFD codes

In the following paragraph first the global strength of the structure will be checked, where walls and floors, comprising of several individual elements, will be considered as a whole. Thereafter these individual elements will be checked on their local strength separately. Note that the combination of these two solicitation mechanisms, i.e. for instance a local bending moment in a floor plate which is part of the larger floor rib, which is also loaded with a global bending moment, has not been taken into account. In practice such combinations can either have a positive or a negative effect on the load – strength ratio of the construction. Therefore it is recommended that superposition of global and local effects should be considered in the next phase.

12.7.2 Global strength

To determine the global strength of the caisson the cross section of the structure can be modelled into three basic schematic components, as can be seen in Figure 12-9 below.

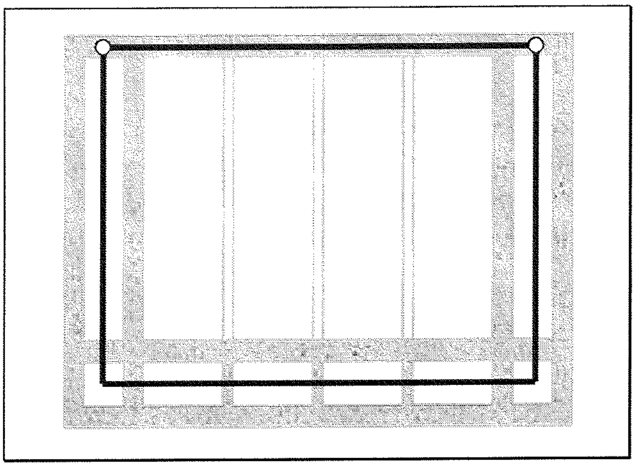


Figure 12-9 Schematisation of caisson structure

These three basic components (wall beams, floor ribs and roof) will each be tested on global strength in the most unfavourable conditions, as has been described below.

- Wall beams

  The wall beams are shown in Figure 12-10 below. Their cross-section has been schematised as an ordinary I-profiled girder.

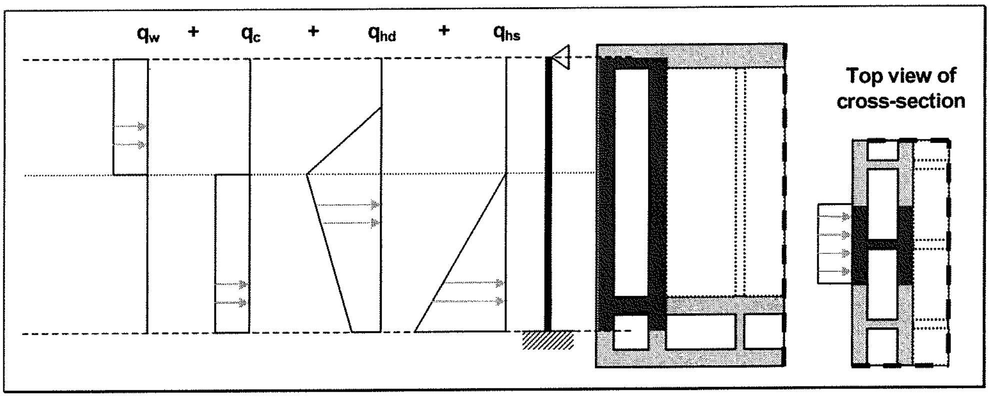


Figure 12-10 Schematisation of wall beam with extreme loading conditions

The Ultimate Limit State (ULS) loading combination for this part of the structure is maximum wind (q_w), maximum current (q_c) and maximum waves (hydrodynamic pressure q_{hd}), all perpendicular to the longest side of the caisson. These loadings have been combined with the hydrostatic pressure (q_{hs}) during the highest possible water level on the outside, and a completely empty LNG storage tank.

- Floor ribs

  The floor of the caisson consists of an inner and an outer slab with ballast compartments in between, together forming a framework of "floor ribs". A top view of the caisson floor is shown in Figure 12-11. Loads on the shaded areas are supposed to be directly transferred to the supporting walls, and will therefore not be taken into account.

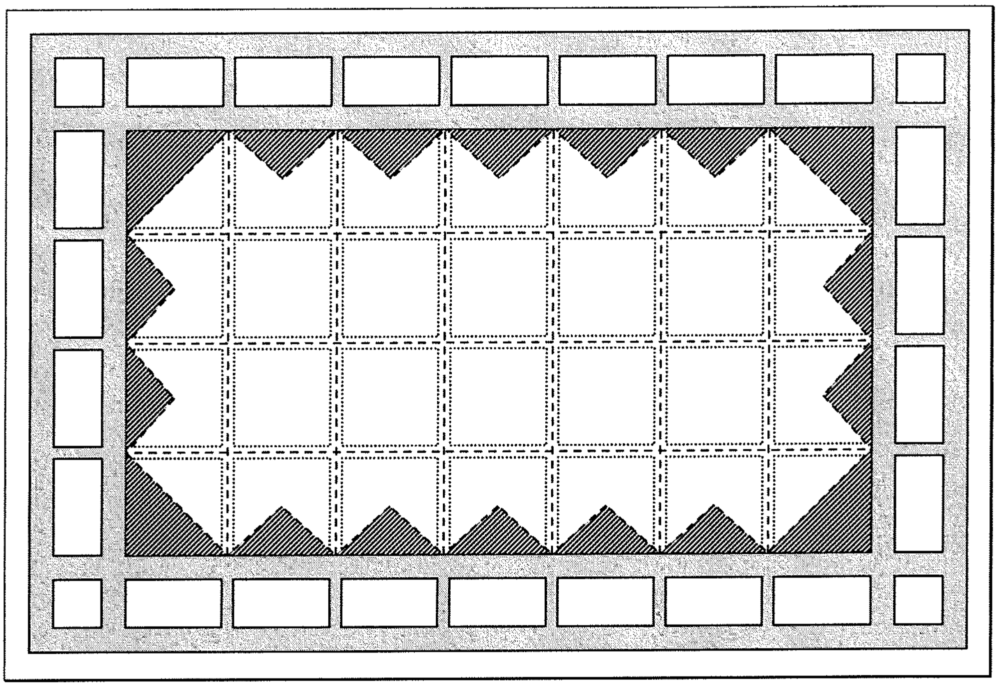


Figure 12-11 Top view of caisson floor structure

The most unfavourable loading combination (ULS) is the weight of the rib itself ($q_{selfweight}$), the weight of the ballast retained in the bottom compartments ($q_{ballast}$), hydrostatic buoyancy forces at maximum water level ($q_{hydrostatic}$), hydrodynamic uplift force due to the highest waves ($q_{hydrodynamic}$) and the soil reaction forces (q_{soil}). The schematisation of the floor rib and the loads are given in Figure 12-12. The cross-section of the rib has been schematised as an I-profiled girder.

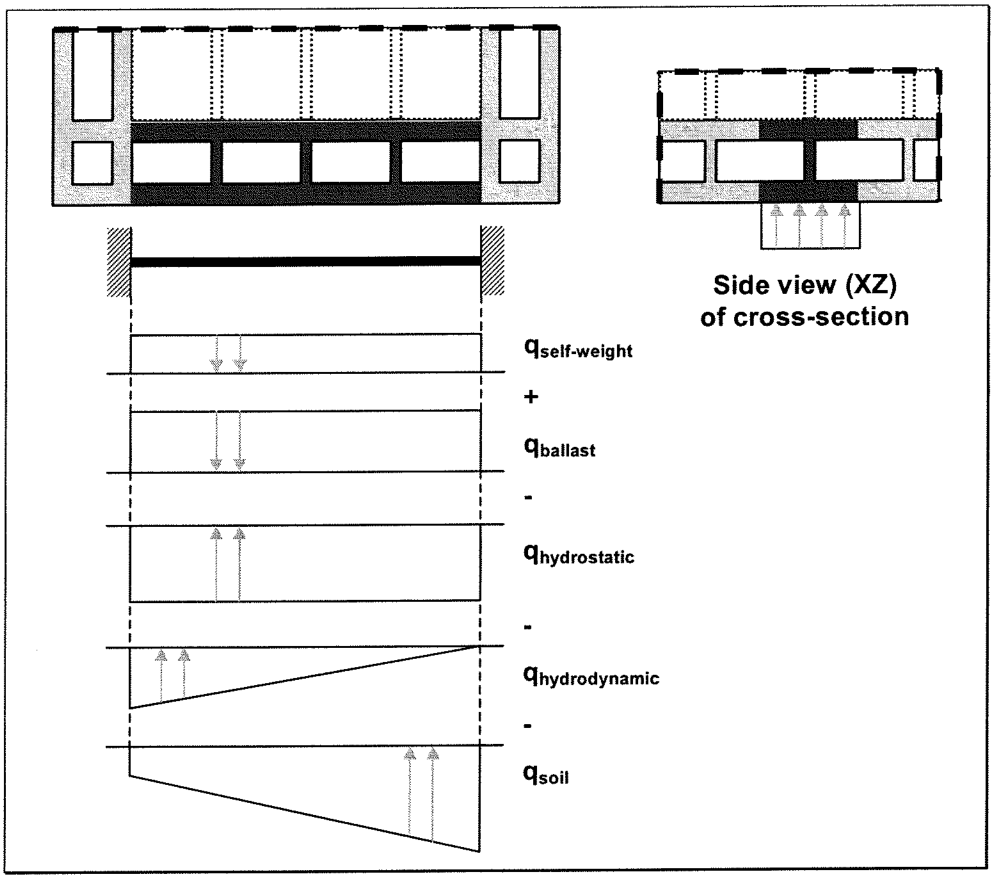


Figure 12-12 Schematisation of floor rib with loads

The floor as a whole can be identified as one plate, restrained at four sides. It can then be assumed that the loads will be redirected to the sides by applying an envelope pattern with angles of 45° (plastic analysis, see Figure 12-13).

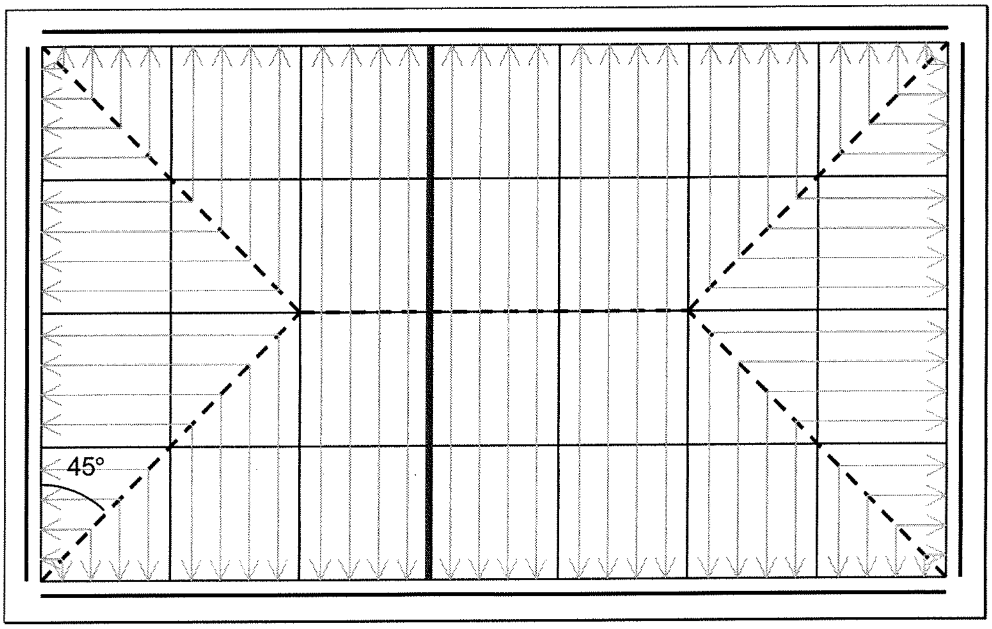


Figure 12-13 Redirection of loads on floor "plate"

Considering this distribution of loads, the red-coloured floor rib in Figure 12-13 has the least favourable position. Hence, the edge moment and local shear stress in that floor rib at the lee-side of the caisson (maximum soil pressure) will determine the required cross-sectional area of the rib.

- Roof construction

  Because the span of the roof is exceptionally large (approximately 55 – 65 m), it has been decided to use prefab concrete girders, a type which is commonly applied for bridges. For the calculation the "VIP"-profiled prefab girders provided by Spanbeton (Ref 63) have been used. A picture of their cross-sectional dimensions is shown in Figure 12-14.

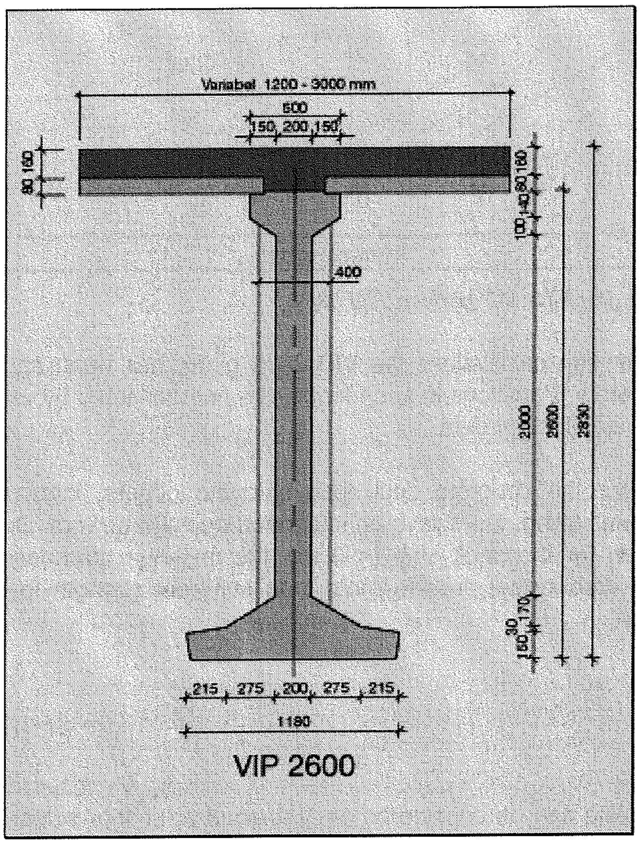


Figure 12-14 Cross-section of VIP 2600 profiled girder (Ref 63)

These girders have been designed for span lengths up to 60 meters. The number of girders that is required depends on the maximum superimposed deck load and their heart-to-heart distance. In the design graph below (Figure 12-15) this relation is shown: a VIP2600 girder with a span of 55 meters can take a maximum bending moment M = 10200 kNm = 1/8 * q * l^2, so q = 27 kN/m (excluding own weight and weight of deck slab). If the superimposed deck load is 10 kN/m^2, the heart-to-heart distances of the girders should be 27 / 10 = 2.7 meters.

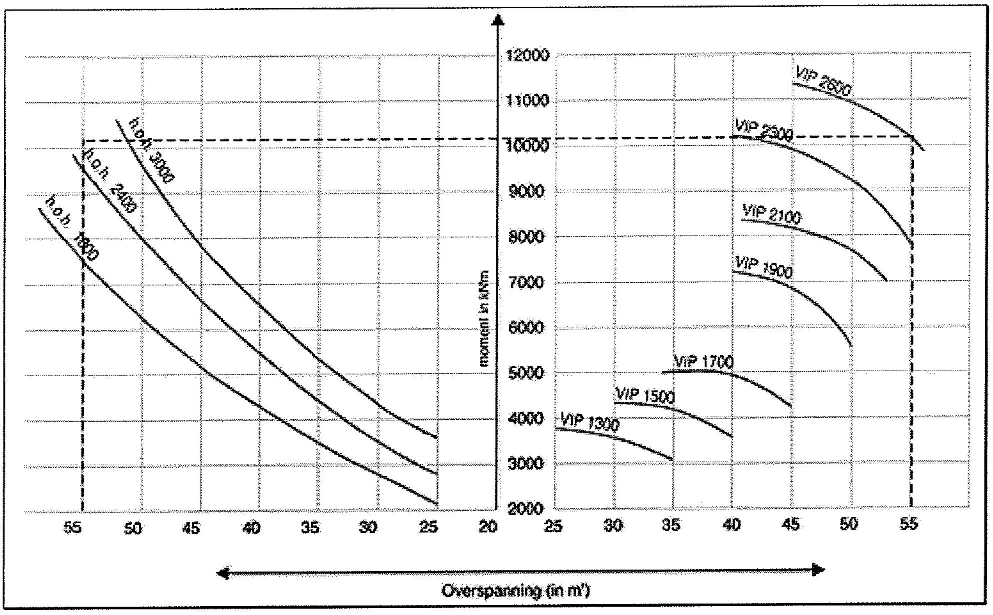


Figure 12-15 Design graph for VIP girders (Ref 63)

In the preliminary design calculations the VIP 2600 girder has been applied for span lengths up to 60 meters. Variations in span length are compensated by variations in the heart-to-heart distance of the girders.

It is advised to pour the concrete deck slab onto the girders, interconnected with crossbeams to prevent tilting, to achieve an integrated deck framework. Considering the relatively small span for the deck slab (< 3 m), the minimum thickness of 450 mm required for constructability will provide more than sufficient strength to deal with the superimposed loads.

12.7.3 Local strength

- Outer bottom plate

  The outer bottom plate can be considered as restrained at all four sides. The Ultimate Limit State condition consists of maximum water level, highest waves, maximum wind and current. To simplify the calculation, the loads, consisting of self-weight, ballast, hydrostatic, hydrodynamic and soil reaction loads, can be modelled as uniformly distributed. Figure 12-16 shows the schematisation of the plate.

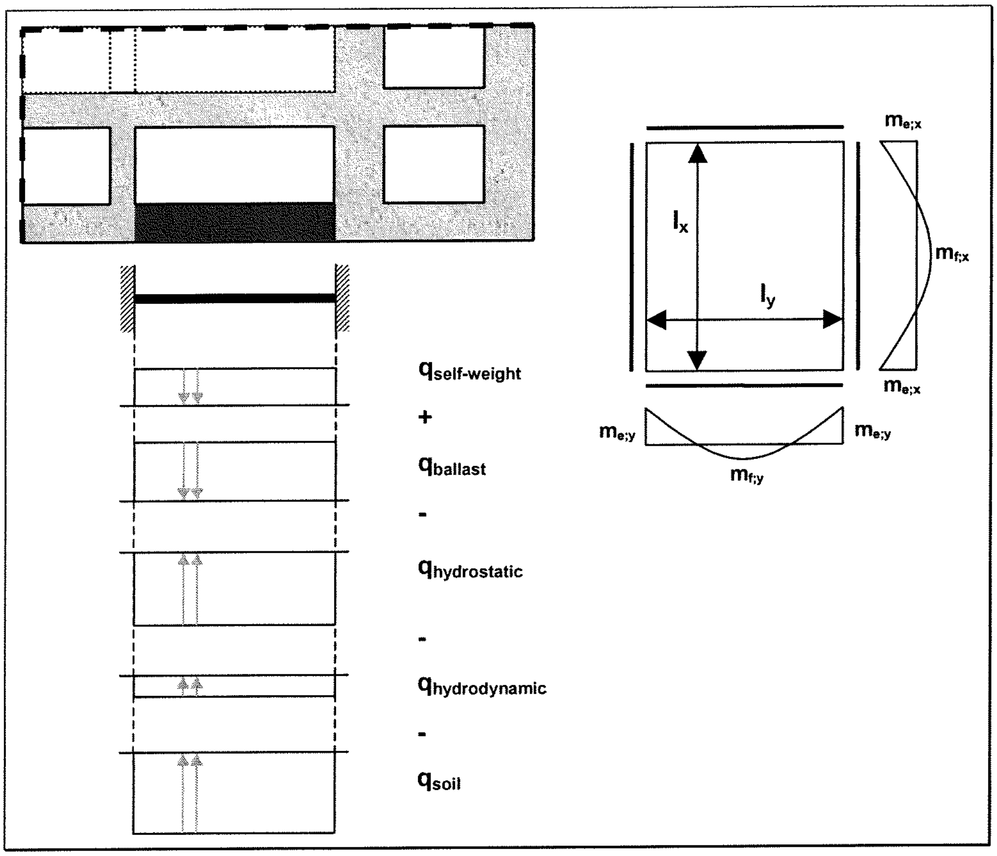


Figure 12-16 Schematisation of outer bottom plate

The NEN 6720 (Ref 48) provides a table for calculating the decisive bending moments in four-sided restrained supported plates with a uniformly distributed load. The table gives various factors for different length-width ratios.

The decisive shear forces have been calculated applying the plastic analysis (explained in the calculation of the floor rib).

- Inner bottom plate

  Except for the different loads on the inner bottom plate, its local strength calculation can be performed using the same method as for the outer bottom plate.

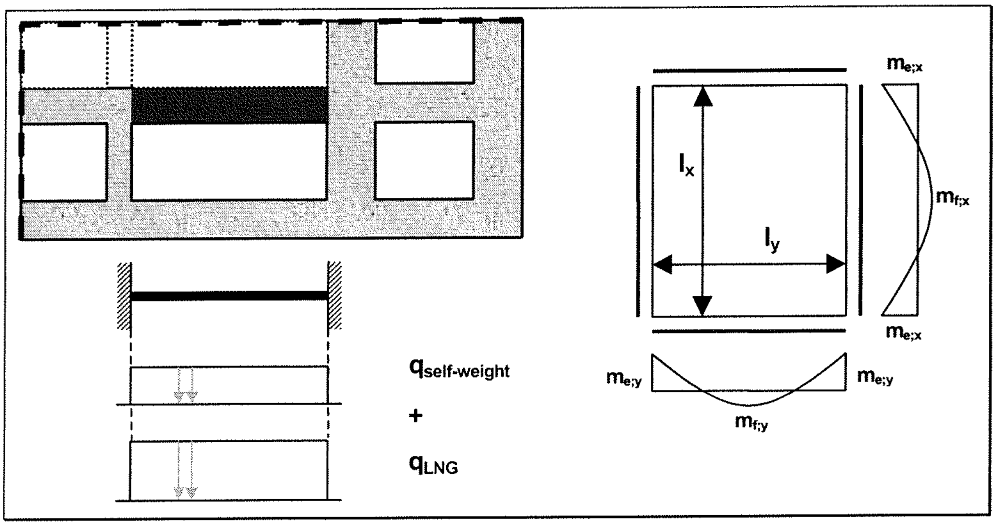


Figure 12-17 Schematisation of inner bottom plate

The least favourable loading case for the inner bottom plate is when the LNG tank is completely filled. Then the total load consists of the weight of the LNG (q_{LNG}) and the weight of the plate itself ($q_{self\text{-}weight}$).

- Outer walls

The dimensions of the outer wall plate are defined by the inner tank height and the size of the wall compartments. Therefore its height is much larger than its width. The most unfavourable (ULS) loading combination consists of the hydrostatic forces (q_{hs}) during the lowest possible water level, no wind, no waves, no current but the wall compartments filled with ballast material ($q_{ballast}$). The schematisation of this plate is shown in Figure 12-18.

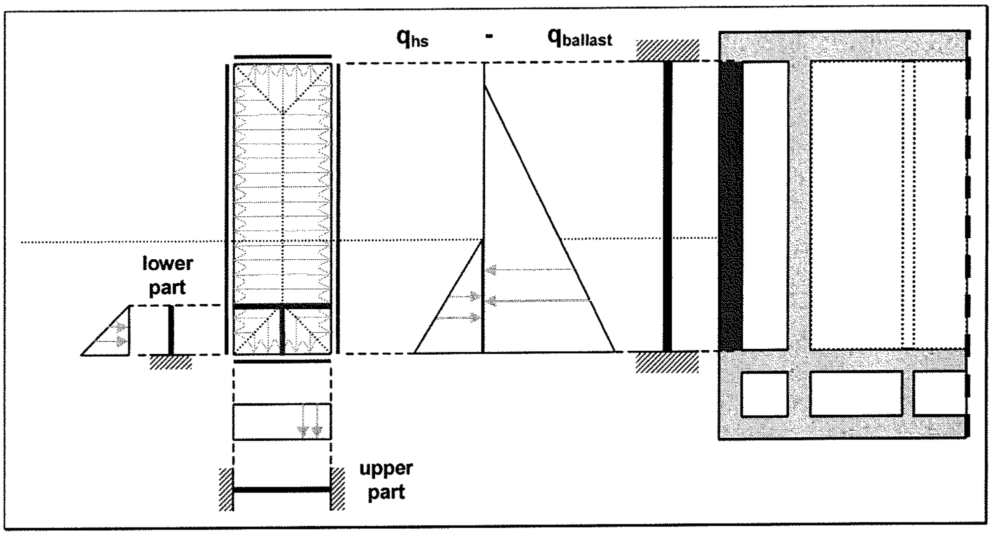


Figure 12-18 Schematisation of outer wall plate

Because the resulting loads on the plate are not uniformly distributed, the plate coefficients used for the bottom slabs cannot be applied. Instead, a plastic analysis with the aid of yield lines has been used to determine the maximum moment and shear stress in the plate. As indicated in Figure 12-18, this has been done for the lower part as well as the upper part of the plate.

- Inner walls

The inner wall plate has the same surface dimensions as the outer wall plate. However, the ULS loading case consists only of a filled wall ballast compartment ($q_{ballast}$) without any LNG in the storage tank. The schematisation of the plate is presented in Figure 12-19.

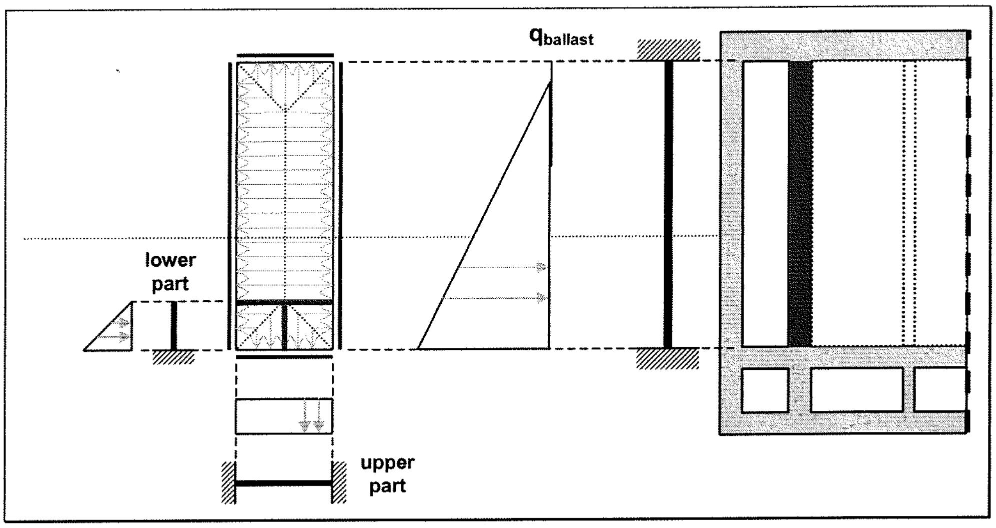


Figure 12-19 Schematisation of inner wall plate

The same plastic analysis, applied for the outer wall plate described above, has been used to determine the decisive moment and shear force in the plate.

12.7.4 Summary of load combinations

Table 12-4 provides a summary of the governing load combination (referring to section 12.4.7) used for the required strength of all calculated structural elements during Ultimate Limit State.

Element	Load combination
Wall beam	I
Floor rib	I
Roof construction	I
Outer bottom plate	I
Inner bottom plate	III
Outer wall plate	IV
Inner wall plate	II

Table 12-4 Governing load combinations for structural elements

12.8 Unit rates

The costs of the construction of the storage tank will mainly be determined by the material costs of the concrete (all-in) and the costs of the membrane containment system. To be able to compare the calculated costs with an alternate GBS design, it was decided to use the comparable unit rates for material costs. The assumed all-inclusive unit rates for concrete and membrane are given in Table 12-5.

Unit	Including	Price (USD)
1 m^3 of concrete (all-in onshore)	concrete, formwork, pre-stressing & embedments, reinforcement	1080
1 m^2 of membrane (all-in)	insulation, membrane panels, vapour barrier, roof blankets, bottom heating	960

Table 12-5 Unit rates for materials

These unit rates will be multiplied with the quantities of concrete and membrane required for the design of the storage tank. The resulting total construction costs only represent an indication comparable to alternative designs. Aspects like construction methodology (graving dock, towage, installation etc.) and construction schedule are not included. The estimated figures will be used to find the optimal solution with minimised (material) costs.

12.9 Calculation method

12.9.1 General

To come to an optimal design all calculations described above have been integrated in one large spreadsheet. The file consists of multiple sheets, all linked to one main sheet to control the optimisation process. An iterative procedure has been applied to find the optimal combination of variables while constantly safeguarding the design limitations. This procedure is described below.

12.9.2 Constants

A number of constants are required to perform the calculation. The values for these constants are calculated assumptions or best estimates. A description of the constants involved is given below. A detailed list of constants and their values is enclosed in Appendix K.

- General constants such as water levels, required freeboard and under-keel clearance, additional draft for skirts and the slope of the seabed
- Safety factors
- Wave, wind and current conditions at site location
- Material properties for concrete, amount of reinforcement and pre-stressing
- Characteristics of LNG and its containment system
- Properties of ballast material
- Superimposed loads
- Characteristics of sub-soil

12.9.3 Input variables

The following input variables can be inserted into the model:

- Inner tank height, width and length
- Thickness of inner walls, outer walls, inner and outer bottom slabs
- Number of compartments over length and width
- Height of bottom compartments
- Width of wall compartments
- Length of skirts

12.9.4 Boundary conditions

The limitations described in paragraph 12.3 together with some practical design limitations are inserted into the model as well:

- Required net storage volume
- Minimum caisson height (overtopping)

- Maximum spanned width of inner tank (prefab limits)
- Maximum caisson length (towage)
- Maximum draft of floating caisson (towage)
- Minimum concrete element thickness (constructability)
- Minimum compartment dimensions (constructability)
- Maximum length of skirts (constructability)

12.9.5 Computation

For the given constants and input variables, the spreadsheet model computes the roof construction height, the outer caisson dimensions, the required material quantities and an estimate of the total costs. At the same time the model checks the local and global strength of the structural elements, as well as the floating and on-bottom stability, the draft of the caisson and the net tank volume. The procedure has been illustrated in Figure 12-20.

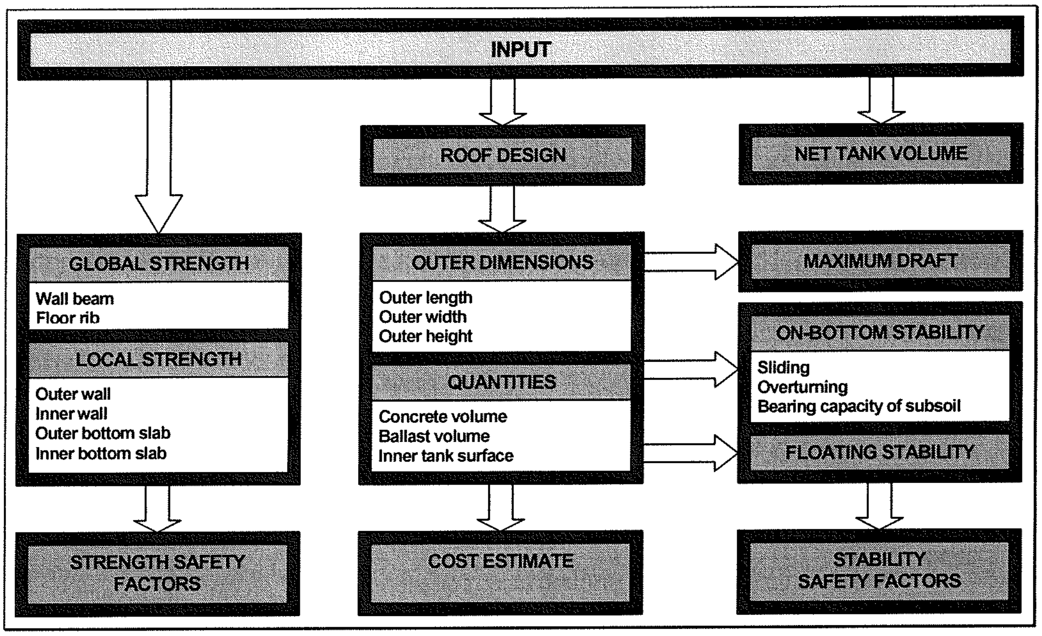


Figure 12-20 Flow scheme of spreadsheet calculation

The listing of the spreadsheet model has been enclosed in Appendix K.

Processing the constants and the initial values for the variables, the model can now compute the optimal solution by iteratively searching for the combination of input variables, within the boundaries, resulting in the lowest total construction costs.

This iterative process will be carried out in the next chapter.

13 Storage tank design optimisation

13.1 General

To come to the optimal design of the storage tank, the spreadsheet model described in the previous chapter will be used. Because of the complexity of the model, a sensitivity analysis has been carried out for the main input parameters, which will be described in the paragraphs 13.2 to 13.7. After the sensitivity analysis the optimal design solution for the storage tank will be presented in section 13.8. Finally the conclusions are given in 13.9.

13.2 Base case

A first calculation has provided the solution given in Table 13-1 for the design of the storage tank, which fulfils all design criteria.

Parameter	Base case value
Inner length x width x height [m]	157.6 x 56.2 x 25.8
Outer wall thickness [mm]	620
Inner wall thickness [mm]	690
Outer bottom slab thickness [mm]	630
Inner bottom slab thickness [mm]	450
Compartment wall thickness [mm]	450
No. of compartments over length / width	28 / 10
Width of wall compartments [m]	5.5
Bottom compartments length x width x height [m]	5.0 x 5.2 x 7.5
Length of skirts [m]	4.5
Outer length x width x height [m]	171.2 x 69.8 x 37.7
Net tank volume [m^3]	200,000
Draft of floating caisson [m]	14.5
Concrete volume [m^3]	62,565
Ballast volume (sand) [m^3]	139,830
Estimated total costs [index]	100

Table 13-1 Base case solution for storage tank design

This solution will be used as the base case for the sensitivity analysis in the following paragraphs. During this analysis the total costs will be expressed as an index number based on the costs for this base case.

13.3 Sensitivity of primary dimensions

13.3.1 Height / width ratio

When the height is increased, while the width is adjusted according to the volume requirements, the model shows a reduction of total construction costs. However, the draft of the structure increases because its mass per square meter bottom area has increased. This also causes an increased load on the bottom slab. Moreover the height of the walls increases. Hence both the wall plates and the wall beam as a whole show a lack of strength.

When the structure is lowered, while increasing the width accordingly, the costs increase and the draft is reduced. On the other hand, due to the reduced weight per area, the resistance against sliding decreases and the span length of the floor rib requires larger dimensions.

The sensitivity has been visualised in Figure 13-1 below. Safety factors are defined as the actual value divided by the critical value. A safety factor of 0.95 therefore means that the actual value is 5% lower than required. The graph only shows those safety factors that have critical values (below 1.0) within the range of variation. From the graph can be concluded that lowering the height is limited by the decreasing sliding resistance and raising the height is restricted by the lack of strength of the wall beam.

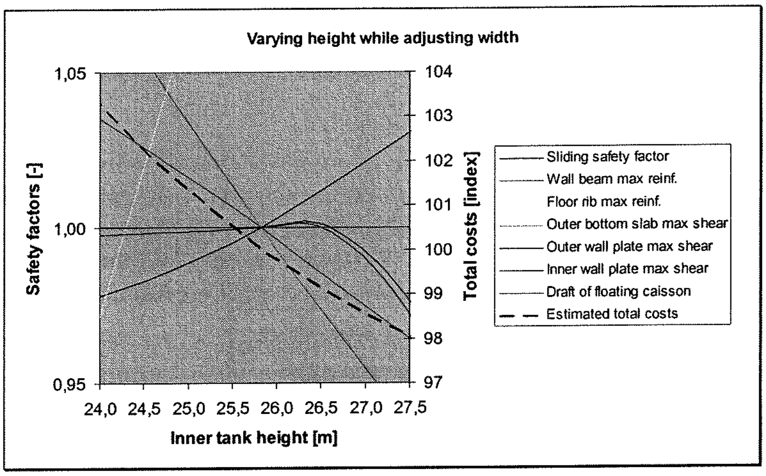


Figure 13-1 Sensitivity analysis for height – width ratio

However it seems possible to overcome these problems by adjusting wall and slab thickness and changing the dimensions of the ballast compartments. From Figure 13-2 can be seen that now costs decrease with increasing inner tank height.

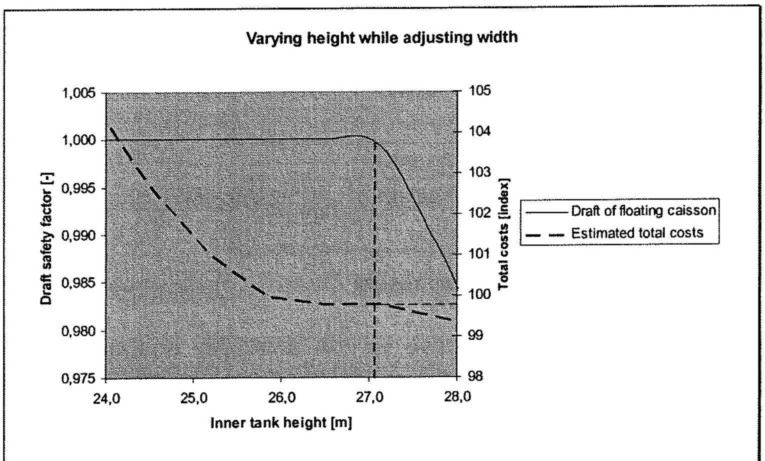


Figure 13-2 Sensitivity analysis for height - width ratio (adjusted)

The draft of the caisson remains sufficiently small until height is increased over 27.1 meters, after which the model is unable to calculate a satisfying combination of dimensions. Hence, according to this analysis the optimal height – width ratio is 27.1 m / 53.3 m.

13.3.2 Height / length ratio

With this new optimal combination of inner dimensions a sensitivity analysis has been carried out on the height – length ratio. When looking at Figure 13-3 it becomes clear the total costs decrease with increasing height. Furthermore it shows that the lower limit of this ratio is given by the decreasing resistance against sliding. The upper limit is set by the lack of strength of the floor rib.

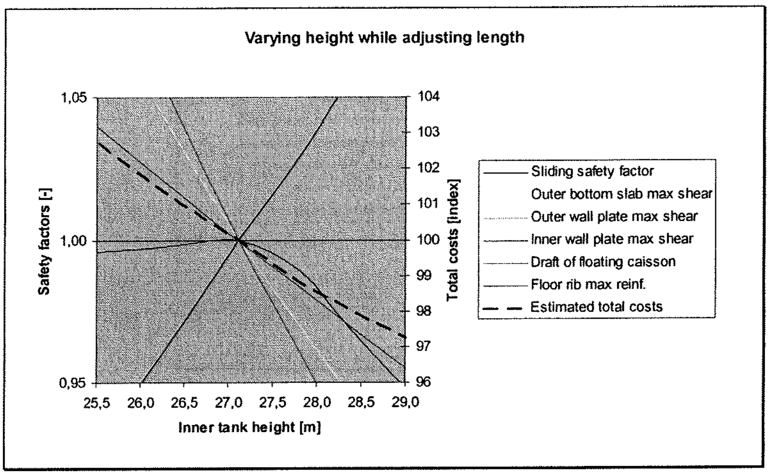


Figure 13-3 Sensitivity analysis for height - length ratio

When again the model is allowed to adjust the element thickness and compartment size to keep all safety factors greater than 1, the optimal solution lies again at H = 27.1 m. Costs are still decreasing when the height is increased even more, but the draft of the caisson becomes the limiting factor. This has been visualised in Figure 13-4.

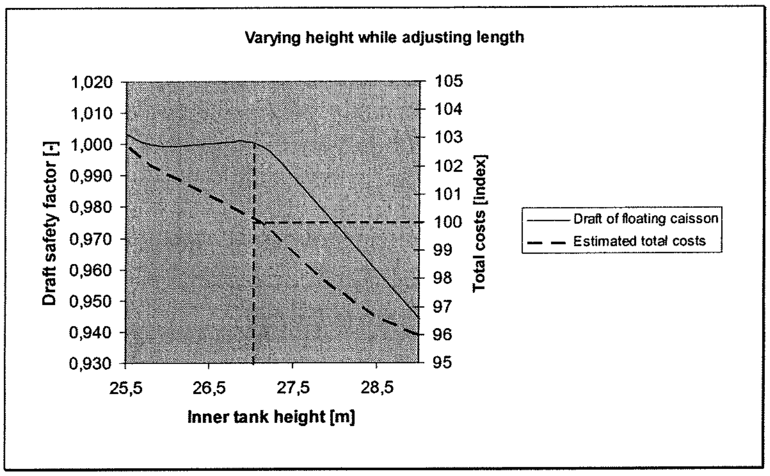


Figure 13-4 Sensitivity analysis for height - length ratio (adjusted)

The conclusion of this analysis is that the optimal height – length ratio of the caisson still is 27.1 m / 157.6 m.

13.3.3 Width / length ratio

The result of the variation of the width – length ratio is shown in Figure 13-5. Decreasing the width while adjusting the length has a positive effect on the total costs, but causes a lack of strength in the outer wall plates. On the other side, increasing the width has a negative impact on the strength of the floor rib.

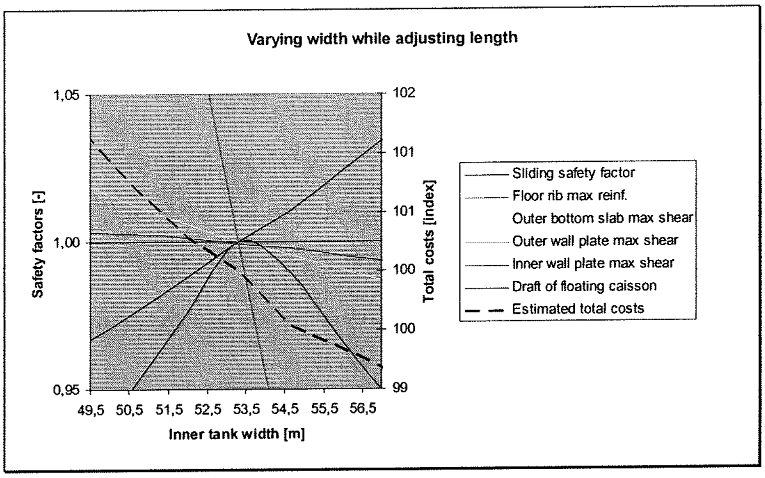


Figure 13-5 Sensitivity analysis for width - length ratio

When trying to overcome these limitations by altering the dimensions of the elements and the compartments, again the draft is the limiting factor, as can been seen in Figure 13-6.

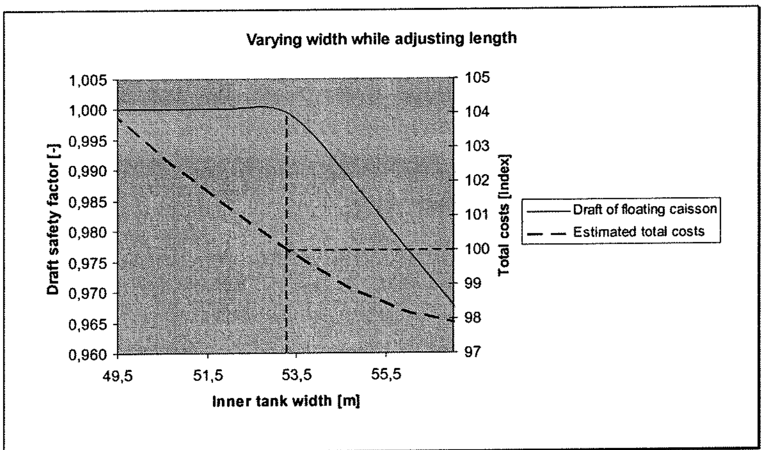


Figure 13-6 Sensitivity analysis for width - length ratio (adjusted)

According to this analysis the optimal width – length ratio is 53.3 m / 157.6 m.

13.4 Sensitivity of skirt length

The length of the concrete skirts attached to the bottom slab has a large impact on the on-bottom stability of the caisson. Decreasing the length of the skirts has to be compensated by increasing the on-bottom weight of the caisson to maintain sufficient resistance against sliding.

The spreadsheet model has been used to calculate the optimal caisson dimensions for different skirt lengths. In Figure 13-7 the resulting relation between the length of the skirts and the total costs of the caisson has been visualised.

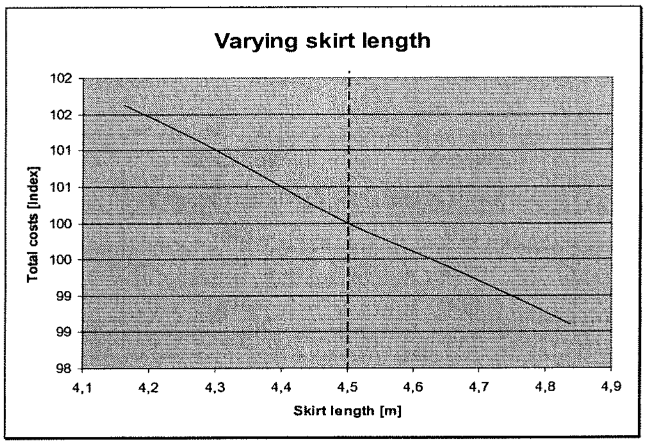


Figure 13-7 Sensitivity analysis of skirt length

It seems that increasing the length of the skirts has a continuing positive effect on the total costs. However there are some limitations to the skirt length that are not incorporated in the model. An important issue for instance is the penetration of the skirts during installation of the caisson. In this analysis is assumed that skirts longer than 4.5 m might be feasible but do not result in lower costs because of construction and installation issues. Skirt length is therefore limited to 4.5 meters. During detailed design this assumption must be verified.

13.5 Sensitivity of compartment dimensions

The length and width of the bottom compartments are determined by the number of compartments over length and width. Because of the bending moments and shear forces in the plates, a square shaped compartment is preferred. Several compartment dimensions have been tested to determine to optimal size. Reducing the number of compartments (increasing their dimensions) has to be compensated by increasing the thickness of the bottom plates. From Figure 13-8 becomes clear that the optimal dimensions for the compartments are 5.3 x 5.4 meters.

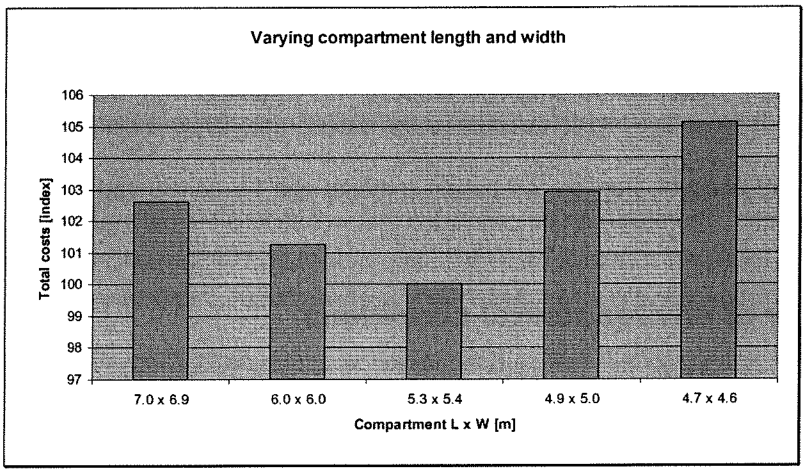


Figure 13-8 Sensitivity analysis of compartment dimensions

13.6 Sensitivity of ballast material

Until now the model has been calculated using sand as ballast material. In this paragraph will be checked whether lighter (water) or heavier (iron ore) ballast material might be a more cost-effective solution.

The spreadsheet model has been used to calculate the optimal solution using either water, sand or iron ore as ballast material. The main differences have been summarised in Table 13-2 below.

Ballast material	Water (ρ = 1030 kg/m^3)	Wet sand (ρ = 2000 kg/m^3)	Wet iron ore (ρ = 3000 kg/m^3)
Outer wall thickness [mm]	520	640	730
Inner wall thickness [mm]	530	710	810
Outer bottom slab thickness [mm]	1290	610	560
Width of wall compartments [m]	14.6	5.5	4.6
Height of bottom compartments [m]	10.7	7.2	6.5
Length of bottom compartments [m]	6.6	5.3	4.7
Width of bottom compartments [m]	5.0	5.4	5.0
Length of skirts [m]	4.5	4.5	0.0
Outer length [m]	213.4	162.3	161.1
Outer width [m]	91.3	71.5	71.7
Outer height [m]	37.2	38.1	36.7
Draft of floating caisson [m]	14.5	14.5	14.4
Concrete volume [m^3]	104,540	60,447	60,069
Ballast volume [m^3]	363,246	134,099	113,860
Estimated total costs [index]	156	100	99

Table 13-2 Sensitivity analysis of ballast material

It can be seen that using water as ballast has huge impact on the dimensions of the caisson. To gain sufficient on-bottom stability, the required ballast volume is almost tripled compared to using sand. Moreover the thickness of the outer bottom slab increases significantly due to the fact that the high buoyancy forces are not compensated by the weight of the ballast material. The resulting solution has very high costs.

When using iron ore as ballast material, the sliding resistance and draft are no longer limiting factors. However, the thickness of the walls (design load based on filled ballast compartment) becomes an issue. The optimum solution becomes slightly cheaper than using sand. On the other hand, the costs of the ballast material itself are not included in these figures. The costs of iron ore are estimated at approximately twice the costs of sand (per cubic meter). Therefore the small cost reduction (99 instead of 100) will turn out to be an increase of costs when the additional costs for the ballast material are included.

Taking this into account, iron ore as ballast material can be considered as unfavourable compared to sand. Therefore can be concluded that wet sand is the optimal ballast material.

13.7 Sensitivity of environmental conditions

13.7.1 Water depth

The minimal water depth required at the site location is calculated at 15 meters, determined by the draft of the LNG carrier (see paragraph 7.3). A maximum water depth has not been established. The advantage of a larger water depth than the minimum of 15 meter is that in that case a higher, heavier caisson with a larger draft can be constructed. This will reduce the ballast requirements. However, while increasing the water depth, some other implications arise, which are discussed below.

- Because the height of the nearshore waves is likely to be limited by the water depth, it is expected that the significant wave height will increase while increasing the water depth.

This means increased wave loads, which will result in higher ballast requirements regarding the stability of the caisson.

- The hydrostatic pressures on the structure increase with the water depth, resulting in higher requirements for the wall thickness of the concrete caisson.
- Most likely a location with deeper water will be situated at a larger distance from the coast. The length of the connecting pipeline to the shore will increase with the depth, depending on the slope of the seabed.

Taking these considerations into account, a larger water depth at the site location will only be attractive when the advantages of a heavier caisson exceed the disadvantages mentioned above.

The effect of increasing the water depth has been assessed using the spreadsheet model. For varying water depth the optimal solution has been determined. The results are shown in Table 13-3 below. Note that the significant wave height has been adjusted to the water depth.

Local water depth [m]	**15**	**16**	**17**	**18**
Significant wave height [m]	7.9	8.5	8.9	9.3
Draft of floating caisson [m]	14.5	15.5	16.5	17.5
Outer length [m]	162.3	160.6	157.8	155.1
Outer width [m]	71.5	71.1	70.7	70.9
Outer height [m]	38.1	39.9	41.6	43.2
Outer bottom slab thickness [mm]	610	800	960	1050
Width of wall compartments [m]	5.5	6.5	7.3	7.7
Concrete volume [m^3]	60,447	63,891	66,713	69,823
Ballast volume [m^3]	134,099	147,110	155,463	164,214
Estimated total costs [index]	**100**	**103**	**106**	**109**

Table 13-3 Sensitivity analysis for water depth at site location

From the table it can be seen that with increasing water depth the increased draft allows for a higher caisson. However due to the increased wave and buoyancy loads the thickness of the bottom slab and the width of the wall compartments increases significantly. Hence the concrete volume and the total costs of the optimal caisson increase with increasing water depth. Therefore the optimal water depth is equal to the minimum water depth, which is 15 meters.

13.7.2 Wave height

The sensitivity of the design significant wave height at the site location has been assessed. A caisson located at a site with reduced wave height requires less ballast weight because of the reduced horizontal loads. The effect on the costs has been visualised in Figure 13-9 below.

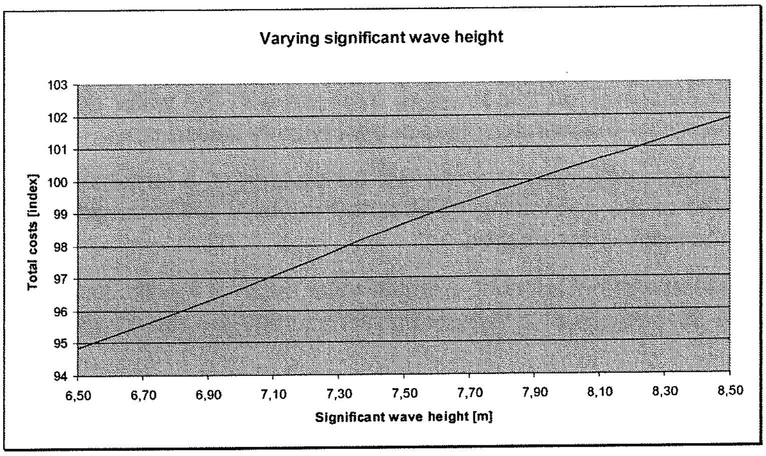


Figure 13-9 Sensitivity analysis for significant wave height

13.7.3 Soil conditions

It has been assumed that the angle of internal friction of the subsoil at the site location amounts to 30 degrees. This value has a huge impact on the on-bottom stability of the caisson. The effect of changing this parameter, by either selecting a different site or improving the subsoil before installation, has been assessed.

The optimal caisson dimensions have been calculated for an angle of internal friction varying between 30 and 40 degrees. Because of the increased sliding resistance, the requirements for ballast weight decrease with increasing ϕ, resulting in lower costs. This has been visualised in Figure 13-10 below.

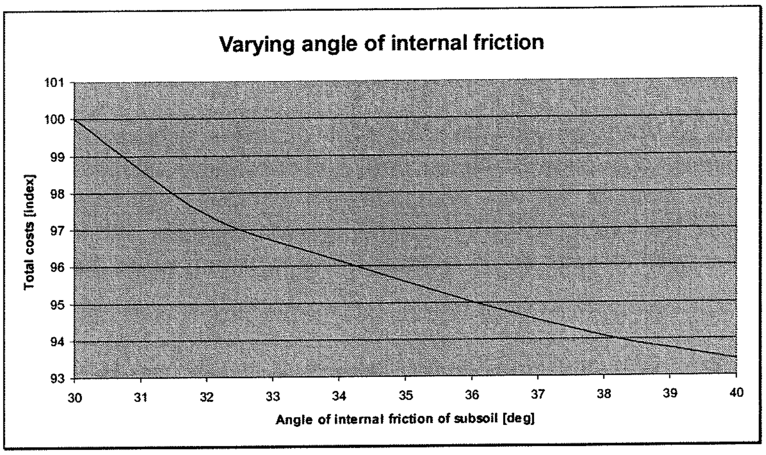


Figure 13-10 Sensitivity analysis for angle of internal friction of subsoil

Obviously, improving the subsoil has a positive effect on the total construction costs. However this figure does not incorporate the costs for improving the soil. It should be investigated whether the reduction in construction costs exceeds the additional costs for soil improvement. This is considered to be outside the scope of this study.

13.8 Optimal solution

As a result from the sensitivity analysis, the optimal solution for a caisson installed at the proposed site location is given in Table 13-4 below.

Parameter	Optimal value
Inner length x width x height [m]	148.6 x 57.7 x 26.6
Outer wall thickness [mm]	640
Inner wall thickness [mm]	710
Outer bottom slab thickness [mm]	610
Inner bottom slab thickness [mm]	450
Compartment wall thickness [mm]	450
No. of compartments over length / width	26 / 10
Width of wall compartments [m]	5.5
Bottom compartments length x width x height [m]	5.3 x 5.4 x 7.2
Length of skirts [m]	4.5
Outer length x width x height [m]	162.3 x 71.5 x 38.1
Net tank volume [m^3]	200,000
Draft of floating caisson [m]	14.5
Concrete volume [m^3]	60,447
Ballast volume (sand) [m^3]	134,099

Table 13-4 Optimal caisson parameters

Based on the unit rates for the required material quantities (membrane and concrete) specified in section 12.8, the costs for this solution are approximately 84 million US Dollars. When considering the storage volume of 200,000 m^3, this equals 420 US Dollars per cubic meter storage.

13.9 Conclusions

Regarding the sensitivity analysis that has been carried out in the previous paragraphs, the following additional conclusions can be drawn:

- When looking the primary dimensions, the first dimension to minimise is length, then width and finally height. In other words, increasing height is the most cost-effective way to gain additional storage volume.
- The resistance against sliding and the draft of the floating caisson are the decisive limitations during optimisation.
- According to the Spreadsheet model increasing the skirt length is cost saving. However because installation issues (penetration) have not been incorporated in the model the maximum length of the skirts has deliberately been limited to 4.5 meters.
- The number of compartments over length and width depends on the total length and width of the structure, but should be chosen such that the compartments are approximately square.
- Water as ballast material results in a huge required ballast volume. Iron ore causes extreme loads on the compartment walls. Wet sand as ballast material is to be preferred.
- Increasing the water depth at the site location allows a caisson with a larger draft. However such as caisson is more expensive due to increased loads of waves and buoyancy. The minimum water depth is therefore the optimal depth.
- Increasing the angle of internal friction of the subsoil, and decreasing the significant wave height at the site location results in reduced construction costs of the optimal solution.
- Because, in this case, a caisson with a larger draft does not have advantages, there is no reason for a multiple phased construction procedure, as has been mentioned in Chapter 9. The caisson should be completely constructed in a dock and then towed to the site location. This procedure will be discussed in the next chapter.

14 Construction and installation of caisson

14.1 General

In this chapter the construction and installation for the concrete storage tank will be discussed in a generic manner. Information from preceding design studies for gravity based structures [Ref 16, Ref 21, Ref 31 and Ref 49] have been consulted for common offshore construction and installation practice.

14.2 Construction

14.2.1 Graving dock

The purpose-built graving dock will be constructed by excavating a large basin surrounded by dikes with a bottom level that is equal to or lower than the bottom level outside. The exit can be sealed by either gates in the form of caissons, or by a temporary dike that can be excavated when the GBS is ready to float.

The dock should be safely dewatered against the maximum high tide plus storm surge and under the maximum rise in water table and rain runoff. Side slopes must be protected against slope failure and excessive erosion under heavy rain, and will have a typical angle of 1:3. At least two surfaced roads should lead into the dock to provide sufficient access to the works. When flooded the dock should provide sufficient draft for the GBS to float. Length and width of the dock will be determined by the dimensions of caisson, increased with some additional space allowing for placing cranes and transporting construction materials (typically 20 – 30 meters extra at all sides).

14.2.2 Construction of concrete caisson

The first items installed are the skirts. The precast concrete skirts will be set down into slots, excavated in the bottom of the dock. Then the base slab can be directly supported on the dock's foundation slab.

The bottom compartment walls are now constructed forming cellular partitions, after which the inner bottom slab is made to cover the bottom compartments. With the bottom structure approaching completion, the mechanical systems are installed, consisting of salt-water ballast piping, an under-base grouting system, skirt drainage and venting system and instrumentation such as bottom clearance sensors.

After the base structure has been finished, the double walls and compartments can be constructed using slip forms. Finally the prefab roof beams can be installed, which will be covered by a concrete deck slab.

14.2.3 Installation of containment system and topsides

As soon as the inner walls and roof are finished the integration of the LNG containment system can be started. Before starting this activity an air test is performed on the concrete tank in order to detect cracks and repair if needed.

After fastening the sandwich insulation panels to the walls, the stainless steel membrane sheets are welded directly onto the insulation. During the construction different sets of tests have to be performed in order to insure the integrity of the membrane containment system.

The most cost-effective solution to install the topsides is in the graving dock, before tow-out. Topside modules will be lifted using a lifting platform alongside the dock, while horizontal transport will take place using trailers. Another advantage of installing the topsides in the dry is that it can be scheduled while construction of the containment system inside is still in progress.

14.3 Installation

14.3.1 Dock tow-out

When the weather window is favourable, the tow-out from the dock can be initiated. First the ballast compartments are partially filled with water to prevent the caisson from floating during the flooding of the dock. Then the dock is flooded by opening the gates or by partially excavating the surrounding dike. After the water ballast is pumped out the trim water ballast system and the air cushion system are implemented.

The injection of compressed air under the bottom slab and between the skirts results in a significant decrease of the draft and partly compensates for the penalty caused by the extra draft created by the skirts. A 50 cm water seal is kept at the tip of the skirts in order to ensure the air-tightness of the air cushion and to avoid any risk of air escaping during the dynamic movements of the GBS. This technique is commonly used for the construction of gravity-based structures when the draft does not meet the site conditions, or when the freeboard is not large enough.

By adjusting these systems the required draft of the caisson can be achieved. Finally the GBS is hauled from the dock by hauling winches and harbour tugs.

14.3.2 Tow from dock to open sea

When the caisson reaches the open sea, the harbour tugs will be replaced by ocean-going tugs. From the moment that the local water depth allows for it, the air cushion will be deflated prior to the sea-tow, because it has a negative impact on marine stability, such as free surface effects and air compressibility effects. Skirt compartments should be arranged in such a way that the marine stability is not too much influenced by these negative effects. The air pressure in the skirt compartments is controlled by means of volume control using water level indicator gauges.

14.3.3 Tow to site location

The required number of tugs for the sea-tow depends on the maximum bollard pull of the tugs, the required towing speed and the expected storm conditions with a return period of 10 years. Typically there will be four tugs with a bollard pull of 150 tons, one escorting tug, travelling with an average towing speed of 2.5 – 3.0 knots. The duration of the sea-tow depends on the distance between dock and site location and the towing speed; it could easily become several weeks.

14.3.4 Arrival at site

When approaching the site, but still at sufficient water depth, the caisson is slowed down and stopped, after which the tugs, in a star formation, keep the GBS into position. After ensuring that the weather conditions will be favourable, the air cushion system is used to achieve the required under-keel clearance. By controlling their pre-installed mooring lines, the tugs in star formation manoeuvre the caisson into its final position.

14.3.5 Installation on site

The structure is carefully water-ballasted downwards until 0.5 meter above the seabed to allow for final repositioning by the tugs. Then the water ballasting is resumed to achieve the initial penetration of the skirts into the soil. Thinner at the bottom than at the top, they will penetrate the ground until the bottom slab ribs arranged below the walls are in contact with the ground. The air cushion is deflated and ballasting continues until the desired penetration is reached. Unequal ballasting can be applied to overcome differential penetration of the skirts due to heterogeneity of the seabed.

14.3.6 Complementary work

In order to secure the GBS onto the seabed, the void space under the base is filled by means of a grout injection. The caisson is filled with additional solid ballast by hydraulic means to reach sufficient on-bottom stability. Finally, temporary equipment and installations can be demobilised.

15 Design of mooring system

15.1 General

In section 10.5 the "Single Point Mooring with limited weathervaning" concept has been selected to work out further. In this chapter the design of this mooring system will be analysed more thoroughly. In paragraph 15.2 a description is given of the individual components of the system. Afterwards the system limitations, which create the boundary conditions for the following chapters, are described in 15.3.

15.2 Component description

15.2.1 General

An overview of the mooring configuration has been given in Figure 15-1 below.

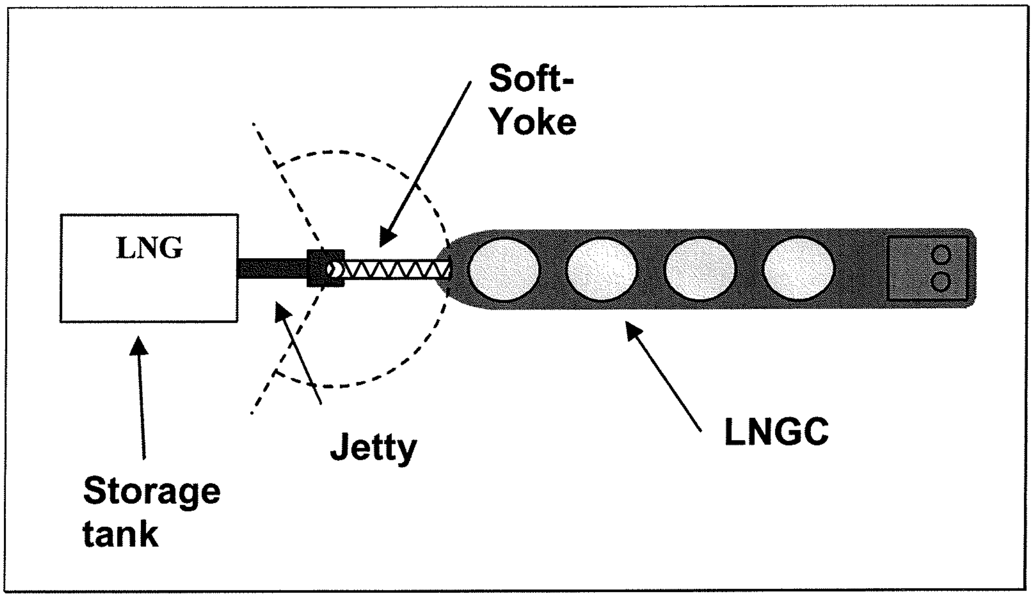


Figure 15-1 General overview of mooring configuration

15.2.2 Soft-Yoke

The Soft-yoke system consists of the following components:

- A steel jacket structure, founded on the seabed, erected to approximately 15 meters above Chart Datum.
- A 40 m high crane fitted on top of the jacket, which has a reach of 40 – 50 meters. Between the jacket and the crane a large rotating swivel is installed to allow the crane to follow the vessel motions.
- Two mooring legs, vertical members hanging down from the crane. They are connected to the crane with universal joints.
- The yoke, triangular shaped structure with a large counterweight. At the tip of the yoke the bow manifold connector is constructed.

A schematic drawing of the soft-yoke system is provided in Figure 15-2.

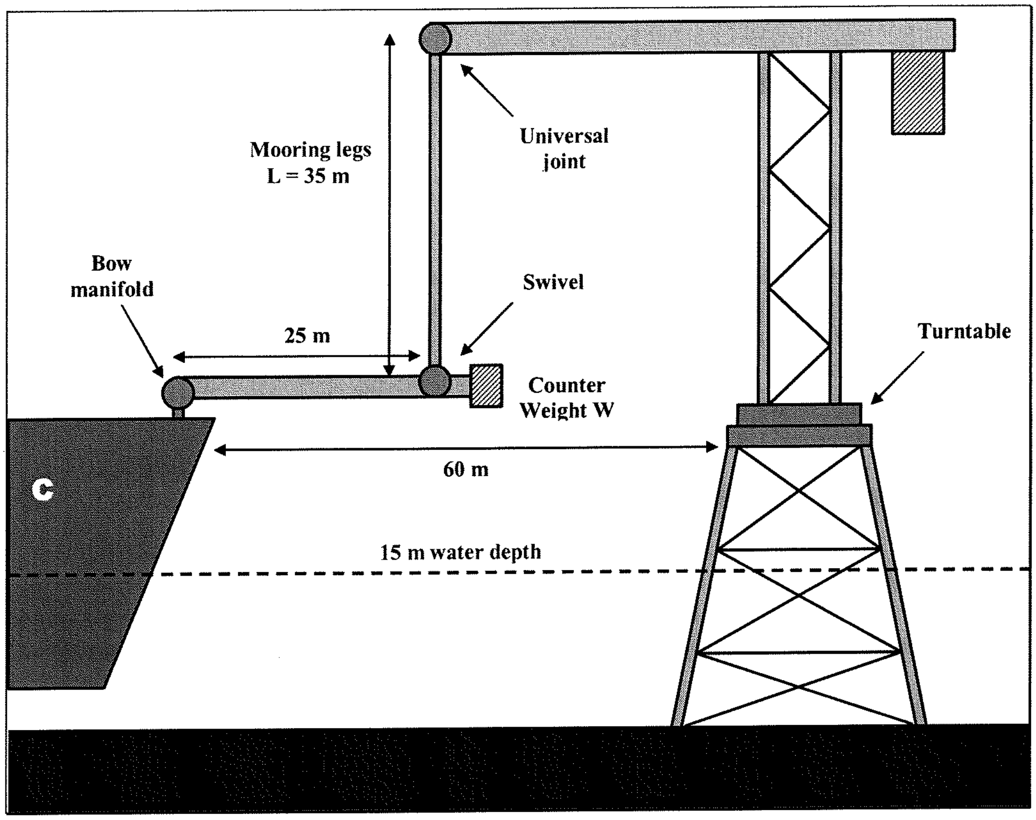


Figure 15-2 Schematic impression of the Soft-Yoke system

The soft-yoke system is designed to maintain a nominal distance between the fixed jacket and the bow of the LNG carrier during offloading. This nominal distance corresponds to the calm water situation (i.e. in the absence of external environmental loads). If external influences, such as wind, waves and current, cause the vessel to surge towards or away from the fixed tower, the yoke reacts with a horizontal force to counteract this motion. In contrast with a hawser mooring, this system also develops a push force when the nominal distance is decreased, thus avoiding contact.

As can be derived from ordinary trigonometry, the restoring force generated by the counterweight becomes greater when the displacement from the nominal position increases. This has been illustrated in Figure 15-3 below.

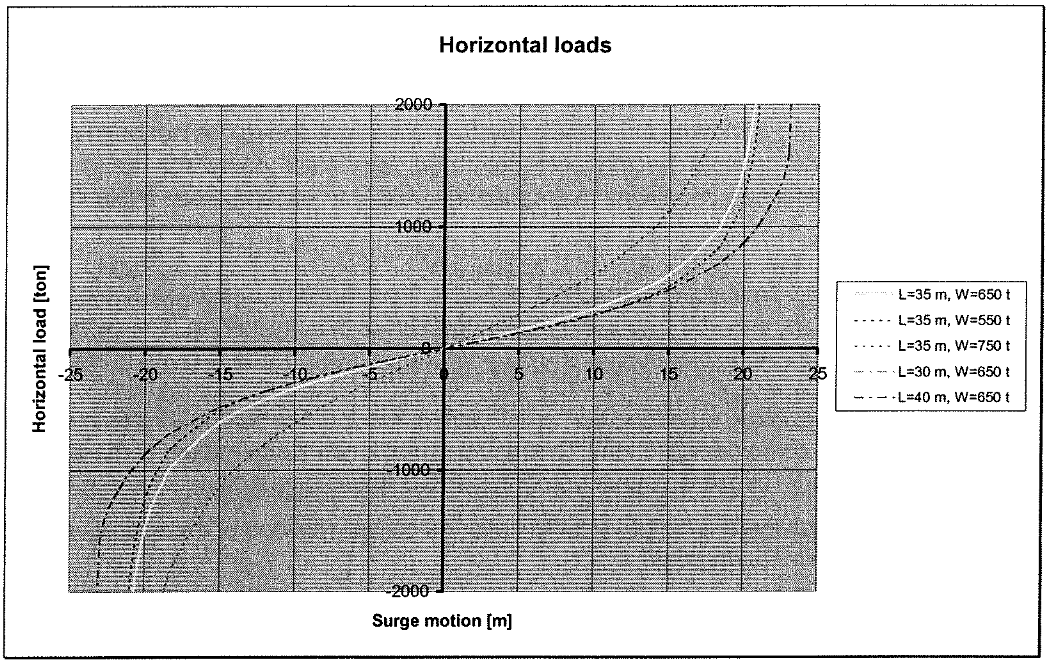


Figure 15-3 Soft-yoke load – excursion curve

Figure 15-3 shows load – excursion curves for yoke configurations with different leg-lengths (L) and counterweights (W). It can be seen that a heavier counterweight results in a more rigid system (steeper curve), while extending the length of the legs creates a system which accepts a larger envelope of surge motions before mooring loads start to increase rapidly.

There is a physical limit to how far apart the vessel can surge away from the tower, determined by the geometry of the system. With a mooring leg length of 35 m, this geometrical limit is set at 81 m (21 m from equilibrium). At this limit, the mooring load is extremely high. Since a high mooring load is to be avoided, it is advantageous to maintain a safety margin between the most probable maximum surge and this physical limit. A Soft-Yoke system with mooring legs of 35 m and a counterweight of 650 tons (yellow curve) will be used as a base case.

The LNG product shall be transferred from the LNG carrier to the jacket via rigid piping. Articulations in the transfer piping will comprise of fluid transfer swivel joints. The axis of rotation of each swivel joint in the transfer piping is in line with the rotation axes of the yoke articulations.

15.2.3 Jetty

A jetty structure between the yoke-jacket and the concrete LNG storage and regasification facility will be constructed to provide sufficient distance between the moored vessel and the terminal. In fact, it will only consist of a simple trestle which bears the LNG piping from the vessel to the terminal as well as a gangway for maintenance access.

The jetty will be fitted to the middle of the shortest side of the caisson to allow maximum ship weathervaning. Another possibility could be to adjust the deck layout proposed in chapter 11 and build the jetty at one corner of the caisson under 45 degrees (in the horizontal plane). This will allow for an even larger angle for weathervaning, but has not been included in this study.

15.3 System limitations

Because the Soft-Yoke LNG transfer system described above has not been constructed yet, assumptions have to be made to determine legitimate values for the limitations of this system. Metocean conditions that cause the vessel to exceed these limitations will result in downtime.

Note that all limitations mentioned here are assumptions based on subjective estimates. They should only be regarded as a tool to obtain insight in the possibilities of this configuration. An explanation of the six possible vessel motions is given in section 16.2.1.

- The maximum mooring force in the yoke is assumed to be 200 ton (or towards the jacket as a push-force: - 200 ton). The resulting maximum surge motion of the ship follows from the load – excursion curve from Figure 15-3 above, and amounts to +/- 6.6 m.
- Although there is no physical limitation to the roll motions of the vessel, workable limit is set at +/- 1.5 degrees.
- The workable limit for the pitch motion of the vessel is set at +/- 1.5 degrees. This pitch will result in a maximum heave motion of the bow of the vessel of +/- 3.8 m.
- Because sway motions can easily be accommodated by rotation of the crane, these are considered not to be governing.
- In this case the yaw motions will be considered rotating around the manifold connection at the vessel's bow; hence yaw is defined as the angle between the heading of the vessel and the angle of the yoke. 45 degrees has been assumed as a maximum to this yaw motion.

The values discussed above have been summarised in Table 15-1 below.

Criteria	Minimum	Maximum
Mooring force	- 200 ton	+ 200 ton
Surge	- 6.6 m	+ 6.6 m
Heave	- 3.8 m	+ 3.8 m
Yaw	- 45°	+ 45°
Roll	- 1.5°	+ 1.5°
Pitch	- 1.5°	+ 1.5°

Table 15-1 Limitations to yoke configuration

Another significant limitation of this configuration is that the vessel should maintain a certain distance from the fixed structures, i.e. the jacket, the jetty and the caisson.

The angle of the sector in which the vessel is allowed to weathervane, while remaining connected (and transferring LNG), depends on the following parameters:

- Dimensions of the caisson
- Size of the vessel
- Length of the loading arm
- Length of the jetty which connects the LNG storage caisson with the jacket
- Minimum required distance between fixed structures and the moored vessel

In this study the dimensions of the caisson, the size of the vessel and the length of the loading arm will be considered as a given. However the jetty can be extended to increase the allowable weathervaning angle of the moored vessel, as can be seen from Figure 15-4 below.

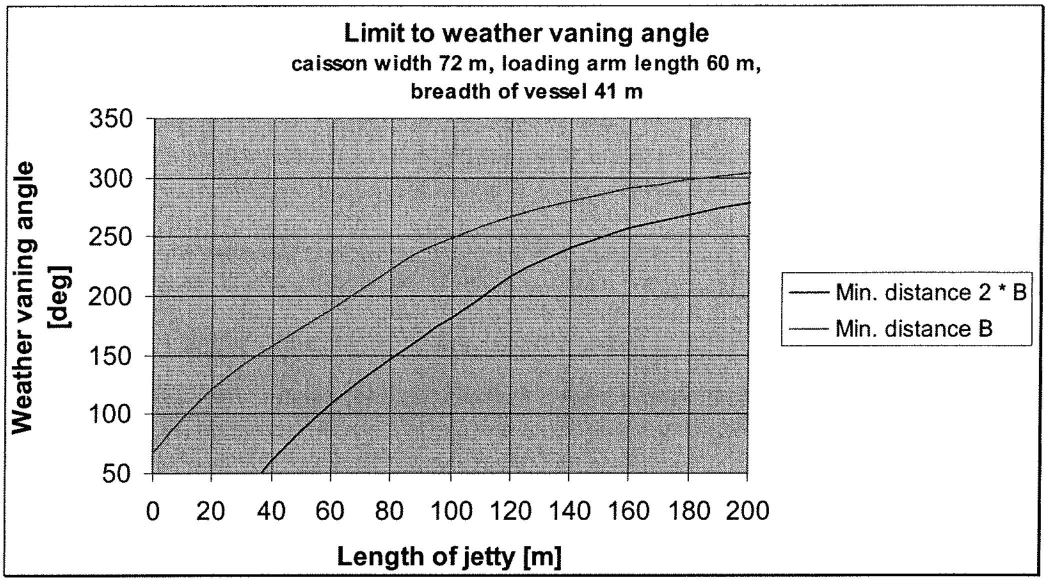


Figure 15-4 Limit to weathervaning angle

Figure 15-4 is the result of simple trigonometry calculations, assuming the heading of the vessel in-line with the angle of the yoke. It shows that the required minimal nominal distance between the vessel and the caisson has large impact on the length of the jetty required to allow the same weathervaning angle. During this study a minimal nominal distance of 2 times the breadth of the vessel will be used as a conservative estimate.

16 Moored ship analysis

16.1 General

The purpose of this part of the study is to identify limiting conditions for wave, wind and currents for the selected mooring configuration.

First some of the basics of ship hydromechanics are discussed in 16.2. Then the TERMSIM model with its assumptions and limitations will be described in section 16.3. Subsequently the mean heading of the vessel as well as the angle of the yoke will be determined for operational conditions in 16.4. Afterwards the critical vessel motions will be identified in paragraph 16.5. Finally conclusions regarding the moored ship analysis will be given in section 16.6.

16.2 Moored ship hydromechanics

To be able to understand the procedure followed in this chapter, some of the basics of moored ship hydromechanics will be explained in this paragraph (Ref 43).

16.2.1 Vessel motions

The movements of a moored ship at a berth are either translations (surge, sway and heave) or rotations (roll, pitch and yaw). These six degrees of freedom can be divided into horizontal (surge, sway and yaw) or vertical (roll, pitch and heave) movements, see Figure 16-1 below.

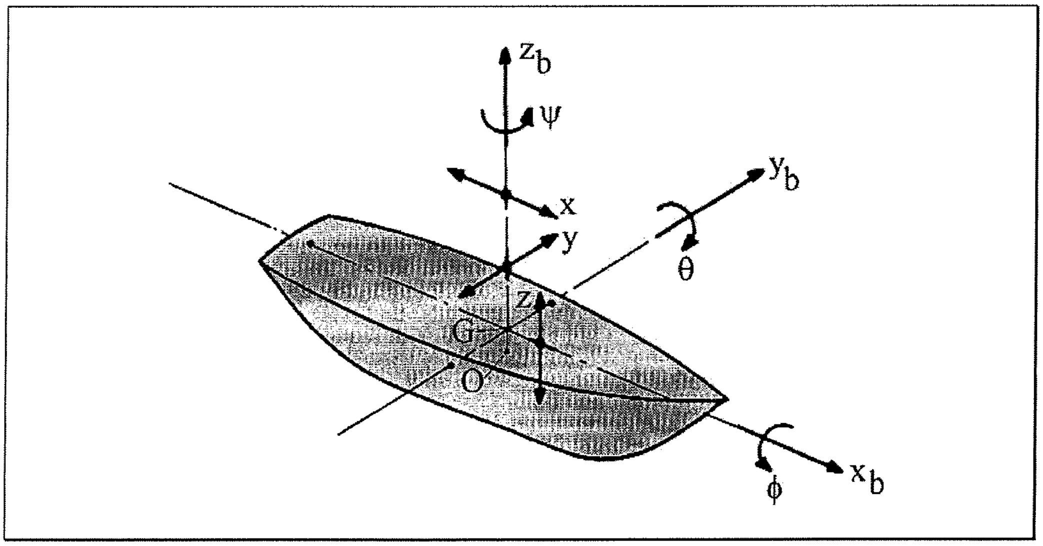


Figure 16-1 Six degrees of freedom (surge x, sway y, heave z, pitch θ, roll φ and yaw ψ) (Ref 50)

Vertical ship motions are almost independent of the mooring system, but horizontal motions are typically dependent on the loading conditions of a ship, the mooring arrangements, i.e. geometry and stiffness of mooring lines and fenders, and the type of berth.

Motions of moored gas tankers are in particular induced by currents and low-frequency wind and wave effects. Acceptable motions are determined by restrictions in the cargo handling systems (loading arms) and the mooring line and fender forces. When ship movements are too large, safe working limits and ultimately safe mooring limits are exceeded.

16.2.2 External actions

A moored vessel is subject to several external actions. The geometric and physical characteristics of the ships' mooring system play a major role in how the ship responds to these external actions. The aim should be that the moored ship can resist the total forces, thus avoiding damage to the ship or the mooring system.

Therefore it is important to know the combination of external actions on the moored vessel, as well as their magnitude and relative importance. The following forces are relevant:

- Wave loads

  For waves offshore it is necessary to distinguish between short and long period waves. In most cases short period waves do not constitute a serious problem for a moored ship, except at exposed berths. The wave periods of storm and swell waves are far from the natural periods of surge, sway and yaw of large ships. Therefore, horizontal motions of significance are normally not occurring due to short waves. The natural periods of heave, pitch and roll are typically within the range of short wave periods and consequently these motion modes can be excited.

- Wind loads

  The wind effect can be decomposed into a static action (constant wind or slow variation in intensity) and a dynamic action (gusty wind, intensity blusters and changing direction). Wind effects on gas tankers are more important compared to oil tankers because of the higher freeboard and the possible presence of spherical tanks.

- Current loads

  Current forces are caused by pressure drag. Under certain circumstances, a current can induce lateral oscillations due to "flutter". Flutter occurs when the arm of the moment exerted by the combination of external forces relative to the centre of gravity of the ship, including the added mass, reaches a value close to the radius of gyration.

  An essential parameter, besides the current speed, is the under keel clearance, defined as the free vertical distance from the bottom of the ship to the bed of the basin. If the under keel clearance is small, the influence of the current can increase up to six times the value in deep water.

Other external actions such as resonance because of long wave phenomena, the effects of astronomical tide as well as the influence of passing ships and loading and unloading operations can also play an important role. However, because only a first insight in the dynamic behaviour of the ship in this mooring system is required, these aspects are considered to be outside the scope of this study.

16.3 TERMSIM Model

16.3.1 General

The computer program TERMSIM developed by MARIN (Ref 67) will be used to simulate the configuration. It is a time domain simulation program to analyse the dynamic behaviour of a moored tanker subject to wind, waves and current. The mooring system can be a Single Point Mooring (SPM), a Multi Buoy Mooring (MBM) or a Jetty terminal. The SPM module will be used for the simulations of the soft-yoke mooring system. The program predicts the mooring loads and motions of the LNG carrier when the system is exposed to operational environmental conditions.

16.3.2 Environmental conditions

The environmental conditions that can be inserted into TERMSIM concern steady current, steady or irregular wind field, swell and long crested irregular waves coming from arbitrary directions. Several spectral formulations for the wind, wave and swell are available. The water depth can be arbitrary.

The actual environmental conditions at the site location have been described in Chapter 4. The buoy from the National Buoy Data Centre (Ref 44) provides a time series of measurements of wind velocity and direction, significant wave height, mean period and mean wave direction from October 1993 to July 1997. During this period the buoy has measured metocean parameters every hour resulting in approximately 32,000 measurements. This dataset will be used as input for the moored ship analysis as well as the downtime assessment in the next chapter.

It should be noted that due to a different location the buoy measurements do not reflect the correct metocean conditions. Local bathymetry has an impact on refraction and diffraction of the incoming waves. Figure 16-2 shows that due to the shape of the coastline the buoy receives waves from a sector different from the sector for the site location. In fact the blue sector from 0 to 20 degrees should be included, while the yellow sector from 90 to 125 degrees should be subtracted from the data. Nevertheless the unaltered buoy data set will be used to get a general impression of the environmental conditions.

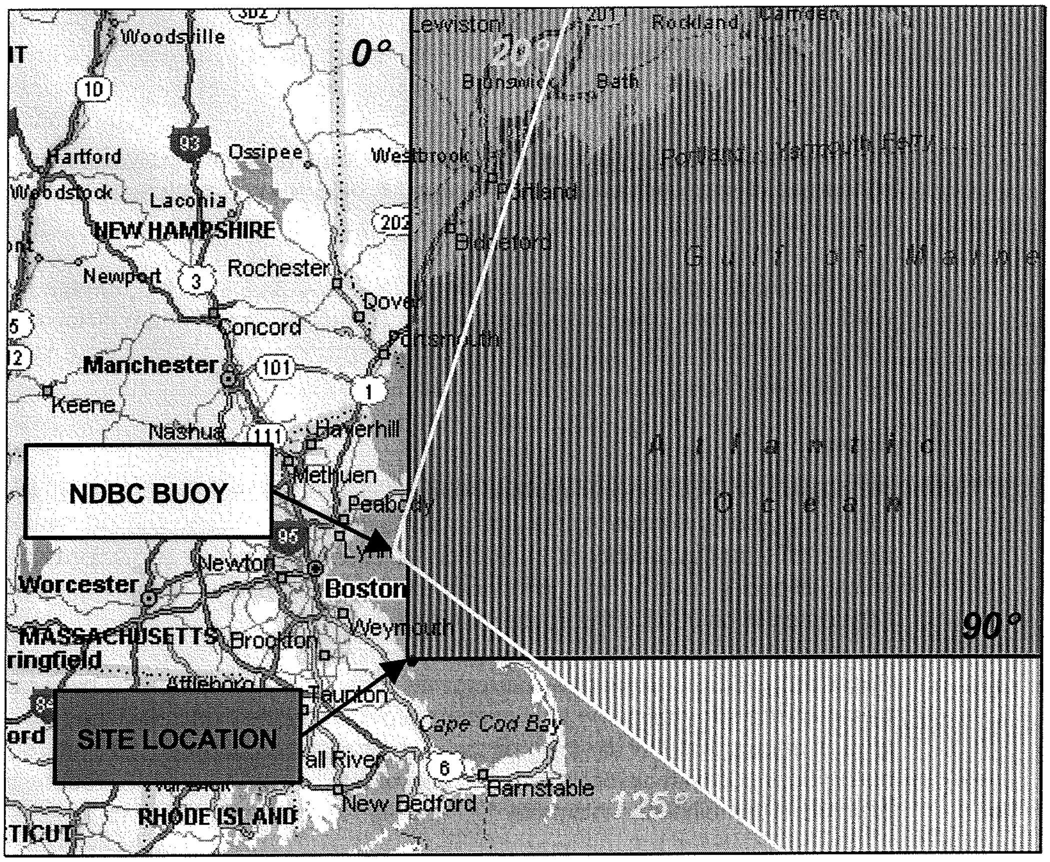


Figure 16-2 Directionality limitations due to bathymetry for buoy and site location

A large number of combinations of metocean conditions should be considered to obtain a clear impression of the vessel's dynamic behaviour. However, to limit the number of possible combinations, some of the environmental conditions have been assumed constant.

- Water depth

In practice there will be variations in water depth due to tide, wind or waves set-up or storm surges. These variations will not be incorporated in the TERMSIM simulations to limit the number of runs. Water depth variations will result in variations of under keel clearance, which is an important parameter for the pressure drag on a vessel caused by current. In all TERMSIM simulations the minimal water depth (15 m) has been used, resulting in the largest current drag forces. As a result, in practice during high water, current drag forces are expected to be less than calculated.

- Waves

Regarding the wave conditions only a mono-directional sea has been used as input for TERMSIM. A multi-directional sea (with wave height, period, spectrum and direction separate for sea and swell waves) would be a more realistic approach, but on the other hand would result in a huge increase of combinations of environmental conditions to be analysed. Moreover the NDBC Buoy does not make a distinction between sea and swell waves. Therefore in TERMSIM only one irregular wave field, characterised by a wave spectrum, a significant wave height and a mean wave period, will be generated for each run. Future studies should be carried out to analyse the effect of multi-directional seas.

The JONSWAP wave spectrum with a peakedness factor $\gamma = 3.3$ has been selected for generation of the irregular wave field. This spectrum is commonly used for not fully developed, fetch-limited (coastal) seas. Although some fully developed swell can be expected from the northeast, the majority of the incoming waves will be fetch-limited (see also sections 4.3.3 and 5.4).

- Wind

Because NDBC Buoy measurements show that wind gusts are likely to occur in the area concerned, it was decided to use an irregular wind field characterised by a mean wind velocity and direction.

The TERMSIM option to use the Harris – DNV wind spectrum to generate an irregular wind field has been used. The spectrum (recommended for offshore installations by Det Norske Veritas (Ref 24)) provides a gusting wind field with varying wind velocities but constant direction.

- Current

Although a time-dependent current velocity/direction series can be used as input for TERMSIM, it was decided to assume a constant current velocity and direction during the simulation time (half an hour). In practice variations in direction and velocity are likely to occur, especially due to tidal influences. However the effects of these phenomena are considered to be outside the scope of this study.

Because the NDBC Buoy does not provide current data, assumptions have been made based on the information on tidal streams provided by the Admiralty Chart (Ref 8) of the area. It shows that the direction of the current switches between 10 and 180 degrees during each tide, with velocities varying between 0.1 and 1.1 knots. To reduce to number of possible combinations this pattern is modelled according to Table 16-1.

Hour	Current direction	Current velocity
1	-	0 m/s
2	180	0.5 m/s
3		
4		
5		
6		
7	-	0 m/s
8	10	0.5 m/s
9		
10		
11		
12		

Table 16-1 Pattern of tidal cycle used in TERMSIM

TERMSIM generates the irregular wave and wind fields based on random seeding numbers. Each seeding number creates a different irregular field, each with its own mean, minimum and maximum values. Therefore, to obtain a good impression of the situation multiple runs with different random seeding numbers have to be carried out. However, instead of doing multiple runs for each combination of environmental conditions, twenty preliminary runs have been carried out with varying random seeding numbers for irregular wind and wave fields. The random seeding numbers that resulted in output closest to the average output of the twenty runs were selected. These numbers have been used for all simulations.

16.3.3 LNG Carrier characteristics

TERMSIM requires a so-called HYD file to describe the hydrodynamic coefficients of a specific vessel. In this study a file has been used of a 130,000 m^3 LNG carrier fitted with spherical tanks, moored at 14 meters of water depth. The HYD file comprises of the specific added mass and viscous damping coefficients as well as the results of a diffraction analysis for the specified ship.

Other main characteristics of the modelled vessel are given in Table 16-2.

Parameter	Value
Capacity	130,000 m^3
Length of all (LOA)	290.5 m
Length between perpendiculars (LBP)	267.15 m
Moulded breadth	41.15 m
Displaced moulded volume	96,361 m^3
Height (from keel to main deck)	26.0 m
Draft (laden)	11.0 m
Projected side area above waterline	7,555 m^2
Projected front area above waterline	1,545 m^2

Table 16-2 Characteristics of LNG carrier used in TERMSIM

In the TERMSIM program, additional viscous damping for surge, sway and yaw can be entered. Since in this case a shallow water situation is considered, 1 – 2 % additional viscous damping may be justified. Runs will be carried out including these values for additional damping.

The actual water depth at the selected site location amounts to 15 m instead of the 14 m used in the hydrodynamic file. Because more suitable hydrodynamic files were not available the 14 m water depth file will be used. Therefore the hydrodynamic calculations will be slightly conservative (shallower water being less favourable), although the deviations are expected to be small. Note that for all other calculations (e.g. wind, current and wave forces) the correct water depth (15 meters) can be entered in the program.

16.3.4 Mooring system

The mooring system will be modelled using TERMSIM's SPM module, which is based on a conventional CALM buoy. The buoy is moored by means of anchor chains to the seabed. The carrier is moored to the buoy by means of a bow hawser.

To adapt the SPM model to the mooring system considered in this project, some important assumptions have to be made.

- The SPM model features a floating buoy anchored to the seabed, while the actual mooring system consists of a fixed jacket structure. Therefore the input characteristics of the buoy should be adjusted to simulate an immovable object. This can be achieved by assuming a number of anchor chains with great stiffness, strength and pretension. Furthermore the diameter of the buoy will be assumed as small as allowed by the program to minimise the wave, wind and current forces acting on the buoy. Preliminary runs have been carried out to check whether these assumptions resulted in a realistic approach. Output shows that the maximum amplitude of the movements of the buoy lies in the order of 10 cm during the most harsh weather conditions.
- TERMSIM allows only for a flexible bow hawser as connection between the ship and the buoy. However the actual mooring configuration comprises of a Soft-Yoke, which is also capable of providing a push-force when the ship comes too close (which obviously a hawser cannot). Unfortunately it is not possible to feed TERMSIM a force – incursion curve for the hawser with negative excursions (incursions) or negative tension (pressure) forces.

  A solution for this problem is to discard all simulations that show a negative excursion of the hawser (slacking of the line). While determining the upper operational limits of the environmental conditions, i.e. relatively strong winds, currents and waves forces, slacking of the hawser is not likely to occur during a simulation. However, when calculating the mean vessel heading, all occurring combinations of environmental conditions have to be considered, including situations where resultant forces direct the vessel towards the buoy. Therefore, an alternative approach is suggested.

  A better solution is to simulate the push-force exerted by the yoke by shifting the equilibrium position of the hawser and applying additional astern propulsion, giving it a sort of 'pretension'. This can be explained best by looking at Figure 16-3.

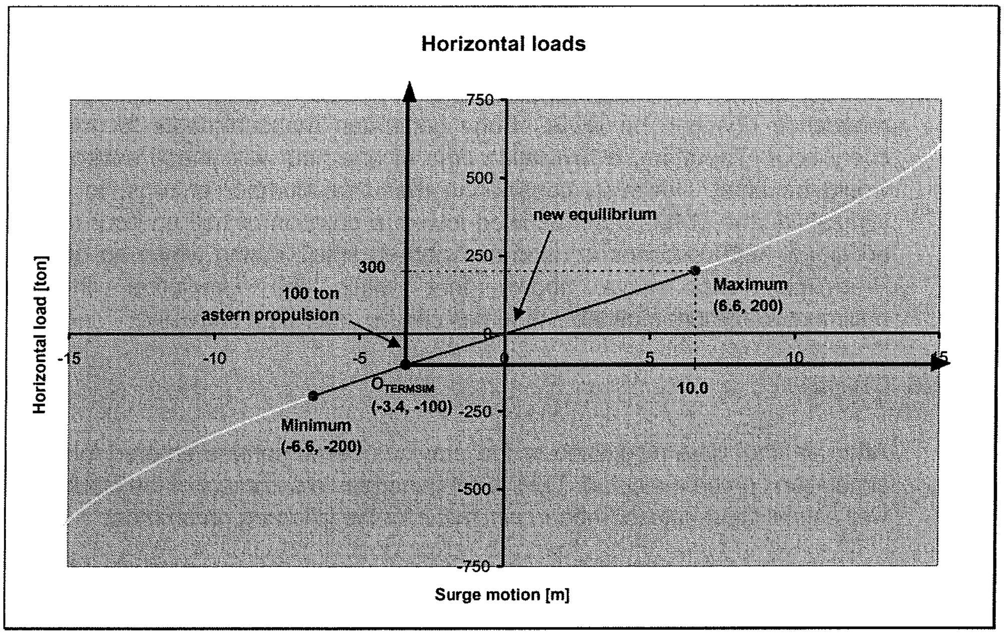


Figure 16-3 Shifted co-ordinate system for TERMSIM load – excursion curve

The theoretical load – excursion curve for the Soft-Yoke with a mooring leg length of 35 m and a counterweight of 650 ton has been indicated in yellow. When assuming a maximum mooring load of 200 ton, the minimum and maximum surge motion envelope can be determined (red curve).

When a 100 ton constant astern propulsion force is included in the TERMSIM input (100 ton is the maximum value allowed by TERMSIM), the hawser is stretched over 3.4 m according to the load – excursion curve. Therefore the origin of the co-ordinate system used in TERMSIM (blue lines) now lies at (-3.4, -100) in the actual co-ordinate system. The new equilibrium of the pre-tensioned hawser will still be at (0, 0) in the actual co-ordinate system, but at (3.4, 100) in TERMSIM. From this equilibrium the hawser in TERMSIM is able to generate a pull-force until 6.6 + 3.4 = 10.0 meters excursion, as well as the desired push-force up to 3.4 meters incursion. In this case only those simulations have to be discarded where the incursion output exceeds this value, which is not likely to occur in practice. However this will be verified for each run.

Main TERMSIM input parameters for the mooring configuration have been summarised in Table 16-3 below. Note that the hawser length of 60 m has been reduced to 56.6 m because, due to the astern propulsion, the hawser is stretched for 3.4 m in the equilibrium position.

Parameter	Value
Astern propulsion [kN]	1962
Hawser fairlead X, Y, Z (with respect to COG) [m]	(143.2, 0, 15)
Unstretched hawser length [m]	56.6
Number of buoy anchor lines [-]	6
Chain pretension [kN]	1000
Chain length [m]	1000
Chain breaking strength [kN]	100,000
Chain elasticity [kN]	500,000
Diameter buoy [m]	0
Draft of buoy [m]	0

Table 16-3 TERMSIM input parameters for mooring configuration

16.3.5 Simulation control parameters

The duration of each TERMSIM simulation should be long enough to include all possible resonance effects. The NDBC Buoy generates measurements for metocean conditions every hour. Therefore, a simulation time of one hour with these environmental conditions would be ideal. However, considering the large number of runs, to decrease the total simulation time, it has been decided to use a duration of half an hour (1800 seconds). An additional 1800 seconds is used as “start-up time”, during which no output is generated. Preliminary runs have shown that within the simulation time all significant minimum/maximum motions and forces can be calculated correctly.

16.3.6 Output

After all input data discussed in the previous paragraphs is entered into the program, the simulation can be executed. TERMSIM generates a summary of the input and time series as well as statistical values (mean, min, max) for the following parameters:

- Significant wave height [m]
- X, Y and Z motions of vessel [m]
- Roll, pitch and yaw motions of vessel relative to COG [deg]
- X and Y motions of buoy [m]
- Mooring force in hawser [kN]
- Mean vessel heading and hawser angle [deg]
- Forces in anchor chains [kN]
- Mooring forces and moments in X, Y and Z direction [kN; kNm]
- Combined environmental forces and moments in X, Y and Z direction [kN; kNm]
- Forces and moment in X, Y, Z direction [kN; kNm] for wind, waves and damping

An example of a TERMSIM output file (with a summary of input included) has been enclosed in Appendix L.

16.4 Mean vessel heading

16.4.1 General

The fixed concrete LNG storage facility limits the weathervaning capability of the mooring system (see also section 15.3). When environmental forces cause the vessel to drift outside the allowed envelope, towards the fixed terminal, an emergency disconnect procedure is required and the combination of environmental conditions can be marked as a “downtime event”. To be able to determine these downtime events it is vital to have insight in the mean vessel heading as a result of varying weather conditions.

In this paragraph the vessel heading as well as the yoke angle will be determined for each event in the NDBC series using TERMSIM.

16.4.2 Analysing dataset

The first step is to reduce to number of combinations that has to be simulated. The 32,000 records each consist of 7 variables which describe the environmental conditions. The values have been categorised by rounding off to certain intervals, which can be seen in Table 16-4 below.

Parameter	Unit	Minimum	Maximum	Interval size	# of intervals
Significant wave height	[m]	0	8	1	9
Mean wave period	[s]	0	12	2	7
Mean wave direction	[deg]	0	330	30	12
Wind velocity	[m/s]	0	20	5	5
Wind direction	[deg]	0	330	30	12
Current velocity	[m/s]	0	0.5	-	2
Current direction	[deg]	10	180	-	2

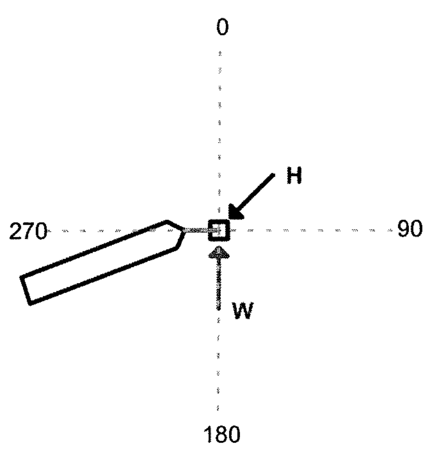
Table 16-4 Categories applied for NDBC data

Simulating all possible combinations of these categories would require 181,440 runs. Fortunately not all combinations will have an equal probability because there are strong correlations between the parameters. For example, waves often come from the same direction as the wind, and higher waves are likely to be accompanied by stronger winds. Therefore, for each combination the probability of occurrence (according to the dataset) has been calculated.

Subsequently, the most probable combinations have been selected until a sufficiently high coverage of the dataset had been reached. Because results are meant to be used for the assessment of downtime (expected to be in the order of 10 – 20 %, see also next chapter), the coverage should preferably be higher than 95%. After analysing the NDBC dataset it appeared that 1300 combinations (x 3 possible current situations, not included in the buoy data = 3900 combinations) were required to cover 98.7% of the data.

16.4.3 TERMSIM results

A batch procedure has been generated to carry out TERMSIM runs for the 3900 combinations of environmental conditions. A program has been written to retrieve the mean vessel heading and hawser angle from each output file. Table 16-5 provides an example of a combination of wind and wave directions resulting in a certain vessel heading and yoke angle, to illustrate the definition of directions, angles and headings used in this report.



Wave direction	45 deg
Wind direction	180 deg
Hawser (yoke) angle	270 deg
Vessel heading	250 deg

Table 16-5 Example to illustrate definition of directions, angles and headings

The results of the program have been inserted into a spreadsheet and have been combined with the values for the probability of each case. Figure 16-4 shows the probability of the mean vessel heading of the moored LNG carrier and the angle of the hawser (in this case the yoke).

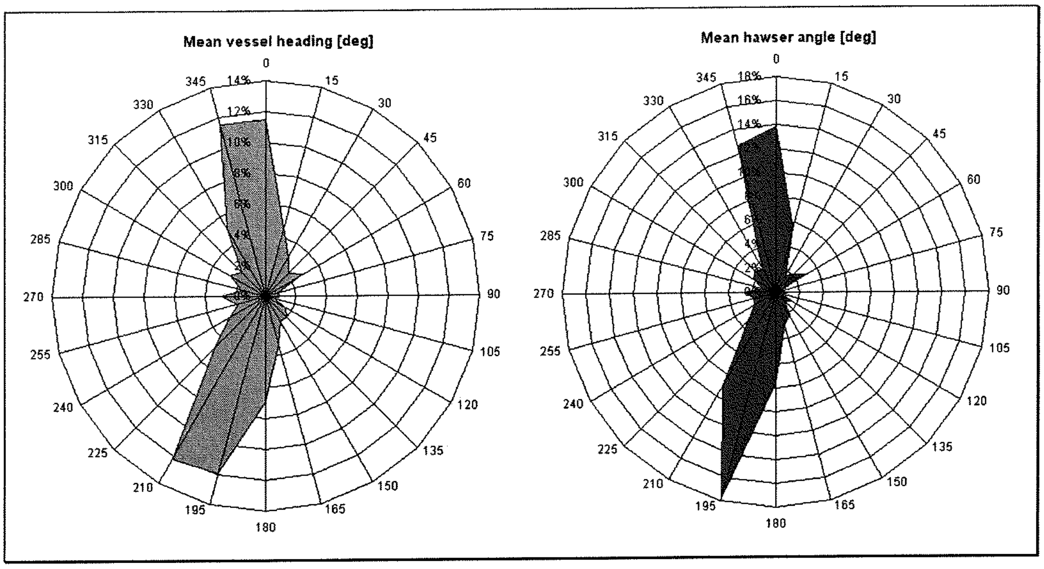


Figure 16-4 Probability of mean vessel heading and mean hawser angle

It can be seen that the current directions (coming from 10 and 180 degrees) have an important influence on the ship's heading. It becomes even more evident when the same graph is plotted for a situation without any current (see Figure 16-5).

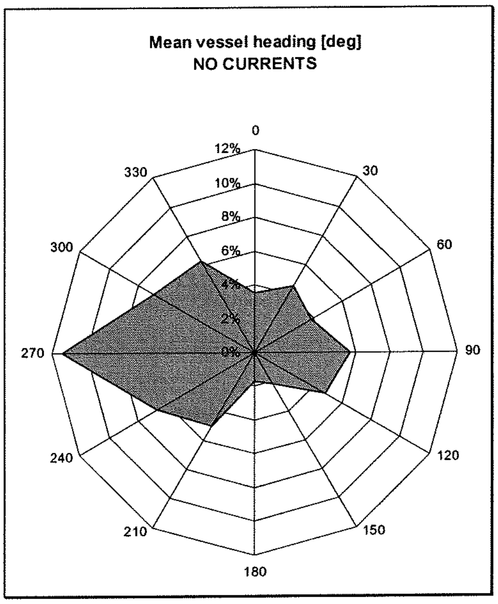


Figure 16-5 Probability of mean vessel heading without currents

In this case the influence of the majority of the waves coming from the east becomes more clear. To get an insight in the relative impact of current in relation to wind and wave forces on the mean vessel heading, two other graphs are presented below.

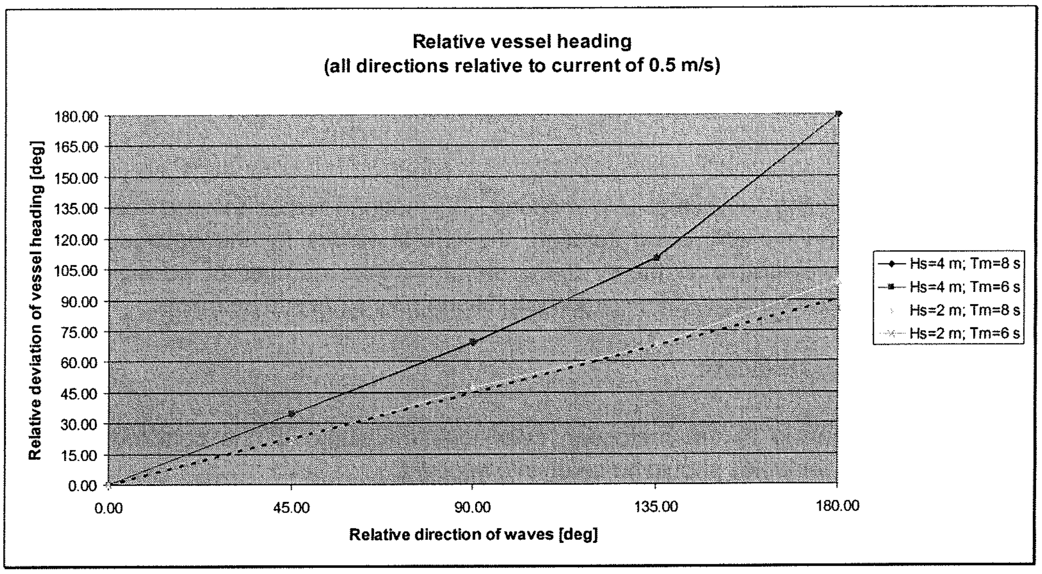


Figure 16-6 Wave influence on vessel heading relative to current

Figure 16-6 shows the influence of waves on the mean vessel heading, while Figure 16-7 does the same for the influence of winds, all relative to a current of 0.5 m/s coming from a direction of 0 degrees. The black dotted line indicates the situation when the vessel's heading lies exactly in the middle of the angle between current and wind / waves, in other words, when they have equal influence.

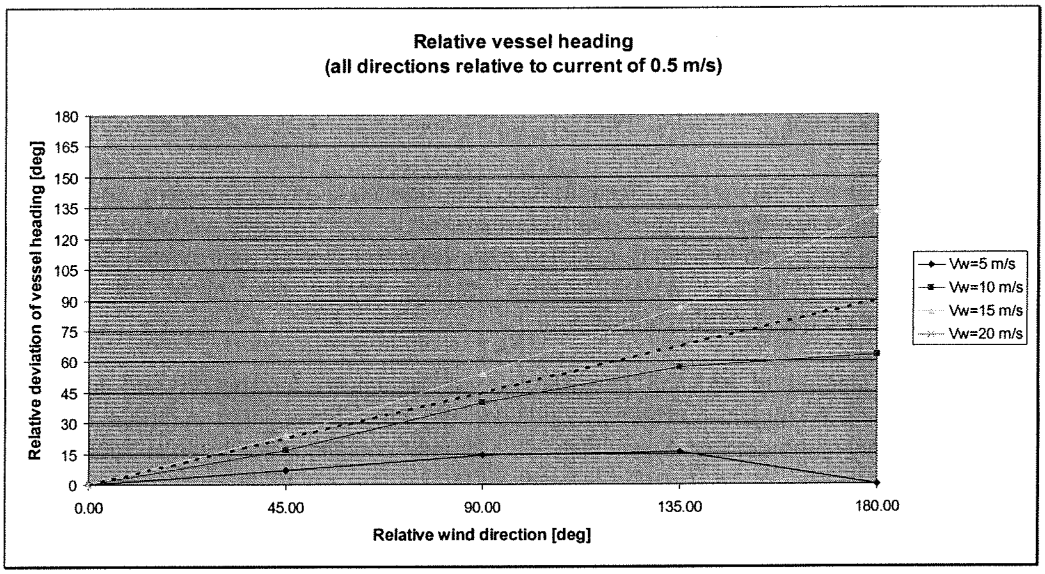


Figure 16-7 Wind influence on vessel heading relative to current

Concluding from these graphs can be said that, when looking at the influence on the mean vessel heading, 0.5 m/s current equals approximately 12 m/s wind or waves with a significant wave height of 2 m and a mean period of 7 s. The reason that, in this case, the vessel heading – probability plot (Figure 16-4) is dominated by current influences, can therefore be explained by the fact that wind velocities of 12 m/s or waves of 2 m, 7 s are seldom exceeded within the dataset (see section 5.6).

16.5 Critical vessel motions

16.5.1 General

By calculating the actual ship motions and mooring forces for each combination of wind, waves and current, using iteration, the critical values for the each combination can be identified. Because in theory there are an infinite number of possible combinations of metocean conditions, some simplifying assumptions have to be made.

16.5.2 Analysing dataset

When looking at the metocean conditions measured at the selected site location, the following variables should be taken into account:

- Current direction (10 or 180 degrees)
- Current velocity (0 or 0.5 m/s)
- Wind direction (all directions)
- Wind velocity (0 to 20 m/s)
- Wave height (0 to 8 m)
- Wave period (0 to 14 s)
- Wave direction (all directions)

Even after categorising these variables into discrete intervals (see also previous paragraph), this will still result in an unacceptable high number (2 current speeds x 2 directions x 5 wind speeds x 12 directions x 9 wave heights x 8 periods x 12 directions = 207,360) of combinations.

To reduce this number even further, the directions of wind, wave and current will be translated to relative angles: each combination is characterised with a ΔCW (angle between current and wind direction), a ΔCH (angle between current and waves) and a ΔWH (angle between wind and waves). These relative angles can either be 0° (in-line), 90° (perpendicular) or 180° (opposite).

Now instead of iterating towards the critical values for motion and forces, a specific combination of environmental conditions will be selected, that is "more severe" than the majority of the dataset. Some of these "ultimate" conditions are given in Table 16-6 below.

H_s (m)	T_m (s)	V_w (m/s)	Probability less severe
2	6	10	78.8%
2	8	10	95.2%
2	10	10	96.0%
3	6	10	80.2%
3	8	10	97.4%
3	10	10	98.1%
3	6	20	81.6%
3	8	20	98.9%
3	10	20	99.6%

Table 16-6 Probability of conditions being less severe

The highlighted condition will be selected as upper limit between "uptime" and "downtime". The 0.4% of the dataset which shows even higher values will be considered instantly as downtime events, regardless of the fact that these conditions might not cause exceedance of the critical vessel motions or mooring forces.

Subsequently, for the remaining dataset (99.6%) TERMSIM simulations will be carried out for 9 types of waves (H_s = 1, 2 or 3 m, T_m = 6, 8 or 10 s) and 2 winds (V_w = 10 or 20 m/s). Together with the current situations and the relative angles between current, wind and waves, this results in a total number of 279 runs to be carried out, as can be seen from Table 16-7.

Current	**Waves**	**Wind**	**Relevant combinations**
No	No	Yes	2 V_w = 2
No	Yes	No	3 T_m x 3 H_s = 9
No	Yes	Yes	3 dir x 3 T_m x 3 H_s x 2 V_w = 54
Yes	No	No	1 U_c = 1
Yes	No	Yes	1 U_c x 3 dir x 2 V_w = 6
Yes	Yes	No	1 U_c x 3 dir x 3 T_m x 3 H_s = 27
Yes	Yes	Yes	1 U_c x 10 dir x 3 T_m x 3 H_s x 2 V_w = 180
TOTAL number of runs			**279**

Table 16-7 Total number of runs

16.5.3 TERMSIM results

For each run the maximum yaw, roll, pitch and mooring force (force in the yoke) are extracted from the TERMSIM output files. The results have been included in Appendix M. The following remarks should be considered while reading the results:

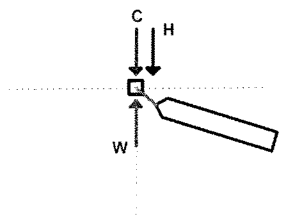
- As already has been explained in section 16.3.4, the yoke system is modelled in TERMSIM by a pre-tensioned hawser. If the forces in the hawser reach zero, the pre-tension is gone and the line goes slack. Now if environmental forces cause the vessel to drift away from the buoy again, the line goes from zero to very high tension in a very short period, resulting in very high mooring forces. In other words, if the forces in the hawser reach zero, the TERMSIM output will not provide reliable results. The results of these runs are marked red in Appendix M.
- When applying the critical motion criteria stated in section 15.3, it seems that the criteria for pitch (max. 1.5 degrees) and yaw (max. 45 degrees) are never exceeded during the simulated conditions.
- Regarding the maximum mooring forces no reliable conclusions can be drawn because simulations with higher (Hs > 2 m) and longer (Tm > 8 s) waves quite frequently resulted in slacking of the hawser (see Appendix M, already mentioned above). However even in these (unreliable) simulations the maximum mooring force still does not exceeded the maximum of 200 tons mentioned in section 15.3. It is therefore considered unlikely that in practice the mooring forces will exceed the critical value of 200 tons during the simulated conditions.
- The maximum roll motions of the vessel (max. 1.5 degrees) are in this case the only limiting criterion. Table 16-8 shows which combinations of environmental conditions result in exceedance of the roll motion criterion. These results will be used to determine the downtime of the system in the next chapter.

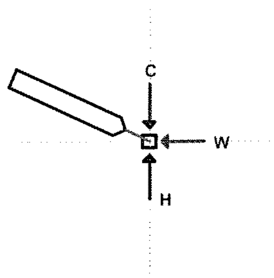
Environment				Relative angles			Maximum value			
H_s [m]	T_m [s]	V_w [m/s]	U_c [m/s]	ΔCW [deg]	ΔCH [deg]	ΔWH [deg]	Yaw [deg]	Roll [deg]	Pitch [deg]	Force [kN]
2	10	0	0.5	N/A	180	N/A	4.6	1.9	0.4	266
3	> 8	20	0.5	180	0	180	23.4	2.6	1.0	674
3	10	20	0.5	0	90	90	7.5	1.7	1.3	665
3	> 8	10	0.5	0	180	180	17.9	1.7	0.9	981
2	10	10	0.5	90	90	180	1.0	1.7	0.9	266
3	10	20	0.5	90	90	180	13.1	1.6	1.3	981
> 2	10	> 10	0.5	90	180	90	4.7	1.6	0.9	459
> 2	8	20	0.5	90	180	90	1.2	1.5	0.2	426

Table 16-8 Exceedance of the roll motion criterion (red numbers indicate slacking of hawser)

16.5.4 Maximum motions of vessel

Combinations of environmental conditions that resulted into the maximum vessel motions for the NDBC dataset are given in Table 16-9.

<table>
<tr><td rowspan="3">Maximum yaw [deg]</td><td rowspan="6">
</td><td>Current velocity [m/s]</td><td>0.5</td></tr>
<tr><td>Significant wave height [m]</td><td>3</td></tr>
<tr><td>Mean wave period [s]</td><td>8</td></tr>
<tr><td rowspan="3">23.4</td><td>Wind velocity [m/s]</td><td>20</td></tr>
<tr><td>Vessel heading [deg]</td><td>107</td></tr>
<tr><td>Yoke angle [deg]</td><td>130</td></tr>
</table>

<table>
<tr><td rowspan="3">Maximum pitch [deg]</td><td rowspan="6">
</td><td>Current velocity [m/s]</td><td>0.5</td></tr>
<tr><td>Significant wave height [m]</td><td>3</td></tr>
<tr><td>Mean wave period [s]</td><td>10</td></tr>
<tr><td rowspan="3">1.4</td><td>Wind velocity [m/s]</td><td>20</td></tr>
<tr><td>Vessel heading [deg]</td><td>294</td></tr>
<tr><td>Yoke angle [deg]</td><td>295</td></tr>
</table>

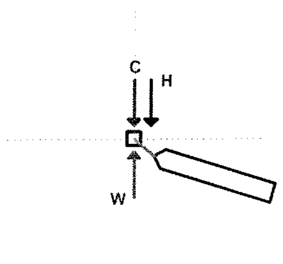
<table>
<tr><td rowspan="3">Maximum roll [deg]</td><td rowspan="6">
</td><td>Current velocity [m/s]</td><td>0.5</td></tr>
<tr><td>Significant wave height [m]</td><td>3</td></tr>
<tr><td>Mean wave period [s]</td><td>10</td></tr>
<tr><td rowspan="3">3.1</td><td>Wind velocity [m/s]</td><td>20</td></tr>
<tr><td>Vessel heading [deg]</td><td>105</td></tr>
<tr><td>Yoke angle [deg]</td><td>126</td></tr>
</table>

Table 16-9 Maximum vessel motions

To provide some insight in the influence of directionality on the maximum motions of the ship, four directionality figures have been plotted (see Appendix N). They show the influence of the incoming angles of a wind with 20 m/s velocity and waves with Hs = 3 m and Tm = 8 s on the yaw, roll, pitch and mooring force respectively. In all situations there is a 0.5 m/s current coming from 0 degrees. Different values for these three components (current, waves, wind) are likely to result in substantially different plots. Note that all graphs are symmetric with respect to point (180, 180).

16.6 Conclusions

From analysis of the moored LNG carrier carried out in this chapter, the following conclusions can be drawn:

- TERMSIM does not seem to be the ideal program to simulate a soft-yoke mooring configuration, because it cannot model a rigid connection between vessel and buoy. The "push" force of the yoke can be simulated by pre-tensioning of the hawser in the form of astern propulsion. However, application of this "trick" is limited for the pre-tension buffer is easily reduced to zero during relatively benign weather conditions, resulting in unreliable output for a small but substantial number of runs.
- Because of the shallow water (15 m) situation the small under keel clearance causes the current to be the dominating factor in the determination of the mean vessel heading. In this case study the total moment exerted on the vessel by a 0.5 m/s current can be seen as of approximately the same magnitude as the moment caused by a 12 m/s wind or a wave field with Hs = 2 m and Tm = 7 s.
- The yaw, pitch and mooring force criteria are never exceeded when considering the dataset of the NDBC Buoy. The 1.5 degree limit to the roll motions of the vessel are occasionally exceeded for certain combinations of conditions specified in Table 16-8.
- Maximum values for yaw, roll, pitch calculated for the NDBC dataset are 23.4, 3.1 and 1.4 degrees respectively. A value for the maximum mooring forces cannot be given because of problems with TERMSIM discussed in the first conclusion.

17 Downtime assessment

17.1 General

The results of the previous chapter with respect to the mean heading of the vessel will be used to optimise the length of the jetty as well as the orientation of the terminal (section 17.2). Afterwards a complete overview of all downtime criteria will be given in 17.3. When the downtime events have been established, the terminal operability can be determined in section 17.4. Conclusions concerning the downtime assessment are stated in paragraph 17.6.

17.2 Optimal berth orientation

Before an assessment of the downtime events can be made, the optimal orientation of the terminal has to be selected. The probability of the mean vessel heading determines the most favourable orientation.

In section 15.3 has already been pointed out that the presence of the fixed terminal generates a restriction on the weathervaning capability of the moored vessel. It was shown that the limiting envelope for the vessel's weathervaning motions depends on the length of the jetty that connects the yoke jacket with the terminal. However because the previous chapter shows that the angle of the yoke is not necessarily in line with the vessel heading, the relation between jetty length and vessel headings will be analysed more thoroughly.

Figure 17-1 shows the combinations of vessel heading and yoke angle that do not provide a safety distance of at least 2 times the breadth of the vessel for a terminal with a jetty length of 120, 150 and 180 m respectively, orientated at 90 degrees (east). The contoured background of the graph indicates the probability of each combination with respect to the NDBC dataset (orange being zero, blue being greater than 10%).

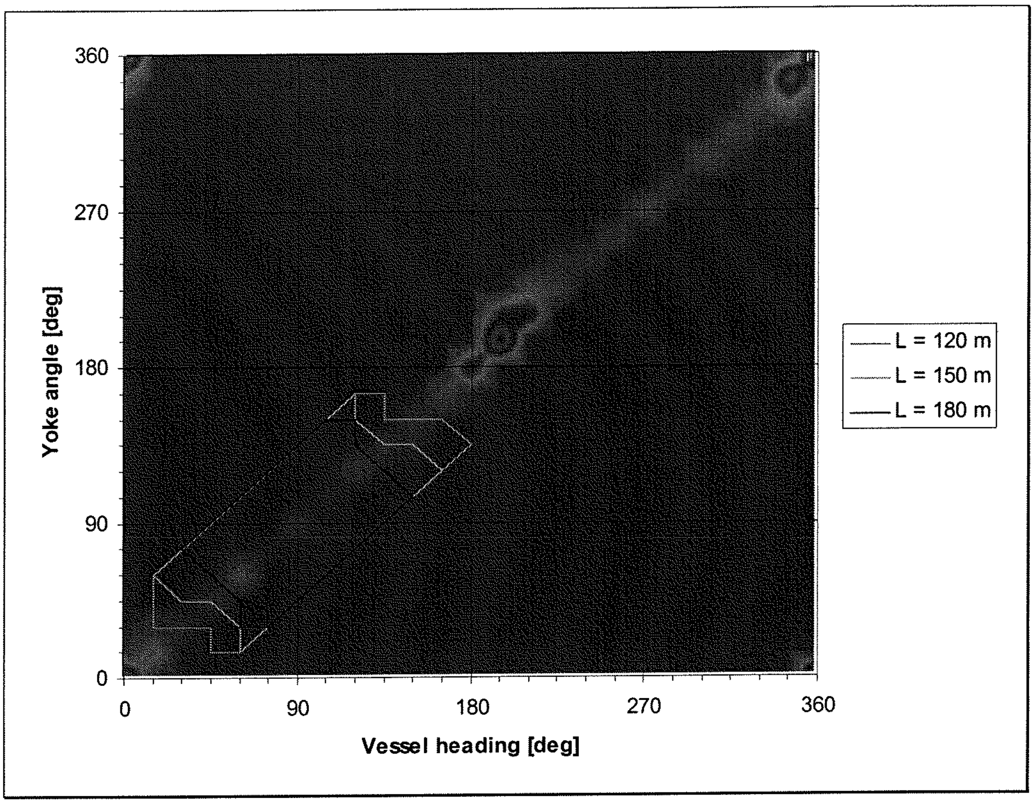


Figure 17-1 Limits to weathervaning due to terminal with jetty length L

In fact the three enclosed areas indicate the combinations of vessel heading and yoke angle that are not allowed because they are already occupied by the terminal. A different terminal orientation would result in three new enclosed areas with the same shape, but shifted along the y = x axis. For each terminal orientation and jetty length the total probability of the occupied area can be calculated. The result of this calculation is the probability of a downtime event caused by the vessel – terminal distance being less than required.

This probability has been calculated for all orientations divided into intervals of 15 degrees. The result of this calculation, as well as the optimal orientation for the terminal, is shown in Figure 17-2.

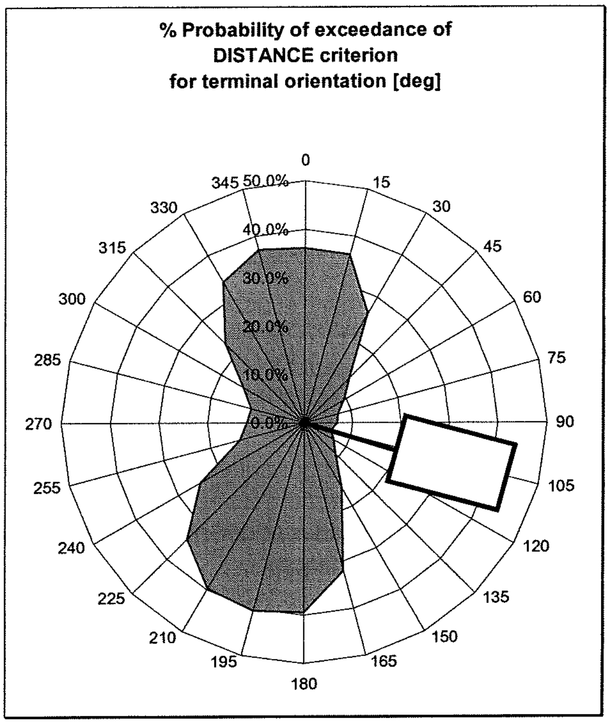


Figure 17-2 Optimal terminal orientation

As can be seen from Figure 17-2 the optimal orientation of the terminal is 105 degrees. By applying the heading / yoke angle restrictions on the dataset, the downtime events generated by the lack of distance between vessel and terminal can be determined for a chosen jetty length.

17.3 Downtime criteria

Events are identified as downtime when one or more of the following criteria are not met (values are derived from the mooring system limitations mentioned in section 15.3):

- YAW: The maximum angle between the mean vessel heading and the yoke angle should be smaller than 45 degrees.
- PITCH: The maximum pitch motion should be smaller than 1.5 degrees.
- ROLL: The maximum roll motion should be smaller than 1.5 degrees.
- FORCE: The maximum mooring force should be smaller than 200 tons.
- WAVE: The significant wave height should be smaller than 3 meters and the mean wave period should be smaller than 10 seconds (simplification made in section 16.5.2).
- WIND: The wind velocity should be smaller than 20 m/s (simplification made in section 16.5.2).
- DISTANCE: The minimal distance between the moored vessel and the fixed terminal should be at least two times the breadth of the vessel.

For each criterion the number of downtime events has been assessed. The results are given in Table 17-1.

Criteria	Limit	Downtime events	Downtime %
YAW	< 45 degrees	0	0%
PITCH	< 1.5 degrees	0	0%
ROLL	< 1.5 degrees	3	0%
FORCE	F < 200 ton	0	0%
WAVE	$H_s < 3$ m, $T_m < 10$ s	428	1.5%
WIND	$V_w < 20$ m/s	0	0%
DISTANCE (L = 120 m)	> 82 m	4244	14.5%
DISTANCE (L = 150 m)	> 82 m	2827	9.6%
DISTANCE (L = 180 m)	> 82 m	1667	5.7%

Table 17-1 Downtime events generated by downtime criteria

From the table can be seen the DISTANCE criterion generates by far the most downtime events. By increasing the length of the jetty there are more combinations of vessel heading and yoke angle allowed resulting in a decreased number of downtime events. However, the reduction itself decreases rapidly with increasing jetty length.

In the following paragraphs only a jetty length of 180 meter will be considered, with 7.1% downtime events (all criteria combined, partially overlapping).

17.4 Terminal operability

The value of 7.1% downtime does not provide sufficient information to say something about the operability of the terminal. The required persistency of the service time (offloading cycle) is crucial, because in practice the moored vessel is not allowed to leave the berth before the unloading process is completed (to prevent sloshing of the LNG inside the tanks). Therefore the captain will not access the berth before he has assured himself that during the service time no downtime events will occur.

Normally the total unloading process takes approximately 18 hours. Assuming that the captain has perfect weather forecasts, this means that the ship does not access the berth unless there are 18 “up-time” events ahead. This assumption can lead to relatively pessimistic results, because in practice it is possible that, while workable (offloading) limiting conditions are exceeded, at the same time the critical motions or forces of the mooring system are not yet reached. In that case, only the unloading process will be interrupted, while the vessel can remain moored until weather conditions have improved, allowing the unloading process to continue. Such a differentiation between workable and critical limits has not been incorporated in this study.

A program has been written to determine for each event the number of events (waiting time) before there are 18 “up-time” events ahead. Figure 17-3 below shows the impact that this “uptime persistency” has on the operability of the terminal.

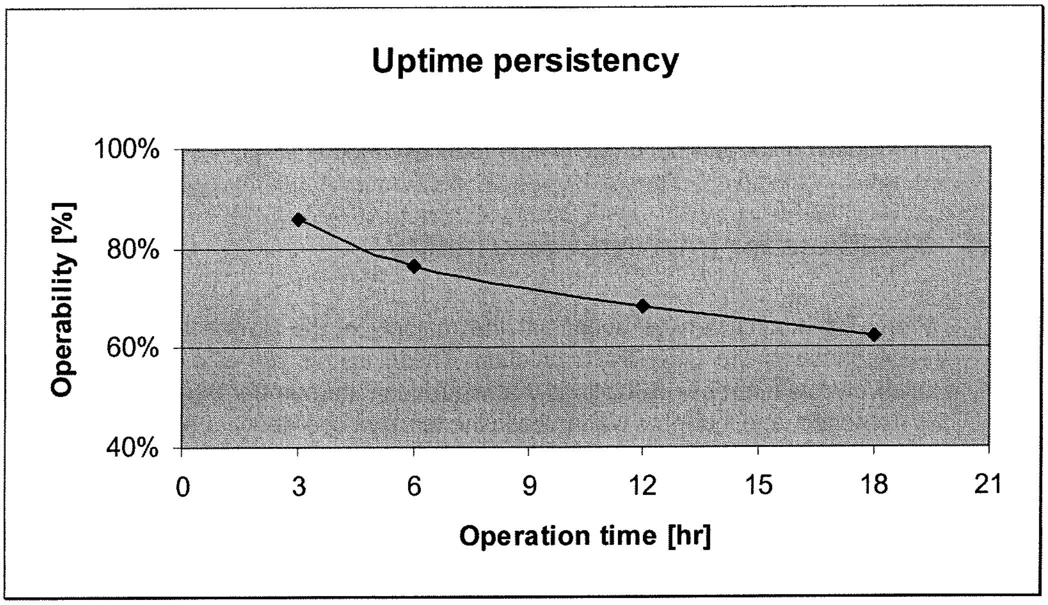


Figure 17-3 Relation between uptime persistency and operability

When assuming an operation (service) time of 18 hours, the operability of this terminal will be 62%. This means that 62% of the time the arriving vessel can immediately enter the berth.

Besides the operability, which defines the chance of the arriving vessel has no waiting time at all, it is also important to know the distribution of the waiting time. The output of the program results in an average waiting time of 11.1 hours. Figure 17-4 below shows the probability of waiting time being exceeded.

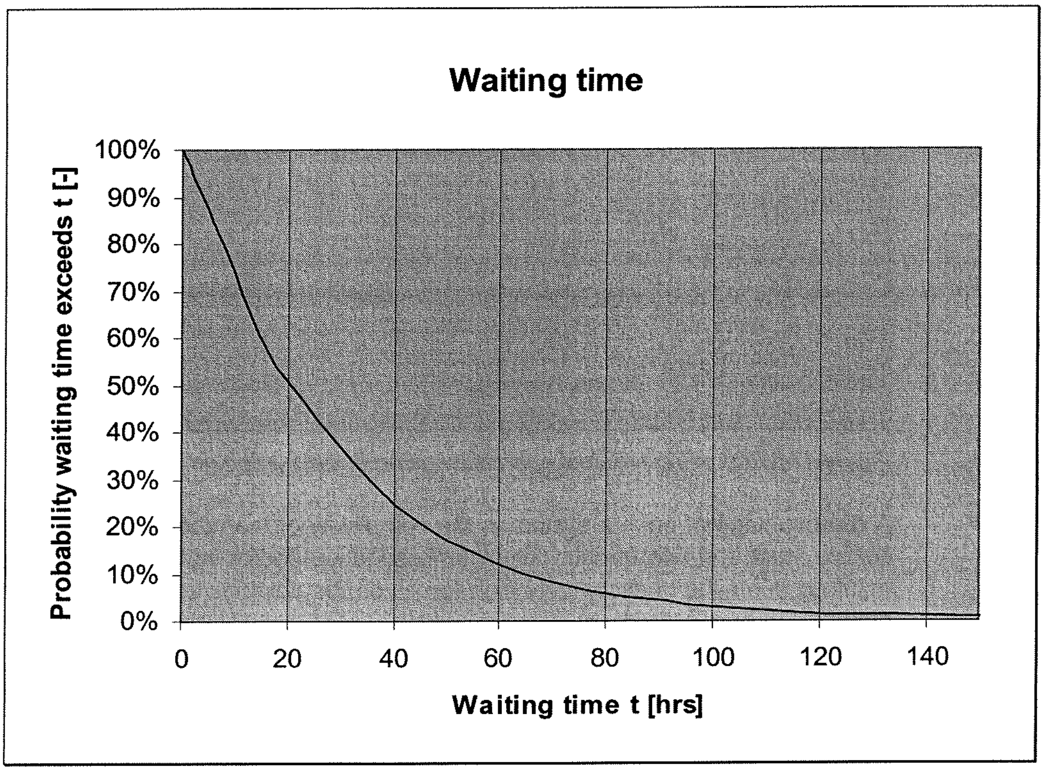


Figure 17-4 Distribution of waiting time

Another important aspect is the maximum number of downtime events in a row. During such a period the natural gas send-out continues while no new cargo loads arrive. At a certain moment the buffer volume contained in the LNG storage tank will be depleted.

In Appendix J already was determined that with a peak send-out rate of 1650 m^3/hour, it will take 1.6 days (39 hours) before the buffer storage is empty. The results of the computer program show that the longest period of consecutive downtime events is 168 hours (for the NDBC dataset). The chance of depletion of the buffer storage capacity, and therefore interruption of the gas send-out, is 26% (see also Figure 17-4).

17.5 Solutions for improved operability

From Table 17-1 can be seen that the majority of the downtime events are caused by exceedance of the DISTANCE criterion. These events are also the main reason for the relatively low operability. Solutions that will reduce the number of events where this criterion is exceeded, and therefore will increase the terminal operability, are given below.

- Construction of flexible fender dolphins

  Instead of initiating an ESD procedure when the ship enters the restricted area around the terminal (DISTANCE < 2 * B), flexible dolphins equipped with fenders can be constructed (see Figure 17-5). Now when the vessel drifts towards the terminal due to the prevailing conditions, it is held into place by the flexible dolphins. Probably such a passive control system allows for a reduction of the minimum distance required between ship and terminal, which can result in a shorter jetty.

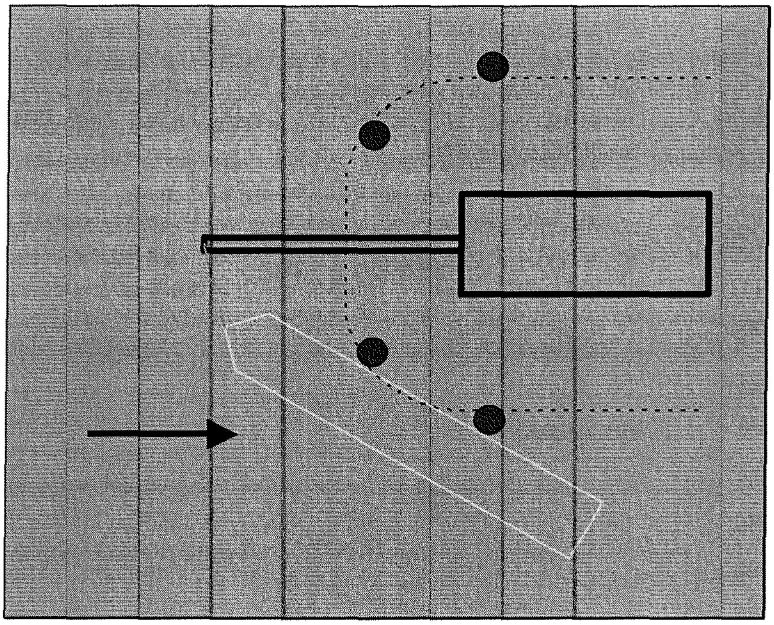


Figure 17-5 Improving terminal operability using flexible dolphins

  A disadvantage of such a system is the complexity of the mooring configuration with a flexible yoke and two flexible dolphins. Resonance and/or instability effects can occur, resulting in maybe unacceptably high forces on the dolphins and, more importantly, the hull.

- Equipping LNG carriers with Dynamic Positioning (DP) system with thrusters

  An active system such as Dynamic Positioning could also offer a solution for the high number of downtime events. An LNG carrier equipped with bow and stern thrusters will be capable of manoeuvring continuously to remain outside the restricted area, even when the prevailing directions of wind, wave and current point towards the terminal.

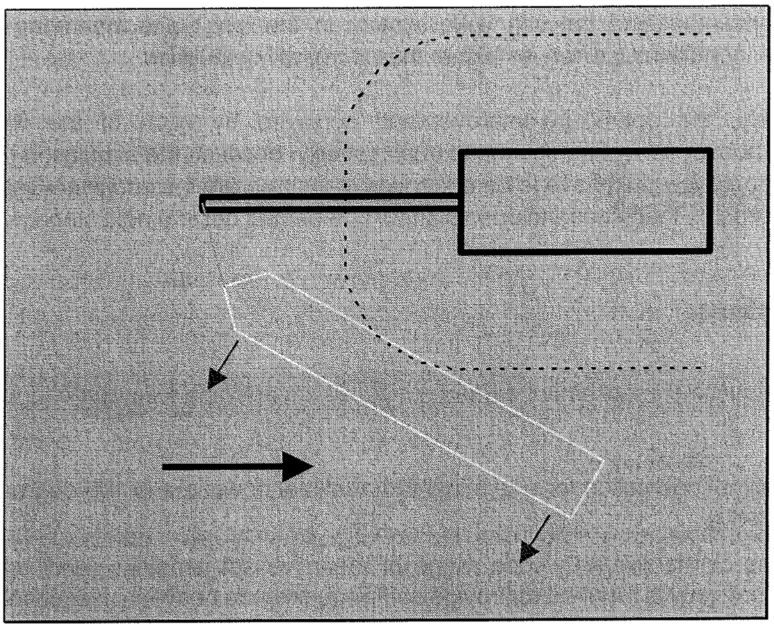


Figure 17-6 Improving terminal operability using a DP system

The main disadvantage of an active system is its reliability. If the system would fail during the unloading operation, still some kind of emergency procedure is required to be able to manoeuvre the ship into a safe area. Therefore, the required minimum distance between terminal and ship cannot be decreased.

- Providing tug assistance

  The most conventional type of active system is providing ordinary tug assistance during the unloading operation. One tug connected to the stern of the LNG carrier can be deployed to keep the vessel outside the restricted envelope.

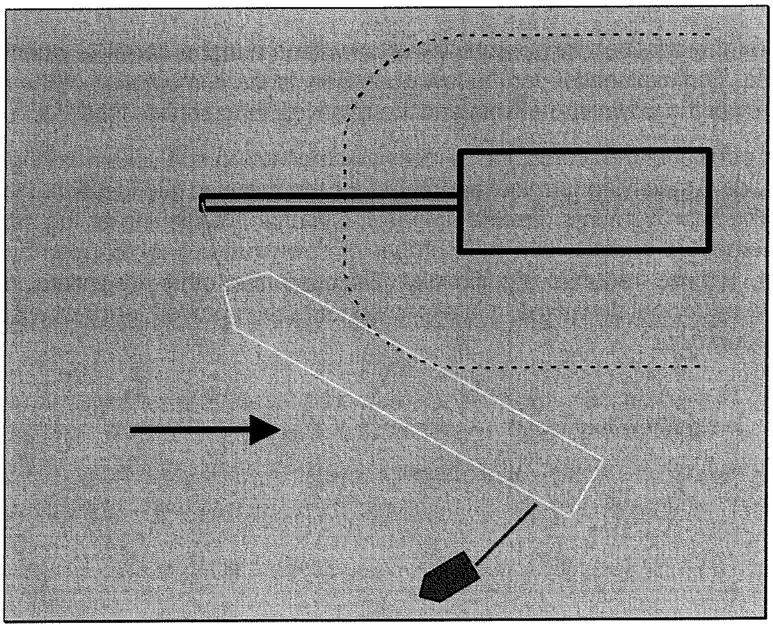


Figure 17-7 Improving terminal operability using assisting tugs

Besides the lack of reliability of the active system, there is another disadvantage when using tugs: In practice, the efficiency of the propulsion of the tug boats becomes too low when manoeuvring in waves higher than 1.5 – 2.0 m. This would mean that one of the main advantages of the soft yoke system in the first place, operating during harsh weather conditions (i.e. waves higher than 3 m), will be nullified.

Unfortunately, the operability improvement, achieved by each of the three solutions mentioned above, cannot be described quantitatively, because the simulation model and the downtime analysis would have to be enhanced to incorporate the additional aspects. A study which implements these enhancements should be carried out in a next stage.

17.6 Conclusions

From the downtime assessment carried out in this chapter, the following conclusions can be drawn:

- The optimal orientation for a terminal with minimal downtime is 105 degrees (0 degrees being north).
- Because of the relatively wide range of mean vessel headings (and yoke angles), a rather long jetty is required to provide sufficient distance between vessel and terminal.
- Regarding the NDBC dataset downtime events are mainly caused by the heading of the vessel being within the sector occupied by the terminal. Maximum vessel motions or mooring forces are seldom exceeded.
- A terminal orientated under 105 degrees with a jetty length of 180 meters has the following operability characteristics:

Probability of downtime	7.1%
Operability	62%
Average waiting time	11.1 hours
Max. no. of downtime events in a row	168 hours
Probability of gas send-out interruption	26%

Table 17-2 Terminal operability characteristics

- Based on the relatively low probability of downtime a higher terminal operability may be expected. The explanation for the low operability is the combination of the relatively high frequency of the downtime events and the relatively long service time.
- Without additional measures, the partially weathervaning soft yoke mooring system does not provide a satisfying terminal operability for the selected site location. However, when one of the improvements (flexible dolphins, Dynamic Positioning or tug assistance) can be implemented successfully, the number of downtime events, caused by the distance criterion, will be reduced significantly. Because the other downtime criteria hardly generate additional downtime, it is expected that the improved terminal will have a much higher operability.

18 Conclusions and recommendations

18.1 General

In this final chapter conclusions and recommendations are given. First the conclusions from the site selection study are discussed in paragraph 18.2. Then conclusions and recommendations are given for each major component of the offshore LNG import terminal, i.e. the LNG storage tank (18.2), layout of the process equipment (18.4) and the mooring configuration (18.5) respectively. Finally in section 18.6 the overall conclusions are drawn.

Recommendations are written in *Italics*.

18.2 Site location

- Based on a preliminary site selection study, the optimal location for an offshore LNG import terminal within the vicinity of Boston, Massachusetts is approximately five kilometres offshore from Gurnet Point, located at the north-west side of Cape Cod Bay.

It is recommended to carry out a more comprehensive assessment, including a thorough analysis of environmental consequences of the terminal, to confirm the optimal location of the site.

18.3 LNG storage tank

- One single LNG storage tank is preferred above several smaller tanks with the same total volume because of rapidly increasing costs for LNG containment material.
- One cylindrical tank of 200,000 m^3 is not feasible due to limitations in height and span length.
- The optimal design for a fixed offshore LNG storage facility of 200,000 m^3 including regasification equipment is a single gravity based structure constructed in pre-stressed, reinforced concrete accommodating a prismatic membrane LNG storage tank. Its outer dimensions are 162 meters long, 72 meters wide and 38 meters high. The structure can be constructed in a purpose-built graving dock after which it can be towed to the site location, where it is sand-ballasted onto the seabed.
- Compared to breakwater-type GBS studies the presented design for the LNG storage tank shows significant reduction on material costs (membrane and concrete) per cubic meter storage.
- While optimising the design of the LNG storage tank, increasing the height is the most cost-effective way to gain additional storage volume. The resistance against sliding (on-bottom stability) and the draft of the floating caisson during towage are the decisive limitations during optimisation.
- Due to increased loads of waves and buoyancy with increased water depth, the minimum water depth is the optimal depth.
- Increasing the angle of internal friction of the subsoil and decreasing the significant wave height at the site location results in reduced construction costs of the optimal solution.

It should be investigated whether the reduction of construction costs achieved by improving the subsoil exceeds the additional costs for carrying out this improvement.

It is recommended to investigate the impact of the requirements for maintenance and inspection of the LNG containment system on the structural design of the caisson as well as on the choice of the type of containment system.

The effects of scouring of the caisson foundation and/or the occurrence of uneven settlements have not been incorporated in this study. Additional research on this subject is recommended.

It is recommended to investigate the consequences of the possibility of ship collision on the design of the concrete caisson and the membrane LNG containment system.

A more detailed structural analysis of the concrete caisson is required to determine the effects of reinforcement details, pre-stressing and temperature gradients on the optimal caisson dimensions and construction costs.

18.4 Process equipment layout

- It is considered technically feasible to fit the process and regasification equipment required for an LNG import terminal with a throughput of 5 mtpa and a peak send-out rate of 1650 m^3/hour on an offshore platform of 150 by 70 meters.

Because the conventional design philosophy for the layout of process equipment in the form of a safety distances matrix has not been applied for the offshore terminal, it is recommended to carry out a detailed quantitative risk assessment to identify the possible safety hazards.

The impact of aspects like maintenance access, modular construction and future expansion on the layout of the process deck, should be investigated more thoroughly in a next stage.

It is recommended to investigate the possibility of the offshore terminal extracting power from the power grid onshore.

It is advised to carry out a detailed assessment of the sea water temperature drop generated by the open rack vaporisers. Local environmental considerations should determine whether additional measurements are required.

18.5 Mooring configuration

- The soft-yoke mooring system with limited weathervaning capability applied for the selected site location near Boston results in a terminal operability of 62% with an average waiting time of 11.1 hours.
- Most of the downtime events are caused by exceedance of the weathervaning limitation. Maximum vessel motions or mooring forces are seldom exceeded.
- When either flexible dolphins, a DP system or tug assistance are incorporated in the mooring configuration, the number of downtime events, caused by the distance criterion, will be reduced significantly. Because the other downtime criteria hardly generate additional downtime, it is expected that in that case terminal will have a much higher operability.

It is recommended to validate the TERMSIM results of the simulation of the LNG carrier moored at the soft-yoke mooring system with a different simulation model.

A detailed, theoretical moored ship analysis is recommended to gain more insight the behaviour of the LNG carrier moored at the soft-yoke mooring system.

The assumed values for the critical limits to vessel motions and mooring forces have to be verified in a next stage.

It is recommended to carry out model simulations including flexible fender dolphins, dynamic positioning systems or tug assistance to determine their effect on terminal operability.

18.6 Overall

The proposed offshore LNG import terminal with a throughput of 5 mtpa comprising of:

- a reinforced concrete GBS with 200,000 m^3 storage capacity,
- regasification equipment for 1650 m3/hour peak send-out installed on top,
- connected with a 180 m long jetty to a jacket-based soft-yoke with limited weathervaning capability,

has a terminal operability of 62%, an average waiting time of 11.1 hours and 26% chance of interruption of gas send-out (buffer under-run) when regarding the environmental conditions at the selected site location near Boston.

Compared to other offshore LNG import terminal concepts, there is a potential saving on material costs with respect to the LNG storage facility. However these savings may be nullified, or even changed into additional expenses, by the increased costs for the jetty and the complex soft-yoke mooring system.

An expensive mooring system may be justified when it results in high terminal operability. For the considered site location this is not the case, unless additional improvements, such as flexible fender dolphins, DP systems or tug assistance, are incorporated. Therefore, without such improvements, the suggested terminal concept is considered not to be a cost-effective solution for Boston.

It is recommended to investigate the operability of an offshore LNG import terminal featuring a fixed storage/regasification facility and an improved soft-yoke mooring system for other site locations with different environmental conditions.

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Design of an
Offshore LNG Import Terminal

APPENDICES

MSc. thesis

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A LNG Import market

A.1 International forecasts

Across the globe, for the short term, there is sufficient natural gas to meet heating and manufacturing needs for the world's major energy markets. However, the world will soon experience supply tightening and even shortages, as soon as economic growth conditions return to normal. An additional point of consumption will be growing domestic needs for gas in Third World countries. Because of high transport costs, natural gas markets and consumption are still regional. This regional emphasis will begin to change, although not rapidly.

A.2 US Market

The US natural gas industry is extremely large by international standards. Consumption in 1997 was 473 Mtpa, which is more than double consumption of Western Europe. The domestic production was also large, about 408 Mtpa, while virtually all the rest was imported from Canada by pipeline. Just 1.9 Mtpa was imported in the form of LNG.

There is a significant potential for LNG to expand in the US, of which the market is demanding more gas for "clean" power generation. Although the US still had a very small (6.24 million m^3) import volume in 2000, the growth is spectacular with 37% increase year-on-year. Most of this demand is supplied by Trinidad, Algeria and Qatar. Natural gas now accounts for 25% of primary US energy demand, a share that could approach 30% by 2010.

Given the huge size of the US natural gas market, there could be considerable opportunities for LNG imports on the US East and Gulf Coasts. The US will need more natural gas from Mexico, and before long will need to import LNG from South America and Africa.

A.3 Existing US import terminals

There are four existing LNG terminals in the USA, of which two are active in the LNG import trade. These terminals received 1.9 Mtpa in 1998. The LNG terminal at Everett, Massachusetts, owned and operated by Distrigas of Massachusetts Corporation, received 0.9 Mtpa in 1998. Some figures of the other terminals are given in Table A-1.

Location	Owner/ Operator	Design throughput (Mcf/d)	Throughput in 1998		Status
			Mcf/d	Mtpa	
Everett, MA	Distrigas of Mass.	285	180	1.2	Operating
Cove Point, MD	Columbia LNG	1000	800	6.3	Reopening
Elba Island, GA	Southern Energy Co.	430	370	2.9	Reopening
Lake Charles, LA	CMS Energy/ Trunkline	600	540	4.4	Operating

Table A-1 Existing US import terminals

B LNG Carriers

B.1 General

Liquefied Natural Gas is transported at its atmospheric boiling point, approximately –160 C. Specially designed ships are required capable of maintaining their cargo at cryogenic temperatures. Therefore these LNGC's (LNG Carriers) need to be provided with a cargo containment system which will maintain its structural integrity in an extreme low temperature environment. The transfer of external heat from ambient air and seawater to the LNG must be prevented by an insulation system. In addition, the ship must be capable of utilising (as fuel for propulsion) or re-liquefying the small amount of vapour that boils off. The average boil-off rate of an LNG carrier amounts to approximately 0.1 – 0.3 % of the cargo per day.

Capacity (1000 m^3)	Membrane	Self-supporting	Total
100 – 140	23	41	64
60 – 100	9	4	13
20 – 60	5	7	12
0 – 20	1	0	1
Total	38	52	90

Table B-1 LNG Carriers – World fleet (operational in 1995; type and number)

B.2 Typical features

Typically the carrier capacities range from 25,000 – 145,000 m^3, but about 60% of the present fleet has a capacity of 125,000 – 135,000 m^3. LNGC's are characterised by their shallow draft and large freeboard due to the low density of their cargo (LNG has a density of about 0.45 ton/m^3). All ships incorporate a double hull to prevent the cargo from spilling in case of a collision. The space between the inner and outer hull is used for ballast water during the return voyage.

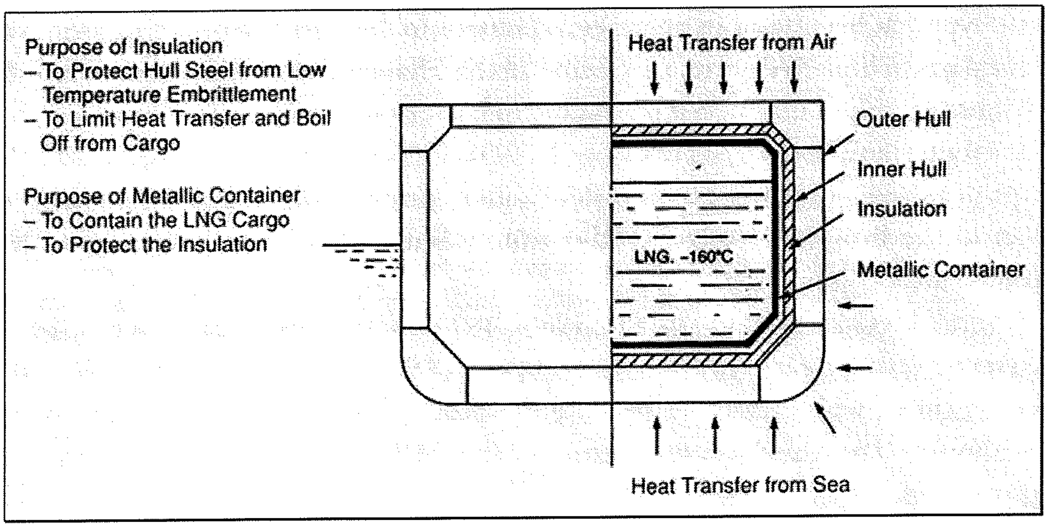

Figure B-1 Principles of LNG ship construction

B.3 Containment systems

It is common to classify the LNG carriers by their containment system. Generally the cargo tanks can grouped as self-supporting (Kvaerner-Moss, IHI-SPB) and membrane (Technigaz, Gaz Transport) systems, which are described in more detail below.

- Kvaerner-Moss

  The Kvaerner-Moss design features spherical aluminium tanks without internal structural support or stiffening. Each tank is supported by a cylindrical skirt that is welded to the inner bottom of the ship. The skirt provides a stainless steel thermal break to reduce the heat transfer from the inner hull to the cargo. The tanks have an internal diameter of about 40 meters. Four or five cargo tanks on a LNGC result in a ship capacity of approximately 135,000 m^3.

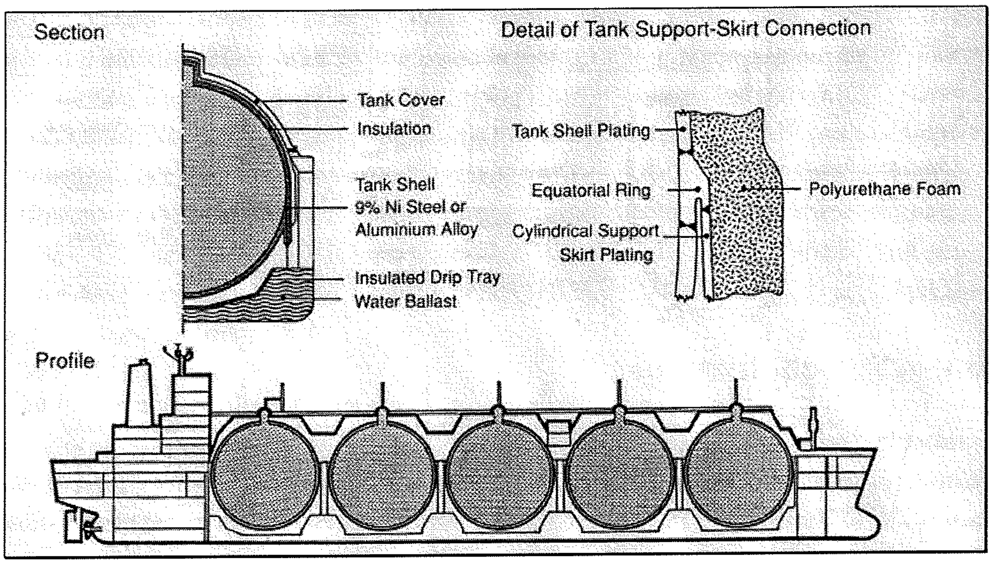

Figure B-2 Kvaerner-Moss containment system

  The outer surface of the tank is covered with 300 mm foam, which is on its turn covered by an aluminium foil as a vapour barrier. Nitrogen between the tank and the insulation prevents moisture from ambient air condensing and freezing on the tank surface. The methane content of the nitrogen is monitored to detect an accidental leakage of the tank.

- IHI-SPB

  Prismatic shaped aluminium alloy tanks are designed to fit the dimensions and efficiently utilise the volume within the ship's hull. The cargo tanks are internally stiffened and divided into liquid-tight compartments to prevent sloshing of the cargo. Foam blocks attached to the outer tank surface provide the necessary insulation. As in the Kvaerner-Moss design, an aluminium foil cover and a monitored nitrogen system are incorporated.

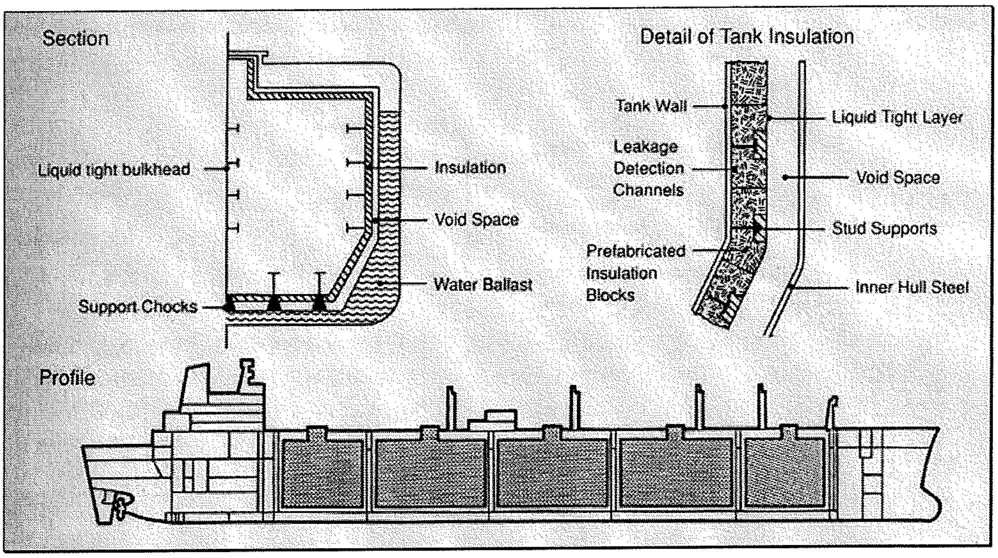

Figure B-3 IHI-SPB Containment system

- Technigaz

  The Technigaz membrane tank consists of 1.2 mm thick stainless steel with longitudinal and transverse corrugations which can absorb mechanical and thermal deflections in two dimensions. The result is a minimum stress in the membrane itself. The load-bearing insulation consists of laminated plywood and rigid polyurethane reinforced with glass fibers. A secondary liquid thight barrier is provided by a composite of aluminium foil and two layers of glass cloth.

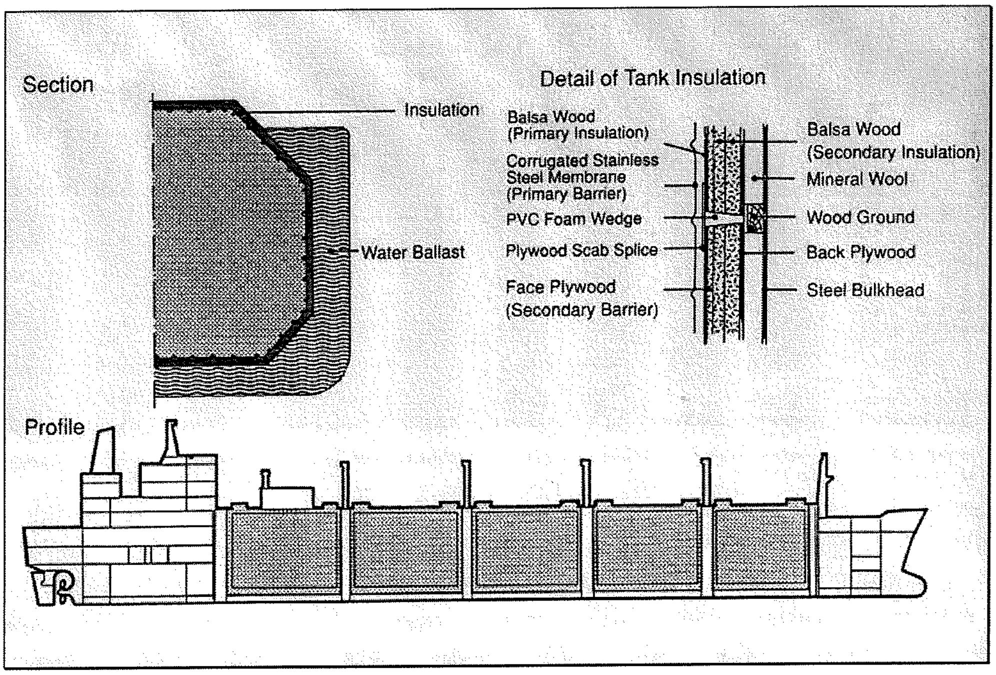

Figure B-4 Technigaz membrane containment system

- Gaz Transport

  The Gaz Transport membrane design uses a 0.7 mm sheet of Invar (36% steel alloy which has a very low thermal expansion coefficient) for both the primary and the secondary barrier. The load bearing insulation consists of plywood boxes filled with perlite, which are fixed to the inner hull.

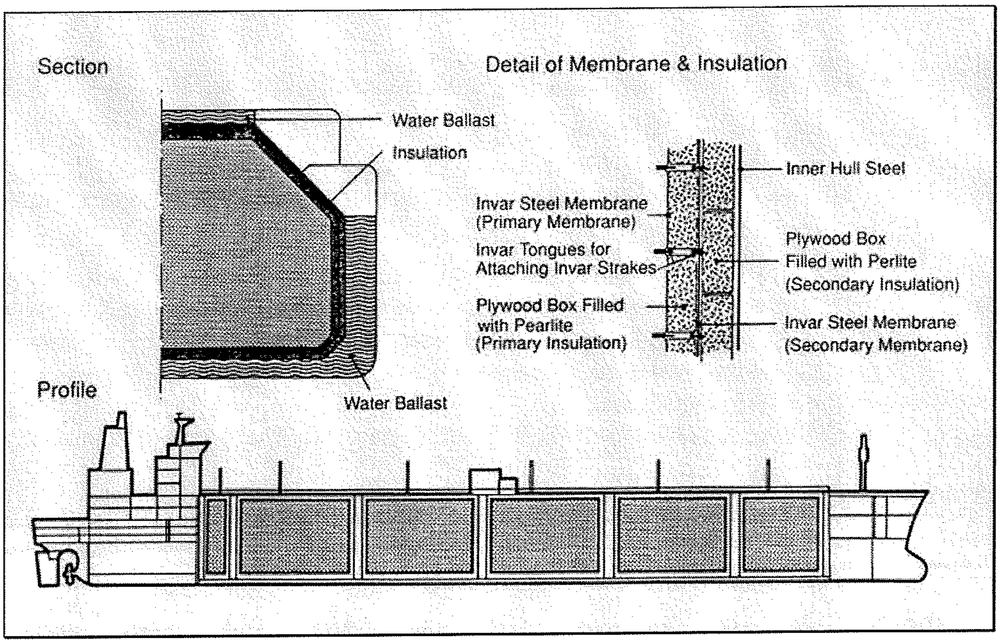

Figure B-5 Gaz Transport membrane containment system

C LNG Receiving terminals

C.1 General

The function of a LNG Import Terminal is to receive liquefied natural gas from LNG carriers, to store it as a buffer, to convert the liquid into gas and to continuously send-out that gas to the customer at the desired pressure levels.

The design of the terminal must accommodate two distinct modes of operation:

- Normal operation, when LNG is pumped from storage, vaporised and distributed to the customers
- Ship unloading mode, during which the cargo of a LNGC is unloaded into terminal storage while maintaining normal vaporisation and send-out rates. 24 hours are typically required for the complete offloading operation.

A typical flow scheme of an LNG receiving terminal is shown in Figure C-1.

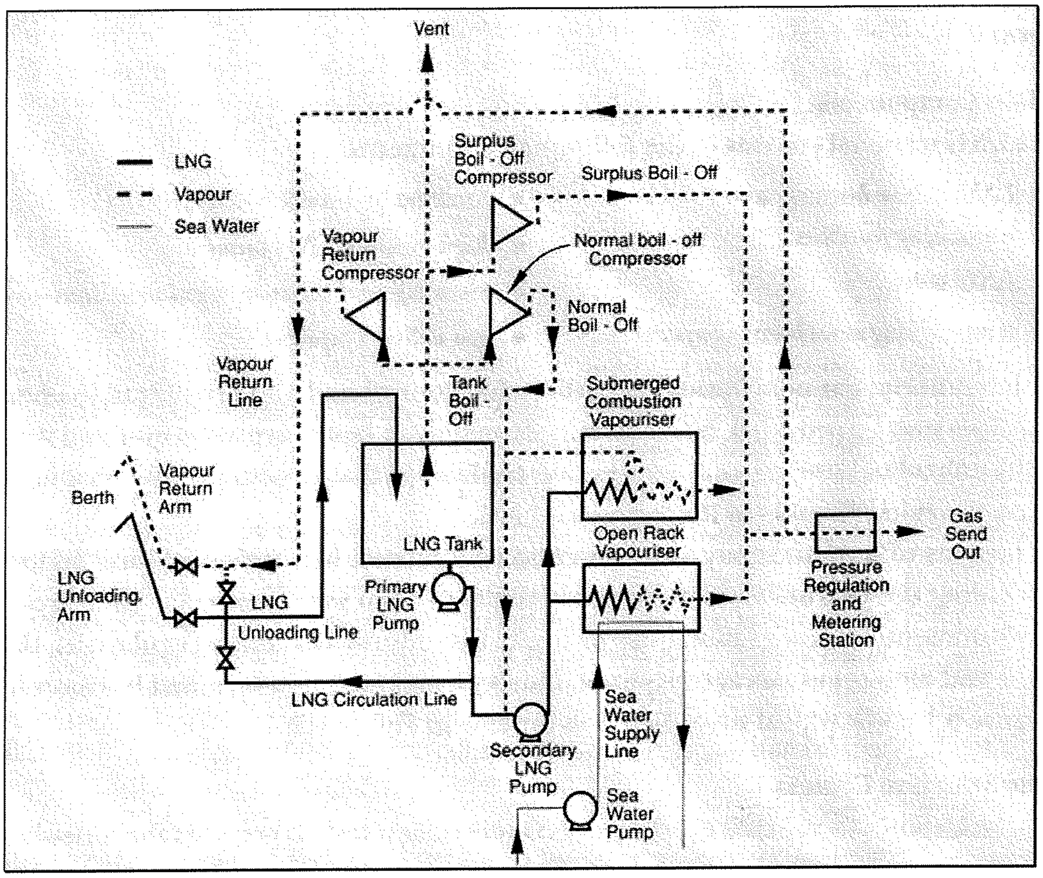

Figure C-1 LNG Import terminal typical flow scheme

Adequate LNG storage must be provided to balance average annual import supply with short-term changes of demand. Vaporisation equipment must have sufficient capacity to meet variable load demand.

When LNG receiving terminals play a critical role in the overall energy supply of an operator or country, extensive equipment redundancy and duplication of facilities are required to assure 100% terminal output except under abnormal circumstances. However, in Europe and the US, LNG plays a minor role in the overall energy picture. Therefore the maximum redundancy, extreme safety measures and excess storage capacity is of less importance.

C.2 Terminal components

Typical components of an onshore LNG receiving terminal are described below.

- Berth or jetty structure

  Typically a berth, dredged to sufficient depth to accommodate the LNG carriers, is located alongside the terminal, or otherwise a jetty is extended to connect a berth to the shore. The length of the jetty should be balanced against the dredging costs. The berth should be secure under prevailing weather and sea conditions and should not be subjected to risk of collision by passing marine traffic. Loading arms, connecting pipelines and supporting facilities are located on the berth. Breasting and mooring dolphins are fitted with quick release hooks and accessible by catwalks (see Figure C-2).

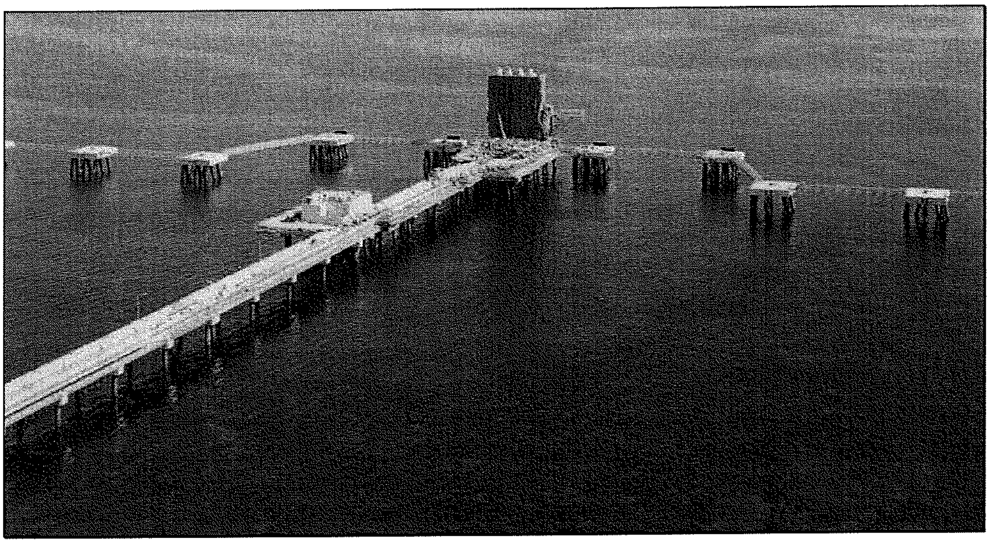

Figure C-2 LNG Unloading jetty

- Unloading system

  Several loading arms are installed to connect with the moored ship's manifold. The loading arms have sufficient swivel joints to deal with ship movements. The LNG is transferred at cryogenic temperature via the loading arms through connecting pipelines to the storage tanks onshore. A vapour return line transports vapour from the terminal into the ship to fill the vacated space. When the unloading process is finished, LNG will still be circulated continuously through the unloading lines to keep them at low temperature. Appendix E will discuss the different loading systems in more detail.

- Storage tanks

  Sufficient storage space for unloaded LNG is provided by one or more cryogenic storage tanks. Typically these tanks are cylindrical concrete or double-walled steel structures provided with insulation to contain the liquid at cryogenic temperature. A more detailed elaboration on LNG tanks is given in Appendix D.

- Regasifying or vaporising system

  A regasification plant vaporising system warms up LNG so that resultant gas is at least 5 degrees C. Three types of vaporisers that are commonly used in LNG import terminals are described below.

The open-rack type of seawater vaporisation is the most common type of LNG vaporiser currently in use. Fuel for heating is not required, but they do need a seawater intake and outflow system that consumes electricity. Because they have little turndown capability, preferably they should be used for continuous services. Another concern of this type is their damage to the local environment in the form of thermal pollution.

A second type is gas-fired direct vaporisers, which are compact and easily shutdown and restarted, but need about 1.5% of their throughput for fuel. The submerged combustion design types are very safe, thermally efficient and well-suited for short-term operations.

A third option is an intermediate fluid vaporiser which transfers heat to the LNG via an intermediate fluid, typically propane or freon. These units offer an alternative method for using seawater to vaporise LNG without the risk of freezing with direct seawater-LNG exchange.

- Boil-Off Gas (BOG) facility

  Continuously heat leaks through insulation of the storage tanks, pipelines and other equipment. As a result, as much as about 1% of the LNG daily boils off as vapour. The vapour can be compressed and added to gas send out or re-mixed with the LNG. Another possibility is to use the gas as fuel to generate electricity for the power requirements of the terminal. As a backup, a terminal is always provided with a vent or a flare to dispose of boil off gas when an equipment failure occurs.

- High pressure LNG pumps

  Because the capital and operating costs of LNG pumps are lower than those of gas compressors, LNG is pumped at high pressure, typically 50 to 80 bar, into the vaporisers, so that the resulting gas needs no further compression. Low pressure LNG circulation pumps are installed inside or close to the LNG storage tanks. LNG delivered by pumps is circulated through the unloading lines and to the high pressure pumps, which raise the pressure to a little above the gas send-out pressure.

- Knock-out (KO) Drums

  A knock-out drum is a device that separates liquid from gas, thus preventing liquid to enter equipment solely designed for gas.

- Metering and pressure regulation station

  Gas before leaving the terminal passes through a pressure regulating and metering station. The quality of the gas will be checked and adjusted if necessary. In most cases it also will be odorised.

- Gas delivery pipeline

  A pipeline transports the gas from the terminal to the distribution grid. Typically transmission pressures can be in the range of 40 to 70 bar, and pipeline diameters from 60 to 120 cm.

- Utility systems

  The utility systems of the terminals consist of the electric power circuit, a seawater supply system (e.g. for the vaporisers), a utility fuel gas system, a fresh water/fire water system and a nitrogen supply.

- Safety systems

  The terminal should have gas detectors, cold detectors, smoke detectors and fire detectors. A fire fighting facility should also include a firewater network across the terminal. Furthermore a process control and safety monitoring facility, and an Emergency Shut Down (ESD) system should be present.

- Buildings

  The terminal should accommodate a control building, an administrative building, a workshop or warehouse, and housing for employees if required.

C.3 Layout

An impression of an LNG receiving terminal layout plan is shown in Figure C-3. Of course capacity requirements and local site conditions will determine the actual layout of a terminal.

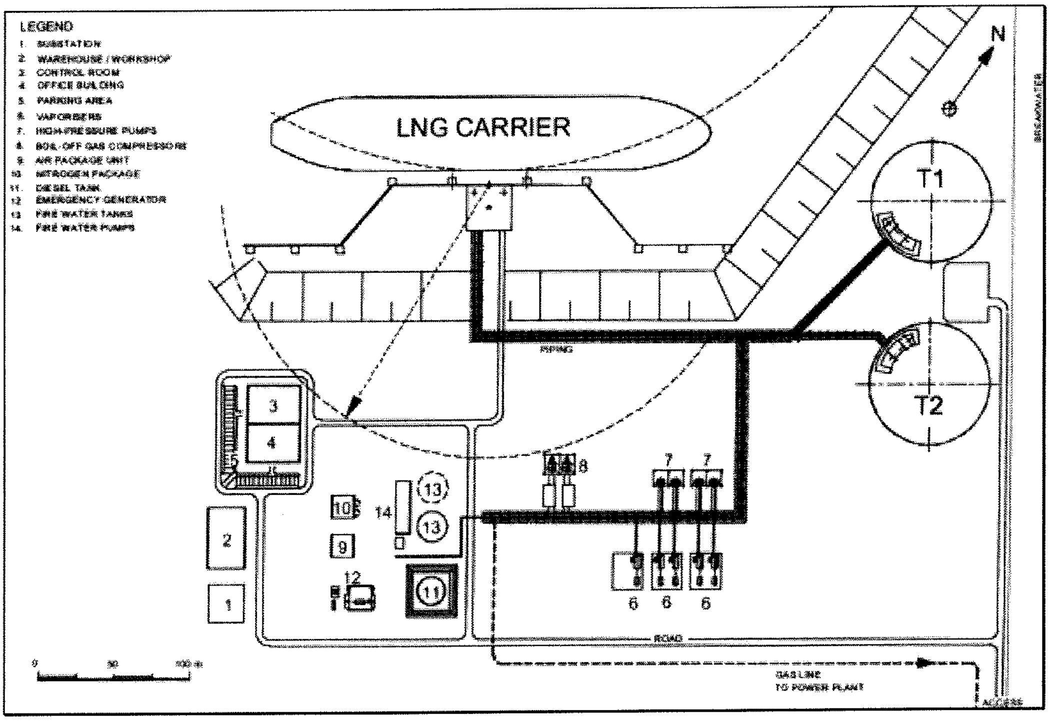

Figure C-3 Typical layout LNG receiving terminal

D Storage tanks

D.1 General

A LNG storage tank must safely perform four main functions:

- The LNG must be contained without any leakage
- The amount of heat entering the tank must be kept as low as possible
- The tank must prevent gas from leaking out and water vapour from leaking in
- The consequential damage resulting from a tank failure must be as small as possible

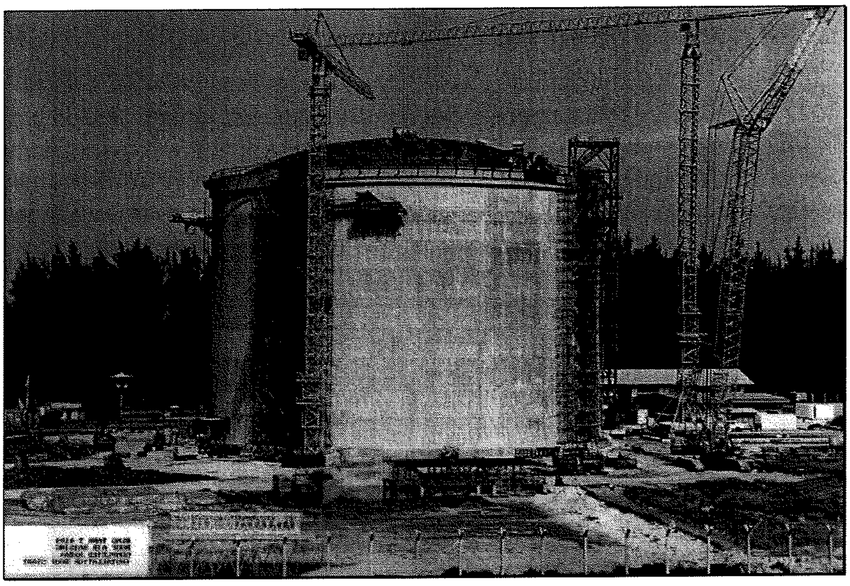

Figure D-1 Construction of concrete LNG storage tank

To fulfil this functions the tank must be deal with all credible events that can occur. Some of the possible causes of failure of an LNG storage tank are given below:

- Unrepaired notch in a plate or a defect in a weld that propagates over time ("Rip-Zip")
- Natural events like hurricanes and seismic activity
- Settlement of the soil below the tank
- Collision with flying objects, blast waves from accidents in adjacent installations
- Thermal radiation resulting from an adjacent fire

D.2 Single containment tanks

Sometime around 1958 the first LNG storage tank, as we know it today, was built. It consisted of a double walled steel outer tank, with a self-supporting, open top inner tank of 9% Nickel steel inside. The tank was founded on a cylindrical shell and covered with a dome roof. The sidewall insulation was provided by granular perlite. Because the LNG was only contained by the inner steel tank, these type of LNG storage tanks are called Single Containment Tanks. Typically the single containment tanks are surrounded by an earthen wall which provides a backup containment system in case of leakage of the inner tank.

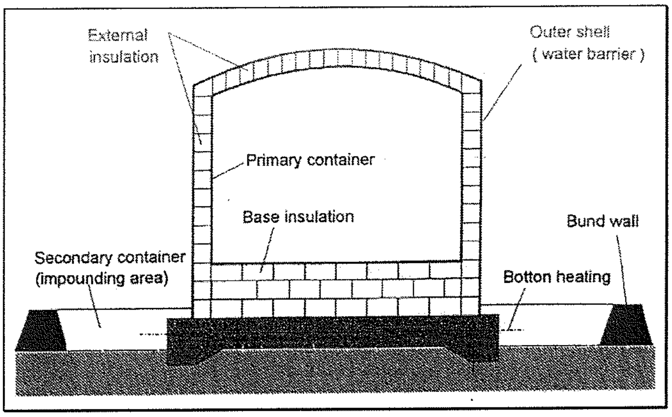

Figure D-2 Typical cross-section of single containment tank

D.3 Double/full containment tanks

Around 1977 Secondary Containment tanks were introduced, which provide a secondary outer tank which also can contain the liquid in case of a leakage. Double Containment Tanks can only contain the liquid while Full Containment Tanks can also contain the vapour resulting from an accidental leakage. A full containment tank is a double wall construction, with a self-supporting primary container surrounded by a concrete secondary container. The outer (secondary) tank consists of a reinforced concrete slab base, a cylindrical wall of pre-stressed concrete, and a reinforced concrete dome roof supported by the cylindrical wall. Again the inner tank is constructed of 9% Nickel steel covered by an aluminium deck. All piping into the tank is routed through the roof to eliminate weak points. The double containment system costs about 45% more than the single containment tank. For the full containment tank this figure goes up to 70%.

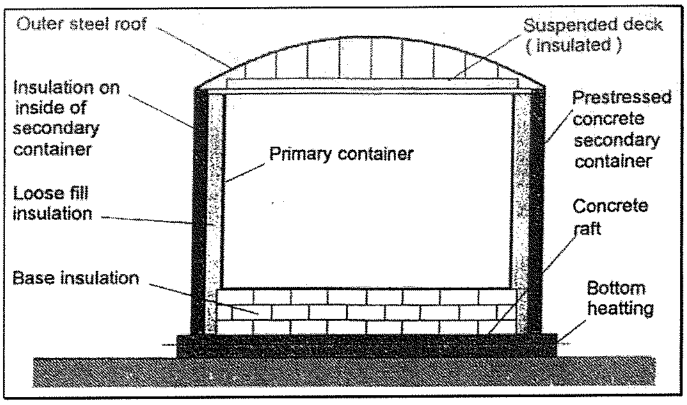

Figure D-3 typical cross-section of full containment tank

D.4 Membrane tanks

An alternative design for LNG tanks is called the Membrane Tank. The structural function is provided by a pre-stressed concrete tank, similar, in every point, to the concrete tank of the full containment storage as described above. The concrete tank constitutes the resisting structure of the storage. The containment function is provided by the membrane made of 1.2 mm thick stainless steel sheets welded together. The membrane incorporates a double network of corrugations allowing free contraction and expansion under thermal solicitations. The membrane transmits the liquid and vapour pressures to the concrete outer tank through the load bearing insulation material. It consists of prefabricated elements made of rigid cellular material with plywood facing. The insulation enables to maintain the concrete of the outer tank in the vicinity of the ambient temperature. The roof is insulated with glass wool laid on a suspended deck.

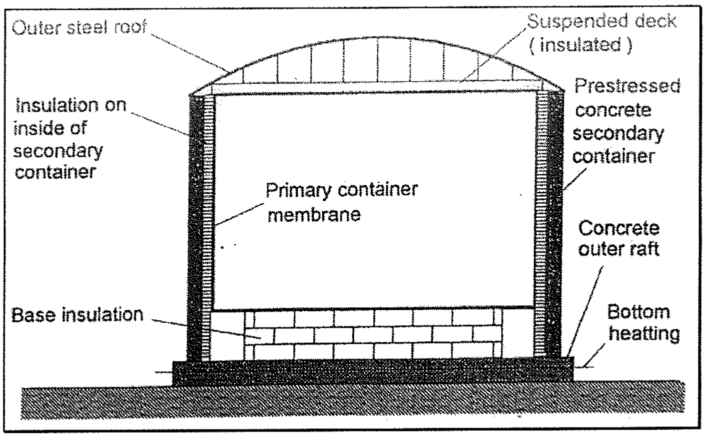

Figure D-4 Typical cross-section of membrane containment tank

D.5 Design aspects

Mostly regional practice dictates the choice of containment system. Whether the spill is contained by an earthen dike or a closed-in concrete wall is the decision of the owner. If there is a small amount of land available the single containment tank will be less appropriate.

It is normally recommended that the tank is built on a concrete slab foundation supported by steel piles that reach into the sound bedrock as most plant areas have soft ground and there is the possibility of soil liquefaction. When a tank is in direct contact with the soil below, a foundation heating system is needed to prevent the water in the soil from freezing, which could damage the foundation. Alternatively a thermal break can be achieved by providing a natural ventilation path between the base slab and the soil.

Obviously the total terminal storage capacity must be sufficient to be able to accept a full ship's cargo of LNG. Furthermore the level of LNG in the tanks must not fall below the limit at which the primary LNG pumps can deliver. There should also be a specific amount of LNG be retained in the tanks to keep the system at low temperature. Finally the storage capacity should have buffer stock to overcome ship downtime and buffer ullage to deal with send-out rates that are lower than expected or to be able to receive a ship that comes in early.

Initially, aboveground storage tanks were of capacity 25,000 – 45,000 m^3, but in recent years this figure has increased to 120,000 – 140,000 m^3. At present the largest membrane tank has a capacity of 180,000 m^3, while the maximum size for a full containment tank design now goes up to 200,000 m^3. It is usually more economical to build one large tank instead of several smaller ones due to reduced materials costs, but also savings in pumps, piping and other equipment.

Typical construction time is about 32 months, and typical costs amount to \$300 - \$350 per m^3 storage.

E Loading systems

E.1 General

LNG is transferred from ship to terminal at a temperature of –162 deg. C. At present, transfer throughout the world is undertaken by the use of hard loading arms. Traditionally in existing shore operations the ship will remain at the berth for about 24 hours, although this may vary according to the metocean conditions. This means generally that the LNG is unloaded at a rate of 10,000 m^3/h which is achieved by the use of two loading arms together with a vapour return arm.

E.2 Traditional loading arms

The traditional loading arm comprises of a system of pipes interconnected by swivel joints to allow the relative movements due to wind, wave, tide, currents and change of cargo weight between the ship and the terminal. A counterbalance system is provided to minimise the load on the manifold to acceptable limits and to reduce the power requirements to manoeuvre the arm into position. An emergency release system (ESD) is employed to allow the emergency disconnection of the loading arm from the ship, should the ship move outside a pre-described operating envelope or if some other form of emergency occurs (see Figure E-1).

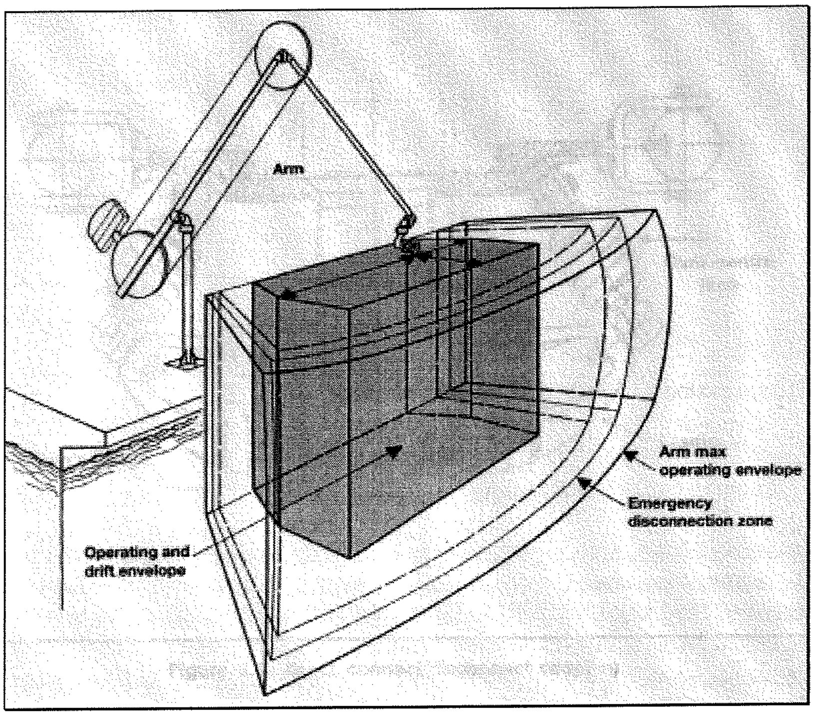

Figure E-1 Loading arm with ESD system

LNG loading arms have been in use throughout the world for a number of years and safety incidents are relatively rare. The use of the traditional LNG loading arms is limited by the physical distance between the loading arm and the loading manifold. Current experience would limit this to between 10 en 15 meters, although construction of greater arms may be possible.

E.3 New developments

The future plans for offshore LNG terminals have led to the development of a new generation of LNG loading systems, capable of compensating for larger carrier movements (surge, heave, sway, roll, pitch and yaw) due to a more exposed location.

Traditionally LNG is unloaded in a side-by-side configuration, where the vessel is moored alongside a fixed quay, jetty or floating structure. However, new offshore concepts often feature single point moorings (SPM) through which the ship is allowed to weathervane around a fixed point. This loading concept is called the tandem-loading configuration. Presently at lot of research is being done on innovative loading concepts for side-by-side as well as tandem loading configurations. The new loading systems should be able to cope with the higher requirements for transferring LNG in an offshore environment. Some companies are working on the development of flexible cryogenic hoses while others focus on enhancing the traditional hard loading arms. An example of the latter is given below.

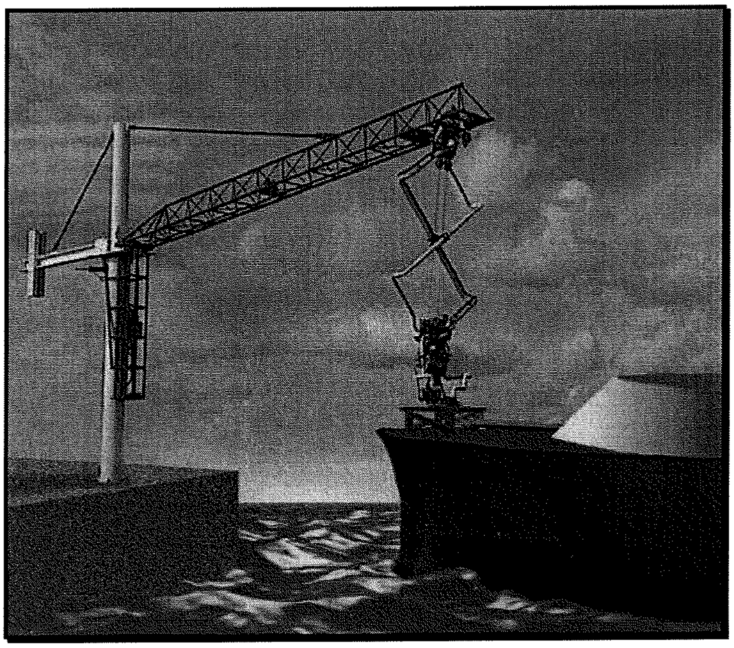

Figure E-2 3D Impression of FMC's BTT loading concept

FMC's BTT (Boom To Tanker) loading arm

The BTT consists of a boom, a double pantograph system and a manifold installed at the bow of the LNG carrier. A 24" diameter line ensures the nominal LNG flow rate of a 10,000 m^3/hour, while the vapour return line is 16" diameter. The double pantograph system together with the jumper assemblies at the bow of the LNG carrier, ensure the six degrees of freedom. The BTT system can operate from an FPSO or SPM in significant wave heights of up to 5.5m following the weathervaning of the LNG carrier for up to 360 degrees.

F Existing offshore import terminal concepts

F.1 General

Recent developments on offshore LNG import terminals focus on various solutions. Below an overview of the most viable concepts has been given. Basically the options can be classified using the following criteria:

- Floating or fixed terminal
- Offshore or onshore storage
- Conventional or dedicated LNGC's required

None have been actually constructed yet, but the LNG industry is, in varying level of detail, studying on these offshore LNG terminals for implementation.

F.2 Dedicated LNG carrier with submerged turret loading (STL) system

This concept consists of a dedicated LNG carrier which hooks up on a submerged turret. The carrier has to be equipped with onboard regasification and gas send-out facilities. The ship is fitted with a receiving cone for a submerged detached turret. It can be moored by pulling in the conical buoy, which is permanently installed at the site. The vessel will be able to weathervane around the turret during the unloading process, which can take place at all times except during a hurricane. An example of the turret is shown in Figure F-1.

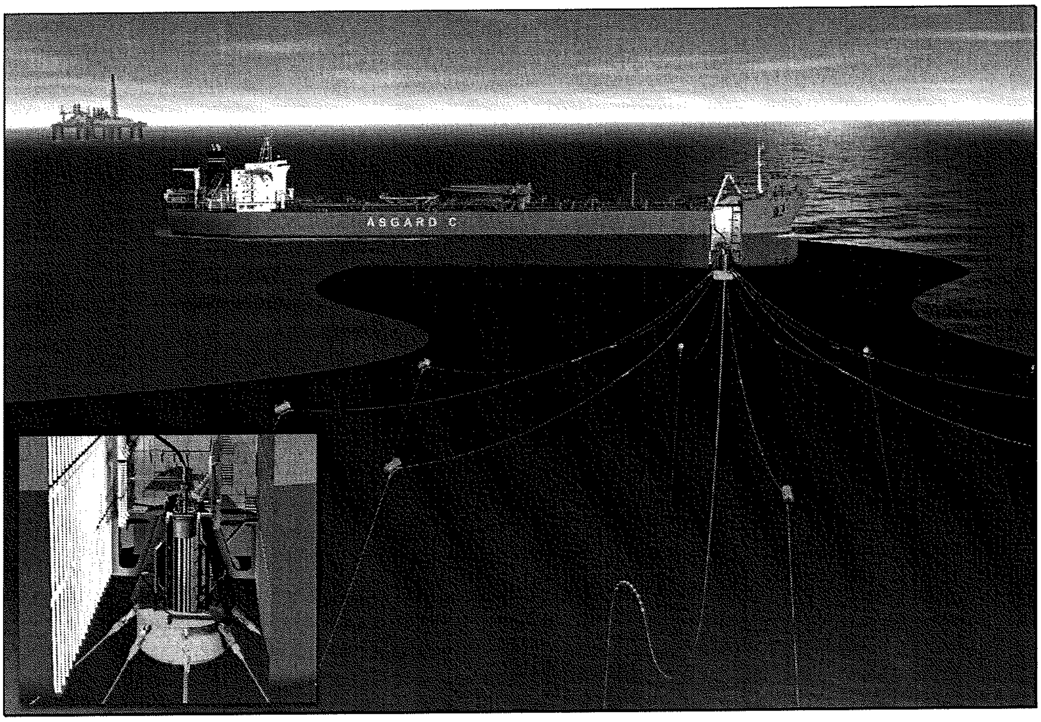

Figure F-1 Submerged Turret Loading concept

Once the turret is locked to the ship the unloading and ESD systems will be connected, and the LNGC can start regasifying its cargo. The gas will be transported via the turret connection through a submerged pipeline to the shore.

An advantage of this offshore solution is that the construction time will be less than for a conventional import terminal. Furthermore the system is able to moor and unload during relatively harsh weather conditions.

The main limitations of the STL system are that the configuration requires a minimal waterdepth of approximately 70 meters, which will probably result in a long and expensive pipeline connection to the shore; also extensive modifications to the LNGC are needed. Due to low capacity of onboard regasification equipment this concept is likely to require multiple vessels unloading at the same time to achieve a substantial terminal throughput.

F.3 Floating storage and regasification unit (FSRU)

The FSRU option consists of a dedicated barge which will be single-point moored to a fixed position. The barge will be able to weathervane around its mooring point and provides onboard storage and regasification facilities. The LNGC will moor alongside the FSRU and unload its cargo using a loading arm capable of absorbing the relative motions between the two floating bodies. The gas will be transferred to the shore through a submerged pipeline. A picture of the floating LNG facility is shown in Figure F-2.

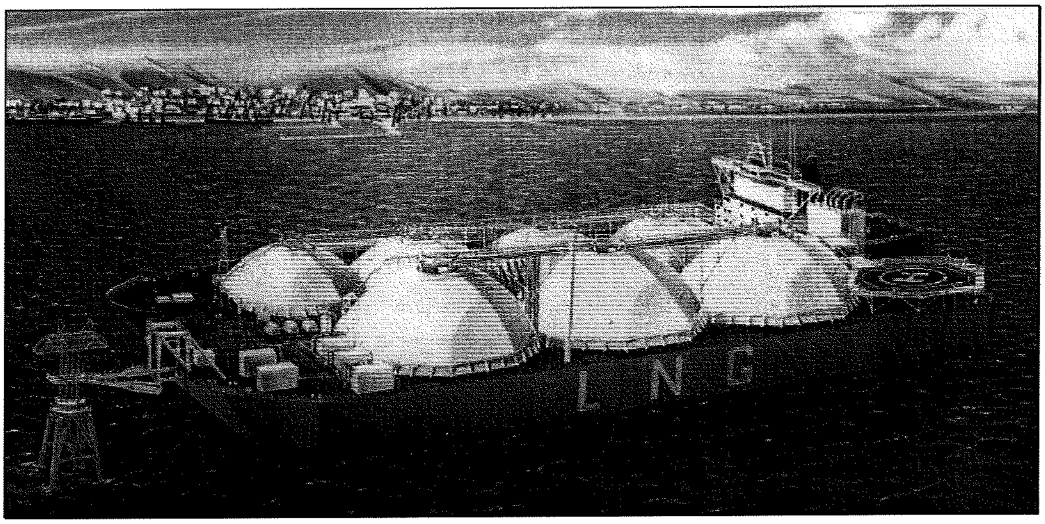

Figure F-2 Floating Storage and Regasification Unit

An advantage of this concept is the fact that the weathervaning ability of this system will reduce the overall downtime. It also has enhanced flexibility because the FSRU can be towed from one site to another without significant modifications.

Disadvantages are the lack of practical experience with the transfer of LNG between two moving bodies and the possibility of sloshing of the LNG in the membrane tanks of the FSRU. Also the weathervaning concept requires a lot of space (minimum circle with a radius of the ship's length).

F.4 Gravity based structure (GBS)

The concrete Gravity Based Structure is a multi-cellar pre-stressed concrete caisson made of high performance concrete. A 3D impression of the GBS is shown in Figure F-3. Typical dimensions of the structure are a width of 60 meters and a length of 400 meters. The main deck level is about 20 meters above the seabed and 5 meters above still water.

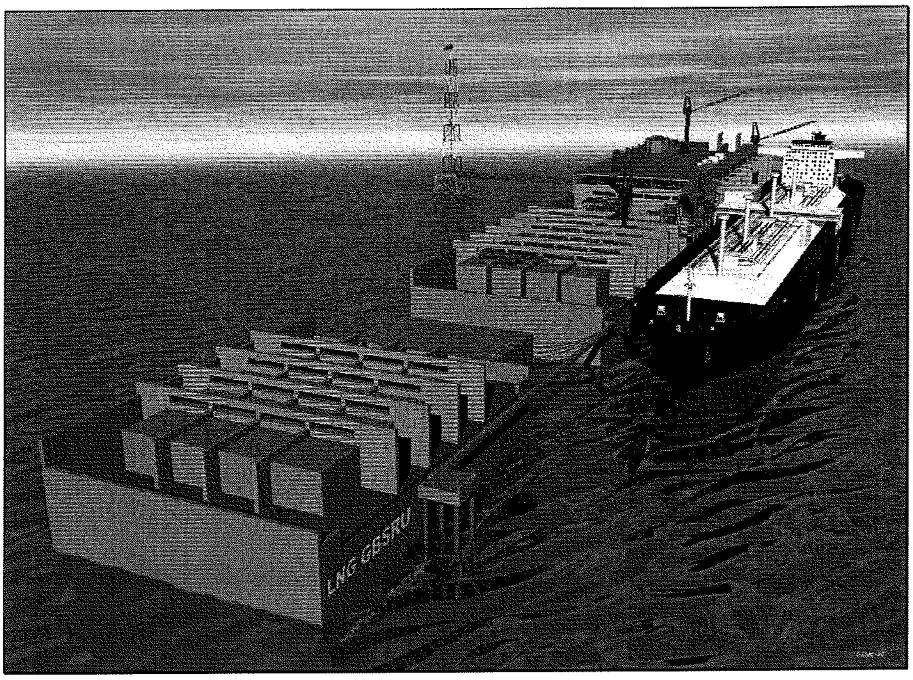

Figure F-3 Gravity Based Structure

This alternative comprises of unloading facilities, prismatic integrated storage tanks, LNG vaporisation and boil-off gas treatment facilities and a natural gas send out. The LNG import terminal receives from standard LNG carriers. Another function of the GBS unit is to provide protection of the moored vessel against excessive waves and current during the unloading operations. The storage and process/utilities units may consist of either one or several elements which can be erected in a dry dock independently. The efficiency of the structure against an open sea environment can be improved by increasing the total length of the construction using a passive breakwater wall.

Prismatic membrane LNG storage tanks are integrated in the concrete caisson and offer space for typically 240,000 m^3 of LNG. The underside of the caisson base is fitted with shallow concrete skirts which provide adequate bearing capacity and sliding resistance for foundation on soft cohesive soil. The caisson base is filled with water and gravel to provide sufficient stability.

The GBS modules are built and equipped onshore in harbour facilities, while the concrete works are performed in a dry dock. Once the GBS is able to float, it will be towed to a quayside where the equipment and storage tanks can be installed. Once completed, each unit is towed floating to the site. Upon arrival, each unit is positioned and water-ballasted until the skirts have penetrated the seabed. Solid ballasting with sand and gravel is then performed to reach the required weight for stability on the seabed under extreme design environmental conditions.

Benefits of this concept are the fact that the terminal structure acts as breakwater providing a sheltered berth and that, because of the fixed structure, conventional side-by-side loading arms can be used. Furthermore, future expansion can easily be achieved by placing extra caissons. Also the pre-stressed concrete has proven to be a perfect material handling cryogenic cargo within a marine environment.

A disadvantage the GBS system is that for larger water depths the costs of the concrete structure will increase significantly. Its application is therefore limited for relatively shallow water depths. However this should not be a problem where the tidal range is not extreme.

G Admiralty Chart

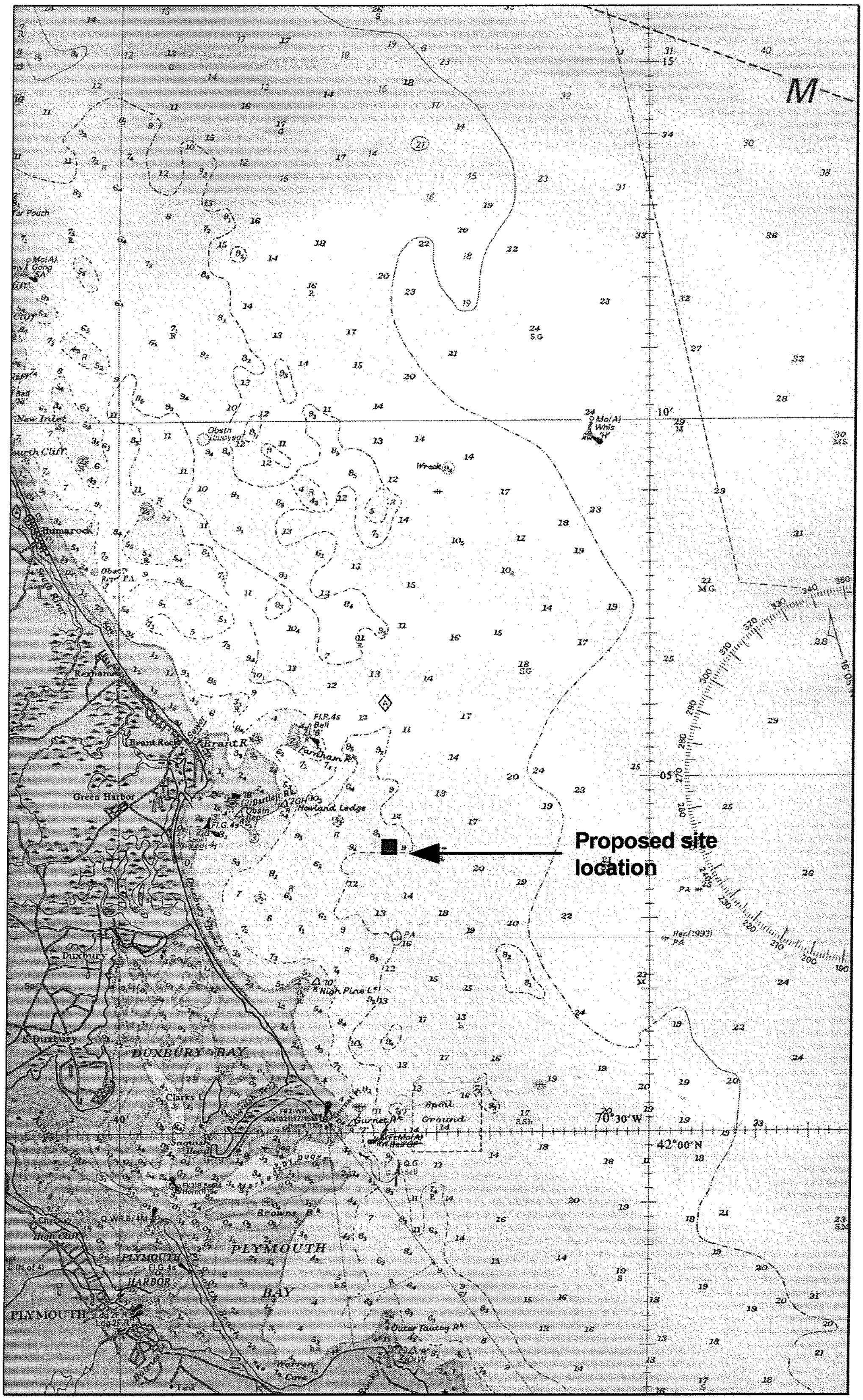

Figure G-1 Admiralty Chart of site (depths in fathoms)

H Wave and wind roses

Wave Rose

Wave parameter: Height of swell (m)
Centre of data area: Location is at 43° 00'N, 69° 30'W
Number of samples: 335 samples from 280 passes

Wave direction "coming from"	0 N	30 NNE	60 ENE	90 E	120 ESE	150 SSE	180 S	210 SSW	240 WSW	270 W	300 WNW	330 NNW	Total
Wave height [m]													
<0.5	0.3	0.3	1.5	3.0	7.2	3.6	3.3	0.0	0.6	0.0	0.0	0.3	20.1
0.5 - 1.0	1.2	2.1	2.1	1.8	5.1	11.6	7.2	2.7	2.1	0.6	0.3	0.6	37.4
1.0 - 1.5	0.6	1.2	0.6	3.6	3.9	2.7	4.8	1.5	0.3	0.6	0.6	0.3	20.7
1.5 - 2.0	0.0	0.0	0.3	0.9	2.4	1.2	3.0	1.2	0.3	0.6	0.0	0.0	9.9
2.0 - 2.5	0.0	0.0	0.0	1.2	1.5	0.9	1.5	0.0	0.3	0.6	0.0	1.2	7.2
>2.5	0.6	0.0	0.0	0.3	1.5	0.3	1.5	0.6	0.0	0.3	0.0	0.0	5.1
TOTAL	2.7	3.6	4.5	10.8	21.6	20.3	21.3	6.0	3.6	2.7	0.9	2.4	100.0

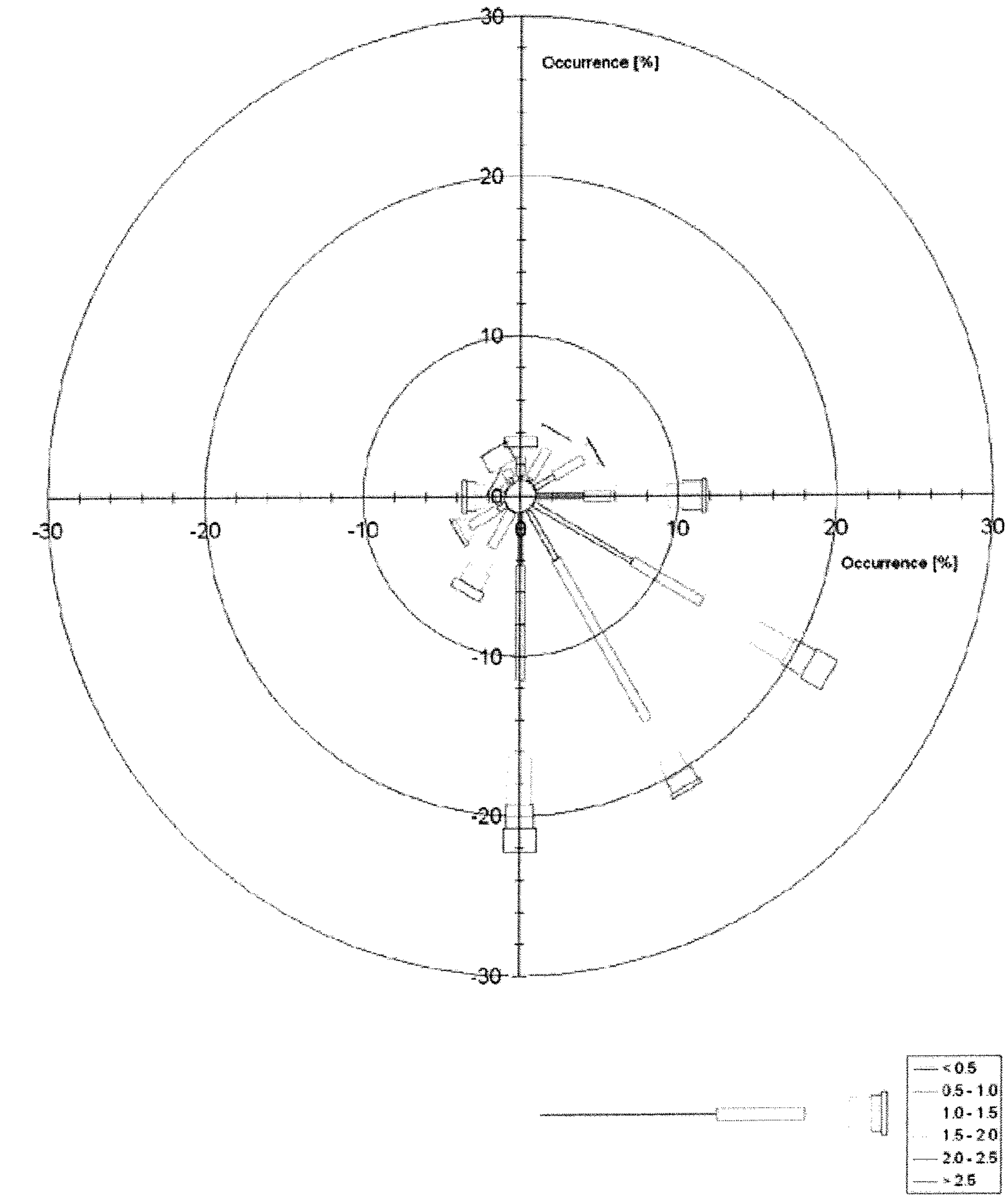

Figure H-1 Wave rose for swell measured in "ARGOSS Area I"

Wave Rose

Wave parameter: Height of wind induced waves (m)
Centre of data area: Location is at 43° 00'N, 69° 30'W
Number of samples: 335 samples from 280 passes

Wave direction "coming from"	0 N	30 NNE	60 ENE	90 E	120 ESE	150 SSE	180 S	210 SSW	240 WSW	270 W	300 WNW	330 NNW	Total
Wave height [m]													
< 0.5	1.8	2.1	2.1	2.1	2.1	1.8	9.0	4.8	1.8	3.0	2.1	1.8	34.5
0.5 - 1.5	2.1	1.5	1.2	1.8	0.9	1.8	2.4	4.5	3.6	3.6	3.0	1.2	27.6
1.5 - 2.5	2.1	0.9	1.8	0.9	1.2	0.0	1.8	1.8	3.0	1.5	2.7	0.6	18.3
2.5 - 3.5	2.1	0.9	0.9	0.9	1.2	1.5	0.0	0.3	0.3	1.2	1.2	0.9	11.4
3.5 - 4.5	0.0	0.6	0.9	0.6	0.6	0.6	1.5	0.0	0.0	0.0	0.6	0.0	5.4
> 4.5	0.3	0.6	0.3	0.0	0.0	0.3	0.3	0.3	0.3	0.3	0.0	0.6	3.3
TOTAL	8.4	6.6	7.2	6.3	6.0	6.0	15.0	11.7	9.0	9.6	9.6	5.1	100.0

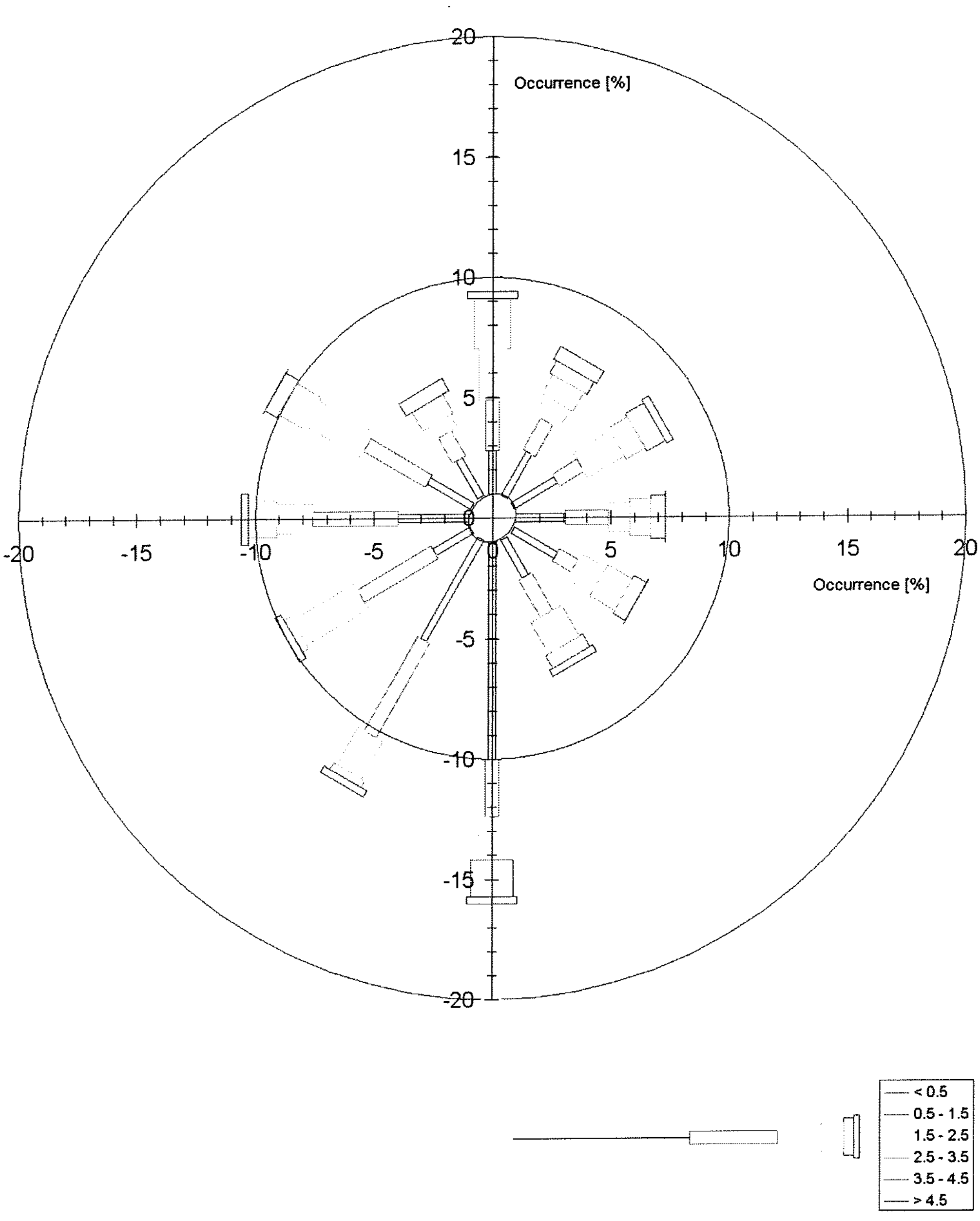

Figure H-2 Wave rose for wind-generated waves measured in "ARGOSS Area I"

Wave Rose

Wave parameter: Significant wave height
Centre of data area: Location is at 43° 00'N, 69° 30'W
Number of samples: 335 samples from 280 passes

Wave direction "coming from"	0 N	30 NNE	60 ENE	90 E	120 ESE	150 SSE	180 S	210 SSW	240 WSW	270 W	300 WNW	330 NNW	Total
Wave height [m]													
< 0.5	0.0	0.3	0.0	0.0	0.9	0.3	0.3	0.0	0.0	0.0	0.0	0.0	1.8
0.5 - 1.0	0.9	1.5	1.2	1.5	1.8	6.0	4.2	3.3	1.2	1.2	0.6	0.0	23.4
1.0 - 2.0	2.4	2.1	1.8	5.7	4.8	4.5	6.9	5.4	3.0	3.3	1.8	1.2	42.9
2.0 - 3.0	2.1	1.2	1.2	2.4	3.0	2.4	3.3	1.2	0.9	1.2	1.8	0.6	21.3
3.0 - 4.0	0.3	0.6	0.3	0.9	0.3	1.2	0.9	0.9	0.3	0.6	0.6	0.9	7.8
> 4.0	0.0	0.6	0.3	0.9	0.3	0.0	0.0	0.3	0.0	0.6	0.0	0.3	3.3
TOTAL	5.7	6.3	4.8	11.4	11.1	14.4	15.6	11.1	5.4	6.9	4.8	3.0	100.0

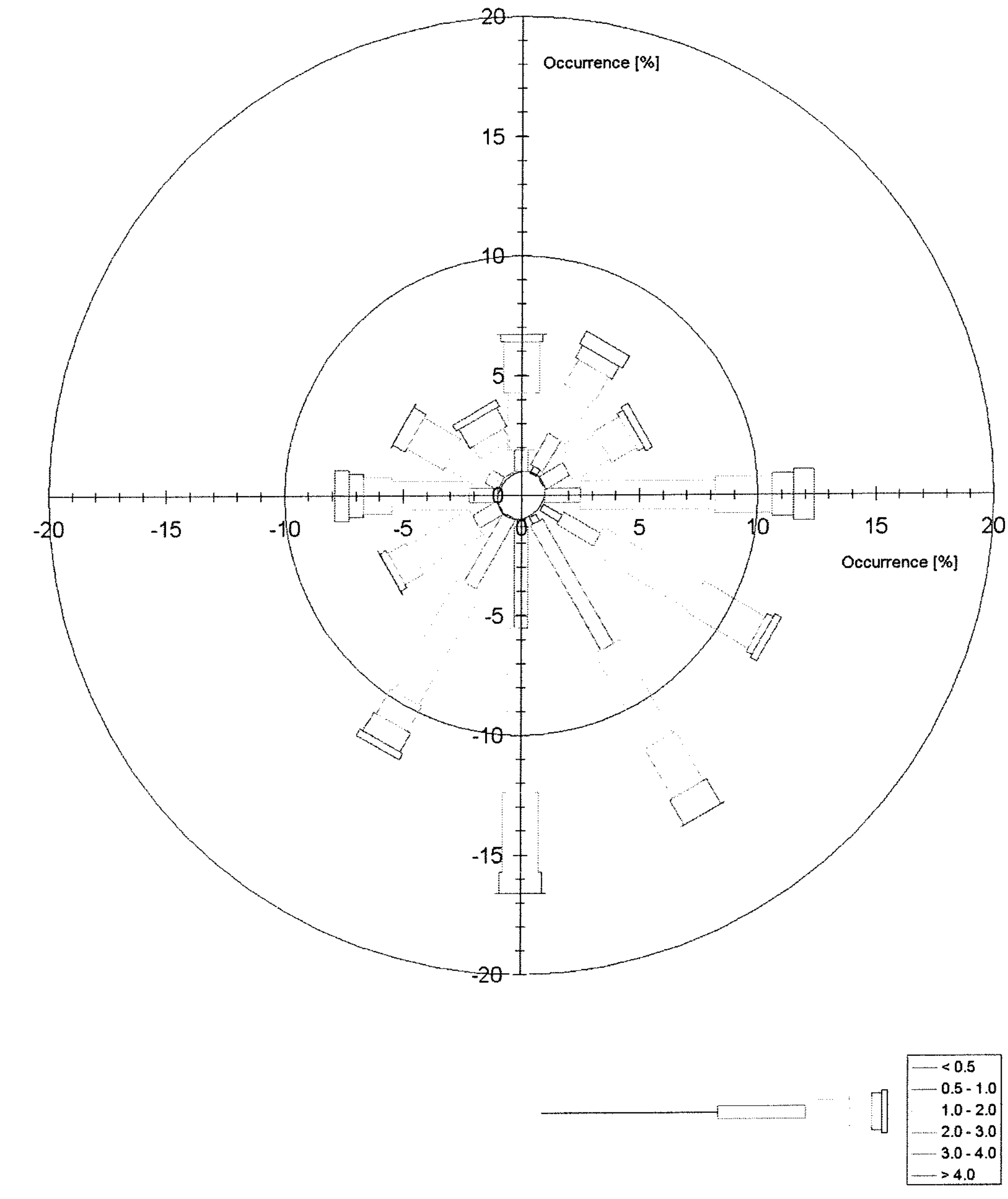

Figure H-3 Wave rose for swell and wind-generated combined, measured in "ARGOSS Area I"

Wind rose

Wind parameter: 10-min sustained wind
Centre of data area: Location is at 43° 00'N, 69° 30'W
Number of samples: 26183 samples of 805 passes

Wind direction "coming from"	0 N	30 NNE	60 ENE	90 E	120 ESE	150 SSE	180 S	210 SSW	240 WSW	270 W	300 WNW	330 NNW	Total
Wind speed [m/s]													
< 2.0	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	9.6
2.0 - 4.0	1.0	1.0	0.9	0.9	1.0	0.9	0.9	1.2	1.4	1.2	1.0	0.9	12.3
4.0 - 6.0	0.9	0.8	0.5	1.0	1.7	1.4	1.0	2.8	2.9	2.0	1.3	1.1	17.4
6.0 - 8.0	2.2	0.7	0.5	0.9	1.1	1.1	2.0	3.3	2.6	2.2	1.3	1.7	19.6
3.0 - 4.0	1.7	1.0	0.5	0.2	0.1	0.7	1.6	2.6	1.9	1.1	1.5	1.8	14.7
> 10	4.2	3.2	1.0	0.8	0.6	0.9	1.5	3.1	1.7	2.5	3.8	3.8	27.1
TOTAL	10.8	7.5	4.2	4.6	5.3	5.8	7.8	13.8	11.3	9.8	9.7	10.1	100.0

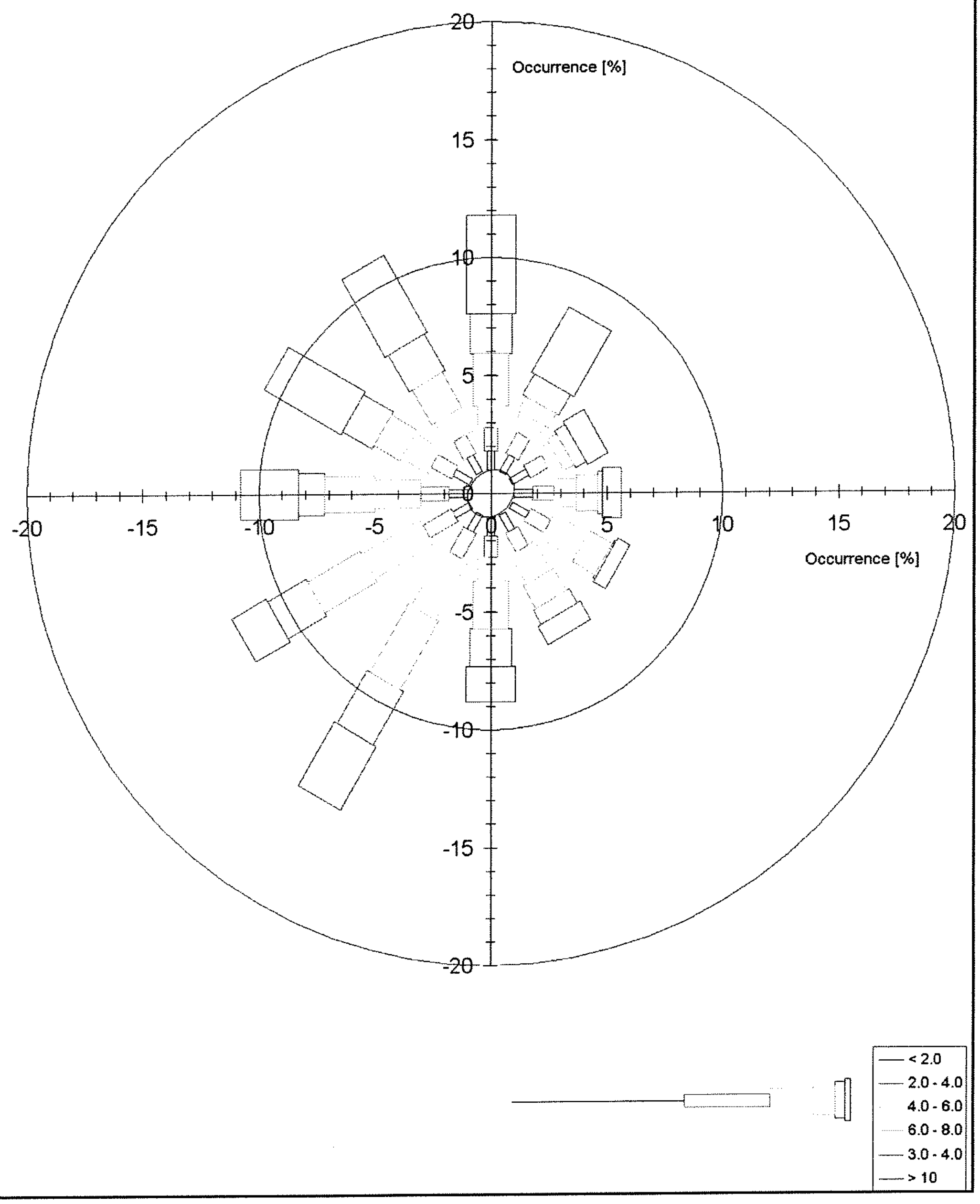

Figure H-4 Wind rose based on measurements in "ARGOSS Area I"

I SWAN input file

```
$************ START-UP ******************************************
 PROJ 'Boston' '01' 'BASIS RUN 1'
 SET  LEVEL 4.2
 SET  MAXERR 1
 MODE STATionary TWODimensional
$*********** MODEL DESCRIPTION ********************************
$ Computational Grid
 CGRID REGular 60000 5000 27.5 80000 120000 160 240 &
       CIRCle  20 0.03 0.3 16
$ Input Grid
 INP  BOTtom REGular 0 0 0 407 359 346 463
 READ BOTtom FAC -1 'boston15a.bot' IDLA 2 NHEDF 0 FREE
 WIND VEL 25 DIR 240
$ Boundary & Initial Conditions
 BOU STATionary SIDE UPP X JONswap GAMMA 3.3 CONstant &
     11.8 PEAK 15.1 240 10
 BOU STATionary SIDE UPP Y JONswap GAMMA 3.3 CONstant &
     11.8 PEAK 15.1 240 10
$ Numerics
 NUM ACCUR 0.02 0.02 0.02 97 0
$*********** OUTPUT *******************************************
$ Output Locations
 POIN 'points' XP 42350  YP 94352 &
          XP 51485  YP 60560 &
          XP 49485  YP 57560
 FRA 'area'  50000 42000 27.5 30000 40000 60 80
$ Write & Plot Commands
 TAB 'points' HEADer 'points.out' XP YP DEP HS TM01 RTP PDIR
 TAB 'area'   HEADer 'area.out' XP YP DEP HS TM01 RTP DIR
 TAB 'COMPGRID' HEADer 'compgrd.out' XP YP DEP HS TM01 RTP DIR
 PLOT 'area' FILE 'areawav.plt' ISO HS 1 0 15
 PLOT 'area' FILE 'areadep.plt' ISO DEP 5 0 100
 PLOT 'area' FILE 'areadir.plt' VEC PDIR 1 1
 PLOT 'COMPGRID' FILE 'cgriddep.plt' ISO DEP 10 0 200
 PLOT 'COMPGRID' FILE 'cgriddir.plt' VEC PDIR 1 1
 PLOT 'COMPGRID' FILE 'cgridwav.plt' ISO HS 1 0 15
$ Main
 COMPUTE
 STOP
```

J Design capacity

The following boundary conditions are given for the capacity of the terminal:

- The annual throughput is 5 million ton of LNG
- The average shipload is 135,000 m^3
- The storage capacity for LNG is 200,000 m^3

When assuming 24 working hours per day, 365 days per year and an LNG density of 0.45 ton/ m^3, the average send-out rate can be calculated:

Annual throughput / (days per year * hours per day) / density of LNG = average send out

5,000,000 / 365 / 24 / 0.45 = 1270 m^3 of LNG per hour

The peak send-out rate is higher than the average rate, which is determined by the swingfactor (relative variation in demand). Applying a swingfactor of 1.3 (130%) the average send-out rate is calculated:

swingfactor * average send-out rate = peak send-out rate

1270 * 1.3 = 1650 m^3/hour

This figure should be the design value for sizing the regasification equipment. The maximum acceptable downtime can be calculated using the following formula:

(total storage volume – one ship load) / (peak send out rate * hours per day) = downtime

(200,000 – 135,000) / (1650 * 24) = 1.6 days

This means that a ship can arrive approximately one and a half days behind schedule before the gas send out will be interrupted (assuming continuous peak demand).

The number of ships per year arriving at the terminal:

5 mtpa / 0.45 ton/m^3 / 135,000 m^3 per ship = 83 ships per year

The inter-arrival time of ships during normal operation will be:

365 days / 83 ships = 4.4 days

The average turn-around time of an LNG ship amounts to 24 hours. The average berth occupancy is computed by:

83 ships/year * 24 hours turn around time / 365 days * 24 hours = 23% occupancy

K Excel model listing

K.1 Input parameters

Parameter	Symbol	Value	Unit
GENERAL			
Sea bottom level at site	d	-15.0	[m above CD]
Lowest low water spring level		0.0	[m above CD]
Mean high water spring level		3.0	[m above CD]
Extreme water level		4.2	[m above CD]
Required freeboard		1.0	[m]
Required underkeel clearance		0.5	[m]
Additional draft due to skirts+air cushion		0.5	[m]
Weight of trim water ballast		2000	[ton]
Slope of seabed	cot(a)	10	[-]
Gravity constant	g	9.81	[N/kg]
WAVES			
Significant wave height at max water level	H_s	7.9	[m]
Significant wave height at min water level	H_s	6.6	[m]
Peak period	T_p	16.2	[s]
Reflection coefficient	K_r	1.75	[-]
Berm reduction coefficient	a	0.75	[-]
Angle of wave incidence	b	0	[deg]
WIND			
Max wind velocity	U_w	24.9	[m/s]
Wind coefficient	C_w	1.0	[-]
Density air	r_{air}	1.23	[kg/m^3]
CURRENT			
Max current velocity	U_c	1.0	[m/s]
Drag coefficient	C_d	2.0	[-]
Density sea water	$r_{seawater}$	1030	[kg/m^3]
SEISMIC			
Seismic coefficient according to UBC	C_a	0.18	[-]
Importance factor according to UBC	I	1.25	[-]
STRUCTURAL MATERIAL			
Concrete quality	C	40	[-]
Reinforcement steel quality	FeB	500	[-]
Density pre-stressed reinforced concrete	$r_{concrete}$	2650	[kg/m^3]
Density reinforcement steel	r_{steel}	7850	[kg/m^3]
Acceptable amount of reinforcement		300	[kg/m^3]

Percentage longitudinal reinforcement		50	[%]
Diameter shear links	Æ	12	[mm]
Centre to centre distance	s	200	[mm]
Amount of prestressing		20	[kg/m³]
LNG			
Density LNG	r_{LNG}	480	[kg/m³]
Unusable height in tank		1.7	[m]
Thickness of insulation	t_{ins}	0.5	[m]
BALLAST			
Density ballast (wet)	$r_{ballast}$	2000	[kg/m³]
Angle of internal friction	$f_{ballast}$	30	[deg]
Percentage of compartments filled		90%	[-]
SUPERIMPOSED LOADS			
Superimposed deck load		10	[kN/m²]
Topsides weight	$W_{topsides}$	10,000	[ton]
SUBSOIL			
Density sub-soil	r_{soil}	2000	[kg/m³]
Angle of internal friction	f_{soil}	30	[deg]
Cohesion	c	0	[kN/m²]
Soil pressure next to foundation	q	0	[kN/m²]

K.2 Optimisation sheet

		Actual	Min Safety	Factor	
	Sliding	1.50	1.50	1.00	[-]
	Overturning	4.51	1.50	3.01	[-]
	Middle third	2.00	1.00	2.00	[-]
	Bearing soil	2.89	2.00	1.45	[-]
		W0 factor	t factor		
	Wall beam	1.01	1.83		
	Floor rib	**1.00**	1.35		
	Outer bottom slab	2.61	**1.00**		
	Inner bottom slab	6.75	2.71		
	Outer wall plate	2.39	1.00		
	Inner wall plate	2.55	1.00		
			min	max	
IN	Inner tank length	148.6	100.0	300.0	[m]
	Inner tank width	57.7	10.0	60.0	[m]
	Inner tank height	26.6	10.0	100.0	[m]

	Outer wall thickness	0.64	0.45	2	[m]
	Inner wall thickness	0.71	0.45	2	[m]
	Outer bottom slab thickness	0.61	0.45	2	[m]
	Inner bottom slab thickness	0.45	0.45	2	[m]
	Compartment wall thickness	0.45	0.45	2	[m]
	No. of compartments over length	26	1.0	50.0	[-]
	No. of compartments over width	10	1.0	50.0	[-]
	Width of wall compartments	5.5	4.0	15.0	[m]
	Height of bottom compartments	7.2	4.0	15.0	[m]
	Length of skirts	4.5	0.0	4.5	[m]
OUT	Length of bottom compartments	5.3	4.0	15.0	[m]
	Width of bottom compartments	5.4	4.0	15.0	[m]
	Outer length	162.3	10.0	350.0	[m]
	Outer width	71.5	10.0	350.0	[m]
	Outer height	38.1	29.5	75.0	[m]
	Net tank volume	200,000	200,000		[m3]
	Draft of floating caisson	14.5		14.5	[m]
	Draft safety factor	**1.00**			
	Concrete volume	60,447			[m3]
	Ballast volume	134,099			[m3]
	Estimated total costs	84.1			[mln USD]
	Cost per m3 storage	420			[USD]

K.3 Calculation of dimensions

Net tank volume	Vin;net	200,000	[m³]
Inner volume	Vin	228,074	[m³]
Inner width of tank	Bin	57.7	[m]
Inner length of tank	Lin	148.6	[m]
Inner height of tank	Hin	26.6	[m]
Wall surface inner tank	Aw	10,969	[m²]

Bottom surface inner tank	Ab	8,582	[m²]
Roof surface inner tank	Ar	8,582	[m²]
# of compartments over width	nw	10	[-]
Width of ballast compartment	Bcomp	5.4	[m]
# of compartments over length	nl	26	[-]
Length of ballast compartment	Lcomp	5.3	[m]
# of compartments over height	nh	1	[-]
Height of ballast compartment	Hcomp	29.4	[m]
Height of roof construction	Hr	3.3	[m]
Thickness outer wall	tw;out	0.64	[m]
Thickness compartment wall	tw;comp	0.45	[m]
Thickness of ballast compartment	tcomp	5.51	[m]
Thickness inner wall	tw;in	0.71	[m]
Thickness of outer bottom slab	tb;out	0.61	[m]
Thickness of bottom compartments	tb;comp	7.15	[m]
Thickness of inner bottom slab	tb;in	0.45	[m]
Compartment roof slab thickness		0.45	[m]
Outer width of total structure	Bout	71.5	[m]
Outer length of total structure	Lout	162.3	[m]
Outer height of total structure	Hout	38.1	[m]
Ballast compartment volume	Vballast	134,099	[m³]
Concrete volume	Vconcrete	60,447	[m³]
Ballast weight	Wballast	241,377	[ton]
Concrete weight	Wconcrete	160,185	[ton]
Total weight [empty]	Wtotal	172,185	[ton]
Total weight [empty w.o. skirts]		167,271	[ton]
Floating draft concrete caisson	d	14.5	[m]
Estimated amount of reinforcement	Vreinforc.	18,134	[ton]
Estimated amount of prestressing	Vprestress.	1,209	[ton]

K.4 Calculation of loads and caisson stability

LOAD COMBINATIONS		I	II	III	IV	
Maximum water depth	dmax	19.2	19.2	15	15	[m]
Significant wave height	Hs	7.9	0.2	6.6	0.2	[m]
Current velocity	uc	1.0	0.1	1.0	0.1	[m/s]
Wind velocity	Vw	24.9	1.0	24.9	1.0	[m/s]
LNG Tank filled		0%	0%	100%	100%	[-]
WAVES						
Significant wave height	Hs	7.9	0.2	6.6	0.2	[m]
Peak period	Tp	16.2	16.2	16.2	16.2	[s]
Maximum wave height	Hmax	14.2	0.4	11.9	0.4	[m]
Deep water wave length	L0	409	409	409	409	[m]
Shallow water wave length	L	219	219	195	195	[m]
Wave number	k	0.029	0.029	0.032	0.032	[-]
Reflection coefficient	K	1.75	1.75	1.75	1.75	[-]
Freeboard		1.0	1.0	1.0	1.0	[m]
Minimum height of caisson	hmin	29.5	20.4	23.8	16.2	[m]
CURRENT						
Density sea water	rho-w	1030	1030	1030	1030	[kg/m^3]
Current velocity	uc	1.0	0.1	1.0	0.1	[m/s]
Drag coefficient	Cd	2.0	2.0	2.0	2.0	[-]
Area affected	A	19.2	19.2	15	15	[m^2/m]
Current force	Fc	194	0	152	0	[kN/m]
WIND						
Density air	rho-a	1.23	1.23	1.23	1.23	[kg/m^3]
Wind coefficient	Cw	1	1	1	1	[-]
Wind velocity	Vw	24.9	1.0	24.9	1.0	[m/s]
Area affected	A	18.9	18.9	23.1	23.1	[m^2/m]
Wind force	Fw	71	0	86	0	[kN/m]
EARTHQUAKE						
Seismic coefficient	Ca	0.18	0.18	0.18	0.18	[-]
Importance factor	I	1.25	1.25	1.25	1.25	[-]
Seismic weight	Ws	325,663	325,663	325,663	325,663	[kN]
Design base shear	Vb	51292	51292	51292	51292	[kN]
Shear per meter length		316	316	316	316	[kN/m]
Shear per meter width		718	718	718	718	[kN/m]

GODA						
Water depth in front of bw	h	19.2	19.2	15	15	[m]
Slope 1:	cot alpha	10	10	10	10	[-]
Water depth at 5xHs	hb	23.15	19.3	18.3	15.1	[m]
Water depth above armor	d	19.2	19.2	15	15	[m]
Water depth at bottom of caisson	h'	19.2	19.2	15	15	[m]
Depth to toe of skirts		23.6994849	23.6994849	19.4994849	19.4994849	[m]
alpha 1	a1	0.94	0.94	0.97	0.97	[-]
alpha 2	a2	0.031	0.000	0.038	0.000	[-]
alpha 3	a3	0.87	0.87	0.89	0.89	[-]
angle of wave attack	beta	0	0	0	0	[deg]
	eta-ster	21.3	0.5	17.8	0.5	[m]
p1	p1	139	3	121	4	[kN/m²]
p2	p2	121	3	108	3	[kN/m²]
p3	p3	121	3	108	3	[kN/m²]
H1		1158	28	810	24	[kN/m]
y1		6.4	6.4	5.0	5.0	[m]
H2		1338	33	906	26	[kN/m]
y2		12.8	12.8	10.0	10.0	[m]
Freeboard to SWL		18.9	18.9	23.1	23.1	[m]
p4		16.1	0.0	0.0	0.0	[kN/m²]
H3		1315	1	1077	1	[kN/m]
y3		25.5	19.4	20.9	15.2	[m]
H4		152	0	0	0	[kN/m]
y4		31.8	19.6	26.9	15.4	[m]
Hwind		71	0	86	0	[kN/m]
ywind		28.6	28.6	26.5	26.5	[m]
Hcurrent		194	0	152	0	[kN/m]
ycurrent		9.6	9.6	7.5	7.5	[m]
U		4309	106	3857	113	[kN/m]
xv		47.6	47.6	47.6	47.6	[m]
W		14179	14179	23013	23013	[kN/m]
xw		35.7	35.7	35.7	35.7	[m]
SARPKAYA						
Hydrodynamic regime factor	D/L	0.74	0.74	0.83	0.83	[-]
	K	0.55	0.01	0.51	0.02	[-]
	ka	2.3	2.3	2.6	2.6	[-]
Wave diffraction coefficient	Cm	0.6	0.6			[-]
Maximum run-up coefficient	Rm/H	1.81	1.81	2.07	2.07	[-]
	Figure	0.93	0.93			[-]

Maximum force	F	2,937	74			[kN/m]
	f(kd)	0.01	0.01			[-]
Goda maximum force		3810	62			[kN/m]
Relative difference with goda		-23%	20%			[-]
Maximum moment	M	27,086	686			[kNm/m]
Goda maximum moment		58,042	619			[kNm/m]
Relative difference with goda		-53%	11%			[-]
Weight of caisson	W_c	1,571,413	1,571,413	1,571,413	1,571,413	[kN]
Weight of ballast	W_b	2,367,912	2,367,912	2,367,912	2,367,912	[kN]
Weight of topsides	W_t	98,100	98,100	98,100	98,100	[kN]
Weight of LNG cargo	W_{LNG}	0	0	941,760	941,760	[kN]
Maximum superimposed deck load	S	115,999	115,999	115,999	115,999	[kN]
Buoyancy force	B	2,250,419	2,250,419	1,758,140	1,758,140	[kN]
Hydrodynamic uplift	U	699,506	17,139	626,159	18,264	[kN]
Horizontal force	P	4,227	63	3,031	52	[kN/m]
Dynamic uplift	U	4,309	106	3,857	113	[kN/m]
Total weight submerged	W	14,179	14,179	23,013	23,013	[kN/m]
Resulting vertical force	V	9,869	14,073	19,155	22,900	[kN/m]
Moment due to hor forces	Mp/a	66,752	627	39,089	403	[kNm/m]
Moment due to hydrodyn uplift	Mu/a	205,292	5,030	183,766	5,360	[kNm/m]
Moment due to weight	Mw/a	506,601	506,601	822,249	822,249	[kNm/m]
Resulting moment	Mt/a	234,556	500,944	599,394	816,485	[kNm/m]
Moment due to hor forces	Mp/c	66,752	627	39,089	403	[kNm/m]
Moment due to hydrodyn uplift	Mu/c	51,323	1,258	45,942	1,340	[kNm/m]
Moment due to weight	Mw/c	0	0	0	0	[kNm/m]
Resulting moment	Mt/c	118,076	1,884	85,031	1,744	[kNm/m]
Submerged weight of soil between skirts		2,951	2,951	2,951	2,951	[kN/m]
Force accepted by soil next to caisson		289	289	289	289	[kN/m]
Effective unit weight of soil	g'	9.5	9.5	9.5	9.5	[kN/m^3]
Depth to base level	D_b	4.5	4.5	4.5	4.5	[m]
Total foundation area	A	11,600	11,600	11,600	11,600	[m^2]
Vertical load at mud level	V_{md}	1,105,398	1,787,765	2,612,785	3,220,680	[kN]
Vertical load at base level	V_{bd}	1,602,059	2,284,426	3,109,446	3,717,340	[kN]
Horizontal load at mud level	H_{md}	686,126	10,173	491,999	8,364	[kN]
Angle of internal friction of soil	f	0.52	0.52	0.52	0.52	[rad]
Horizontal passive soil reaction factor	K_p	3.0	3.0	3.0	3.0	[-]

Drained horizontal soil reaction factor	K_{rd}	2.7	2.7	2.7	2.7	[-]
Soil cohesion intercept	c'	0.0	0.0	0.0	0.0	[kN/m²]
Characteristic angle of internal friction	f'	0.52	0.52	0.52	0.52	[rad]
Characteristic soil attraction	a	0.00	0.00	0.00	0.00	[kN/m²]
Embedded vert area of foundation	A_h	730.4	730.4	730.4	730.4	[m²]
Hor. Resist. force between mudline and base	DH	41,696	41,696	41,696	41,696	[kN]
Horizontal load at base level	H_{bd}	644,430	0	450,302	0	[kN]
Moment arm of DH above base level	d_q	1.5	1.5	1.5	1.5	[m]
M by vert shear betw mudline and base level	DM_t	0	0	0	0	[kNm]
M ass. with hor and vert loads at mud level	M_{md}	19,166,891	305,903	13,802,828	283,019	[kNm]
M ass. with hor and vert loads at base level	M_{bd}	22,191,570	289,141	15,954,031	258,117	[kNm]
Eccentricity of the resultant vertical load	e	14	0	5	0	[m]
Effective foundation width	B'	44	71	61	71	[m]
Effective foundation area	A'	7,103	11,559	9,934	11,577	[m²]
Design horizontal capacity	Q_{hd}	966,645	1,360,610	1,836,936	2,187,904	[kN]
Friction angle factor for overburden pressure	N_q	18.40	18.40	18.40	18.40	[-]
Inclination factor for overburden pressure	i_q	0.33	1.00	0.69	1.00	[-]
Shape factor for overburden pressure	s_q	1.04	1.22	1.13	1.22	[-]
Depth factor for overburden pressure	d_q	1.03	1.02	1.02	1.02	[-]
Bearing correction factor for overburden pressure	K_q	0.35	1.24	0.79	1.24	[-]
Friction angle factor for soil	N_g	15.07	15.07	15.07	15.07	[-]
Inclination factor for soil	i_g	0.19	1.00	0.59	1.00	[-]
Shape factor for soil	s_g	0.98	0.82	0.91	0.82	[-]
Depth factor for soil	d_g	1.00	1.00	1.00	1.00	[-]
Bearing correction factor for soil	K_g	0.19	0.82	0.53	0.82	[-]
Effective overburden pressure at base level	p_0'	43	43	43	43	[kN/m²]
Design unit drained bearing capacity	q_d	864	5,188	2,968	5,193	[kN/m²]
Minimum soil pressure		-23	195	153	319	[kN/m²]
Maximum soil pressure		299	199	384	322	[kN/m²]
Sliding criterion	g_s	1.5	13,606,099.8	4.1	21,879,036.5	[-]
Overturning criterion	g_o	4.5	800.0	16.3	2024.6	[-]
Middle third criterion	g_m	2.0	3.0	2.6	3.0	[-]
Bearing capacity criterion	g_b	2.9	26.1	7.7	16.1	[-]
FLOATING STABILITY						
Draft of caisson		14.5	14.5	14.5	14.5	[m]
Center of buoyancy	zb	7.3	7.3	7.3	7.3	[m]
Freeboard floating		23.6	23.6	23.6	23.6	[m]
Volume of skirts	Vskirt	1854.2	1854.2	1854.2	1854.2	[m³]

	zskirt	12.3	12.3	12.3	12.3	[m]
Volume of outer bottom slab	Vobs	7,032	7,032	7,032	7,032	[m^3]
	zobs	9.7	9.7	9.7	9.7	[m]
Volume of bottom compartments	Vbc	14,313	14,313	14,313	14,313	[m^3]
	zbc	5.8	5.8	5.8	5.8	[m]
Volume of inner bottom slab	Vibs	5,220	5,220	5,220	5,220	[m^3]
	zibs	2.0	2.0	2.0	2.0	[m]
Volume of walls	Vw	21,067	21,067	21,067	21,067	[m^3]
	zw	-11.5	-11.5	-11.5	-11.5	[m]
Volume of deck slab	Vds	10,962	10,962	10,962	10,962	[m^3]
	zds	-26.4	-26.4	-26.4	-26.4	[m]
Total volume	Vtotal	60,447	60,447	60,447	60,447	[m^3]
Center of gravity	zg	-6.1	-6.1	-6.1	-6.1	[m]
Distance between B and M	BM	29.3	29.3	29.3	29.3	[m]
	zm	-22.1	-22.1	-22.1	-22.1	[m]
Distance between G and M	GM	16.0	16.0	16.0	16.0	[m]

L Example of TERMSIM output file

```
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                  SPM terminal
                  General data  and  vessel particulars

                   Review of input


     Project    : BOSTON

     Outputfile : bat40275

                    Case         Description
     Environment :  0300         output
     Vessel      :  0300
     Mooring     :  0300
     Control     :  0300
     Output      :  0300         environment


     Type of vessel                      : Gas carrier
     Type of tanks                       : Spherical tanks
     Length between perpendiculars       :     276.15       [m]
     Moulded breadth                     :      41.15       [m]
     Displaced moulded volume            :   96361.00    [m**3]
     Draft                               :      11.00       [m]
     Projected side  area above waterline :   7555.00    [m**2]
     Projected front area above waterline :   1545.00    [m**2]
     Loading condition in % of max. draft :    100.00       [%]
     Water depth                         :      15.00       [m]
     Additional damping for SURGE        :     600.00    [kNs/m]
     Additional damping for SWAY         :    3000.00    [kNs/m]
     Additional damping for YAW          : **********[kNms/rad]


     User supplied hydrodynamic datafile  : bajal4m2.hyd


     Height of COG above keel            :      14.30       [m]
     Transverse radius of inertia        :      16.46       [m]

     Centre Of Gravity (Fwd of Station 10):      0.00       [m]


S C A L E D   V E S S E L   P A R T I C U L A R S

SCALING FACTOR                      =       1.00
WATER DEPTH (0 = INFINITE)          =      14.00 M
VESSEL DRAFT                        =      11.00 M
Z-COORDINATE OF M.S.L.  (WRT ORIGIN) =     -3.30 M
Z-COORDINATE OF KEEL    (WRT ORIGIN) =    -14.30 M
VESSEL DISPLACEMENT                 =   96361.00 M**3
VESSEL MASS                         =   98770.02 T
VESSEL WEIGHT                       =     968.93 MN
WATER PLANE AREA                    =    9324.06 M**2
X-COORDINATE OF C.O.F.  (WRT COG)   =      -6.68 M
X-COORDINATE OF C.O.B.  (WRT COG)   =       0.00 M
C.O.G. ABOVE KEEL            KG     =      14.30 M
TRANSV. METAC. ABOVE KEEL    KMT    =      17.80 M
LONGIT. METAC. ABOVE KEEL    KML    =     445.60 M
TRANSV. METAC. HEIGHT        GMT    =       3.50 M
LONGIT. METAC. HEIGHT        GML    =     431.30 M
LIST ANGLE                          =       0.00 DEGREES
TRIM ANGLE                          =       0.00 DEGREES

H Y D R O S T A T I C   R E S T O R I N G   M A T R I X   (KN-M-S, WRT C.O.G.)

------------------------------------------------------------------------------
MODE:    S U R G E     S W A Y  H E A V E     R O L L  P I T C H        Y A W
------------------------------------------------------------------------------
SURGE           0.0         0.0        0.0          0.0          0.0       0.0
SWAY            0.0         0.0        0.0          0.0          0.0       0.0
HEAVE           0.0         0.0    93755.8          0.0     625822.1       0.0
ROLL            0.0         0.0        0.0    3391327.3          0.0       0.0
PITCH           0.0         0.0   625822.1          0.0  417902720.0       0.0
YAW             0.0         0.0        0.0          0.0          0.0       0.0
------------------------------------------------------------------------------
    Method to calculate wave drift forces: Double Fourier Transform
```

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Environmental conditions

Review of input (Continued)

Water depth	:	15.00	[m]
Type of wave spectrum	: JONSWAP		
Significant wave height	:	2.00	[m]
Mean wave period	:	10.00	[s]
Gamma	:	3.30	[-]
Wave direction	:	90.00	[deg]
Type of swell spectrum	: No swell		
Wind velocity	:	20.00	[m/s]
Wind direction	:	180.00	[deg]
Type of gust spectrum	: Harris-DNV		
Current velocity	:	0.50	[m/s]
Current direction	:	0.00	[deg]
Initial seed for random waves	:	7654327	[-]
Initial seed for random wind gust	:	345679074	[-]

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Environmental conditions

Review of input (Continued)

Mean wave drift force in starting condition

Direction [deg]	X-mode [kN]	Y-mode [kN]	N-mode [kNm]
90.00	-21.70	925.93	1671.08
100.00	-57.48	786.63	2123.38
110.00	-79.21	657.09	6051.15
120.00	-86.89	537.29	13454.41
130.00	-73.87	488.38	9123.61
140.00	-64.75	402.03	5735.79
150.00	-59.53	278.26	3290.95
160.00	-50.84	152.79	2806.34
170.00	-45.68	76.40	1403.17
180.00	-40.53	0.00	0.00
190.00	-45.68	-76.40	-1403.17
200.00	-50.84	-152.79	-2806.34
210.00	-59.53	-278.26	-3290.95
220.00	-64.75	-402.03	-5735.79
230.00	-73.87	-488.38	-9123.61
240.00	-86.89	-537.29	-13454.41
250.00	-79.21	-657.09	-6051.15
260.00	-57.48	-786.63	-2123.38
270.00	-21.70	-925.93	-1671.08
280.00	39.68	-791.29	3093.59
290.00	79.68	-654.82	6459.67
300.00	98.28	-516.51	8427.18
310.00	83.62	-452.37	5062.43
320.00	75.60	-357.25	3535.13
330.00	74.24	-231.16	3845.26
340.00	68.92	-128.55	3697.04
350.00	64.61	-51.49	2415.29
360.00	61.32	0.00	0.00
10.00	64.61	51.49	-2415.29
20.00	68.92	128.55	-3697.04
30.00	74.24	231.16	-3845.26
40.00	75.60	357.25	-3535.13
50.00	83.62	452.37	-5062.43
60.00	98.28	516.51	-8427.18
70.00	79.68	654.82	-6459.67
80.00	39.68	791.29	-3093.59
90.00	-21.70	925.93	1671.08

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Environmental conditions

Review of input (Continued)

Zero frequency wave drift force spectrum in starting condition

Direction [deg]	X-mode [kN^2.s]	Y-mode [kN^2.s]	N-mode [kNm^2.s]
90.00	0.2111E+04	0.3577E+07	0.1162E+08
100.00	0.1590E+05	0.2521E+07	0.3633E+08
110.00	0.2851E+05	0.1683E+07	0.3782E+09
120.00	0.3993E+05	0.1064E+07	0.1037E+10
130.00	0.2798E+05	0.9287E+06	0.5694E+09
140.00	0.2019E+05	0.6736E+06	0.2546E+09
150.00	0.1658E+05	0.2984E+06	0.9274E+08
160.00	0.1257E+05	0.8719E+05	0.4034E+08
170.00	0.1059E+05	0.4359E+05	0.2017E+08
180.00	0.8601E+04	0.5249E-07	0.1701E-03
190.00	0.1059E+05	0.4359E+05	0.2017E+08
200.00	0.1257E+05	0.8719E+05	0.4034E+08
210.00	0.1658E+05	0.2984E+06	0.9274E+08
220.00	0.2019E+05	0.6736E+06	0.2546E+09
230.00	0.2798E+05	0.9287E+06	0.5694E+09
240.00	0.3993E+05	0.1064E+07	0.1037E+10
250.00	0.2851E+05	0.1683E+07	0.3782E+09
260.00	0.1590E+05	0.2521E+07	0.3633E+08
270.00	0.2111E+04	0.3577E+07	0.1162E+08
280.00	0.1568E+05	0.2516E+07	0.9228E+08
290.00	0.3429E+05	0.1658E+07	0.2343E+09
300.00	0.5796E+05	0.1003E+07	0.4376E+09
310.00	0.3836E+05	0.8120E+06	0.2030E+09
320.00	0.2690E+05	0.5476E+06	0.8013E+08
330.00	0.2358E+05	0.2097E+06	0.6909E+08
340.00	0.2031E+05	0.8590E+05	0.5897E+08
350.00	0.1781E+05	0.1599E+05	0.3594E+08
360.00	0.1608E+05	0.2119E-07	0.1013E-03
10.00	0.1781E+05	0.1599E+05	0.3594E+08
20.00	0.2031E+05	0.8590E+05	0.5897E+08
30.00	0.2358E+05	0.2097E+06	0.6909E+08
40.00	0.2690E+05	0.5476E+06	0.8013E+08
50.00	0.3836E+05	0.8120E+06	0.2030E+09
60.00	0.5796E+05	0.1003E+07	0.4376E+09
70.00	0.3429E+05	0.1658E+07	0.2343E+09
80.00	0.1568E+05	0.2516E+07	0.9228E+08
90.00	0.2111E+04	0.3577E+07	0.1162E+08

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Environmental conditions

Review of input (Continued)

Mean wind force in starting condition

Direction [deg]	X-mode [kN]	Y-mode [kN]	N-mode [kNm]
180.00	-385.01	0.00	-0.01
190.00	-378.65	-221.79	-12158.65
200.00	-356.98	-513.34	-23707.24
210.00	-315.24	-896.46	-33632.88
220.00	-253.23	-1328.28	-40379.03
230.00	-179.67	-1725.65	-42101.59
240.00	-109.78	-2013.23	-37260.73
250.00	-56.76	-2162.73	-25312.46
260.00	-22.89	-2198.38	-7192.12
270.00	3.12	-2168.10	14651.14
280.00	39.61	-2103.61	36731.21
290.00	99.93	-1998.96	55364.12
300.00	182.77	-1822.18	67569.81
310.00	271.57	-1549.67	71716.27
320.00	343.85	-1196.23	67701.80
330.00	384.54	-816.64	56671.01
340.00	394.49	-476.05	40450.27
350.00	388.43	-209.58	20982.64
360.00	383.86	0.00	-0.02
10.00	388.43	209.58	-20982.63
20.00	394.49	476.05	-40450.25
30.00	384.54	816.64	-56671.00
40.00	343.85	1196.23	-67701.79
50.00	271.57	1549.67	-71716.27
60.00	182.77	1822.18	-67569.81
70.00	99.93	1998.96	-55364.13
80.00	39.61	2103.61	-36731.23
90.00	3.12	2168.10	-14651.15
100.00	-22.89	2198.38	7192.17
110.00	-56.76	2162.73	25312.50
120.00	-109.78	2013.23	37260.75
130.00	-179.67	1725.65	42101.60
140.00	-253.23	1328.28	40379.03
150.00	-315.24	896.46	33632.88
160.00	-356.98	513.34	23707.23
170.00	-378.65	221.79	12158.64
180.00	-385.01	0.00	0.00

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                    Environmental  conditions

                 Review of input  (Continued)


         Mean current force in starting condition
         ----------------------------------------

    Direction      X-mode      Y-mode      N-mode
      [deg]         [kN]        [kN]        [kNm]

       0.00        14.15        0.00         0.00
      10.00         9.02      130.88     -9425.43
      20.00        -5.84      261.75    -18850.86
      30.00       -19.99      392.63    -28276.29
      40.00       -17.51      483.01    -27067.99
      50.00        -9.55      573.38    -25859.68
      60.00         2.65      663.76    -24651.38
      70.00         8.31      696.28    -18277.65
      80.00         7.96      728.79    -11903.92
      90.00         3.36      761.31     -5530.19
     100.00        -1.59      732.24      1976.93
     110.00        -2.83      703.16      9484.05
     120.00         1.59      674.09     16991.18
     130.00        19.99      602.06     20658.43
     140.00        29.72      515.51     21876.51
     150.00        29.54      414.46     20645.41
     160.00         6.37      285.44     14991.45
     170.00        -7.01      147.28      8109.65
     180.00       -11.85        0.00         0.00
     190.00        -7.01     -147.28     -8109.65
     200.00         6.37     -285.44    -14991.45
     210.00        29.54     -414.46    -20645.41
     220.00        29.72     -515.51    -21876.51
     230.00        19.99     -602.06    -20658.43
     240.00         1.59     -674.09    -16991.18
     250.00        -2.83     -703.16     -9484.05
     260.00        -1.59     -732.24     -1976.93
     270.00         3.36     -761.31      5530.19
     280.00         7.96     -728.79     11903.92
     290.00         8.31     -696.28     18277.65
     300.00         2.65     -663.76     24651.38
     310.00        -9.55     -573.38     25859.68
     320.00       -17.51     -483.01     27067.99
     330.00       -19.99     -392.63     28276.29
     340.00        -5.84     -261.75     18850.86
     350.00         9.02     -130.88      9425.43
     360.00        14.15        0.00         0.00

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                    Control information

                 Review of input  (Continued)

    Length of simulation                :    1800.00  [sec]
    Post processing option              :          2    [-]

     No break force or time specified for leg number  1
     No break force or time specified for leg number  2
     No break force or time specified for leg number  3
     No break force or time specified for leg number  4
     No break force or time specified for leg number  5
     No break force or time specified for leg number  6

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                      SPM  terminal


                 Review of input  (Continued)



    Astern propulsion                       :    -981.00   [kN]
    X-position of hawser FAIRLEAD           :     143.19    [m]
    Y-position of hawser FAIRLEAD           :       0.00    [m]
    Z-position of hawser FAIRLEAD           :      15.00    [m]

    Type of HAWSER                          : User supplied
    Unstreched (initial) length of HAWSER :      56.60    [m]
    Number of grommets of the HAWSER        :          0
    Diameter of HAWSER                      :       4.00   [cm]
    Breaking strength of HAWSER             :    5952.00   [kN]
    Total breaking strength                 :    5952.00   [kN]
    X-position of reference point           :       0.00    [m]
    Y-position of reference point           :       0.00    [m]
    Z-position of reference point           :       0.00    [m]

    Buoy data
    ---------

    Vertical position of HAWSER attachment
    point                                   :      15.00    [m]
    Vertical position of CHAIN  attachment
    points                                  :       0.00    [m]
    Diameter of buoy                        :       0.00    [m]
    Draft of buoy                           :       0.00    [m]
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T E R M S I M Phase II v2002_0  Date :08/28/02  Time:21:50:57  Page:  9

                        SPM  terminal
                User defined hawser characteristics

                     Review of input  (Continued)

  Excursion     Tension
     [m]          [kN]
    0.000        0.000
    1.700      495.000
    3.400      981.000
    5.100     1467.000
    6.800     1961.000
    8.500     2475.000
   10.200     3020.000
   11.900     3612.000
   13.600     4272.000
   15.300     5034.000
   17.000     5952.000

T E R M S I M Phase II v2002_0  Date :08/28/02  Time:21:50:57  Page: 10

                        SPM  terminal
                User defined hawser characteristics

                     Review of input  (Continued)

 Chain number                          :          1
 Chain from                            : User supplied
 Orientation angle                     :       0.00  [deg]
 Pre-tension                           :    1000.00   [kN]
 Length                                :    1000.00    [m]
 Breaking strength                     :  100000.00   [kN]
 Elasticity                            :  500000.00   [kN]
 Weight in Air                         :       1.25 [kN/m]

 Chain number                          :          2
 Chain from                            : User supplied
 Orientation angle                     :      60.00  [deg]
 Pre-tension                           :    1000.00   [kN]
 Length                                :    1000.00    [m]
 Breaking strength                     :  100000.00   [kN]
 Elasticity                            :  500000.00   [kN]
 Weight in Air                         :       1.25 [kN/m]

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                        SPM  terminal
                User defined hawser characteristics

                     Review of input  (Continued)

 Chain number                          :          3
 Chain from                            : User supplied
 Orientation angle                     :     120.00  [deg]
 Pre-tension                           :    1000.00   [kN]
 Length                                :    1000.00    [m]
 Breaking strength                     :  100000.00   [kN]
 Elasticity                            :  500000.00   [kN]
 Weight in Air                         :       1.25 [kN/m]

 Chain number                          :          4
 Chain from                            : User supplied
 Orientation angle                     :     180.00  [deg]
 Pre-tension                           :    1000.00   [kN]
 Length                                :    1000.00    [m]
 Breaking strength                     :  100000.00   [kN]
 Elasticity                            :  500000.00   [kN]
 Weight in Air                         :       1.25 [kN/m]

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                        SPM  terminal
                User defined hawser characteristics

                     Review of input  (Continued)

 Chain number                          :          5
 Chain from                            : User supplied
 Orientation angle                     :     240.00  [deg]
 Pre-tension                           :    1000.00   [kN]
 Length                                :    1000.00    [m]
 Breaking strength                     :  100000.00   [kN]
 Elasticity                            :  500000.00   [kN]
 Weight in Air                         :       1.25 [kN/m]

 Chain number                          :          6
 Chain from                            : User supplied
 Orientation angle                     :     300.00  [deg]
 Pre-tension                           :    1000.00   [kN]
 Length                                :    1000.00    [m]
 Breaking strength                     :  100000.00   [kN]
 Elasticity                            :  500000.00   [kN]
 Weight in Air                         :       1.25 [kN/m]
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T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 13

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Load excursion of the hawser

Excursion [m]	Tension [kN]
0.000	0.000
1.700	495.000
3.400	981.000
5.100	1467.000
6.800	1961.000
8.500	2475.000
10.200	3020.000
11.900	3612.000
13.600	4272.000
15.300	5034.000
17.000	5952.000

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 14

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Load excursion of chain number : 1

Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]
-50.000	16.313	-16.000	16.323	18.000	9686.313
-49.000	16.313	-15.000	16.623	19.000	10186.313
-48.000	16.313	-14.000	17.118	20.000	10681.313
-47.000	16.313	-13.000	17.818	21.000	11176.313
-46.000	16.313	-12.000	18.783	22.000	11676.313
-45.000	16.313	-11.000	20.113	23.000	12171.313
-44.000	16.313	-10.000	21.963	24.000	12671.313
-43.000	16.313	-9.000	24.563	25.000	13166.313
-42.000	16.313	-8.000	28.413	26.000	13666.313
-41.000	16.313	-7.000	34.363	27.000	14166.313
-40.000	16.313	-6.000	44.313	28.000	14661.313
-39.000	16.313	-5.000	61.813	29.000	15161.313
-38.000	16.313	-4.000	96.313	30.000	15656.313
-37.000	16.313	-3.000	171.313	31.000	16156.313
-36.000	16.313	-2.000	336.313	32.000	16656.313
-35.000	16.313	-1.000	621.313	33.000	17151.313
-34.000	16.313	0.000	996.313	34.000	17651.313
-33.000	16.313	1.000	1421.313	35.000	18146.313
-32.000	16.313	2.000	1876.313	36.000	18646.313
-31.000	16.313	3.000	2341.313	37.000	19146.313
-30.000	16.313	4.000	2811.313	38.000	19646.313
-29.000	16.313	5.000	3291.313	39.000	20146.313
-28.000	16.313	6.000	3776.313	40.000	20641.313
-27.000	16.313	7.000	4261.313	41.000	21141.313
-26.000	16.313	8.000	4751.313	42.000	21641.313
-25.000	16.313	9.000	5241.313	43.000	22141.313
-24.000	16.313	10.000	5731.313	44.000	22636.313
-23.000	16.313	11.000	6226.313	45.000	23136.313
-22.000	16.313	12.000	6716.313	46.000	23636.313
-21.000	16.313	13.000	7211.313	47.000	24136.313
-20.000	16.313	14.000	7706.313	48.000	24636.313
-19.000	16.313	15.000	8201.313	49.000	25136.313
-18.000	16.313	16.000	8696.313	50.000	25631.313
-17.000	16.313	17.000	9191.313		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 15

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Length of chain at seabed : 1

Excursion [m]	Length [m]	Excursion [m]	Length [m]	Excursion [m]	Length [m]
-50.000	985.000	-16.000	984.991	18.000	483.296
-49.000	985.000	-15.000	984.718	19.000	470.117
-48.000	985.000	-14.000	984.277	20.000	457.385
-47.000	985.000	-13.000	983.675	21.000	444.944
-46.000	985.000	-12.000	982.879	22.000	432.656
-45.000	985.000	-11.000	981.839	23.000	420.746
-44.000	985.000	-10.000	980.484	24.000	408.960
-43.000	985.000	-9.000	978.726	25.000	397.519
-42.000	985.000	-8.000	976.361	26.000	386.179
-41.000	985.000	-7.000	973.113	27.000	375.045
-40.000	985.000	-6.000	968.418	28.000	364.214
-39.000	985.000	-5.000	961.527	29.000	353.457
-38.000	985.000	-4.000	950.686	30.000	342.982
-37.000	985.000	-3.000	932.912	31.000	332.568
-36.000	985.000	-2.000	904.855	32.000	322.314
-35.000	985.000	-1.000	869.944	33.000	312.313
-34.000	985.000	0.000	834.896	34.000	302.356
-33.000	985.000	1.000	802.557	35.000	292.637
-32.000	985.000	2.000	772.986	36.000	282.954
-31.000	985.000	3.000	746.302	37.000	273.399
-30.000	985.000	4.000	721.920	38.000	263.969
-29.000	985.000	5.000	699.052	39.000	254.658
-28.000	985.000	6.000	677.589	40.000	245.553
-27.000	985.000	7.000	657.468	41.000	236.467
-26.000	985.000	8.000	638.274	42.000	227.487
-25.000	985.000	9.000	620.049	43.000	218.611
-24.000	985.000	10.000	602.659	44.000	209.921
-23.000	985.000	11.000	585.832	45.000	201.240
-22.000	985.000	12.000	569.823	46.000	192.652
-21.000	985.000	13.000	554.234	47.000	184.155
-20.000	985.000	14.000	539.172	48.000	175.745
-19.000	985.000	15.000	524.587	49.000	167.420
-18.000	985.000	16.000	510.436	50.000	159.259
-17.000	985.000	17.000	496.683		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 16

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Load excursion of chain number : 2

Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]
-50.000	16.313	-16.000	16.323	18.000	9686.313
-49.000	16.313	-15.000	16.623	19.000	10186.313
-48.000	16.313	-14.000	17.118	20.000	10681.313
-47.000	16.313	-13.000	17.818	21.000	11176.313
-46.000	16.313	-12.000	18.783	22.000	11676.313
-45.000	16.313	-11.000	20.113	23.000	12171.313
-44.000	16.313	-10.000	21.963	24.000	12671.313
-43.000	16.313	-9.000	24.563	25.000	13166.313
-42.000	16.313	-8.000	28.413	26.000	13666.313
-41.000	16.313	-7.000	34.363	27.000	14166.313
-40.000	16.313	-6.000	44.313	28.000	14661.313
-39.000	16.313	-5.000	61.813	29.000	15161.313
-38.000	16.313	-4.000	96.313	30.000	15656.313
-37.000	16.313	-3.000	171.313	31.000	16156.313
-36.000	16.313	-2.000	336.313	32.000	16656.313
-35.000	16.313	-1.000	621.313	33.000	17151.313
-34.000	16.313	0.000	996.313	34.000	17651.313
-33.000	16.313	1.000	1421.313	35.000	18146.313
-32.000	16.313	2.000	1876.313	36.000	18646.313
-31.000	16.313	3.000	2341.313	37.000	19146.313
-30.000	16.313	4.000	2811.313	38.000	19646.313
-29.000	16.313	5.000	3291.313	39.000	20146.313
-28.000	16.313	6.000	3776.313	40.000	20641.313
-27.000	16.313	7.000	4261.313	41.000	21141.313
-26.000	16.313	8.000	4751.313	42.000	21641.313
-25.000	16.313	9.000	5241.313	43.000	22141.313
-24.000	16.313	10.000	5731.313	44.000	22636.313
-23.000	16.313	11.000	6226.313	45.000	23136.313
-22.000	16.313	12.000	6716.313	46.000	23636.313
-21.000	16.313	13.000	7211.313	47.000	24136.313
-20.000	16.313	14.000	7706.313	48.000	24636.313
-19.000	16.313	15.000	8201.313	49.000	25136.313
-18.000	16.313	16.000	8696.313	50.000	25631.313
-17.000	16.313	17.000	9191.313		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 17

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Length of chain at seabed : 2

Excursion [m]	Length [m]	Excursion [m]	Length [m]	Excursion [m]	Length [m]
-50.000	985.000	-16.000	984.991	18.000	483.296
-49.000	985.000	-15.000	984.718	19.000	470.117
-48.000	985.000	-14.000	984.277	20.000	457.385
-47.000	985.000	-13.000	983.675	21.000	444.944
-46.000	985.000	-12.000	982.879	22.000	432.656
-45.000	985.000	-11.000	981.839	23.000	420.746
-44.000	985.000	-10.000	980.484	24.000	408.960
-43.000	985.000	-9.000	978.726	25.000	397.519
-42.000	985.000	-8.000	976.361	26.000	386.179
-41.000	985.000	-7.000	973.113	27.000	375.045
-40.000	985.000	-6.000	968.418	28.000	364.214
-39.000	985.000	-5.000	961.527	29.000	353.457
-38.000	985.000	-4.000	950.686	30.000	342.982
-37.000	985.000	-3.000	932.912	31.000	332.568
-36.000	985.000	-2.000	904.855	32.000	322.314
-35.000	985.000	-1.000	869.944	33.000	312.313
-34.000	985.000	0.000	834.896	34.000	302.356
-33.000	985.000	1.000	802.557	35.000	292.637
-32.000	985.000	2.000	772.986	36.000	282.954
-31.000	985.000	3.000	746.302	37.000	273.399
-30.000	985.000	4.000	721.920	38.000	263.969
-29.000	985.000	5.000	699.052	39.000	254.658
-28.000	985.000	6.000	677.589	40.000	245.553
-27.000	985.000	7.000	657.468	41.000	236.467
-26.000	985.000	8.000	638.274	42.000	227.487
-25.000	985.000	9.000	620.049	43.000	218.611
-24.000	985.000	10.000	602.659	44.000	209.921
-23.000	985.000	11.000	585.832	45.000	201.240
-22.000	985.000	12.000	569.823	46.000	192.652
-21.000	985.000	13.000	554.234	47.000	184.155
-20.000	985.000	14.000	539.172	48.000	175.745
-19.000	985.000	15.000	524.587	49.000	167.420
-18.000	985.000	16.000	510.436	50.000	159.259
-17.000	985.000	17.000	496.683		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 18

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Load excursion of chain number : 3

Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]
-50.000	16.313	-16.000	16.323	18.000	9686.313
-49.000	16.313	-15.000	16.623	19.000	10186.313
-48.000	16.313	-14.000	17.118	20.000	10681.313
-47.000	16.313	-13.000	17.818	21.000	11176.313
-46.000	16.313	-12.000	18.783	22.000	11676.313
-45.000	16.313	-11.000	20.113	23.000	12171.313
-44.000	16.313	-10.000	21.963	24.000	12671.313
-43.000	16.313	-9.000	24.563	25.000	13166.313
-42.000	16.313	-8.000	28.413	26.000	13666.313
-41.000	16.313	-7.000	34.363	27.000	14166.313
-40.000	16.313	-6.000	44.313	28.000	14661.313
-39.000	16.313	-5.000	61.813	29.000	15161.313
-38.000	16.313	-4.000	96.313	30.000	15656.313
-37.000	16.313	-3.000	171.313	31.000	16156.313
-36.000	16.313	-2.000	336.313	32.000	16656.313
-35.000	16.313	-1.000	621.313	33.000	17151.313
-34.000	16.313	0.000	996.313	34.000	17651.313
-33.000	16.313	1.000	1421.313	35.000	18146.313
-32.000	16.313	2.000	1876.313	36.000	18646.313
-31.000	16.313	3.000	2341.313	37.000	19146.313
-30.000	16.313	4.000	2811.313	38.000	19646.313
-29.000	16.313	5.000	3291.313	39.000	20146.313
-28.000	16.313	6.000	3776.313	40.000	20641.313
-27.000	16.313	7.000	4261.313	41.000	21141.313
-26.000	16.313	8.000	4751.313	42.000	21641.313
-25.000	16.313	9.000	5241.313	43.000	22141.313
-24.000	16.313	10.000	5731.313	44.000	22636.313
-23.000	16.313	11.000	6226.313	45.000	23136.313
-22.000	16.313	12.000	6716.313	46.000	23636.313
-21.000	16.313	13.000	7211.313	47.000	24136.313
-20.000	16.313	14.000	7706.313	48.000	24636.313
-19.000	16.313	15.000	8201.313	49.000	25136.313
-18.000	16.313	16.000	8696.313	50.000	25631.313
-17.000	16.313	17.000	9191.313		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 19

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Length of chain at seabed : 3

Excursion [m]	Length [m]	Excursion [m]	Length [m]	Excursion [m]	Length [m]
-50.000	985.000	-16.000	984.991	18.000	483.296
-49.000	985.000	-15.000	984.718	19.000	470.117
-48.000	985.000	-14.000	984.277	20.000	457.385
-47.000	985.000	-13.000	983.675	21.000	444.944
-46.000	985.000	-12.000	982.879	22.000	432.656
-45.000	985.000	-11.000	981.839	23.000	420.746
-44.000	985.000	-10.000	980.484	24.000	408.960
-43.000	985.000	-9.000	978.726	25.000	397.519
-42.000	985.000	-8.000	976.361	26.000	386.179
-41.000	985.000	-7.000	973.113	27.000	375.045
-40.000	985.000	-6.000	968.418	28.000	364.214
-39.000	985.000	-5.000	961.527	29.000	353.457
-38.000	985.000	-4.000	950.686	30.000	342.982
-37.000	985.000	-3.000	932.912	31.000	332.568
-36.000	985.000	-2.000	904.855	32.000	322.314
-35.000	985.000	-1.000	869.944	33.000	312.313
-34.000	985.000	0.000	834.896	34.000	302.356
-33.000	985.000	1.000	802.557	35.000	292.637
-32.000	985.000	2.000	772.986	36.000	282.954
-31.000	985.000	3.000	746.302	37.000	273.399
-30.000	985.000	4.000	721.920	38.000	263.969
-29.000	985.000	5.000	699.052	39.000	254.658
-28.000	985.000	6.000	677.589	40.000	245.553
-27.000	985.000	7.000	657.468	41.000	236.467
-26.000	985.000	8.000	638.274	42.000	227.487
-25.000	985.000	9.000	620.049	43.000	218.611
-24.000	985.000	10.000	602.659	44.000	209.921
-23.000	985.000	11.000	585.832	45.000	201.240
-22.000	985.000	12.000	569.823	46.000	192.652
-21.000	985.000	13.000	554.234	47.000	184.155
-20.000	985.000	14.000	539.172	48.000	175.745
-19.000	985.000	15.000	524.587	49.000	167.420
-18.000	985.000	16.000	510.436	50.000	159.259
-17.000	985.000	17.000	496.683		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 20

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Load excursion of chain number : 4

Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]
-50.000	16.313	-16.000	16.323	18.000	9686.313
-49.000	16.313	-15.000	16.623	19.000	10186.313
-48.000	16.313	-14.000	17.118	20.000	10681.313
-47.000	16.313	-13.000	17.818	21.000	11176.313
-46.000	16.313	-12.000	18.783	22.000	11676.313
-45.000	16.313	-11.000	20.113	23.000	12171.313
-44.000	16.313	-10.000	21.963	24.000	12671.313
-43.000	16.313	-9.000	24.563	25.000	13166.313
-42.000	16.313	-8.000	28.413	26.000	13666.313
-41.000	16.313	-7.000	34.363	27.000	14166.313
-40.000	16.313	-6.000	44.313	28.000	14661.313
-39.000	16.313	-5.000	61.813	29.000	15161.313
-38.000	16.313	-4.000	96.313	30.000	15656.313
-37.000	16.313	-3.000	171.313	31.000	16156.313
-36.000	16.313	-2.000	336.313	32.000	16656.313
-35.000	16.313	-1.000	621.313	33.000	17151.313
-34.000	16.313	0.000	996.313	34.000	17651.313
-33.000	16.313	1.000	1421.313	35.000	18146.313
-32.000	16.313	2.000	1876.313	36.000	18646.313
-31.000	16.313	3.000	2341.313	37.000	19146.313
-30.000	16.313	4.000	2811.313	38.000	19646.313
-29.000	16.313	5.000	3291.313	39.000	20146.313
-28.000	16.313	6.000	3776.313	40.000	20641.313
-27.000	16.313	7.000	4261.313	41.000	21141.313
-26.000	16.313	8.000	4751.313	42.000	21641.313
-25.000	16.313	9.000	5241.313	43.000	22141.313
-24.000	16.313	10.000	5731.313	44.000	22636.313
-23.000	16.313	11.000	6226.313	45.000	23136.313
-22.000	16.313	12.000	6716.313	46.000	23636.313
-21.000	16.313	13.000	7211.313	47.000	24136.313
-20.000	16.313	14.000	7706.313	48.000	24636.313
-19.000	16.313	15.000	8201.313	49.000	25136.313
-18.000	16.313	16.000	8696.313	50.000	25631.313
-17.000	16.313	17.000	9191.313		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 21

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Length of chain at seabed : 4

Excursion [m]	Length [m]	Excursion [m]	Length [m]	Excursion [m]	Length [m]
-50.000	985.000	-16.000	984.991	18.000	483.296
-49.000	985.000	-15.000	984.718	19.000	470.117
-48.000	985.000	-14.000	984.277	20.000	457.385
-47.000	985.000	-13.000	983.675	21.000	444.944
-46.000	985.000	-12.000	982.879	22.000	432.656
-45.000	985.000	-11.000	981.839	23.000	420.746
-44.000	985.000	-10.000	980.484	24.000	408.960
-43.000	985.000	-9.000	978.726	25.000	397.519
-42.000	985.000	-8.000	976.361	26.000	386.179
-41.000	985.000	-7.000	973.113	27.000	375.045
-40.000	985.000	-6.000	968.418	28.000	364.214
-39.000	985.000	-5.000	961.527	29.000	353.457
-38.000	985.000	-4.000	950.686	30.000	342.982
-37.000	985.000	-3.000	932.912	31.000	332.568
-36.000	985.000	-2.000	904.855	32.000	322.314
-35.000	985.000	-1.000	869.944	33.000	312.313
-34.000	985.000	0.000	834.896	34.000	302.356
-33.000	985.000	1.000	802.557	35.000	292.637
-32.000	985.000	2.000	772.986	36.000	282.954
-31.000	985.000	3.000	746.302	37.000	273.399
-30.000	985.000	4.000	721.920	38.000	263.969
-29.000	985.000	5.000	699.052	39.000	254.658
-28.000	985.000	6.000	677.589	40.000	245.553
-27.000	985.000	7.000	657.468	41.000	236.467
-26.000	985.000	8.000	638.274	42.000	227.487
-25.000	985.000	9.000	620.049	43.000	218.611
-24.000	985.000	10.000	602.659	44.000	209.921
-23.000	985.000	11.000	585.832	45.000	201.240
-22.000	985.000	12.000	569.823	46.000	192.652
-21.000	985.000	13.000	554.234	47.000	184.155
-20.000	985.000	14.000	539.172	48.000	175.745
-19.000	985.000	15.000	524.587	49.000	167.420
-18.000	985.000	16.000	510.436	50.000	159.259
-17.000	985.000	17.000	496.683		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 22

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Load excursion of chain number : 5

Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]
-50.000	16.313	-16.000	16.323	18.000	9686.313
-49.000	16.313	-15.000	16.623	19.000	10186.313
-48.000	16.313	-14.000	17.118	20.000	10681.313
-47.000	16.313	-13.000	17.818	21.000	11176.313
-46.000	16.313	-12.000	18.783	22.000	11676.313
-45.000	16.313	-11.000	20.113	23.000	12171.313
-44.000	16.313	-10.000	21.963	24.000	12671.313
-43.000	16.313	-9.000	24.563	25.000	13166.313
-42.000	16.313	-8.000	28.413	26.000	13666.313
-41.000	16.313	-7.000	34.363	27.000	14166.313
-40.000	16.313	-6.000	44.313	28.000	14661.313
-39.000	16.313	-5.000	61.813	29.000	15161.313
-38.000	16.313	-4.000	96.313	30.000	15656.313
-37.000	16.313	-3.000	171.313	31.000	16156.313
-36.000	16.313	-2.000	336.313	32.000	16656.313
-35.000	16.313	-1.000	621.313	33.000	17151.313
-34.000	16.313	0.000	996.313	34.000	17651.313
-33.000	16.313	1.000	1421.313	35.000	18146.313
-32.000	16.313	2.000	1876.313	36.000	18646.313
-31.000	16.313	3.000	2341.313	37.000	19146.313
-30.000	16.313	4.000	2811.313	38.000	19646.313
-29.000	16.313	5.000	3291.313	39.000	20146.313
-28.000	16.313	6.000	3776.313	40.000	20641.313
-27.000	16.313	7.000	4261.313	41.000	21141.313
-26.000	16.313	8.000	4751.313	42.000	21641.313
-25.000	16.313	9.000	5241.313	43.000	22141.313
-24.000	16.313	10.000	5731.313	44.000	22636.313
-23.000	16.313	11.000	6226.313	45.000	23136.313
-22.000	16.313	12.000	6716.313	46.000	23636.313
-21.000	16.313	13.000	7211.313	47.000	24136.313
-20.000	16.313	14.000	7706.313	48.000	24636.313
-19.000	16.313	15.000	8201.313	49.000	25136.313
-18.000	16.313	16.000	8696.313	50.000	25631.313
-17.000	16.313	17.000	9191.313		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 23

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Length of chain at seabed : 5

Excursion [m]	Length [m]	Excursion [m]	Length [m]	Excursion [m]	Length [m]
-50.000	985.000	-16.000	984.991	18.000	483.296
-49.000	985.000	-15.000	984.718	19.000	470.117
-48.000	985.000	-14.000	984.277	20.000	457.385
-47.000	985.000	-13.000	983.675	21.000	444.944
-46.000	985.000	-12.000	982.879	22.000	432.656
-45.000	985.000	-11.000	981.839	23.000	420.746
-44.000	985.000	-10.000	980.484	24.000	408.960
-43.000	985.000	-9.000	978.726	25.000	397.519
-42.000	985.000	-8.000	976.361	26.000	386.179
-41.000	985.000	-7.000	973.113	27.000	375.045
-40.000	985.000	-6.000	968.418	28.000	364.214
-39.000	985.000	-5.000	961.527	29.000	353.457
-38.000	985.000	-4.000	950.686	30.000	342.982
-37.000	985.000	-3.000	932.912	31.000	332.568
-36.000	985.000	-2.000	904.855	32.000	322.314
-35.000	985.000	-1.000	869.944	33.000	312.313
-34.000	985.000	0.000	834.896	34.000	302.356
-33.000	985.000	1.000	802.557	35.000	292.637
-32.000	985.000	2.000	772.986	36.000	282.954
-31.000	985.000	3.000	746.302	37.000	273.399
-30.000	985.000	4.000	721.920	38.000	263.969
-29.000	985.000	5.000	699.052	39.000	254.658
-28.000	985.000	6.000	677.589	40.000	245.553
-27.000	985.000	7.000	657.468	41.000	236.467
-26.000	985.000	8.000	638.274	42.000	227.487
-25.000	985.000	9.000	620.049	43.000	218.611
-24.000	985.000	10.000	602.659	44.000	209.921
-23.000	985.000	11.000	585.832	45.000	201.240
-22.000	985.000	12.000	569.823	46.000	192.652
-21.000	985.000	13.000	554.234	47.000	184.155
-20.000	985.000	14.000	539.172	48.000	175.745
-19.000	985.000	15.000	524.587	49.000	167.420
-18.000	985.000	16.000	510.436	50.000	159.259
-17.000	985.000	17.000	496.683		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 24

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Load excursion of chain number : 6

Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]	Excursion [m]	Tension [kN]
-50.000	16.313	-16.000	16.323	18.000	9686.313
-49.000	16.313	-15.000	16.623	19.000	10186.313
-48.000	16.313	-14.000	17.118	20.000	10681.313
-47.000	16.313	-13.000	17.818	21.000	11176.313
-46.000	16.313	-12.000	18.783	22.000	11676.313
-45.000	16.313	-11.000	20.113	23.000	12171.313
-44.000	16.313	-10.000	21.963	24.000	12671.313
-43.000	16.313	-9.000	24.563	25.000	13166.313
-42.000	16.313	-8.000	28.413	26.000	13666.313
-41.000	16.313	-7.000	34.363	27.000	14166.313
-40.000	16.313	-6.000	44.313	28.000	14661.313
-39.000	16.313	-5.000	61.813	29.000	15161.313
-38.000	16.313	-4.000	96.313	30.000	15656.313
-37.000	16.313	-3.000	171.313	31.000	16156.313
-36.000	16.313	-2.000	336.313	32.000	16656.313
-35.000	16.313	-1.000	621.313	33.000	17151.313
-34.000	16.313	0.000	996.313	34.000	17651.313
-33.000	16.313	1.000	1421.313	35.000	18146.313
-32.000	16.313	2.000	1876.313	36.000	18646.313
-31.000	16.313	3.000	2341.313	37.000	19146.313
-30.000	16.313	4.000	2811.313	38.000	19646.313
-29.000	16.313	5.000	3291.313	39.000	20146.313
-28.000	16.313	6.000	3776.313	40.000	20641.313
-27.000	16.313	7.000	4261.313	41.000	21141.313
-26.000	16.313	8.000	4751.313	42.000	21641.313
-25.000	16.313	9.000	5241.313	43.000	22141.313
-24.000	16.313	10.000	5731.313	44.000	22636.313
-23.000	16.313	11.000	6226.313	45.000	23136.313
-22.000	16.313	12.000	6716.313	46.000	23636.313
-21.000	16.313	13.000	7211.313	47.000	24136.313
-20.000	16.313	14.000	7706.313	48.000	24636.313
-19.000	16.313	15.000	8201.313	49.000	25136.313
-18.000	16.313	16.000	8696.313	50.000	25631.313
-17.000	16.313	17.000	9191.313		

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 25

SPM terminal
User defined hawser characteristics

Review of input (Continued)

Length of chain at seabed : 6

Excursion [m]	Length [m]	Excursion [m]	Length [m]	Excursion [m]	Length [m]
-50.000	985.000	-16.000	984.991	18.000	483.296
-49.000	985.000	-15.000	984.718	19.000	470.117
-48.000	985.000	-14.000	984.277	20.000	457.385
-47.000	985.000	-13.000	983.675	21.000	444.944
-46.000	985.000	-12.000	982.879	22.000	432.656
-45.000	985.000	-11.000	981.839	23.000	420.746
-44.000	985.000	-10.000	980.484	24.000	408.960
-43.000	985.000	-9.000	978.726	25.000	397.519
-42.000	985.000	-8.000	976.361	26.000	386.179
-41.000	985.000	-7.000	973.113	27.000	375.045
-40.000	985.000	-6.000	968.418	28.000	364.214
-39.000	985.000	-5.000	961.527	29.000	353.457
-38.000	985.000	-4.000	950.686	30.000	342.982
-37.000	985.000	-3.000	932.912	31.000	332.568
-36.000	985.000	-2.000	904.855	32.000	322.314
-35.000	985.000	-1.000	869.944	33.000	312.313
-34.000	985.000	0.000	834.896	34.000	302.356
-33.000	985.000	1.000	802.557	35.000	292.637
-32.000	985.000	2.000	772.986	36.000	282.954
-31.000	985.000	3.000	746.302	37.000	273.399
-30.000	985.000	4.000	721.920	38.000	263.969
-29.000	985.000	5.000	699.052	39.000	254.658
-28.000	985.000	6.000	677.589	40.000	245.553
-27.000	985.000	7.000	657.468	41.000	236.467
-26.000	985.000	8.000	638.274	42.000	227.487
-25.000	985.000	9.000	620.049	43.000	218.611
-24.000	985.000	10.000	602.659	44.000	209.921
-23.000	985.000	11.000	585.832	45.000	201.240
-22.000	985.000	12.000	569.823	46.000	192.652
-21.000	985.000	13.000	554.234	47.000	184.155
-20.000	985.000	14.000	539.172	48.000	175.745
-19.000	985.000	15.000	524.587	49.000	167.420
-18.000	985.000	16.000	510.436	50.000	159.259
-17.000	985.000	17.000	496.683		

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SPM terminal
User defined hawser characteristics

Output : Environmental forces

Mode	Mean environmental force
X	-336.27 [kN]
Y	-434.05 [kN]
N	-59769.09 [kNm]

Forces are vessel bound wrt centre of gravity

T E R M S I M Phase II v2002_0 Date :08/28/02 Time:21:50:57 Page: 27

SPM terminal
User defined hawser characteristics

Project : BOSTON

Outputfile : bat40275

	Case	Description
Environment :	0300	output
Vessel :	0300	
Mooring :	0300	
Control :	0300	
Output :	0300	environment

Output : SPM

Mean X-displacement of buoy	=	-1.06	[m]
Mean Y-displacement of buoy	=	0.46	[m]
Mean heading of vessel	=	318.52	[deg]
Mean hawser angle	=	336.25	[deg]
Mean hawser force	=	1381.	[kN]
Mean hawser elongation	=	4.80	[m]
Safety factor of hawser	=	3.16	[-]

Chain	Mean force [kN]	Safety factor [-]
1	1448.92	60.63
2	1050.55	88.70
3	650.16	129.35
4	608.42	133.96
5	948.46	100.78
6	1392.64	63.95

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T E R M S I M Phase II v2002_0  Date :08/28/02  Time:21:50:57  Page: 28

                 SPM terminal

    Project     : BOSTON
    Outputfile  : bat40275

                    Case         Description
    Environment :   0300         output
    Vessel      :   0300
    Mooring     :   0300
    Control     :   0300
    Output      :   0300         environment

Signr     Name        Unit      Mean       Stdev       Min         Max
    1Wave              m 0.28757E-03 0.49494E+00-0.16461E+01 0.14397E+01
    2X-mot             m 0.35331E+02 0.18166E+01 0.29283E+02 0.39138E+02
    3Y-mot             m 0.11998E+03 0.30736E+01 0.10982E+03 0.12527E+03
    4Z-mot             m 0.41046E-04 0.83423E-01-0.24113E+00 0.25405E+00
    5Roll-mot        deg 0.29066E-03 0.22526E+00-0.75399E+00 0.85510E+00
    6Pitch-mot       deg 0.96482E-04 0.19996E+00-0.59758E+00 0.58212E+00
    7Yaw-mot         deg 0.31852E+03 0.90305E+00 0.31631E+03 0.32111E+03
    8X-buoy            m-0.10580E+01 0.16417E+00-0.15010E+01-0.66624E+00
    9Y-buoy            m 0.46348E+00 0.83038E-01 0.24240E+00 0.69541E+00
   10F-hawser         kN 0.13808E+04 0.20131E+03 0.88893E+03 0.18837E+04
   11Haws-ang        deg-0.23750E+02 0.27936E+01-0.29434E+02-0.16458E+02
   12F-chain 1        kN 0.14489E+04 0.73205E+02 0.12795E+04 0.16493E+04
   13F-chain 2        kN 0.10505E+04 0.26699E+02 0.10009E+04 0.11273E+04
   14F-chain 3        kN 0.65016E+03 0.48503E+02 0.53212E+03 0.77312E+03
   15F-chain 4        kN 0.60842E+03 0.51434E+02 0.47853E+03 0.74647E+03
   16F-chain 5        kN 0.94846E+03 0.23558E+02 0.88070E+03 0.99230E+03
   17F-chain 6        kN 0.13926E+04 0.61416E+02 0.12493E+04 0.15637E+04
   18F-moor X         kN 0.13129E+04 0.18927E+03 0.86420E+03 0.18067E+04
   19F-moor Y         kN 0.42138E+03 0.10096E+03 0.20130E+03 0.66827E+03
   20F-moor Z         kN 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   21F-moor K        kNm 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   22F-moor M        kNm 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   23F-moor N        kNm 0.60338E+05 0.14457E+05 0.28825E+05 0.95689E+05
   24F-wind X         kN-0.24875E+03 0.42495E+02-0.38593E+03-0.11615E+03
   25F-wind Y         kN-0.14216E+04 0.23034E+03-0.21485E+04-0.66623E+03
   26F-wind N        kNm-0.41663E+05 0.68121E+04-0.62731E+05-0.19499E+05
   27F-lfdamp X       kN-0.14833E+02 0.14813E+02-0.45835E+02 0.40927E+02
   28F-lfdamp Y       kN 0.50758E+03 0.79992E+02 0.31000E+03 0.67935E+03
   29F-lfdamp N      kNm-0.26488E+05 0.40294E+04-0.38260E+05-0.13160E+05
   30F-wave X         kN-0.72684E+02 0.24712E+03-0.98594E+03 0.95815E+03

T E R M S I M Phase II v2002_0  Date :08/28/02  Time:21:50:57  Page: 29

                 SPM terminal

    Project     : BOSTON
    Outputfile  : bat40275

                    Case         Description
    Environment :   0300         output
    Vessel      :   0300
    Mooring     :   0300
    Control     :   0300
    Output      :   0300         environment

Signr     Name        Unit      Mean       Stdev       Min         Max
   31F-wave Y         kN 0.48002E+03 0.84834E+03-0.20693E+04 0.42963E+04
   32F-wave Z         kN 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   33F-wave K        kNm 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   34F-wave M        kNm 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   35F-wave N        kNm 0.83815E+04 0.44210E+05-0.14399E+06 0.22624E+06
   36F-envir X        kN-0.33627E+03 0.24807E+03-0.12726E+04 0.64339E+03
   37F-envir Y        kN-0.43405E+03 0.84550E+03-0.31551E+04 0.28571E+04
   38F-envir Z        kN 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   39F-envir K       kNm 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   40F-envir M       kNm 0.00000E+00 0.00000E+00 0.00000E+00 0.00000E+00
   41F-envir N       kNm-0.59769E+05 0.44842E+05-0.21415E+06 0.16216E+06

T E R M S I M Phase II v2002_0  Date :08/28/02  Time:21:50:57  Page: 30

                 SPM terminal
                 General data  and  vessel particulars

    Project    : BOSTON

    Outputfile : bat40275

                    Case         Description
    Environment :   0300         output
    Vessel      :   0300
    Mooring     :   0300
    Control     :   0300
    Output      :   0300         environment

    Single amplitude significant value of
      horizontal wave force on the buoy
      (in mean hawser direction)           = 0.0000       [kN]

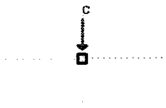
    Single amplitude significant value of
      fairlead point motion
      (in mean hawser direction)           =       0.37   [m]

    Single amplitude significant value of
      hawser load due to fairlead point
      motion (in mean hawser direction)    =  86.73       [kN]

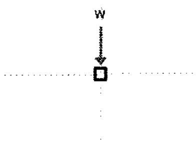
    Total maximum hawser load              =  1970.       [kN]
```

M Summary of TERMSIM results

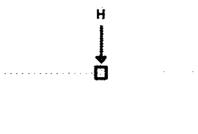
M.1 Current only

	Maximum	
	Yaw [deg]	0.00
	Roll [deg]	0.00
	Pitch [deg]	0.00
	Force [kN]	12

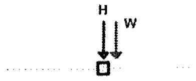
M.2 Wind only

	Maximum	*Vw = 10 m/s*	*Vw = 20 m/s*
	Yaw [deg]	0.00	0.00
	Roll [deg]	0.00	0.00
	Pitch [deg]	0.01	0.01
	Force [kN]	99	392

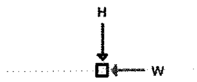
M.3 Waves only

	Maximum	Hs [m]	Tm = 6 s	Tm = 8 s	Tm = 10 s
	Yaw [deg]	1	0.00	0.00	0.00
		2	0.00	0.00	0.00
		3	0.00	0.00	0.00
	Roll [deg]	1	0.00	0.00	0.00
		2	0.00	0.00	0.00
		3	0.00	0.00	0.00
	Pitch [deg]	1	0.03	0.06	0.10
		2	0.09	0.18	0.27
		3	0.13	0.25	0.37
	Force [kN]	1	62	249	562
		2	141	569	981
		3	189	762	981

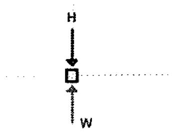
M.4 No current



Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	0.00	0.00	0.00	0.00	0.00	0.00
	2	0.00	0.00	0.00	0.00	0.00	0.00
	3	0.00	0.00	0.00	0.00	0.00	0.00
Roll [deg]	1	0.00	0.00	0.00	0.00	0.00	0.00
	2	0.00	0.00	0.00	0.00	0.00	0.00
	3	0.00	0.00	0.00	0.00	0.00	0.00
Pitch [deg]	1	0.03	0.06	0.10	0.04	0.07	0.11
	2	0.09	0.18	0.27	0.08	0.17	0.27
	3	0.12	0.25	0.37	0.12	0.24	0.36
Force [kN]	1	104	136	447	398	413	439
	2	106	502	981	400	422	969
	3	109	662	981	402	441	981

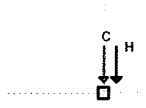


Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	2.28	2.28	0.83	3.64	7.21	8.31
	2	2.10	2.94	1.39	4.35	6.44	8.35
	3	2.02	2.10	0.64	4.44	6.23	5.69
Roll [deg]	1	0.12	0.24	0.30	0.21	0.36	0.47
	2	0.26	0.57	0.88	0.56	0.80	1.24
	3	0.52	0.63	0.98	0.76	1.18	1.29
Pitch [deg]	1	0.11	0.09	0.13	0.15	0.21	0.15
	2	0.29	0.20	0.28	0.28	0.60	0.48
	3	0.45	0.45	0.44	0.36	0.91	1.06
Force [kN]	1	97	115	421	399	395	375
	2	102	370	981	402	415	478
	3	107	560	981	401	431	775

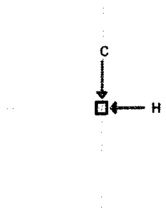


Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	6.29	9.34	0.00	0.00	0.00	5.85
	2	0.00	10.36	0.00	0.00	0.00	10.65
	3	0.00	8.08	0.00	0.00	0.00	10.08
Roll [deg]	1	0.23	0.37	0.00	0.00	0.00	0.11
	2	0.00	0.78	0.00	0.00	0.00	0.59
	3	0.00	1.35	0.00	0.00	0.00	0.87
Pitch [deg]	1	0.17	0.26	0.10	0.07	0.12	0.17
	2	0.11	0.58	0.27	0.12	0.23	0.34
	3	0.16	0.90	0.37	0.16	0.31	0.50
Force [kN]	1	32	263	677	380	342	288
	2	84	498	981	378	335	981
	3	101	478	981	377	412	981

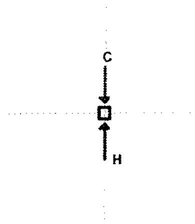
M.5 No wind



Maximum	Hs [m]	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	0.00	0.00	0.00
	2	0.00	0.00	0.00
	3	0.00	0.00	0.00
Roll [deg]	1	0.00	0.00	0.00
	2	0.00	0.00	0.00
	3	0.00	0.00	0.00
Pitch [deg]	1	0.03	0.06	0.11
	2	0.09	0.18	0.27
	3	0.13	0.25	0.37
Force [kN]	1	54	255	592
	2	139	597	981
	3	183	773	981

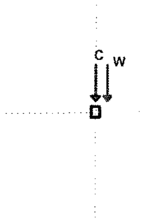


Maximum	Hs [m]	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	4.43	5.91	3.18
	2	5.09	6.46	3.97
	3	5.12	5.37	3.75
Roll [deg]	1	0.16	0.26	0.33
	2	0.41	0.61	0.97
	3	0.68	0.79	1.04
Pitch [deg]	1	0.19	0.10	0.14
	2	0.32	0.22	0.29
	3	0.44	0.54	0.47
Force [kN]	1	16	177	555
	2	36	409	981
	3	46	582	981

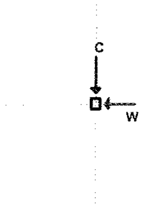


Maximum	Hs [m]	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	0.00	1.38	10.73
	2	0.00	5.91	8.54
	3	0.00	4.61	9.98
Roll [deg]	1	0.00	0.66	0.29
	2	0.00	1.45	0.84
	3	0.00	1.93	1.01
Pitch [deg]	1	0.06	0.16	0.13
	2	0.11	0.42	0.28
	3	0.16	0.44	0.47
Force [kN]	1	58	30	442
	2	173	237	981
	3	230	266	981

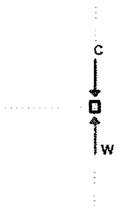
M.6 No waves



Maximum	*Vw = 10 m/s*	*Vw = 20 m/s*
Yaw [deg]	0.00	0.00
Roll [deg]	0.00	0.00
Pitch [deg]	0.01	0.02
Force [kN]	111	404

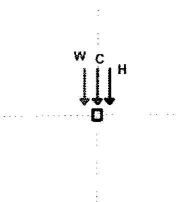


Maximum	*Vw = 10 m/s*	*Vw = 20 m/s*
Yaw [deg]	4.33	5.13
Roll [deg]	0.04	0.07
Pitch [deg]	0.01	0.01
Force [kN]	19	338

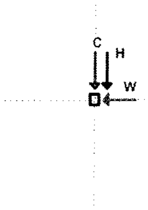


Maximum	*Vw = 10 m/s*	*Vw = 20 m/s*
Yaw [deg]	12.81	14.74
Roll [deg]	0.06	0.06
Pitch [deg]	0.00	0.01
Force [kN]	27	405

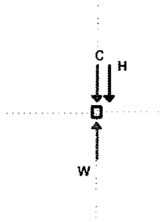
M.7 Current, wind and waves



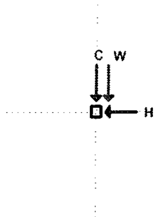
Maximum	Hs [m]	*Vw = 10 m/s*			*Vw = 20 m/s*		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	0.00	0.00	0.00	0.00	0.00	0.00
	2	0.00	0.00	0.00	0.00	0.00	0.00
	3	0.00	0.00	0.00	0.00	0.00	0.00
Roll [deg]	1	0.00	0.00	0.00	0.00	0.00	0.00
	2	0.00	0.00	0.00	0.00	0.00	0.00
	3	0.00	0.00	0.00	0.00	0.00	0.00
Pitch [deg]	1	0.03	0.07	0.11	0.04	0.07	0.11
	2	0.09	0.17	0.27	0.09	0.17	0.27
	3	0.12	0.25	0.37	0.12	0.24	0.36
Force [kN]	1	116	141	476	409	425	451
	2	118	529	981	411	434	981
	3	120	685	981	414	465	981



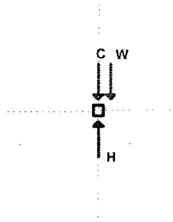
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	4.76	4.97	4.84	4.93	4.28	2.79
	2	4.60	4.52	4.55	4.62	4.05	4.00
	3	4.74	4.86	5.07	4.50	3.16	1.97
Roll [deg]	1	0.08	0.17	0.22	0.16	0.28	0.46
	2	0.21	0.48	0.67	0.23	0.56	1.05
	3	0.27	0.59	0.78	0.47	0.89	1.21
Pitch [deg]	1	0.04	0.08	0.12	0.10	0.10	0.15
	2	0.10	0.18	0.28	0.28	0.37	0.33
	3	0.18	0.30	0.42	0.44	0.70	0.83
Force [kN]	1	38	183	475	330	303	278
	2	111	475	981	336	316	780
	3	169	745	981	343	340	981



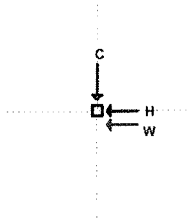
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	0.00	0.00	0.00	17.96	22.61	17.14
	2	0.00	0.00	0.00	18.19	23.03	23.35
	3	0.00	0.00	0.00	18.12	22.90	20.45
Roll [deg]	1	0.00	0.00	0.00	0.11	0.27	1.26
	2	0.00	0.00	0.00	0.26	0.65	2.57
	3	0.00	0.00	0.00	0.40	0.83	3.06
Pitch [deg]	1	0.03	0.06	0.10	0.07	0.12	0.36
	2	0.09	0.18	0.27	0.12	0.26	1.01
	3	0.13	0.25	0.37	0.19	0.42	1.32
Force [kN]	1	191	370	709	400	330	457
	2	228	665	981	398	351	674
	3	302	893	981	396	505	981



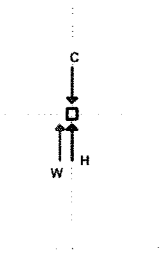
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	4.39	8.58	6.94	4.14	9.65	12.29
	2	5.30	7.59	7.89	5.10	9.84	10.40
	3	5.41	5.69	6.26	5.15	9.87	7.46
Roll [deg]	1	0.19	0.27	0.40	0.22	0.37	0.48
	2	0.53	0.59	0.99	0.56	0.88	1.26
	3	0.76	1.02	1.13	0.80	1.39	1.67
Pitch [deg]	1	0.18	0.13	0.14	0.09	0.30	0.23
	2	0.32	0.45	0.30	0.19	0.63	0.71
	3	0.42	0.90	0.73	0.25	0.90	1.32
Force [kN]	1	117	101	353	410	422	400
	2	120	262	906	414	437	442
	3	119	354	981	413	447	665



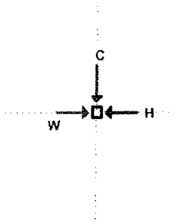
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	0.00	0.00	16.51	0.00	0.00	0.00
	2	0.00	0.00	17.88	0.00	0.00	0.00
	3	0.00	0.00	13.24	0.00	0.00	0.00
Roll [deg]	1	0.00	0.00	1.01	0.00	0.00	0.00
	2	0.00	0.00	1.71	0.00	0.00	0.00
	3	0.00	0.00	2.64	0.00	0.00	0.00
Pitch [deg]	1	0.06	0.12	0.60	0.07	0.12	0.17
	2	0.11	0.22	0.91	0.12	0.23	0.34
	3	0.16	0.30	1.36	0.16	0.31	0.46
Force [kN]	1	98	175	705	391	354	291
	2	96	600	981	390	347	981
	3	101	812	981	389	446	981



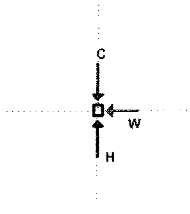
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	3.45	1.80	0.16	5.15	5.25	5.32
	2	3.59	2.22	0.64	5.06	5.00	4.89
	3	3.38	1.68	0.74	5.14	5.10	4.50
Roll [deg]	1	0.13	0.19	0.30	0.10	0.17	0.26
	2	0.27	0.58	0.87	0.23	0.47	0.69
	3	0.33	0.64	0.96	0.31	0.57	0.81
Pitch [deg]	1	0.05	0.08	0.13	0.05	0.09	0.13
	2	0.13	0.19	0.29	0.10	0.19	0.28
	3	0.26	0.35	0.45	0.15	0.29	0.42
Force [kN]	1	44	181	500	353	391	440
	2	79	490	981	355	398	981
	3	118	689	981	357	640	981



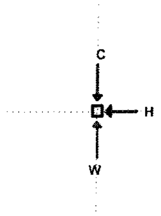
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	0.98	15.32	0.00	12.81	0.00	0.00
	2	1.21	14.07	0.00	12.74	0.00	0.00
	3	0.96	15.01	9.00	12.92	0.00	0.00
Roll [deg]	1	0.22	0.27	0.00	0.08	0.00	0.00
	2	0.59	0.59	0.00	0.18	0.00	0.00
	3	0.89	0.77	0.78	0.26	0.00	0.00
Pitch [deg]	1	0.07	0.10	0.11	0.05	0.07	0.11
	2	0.05	0.20	0.27	0.09	0.17	0.27
	3	0.07	0.52	0.42	0.14	0.24	0.36
Force [kN]	1	13	160	498	411	399	425
	2	22	361	981	413	407	981
	3	22	889	981	416	489	981



Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	4.04	3.31	4.19	5.60	7.34	11.33
	2	3.63	2.12	1.82	5.66	7.54	12.28
	3	3.60	0.99	3.94	5.73	7.77	13.05
Roll [deg]	1	0.17	0.58	0.44	0.11	0.20	0.59
	2	0.25	1.41	1.00	0.26	0.54	1.40
	3	0.49	1.69	1.30	0.40	0.77	1.57
Pitch [deg]	1	0.09	0.29	0.15	0.07	0.12	0.21
	2	0.29	0.67	0.30	0.12	0.23	0.96
	3	0.45	0.88	0.81	0.19	0.37	1.33
Force [kN]	1	34	149	698	318	243	513
	2	132	225	981	314	436	981
	3	178	266	981	313	673	981

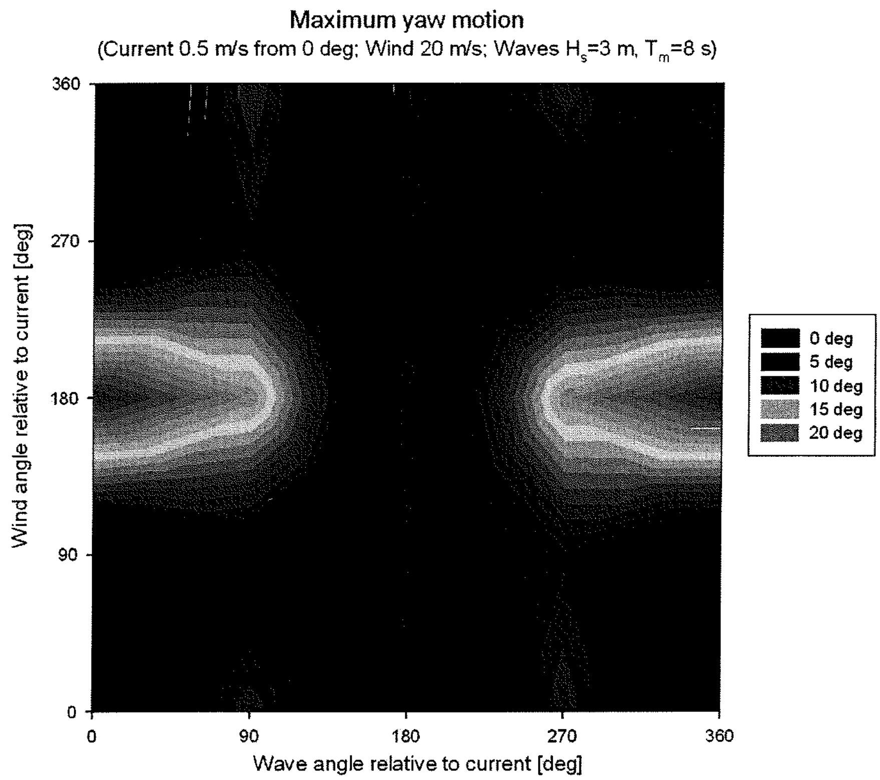


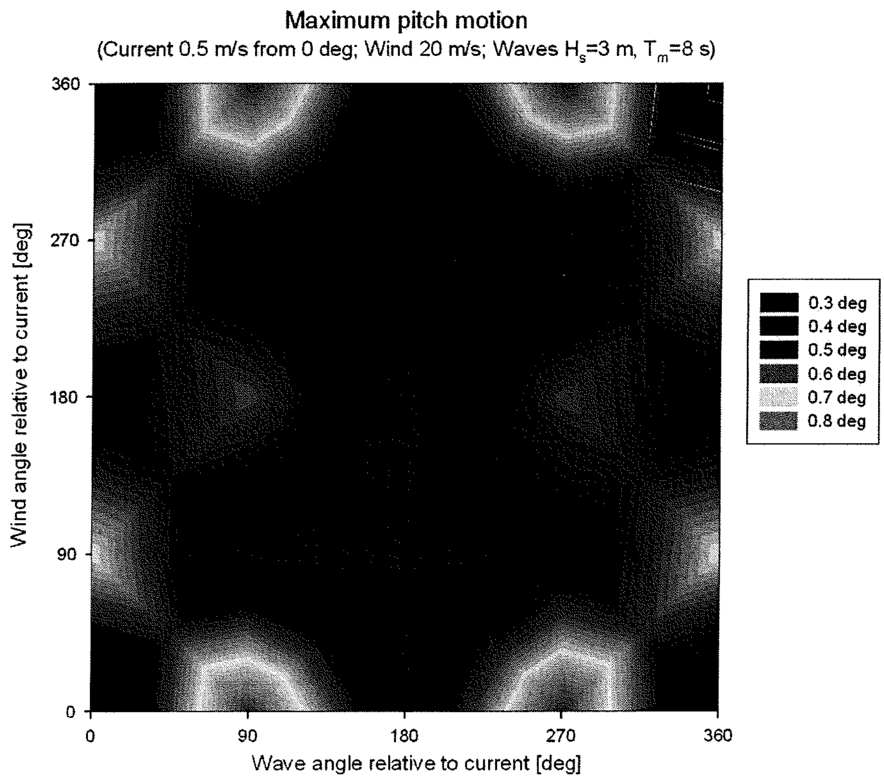
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	4.56	2.12	10.11	4.39	2.36	2.15
	2	4.46	3.89	7.79	4.13	1.15	2.15
	3	4.55	4.65	10.99	4.19	0.98	1.27
Roll [deg]	1	0.11	0.68	0.49	0.09	0.70	0.65
	2	0.25	1.47	1.12	0.23	1.51	1.60
	3	0.35	1.61	1.86	0.61	2.02	2.22
Pitch [deg]	1	0.07	0.13	0.15	0.15	0.10	0.39
	2	0.14	0.34	0.36	0.33	0.23	0.91
	3	0.27	0.89	1.37	0.49	0.25	1.38
Force [kN]	1	46	107	495	346	406	467
	2	131	146	946	341	426	686
	3	180	459	981	334	421	841



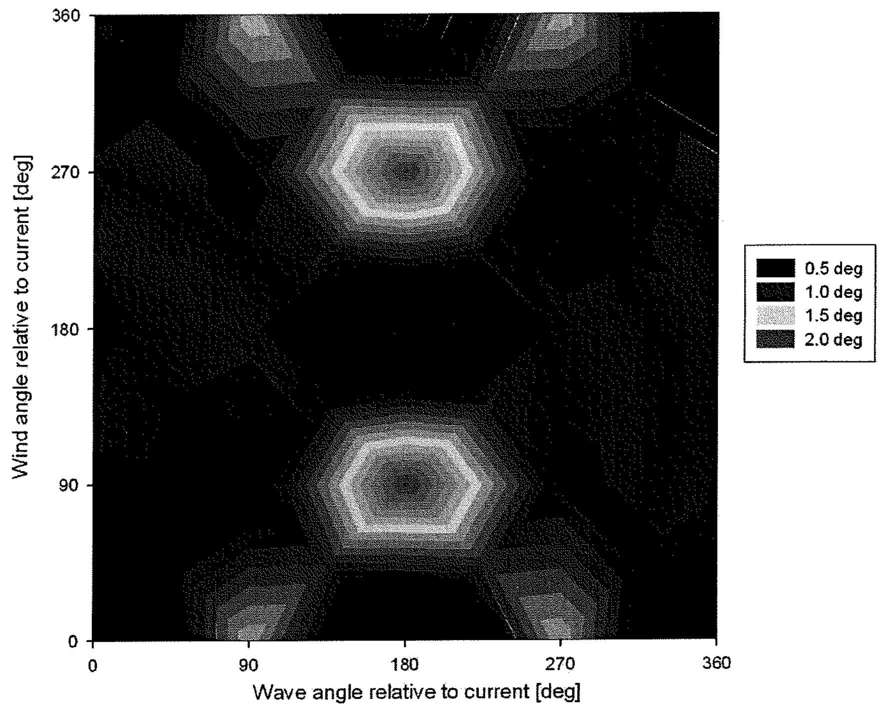
Maximum	Hs [m]	Vw = 10 m/s			Vw = 20 m/s		
		Tm = 6 s	Tm = 8 s	Tm = 10 s	Tm = 6 s	Tm = 8 s	Tm = 10 s
Yaw [deg]	1	9.39	5.34	2.74	18.01	18.70	17.83
	2	9.43	5.53	3.41	17.73	19.04	18.31
	3	9.59	5.89	4.70	17.34	17.73	16.56
Roll [deg]	1	0.09	0.12	0.13	0.16	0.29	0.40
	2	0.19	0.29	0.34	0.23	0.59	0.90
	3	0.26	0.37	0.41	0.46	0.86	1.05
Pitch [deg]	1	0.04	0.07	0.11	0.09	0.10	0.13
	2	0.09	0.18	0.27	0.27	0.29	0.30
	3	0.14	0.27	0.40	0.43	0.60	0.63
Force [kN]	1	66	247	567	409	371	311
	2	114	506	981	414	377	568
	3	171	738	981	420	400	981

N Directionality plots





Maximum roll motion
(Current 0.5 m/s from 0 deg; Wind 20 m/s; Waves H_s=3 m, T_m=8 s)



Maximum mooring force
(Current 0.5 m/s from 0 deg; Wind 20 m/s; Waves H_s=3 m, T_m=8 s)