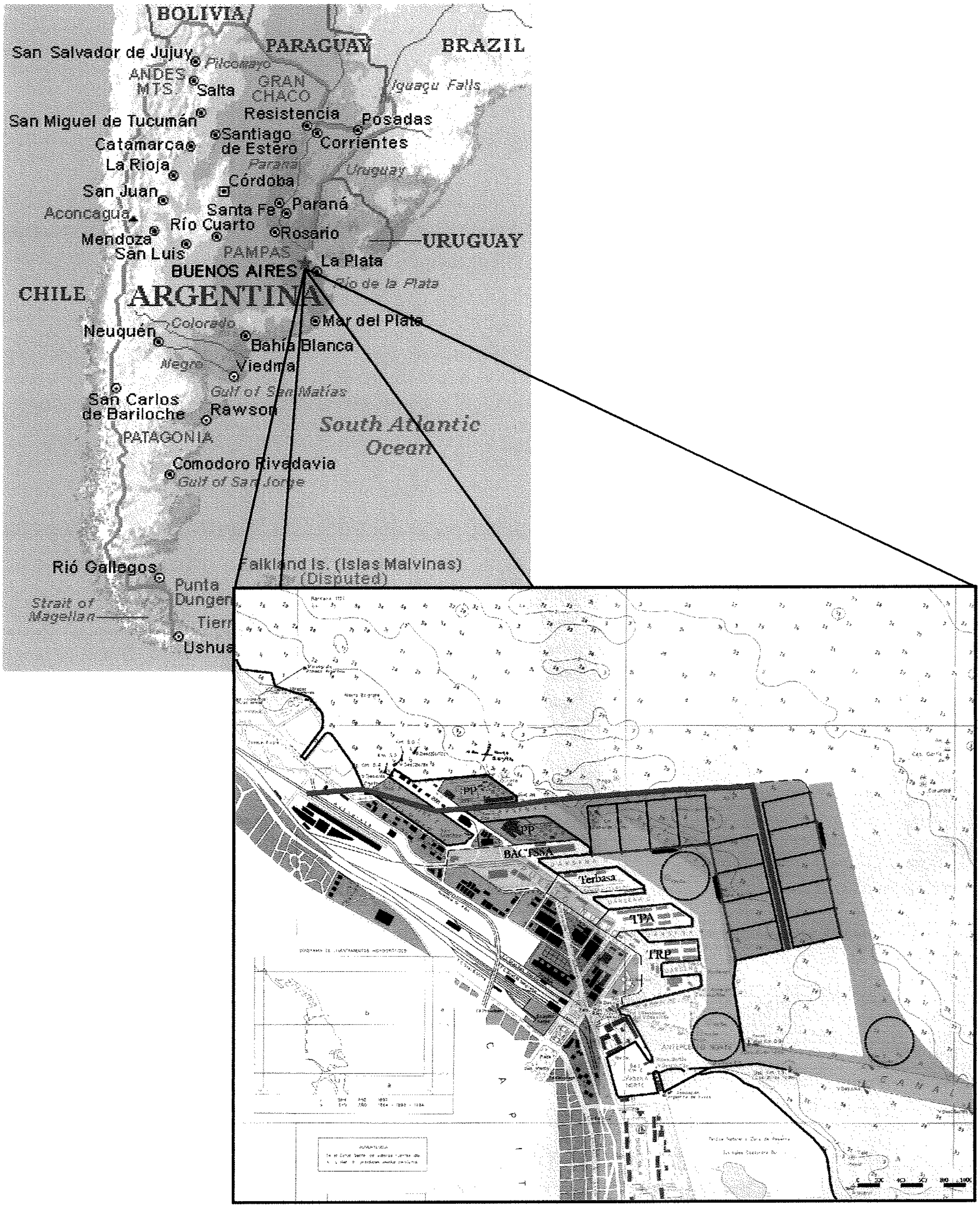
BUENOS AIRES PORT EXPANSION 2020



Final report, by R.D. van den Bosch

Under supervision of:

Prof. ir. H. Ligteringen	TU Delft
Ir. R. Groenveld	TU Delft
Prof. dr. ir. J.A. Pinkster	TU Delft
Ir. D.C. Roukema	Ballast Nedam Dredging
Ir. W. Molenaar	Ballast Nedam Engineering

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PREFACE

This is the final report of the feasibility study *'Buenos Aires Port Expansion 2020'*. It has been conducted as graduation project of my study Civil Engineering; section Ports and Waterways, at Delft University of Technology.

The aim of this study is to design a port expansion for the port of Buenos Aires, which offers sufficient container handling capacity for the port to operate safely and efficiently well into the first decades of this century.

This study has been made at the request of Ballast Nedam Dredging, because an expansion of the port of Buenos Aires could mean a possible feasible dredging project. A thorough study of this expansion gives them more knowledge about the possible risks and opportunities of these expansion works.

I would like to thank Prof ir. H. Ligteringen and ir. R. Groenveld from the section Ports and Waterways, Prof. J. Pinkster and ir. A. Versluis from Maritime Technology, ir. D.C. Roukema from Ballast Nedam Dredging and ir. W. Molenaar from Ballast Nedam Engineering for their support, help and advice during my graduation study.
Finally, I am very glad that my family, my friends and the people at Ballast Nedam Dredging have been very interested in my graduation project and have supported me as much as they could.

Rolf D. van den Bosch
Zeist, May 2000

SUMMARY

Economic outlook and capacity of the port

The port of Buenos Aires has known a large growth of container throughputs in the last few years: in 1991 233,000 TEU were handled, in 1995 638,000 and in 1998 over 1.1 million TEU were handled by the container terminals in Puerto Nuevo and Dock Sud. The end of this growth is not expected soon and various growth scenarios result in a throughput range of 4.2 to 6.6 million TEU in 2020. The current capacity of the port amounts approximately 2.0 million TEU, which is sufficient to operate till 2004 – 2005. To meet the future throughput demands, the capacity of the port must be increased.

There is no space for new terminals in the existing port and the capacity can be increased either by building new terminals or by reorganisation of the old container terminals. This study is restricted to a port expansion, where new terminals are constructed on reclaimed land in the Rio de la Plata, although reorganisation of existing terminals can be beneficial in technical and operational aspects.

The port of Buenos Aires will have to deal with more and larger ships in the future and the existing port is badly equipped to do so because:

- the Rio de la Platá is shallow and access to the port is by means of a long dredged access channel with insufficient depth for large deep draft ships
- the basins in the port are shallow and narrow
- quays are bended and curved

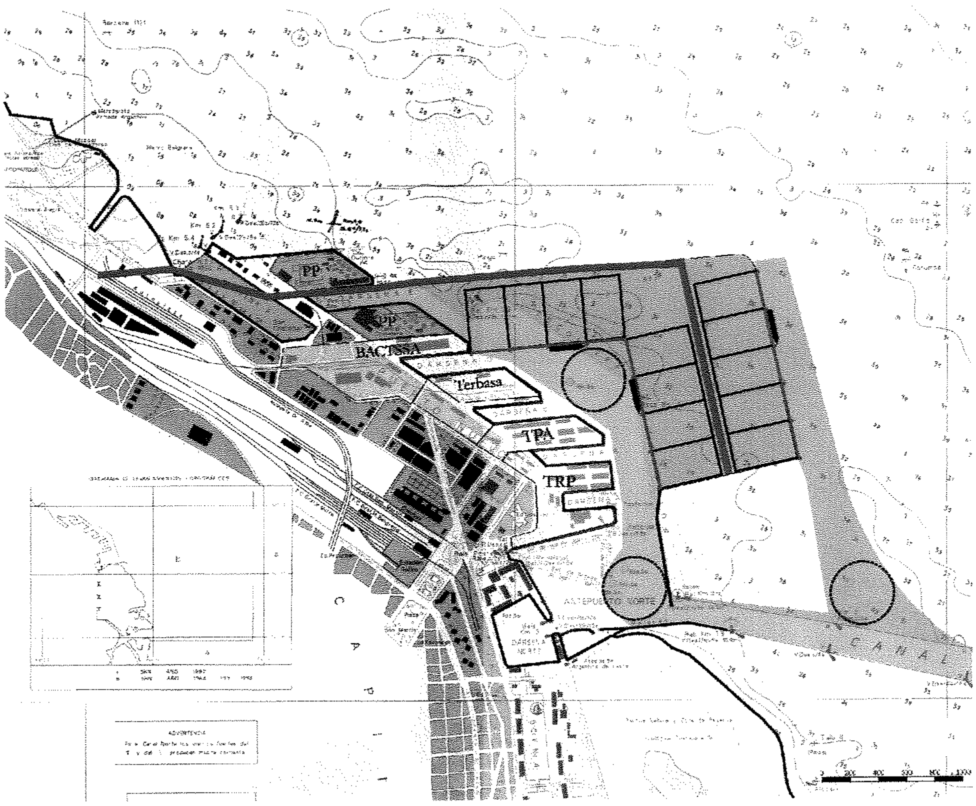
Aim of this study

The main goal of this study is to design a modern, safe and efficient port expansion, which has the necessary capacity to handle the containers coming to Buenos Aires within the next 20 years.

Besides that, the downtime of the container terminals is calculated, a possible construction phasing is determined, a cost estimate is made and the feasibility of the project is analysed.

Design of port expansion

A port expansion has been designed to meet future capacity demands, when a medium growth scenario is adopted (5.7 million TEU in 2020). In the chosen design, presented in the figure below, container ships will berth at both sides of an artificial peninsula. The old port will remain sheltered, but the new terminals on the eastern side of the peninsula unprotected and directly exposed to wind, waves and currents.



Downtime
The terminals on the eastern side of the peninsula are directly exposed to waves, wind and currents and this could have some consequences for the workability of these terminals. A downtime analysis for the port has been made to measure both this impact and that of other parameters:

- channel depth
- wind, waves and currents
- traffic intensity
- wave induced motions of moored ships

From this analysis, it is concluded that the access channels and port basins will be dredged to CD -13.2 m and that no unacceptable downtimes or waiting times will occur for the terminals of the port expansion.

Quay wall
A preliminary design has been made for the quay wall of the new container terminal, because it is one of the most important and expensive parts of the expansion. The designed quay structure consists of a combiwall, anchored by means of tie-roads and an anchorwall.

Construction of the port expansion
The port expansion can be constructed in 3 phases. The construction of phase 1 must start at the beginning of 2002 to prevent capacity problems in the port. In this phase, four terminals are built on the port side of the island, the access channel is widened and deepened and the connection from the peninsula to the main land is made. Phase 1 is completed in 2004. The second phase can start in 2009, in which another four terminals are constructed on the port

side. In the third phase, starting in 2014, five terminals are constructed on the eastern side of the peninsula, together with a new access channel to these berths.

The sand, which is needed for the reclamation is dredged from various near-shore sand pits by a deep suction dredger. However, before the sand layer is reached a large amount of soft material will have to be dredged by cutter suction dredgers. The soft material from the first sand pit is taken offshore by trailing suction hopper dredgers; soft material from other sand pits is dumped directly into depleted sand pits.

Feasibility

The port expansion requires an investment over the years of US$ 671 million: US$ 382 million for the capital works (reclamation, quay walls, roads and dredging of channels and basins) and US$ 289 million for maintenance of the port up to 2020. A discounted cash flow calculation shows that with an annual governmental subsidy of US$ 15 million the port expansion has an internal rate of return of 20% until 2020, making the port expansion a financially feasible and attractive project.

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PART 1:

INTRODUCTION AND FORECAST OF CONTAINER TRAFFIC

1. INTRODUCTION TO ARGENTINA AND THE PORT OF BUENOS AIRES

1.1 INTRODUCTION TO ARGENTINA

1.1.1 Geographic location of Argentina

Argentina is situated on the east coast of South America's tail. The country borders Chile to the west and Bolivia, Paraguay, Brazil and Uruguay to the north and east. It covers an area of about 2,767,000 square km[1], which makes Argentina the 8th largest country in the world.

Because of the size of the country its landscape is quite varied. It can be divided in four major physiographic regions:

- the Andes in the west, where arid basins, grape-filled foothills, glacial mountains and the Lake District can be found. The highest mountains are positioned in the northwest, where highlands (*punas*) are surrounded by mountains of 5000 to 6000 m in altitude
- the fertile lowland, *chaco*, in the north, with subtropical rainforests
- the central plains, the *pampa*, a flat mix of humid and arid expanses
- Patagonia in the south, a combination of steppes and glacial regions

Argentina has a population of 36 million, 12.7 million of which live in and around the capital of the country, Buenos Aires. Other big cities include Córdoba, Mendoza and Rosario.

Figure 1-1 Argentina and situation of Buenos Aires, Source: CIA Factbook

1.1.2 History of Argentina

Pre-Columbian Argentina was sparsely populated by Indian farmers and nomadic tribes, which used the plains as hunting ground. Indian resistance prohibited Spanish settlement at first, but Spain gradually gained control over the Rio de la Plata region. Buenos Aires was only established in 1580, but for 200 years, Argentina remained the least developed part of Latin America, as an outpost of the Peruvian sub-kingdom.

Buenos Aires became the capital of the new Vice-Royalty of the Rio de la Plata in 1776, when the region had outgrown Spain's economical and political domination. The

[1] That is 80 times the size of the Netherlands or one third of Brazil

Indian population, living in the *pampa*, had been put to work on huge cattle ranges (*haciendas*), which were the origin of the legendary *gauchos*. Trade was liberalized in the Spanish kingdom and the export of hides flourished.
Although Argentina enjoyed some sort of independence, dissatisfaction with the continuing Spanish interference led to the revolution of 25 May 1810 and eventually complete independence in 1816. Despite this unity, Federalists advocated provincial autonomy, but the Unitarists upheld Buenos Aires' central authority. After decades of dictatorship and political turmoil, eventually the Unitarists prevailed, which led to the constitution of 1853.
The export of cattle and hides remained the most important for economic and political development for years, but it was joined by the export of wool and grain in the second half of the 19th century. Liberalisation of the economy attracted foreign investors and immigrants. Between 1870 and 1914 around 6 million European immigrants came to Argentina, having heard the possibilities of having an own hacienda on the *pampa*. Many of them were disappointed, because most of the *pampa* was controlled by a very few.

Nevertheless, Argentina made an enormous economic progress in the second half of the 19th century and by 1910, Buenos Aires had become the largest and most modern city in Latin America. With mainly British investments port facilities and the railway network had been expanded in such a way that the products from the *pampa* could easily reach one of the ports. When World War I (1914-1918) started in 1914, Argentina had become Europe's main supplier for meat, grain, hides and wool. Argentina was as wealthy as Germany and the Netherlands and the average income per capita was higher than that of many European countries.

All the foreign investment however, made Argentina vulnerable to world economic downturns and the crisis in the early thirties hit very hard. Following the crisis, a weak civilian rule, high unemployment rates, economic failure and continuing resentment of the controlling elite led to political unrest and various military coups. During World War II (1939-1945), the export of grain and meat declined and unemployment rose even further. The labour unions got very powerful and Juan Perón won the presidential elections in 1946. He initiated a stringent economic program, which focused mainly on domestic industrialization. Unfortunately, this economic program led to high expenses for the government and inflation and unemployment rates increased even more. After a three-day civil war in 1955, Perón resigned and was banned from the country.
A period of unrest began, in which military rule was only interspersed by brief periods of civilian rule. Economic stability was only regained by means of foreign loans. Perón returned briefly in 1973, but when he died a year later political and economic conditions deteriorated rapidly. With another coup in 1976 a military group gained control and launched its own terror campaign against its opponents. Uncontrolled economic liberalization and political repression led to severe violations of Human Rights, hyperinflation, the total degradation of the national industry and severe poverty.
When in 1982 economic deterioration and popular discontent brought mass demonstrations a desperate General Leopoldo Galtieri, president at that time invaded the British-controlled Falkland Islands. The occupation of the Falkland Islands unleashed a wave of nationalist euphoria, but Argentina's ill-trained and poorly motivated forces soon surrendered.
In 1983, the military retreated from the government and Argentina returned to its constitution of 1853, burdened by an enormous international debt and hyperinflation. The Argentines elected Raul Alfonsin in the first presidential elections in a decade.
He reorganised the army, charged former leaders with Human Rights abuses, restructured foreign debt and introduced economic reforms. However, he could not

put an end to the inflation. His successor, Carlos Menem, was inaugurated in 1989 and he imposed an extensive austerity program.

Developments under Carlos Menem

When Menem took office in 1989 the economic situation was still a mess, with hyperinflation at 5000%, supermarket lootings and strike chaos.
He linked the peso to the dollar, which reduced the hyperinflation, he imposed free market policies to the economy, the nation's debts were rescheduled to commercial banks and state enterprises, like the ports, railroads and waterways were deregulated and privatised.
The economy of Argentina has grown 50 percent since 1990, but the free market policies caused serious social problems. Poverty and inequality increased, corruption rose and today unemployment rates are twice as high as in 1989, when Menem started.
In the recent elections of October 1999, the centre-left Alliance of Fernando de la Rua , the former mayor of Buenos Aires, defeated the peronist party of Eduardo Duhalde (and Menem) He is planning to make an end to corruption, frivolity, poverty and unemployment (*CNN & Reuters, october 1999*)

Consequences of privatisation

The privatisation of the government enterprises has caused some difficulties, which are important for the development of the port of Buenos Aires.
In the past the ports were governed by one federal authority: Adminstracion General de Puertos (AGP). But after the privatisation, several ports are managed by different authorities. For example: the port of Buenos Aires has been split into two parts: Puerto Nuevo and Dock Sud. Puerto Nuevo is managed by AGP but the land belongs to the city of Buenos Aires. Dock Sud in the south of Buenos Aires is managed by the province of Buenos Aires and the land belongs to the province. So both ports are managed by completely different authorities under completely different rules.
The privatisation of the railroads has resulted in several railway companies, which are all focussing on passenger transport. These companies have priority to use the tracks, making cargo transport by rail almost impossible.

1.2 Introduction to the Port of Buenos Aires

1.2.1 Location of the Port of Buenos Aires

The port of Buenos Aires is situated in eastern Argentina, 250 km from the mouth of the Rio de la Plata (see Figure 1-3), at 34.36 degrees south and 58.22 degrees west. The port extends approximately 12 kilometres facing the city of Buenos Aires along the river bank. The port of Buenos Aires consists of three main parts, Puerto Nuevo in the north, south of that Puerto Madero and Dock Sod in the far south. Puerto Madero is not used as a port anymore, but as an urban area (a beautiful area with old warehouses etc.). It is of no importance of the port expansion and will not be mentioned any further. (see Figure 1-2).

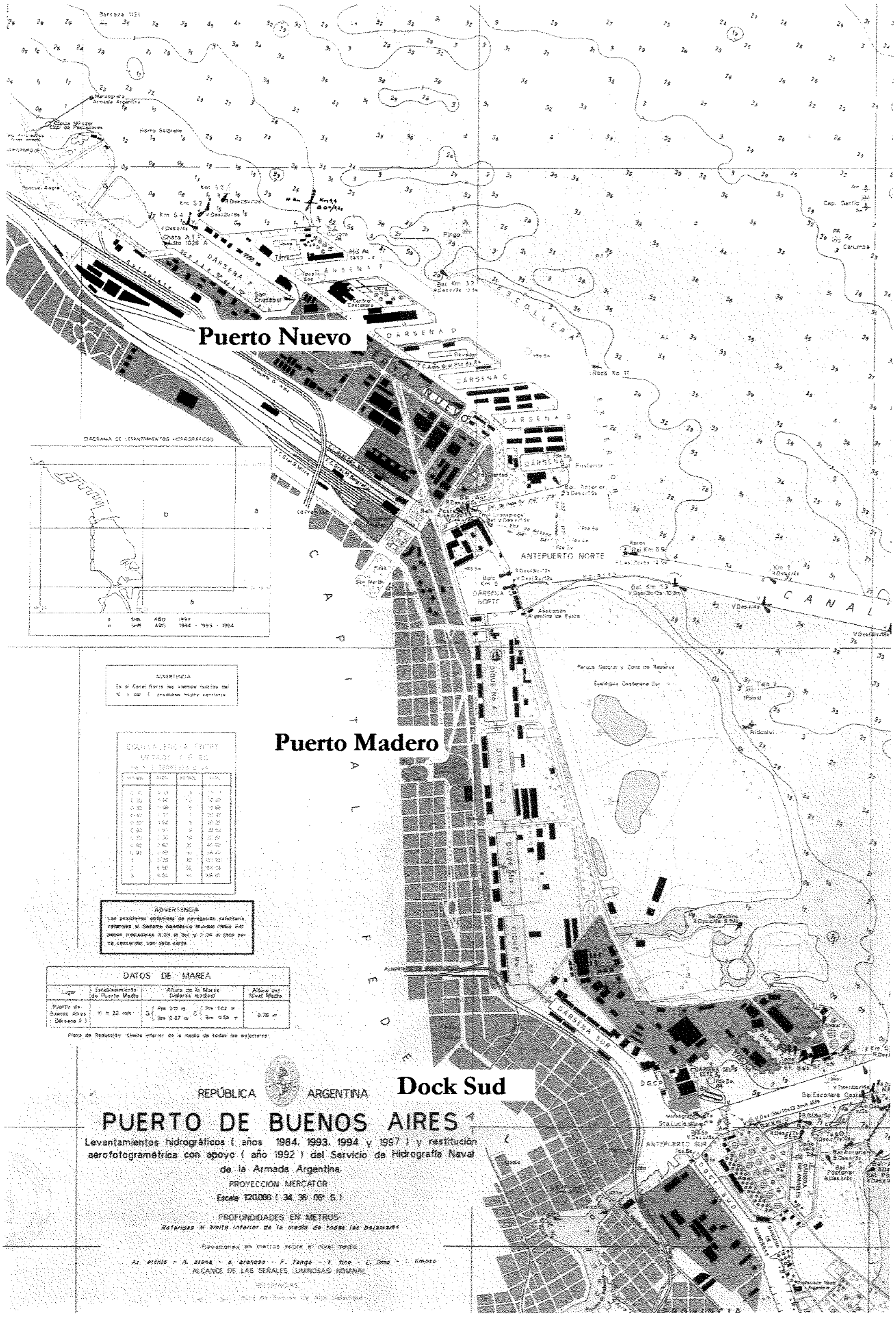

Figure 1-2 Port of Buenos Aires

1.2.2 Entrance to Buenos Aires

The Rio de la Plata is a shallow estuary with depths mostly ranging from 3 to 5 metres below Chart Datum[2]. Entrance to the various ports for ocean vessels is therefore by

[2] Chart Datum (CD) is defined as the level of the river Riachuelo (near Dock Sod)

means of dredged channels through the Rio de la Plata. Sailing from the Atlantic Ocean, until the *Recalada* lighthouse, 30 km south of Montevideo and 208 km from Buenos Aires, the navigational depth is more than 10 metres. From there *Canal Punta Indio* runs to an intersection 37 km from the port. *Canal de Accesso al Puerto de Buenos Aires* starts here and a *Canal* Principal goes up north in the direction of the Rio Uruguay and Rio Uruguay. *Canal de Accesso* continues for 25 km WNW where it splits; the *Canal Emilio Mitre* goes NW up the *Rio Paraná* to Rosario, *Canal Sud* goes SW to Dock Sur and *Canal North* continues WNW to Puerto Nuevo.
For the design of the port expansion *Canal Punto Indio*, *Canal de Accesso* and *Canal Norte* are the most important.

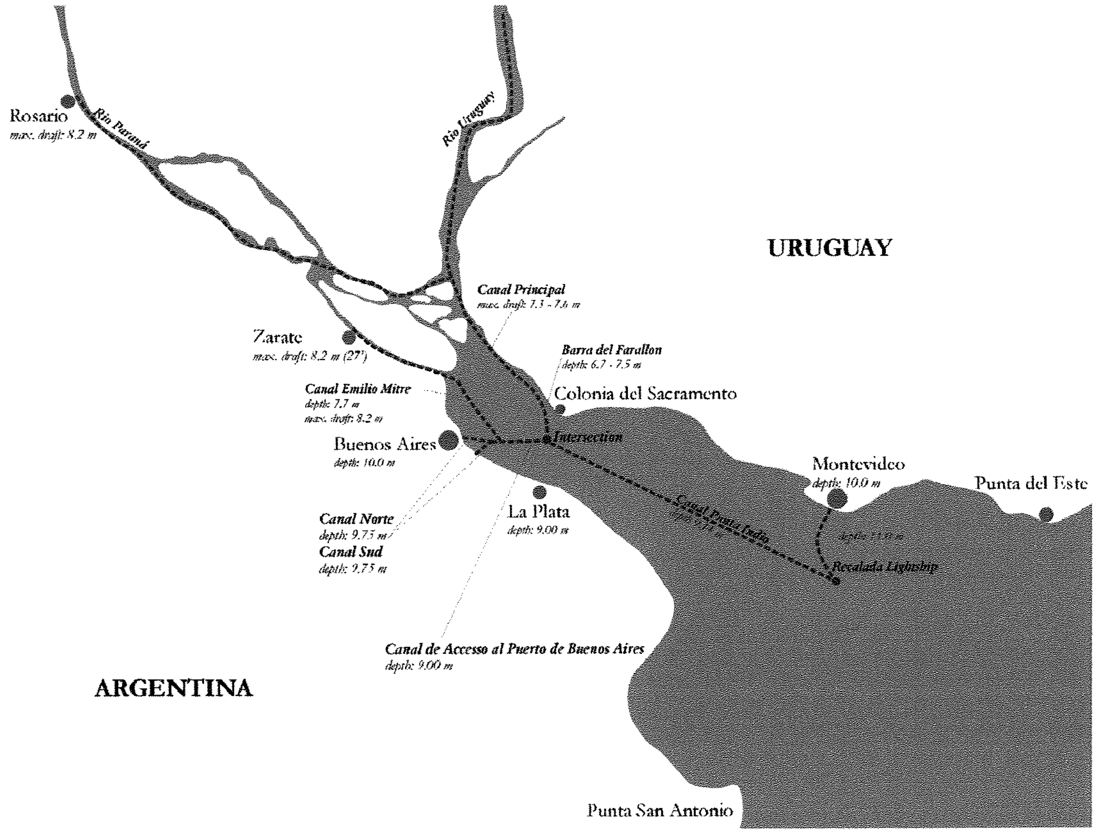

Figure 1-3 Channels in the Rio de la Plata, access to Buenos Aires, scale: 1 cm = 39 km

The dredged depth of the access channels are shown in the table below:

Channel name	Dredged Depth	Max. Draft Allowed
Canal Punta Indio	9.75 m -CD	-
Canal de Accesso	9.75 m -CD	-
Canal Norte	9.75 m -CD	-
Canal Sud	9.75 m -CD	-
Canal Emilio Mitre	9.75 m -CD	10.97 (36 ft)
Note 1: water levels are subject to considerable change due to wind variations. Values are approximates		

Table 1-1 Depths of Channels in the Rio de la Plata,
Source: Anuario Portuario y Marítimo 1999

1.2.3 History of the Port of Buenos Aires

The first activities in the port of Buenos Aires are recorded at the beginning of the 16th century, when the Spaniards arrive in South America. At the mouth of a small river, the Riachuelo, a small fortress with a small harbour is made, which is used as a mooring place for colonization expeditions.
After the destruction of the fortress by Indians, Juan de Garay founds the port of Buenos Aires (Puerto de Santa María del Buen Ayre) again at approximately the same place in 1580. The port is then heavily defended to withstand Indian attacks and at first its character is more military than civil. The first real berths are constructed early in the 17th century, but for 200 years the port does not grow much. This situation changes when Buenos Aires becomes the capital of the Vice-Royalty of the Rio de la Plata in 1776. Following the expansion of hacienda's on the *pampa* trade grows and mainly hides are transported through the port, making it the main port of Argentina. When Argentina becomes independent of Spain in 1816, trade is liberarized and the port is opened to free trade. Argentina sees an enormous economic expansion at the end of the 19th century and most of the products from the *pampa* (hides, meat, grain and wool) are shipped all over the world from the port of Buenos Aires.
In the 19th century a passenger terminal is opened and a better connection with the city of Buenos Aires is made. Because 3 piers are not sufficient to accommodate the growth of the port, an expansion of the port is ordered in 1882; 4 locked docks are built on the riversides. The first ship enters the new Puerto Madero in 1889. Because of the huge growth in transport of grain (in 1910 30,000 tons are transported per day), Buenos Aires becomes a very important port, but it lacks the necessary capacity to meet the international commercial needs. Therefore, in 1911 works start on what would later become Puerto Nuevo. These works, completed in 1926, turn Buenos Aires into the first large port of Latin America.
Shortly after, the crises of the 1930's affect Argentina so badly that trade volumes decline, inflation rises and no more investments are made in the port for a long time.

1.2.4 Containerisation in Buenos Aires

Due to the lack of investors, political unrest and low trade volumes, the port of Buenos Aires has only had minor adjustments after the completion of Puerto Nuevo. In the last two decades however, international maritime trade has dramatically changed in volume and character (see Chapters 3 and 4). Not only the dimensions of vessels have increased a lot but also most of the cargo, which used to be shipped in bulk or as general cargo is nowadays shipped in containerised form.

The conventional general cargo berths started to handle containers as well, in order to adjust to these changes. However, the government owned port was not operating very efficiently and new developments were difficult to initiate. Therefore the Argentine government, being the first in South America, commenced a privatisation process in 1991 for all its ports. For the port of Buenos Aires this process of deregulation and privatisation ended in 1994.
It resulted in two ports for Buenos Aires, Puerto Nuevo in the North, still under the authority of AGP and Dock Sud in the South, under the authority of the Province of Buenos Aires. Of these two ports, Puerto Nuevo is the general cargo / container port and Dock Sud is more focussed on bulk goods.
After the privatisation, 5 concessions to operate the terminals for a period of 18 to 30 years were granted in Puerto Nuevo to the highest bidders, mostly alliances of multinationals and Argentine companies.

Of these terminals, two are container terminals (terminals 1&2 and 5), one started a container/ro-ro operation (terminal 3), one is a multi purpose terminal (terminal 4) and one failed to start (see Table 1-2).

Terminal	Operator	Type
PUERTO NUEVO		
Terminal 1 & 2	Terminales Rio de la Plata (TRP)	Containers
Terminal 3	Terminales Portuarias Argentinas (TPA)	Multipurpose: containers, roro, cars, general cargo
Terminal 4	Gabriel y Cia / ENCYM	Multipurpose, grain
Terminal 5	Buenos Aires Container Terminal Services SA (BACTSSA)	Containers
Terminal 6	Inoperative	-
TERBASA	Terminal Buenos Aires SA (TERBASA)	Grain
DOCK SUD		
Exolgan	Exolgan	Containers

Table 1-2 Terminal operators in the Port of Buenos Aires and Dock Sud

In 1995, a new container terminal started its operations in Dock Sud, the port situated just south of Puerto Madero in the Province of Buenos Aires (see Table 1-2). This had important consequences; firstly, the new terminal Exolgan had been awarded a concession on more beneficial terms compared to the concessions granted in Puerto Nuevo. The other consequence was the creation of two adjacent container ports in Buenos Aires, either of them operating under entirely different terms and the latter enjoying an unfair competitive advantage. The terminal operators of Puerto Nuevo had quite a set back in the first years of Exolgan's operation, but due to the rapid growth in container throughput volumes they do not suffer much today.

1.3 CARGO THROUGHPUT IN BUENOS AIRES TODAY

The port of Buenos Aires, and especially Puerto Nuevo, is the largest general cargo and container port of Argentina[3]. The container terminals of Puerto Nuevo and Dock Sud handle about 98% of the country's container traffic.
The contribution of Exolgan, the container terminal in Dock Sud to the total container throughput of Buenos Aires is quite large (28 % of 1998 container traffic), but for future port throughputs its influence is not very important as new berths of the port expansion will be constructed near Puerto Nuevo. Therefore, for the distribution of cargo per type, only figures for Puerto Nuevo will be used.
Especially the container throughput in the port of Buenos Aires has known a continuous growth during the last years. To illustrate this, the cargo throughput in tons per type of cargo for 1995 – 1998 has been shown in Figure 1-4.

[3] Buenos Aires has even been the largest container port of South America the last few years

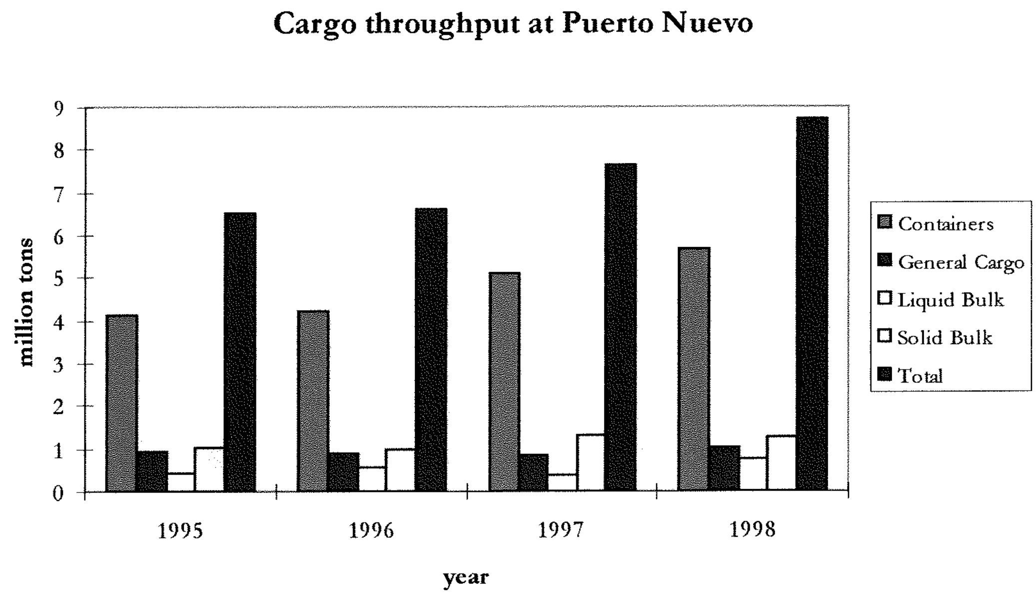

Figure 1-4 Throughput of Puerto Nuevo 1995-1998 (in million tons)

The distribution of cargo types as a percentage of the total throughput is shown in Figure 1-5. Figures for containers can be found in Appendix A.1.

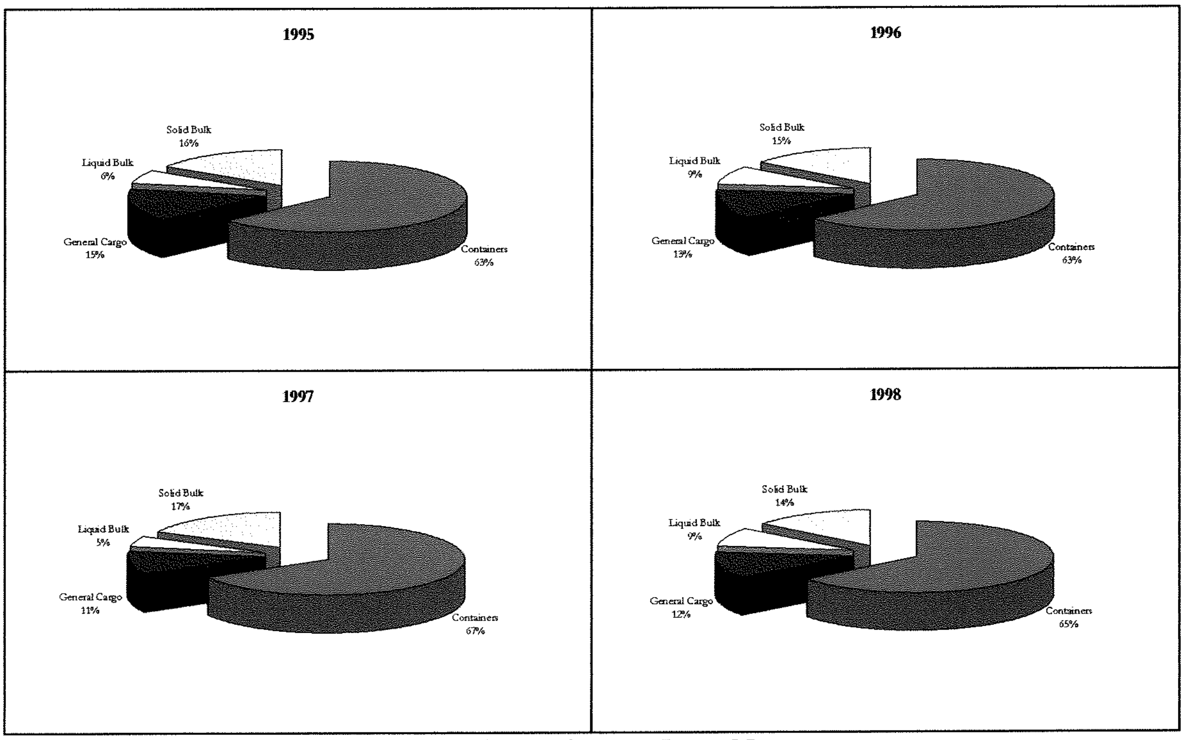

Figure 1-5 Distribution of cargo in Puerto Nuevo

65% of the cargo handled in Puerto Nuevo is containerised and this containerisation rate is expected to grow to about 70% of the total cargo load. The cargo handled in Puerto Nuevo is mostly domestic freight. Most of the hinterland transport is carried out by truck, because of the low freight rates for this type of transport. Although the railway network is well developed, almost no freight is transported by rail, as rail transport is mainly focused on passenger transport. Only a slight volume of cargo is transhipped to serve ports upstream in the Rio Paraná for mainly bulk cargo. Terminal 1 & 2 already tranship some containers to barges, to serve upstream ports. The volume of this transhipment is still negligible, but it is growing. When future container terminals are developed upstream, more transhipment is expected, but its share will remain small.

In Table 1-3 the development of container traffic in Buenos Aires (Puerto Nuevo and Dock Sud) is shown, expressed in TEU.

TEU stands for Twenty feet Equivalent Units, which is an international standard length unit for containers (6,10 m). Other standard length measures are 40' (12.19 m), which accounts for 2 TEU and 45' (2.25 TEU and called over-sized containers) . The standard width for containers is 8' (2.44 m) and the standard heights are 8' and 8'6" (2.59 m). This does not mean that other dimensions do not exist, on the contrary: especially in North America more measures are found.

Year	Puerto Nuevo	Dock Sud	Total
1991	233,002	-	233,002
1992	267,228	-	267,228
1993	448,219	-	448,219
1994	532,681	-	532,681
1995	504,630	133,645	638,275
1996	530,346	249,208	779,554
1997	720,247	307,910	1,028,157
1998	818,334	319,286	1,137,620

Table 1-3 Container throughput in Buenos Aires, including Dock Sud, expressed in TEU's, Source: AGP

1.4 SHIPS CALLING AT THE PORT OF BUENOS AIRES

1.4.1 Number of ships

The majority of ships calling at Puerto Nuevo and Dock Sud are nowadays containerships. Looking at the ships arriving in Puerto Nuevo in 1998, about 80 percent of them were carrying containers (no data available for Dock Sud). Others were carrying grain, other types of solid bulk, liquid bulk, break bulk or cars. In 1998, a total of 1961 ships called at Puerto Nuevo, an average of 5.4 ships per diem.

1.4.2 Destinations

Buenos Aires is usually at the end or beginning of shipping routes, because of its southern location and shallow draft. On various shipping routes Buenos Aires is a scheduled port of call, but the three most important are: one east-west line to Africa and Asia and two north-south lines, one of which is going to Europe and one of which is calling at the ports on the East Coast of North America. All of the lines stop at other South American and Caribbean ports as well. The main destinations are:

Ports of call in South America and Caribbean

Brazil:	Santos, Rio de Janeiro, Rio Grande, Paranagua, Sao Francisco du Sul
Bahamas:	Freeport
Venezuela:	Puerto Cabello
Jamaica:	Kingston
Uruguay:	Montevideo
Neth. Antilles:	Curaçao, Aruba
Mexico:	Vera Cruz, Altamira
Trinidad	

Ports of call on North American line:

Canada:	Montreal, Toronto
North East America:	New York, Baltimore, Charleston, Miami, Norfolk, Philadelphia, Savannah
Gulf of Mexico	New Orleans, Houston

Ports of call on European line:

Netherlands:	Rotterdam
Belgium:	Antwerp
France:	LeHavre, Marseilles
Germany:	Bremerhaven, Hamburg
Portugal:	Lisbon, Leixoes
Spain:	Algericas, Bilbao
United Kingdom:	Tilbury, Felixstowe, Thamesport

Ports of call on East Asia line:

South Africa:	Cape Town, Durban
Taiwan:	Kaohsuing
Singapore	
Hong Kong	

1.4.3 Dimensions of ships

The depth of (the access channels to) the port of Buenos Aires is not very large 32 ft (9.75 m) and consequently deep draft ships are not able to enter the port. Traffic volumes have not been very big in the past and shipping lines are not able send their biggest ships to Argentina. The largest ships calling regularly at Buenos Aires, are ships operated by shipping line Evergreen. The company is operating their G-type of fully cellular containerships on this line, which have a capacity of 2,728 TEU, a length (LOA, length over all) of about 230 m, a beam of 32.2 metre and a draft of 11.62 metres. These ships cannot enter the port of Buenos Aires fully loaded, but after calling at some other South American ports, they can make the voyage to the Far East fully loaded. When the port and its access channels are dredged to 36 ft (10.95 m), for which plans have been announced by AGP, larger Panamax ships of up to 4,000 TEU could enter the port. Dredging to 40 ft (12.2 m) would even allow some Post-Panamax ships (up to 5,000 TEU) to enter the port partially loaded.
The largest main line ships with capacities of around 8,000 TEU are not able to enter the port of Buenos Aires, due to their average draft of 13 - 14 m. Even larger ships (up to 15,000 TEU), which have been announced and are planned to sail the globe within 10 years, will be able to call at only a few ports in the world and will not call at Buenos Aires. More realistic for Buenos Aires is to focus on Panamax ships and the smaller, shallow draft Post Panamax ships with capacities as mentioned above.

1.5 Terminals and capacity of Buenos Aires

1.5.1 Container terminals in Buenos Aires

In Buenos Aires 7 major terminals are present, see table Table 1-2. Three of them are solely container terminals (TRP, BACTSSA and Exolgan) and one terminal (TPA) handles some other goods as well. All container terminals are using transtainers[4] for stacking, but they also use forklifts and frontloaders.

Terminales Rio de la Plata (TRP)

[4] Rubber Tyres Gantry cranes, see chapter 2

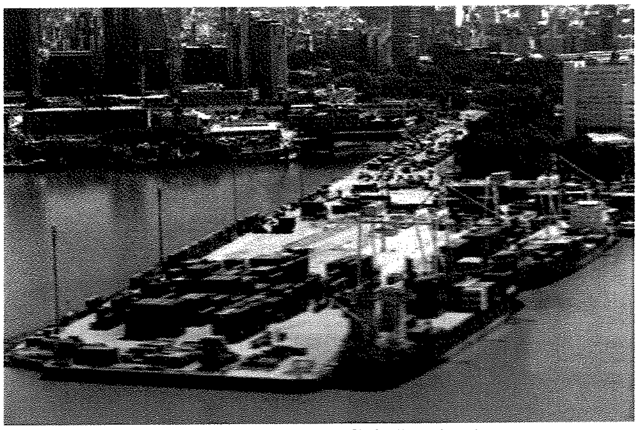

Figure 1-6 View of TRP, pier 1

Terminales Rio de la Plata, or TRP is operating piers 1 and 2 on Basins A&B. It is the largest dedicated container terminal in Puerto Nuevo.

The company is a subsidiary of P&O ports, and has a concession to operate piers 1 and 2 for 25 years. After the company started in 1994 it has completely remodelled piers 1 and 2 to adjust the terminal to container handling operations. In 1997, about 380,000 TEU were handled, both on ocean vessels as on barges. The available quay length is 980 meters with a draft of 32' alongside and the company states that after the development of stage 2 it will have a capacity of about 1 million TEU.

For container operations, TRP has 5 ship to shore gantry cranes

Terminales Portuarias Argentina (TPA)

TPA is a multipurpose terminal owned by Mi Jack from the Unites States, ATA (a local company) and the International Finance Corporation. After they were granted the 25-year concession to operate pier 3 on the north of Basin B and the south of Basin C, they incorporated TPA and started business on november 1994. The terminal has a total quay length of about 1,350 m, with a depth of 32 ft. alongside, some of which cannot be used efficiently, because the area behind it is rather small (total area: 16 hectare).

The terminal is a multipurpose terminal, which focuses on general cargo, roll on roll off vessels, car vessels and container vessels. In 1997 about 94,000 TEU were handled. For container operations, TPA has 2 Post-Panamax container cranes, positioned alongside Basin C.

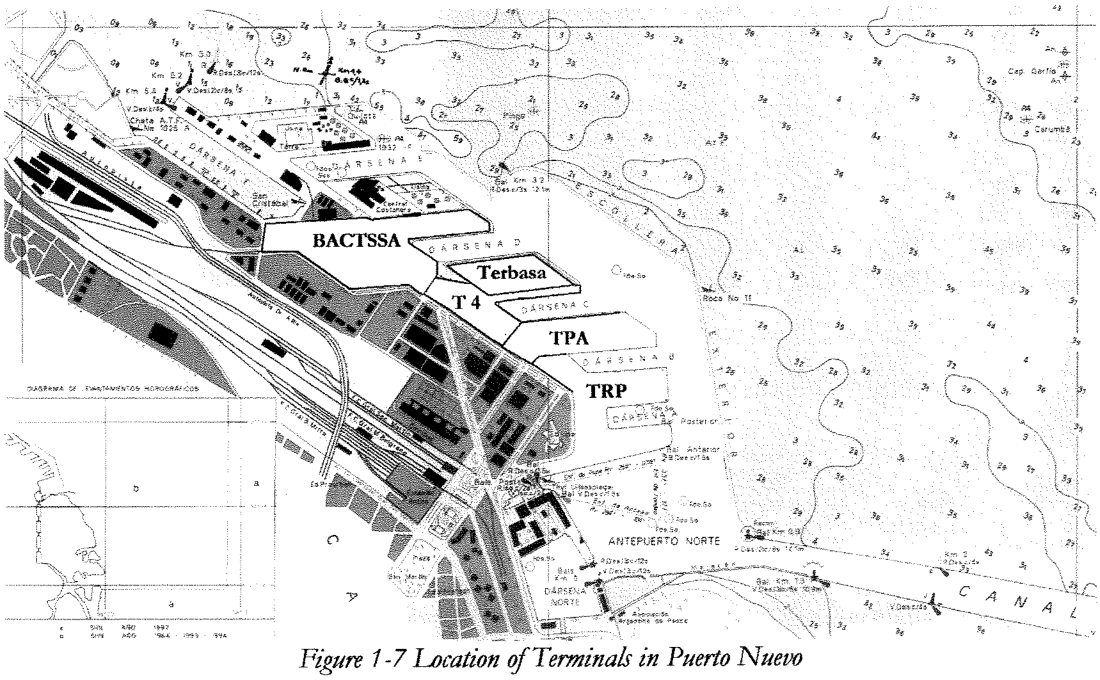

Figure 1-7 Location of Terminals in Puerto Nuevo

Ballast Nedam Dredging

Buenos Aires Container Terminal Services S.A. (BACTSSA)
BACTSSA is the second dedicated container terminal in Puerto Nuevo and it has a concession to operate pier 5 on the north of Basin D for 18 years. It is a consortium, consisting of ICTSI (a Philippine terminal operator) and Bemberg Inversiones (an Argentine investing company). After starting business in 1994, it completely reorganised its 21.5 hectares and purchased 3 ship to shore Panamax gantry cranes. On its 885 meters of quay (depth 31'), it handled about 242,000 TEU in 1997 and the company states it has an annual capacity of 450,000 TEU.

Exolgan
Exolgan is the second largest container terminal in Buenos Aires, a joint venture of an Argentine company International Trade Logistics and German HHLA. It is not situated in Puerto Nuevo, but in *Dock Sud*, where it started business in 1995. It got a concession on beneficial terms compared to terminals operation in Puerto Nuevo, so it could undercut the prices of those operators by 40%. It got a large share of the containers passing through Buenos Aires. The terminal has an area of 45 hectares and a quay length of 1,000 meters, 700 of which were operative in 1997. The depth alongside its berths is 32 ft. The terminal handled about 320,000 TEU in 1997, with 3 container cranes. The terminal has 4 Post-Panamax container gantries today and its capacity is estimated to be 500,000 TEU.

The distribution of the 1997 container traffic between the various terminals is shown in Table 1-4

Terminal Operator	TEU (1997)	Area (ha)	Quay length (m)	TEU/ha	TEU/m
TRP (T 1&2)	383,322	28.5	980	13,450	391
TPA (T 3)	94,623	16.0	1,345	5,914	70
BACTSSA (T5)	242,302	21.5	885	11,270	274
Exolgan (Dock Sud)	307,910	45.0	700	6,842	440
Total / Average	1,028,157	111.0	3,910	9,690	256

Table 1-4 Throughput and benchmark values of Buenos Aires' terminals in 1997

As can be seen from the table above, Terminal 3, the multipurpose and ro-ro terminal, is not operating very efficiently in terms of container throughput per metre quay. Handling of containers is not the core business of this terminal; it is after all a multipurpose terminal. It does not have a large container yard, but uses it efficiently. Exolgan is performing better than average looking at the number of containers handled per metre of quay, because it is better situated, not on fingerpiers, and it does not have surplus quay space.
The annual number of handled TEU per hectare and per metre have been stated as a benchmark of the port for comparison purposes to other (similar) ports in the world. The data for this comparison can be found in Appendix A.

1.5.2 Capacity of the container terminals in Buenos Aires

The estimated capacity of terminal 3 is unknown and the estimated capacity of TRP seems a bit optimistic, but the annual capacity of the terminals in Buenos Aires has been estimated to be 2.0 million TEU after all, when all terminals have been optimised. The capacity of Dock Sud will be about 500,000 TEU and of Puerto Nuevo about 1,500,000 TEU. Exolgan in Dock Sud and BACTSSA are at the moment operating near capacity today and it won't be long until all terminals will reach their capacity, see chapter 3.

1.5.3 Comparison with other ports: benchmarks

To compare different ports, three benchmark values have been calculated for the different terminals in the described ports:

- the annual throughput per metre of quay
- the annual throughput per hectare
- the annual throughput per berth

The values of these benchmarks are dependent on various parameters, some of which can be found in Appendix A. Others are difficult to obtain, but can be approximated. The most important parameters on which the benchmarks are dependant are:

- type of terminal; a dedicated container terminal is likely to have better performance than a multipurpose terminal handling some containers
- type of terminal operation; road chassis can be used if the terminals have a lot of space for parking the chassis and high density stacking is not needed. If higher density stacking is needed, forklifts and front-end loaders are usually used in low volume terminals; straddle carriers and transtainers are used in high volume terminals.
- shape of the terminal, strange shapes cannot be used for the stacking of loaded containers, however empties can be stored there.
- type and number of ship to shore cranes; mobile, floating and quay cranes are not adapted to handle lots of containers and will perform less than specialized ship-to-shore gantry cranes
- depth of access channels and basins; ports with both a large (natural) depth and high economic activity will attract a lot of cargo and large ships (Rotterdam, Hong Kong)
- dwell times for import, export and empty containers influence necessary storage space and the throughput per hectare dramatically. Dwell times can be influenced by adjusting the storage rates of containers after a few days.

The first two benchmarks are easily calculated, when the throughput figures for a terminal are available. The last benchmark however is quite difficult to calculate, because berth length is calculated in different ways in different ports. Values for the last benchmark have been stated in the appendix, but will not be mentioned in the following text. To make a fair comparison between terminals in different ports, one should look at dedicated container terminals, because comparing a general cargo berth to a container berth does not make sense. The ports described hereafter have been chosen, because there is some similarity to the (future) port of Buenos Aires or because it is very different from Buenos Aires.

Antwerp

The port of Antwerp is situated 75 km from the North Sea on the river Schelde. It is the 9th container port in the world and had a throughput of almost 3.0 million TEU in 1997. The Schelde is navigable for ocean going vessels, but some of the terminals of the port are situated in docks. Independently of tide, ships with drafts up to 12.2 metres can use the Schelde. The maximum draft is 14.5 metres, taking advance of rising tide.
Most of the terminals in Antwerp use straddle carriers for their operations within the terminal. Only the largest terminal is mentioned here, because most of the other terminals are multipurpose terminals. Hessenatie (762 TEU/m, 13,457 TEU/ha) handled 1.8 million TEU in 1997 and is a clear example of an optimised straddle carrier operation. The terminal, almost operating at maximum capacity achieves very good benchmarks for a straddle carrier operation (higher than the terminals in Hamburg).

Values for throughput per metre of quay and per ha are very high compared to European average and also higher than in Buenos Aires.

Hamburg

Hamburg is the 7th largest container port in the world and had a throughput in 1997 of more than 3.3 million TEU. It is situated on the river Elbe, 121 km from the open sea. Ships with drafts up to 12.3 metres can enter the port at any time. Even bigger ships, with drafts up to 14.7 metres can enter the port during a tidal window.

In Hamburg, the container terminals have been optimised for their purpose too and high benchmark values are found. The average throughput per ha is over 10,000 TEU for the specialized container terminals, which is comparable to Antwerp. When the benchmarks for Hamburg are compared to Buenos Aires, the annual throughput per metre of quay is higher for Hamburg, but the number of handled containers per ha is approximately equal. The difference in annual quay throughput can partially be explained by looking at the shape of the container terminals in Buenos Aires and Hamburg. In Buenos Aires, the container terminals are situated on fingerpiers, and the terminals have a surplus of quay length, which can not be used to moor ships on. The container berths in Hamburg are positioned in long straight lines, with container yards directly behind them. This guarantees a better use of the quay and a higher benchmark value. .

All the terminals in Buenos Aires are using the transtainer (RTG) type of operation, which enables higher stacking density of containers and thus higher throughputs per ha than the used straddle carrier operation in Hamburg. But the RTG-operation is not so efficient, due to the strange shapes of the container yards and because dwell times can be high in Buenos Aires, resulting in medium values for the throughput per ha for this type of yard, while on the other hand the straddle carrier operation in Hamburg has been optimised, resulting in high values for straddle carrier operations.

Manila

Manila was the 16th container port in the world in 1997. It is the main cargo port of the Philippines and had a throughput of 2.1 million TEU. For two reasons Manila is a good comparison for Buenos Aires: Firstly the two major container terminals in Manila have adopted the RTG system, and secondly one container terminal is situated on a new piece of land, adapted to modern container terminal operations, while the other terminal is situated on various fingerpiers, just like in Buenos Aires. When the two main terminals in Manila are compared, it can be concluded that the terminal situated on the fingerpiers is performing worse than the terminal with the optimised layout for container operation, just as is the case in Buenos Aires. The average throughput of the container yard is good in Buenos Aires, while the throughout per metre of quay is still low, compared to Manila.

Nagoya

The port of Nagoya, situated on the Honshu island in Japan, has only a slightly larger container throughput than Buenos Aires (1.5 million TEU). It has a similar depth (max. draft 12.6 m) and about the same area of container terminals as Buenos Aires. The higher benchmark values are achieved by the high rate of transhipment in Nagoya. Many containers pass the quay, but never go through the container yard, just like in Hong Kong and Singapore.

Oakland

Oakland is situated on the West Coast of the Unites States in the San Francisco Bay. It is the biggest North Californian container port and had a throughput in 1997 of 1.4

million TEU. The west coast of the USA has no natural deep-water access, and the maximum draft for Oakland is 11.5 m. American ports usually have quite a lot of space and terminals have a lot of parking space for trucks and chassis. The stacking density and thus the annual container yard throughput is therefore low, lower than in Buenos Aires. The quay throughput is about the same.

Rotterdam

The port of Rotterdam is one of the largest container ports in the world a main container hub port of Western Europe. Its access channels are very deep (18 metres and more) and for the largest containerships that sail the world, Rotterdam is a frequent destination. Rotterdam's largest and well-known container terminal ECT is an innovative and its terminal on the *Maasvlakte* had a throughput of 2.72 million TEU in 1997. The total throughput of Rotterdam in this year was 5.43 million TEU, which is of the same scale as the forecasted throughput of Buenos Aires in 2020, see chapter 2.
For Europe, the port of Rotterdam has good benchmarks values, comparable to Antwerp and Hamburg, but still not near the values of Asian terminals. Some of the container terminals on the *Maasvlakte* are dedicated terminals and have low berth occupancy.

Shanghai

Shanghai, China's major port is situated on the mouth of the Yangtze. It is one of the major container ports in the world, positioned 12th in 1997, and had an annual throughput of 2.5 million TEU. It is not a very deep port, maximum draft 10.2 meter and a comparison is made, because the Yangtze is discharging large volumes of sediment just as the Rio Parana, which makes dredging of deeper channels difficult. Shanghai's terminals are clear examples of East Asian optimised RTG terminals, such as those in Busan (see Appendix C), Hong Kong and Singapore (not in the Appendix). The benchmark values are very high compared with those of Buenos Aires, but not compared with the rest of Asia. The difference in the throughput per hectare can be explained by the short dwell times and large rate of transhipment in these ports.

Port	Terminal Name	Throughput (in 1,000 TEU/year)	TEU/m	TEU/ha
Buenos Aires				
	Exolgan	308	440	6,842
	TRP	383	391	11,270
	BACTSSA	242	274	11,270
Antwerp	Hessenatie	1,829	762	13,457
Busan	Jasungdae Container Terminal	1,808	1,210	27,922
Hamburg	HHLA	2,207	504	11,119
	Eurokai	908	478	12,972
Manila	MICT	907	698	9651
Nagoya	NCB	649	721	22,440
	Kinjo	279	348	15,839
Oakland	Sea Land	330	472	12,453
Rotterdam	ECT (Maasvlakte)	2,752	754	12,568
Shanghai	SCT	1,766	774	20,583
Yokohama	Honmuku Private	950	559	15,942

Table 1-5 Benchmark values in 1997 for different terminals in the world, based on Containerisation Yearbook 1999 data

2. PROBLEM DEFINITION

2.1 PROBLEM DEFINITION

As can be read in the following chapters, the world container port throughput is expected to keep on growing. Buenos Aires has the potential to grow faster than average, but lacks the capacity it needs to do so. The total annual capacity of the container terminals in Buenos Aires is estimated to 2.0 million TEU. Growth scenarios predict that in 2020 a throughput of 4.2 to 6.6 million TEU may be reached. However an expansion of the port of Buenos Aires is necessary to reach this goal.

2.2 AIM OF THIS STUDY

The aim of this study is

- to design a modern, efficient and safe container port expansion for Buenos Aires, which has the necessary capacity to handle the containers coming to Buenos Aires within the next 20 years.
- to determine in which phases the port has to be developed
- to determine the construction (dredging) method for the port expansion
- to evaluate the costs, profits and feasibility of first phase of the project
- to make a preliminary design of the quay wall

3. FORECAST OF CONTAINER TRAFFIC

Forecasts of future container throughput at the port of Buenos Aires cannot be made without looking at the economic situation of Argentina and the Rio de la Plata region. This chapter describes the economic situation of the region and the port of Buenos Aires in paragraph 3.1. Three cargo forecast scenarios for Buenos Aires' port will be described in paragraph 3.2.

3.1 ECONOMIC SITUATION OF THE RIO DE LA PLATA REGION

In the Rio de la Plata region and South Brazil, four large container ports can be found: Buenos Aires (Argentina), Montevideo (Uruguay), Santos and Sepetiba (Brazil). The majority of the container traffic in this region is international trade, but the share of Mercosur trade is growing fast. As both Mercosur and international traffic are growing rapidly, shipping lines are looking for a hub port, in which their large container Post-Panamax vessels can tranship their cargo to smaller feeder vessels.
Requirements for such a port are high efficiency and deep water access (14 meters at least). In the region no hub has been developed so far, because none of above mentioned ports in this region meet all the requirements. In the Rio de la Plata region (Buenos Aires and Montevideo) especially the deep water criterium is difficult to meet as the river is discharging very large quantities of sediment. The larger vessels could discharge some of their cargo first at ports in the South of Brazil and navigate with reduced draft to Buenos Aires. More likely, smaller vessels will navigate the Rio de la Plata region, and a hub port will grow in the South of Brazil where natural deep water access is available.
However, the Brazilian ports are not operating efficiently enough until now and are coping with huge labour problems.

Although the port of Buenos Aires is not likely to become a hub port, it will still be the main container port for Argentina, with its 36 million inhabitants. Therefore traffic quantities will still be very large, but the quantity of transhipment containers will be limited to those containers with destinations up the Rio de Uruguay and Rio Parana.

3.2 CARGO FORECAST SCENARIOS

To forecast the container throughput of the port of Buenos Aires in the coming years, various scenarios, based on different approaches have been opted. The scale and trend of cargo traffic in following scenarios are more important than the exact numbers. As can be seen in Table 3-4, the growth of container traffic will be large in the coming years and the capacity of the present container terminals is Puerto Nuevo and Dock Sud, which is estimated to be approximately 2.0 million TEU, will be reached around 2005.
The following growth scenarios have been developed for container throughput of both Puerto Nuevo and Dock Sud. They are based on: world economic development, world container port throughput forecasts, development of South American ports and rules of thumb.

3.2.1 Low growth scenario (Drewry)

In '*World Container Terminals*', published by Drewry Shipping Consultants, a world container port throughput up to 2005 is made. Their forecast is based on an analysis of world container trade and takes into account trade growth predictions, transhipment incidence and the anticipated effects of the Asian crisis. Despite downsizing of forecasts as a result of the recent economic crisis in the Far East and South East Asia, continuing growth is expected. The world container port throughput is expected to grow from 156 million TEU in 1996 (151 million TEU according to '*Containerisation International Yearbook 1999*') to 271 million TEU by 2005, approximately a 60% increase.
Global growth is predicted to slow down from 6.25 percent per annum in 2000 to 5.72 percent per annum in 2005 (see Appendix B.2). From the same figures can be derived that the growth rate in South America will be somewhat higher than the world average. South America accounted for 6.60 percent of the global throughput in 1996, 6.98 percent in 1998 and is predicted to amount 7.26 percent in 2000.
To make a long term forecast for world container port throughput, these figures have been extrapolated beyond 2005. Because nothing valid can be said of long term global economic development, due to occurring crises and revivals of economies, the 2005 global growth rate of 5.72% is maintained till 2020 (see Appendix B.2). The global container throughput forecast for 2020 with these growth rates is 625 million TEU. The share of South America will increase in time as container traffic on this continent is growing more rapidly compared to global average. The main reasons for this catching up are that the degree of containerisation is still low compared to developed countries, developments started later and that large scale improvements will continue for a longer time. South America's container traffic is estimated to amount about 7.76 percent in 2010, 7.88 percent in 2015 and 8.00 percent in 2020.
To estimate container throughput through Argentine ports, the throughput is assumed to be a percentage of South American traffic. According to the '*OKITA II*' report by JICA the Argentine throughput would eventually amount 8.19 percent of the Latin American region, but recent figures have proven that this is already 9.08 percent.
Using this percentage, the throughput through Argentine ports will amount about 4.6 million TEU per year in 2020 (see Table 3-1). Considering these figures, growth rates for Argentina are almost constant between 2005 and 2020, about 6.1 percent, not an exceptional long-term growth rate for countries with an already established container market. Because the growth rate in this scenario is the lowest of the three scenarios presented here, this scenario will be called the low scenario.

Year	Containers (TEU)
1998	1,137,620
1999	1,240,148
2000	1,338,246
2005	1,884,155
2010	2,681,711
2015	3,392,508
2020	4,550,522

Table 3-1 Argentine container throughput in TEU, low scenario

3.2.2 Medium growth scenario

This scenario is based on a rule of thumb often used to estimate future container throughput, which says that the growth in container traffic amounts about 2 to 3 times the growth of the GDP[5] (Ocean Shipping Consultants).
In developing countries large growth rates of GDP (sometimes more than 10 percent per annum) can be seen and the lower multiplier is used resulting in growth rates in the order of 15-20 percent per annum. In developed countries the higher multiplier is used. According to the Worldbank Argentina's GDP is estimated to grow with an average of 4 percent per annum from 2000 to 2005, so that the container growth rate will amount 12 percent. This is not an exceptional figure when considering the double digit figures over the past years (average over 1994-1998: 21 per cent) After 2005 year the growth is assumed to slow down to an average of about 6 percent over 2006-2020. Using the 1998 container throughput in Argentina (1.137 million TEU) with this scenario the throughput in 2020 is calculated to be 5.7 million TEU (see Table 3-2).

Year	Containers (TEU)
1998	1,137,620
1999	1,274,134
2000	1,427,031
2005	2,381,792
2010	3,187,375
2015	4,265,426
2020	5,708,102

Table 3-2 Container throughput in TEU, medium scenario

3.2.3 High growth scenario (OSC)

This scenario is based on a container throughput forecast presented by Ocean Shipping Consultants in '*World Container Port Market to 2010*' (see Appendix A). Although the figures presented in the report have been overestimated (in a new report, published in 1999 lower figures are used), the percentages for the Latin American region, used in their high growth scenario, have been quite right and correspond to the growth in 1998 in Argentina. This growth rate (10.5 percent) is estimated to last for a longer period (until 2010) than the 12 percent growth rate from the previous scenario. After 2010, growth has been extrapolated. As high growth rates seem unlikely to last forever, growth rates are expected to decline; over 2011-2020 the throughput growth is estimated to be 6 percent. The throughput in 2020 with this scenario will be 6.6 million TEU (see Table 3-3). Because of the longer period in which the high growth rate is used, the last scenario gives the highest number of containers in 2020 and will be called high scenario.

Year	Containers (TEU)
1998	1,137,620
1999	1,257,070
2000	1,389,062
2005	2,288,406
2010	3,684,733
2015	4,931,004
2020	6,598,796

Table 3-3 Container throughput in TEU, OSC scenario

[5] Gross Domestic Product

3.3 Summary of Growth Scenarios: Conclusion

Year	Drewry	Medium	OSC
1998	1,137,620	1,137,620	1,137,620
1999	1,240,148	1,274,134	1,257,070
2000	1,338,246	1,427,031	1,389,062
2005	1,884,155	2,381,792	2,288,406
2010	2,511,499	3,187,375	3,684,733
2015	3,278,938	4,265,426	4,931,004
2020	4,232,894	5,708,102	6,598,796

Table 3-4 Container throughput in TEU, all scenarios

Note: Because of the longer period in which the high growth rate is used, the OSC scenario gives the highest number of containers in 2020. Until 2005 the medium scenario gives higher throughput values.

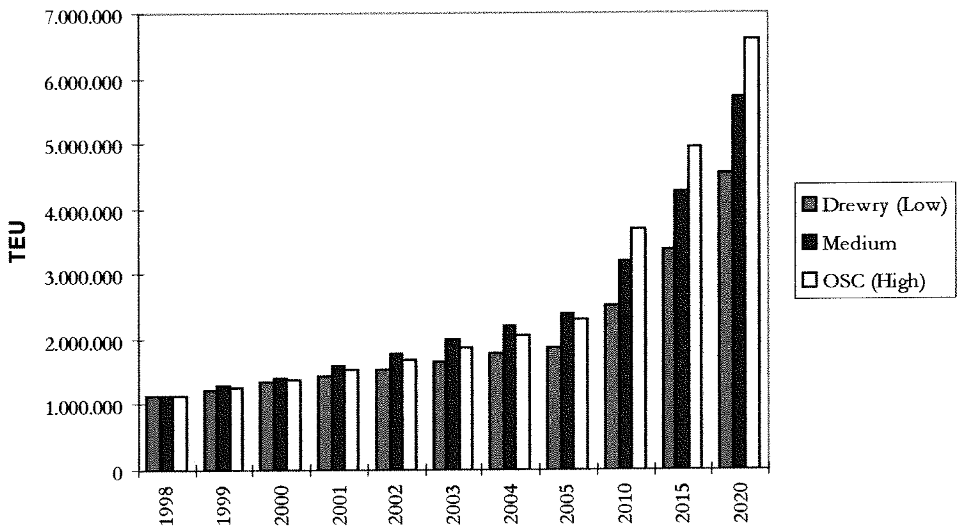

Figure 3-1 Container throughput in TEU, all scenarios

As can be seen from the growth scenarios presented above and Figure 3-1 the number of containers to be handled at the port of Buenos Aires in 2020 vary between 4.2 and 6.6 million TEU. No matter what scenario is used or whatever economic development will occur, the conclusion is that the capacity of the port of Buenos Aires is not sufficient and will be reached around 2005. An expansion of the existing port is therefore needed, and the size and rate of this expansion in time will be determined in the following chapters, using the medium scenario.

4. DEVELOPMENT OF CONTAINER TRANSPORT

4.1 GENERAL HISTORY

For a very long time, long distance transport of large quantities of cargo has been carried out by ship. For centuries this has been general cargo, stowed in the holds of ships in crates, bundles and barrels. Ships were loaded and unloaded by hand, as is still the case in some developing countries. As mechanisation went on, manual labour was gradually replaced by the use of ship-to-shore cranes and derricks. To ensure faster and more efficient loading/unloading and to reduce the damage of cargo, goods were placed on standardised pallets, which could be picked up by forklifts.

A real transformation in the character of general cargo took place when in the early 1950's the first containers were introduced by 'Sea-Land' on their east and West Coast traffic routes in the USA. First meant to be a domestic operation, the containerised transport started spreading the maritime market soon after that. It was first introduced in the developed countries of Europe, North America and Japan and was followed later on by the newly industrialising countries of East Asia. Many countries, in particular developing countries, opposed the arrival of containers at first, but eventually had to give in, as goods did only arrive in containerised form.

Large growth of containerisation and traffic volumes in the past decades has been accompanied by the increase in the size of ships (see Table 4-1). These vessels have placed demands on the facilities of ports. Cranes and terminal areas have increased as well and the main ports in the world are currently preparing to handle 8,000+ TEU vessels.

Year	LOA (m)	Draft(m)	Beam(m)	Gross tonnage	Capacity (TEU)
1980	221	11.0	31.2	31,300	1,280
1983	217	11.5	32.2	25,210	1,728
1986	248	11.5	32.2	32,300	2,568
1990	290	13.0	32.2	49,000	4,000
1994	294	12.6	32.2	46,800	4,229
1994	300	13.0	37.1	61,000	4,743
1995	276	14.0	40.0	66,385	4,826
1996	318	14.0	42.8	84,900	6,000-7,500
1997	347	14.5	42.8	104,700	6,600-8,700
1998	300	14.0	42.8	80,942	6,716

Table 4-1 Yearly changes of container vessel size from 1980 to 1998,
Source: JICA / Fairplay Shipping Database

4.2 HISTORY OF SOUTH AMERICAN CONTAINER TRANSPORT

In South America containerisation got off the ground very slowly: the main reason for this are a lack of investment in both container facilities and transport infrastructures and slow economic growth. Furthermore, technical limitations of the ports' container handling facilities and dramatic inefficiencies in port operations gave rise to very high handling costs, so that containerisation did not bring any benefits. Therefore container traffic did not grow as expected and throughputs were structurally overestimated in the

1980's. However, the last few years container traffic volumes in South America have been growing enormously with rates in the order of 10 to 15 percent as a result of higher economic growth, privatisation and deregulation of port activities, which result in more efficient port operations and lower handling costs.

4.3 CURRENT AND FUTURE DEVELOPMENTS

4.3.1 World container port throughput

At a global scale of container traffic the trend has been continuous growth. Even though the crises in the Far East and South America have had serious impact on international economies (especially Japan) and international trade, global container port trade volumes still have increased over the past few years (growth from 1980 to 1997: 350.5 percent). There are still no signs that this trend will change after the 1990's (see Table 4-2) and therefore it will not be long before global container port throughput will reach 200 million TEU per annum.

Year	Global container port throughput (in million TEU)
1980	36.35
1986	61.09
1990	86.64
1995	141.59
1996	150.72
1997	163.74

Table 4-2 World container throughput, 1980-1997,
Source: Ocean Shipping Consultants Ltd / CIY

4.3.2 Ships and shipping lines

As in every business, shipping lines are trying to make as much profit as possible by maximizing the revenues and minimizing the costs of their operations.
Fees are usually established for the transportation of a specific type of cargo over a given distance and after determination they are fixed. Therefore, if costs can be reduced, higher profits are made.

Costs can be distinguished in two ways, costs regarding the actual transportation of containers and overhead costs.

Transportation costs
Reducing unit costs is what the shipping business is about. As economy of scale proves, [Wijnolst, 1995], building larger capacity vessels is cheaper in terms of cost per container slot than building several smaller ships. It therefore reduces the cost per shipped container. Operating a larger vessel is also more efficient than operating a smaller vessel. The operation of the larger vessel is of course more expensive, but not proportionally. Larger ships are able to move much more containers with the same crew and approximately the same fuel consumption. Other savings come from reduced port, canal and berth fees, including pilotage and towing, which are usually calculated per ship.

Overhead costs
Another way of reducing unit costs, or to put it in a better way, to enhance revenues, is to make better usage of available ship capacity on shipping routes. In order to do so, various shipping lines have merged. Recent examples are the merger of P&O and Nedlloyd and the merger of Maersk and Sea-Land. Besides these examples of mergers, on many trade lines alliances have been formed. After the merger savings through reductions in staffing levels and reductions in other overhead costs can be achieved.

Consequences
Because of the very high costs of operating large container vessels, every delay is extremely costly and will rule out any reduction of costs, achieved by using the larger ships in the first place. To avoid delays as much as possible, the arrivals of vessels are scheduled and shipping lines have placed stringent demands on the ports and terminal operators. Some of these demands are:

- dedicated terminals with reserved berths
- minimum turn around time (within 24 hours is the norm)
- fast operating of tugboats
- unrestricted entrance of the port

Shipping lines will do anything to further reduce the transportation cost per TEU and therefore will continue to order larger ships (ships up to 15,000 TEU are expected to be in service within 10 years, *source: Lloyds*). Only a few major ports in the world will be able to accommodate this type of ships, as they require deeper approach channels and basins, longer berths, larger cranes and greater storage capacity. The largest ships will eventually sail the world and call at only a few ports on each continent. In these large volume ports, called hubs, the containers will transhipped to smaller ships. These smaller ships sail the smaller routes (spokes), on smaller ports will be called at. Panamax ships with capacities of 2000-4000 TEU, which were the largest ships a few years ago, are gradually shifted from main lines to feeder lines. As the hub and spoke system develops in the future and larger ships come into operation, it may be expected that even bigger ships will become operational on 'feeder' lines.

4.3.3 Terminals

Terminal operators also wish to make as high profits as possible. They will only start business in a certain port if they are convinced that business in this port will be profitable. Cargo forecasts should appear positive, the economic position of the surrounding area must seem promising and the concession under which a terminal can run its business must be advantageous (flexibility of operations, low tax and rent). Different ports and terminals are always trying to attract as many customers as possible and are competing by using lower handling costs and providing better service. Handling costs applicable to large customers are usually determined in lump sump contracts, but in some ports, the port authority determines fixed handling tariffs for all terminals. Attraction of customers is then solely a matter of offering the best service. Ways to achieve this are: the availability of sufficient berth length, quick turn-around times, guaranteeing minimal waiting time for customers, good hinterland connections, availability of sufficient storage space for import and export containers against low costs etc.

As container traffic is growing and containerisation is proceeding, many countries are converting their conventional ports with general cargo terminals into container ports with modern container terminals. To ensure higher efficiency in port operations, in order to attract more container traffic, many ports in developing countries have been going through a privatisation process. Large international stevedoring companies operate strategic terminals, and after privatisation generally handling costs will drop and handling rates increase. However, these terminals are often positioned in ports with difficult layouts for highly efficient container handling operations. Newly developed terminals are set up on rectangular land locations to ensure efficient operations. Sometimes only if high volumes are expected in a new terminal, a shipping line (or alliance) will have a dedicated terminal.

As shipping lines are using larger and larger ships, their demands get stricter and are more difficult to meet. Only some ports in the world can keep up with the development of larger ships (e.g. Rotterdam, Singapore, Hong Kong)
When only few, but very large ships call at a port, berth occupation will be low and a large amount of containers is discharged in a short span of time. This requires many fast container cranes, capable of lifting up to 50 containers per hour, fast transportation within the terminal to the stockyard and very large storage areas. Therefore. cargo handling equipment will continue to get larger and faster, as main ports will have to be capable of handling these very large vessels. Post-Panamax cranes (16 container rows wide) are already the norm and some ports are investing in Super Post Panamax cranes (18-23 container rows wide)

Information systems have been developed for faster services and cargo information is communicated all over the world. It is already possible for customers to track their cargo on-line (e.g. on the Internet).

In most ports, road transport is the dominating form of hinterland transport due to its flexibility and speed. However, transport by rail and inland navigation is increasing, because in many countries the road network is coping with congestion.

4.4 TERMINAL OPERATIONS

In the container transport sector speed, costs and efficiency are of utmost importance. Therefore most of the containers are handled in special terminals adapted to containers.
In a container terminal three types of transport can be distinguished: ship to shore (and vice versa), horizontal transport from the quay to the container yard and vertical transport when containers are stacked. For each type of transport specialised equipment has been developed.

4.4.1 Ship to shore transport

In the past, ships used to have their own loading and unloading gear, but today this is only found on some feeder vessels. Today container loading and unloading from a ship is done with large ship-to-shore container gantry cranes (portainers), which are positioned on the quay.
These large portal cranes have an outreach of 45 m (Post Panamax) or even 55 m (Super Post-Panamax, up to 20 container rows wide). After lifting a container from the ship, the crane puts the container either on the quay between its legs or directly on a (terminal) trailer. This depends of the choice of the transport system from the quay to the stack.

Figure 4-1 Container crane, source: Liebherr

4.4.2 Transport systems in a container terminal

For the horizontal transport from the quay to the container yard and within the container yard (horizontal and vertical) various types of operations have been developed, each of them with their own characteristics. In the following text four types of terminal operations will be mentioned;

- trailer operation
- straddle carrier operation
- front-end loader operation (forklift and reachstacker)
- combination of above and Yard Gantries (RTG/RMG)

Trailer operation

When the portainer has lifted the container off the ship it is put on a road trailer between its legs. The container will stay on this trailer until delivery to the customer; the container is only horizontally transported.

A terminal tractor tows the container to the storage area, where it remains until collected by a road tractor. This type of operation is very simple to organise, is very safe, but it requires a large area to park the trailers on. Selectivity is high, any container can be reached at any time. Because there is no stacking, the loads on the pavement are low and no special measures have to be taken. The operation with trailers requires a lot of trailers however and peak throughputs are difficult to absorb.

Straddle carrier operation

With this system, the container is picked up by a straddle carrier on the quay. The straddle carrier (see Figure 4-2) takes the container to the container yard where it stacks the container 2 to 4s high. When a container is needed the straddle carrier removes the container from the stack. Often a few containers have to be removed before the specific container is reached; re-handling of containers is needed. An advantage of this type of operation is that straddle carriers can be used for horizontal and vertical transport at the same time. Disadvantages are that straddle carriers are expensive in price and maintenance. They are also rather dangerous, because they drive very fast and the driver has not always a good visibility. The wheel loads of the machines are high and make the necessary civil engineering investment higher than with the trailer system.

Figure 4-2 Straddle carrier, source: Nelcon

Front-end loader operation

There are two main types of front-end loaders; heavy duty forklift trucks, which lift the container from underneath and reachstackers, which lift the container from above using an integral lifting frame (see Figure 4-3and Figure 4-4)

The front-end loaders are used to transfer the container from the quay to the container stack where they stack the container 2 or 3 high. The same type of equipment is used for both horizontal and vertical transport.

A reachstacker has an extendable arm and is able to work in the second row, thus making more efficient use of the available land. This system places heavy loading on the surface of the terminal and adequate measures have to be provided.

Compared to the straddle carrier, this type of operation needs wider traffic lanes and it has a lower stacking density[6], thus making less efficient use of the land.

The front-end loader operation is slower then the straddle carrier operation and it is dangerous as well, but its operation is very simple to organise. When transporting fully loaded containers, the wheel loads are even higher than with straddle carriers. The front-end loader operation is therefore not often used for the stacking of full containers, but because forklifts are cheaper in operation and price they are often used for the stacking of empty containers and in terminals with low throughputs.

[6] Both stack the containers to the same height, but the reachstacker is not able to stack two adjacent rows both to 3 high.


Figure 4-3 Forklift truck

Figure 4-4 Reachstacker

Yard gantry operation (RTG/RMG)

With this system horizontal and vertical transport are carried out by different pieces of equipment. The horizontal transport to and from the quay is carried out either by straddle carrier or by truck and trailer. The container is placed on a temporary parking place within the reach of the yard crane, from where it is placed into the stack by specialised gantry cranes.

There are two different types of yard gantry cranes: the Rubber Tyred Gantry (RTG) and the Rail Mounted Gantry (RMG).

RMG's can be made with a larger span than RTG's and because they can stack the containers up to 6 high they are a bit more space efficient than the RTG's. Furthermore, the rail mounted type can be automated relatively easily. But this type is quite inflexible in its operation; where RTG's can be moved from one stack to another, RMG's cannot.

Using yard gantries requires a large investment both in terminal development and in equipment costs. The crane beam of the RMG must have a solid foundation and high quality paving is required for RTG's.

But as mentioned above, operations are very fast, the operational cost is low, reliability is high, the stacking density is high and this type of operation does not require much maintenance.

The various stacking systems and their characteristics are summarized in Table 4-3.

Mixed systems

In many terminals, different types of equipment are used to carry out different tasks. In different parts of the terminal the equipment is used that is most favourable for the task at hand. Par example: for the stacking of empty containers, forklifts or reachstackers are used, for the horizontal transport the tractor/trailer system and for the receipt and for the stacking of containers either a straddle carrier or a stacking crane.

Remark:

The terminal operator decides which system is used and also the maximum stacking height. These choices very much determine the annual throughput that can be achieved within the terminal area (TEU/ha/yr). Extremely high rates, like some ports in South East Asia (see chapter 1 & appendix A), can only be achieved by means of special operational circumstances.

System	Trailer	Straddle carrier	Forklift	RTG	RMG
Stacking height	none	2-4	2-3	2-5	3-6
Land utilisation	low, large area needed, due to large lanes and large area per container	good, smaller lanes required	poor, large traffic lanes needed	very efficient	most efficient of all
Selectivity	high, every container available	medium-high, some rehandling may be necessary	medium-high	medium-low, rehandling necessary	lowest of all systems, rehandling necessary
Terminal development costs	low, no high quality paving required	medium, hard surface is needed due to high wheelloads	high, very high wheelloads on surface	high, high load paving is needed for crane wheels	high, piled rails might be needed
Equipment costs	high, many trailers required	high, 4-6 straddle carriers per ship-to-shore crane	medium: for low throughputs cost effective	high, gantries are expensive	very high, railmounted gantries are expensive
Equipment maintenance costs	low, reliable system and easy maintenance	very high, most expensive in maintenance	medium: easy maintenance	low, not much maintenance is needed	low, not much maintenance is needed
Manning levels	high	low	medium	medium/high	medium/high
Flexibility	high	high	high	medium-low, high load paving is not available everywhere	almost none, rails are mostly fixed
Capacity	due to limited number of trailers are peaks diffucult to absorb	can absorb peaks quite easily	low	high, gantries can work very fast	high, gantries can work very fast
Remarks	also used to bring containers to a temporary storage, where a crane or straddle carrier can pick it up	most suitable for terminals requiring easy selection of containers from stack	best for stacking of empty containers	extra horizontal transport needed	horizontal transport needed, suitable for automation

Table 4-3 Characteristics of different terminal operations

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BUENOS AIRES PORT EXPANSION 2020

PART 2:

DESIGN PARAMETERS AND DESIGNS OF PORT EXPANSION

5. CHARACTERISTICS OF THE PORT OF BUENOS AIRES AND PORT DESIGN PARAMETERS

In this part of the report a first design of the required port expansion is presented. In chapter 5 and 6 design parameters and restrictions have been stated, with which six possible designs of the port expansion have been developed, chapter 7. A first comparison of these alternatives is made in chapter 8 and the two most promising alternatives have been analysed in further detail in chapter 9.

As mentioned above, in this chapter first the characteristics of the port of Buenos Aires have been mentioned and after that other port design parameters, such as minimum terminal area and length of berths have been calculated.

5.1 CHARACTERISTICS OF THE PORT OF BUENOS AIRES

5.1.1 Depth and width of access channels and basins

The Rio de la Plata basin is a shallow river delta with an average depth of about 5 metres. Depths around Buenos Aires (within a range of 15 kilometres) vary from CD -2.5 to CD -5 metres. In order for ships to enter the port, channels through the river have been dredged (see chapter 1). The *Canal de Accesso*, *Canal Norte* and *Canal Sud* have a dredged depth of CD -9.75 metres (32 feet). The channels are subject to siltation and regular maintenance dredging is required. Nominal depths and approximate widths are given in Table 5–1.

Channel name	depth	width
Canal Punta Indio	9.75 m	100 m
Canal de Accesso Al Puerto de Buenos Aires	9.75 m	100 m
Canal Norte	9.75 m	100 m
Canal Sud	9.75 m	60 m

Table 5–1 Current depths and widths of access channels to Buenos Aires (measured from CD), Source: Shipping Guide 1999

The port authority AGP has announced that it is likely that in the coming years, the channels will be dredged to 36 feet (10.97 metre). The width of *Canal de Accesso* and *Canal Norte* will be extended to 140 metres, the width of *Canal Sud* to 100 metres.

The harbour basins used for container operations in *Dock Sud* and the basins in *Puerto Nuevo* currently have a charted depth of approximately 10 metres (33 feet), although this is rather poorly maintenanced.

5.1.2 Water levels

Water level fluctuations at a port are dependent of waves, wind and obviously tides. These fluctuations combined with wind, currents and soil conditions determine how a specific port can be entered, whether or not it can be entered (should a tidal window be used for some deep draft ships). The wind and currents also determine the orientation and dimensions of the access channel and berths.

Tides occurring at the port of Buenos Aires are described in paragraph 5.1.3. As the wind in the Rio de la Plata can make a large difference on the water level its influence is described in paragraph 5.1.4 and 5.1.5. Waves are described in 5.1.6, currents in 5.1.7 and the soil conditions in 5.1.8.

5.1.3 Water levels: tides

Water levels at the port of Buenos Aires are measured in metres from Chart Datum. Chart Datum (C.D.) is defined as *zero de Riachuelo*, the level of the Riachuelo river, which flows into the Rio de la Plata near Dock Sur.
At the port of Buenos Aires a weak semidiurnal tide is occurring, which has considerable differences through the seasons. The tide is built up as shown in following table.

Component	Phase	Amplitude (m)
Mean Sea Level Z_0		0.79
M_2	204°	0.29
S_2	283°	0.06
K_1	040°	0.08
O_1	216°	0.16

Table 5–2 Tidal components at the port of Buenos Aires,
Source: Admiralty Tide Tables, 1991

The mean sea level has a seasonal variation of 0.1 metre (see Table 5–3).

	Jan	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov	Dec.
Variation (m)	0.0	+0.1	+0.1	+0.1	0.0	-0.1	-0.1	-0.1	-0.1	0.0	0.0	0.0

Table 5–3 Seasonal variations of Mean Sea Level,
Source: Admiralty Tide Tables, 1991

Without disturbance of wind, the following tidal levels are observed.

Tidal Level	Height (m)
Lowest Astronomical Tide (L.A.T.)	- 0.1
Mean Lower Low Water (M.L.L.W.)	+ 0.3
Mean Higher Low Water (M.H.L.W.)	+ 0.5
Mean Sea Level (M.S.L.)	+ 0.8
Mean Lower High Water (M.L.H.W.)	+ 1.0
Mean Higher High Water (M.H.H.W.)	+ 1.3
Highest Astronomical Tide (H.A.T.)	+ 1.7

Table 5–4 Tidal levels measured from CD at the port of Buenos Aires,
Source: Admiralty Tide Tables, 1991

5.1.4 Water levels: wind

The wind is the most important factor influencing the water levels at the port of the Buenos Aires, not the tides. Although the wind in Buenos Aires is usually not so strong, it can cause quite high levels of set-up or set-down. Strong off-shore winds blow the water out of the Rio de la Plata and reduce the water level, while on shore winds will increase the water level (in extreme cases as much as 2.4 m below or above Chart Datum). All values of the set-up and set-down for certain wind directions at certain wind speeds can be found in Appendix C.1.

The tides including the wave set-up or set-down and extreme discharges of the Rio de la Plata result in some significant water levels, which are shown in Table 5–5.

Return Period (Years)	Maximum level (m)	Minimum level (m)
2	+3.00	-1.20
10	+3.50	-2.15
25	+3.75	-2.70
50	+4.10	-3.10
100	+4.30	-3.40

Table 5–5 Significant water levels in Buenos Aires,
Source: Naval Hydrographic Service

Normally the water level elevation does not vary much, as is shown in Table 5–4; for 48 percent of the time it varies between 0.4 and 1.0 m +CD. The maximum level, which has occurred since water levels have been measured, is CD +4.44 m (recorded in 1940) and the lowest level is CD -3.63 m (recorded in 1984).

The container terminals are constructed on a level of 4.75 m +CD to prevent them from flooding at an extreme water level. The shore protection will be constructed on a design crest level of 5.00 m +CD.

5.1.5 Wind

Wind plays a predominant role in port planning. It can cause set-up and set-down (see paragraph 5.1.4), it determines the manoeuvres of ships navigating the access channel and basins to a large extent, it causes great forces on moored ships and quays and it influences the operations of cargo handling equipment.
The wind climate in the Rio de la Plata is moderate, 93 percent of the time velocities are below 15 knots (Beaufort 4), and 99.5 percent of the time below 33 knots (Beaufort 7). The dominating direction of strong winds is between ENE and SSW (on-shore winds). The distribution and occurrence of wind speeds and directions can be found in Appendix C.2.

5.1.6 Wind generated waves and swell

Wind generated waves
The development of waves generated by the winds, blowing over the Rio de la Plata, is limited by fetch, storm duration and water depth. In Appendix C.3 two figures show the distributions of significant wave heights and zero crossing periods for the port of Buenos Aires. In the following table the design waves with their return period are shown.

Return Period (Years)	Significant wave height (m)	Significant wave period (s)
10	1.80	6.10
50	1.95	6.30

Table 5–6 Design waves for Buenos Aires,
Source: Informe d'Avance No.1

For the design of port structures, the design wave with a return period of 50 years is used. To validate the values in Table 5–6, a fetch calculation for an extreme wind condition has been made in chapter 11, using the wind-generated wave theory. The calculation results in a significant wave height H_s of 1.92 m.

Swell
As the port of Buenos Aires is situated more than 200 km from the Atlantic Ocean and the Rio de la Plata is very shallow, swell does not occur near Buenos Aires.

5.1.7 Currents
The currents in the Rio de la Plata are small; according to the Admiralty maps about 0.5 to 1 knots (0.25 - 0.5 m/s), with extreme values of 2 knots (1 m/s). The main direction of the current is NW/SE.

5.1.8 Soil Conditions (see chapter 10)
The various soil layers and their properties are described in detail in 11chapter 10.
The first layer of soil in the Rio de la Plata consists of soft material originating from the Rio Parana and the Rio Uruguay. It has a thickness ranging from 8 to 14 metres. Below that a pampean formation can be found. It consists of a stiff to very stiff clayey silt layer and a stiff to very stiff clay layer and has a thickness of about 14 metres. This layer is called the *tosca* layer and the soil can be cemented at some places. Below the *tosca* layer, at a depth of about 22 metres a sand layer can be found.

5.2 PORT DESIGN PARAMETERS: NAVIGATION

5.2.1 Design ships
For the design of the port expansion two design ships have been chosen: a Panamax ship and a Post Panamax ship, see Table 5–7. Panamax ships will enter the port frequently in the future; Post Panamax ships less frequently. For a first design of the required channel depth, the maximum *allowable* draft for the Post Panamax ship has been chosen as 12.0 metres, which corresponds approximately to an 75% loaded ship (chapter 11.1)

Parameter	Panamax ship	Post Panamax
Capacity (TEU)	3,100	5,400
LOA (m)	252	282
Lpp (m)	232	268
Beam (m)	32.20	40.0
Max. draft (m)	12.0	13.3
DWT	46,000	64,000

Table 5–7 Dimensions of design ships for Buenos Aires port expansion

Note: The dimensions of the design ships are averages for those types of ships. (source: Fairplay Encyclopaedia 1998)

5.2.2 Sailing speeds
The maximum allowed sailing speed in the access channel is 8 knots (4 m/s), [*South America Pilot, 1996*]. The maximum sailing speed in the port is 3 knots.

5.2.3 Channel width: PIANC
For the determination of the channel width a first estimate has been made with the guidelines presented by PIANC. The port of Buenos Aires will be a busy port in the future, approximately 7000 ships will enter and leave the port every year, see 5.5. The access channel will therefore be a two-way channel, at least for Panamax ships. For Post-Panamax ships, which will enter the port less frequently, the access channel is designed for one way traffic.

The PIANC guidelines state that for a first estimate the required width for a two way channel is:

$$w = 2w_{BM} + 2\sum_{i=1}^{n} w_i + w_{BR} + w_{BG} + \sum w_P$$

in which:

- w = width at bottom of channel
- w_{BM} = basis manoeuvring lane
- w_i = extra width to compensate wind, waves etc.
- w_{BR} = distance bank-ship, starboard (red) side
- w_{BG} = distance bank-ship, port (green) side
- w_P = passing distance

Parameter	Value	Category in PIANC calculation	(additional)width
Design condition:	Panamax ship 2-way channel	-	
Channel condition	Outer channel exposed to open water		
Ship manoeuvrability		moderate	$w_{BM} = 2 * 1.8$ B
a. Speed in access channel	8 knots (4 m/s):	Slow	-
b. Cross wind	93 % below 15 knots:	Mild	-
c. Prevailing cross current	0.5 knot	Mild	$w_i = 2 * 0.3$ B
d. Longitudinal current:	< 1.5 knot	Low	-
e. Waves:	Hs < 1 m	Mild	-
f . Aids to navigation:		good	$w_i = 2 * 0.1$ B
g. Bottom surface		smooth and soft	-
h. Depth:	1.10 * draft	< 1.25 T	$w_i = 2 * 0.2$ B
i . Cargo hazard level	containers	low	-
Passing distance: Vessel speed		8 knots	w_P= 1.2 B
Passing distance: Encounter traffic density	0 - 1 ship/hour	light	-
Bank Clearance	shoals, slow speed		$w_{bg} = 2 * 0.3$ B
TOTAL			6.6 B

Table 5–8 Determination of channel width

Table 5–8 above shows the qualitative values of the parameters have been presented as well as the required additional width of the access channel. The total channel using two-way traffic for Panamax ships comes to: 6.6 * 32.2 = 213 m.

5.2.4 Channel depth

As a first estimate, the channel depth below C.D. is determined as a ratio of the maximum draft.

PIANC: $h=1.10$ to $1.15 * d_{max} = 13.5$ m.

Note: this level is measured from MLLW and leads to a channel depth of CD - 13.2 m

For the draft of the ship, the Panamax design ship has been chosen, which should be able to enter the port under all circumstances. A better estimate, in relation with down-time calculations is presented in chapter 11

5.2.5 Length of access channel in port

Ships entering the Port of Buenos Aires already have tugs attached to them outside the port, so no extra distance is needed in the port to enter safely. At least space for a turning circle has to be available.

5.2.6 Turning circle

The required diameter for a turning circle has been mentioned in various sources, [*PIANC, 1997*] & [*Thoresen, 1988*]. The required diameter varies between 1.6 and 2.0 * L_{max}, depending on the wave climate of the considered port. In Buenos Aires, where the wave climate is mild, the diameter of the turning circle is determined to be:

$$D_{TC} = 1.6 \cdot L_{\max} = 1.6 \cdot 282\,m = 451\ m$$

5.2.7 Width of basins

A preliminary width of the basins can be been calculated with the following rules of thumb.

4 B + 100	UNCTAD Mooring on 1 side, containerships
5 B + 100	UNCTAD Mooring on 2 sides, containerships
L + B + 50	UNCTAD Long basins (>1000 m)
2 B_{max} + 30	Thoresen (short basins)
2 B_{max} + 50	Thoresen (long basins)

This leads to:

UNCTAD for mooring on 2 sides:	261 m (design ship: Panamax)
UNCTAD long basins:	375 m (design ship: Post-Panamax should be able to turn around)

5.2.8 Terminals

Type of cargo:		containers
Cargo handling method:		RTG
Net. working hours per day:		21
Operational days per year:		363
Average quantity of cargo per vessel:		600 - 650 TEU (today)
Average quantity of cargo per vessel:		800 - 1200 TEU (in 2020)
Vessel arrival pattern		M (negative exponential distribution)
Vessel waiting time		maximum 15% of service time
Service time pattern		Erlang 2
Gross number of moves		25 moves / working hour
Dwell times:	import	7 days
	export:	5 days
	empty:	10 days

5.3 BERTH LENGTH AND ORIENTATION

5.3.1 Berth length

As a first estimate of the berth length, UNCTAD gives as a rule of thumb:
For container vessels:

for 1 or 2 berths	300 m per berth
for 3 or more berths	250 m per berth

For the design of a container port more than 2 berths are preferably positioned in the same direction. This leads to a berth length of 250 m. Taking into account, the extra 25 metres between 2 ships (see below), the total berth length for a first design is:

250+25 = 275 m.

Note: in a detailed design, the quay length will not be determined based on a discrete number of berths, but the total quay length will be optimised with simulation models based on acceptable queuing times.

Space between ships
The minimum space between ships is, according to Thoresen:

$0.1\ L_{max} = 0.1 \quad * 282 = 28.2$ m ($\approx$ 25 m)

Space between end of ship and end of quay
The required space between the end of a ship and the end of the quay varies in various sources from 10 - 15 metres [UNCTAD] to 0.1-0.15 L [Thoresen, 1988]. For the design of the alternatives an average will be taken: 25 metres.

5.3.2 Orientation of berths

Generally, PIANC advises to chose the orientation of the berths in such a way that the angle between the prevailing wind direction and the berth is no more than 30°. But because the wind climate is mild in Buenos Aires, storms are rare and there are no prevailing wind directions, no special attention has to be paid to the direction of the berths.

5.4 PORT DESIGN PARAMETERS: LIMITING VALUES

5.4.1 Limiting wave heights

tug operation: $H_{S,max}$=1.5 m PIANC/UNCTAD

Limiting wave heights for moored ships are generally mentioned for waves with significant zero crossing periods of around 8 – 12 seconds as these influence the motions of the moored ship to a great extend. Wave measurements in Appendix C.3 show that these waves do not occur in Buenos Aires. Therefore not limiting wave heights, but limiting motion amplitudes for berthed ships are used, see Table 5–9

Motion	90-100 % efficiency Operational limit	50% efficiency Safety limit
Surge	0.25 m	0.50 m
Sway	0.40 m	1.00 m
Heave	0.23 m	0.30 m
Roll	1.50°	3.00°
Pitch	0.75°	1.25°
Yaw	0.25°	0.75°
Note: For 6,000+ TEU vessels, the limiting roll motion for the operational limit becomes 1.00°		

Table 5–9 Limiting ship motion amplitudes (Source: Thoresen)

5.4.2 Limiting wind speeds

Mooring	Beaufort 7 (15 m/s)	Shell, containerships
Moored	Beaufort 9 (20 m/s)	Thoresen
Container cranes	Beaufort 8 - 9	Various sources

5.4.3 Limiting flow velocity

The following limiting values for flow at the quay are found:

Flow perpendicular to quay:	0.4 – 0.5 m/s	PIANC/Thoresen
Flow parallel to quay:	1.5 m/s	PIANC/Thoresen

5.5 TERMINAL DIMENSIONS: PRELIMINARY DESIGN

In this paragraph the most import dimensions of the container terminals have been estimated with rules of thumb. As only the dimensions of the new terminals are of importance, only the container throughput of these new terminals is taken into account; the current capacity of the port of Buenos Aires (*Puerto Nuevo* and *Dock Sud*), which amounts 2.0 million TEU has been subtracted from the throughput values in 2020 of the different growth scenarios as developed in chapter 3.

5.5.1 Berth length with rules of thumb

The required berth length of the new terminals depend on the capacity of a berth. This capacity depends on the number of cranes on the quay, the handling rate, occupancy rate of the berth, the number of working hours per year and the percentage of 40' containers.

Below for these parameters a best estimate has been stated and the capacity of the new berths is determined.

Working days per year:	363
Net. working hours per day:	21
Gross crane production	25 moves per working hour,
Interference + idle time:	12.5 % (10% + 2.5%)
Net. crane production	21.8 moves per hour
Containers per TEU	1.5 TEU/ move

The theoretical crane capacity becomes (100% occupancy of the berth)

$$363 * 21 * 21.8 * 1.5 = 250{,}000 \text{ TEU/crane/year}$$

According to UNCTAD a 60 % berth occupancy does not cause considerable waiting times in large ports, when 2 cranes are used on each berth. The annual berth capacity becomes:

$$2 \cdot 60\% \cdot 250{,}000 = 300{,}000 \; {}^{TEU}\!\!/_{berth\,*\,year}$$

The necessary number of berths is calculated by dividing the annual throughput by the berth capacity:

Throughput of new terminals	Scenario Low 2,200,000 TEU	Medium 3,700,000 TEU	High 4,600,000
Number of berths	8	13	16
Quay length	2,200 m	3,575 m	4,400 m

Table 5–10 Number of new terminals in Puerto Nuevo and quay length for different scenarios

The calculated number of berths, determined in Table 5–10, is based on the assumption that a berth occupancy of 60% does not cause unacceptable waiting times for arriving ships. This occupancy has to be checked with the queuing theory or with a simulation.

Service times and inter arrival times
In order to make a calculation with the queuing theory, both service times and inter-arrival times have to be known. In order to maintain the 24-hour limit for turn-around, more cranes or faster handling rates per crane have to be used. Besides that, using more cranes can reduce the required number of berths; using 1 more crane per berth can reduce the number of berths by 30-50 percent, (a crane is cheaper than a complete berth as well). A trade-off has to be made between the purchase of more quay cranes and the creation of an extra berth.
Note: the rest of the transport in the terminal has to be adapted to the berth capacity.

Scenario Throughput in 2020 in PN	Low 3,700,000 TEU	medium 5,200,000 TEU	high 6,100,000 TEU
800 TEU / ship	4625 ships/year 12.7 ships/day	6500 ships/year 17.8 ships/day	7625 ships/year 20.9 ships/day
1200 TEU / ship	3083 ships/year 8.4 ships/day	4333 ships/year 11.9 ships/day	5083 ships/year 13.9 ships/day

Table 5–11 Number of arriving ships in Puerto Nuevo for different growth scenarios

5.5.2 Stacking area

The required stacking area can be calculated with the following formula:

$$O = \frac{C_i \times t_d \times F}{r \times 365 \times m_i}$$

in which:

O Required stacking area per type of stack

C_i : Number of containers moved per type of stack
Percentage of full import containers in 2020: 45 %
Percentage of full export containers in 2020: 35 %
Percentage of empty containers in 2020: 20%

t_d Average dwell time for type of cargo
Import containers 7 days

	Export containers	5 days
	Empties	10 days (low value, 20 days have been measured)
F:	required area per TEU (equipment travelling lanes included)	
	RTG operation:	4 high 10 m^2 / TEU
		5 high 7 m^2 / TEU
r:	average stacking height / nominal stacking height	
	import / export:	0.7
	empties	0.8
m_i:	acceptable average occupancy rate	
	import	0.6
	export	0.7
	empties	0.9

For 1 terminal with a capacity of 300,000 TEU, this leads to:

Import: $$O = \frac{45\% \cdot 300{,}000 \cdot 7 \cdot 10}{0.7 \cdot 0.6 \cdot 365} = 6.2\ ha$$

Export: $$O = \frac{35\% \cdot 300{,}000 \cdot 5 \cdot 10}{0.7 \cdot 0.7 \cdot 365} = 2.9\ ha$$

Empties: $$O = \frac{20\% \cdot 300{,}000 \cdot 10 \cdot 7}{0.9 \cdot 0.8 \cdot 365} = 1.6\ ha$$

The necessary stacking are becomes when these values are added up:10.7 ha
The stacking area depth in the case of a berth length of 275 m becomes
389 m. Some space has to be reserved for gates, parking space and terminal buildings, so an average depth of 400 m has been chosen.

For the new terminals the following total stacking areas are required:

	Scenario		
Throughput of new terminals	**2,200,000 TEU**	**3,700,000 TEU**	**4,600,000 TEU**
Number of new terminals	8	13	16
Import	50 ha	81 ha	100 ha
Export	24 ha	38 ha	47 ha
Empties	13 ha	21 ha	26 ha
Total	88 ha	143 ha	176 ha

Table 5–12 Required new stacking areas in 2020 for different scenarios

Note: This is the required terminal area without quay and transport corridor. Thoresen states that if future throughputs and capacities are uncertain another 25 to 40 percent of stacking space should be reserved.

The width of the quay is taken as 50 m, sufficient for the design presented in chapter 14 and 15. The total required terminal area comes to:

Scenario	2,200,000 TEU	3,700,000 TEU	4,600,000 TEU
Stacking area	88 ha	143 ha	176 ha
Quay	11 ha	18 ha	22 ha
Total	99 ha	161 ha	198 ha

Table 5–13 Total required terminal area in 2020 for different scenarios

6. CONSIDERATIONS, BOUNDARY CONDITIONS, AND RESTRICTIONS

The designs of possible alternatives for the future expansion have to comply with boundary conditions and assumptions. In this chapter first considerations on some very important design parameters are mentioned. After that the assumptions and design parameters are stated and finally, the most important design parameters from chapter 5 and 6 have been summarized at the end of this chapter in Table 6–4. The designs of the port expansion are presented in chapter 7 and Appendix D.

6.1 CONSIDERATIONS CONCERNING THE DESIGN OF THE PORT EXPANSION

6.1.1 Growth scenario

The main requirement for a port expansion is that it meets the required capacity in time. To predict the necessary capacity in time, growth scenarios have been developed (see chapter 3). As stated in chapter 3, for the design of the masterplan the medium growth scenario has been chosen. All of the presented alternatives will be constructed in phases, which gives some flexibility in the planning of the expansion to react on higher or lower growth rates.

Year	Total container throughput (TEU)
2000	1,427,000
2005	2,382,000
2010	3,187,000
2015	4,265,000
2020	5,708,000

Table 6–1 Container throughput of Buenos Aires in TEU, medium scenario

6.1.2 Place of expansion

Theoretically, an expansion of the capacity of the port of Buenos Aires can be made in two ways; first, the old port can be reorganised in order to enhance the current capacity. Secondly, new land can be reclaimed in the Rio de la Plata, on which new terminals are built.

The first option, reorganising old terminals, may increase the current capacity of *Puerto Nuevo* to about 2.1 million TEU[1], but this is still not sufficient to meet the ports needs and new terminals on reclaimed land are required after all. In addition, all of the existing container terminals are working under long term concessions, which makes it difficult to reorganise *Puerto Nuevo* completely. In the future, reorganisation of the existing terminals may be possible, but for now new terminals have to be constructed on newly reclaimed land. The current capacity of *Puerto Nuevo* and *Dock Sud* together is estimated to be 2.0 million TEU, so 3.7 million TEU will have to be handled on the new terminals of the port expansion, see Table 6–2.

[1] In either way, some new terminals will have to be constructed on reclaimed land

Throughput in 2020	
Total Buenos Aires	5,700,000 TEU
Dock Sud	500,000 TEU
Current Terminals *Puerto Nuevo*	1,500,000 TEU
New Terminals *Puerto Nuevo*	3,700,000 TEU

Table 6–2 Throughput figures in 2020, medium scenario

6.1.3 Power plants

In the north of *Puerto Nuevo*, two power plants are situated; *Central Puerto Nuevo* and *Central Nuevo Puerto*. These power plants are very important for the power supply of the city of Buenos Aires and cannot be removed. For their operations, water of the Rio de la Plata is used as cooling water. Both the intakes of the power plants are situated in Basin E. One power plant discharges its water at the head pier 5, and the other one discharges directly into the Rio de la Plata. When basin E is refilled, measures have to be taken to ensure the availability of cooling water for both power plants.

6.1.4 Number of berths

One of the most important elements of a future port expansion is the number of berths, which determines the throughput capacity of the port. For now, a first estimate for the required number of berths is made by dividing the total throughput of the medium scenario in 2020 by the capacity of a single berth. This berth capacity is calculated to be 300,000 TEU (chapter 5). The estimated number of berths will have to be validated with queuing theory or simulation in a later stage. When a simulation is made, not the number of berths is determined, but the total required quay length per terminal, for which waiting times (or waiting chances) are acceptable.
A flexible design offering the option to increase the number of berths (or quay length) when growth is higher than expected is preferred. In Table 6–3 the number of required berths in time has been stated, using the medium scenario and an estimated capacity of the existing berths of *Puerto Nuevo* and *Dock Sud* of 2.0 million TEU.

Year	Total throughput of Puerto Nuevo (TEU)	Throughput of Expansion (TEU)	Estimated number of new berths required
2000	1,427,000	-	
2005	2,382,000	382,000	2
2010	3,187,000	1,187,000	4
2015	4,265,000	2,265,000	8
2020	5,700,000	3,700,000	13

Table 6–3 Number of required berths in time, medium growth scenario

6.1.5 Quay wall

Generally the quay wall is one of the most expensive parts of any port expansion. Therefore, the shorter the quay can be to meet the required capacity, the better. Any quay wall that is not used for port operations makes the expansion more expensive than necessary. The required quay length per berth has been assumed to be 275 m when many berths are positioned in the same direction or 300 m when only 1 or 2 berths are positioned like that (source: UNCTAD). With the required 13 berths the minimum length will be 13*275 = 3575 m.

In many designs, the existing smaller breakwaters have to be removed to give access to the most northern new berths. Other berths, which are situated more to the south are frequently positioned near the existing (large) breakwater. To give access to these new berths, the existing breakwater can be removed so the quay wall can be positioned in any way, or the existing breakwater remains and the design of the quay wall is adapted to the position of the breakwater.

When the breakwater remains, the quay wall is positioned on the port side of the breakwater and the breakwater itself is (partially) included in the reclamation. It does not seem logical to position the quay wall on top of the breakwater, because this makes the construction difficult, costly and measures have to be taken to secure the stability of the existing breakwater. Including the breakwater in the reclamation is an easy and cheap solution, but results in a reduction of the basin width of the port and differential settlements of the reclamation area. On some places, this width is already considered to be small, so this option does not contribute to the nautical safety and manoeuvring convenience in the port.
Therefore, for the design of the alternatives, the quay wall has been chosen to be constructed behind the existing breakwater. The breakwater is removed where necessary after the completion of the berths.

6.1.6 Reclamation

Another expensive part is the reclamation, the land area, itself. The total area to be reclaimed consists of terminal area, roads and other reclaimed land. The ratio between total area and net terminal area shows how efficient the reclaimed area is used. The smaller this ratio is, the more efficient the usage of land. Other parts of the reclamation, which are not used for terminal operations could be used for distribution parks and container freight stations, but large spare areas are not considered beneficial.
In chapter 5 can be found that for the design of this expansion, the standard terminal depth has been calculated to be 450 metres; 50 metres of quay and 400 metres of stacking space, gates and manoeuvring areas. In this figure, the transport corridor is not included and for this purpose another 100 metres has been reserved. The minimum net terminal area is: 3575 m x 450 m =161 ha.
The minimum reclaimed area, including roads (3575 x 100 = 36 ha) is 197 ha.

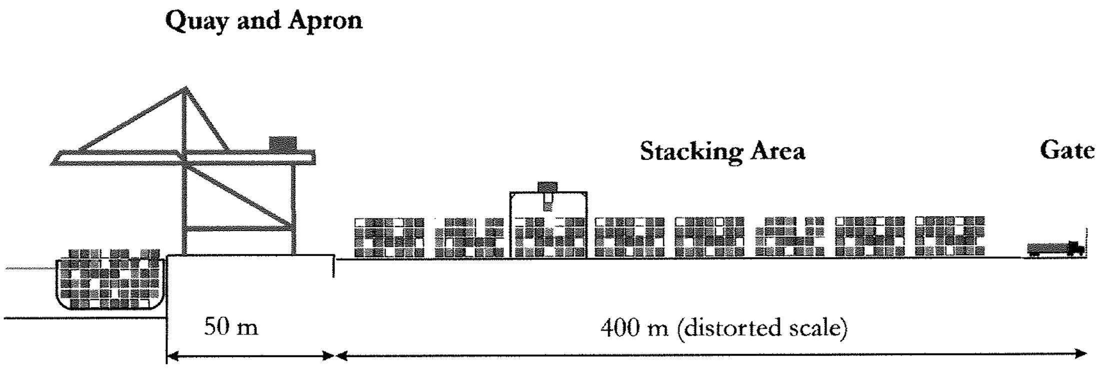

Figure 6-1 Cross section of terminal

6.1.7 Nautical areas

The necessary dimensions of the nautical areas have been calculated in chapter 5. Over these dimensions the full depth of CD -13.2 m has to be available. The old quay wall of Puerto Nuevo has not been designed for such depths, so in and around basins A to E the maximum bottom level of about CD -10 metres will be maintained and only on a certain distance, which secures the stability of the old quay wall, the basins will be dredged to their full depth. The new basins will therefore seem wider (400 m) than necessary. This is shown in the maps with different shades of blue.

6.1.8 Shore protection and breakwater

Shore protection

The reclaimed area needs a shore protection in order to protect the new land from erosion by waves and currents. The wind and wave climate in the Rio de la Plata is mild, see appendix C; high waves and storms are very rare. Currents are usually small (0.5 to 1 knot, extremes of 2 knots, see chapter 5).

Therefore, a soft shore protection (artificial beach) seems likely, but the soil of the Rio de la Plata, silty mud, is not suitable for this and sand, which is suitable, is difficult to obtain. The shore protection will be a small rock protection and even though rocks will be imported from Uruguay it is not a very expensive part of the expansion.

Breakwater

As stated above, the wave climate in the Rio de la Plata is mild. Besides, even the large waves have quite small zero crossing periods (up to 6 seconds), which do not influence the mooring and operating conditions for large container ships (LOA approximately 200 metres and up) much[2]. In chapter 12, it is shown that it is not necessary to construct a new breakwater when the port is expanded. The old port used to have a breakwater because small ships use the port as a shelter when storms occur. In the future small ships (feeders) will still use Puerto Nuevo for this purpose. Consequently, it is necessary to keep the old port sheltered. For this purpose in some designs new breakwaters are constructed. The new constructed berths, which serve larger ships, do not have to be sheltered from waves and currents if these are not affecting berthing, mooring and loading operations.

6.1.9 Environmental impact

Growing port activities lay a demand on nature; the reclaimed peninsula itself affects current patterns and causes some local scour, the port related activities cause pollution. In a final design stage an environmental impact analysis has to be done, here only the impact of the peninsula on the current and ecological reserve is mentioned. Some research into current diversion and siltation has already been carried out, [*INAA, Estudio del impacto.., 1997*]

Because of the width of the Rio de la Plata (near Buenos Aires approximately 45 km) the impact of the expansion will be limited to a relatively small area in the order of a few 100 metres, see chapter 11. Within this area changes in the sedimentation/erosion pattern will occur.

There will be no impact on the ecological reserve located south of *Canal Norte*, because of its sea defence. If deemed beneficial, the ecological reserve could be expanded during the dredging works in the port.

If the dredged material from the port is not heavily polluted, it will be dumped on an offshore dump location in an early stage. Spoil, dredged at a later stage, can be transported to depleted sand pits near the port.

6.2 BOUNDARY CONDITIONS AND ASSUMPTIONS

6.2.1 General - boundary conditions

- Expected moment of maximum capacity Puerto Nuevo: 2004
- Moment of completing expansion: 2020
- Full capacity of expansion is reached just after 2020
- Design ships as stated in paragraph 5.2.1

[2] Larger containerships have natural frequencies in the order of 20 seconds

- Looking at the economic growth in South America, the medium growth scenario gives the most realistic scenario for future throughputs
- The power plants will remain operative beyond 2020
- The supply and discharge of cooling water for the power plants has to be guaranteed
- The grain terminal TERBASA will be operative until 2020

6.2.2 General - assumptions

- Economical life - time of port expansion: 30 years
- Design life-time of port structures: 60 years (source: BS 6349)
- Tugs can operate outside the port, all container-ships will enter the port guided by tugs and pilots.
- All alternatives will be constructed in phases, first phase (2 terminals) must be operational at capacity in 2005
- Most of the hinterland transport will be carried out by truck, but the share of railway and transhipment will grow.
- Several terminal operators will use the new terminals

6.2.3 General - quantitative

- The current terminals in Puerto Nuevo will not be reorganised until 2020, but will expand their capacity with minor adjustments to 1.5 million TEU / year. The capacity of *Dock Sud* is 500,000 TEU.
- Throughput volumes as calculated in medium growth scenario (in TEU)

Year	Buenos Aires	Total throughput Puerto Nuevo	New Terminals Puerto Nuevo
2000	1,427,000	927,000	-
2005	2,382,000	1,882,000	332,000
2010	3,187,000	2,687,000	1,187,000
2015	4,265,000	3,765,000	2,265,000
2020	5,708,000	5,208,000	3,708,000

- Number of new berths in 2020: 13
- Length of berth: 300 m (1 or 2 berths), 275 m (3+ berths)
- Depth of terminal: 450 m (50 m quay & 400 m stacking area)
- Minimum new quay length: 3575 m
- Minimum new stacking area: 139 ha (Calculation chapter 5),
- Minimum new terminal area: 161 ha
- Width of transport corridor: 100 m
- Diameter turning circle: 450 m
- Width of basins (turning in basins is allowed): 375 m
- Depth of new access channel & new basins: CD -13.2 m, chapter 11
- Bottom width of new access channel: 219 m

6.2.4 Functional - boundary conditions

- Puerto Nuevo must be sheltered, in order to secure safe mooring for all sizes of ships
- Expansion takes place by reclaiming new land north of *Canal Norte*
- Wind, wave and tide conditions as presented in Appendix A, C & D

6.2.5 Functional - assumptions

- Larger ships (LOA approximately 200 metres and up) can moor directly at the Rio de la Plata side
- Depth of new basins is equal to the design depth of access channel (CD -13.2 m)
- There will be only one type of quay for deep-sea, short sea and transhipment (feeder) vessels
- Long straight quays are preferred over short bended quays
- Widening and deepening of access channel is necessary
- Current breakwater is removed when necessary
- For a transport corridor, road and railroad over the full length of the reclamation an area of 100 meters has been reserved.
- A connection to the freeway can be made in the north

6.3 SUMMARY OF DESIGN PARAMETERS

In the following table the most important design parameters from chapter 5 and 6 have been restated.

Parameter	Value	Comment
Capacity		
Growth scenario	Medium	
Throughput BA in 2020	5,700,000 TEU	
Throughput of new terminals in 2020	3,700,000 TEU	
Levels & angles		
Maximum water level	CD +4.30 m	Return Period 100 years
Design level container terminal	CD +4.75 m	
Design top-level shore protection	CD +5.00 m	
Design lowest level shore protection	1 m below seabed	Assumption
Design wave shore protection	H_s= 1.95 m, T_s=6.30 s	Return Period 50 years
Angle of shore protection	1:2	Assumption
Soil		
Average depth of sea bed	CD -3.00 m	
Average starting level of *tosca*	CD -7.00 m	
Average starting level of sand	CD -21.00 m	
Settlements soft layers after reclamation	1.50 m	Assumption
Access channel & basins		
Channel depth	CD -13.2 m	Calculation chapter 11
Channel width	219 m (6.8 B)	PIANC - 2 way channel
Turning circle diameter	450 m (1.6 * L_{max})	Calculation chapter 5
Basin width	375 m	Calculation chapter 5
Terminal dimensions		
Number of new berths	13	Medium Scenario
Berth length	300 m	1-2 container berths
	275 m	3+ container berths
Berth capacity	300,000 TEU	Calculation chapter 5
Terminal depth	450 m	
Transport corridor	100 m	Assumption
Minimum stacking area	143 ha	Calculation chapter 5
Minimum terminal area	161 ha	Calculation chapter 5

Table 6–4 Summary of design parameters

7. VARIOUS ALTERNATIVES FOR PORT EXPANSION

Enlargement figures of the designs than the ones in this chapter can be found in Appendix D.

7.1 Base layout: 'AGP-proposal'

The base layout, which has been designed by AGP, consists of an artificial island, positioned along the coast in front of the old port. The quay of pier 6 is extended with 700 metres (2 berths), then a bend is made and on 1400 metres of quay 5 berths are created. The quay bends again and then follow the last 5 berths on again 1400 metres of quay.

Summarized: 12 new berths are created on 3500 metres of quay. In the south a large area of land is reclaimed, but this is not considered to be necessary.
This alternative needs one more berth to meet the required capacity in 2020 using the medium growth scenario. Although it has been designed using different considerations and assumptions, this alternative has been evaluated because for many criteria it can function as a reference alternative. In other aspects it cannot be compared with the other designed alternatives.

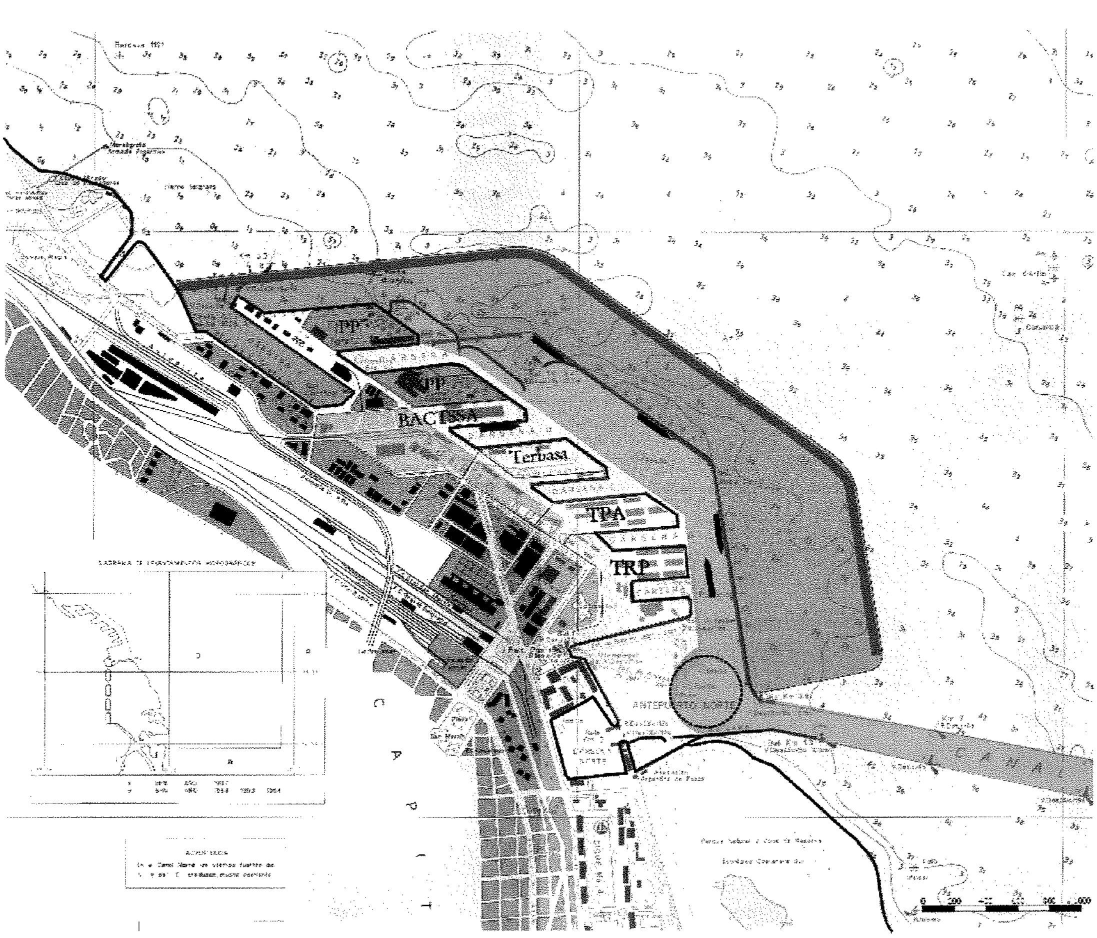

Figure 7-1 Alternative 1: Base layout

7.2 ALTERNATIVE 2: MODIFIED BASE LAYOUT

Alternative 2 is a modification of alternative 1, in order to overcome its shortcomings and adjust the design to the assumptions and considerations as stated in chapter 5. Therefore the quay starts more to the north and extra space is created for the extra berth required. Eight berths are constructed parallel to the current port on 2200 metres of quay wall. Then the quay bends and on 1375 metres of quay 5 new berths are positioned. The last berth, just before the bend ends approximately 200 metres from the bend of the existing breakwater, so the basins can be widened to ensure safer manoeuvring in the port. The existing breakwaters have to be removed in order to give access to the new constructed berths. In the south and the north no more land is reclaimed than necessary for terminal operations and the transport corridor. This results in a smaller area of land to be reclaimed (259 ha instead of 278 ha) and a shorter exposed shore protection (6000 m instead of 6700 m). In the port about 950 m shore protection is needed.

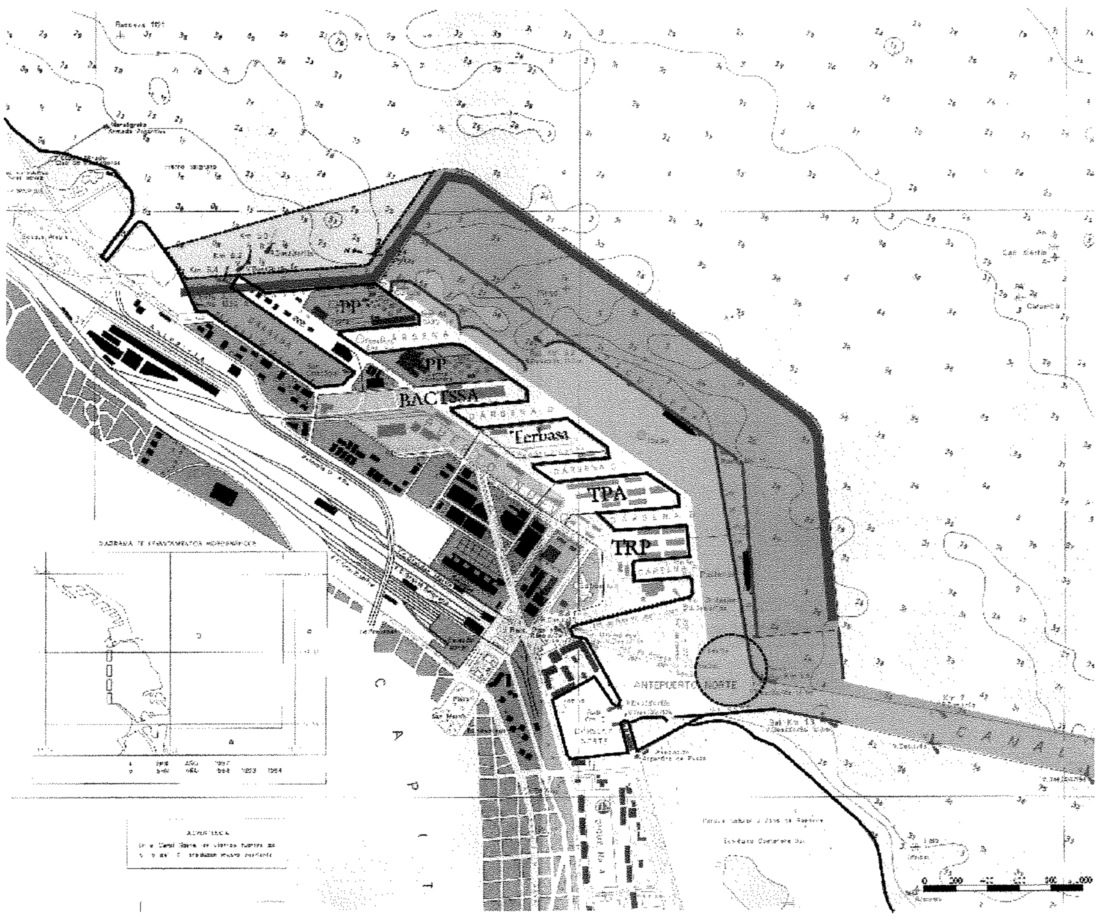

Figure 7-2 Alternative 2: Modified base layout

7.3 Alternative 3: Compact island

Looking at the current situation of Puerto Nuevo, one can see that north of basin D not many port related activities can be found; two power plants occupy piers 5 and 6. This area is not considered to be part of the port anymore and the new quay will be extended from where currently BACTSSA operates. Large ships will berth directly at the unprotected Rio de la Plata side and smaller ships will enter the port and berth on the landside of the island, which is constructed in front of the old port. Most of the current breakwater has to be removed, but the most southern part (see map) will be preserved to give shelter to the existing port. The reclamation can easily be expanded in the future by extending the quays on both sides of the island. In this way 4 more berths can be constructed.

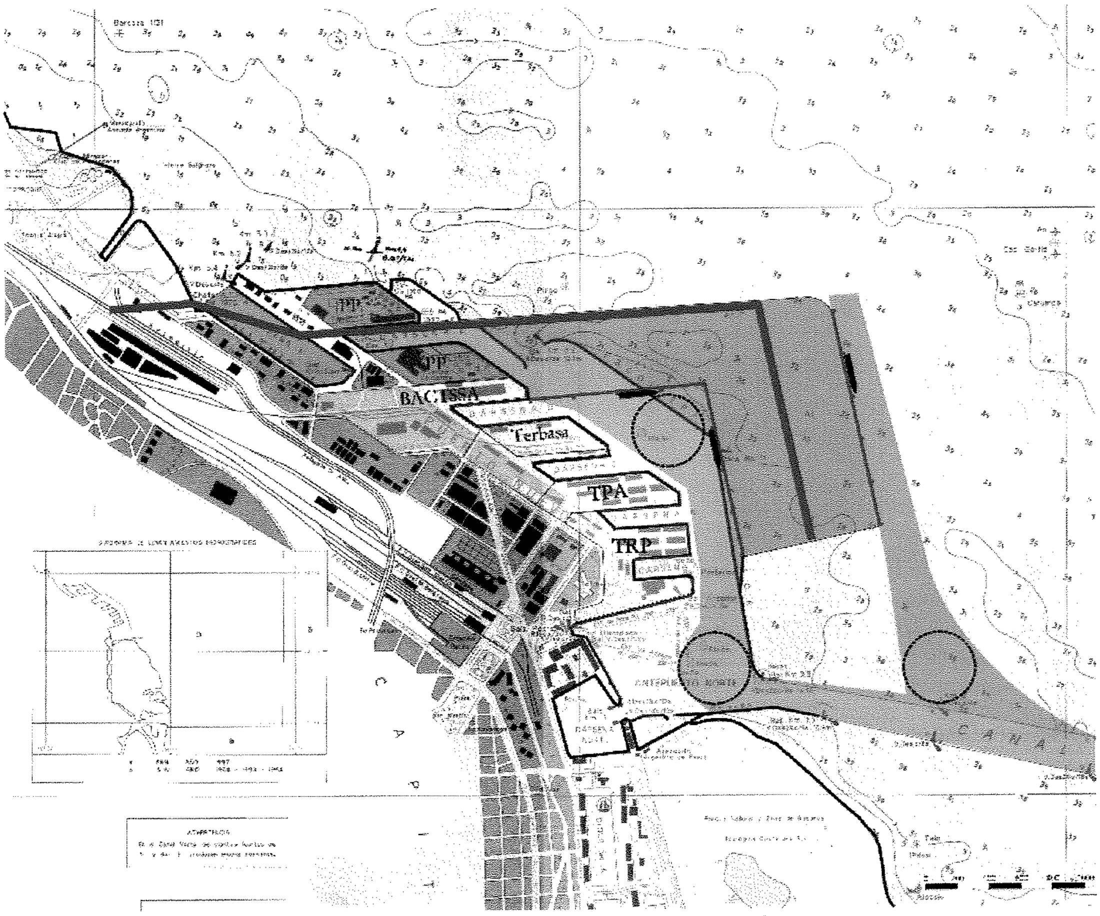

Figure 7-3 Alternative 3: Compact peninsula

7.4 ALTERNATIVE 4: ONE LONG QUAY

In this alternative, one straight quay of 3575 metres is constructed parallel to the northern part of the port. This creates wide basins, but requires the dredging of a large area and the complete removal of both the small and large breakwaters. New breakwaters have to be constructed because the orientation of the quay is such that without a breakwater small ships will be hindered by oscillations in the basins.

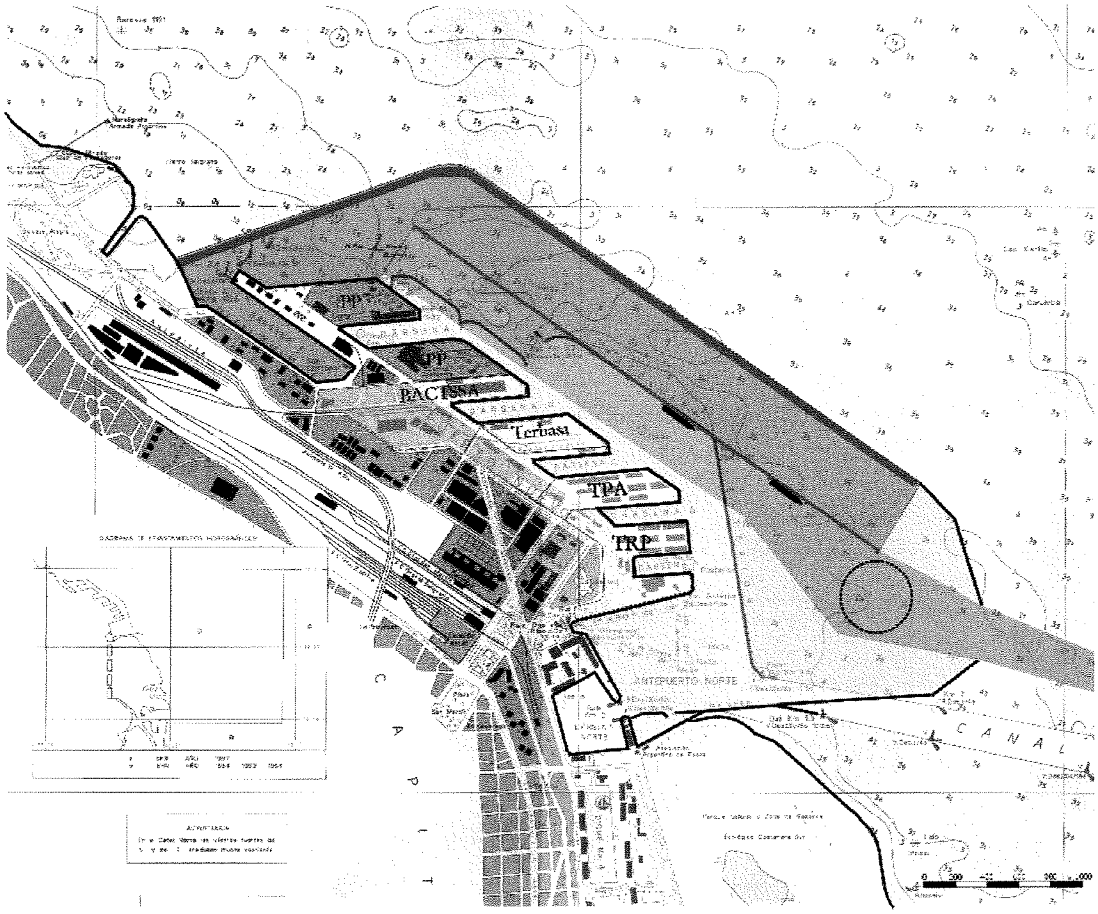

Figure 7-4 Alternative 4: One long quay

7.5 ALTERNATIVE 5: NORTHERN EXPANSION

In this alternative the expansion of the port is mainly in the north of Puerto Nuevo. North of basin E, 4 berths are constructed, then the quay bends and 9 more berths are constructed. The last berth ends in the bend of the existing breakwater. A large expansion will be possible but the ships will have to sail a large distance through the port to the northern berths. The old breakwater will be partially removed.

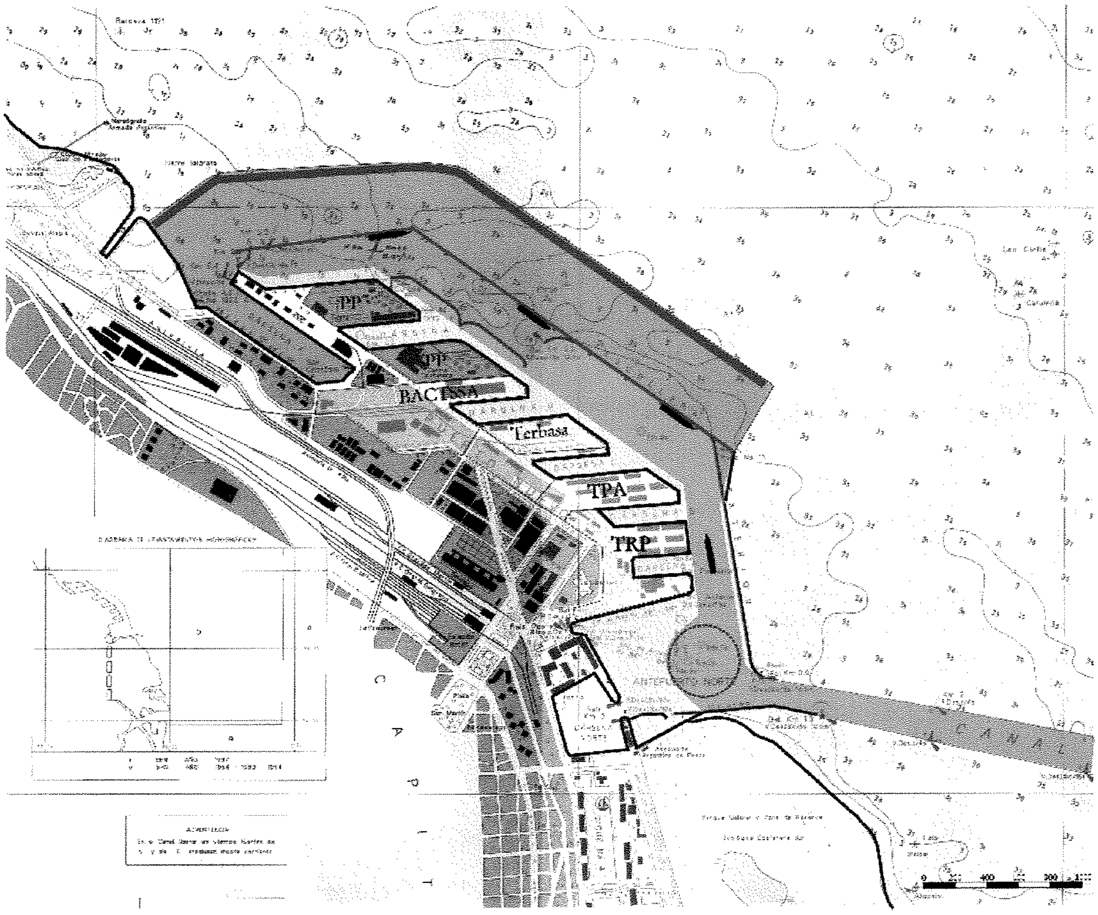

Figure 7-5 Alternative 5: Northern expansion

7.6 ALTERNATIVE 6: BOOMERANG

In this alternative the northern quay of basin E is extended with 6 berths. Then the quay bends and another 7 berths follow in the NNW-SSE direction. The last berth ends near the end of pier 1 (TRP) and a large turning basin in the northern port is automatically created. In order to maintain the mild climate in the old port the breakwaters have to be extended a bit. The old breakwater will be removed and a large amount of dredging is needed, but not as large as in alternative 2 or 3.

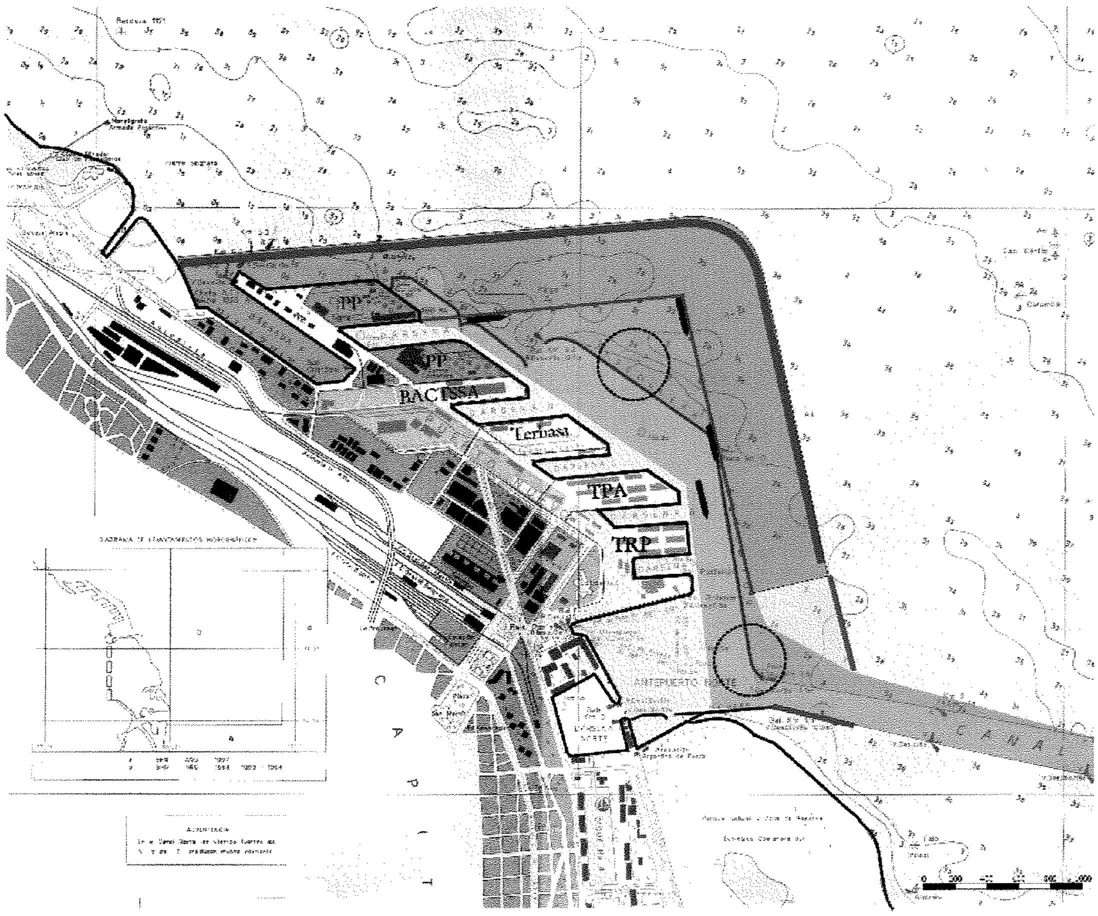

Figure 7-6 Alternative 6: Boomerang

8. FIRST SELECTION OF ALTERNATIVES

The six alternatives, mentioned above, have been evaluated qualitatively in order to make a first selection and to find the strengths and weaknesses of each of the alternatives. First, the alternatives will be discussed and later a summary will be given in Table 8–1.
Attention is paid to capacity, nautical safety, efficiency, area of reclaimed land, expansion possibilities and phasing, approximate costs and environmental impact.

8.1 Base layout: AGP Proposal

The considerations and assumptions with which this alternative has been designed differ from those with which alternatives 2 to 6 have been designed. Therefore not for all aspects a valid comparison can be made. For aspects on which the alternatives can be compared, the base layout will act as a reference alternative.

Capacity, expansion possibilities & phasing
First of all, this alternative does not have enough capacity for the expected growth, using the medium scenario. Secondly, when the growth is larger than expected, it will be quite difficult to expand this alternative. Phasing is quite easy, when starting in the north, terminals can be added one by one.

Nautical access, capacity of nautical area and traffic intensity
All the alternatives comply to nautical safety standards; the basins in this alternative are generally wide enough (300 m) for berthing on two sides of the basin and ships are assisted by tugs in the port. However, the passage near pier 1 and 2 is small (180 m) and with increasing traffic intensity, it may cause additional waiting times in the port. Turning is done in the turning circle in *Antepuerto Norte*, but at places where basins are wide enough (approximately B+L+50), ships can turn there.

Reclaimed area and efficiency
This alternative has the largest area of reclaimed land, although it lacks one terminal to meet the required capacity in 2020. In the port-area not many other activities will be allowed and it is questionable whether or not this extra space will be used efficiently.

Approximate costs incl. maintenance dredging
The net terminal area is small compared to the total reclaimed area, thus the relative costs per m2 terminal will be high. Approximately 200 ha of nautical area have to be maintained (an estimated volume of 3 million m3 /year)

Environmental impact
The base alternative has quite a streamlined layout and environmental impact will be limited to local effects due to current diversion.

8.2 Alternative 2: Modified AGP Proposal

Capacity, expansion possibilities & phasing
This alternative is a modification of alternative the base layout, to overcome its shortcomings. It meets the required capacity in 2020, using the medium scenario. Future expansion will still be difficult and has to be found either in reorganisation of the current terminals of *Puerto Nuevo* or in a new reclamation on the river side.

Construction in phases is very easy, terminals can be added one by one without causing large problems.

Nautical access, capacity of nautical area and traffic intensity
In order to enhance the capacity of the nautical areas, the most northern quay is extended with one berth and then the bend is made. The smallest passage near pier 1 and 2, which could cause congestion in the port, is hereby removed. (see Figure 7-1 and Figure 7-2)
The basins are longer because of the extra berth (max. 4300m). The turning circle is situated near the entrance of the port in *Antepuerto Norte*. The towing distance for the larger ships, which turn in the turning circle (Post Panamax), is larger than in the base alternative, smaller ships will turn in the basins where possible.

Reclaimed area and efficiency
This alternative has a smaller area and the ratio of terminal area to total area is more beneficial than in the base layout. In the north, a large area may be reclaimed, for other port related activities, such as a Container Freight Station or a distribution centre. The long straight quays are beneficial for terminal efficiency.

Approximate costs incl. maintenance dredging
Because the net terminal area is still small compared to the total reclaimed area, the relative costs per m^2 terminal will be high. The nautical area is somewhat larger than the nautical area of the base alternative, and the maintenance volume will be a bit larger too (3.5 – 4.0 million m3)

Environmental impact
Alternative 2 has a streamlined layout and environmental impact will be limited to local effects due to current diversion, which are expected to be less than in the base alternative.

8.3 ALTERNATIVE 3: COMPACT ISLAND

Capacity, expansion possibilities & phasing
Alternative 3 meets the capacity, required by the medium growth scenario in 2020. If the growth is larger than expected an expansion with 4 berths is relatively easy to accomplish. Phasing can be done in various ways, making it a flexible alternative.

Nautical access, capacity of nautical area and traffic intensity
In this alternative two sides of the island can be distinguished. Smaller ships enter the port for quiet water, while the larger ships, which are less sensitive to wave action (LOA 200 metres and up), can berth at the east side of the island.
Because the traffic of vessels in the port of Buenos Aires is now divided over two sides of the reclaimed island, the traffic intensity on both sides will be smaller then in the other alternatives. In the old port the, ships can turn in or in one of the two turning circles, which are situated in *Antepuerto Norte* and in the northern basin. Because of the lower traffic intensity on the east side of the island, the newly dredged channel has been designed for one-way traffic, two way traffic is not necessary. The ships turn in the turning circle, just north of the access channel.

Reclaimed area and efficiency
Because of its compactness, with this design the most efficient use of land is reached. The relatively small area is mostly used for terminals operations.

Approximate costs incl. maintenance dredging
In this alternative less shore protection is needed because the quay wall on the east side of the island will act as one. A large investment has to be done for the dredging of a new channel and turning circle on the east side of the island. The compactness of the island result in very efficient use of land however, so the price per m^2 terminal could be low after all. The nautical area, which requires maintenance, is large, but it is not quite sure what the required maintenance is for the exposed channel. Assumed is that the maintenance per m^2 of the exposed channel is about the same as the regular access channel (1.5 m / year). The annual maintenance volume will be approximately 3.5 million m^3.

Environmental impact
Alternative 3 is does not have a streamlined layout like alternative 2. The expansion will not have an impact on the complete system, but local impacts will be more severe than in the base alternative and alternative 2. What the impact will be near the exposed quay will have to be studied in more detail.

8.4 ALTERNATIVE 4: ONE LONG QUAY

Capacity, expansion possibilities & phasing
This alternative meets the capacity, required by the medium growth scenario in 2020. If the growth will be larger than expected an expansion with 1 berth can be made, or an expansion of 3 berths in the south. Phased construction of this alternative is quite easy up to 8 berths. After that, a new access has to be made to the new berths and a new breakwater has to be constructed. This required a large investment in dredging and shore protection.

Nautical access, capacity of nautical area and traffic intensity
Of all alternatives, this alternative has the largest area of water and the widest basins, so no congestion problems are to expect. On the other hand this large area of approximately 375 ha requires maintenance too (approximately 5.5 million m3 / year) and travel (towing) distances from the port entrance are large. In order to secure safe mooring for smaller ships, two breakwaters will have to be constructed.

Reclaimed area and efficiency
This alternative has a relatively small area. Because of the straight quay not much space is lost, but the connection from the straight island to the main land takes quite a lot of space.

Approximate costs incl. maintenance dredging
The straight long quay is beneficial for terminal operations, but in order to accomplish this long quay a lot of dredging has to be carried out, and two new breakwaters will have to be constructed. Besides that, the new nautical area requires a lot of maintenance dredging; all of these measures are quite costly and will affect the price per m^2 reclaimed land greatly.

Environmental impact
The most severe impact on the environment will occur in the south, near the new breakwaters. Current diversion around the breakwaters will cause a change in cross currents and sedimentation and erosion patterns. The determination of the exact

impact is beyond the scope of this report and a morphological study has to be carried out to estimate this impact.

8.5 ALTERNATIVE 5: NORTHERN EXPANSION

Capacity, expansion possibilities & phasing

Just as alternatives 2, 3, 4 and 6, alternative 5 meets the capacity, required by the medium growth scenario in 2020. If the growth is larger than expected an expansion can be made with about 4 berths. To do so the quay wall is extended as in alternative 2. Starting in the north the berths can be constructed one by one, not requiring very large investments at a time, and not effecting the environment much

Nautical access, capacity of nautical area and traffic intensity

Of all of the alternatives mentioned, the travelling distances are the longest in this alternative. When the current breakwater is left as it is in the southern part of the port, the small passage near pier 1 and 2 may cause congestion[3]. There is no specific turning basin in the vicinity of the berths, but only in *Antepuerto Norte*. The basins are the wide on the other hand so that turning around near the quay is possible for smaller ships. Safe mooring conditions for all sizes of ships is guaranteed.

Reclaimed area and efficiency / Approximate costs incl. maintenance dredging

This alternative has a relatively small area as well, because the terminals are positioned very close to the main land. Together with the short distances for shore protection, breakwater removal and breakwater construction, this will keep the prices per m^2 low.

Environmental impact

The environmental impact of alternative 5 will be mostly limited to local diversions of the current. Especially in the south this may have some consequences. In the north, the current is more regulated. The determination of the exact impact is beyond the scope of this report and a morphological study has to be carried out to estimate this impact.

8.6 ALTERNATIVE 6: THE BOOMERANG

Capacity, expansion possibilities & phasing

Like the previous mentioned alternatives, this alternative has been designed to meet the capacity, required by the medium growth scenario in 2020. If the growth will be larger than expected an expansion with 1 berth can be made. Other adjustments must be found in new reclamation or reorganisation of the old (current) Puerto Nuevo. Construction of this alternative in phases in requires large investments at an early stage to give access to the new berths

Nautical access, capacity of nautical area and traffic intensity

This alternative has a large area of water and the wide basins, so no congestion problems are expected. Travelling distances are not that long and 2 turning circles are available. Only a small breakwater has to be constructed to assure mild mooring conditions for smaller ships

Reclaimed area and efficiency

The use of the land is efficient because of the relatively small area and short connections to the main land.

[3] The minimum width may seem about 240 metres, but about 180 m will be available at full depth of 13.2 m. Otherwise adjustments to the existing quay walls are necessary.

Approximate costs
This alternative is quite expensive because of its large investments in capital dredging works (northern basin). The price per m^2 reclaimed land will be quite high, but efficient use of land makes the price of net terminal area competitive. Because of the large nautical area, this alternative requires a lot of maintenance (4.5 million m3 /year).

Environmental impact
The environmental impact of alternative 6 will be the worst in the north, where the peninsula bends south an approximately 2000 metre long east-west expansion (see appendix D)

8.7 EVALUATION OF ALTERNATIVES

8.7.1 Comparison

In Table 8–1 below the most important design characteristics have been summarized.

RECLAMATION	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6	
Area of reclaimed land	278	259	259	245	262	278	ha
Efficiency	-	+	++	-/+	+	+	-
Quay length	3500	3600	3575	3575	3575	3575	m
Shore protection	6700	6850	3680	6330	5200	6360	m
Breakwater construction	0	500	200	2800	250	1050	m
Breakwater removal (small)	1150	1150	600	1150	1150	1150	m
Breakwater removal (large)	2815	2815	2100	2815	1550	2815	m

Table 8–1 Summary of most important alternative characteristics[4]

[4] The yellow cells show the most favourable values, the red ones the unfavourable values.

In Table 8–2 alternatives 1 to 6 are compared qualitatively on the same aspects on which the alternatives have been evaluated in paragraphs 8.1 to 8.6. It shows the relative merits of the alternatives for each aspect.

	Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5	Alt. 6	Remarks / units
CAPACITY AND BERTHS							
Number of berths	12	13	13	13	13	13	
Expansion possibility	0	0	++	+	+	+	
Number of possible extra berths	0	0	4	1	4	1	
Subtotal capacity & berths	-	*1*	*5*	*3*	*4*	*3*	
CONSTRUCTION							
Phasing	+	++	+	+/-	+	-	
Subtotal construction		*5*	*4*	*2*	*4*	*1*	
NAUTICAL AREA							
Convenience	-	+	++	+	-	+	+ = high safety
Traffic intensity	- -	-	++	+	- -	+	+ = low intensity
Subtotal nautical area		*3*	*5*	*4*	*1*	*4*	
MAINTENANCE							
Maintenance to nautical area (dredging)	0	0	-	- -	-	-	0 = reference + = less maintenance
Maintenance to quay wall	0	0	-	0	0	0	0 = reference + = less maintenance
Maintenance to shore Protection	0	0	+	0	+	0	0 = reference + = less maintenance
Maintenance to breakwater	0	0	0	- -	-	-	0 = reference + = less maintenance
Subtotal maintenance		*5*	*3*	*1*	*4*	*3*	
COSTS							
Reclamation costs	0	+	+	-	+	-	0 = reference + = less costs
Capital dredging costs	0	0	0	- -	0	-	0 = reference + = less costs
Maintenance dredging costs	0	0	0	- -	0	- -	0 = reference + = less costs
Total costs per m² terminal (includes efficiency)	0	+	++	-	+	-	0 = reference + = less costs
Subtotal costs		*3*	*4*	*1*	*3*	*2*	
ENVIRONMENTAL IMPACT							
Diversion of current in final phase	0	+	-	-	0	-	0 = reference + = less diversion
Diversion of current during construction	0	0	-	- -	0	- -	0 = reference + = less diversion
Impact on ecological Reserve	0	0	0	-	0	0	0 = no impact - - = large impact
Subtotal environmental impact		*5*	*3*	*2*	*4*	*2*	

Table 8–2 Evaluation table of alternatives

8.7.2 Multi Criteria Analysis

Table 8–2 shows that every alternative has its advantages and its disadvantages. A choice between these alternatives is made with a Multi Criteria Analysis (MCA).
The scores of the alternatives in table 8.2 are transformed into values of the MCA by applying a multiplier for each aspect (results in Table 8–3). These multipliers show the relative importance of each aspect (The exact values of the multipliers are considered to be less important than the ranking)

Relative importance of comparison aspects:

- The most important element of the port expansion is *costs*, both construction costs and maintenance costs. These are not shown quantitatively in Table 8–2, because it is difficult to estimate the costs of all alternatives. Qualitatively an estimate has been be made, for example: large maintenance works mean large annual costs and a large area, either land area or nautical area, which is inefficiently used, causes unnecessary construction (and maintenance) costs. Used multiplier in MCA: 8

- The second element is the *nautical access and traffic intensity*. When ships cannot manoeuvre and berth safely in the port, this will affect the port operations and can cause additional congestion and downtime for the terminals.
  Used multiplier in MCA: 7

- The third very important element of port expansion is *capacity*. Not having enough capacity means queuing of ships and long waiting times.
  Used multiplier in MCA: 5

- The fourth element is *construction*. Construction of the expansion has to be done in phases, but when phased construction causes large problems (impact on environment, very large investments, unsafe mooring conditions) it could be necessary to reduce the number of phases.
  Used multiplier in MCA: 4

- The fifth element is *maintenance*. Large nautical areas require a lot of maintenance, which is not only costly, but dredging operations can hinder the manoeuvring of ships as well.
  Used multiplier in MCA: 3

- The sixth and last element is *environmental impact*. None of the alternatives will have a large impact on the system of the Rio de la Plata. However, local impacts can be more severe and require countermeasures like extra bottom protection, heavier quay walls, additional maintenance dredging.
  Used multiplier in MCA: 3

Multi criteria analysis (MCA) in tabular form:

Criterium	Factor	Alt. 2	Alt. 3	Alt. 4	Alt.5	Alt. 6
Costs	8	24	32	8	24	16
Nautical Access	7	21	35	28	7	28
Capacity and berths	5	5	25	15	20	15
Construction (phasing)	4	20	16	8	16	4
Maintenance	3	15	9	3	12	9
Environmental Impact	3	15	9	6	12	6
Total		***100***	***126***	68	91	78

Table 8–3 Multi Criteria Analysis of alternatives 2 to 6

The multi criteria analysis (MCA) above in Table 8–3 shows that alternatives 2 and 3 are the two best alternatives.

Besides this qualitative evaluation, there can be other reasons, why an alternative is not chosen. These reasons are described below:
Alternative 4 is not chosen, because of the large area of water, which has to be dredged and maintained. In addition two long breakwaters will have to be constructed. All together this will effect the costs and maintenance volumes in a negative way.
Alternative 5 is not chosen, because the sailing distances are too long in narrow basins, this will cause congestion and affects sailing manoeuvres in the port. Turning at the most northern quays is not possible, not even for smaller ships (LOA 100-200 m).
Alternative 6 seems a good alternative in the final phase. During constructing phasing is not so easy, the environment is affected and a large investment is required in an early stage.

Alternatives 2 and 3, which remain after the MCA will be further analysed in the next chapter.

9. ANALYSIS OF MOST PROMISING ALTERNATIVES

In this chapter, alternatives 2 and 3 will be further analysed. First the key data for both alternatives are presented, then cost estimates for both alternatives, based on unit prices are given and finally total comparison is made between the alternatives

9.1 KEY DATA FOR ALTERNATIVES 2 AND 3

Criterion	Alternative 2	Alternative 3
CAPACITY		
Quay length in port	3600 m	2200 m
Quay length exposed	0 m	1375 m
Number of berths	13	13
Possible extension	0	4
RECLAIMED LAND		
Net terminal area	161 ha	161 ha
Other terminal area	10 ha	28 ha
Transport corridor	61 ha	48 ha
Refill of basin F	17 ha	13 ha
Other reclaimed land	102 ha	9 ha
Total	259 ha	259 ha
Ratio (terminal area / total area)	1.51	1.37
TRANSPORT CORRIDOR		
Road	6500 m	5500 m
Railroad	p.m.	p.m.
SHORE PROTECTION & BREAKWATERS		
Riverside protection	5980 m	3680 m
Shore protection in port	870 m	0 m
Breakwater construction	500 m	250 m
Breakwater removal (small)	1150 m	600 m
Breakwater removal (large)	2815 m	2100 m
NAUTICAL AREA		
Nautical area (basins)	259 ha	253 ha
Number of turning circles	1	2 (+1)
Maximum distance from nearest TC	3000 m	2250 m
Minimum width in Puerto Nuevo	300 m	240 m

Table 9-1 Key data for alternatives 2 and 3

9.2 COSTS OF ALTERNATIVES

9.2.1 Cost estimate parameters: unit prices

To make an estimate for the construction and maintenance costs of both alternatives, assumptions and estimates have been made for the unit price of different components of the port expansion. These have been summarized in Table 9-2.
N.B. The costs for e.g. terminal equipment and paving of terminal stacking area have not been included. These are the choice of the terminal operator

Component	Unit price	Source:
QUAYS AND BERTHS		
Quay wall not- exposed to waves	$33,000 / m	Prelim. Calculation (+/- 30%), Ch. 14
Quay wall exposed to waves	$33,000 / m	Prelim. Calculation (+/- 30%), Ch. 14
DREDGING WORKS[5]		
Dredging alluvial deposits, dump at sea	$ 4.50 / m³	Prelim. Calculation , Ch. 15
Dredging alluvial deposits, pump to pit	$ 2.00 / m³	Estimate
Dredging Tosca (large CSD)	$ 3.00 / m³	Prelim. Calculation, Ch. 15
Dredging Sand (large DSD)	$ 2.25 / m³	Prelim. Calculation, Ch. 15
Maintenance dredging	$ 2.00 / m³	Estimate
Dredging of basins	$ 3.50 / m³	Estimate
Vertical drains	$ 40,000 / ha	Prelim. Calculation
SHORE PROTECTION & BREAKWATERS		
Protection Riverside	$ 3,000 / m	Estimate
Removal of small breakwater	p.m.	
Removal of large breakwater	p.m.	
Construction of breakwater (materials)	$ 7,500 / m	Estimate (+/- 30%)
Construction of breakwater (operational costs)	$ 2,500 / m	Estimate (+/- 30%)
Reuse of armour stone	p.m.	
TRANSPORT CORRIDOR		
Road	$ 3,000 / m	Estimate
Railroad	p.m.	

Table 9-2 Cost estimates for various components of port expansion

[5] Reclaimed land + nautical area

9.2.2 Costs of alternatives

In the table below the costs of both alternatives have been summarized. The calculations are based on the unit prices in paragraph 9.2.1 and have not been corrected for inflation and interest of future expenses. Detailed costs can be found in Appendix H.

Criterion	Alternative 2	Alternative 3
QUAYS AND BERTHS		
Quay in port	118.80	72.60
Quay exposed	0	45.38
Total quays and berths	**118.80**	**117.98**
RECLAIMED LAND		
Dredging & dumping of alluvial deposits	20.61	20.61
Dredging + reclaiming *tosca*	50.31	50.31
Dredging + reclaiming sand	43.63	43.63
Vertical Drains	10.36	10.36
Total reclamation dredging	**124.91**	**124.91**
SHORE PROTECTION & BREAKWATERS		
Riverside protection	20.55	11.04
Breakwater construction	5.00	2.50
Breakwater removal (small)	p.m.	p.m.
Breakwater removal (large)	p.m.	p.m.
Total shore protection & breakwaters	**25.55**	**13.54**
TRANSPORT CORRIDOR		
Road	19.50	16.50
Rail Road	p.m.	p.m.
Total transport corridor	**19.50**	**16.50**
NAUTICAL AREA		
Capital dredging basins	40.46	20.13
Capital dredging eastern channel	-	9.30
Capital dredging access channel	72.25	72.25
Total capital dredging	**112.71**	**101.68**
Total construction costs	**401.47**	**374.60**
MAINTENANCE		
Annual maintenance Basins	8.08	5.91
Annual maintenance eastern channel	-	1.62
Annual maintenance access channel	7.26	7.26
Total Annual maintenance	**15.34**	**14.79**
Total Capital Construction + 20 years of maintenance	***708 M$***	***670 M$***
Costs per m² reclamation	***$ 273/m²***	***$ 259/m²***
Costs per m² terminal	***$ 414/m²***	***$ 355/m²***

Table 9-3 Estimated costs for alternatives 2 and 3, costs in millions of US $, unless stated otherwise

Not only the capital construction costs, but the maintenance costs have been stated as well. The total estimated amount for the port expansion has been chosen to be an addition of the capital construction costs and the maintenance costs for 20 years.

9.3 COMPARISON OF SELECTED ALTERNATIVES

Alternatives 2 and 3 both meet the required capacity in 2020, which follows from the medium growth scenario, but they differ in quite a few aspects: the use of the reclaimed land, the phasing of construction, expansion possibilities, nautical access, intensity of traffic, length of shore protection and costs.

9.3.1 Reclaimed area and efficiency

Alternatives 2 and 3 have the same reclaimed land area, 259 ha, but the difference in the shape of the designs causes some differences in land use.
Alternative 2 is stretched along a long part of the current port and needs a long transport corridor. Alternative 3 on the other hand, has terminals positioned on both sides of the peninsula terminals and both can use the same transport corridor. This results in a smaller area and in a slightly more efficient use of land.
For both alternatives goes that in the areas, which cannot be used as terminal area (for example basin F), distribution centres or container freight stations will be built.

9.3.2 Phased construction and expansion possibilities

Both alternatives will be constructed in more than one phase, because not all terminals are required at once and investments can be spread in time. Alternative 2 offers the best possibilities for construction in phases; it will not effect the environment much in the first phases and terminals can be built one at a time. However, after completion of the last terminal, there is not much possibility to expand the capacity of the port; either the old terminals must be reorganised to enhance their capacity or another large expansion into the Rio de la Plata has to be made.
Alternative 3 can be constructed with different phasing methods. One method starts the construction just as the previous alternative, and when the 8 berths at the port side are completed, the riverside berths are built. The investment of dredging the eastern access can be made at a later stage when the first riverside terminals are built. If throughput forecasts predict larger growth during the construction of the different phases, four extra berths can be added to the design of this alternative by extending the berths on both sides of the peninsula.

9.3.3 Nautical access & traffic intensity

Alternative 2 has the most conventional layout of both alternatives, but the long quay and single turning circle result in large travelling distances in the port and high traffic intensity. Turning in the basin is possible for smaller vessels, but widening the basins to make turning possible for all vessels results in an excessive large nautical area and consequently more costs of maintenance.
Alternative 3 has the best nautical access of the alternatives posed here. Because of the secondary access channel at the Rio de la Plata side, the traffic intensity will be lower and the three turning circles secure safe turning. Of course this requires an additional investment in channel dredging (see costs Table - Capital Dredging of Eastern Channel). Compared to alternative 2, distances in port are short.

9.3.4 Shore protection

Alternative 2 has the longest shore protection of both alternatives, but requires the construction of only 500 metres of new breakwater. The stones of the old breakwater, which will be removed, might be used both in the shore protection and in the new breakwater.
As mentioned above, alternative 3 has an exposed quay of 1375 metres length. This makes the design compact and results in a smaller shore protection. To protect the old

port from waves and currents, only a small length of new breakwater has to be constructed, for which stones from the current breakwater can be used.

9.3.5 Costs

Table 9-3 shows that alternative 3 is the cheapest alternative, both in costs per m^2 reclamation (US$ 259) and in costs per m^2 terminal (US$ 355), because it makes more efficient use of its land and it requires a smaller investment. However, the cost difference with alternative 2 is small.

9.4 CONCLUSIONS

Alternative 2, the modified AGP-proposal, is only slightly more expensive than alternative 3, it has the lowest impact on the environment and its terminals can be constructed one at a time. On the other hand there is a lot of spare area, a higher traffic intensity, only one turning circle and longer travelling distances.
Alternative 3, the compact island, requires the lowest total investment, has the lowest price per m^2 terminal and thus uses its land the most efficient. The traffic intensity is lower and the nautical safety is high. It can easily be constructed in phases and the large investment in dredging the eastern access channel can be delayed for some time.

As described above both alternatives have their advantages and disadvantages. On basis of the cost criterion, alternative 3 is chosen as the most promising alternative. In the preliminary design, presented in previous chapters an important assumption has been made, which needs to be verified:

> *Large containerships are assumed to berth safely at the exposed berths on the eastern side of the island; waves and currents do not cause significant down time for the container terminals.*

A downtime calculation will be made for alternative 3 in the next part of the report. For general parameters, which can cause down time for all previously described alternatives a downtime estimated has been made in chapter 10. In following chapters 11 and 12 specific phenomena have been analysed, which can cause additional downtime of the exposed berths of alternative 3.
These phenomena are:

- currents near the berth
- wave-induced ship motions

The costs of the quay wall depend largely on its design and the construction method. In chapters 13 and 14 a preliminary design for the quay wall is presented and construction and costs are determined.

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BUENOS AIRES PORT EXPANSION 2020

PART 3:

DOWNTIME CALCULATION

10.DOWN TIME CALCULATION OF PORT INFRASTRUCTURE

In order to estimate the down time of the terminals and the accessibility of the port in 2020, first a general down time calculation has been made for alternatives 2 and 3. The following several parameters have been analysed in this chapter:

- Channel depth
- Wave height for sailing ships and tug operations
- Wind
- Traffic intensity in turning circle

After these calculations some specific parameters that can cause additional down time for the exposed berths of alternative 3 have been calculated in the next chapters:

- Flow pattern (velocity) near exposed berth (chapter 11)
- Wave induced motions of ship moored at exposed berth (chapter 12)

The down time of the port has been determined for each parameter by checking their values and their rate of occurrence against general accepted limiting values as stated in chapter 5. Reasonable accessibility rates are based on assumptions made by the author:

- The new port has to be accessible for the largest ships for at least 95 percent of the time
- Smaller ships or partially loaded ships must be able to enter the port at all times (> 99 percent of the time)
- Loading and unloading operations must not be hindered by waves, currents or wind for at least 95 percent of the time

10.1 DOWN TIME DUE TO CHANNEL DEPTH

The down time of the channel depends largely on the available net channel depth in various weather conditions. Down time caused by channel width and traffic intensity is neglected because the channel has been designed as a two-way channel in previous chapters and the amount of traffic is not that large that congestion in the channel is expected.

Different types of ships will navigate the channel and for most of the smaller ships, the design depth of the channel (CD -13.2 m) is sufficient. The design ships however, which are governing for the dimensions of the access channel may have some difficulty to enter the port in extreme cases. The accessibility of these design ships is analysed below.

The required navigating depth[1] of the channel, h, is calculated as an addition of various parameters:

Z: water level variation [m rel. to CD]
T: at-rest draft of design ship
z_{sq}: squat of the ship at a certain sailing speed
z_{mov}: wave-induced motion of the ship
z_{UKC}: under keel clearance (required depth between seabed and keel of ship)

[1] The construction depth is not equal to the navigating depth; sounding tolerances, dredging tolerances and some space for accretion have to be taken into account

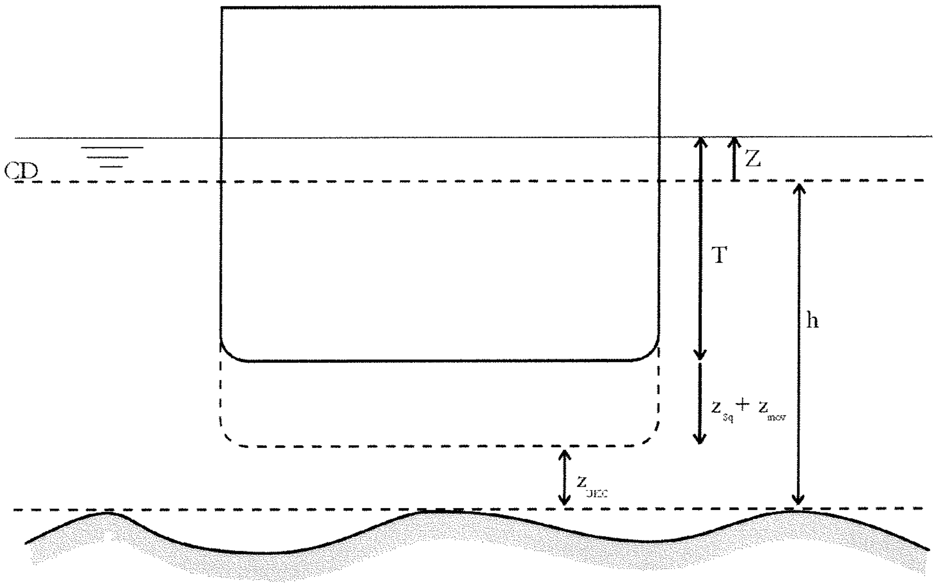


Figure 10-1 Definition of navigating depth

Written as an equation:

$$h = Z - (T + z_{squat} + z_{mov} + z_{UKC}) \quad [metres - C.D.] \quad \textit{Equation 10–1}$$

The channel is accessible when the available channel depth (Z-h) is larger than the required channel depth (T $+z_{squat}+z_{mov}+z_{UKC}$)

10.1.1 Available channel depth

The available channel depth (Z-h) is determined by:

- Water level variation in time (tidal cycle, discharge of Rio de la Plata)
- Set-up / set-down (wind direction, wind force)
- Dredged bottom level

Unlike the *required channel depth*, which can be calculated with various theories, formulas and measurements, the variation of the *available channel depth* in time is difficult to predict. The available channel depth is determined by natural conditions: tides, the discharge of the Rio de la Plata, wind direction and wind force. The deviation of the water level from the tidal cycle, which is often caused by wind (either set-up or set-down), is described here in an statistical way.

As can be seen from Table 10–1, which is obtained by combining appendix C.1 and C.2, most of the time the values for set-up and set-down are quite small. For the calculation of the channel depth only set-down is important, because it reduces the maximum allowable draft. The set-down value is the only statistical parameter determining the available channel depth and down time percentages are directly related to the statistical distribution of the set-up and set-down.

Value of set-up / set down	Occurrence (% of time)
> 0.70	2.16 %
0.70 – 0.60 +	2.52 %
0.60 – 0.50 +	4.26 %
0.50 – 0.40 +	2.83 %
0.40 – 0.30 +	8.78 %
0.30 – 0.20 +	9.93 %
0.20 – 0.10 +	6.73 %
0.10 + – 0.10 -	30.19 %
0.10 – 0.20 -	10.89 %
0.20 – 0.30 -	16.53 %
0.30 – 0.40 -	3.13 %
> 0.40 -	2.05 %

Table 10–1 Chance of occurrence of set-up and set-down values

10.1.2 Required channel depth

The required channel depth is an addition of the following parameters:

- At-rest draft of the ship
- Squat at a certain sailing speed
- Wave induced ship motions
- Required Under Keel Clearance (UKC)

Draft of ship: relation between draft and load

The required accessibility of a port will not be the same for all types of ships; smaller ships or partially loaded ships, calling frequently at the port must be able to enter the port at all times (extreme cases excluded), while very large ships, which call less frequently, may have restricted access to the port, for example a tidal window. To determine the accessibility for the design ships, a relation between the draft of the ship and the amount of cargo it is carrying is used in this report. According to a source at P&O Nedlloyd, the Netherlands, a Panamax containership with a maximum draft of 13 metres has the following distribution of draft vs. load (see Figure 10-2)

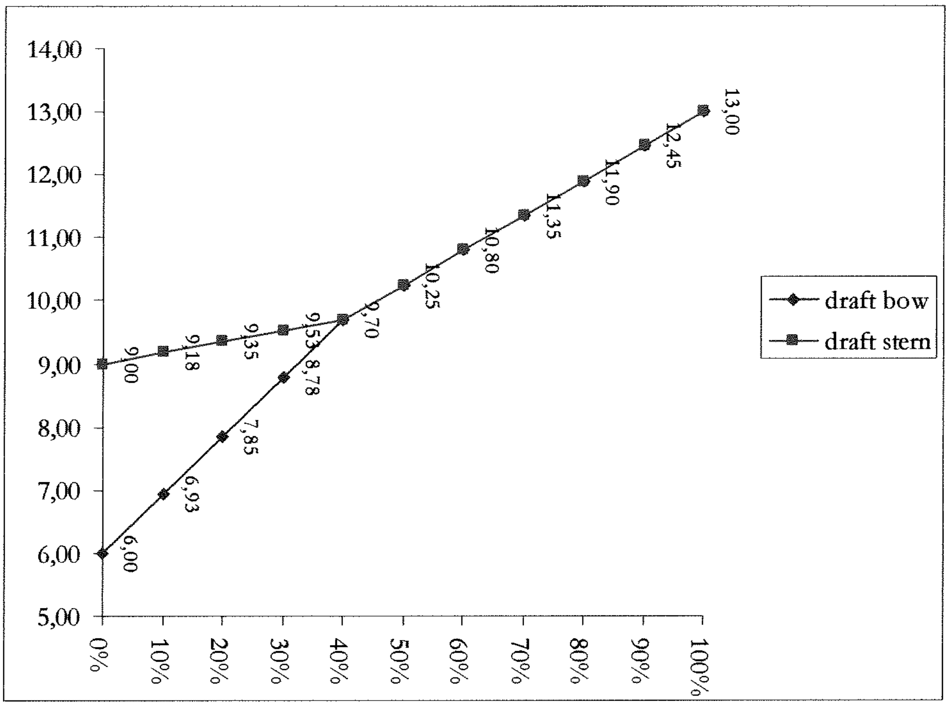


Figure 10-2 Draft vs. Load for a large Panamax container ship, maximum draft 13 m, Source: Personal communication

The stern of an empty ship usually has a larger draft than the bow, because the engine room and the bridge of the ship are situated at the back of the ship. When the ship is loaded, the bow will sink faster than the stern and at approximately 40% of the load, the draft of the stern and bow becomes about equal.
The relation of Figure 10-2 has been scaled to fit the maximum draft of the design ships.

Panamax Containership (PX)
The Panamax design ship has a maximum draft of 12 metres. When the above used relation between draft and load for the Panamax container ship is scaled the following relation between draft and load is found, Figure 10-3.

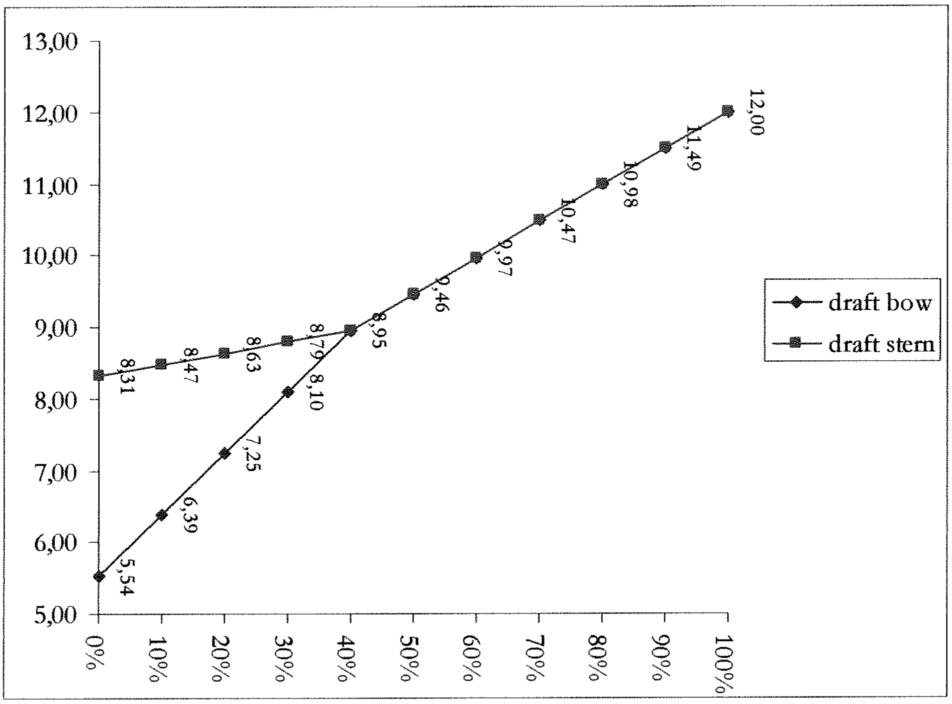


Figure 10-3 Draft vs. Load for a Panamax container ship, maximum draft 12 m

When the ship is only loaded for 50 % it has a draft of approximately 9.5 metres.

Post Panamax containership. (PPX)

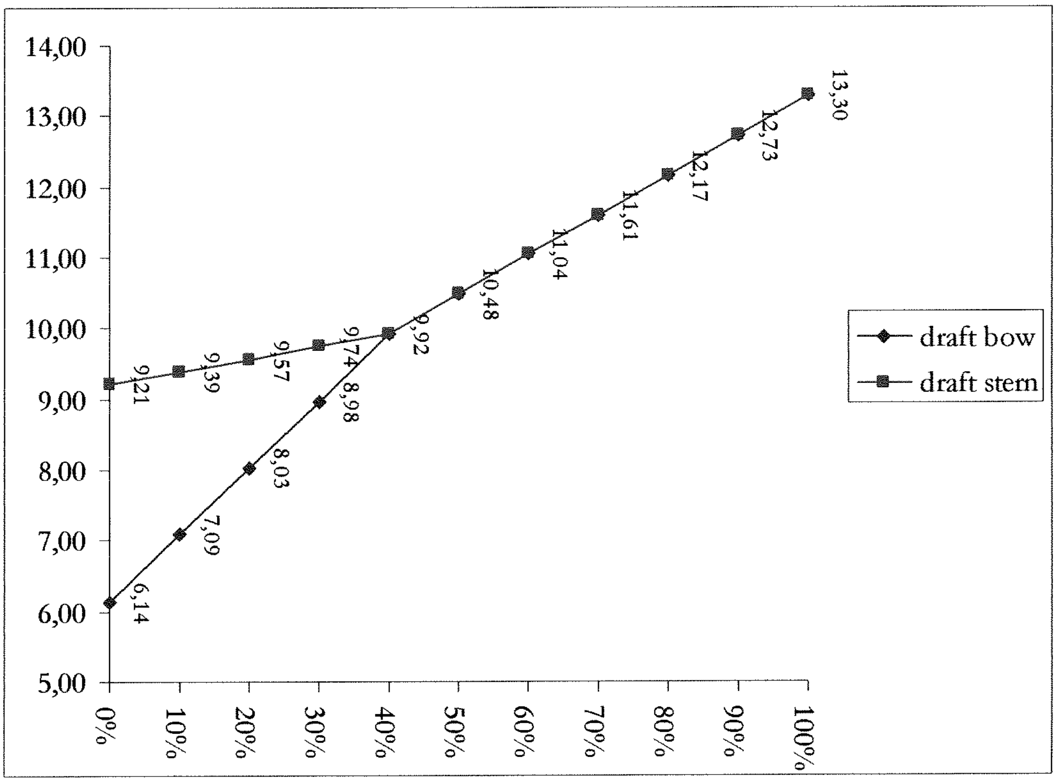


Figure 10-4 Load vs. draft for a Post-Panamax ship, maximum draft 13.3 m

When Figure 10-2 is scaled to a similar graph for the Post-Panamax design ship, it is found that fully loaded, this ship has a draft of 13.3 metres and when loaded for 50% the draft is approximately 10.50 metres (see Figure 10-4).

For both the fully and half loaded design ships, the channel depth has been validated.

Squat
The squat of a ship depends on the dimensions of the ship, the water depth and the sailing speed of the ship and can be calculated with the following formula, [*PIANC, 1995*]:

$$z_{sq} = 2.4 - \frac{\nabla}{L_{pp}^2} \cdot \frac{F_{nh}^2}{\sqrt{(1 - F_{nh}^2)}} \quad , \quad F_{nh} = \frac{V}{\sqrt{(g \cdot h)}}$$

in which:

V:	sailing speed in m/s: maximum speed in access channel: 8 knots (4 m/s)
∇:	displacement
L_{pp}:	length between perpendiculars
F_{nh}:	Froude number "for water depth"
h:	water depth

The squat values for both design ships sailing at 8 knots (4 m/s) are stated below.

Ship, load	At-rest draft	Squat	Sailing draft
Panamax, 100 %	12.00 m	0.32 m	12.32 m
Panamax, 50%	9.46 m	0.25 m	9.71 m
Post Panamax, 75 %	12.00 m	0.36 m	12.36 m
Post Panamax, 50 %	10.48 m	0.31 m	10.79 m

Table 10–2 Squat values for sailing design ships

As the *sailing* draft of the fully loaded Panamax ship and the partially loaded Post Panamax ship does not differ much (only 0.04 m), these two ships considered together in the remainder of this chapter; *sailing* draft 12.36 m.

Wave induced motions of sailing ship
As mentioned above, the channel depth is validated for off-shore winds, as set-down of the water level reduces the available channel depth. The highest waves, which occur from off-shore wind directions (SW-N-NE), are smaller than the design waves for Buenos Aires. A significant wave height is chosen, which has an occurrence of less than one percent: H_s=1.4 m, T_s=4.7 s (See Appendix C)

To estimate the wave induced ship motion, calculations have been made with the computer program SEAWAY, from the faculty of Maritime Technology. The program calculates the significant motion amplitudes of the ship for a complete spectrum of waves using the strip theory. Different sailing speeds, wave directions, significant wave heights and periods can be chosen freely. The calculated ship motions are linear proportional to the wave height in this computer program and the significant wave height has been chosen as 1.0 metre. The motions per metre of wave height are calculated in this way.
As the used computer program was licensed for the calculation of one ship only, the motion due to wave action has been calculated only for the loaded Panamax container ship sailing the access channel (at-rest draft: 12.0 m, water depth of access channel 13.2

m [2], sailing speed 8 knots). Three wave directions have been chosen: 45, 90 and 135 degrees off stern and two sailing speeds: 4 and 8 knots, see Figure 10-5

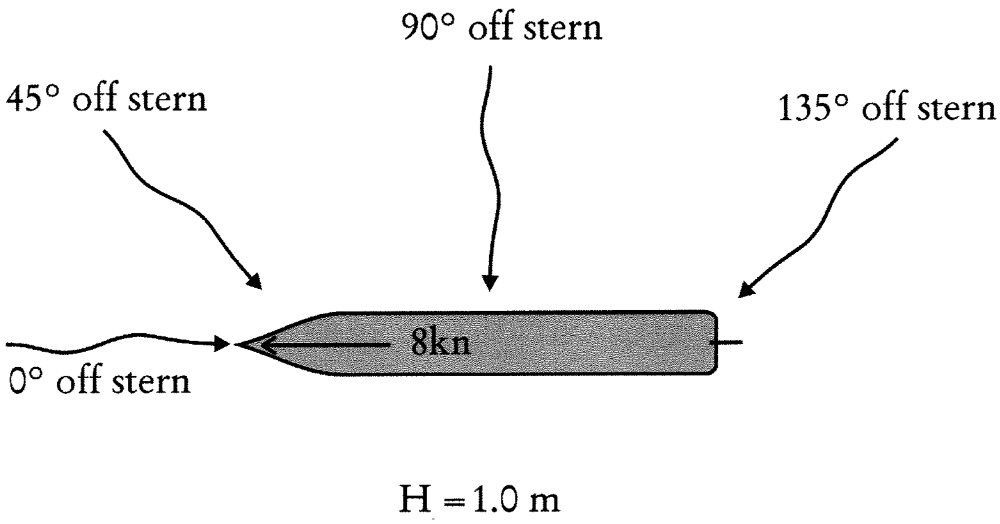


Figure 10-5 Considered wave directions

The limiting wave direction is found to be 45 degrees off stern, for a ship sailing with a speed of 8 knots. The significant motion amplitude of the bow of the ship with a wave of 1.0 metre and a zero crossing period of 5.0 seconds (the maximum period from that direction) amounts 0.15 metres. As the actual design wave height amounts 1.4 metres instead of 1.0 metre, the *maximum* amplitude of the bow amounts: 1.8 * 1.4 * 0.15 = 0.38 metres[3]. This value has used for both design ships sailing either fully loaded or half loaded.

Note: There are other wind directions, from which higher waves occur, but these wind directions are accompanied by set-up. Movements will obviously increase, but due to the increased water depth, these situations are not governing. The maximum movement from those directions is with a wave of 1.0 metre and a period of 6.3 seconds: 0.25 metres.
This leads to the total amplitude of 1.8 * 1.8 * 0.25 metres = 0.81 metres

Under Keel Clearance
According to *Thoresen*, the net underkeel clearance z_{UKC} in access channels with a soft bottom surface has to be at least 0.5 metres.

10.1.3 Validation of the channel depth

Required water depth
The parameters, which influence the draft of the ship (draft at certain load, squat and motions of the ship and UKC), have been added to the normal draft of the ship leading to an increased (virtual) draft. This draft has been subtracted from the normal sea water level variation and this transposed sine wave shows the position of the ship's keel in time during a tidal cycle: the increased required water depth.

---

[2] As the calculated channel depth h amounts CD-13.2 and MLLW is CD+0.3 m, the depth of the access channel usually amounts 13.5 m or more. A variation in the chosen water depth of some decimetres hardly influences the ship motions.
[3] The factor 1.8 converts the significant amplitude to a maximum amplitude of the motion

Available water depth

The parameter, influencing the normal sea level variation (set-down), is a statistical parameter. Several values of this parameter have been added to the seabed position, resulting in several reduced bottom levels. The distance between both lines is the net distance between keel and sea bottom for certain natural conditions; when the translated sea-level (the keel's position) crosses the translated seabed the available depth is not sufficient, which results in down time.

The following considerations have been taken into account:

- The set-up and set-down values have been calculated for certain intervals. The maximum value of that interval has been used in the calculation. In the last interval all remaining set-down values, which are greater than 0.40 m have been put together. The maximum value of this interval which amounts approximately 1.0 m is considered to be too large and an average value of 0.60 m has been taken as representative value of the interval, see Table 10–1.
- Sailing time of the access channel to Buenos Aires (11.4 kilometres) is estimated to be 1 hour (sailing speed 8 knots and slowing down). When the water level is not sufficient during for example 2 hours, the channel is not available for 2 + 1 = 3 hours.
- For each set-down value, which limits the accessibility of the port, the down time has been calculated as a percentage of time.
- For the tidal variation of the water level an average asymmetrical tide has been assumed; a high tide is followed by a lower tide (MHHW-MLLW-MLHW-MHLW)

10.1.4 Accessibility calculation for channel depth of CD -13.1 metres

As can be seen from Figure 10-6, the access channel with a depth of CD -13.1 m is unavailable during part of the tidal cycle in many weather conditions. The accessibility of the port for the largest ships less than the required 95 percent: 94,07%, see Table 10–3 below.

For ships with smaller drafts than approximately 11.50 metres, accessibility is guaranteed during the total tidal cycle for all weather conditions.

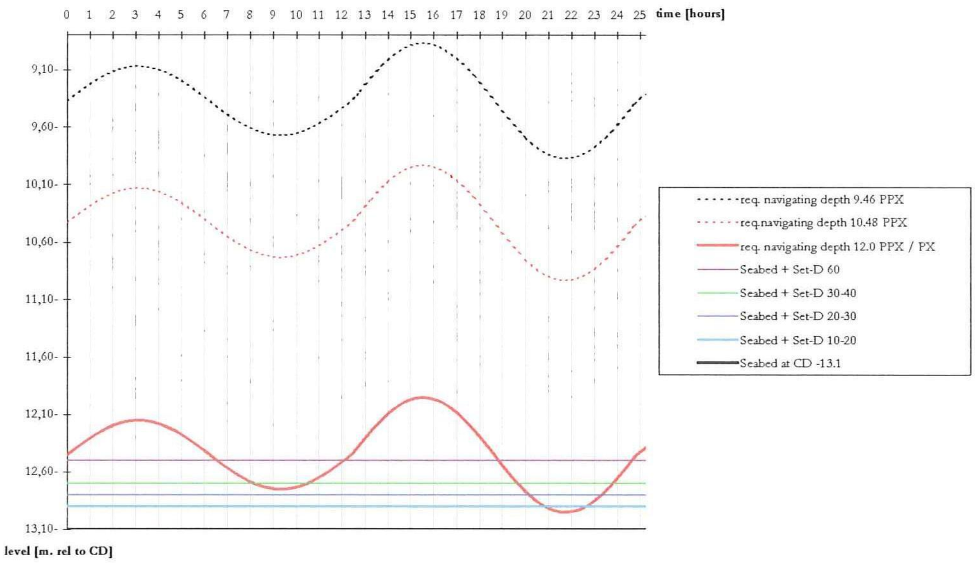


Figure 10-6 Accessibility determination at channel depth CD-13.1 m

Set-down interval	Occurrence	Value used	# of hours, depth not available	# of hours port not accessible	% of time, port not available
< -0.40	2.05 %	-0.60	5 & 5.83	12.83	1.06%
-0.40 to -0.30	3.13 %	-0.40	2 & 4.5	8.5	1.07%
-0.30 to -0.20	16.53 %	-0.30	3	4	2.66%
-0.20 to -0.10	10.89%	-0.20	1.6	2.6	1.14%
> 0.10	67.40 %	-	0	0	0
Total	100.00%				5.93%
PORT ACCESSIBILITY					94.07 %

Table 10–3 Accessibility determination at channel depth CD-13.1 m

10.1.5 Accessibility calculation for channel depth of CD -13.2 metres

As can be seen from Figure 10-7, the available depth of the channel is not sufficient during 3.50 hours of the tidal cycle, when a set-down of 0.60 metres occurs. This results in an accessibility of more than 99 percent for the deepest ships (12 metres draft), Table 10–4. For ships with smaller drafts channel depth is not limiting the access to the port.

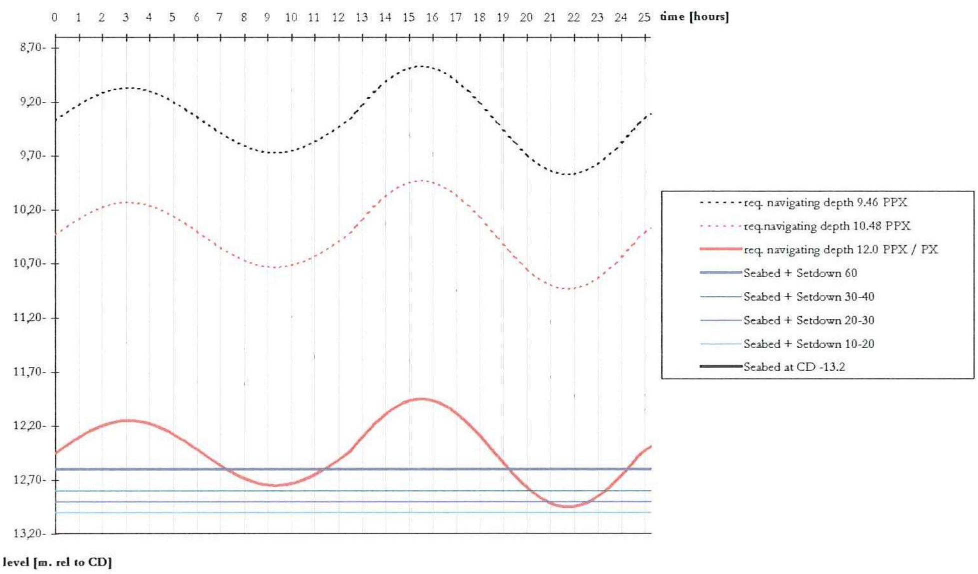


Figure 10-7 Accessibility determination at channel depth CD -13.2 metres

Set-down interval	Occurrence	Value used	# of hours, depth not available	# of hours port not accessible	% of time, port not available
< -0.40	2.05 %	-0.60	4 & 5	11	0.91%
-0.40 to -0.30	3.13 %	-0.40	3	4	0.50%
-0.30 to -0.20	16.53 %	-0.30	1.8	2.8	1.86%
-0.20 to -0.10	10.89%	-0.20	0	0	0
> 0.10	67.40 %	-	0	0	0
Total	100.00%				3.28%
PORT ACCESSIBILITY					96.72 %

Table 10–4 Accessibility determination at channel depth CD-13.20 m

10.1.6 Accessibility calculation for channel depth of CD -13.3 metres

As can be seen from Figure 10-7, the available depth of the channel is not sufficient during 3.50 hours of the tidal cycle, when a set-down of 0.60 metres occurs. This results in an accessibility of more than 99 percent for the deepest ships (12 metres draft), Table 10–4. For ships with smaller drafts channel depth is not limiting the access to the port.

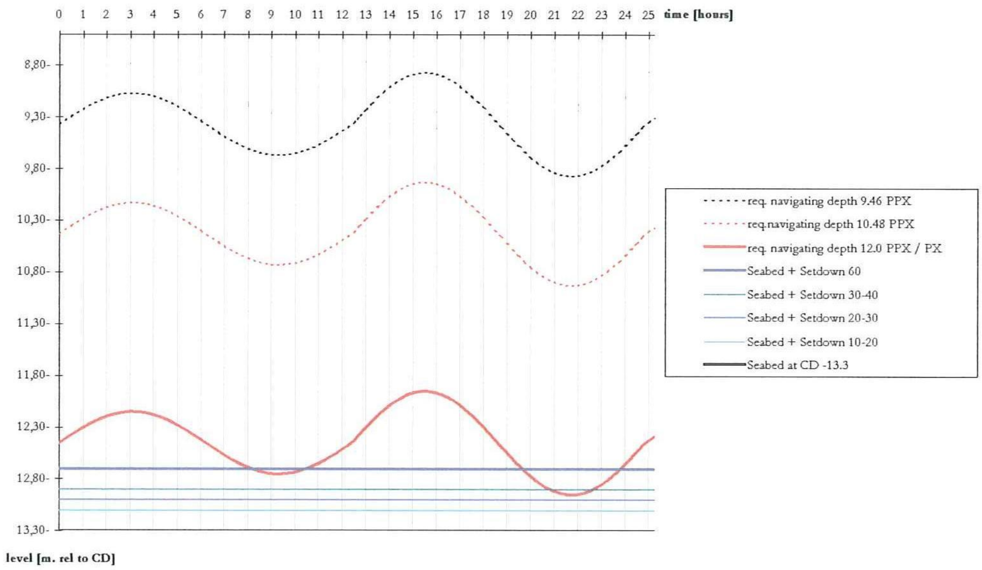


Figure 10-8 Accessibility determination at channel depth CD -13.3 metres

Set-down interval	Occurrence	Value used	# of hours, depth not available	# of hours port not accessible	% of time, port not available
< -0.40	2.05 %	-0.60	2 & 4	8	0.66%
-0.40 to -0.30	3.13 %	-0.40	1.8	2.8	0.35%
-0.30 to -0.20	16.53 %	-0.30	0	0	0
-0.20 to -0.10	10.89%	-0.20	0	0	0
> 0.10	67.40 %	-	0	0	0
Total	100.00%				1.01%
PORT ACCESSIBILITY					98.99 %

Figure 10-9 Accessibility determination at channel depth CD -13.30 m

10.1.7 Conclusion

For the chosen channel depth of CD -13.2 m the down time amounts 3.28% for only the largest types of ships; smaller ships may enter the port at all times. This channel depth guarantees a good accessibility of the port. When the channel depth is increased to CD -13.3 metres the accessibility of the port reaches almost 100% for all types of ships. Decreasing the channel depth to CD -13.1 m will cause more down time than is allowed.

A trade-off between the reduction in down time and dredging investments must be made to decide which depth to choose for the access channel to the new port. The chosen depth of CD -13.2 m is maintained for the remainder of the report.

10.2 DOWN TIME DUE TO WIND FORCE

Wind does not only cause set-up, set-down and waves, it also influences the manoeuvring of the ships, the berthed ships and crane operations directly.

- Wind velocities over 15 m/s (Bft. 7) affect the mooring and berthing of ships, depending on tug capacity [*Thoresen, 1988*]
- When the wind velocity reach values of Bft. 8 or 9 crane handling operations are stopped because of safety precautions [*various sources*]
- When extreme wind conditions (25+ m/s, Bft. 9 - 10) occur ships may have to leave the berth [*Thoresen, 1988*]

When these limits are compared to the wind climate in Buenos Aires, the additional down time for the container terminals is found. The maximum interval of the measured wind speeds is 12.5 m/s and over and this interval is taken as limiting value, appendix C.2. Wind speeds of 12.5 m/s or more have a chance of occurrence of 0.51% and the down time of the terminals becomes:

0.51 % * 24 * 365 = 44.7 hours / year.

A down time of less than 45 hours per year is considered to be acceptable for the container terminals.

10.3 DOWN TIME DUE TO WAVE HEIGHT

Wave heights can be limiting in several cases:

- when tugs have to make fast to the sailing ship
- when ships are loaded and unloaded at an exposed berth.

In this paragraph first the design waves are checked against a calculation made with wind induced wave theory. Then the limits for tug operations are checked against occurring conditions. Wave induced motions of moored ships are calculated in chapter 13.

Wave calculation

For the development of waves the following parameters are of major importance:

- fetch length
- water depth
- storm duration
- wind force

As can be seen from wind and water level measurements, presented in Appendix C, strong winds from eastern and southern direction cause set up of the water level of the Rio de la Plata. The increased water depth gives larger waves the opportunity to develop and the largest waves are found to come from these directions as well, see Appendix C.3. The limiting direction, from which waves can directly reach the port is SE, because the wind has a long fetch, causes set-up and strong winds are most frequent. The wave height is calculated with the *wind induced wave theory*, which is presented in [*Holthuijsen, 1998*]

As no clear data is found on storm duration and fetch length, here a fetch of 200 km, and infinite wind duration is assumed. The only factor limiting the wave development remains the shallow water in the Rio de la Plata[4]. Assuming a wind speed of 25 m/s

[4] When the water would have been very deep, the wave height calculated by the wave induced theory, would have been around 5.5 metres

(Beaufort 10), an average undisturbed water depth of the Rio de la Plata of 5 metres, at higher high water and a set-up of 1.26 metres the water depth becomes: 5 + 1.3 + 1.26 = 7.56 metres.

The wave height becomes:

$$\frac{g\,H_s}{v^2} = 0.283\tanh\left[0.530\left(\frac{g\cdot d}{v^2}\right)^{0.75}\right]\tanh\left[\frac{0.0125\left(\frac{g\cdot F}{v^2}\right)^{0.42}}{\tanh\left[0.530\left(\frac{g\cdot d}{v^2}\right)^{0.75}\right]}\right] =$$

$$0.283\ \tanh\left[0.530\cdot 0.1187^{0.75}\right]\tanh\left[\frac{0.0125\cdot 3140^{0.42}}{\tanh\left[0.530\cdot 0.1187^{0.75}\right]}\right] = 0.0301 \rightarrow H_s = 1.92m$$

This is very close to the maximum wave height, which has been given as design wave height with a returning period of 50 years: 1.95 m [*AGP, Informe de Avance,* 1997]. The zero crossing period of the wave is calculated to be:

$$\frac{g\cdot T_s}{v^2} = 7.5398\tanh\left[0.833\left(\frac{g\cdot d}{v^2}\right)^{0.375}\right]\tanh\left[\frac{0.077\left(\frac{g\cdot F}{v^2}\right)^{0.25}}{\tanh\left[0.833\cdot\left(\frac{g\cdot d}{v^2}\right)^{0.375}\right]}\right] =$$

$$7.5398\cdot\tanh\left[0.833\cdot 0.1187^{0.375}\right]\tanh\left[\frac{0.077\cdot 3140^{0.25}}{\tanh\left[0.833\cdot 0.1187^{0.375}\right]}\right] = 2.487 \rightarrow T_s = 6.34s$$

This is again the same value as the design wave period with a return period of 50 years. Smaller wave heights are distributed as presented in Appendix C.3.

Operational limitations

The maximum wave height at which tugs can make fast to a sailing ship is about 1.5 metres, depending on the type of tug used. From appendix C.3 can be found that a wave of 1.5 metres or higher occurs less than 0.5 percent of the time, resulting in a down time of 44 hours per year. In extreme conditions ships will have difficulty to enter the port, but generally no problems are expected for tug operations.

10.4 QUEUING THEORY CALCULATION FOR TURNING CIRCLE

In order to calculate the utilisation of the turning circle in 2020, and the waiting times for the ships, the turning circle has been modelled as one service point.

Inter arrival times of the ships have been estimated to be negative exponential distributed. The service time in the turning circle have been assumed to be Erlang-2 distributed (a conservative approach). Written in Kendall-notation: the turning circle is modelled as a *M/E2/1* system.

First the notation of both distributions is given, then the equations with which the waiting times can be calculated are stated and finally the results of the calculations haven been stated

M: The negative exponential distribution (Markov).
The negative exponential distribution, or Markov distribution, is used in the queuing theory when ships arrive at random. Ships in container ports are scheduled per terminal, but looking at all of the port, the distribution of inter arrival time is still random.
Probability density function: $f(t) = \lambda\, e^{-\lambda t}$, in which λ is the mean value of the statistical parameter.

E2 - Erlang 2 distribution.
The Erlang 2 distribution is a more general distribution than the negative exponential distribution and it is usually used for service systems. The general notation for the probability density function of this Erlang-2 distribution is:

$$f(t) = (2 \cdot \mu)^2 \cdot t \cdot e^{-2 \cdot \mu \cdot t}.$$

It only requires the mean value μ of the statistical parameter with this distribution.

M/E2/1 system
The parameters, which are calculated for the *M/Ek/1* system are:

Average number of ships in system:

$$N_a = \rho + \frac{1+k}{2k} \cdot \frac{\rho^2}{1-\rho} = \text{utilisation} + \text{queuelength}$$

Average time of ship in system:

$$T = \mu^{-1} + \frac{1+k}{2k} \cdot \frac{\rho\mu^{-1}}{1-\rho} = \text{service time} + \text{waiting time}$$

Average waiting time:

$$W = \frac{1+k}{2k} \cdot \frac{\rho\mu^{-1}}{1-\rho}$$

Queue length:

$$N_w = \frac{1+k}{2k} \cdot \frac{\rho^2}{1-\rho}$$

For the Erlang 2 system, the value of $k = 2$.

Queuing theory calculation
In the turning circle in Puerto Nuevo, which is situated in *Antepuerto Norte,* all of the ships entering and leaving Puerto Nuevo have to be stopped and turned. Considering that the capacity of the *Exolgan* container terminal in *Dock Sud* amounts 500,000 TEU, in the medium scenario 5,200,000 TEU are handled in the container terminals of Puerto Nuevo. The average amount of containers to be handled per ship amounts today 600 to 650 TEU/ship, but in 2020 this amount will range from 800 TEU/ship to 1200 TEU/ship. From these figures the average number of arriving and leaving ships is calculated, Table 10–5.

Throughput per ship	Ships/year	Ships/day
800 TEU/ship	6,500	17.8
1200 TEU/ship	4,333	11.9

Table 10–5 Traffic intensity medium scenario for different loads per ship

At the turning circle between 11.9 and 17.8 ships arrive and leave every day. This results in 23.8 to 35.6 moves per day. When a ship arrives at the port it occupies the turning circle for an average of 15 minutes, because it has to come to a complete stop, turn and towed away by tugs. Adding 5 more minutes as a following safety time results in an average service time of 20 minutes. When leaving the port it only has to sail through the turning circle and from there it leaves the port without tug assistance. The total time in which the turning circle is occupied by leaving ships $T_{leaving}$ is estimated as the number of ships * 10 minutes (see Table 10–6) For the remaining time, the turning circle is available for arriving ships: $T_{arrival}$, which leads to an average inter arrival time
λ^{-1}. Together with the average service time in the Turning Circle μ^{-1} of 20 minutes, the utilisation ρ, average waiting time W, and number of ships in the system N_w can be calculated:

Parameter	Symbol	Unit	Scenario 800 TEU / ship	Scenario 1200 TEU /ship
Number of ships / day		-	17.8	11.9
Total leaving time	$T_{departure}$	min	178	119
Total arrival time	$T_{arrival}$	min	1262	1321
Average inter arrival time	λ^{-1}	min	70.9	111.3
Average service time	μ^{-1}	min	20	20
Utilisation	$\rho = \lambda / \mu$	-	0.28	0.18
Average waiting time in percentage of service time			29%	16%
Average waiting time	W	min	5.9	3.3
Average number of ships in queue	N_W	-	0.11	0.04

Table 10–6 Results of queuing theory calculations

Sensitivity analysis:
In Table 10–7, Figure 10-10 & Figure 10-11 below is shown, what the average waiting time will be for different values of the average service time.

Average Service Time arriving ships	Waiting time 800 TEU / ship		1200 TEU /ship	
	min	% of μ	min	units of μ
10 minutes	1.2	12%	0.7	7%
15 minutes	3.0	20%	1.8	12%
20 minutes	5.9	29%	3.3	16%
25 minutes	10.2	41%	5.4	22%
30 minutes	16.5	55%	8.3	28%

Table 10–7 Average waiting times for different service times; Throughput of Puerto Nuevo 5.2 million TEU

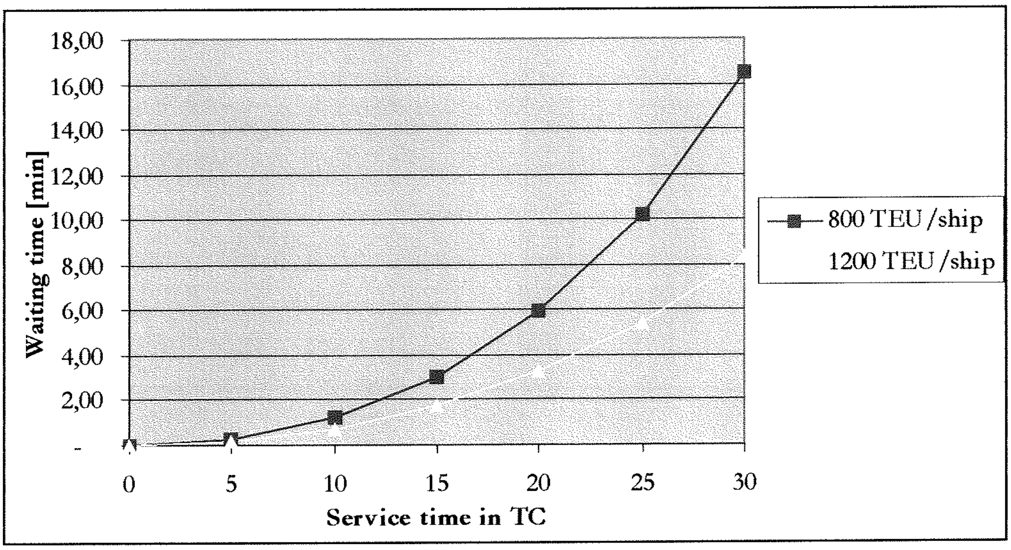


Figure 10-10 Waiting time in minutes vs. service time in TC for different ship loads

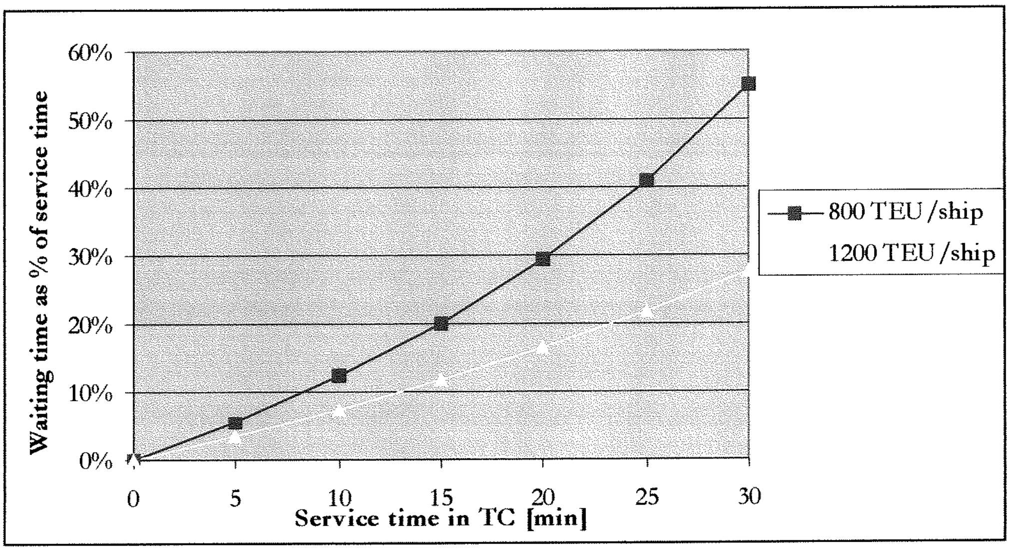


Figure 10-11 Waiting time as % of service time vs. service time

When the service time in the turning circle becomes higher than 20 minutes, the waiting times for arriving ships become substantial. Taking the average throughput of 800 TEU per ship and an average service time of 20 minutes, the waiting time amounts 6 minutes; when the average service time is 25 minutes, the waiting time becomes more than 10 minutes and more than 40 percent of the service time (Table 10–7). When higher service times occur, one turning circle will not be sufficient for the port. Turning must be done in the basins itself, or another turning circle must be constructed.

As can be seen in Figure 10-12 below, the average waiting time decreases as the average shipload increases. This is logical, because with the same throughput of the port, fewer ships call at the port. Today the average throughput per ship is 600 to 650 TEU/ship; in the future this will probably increase to about 800-1200 TEU/ship. In the figure below the waiting time is shown as a function of this average throughput per ship, assuming a service time of 20 minutes (throughput of Puerto Nuevo 5.2 million TEU).

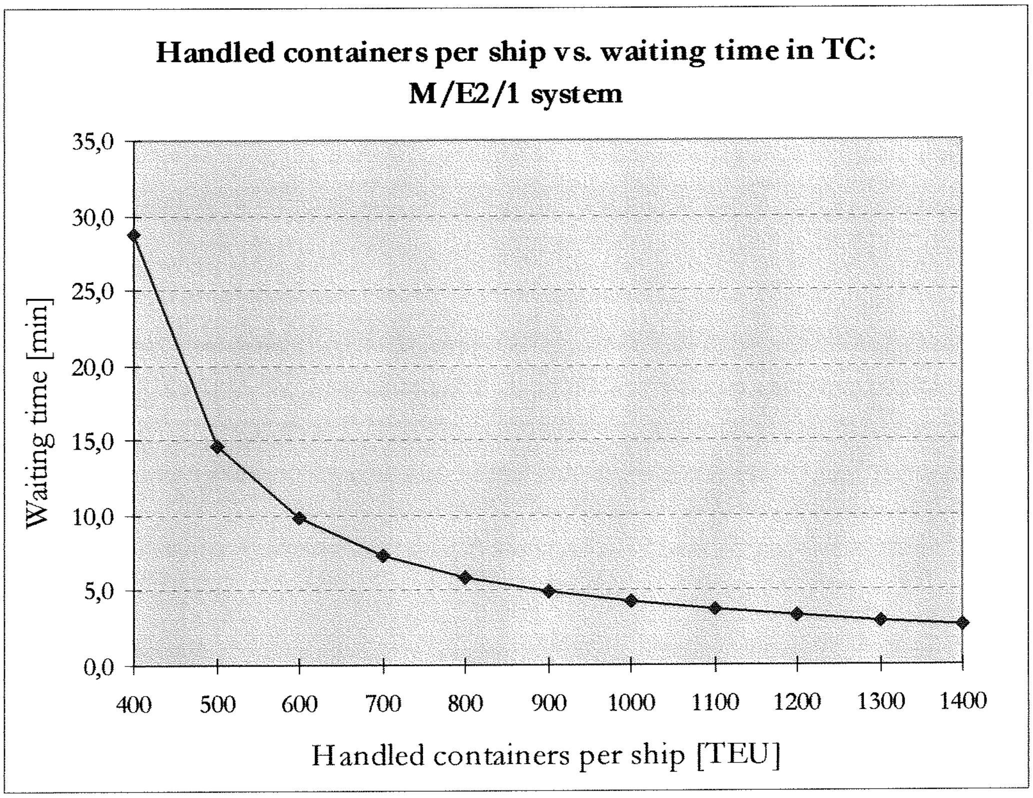


Figure 10-12 Waiting time vs. shipload, throughput of Puerto Nuevo 5.2 million TEU, Service time 20 minutes

10.5 CONCLUSION

As long as the service time in the turning circle does not exceed 20 minutes, no congestion problems are expected in the turning circle. Here the port is modelled as if all ships use just 1 turning circle. This is the case in alternative 2, but not in alternative 3, where after the final construction phase 2 turning circles are present. This leads to a more beneficial situation for alternative 3 than the one described above.

The access channel and basins for both alternatives 2 and 3 will be dredged to a level of CD –13.2 m, which secures an accessibility of the port of more than 95 percent for the largest ships.
For the other parameters in this chapter, no additional downtime is expected; the mild wind and wave climate does not cause problems for entering ships. In the next chapters the influence of the currents in the Rio de la Plata and the influence of waves on ships moored at the exposed berths are described for alternative 3.

11. FLOW PATTERN AFTER RECLAMATION

The construction of a port expansion in the Rio de la Plata will influence the existing current patterns; near the peninsula, where the flow profile is constricted, both the flood and the ebb stream will accelerate and flow velocities will increase.
Of the two mentioned alternatives in chapter 9, alternative 3 has the most severe impact on the current and this alternative will be dealt with in this chapter. When the current has passed the smallest cross-section in the northeastern corner of the peninsula, the river will widen again, but the stream will probably not be able to follow the shape of the peninsula directly. On either the north or the south side of the peninsula an area with eddies will occur for half the tidal cycle. See figures in Appendix I.

11.1 CURRENT DETERMINATION PARALLEL TO QUAY

11.1.1 Existing currents

Looking at the flow patterns the Rio de la Plata near Buenos Aires, the currents within a radius of 10 kilometres are small, on average about 0.5 knot (approximately 0.25 m/s), see Admiralty Map 3561 & figures in Appendix D. In extreme cases (Return Period 10 to 50 years) this current can be 1 knot within this radius or 2 knots further away, in the middle of the Rio de la Plata. The incoming stream runs for about 5 hours and 20 minutes, and the outgoing stream for about 7 hours and 5 minutes (source: Admiralty Pilot, NP5).

11.1.2 Currents after construction of peninsula: flow net

Before an estimate can be made of the currents after the construction of the port expansion, first has to determined over what distance a significant diversion of the current occurs. From a study on current diversion, which has been carried out for alternative 1, [*AGP, 1997*], can be concluded that the limit of influence lies around 2 times the contraction width. The current direction is generally NW-SE, and the width has been determined perpendicular to this direction. The area in which the current diversion is analysed in the next paragraphs has a minimum width before the port expansion of 4150 m. The maximum constriction after the reclamation is 1450 m, thus about 2/3rd of the flow profile remains, 2700 m, see Figure 11-1

The impact in the direction of the current is more significant, but depends greatly on the shape of the constructed peninsula. When sudden curves exist in the peninsula or in the coastline, the stream is not able to follow this bend and an area with eddies will be formed.

Coming back to alternative 3: the *flood stream* contracts at the southern side, where the near-shore streamlines make a sudden bend and eventually flow parallel to the quays on eastern side of the peninsula. The current accelerates along this quay and this increasing flow will affect sailing and mooring ships in the navigation channel and at berth. Then the current will pass the northeastern corner at its maximum speed and is probably not able to follow the sudden curve of the peninsula. On the western side of the flow, near shore an area with eddies is formed at the northwestern side of the island. This eddies are of no importance to navigating or moored ships, but can cause important morphological changes in the area.

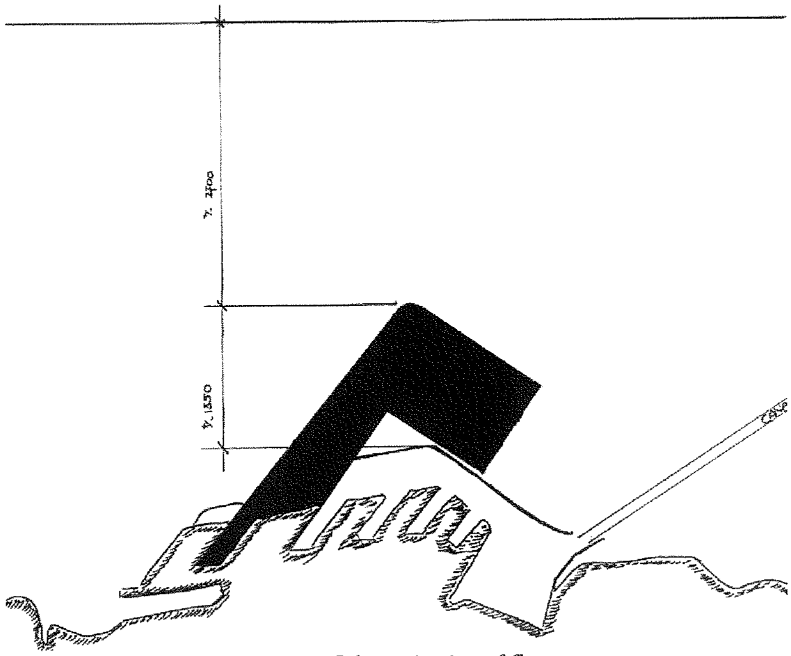


Figure 11-1 Schematisation of flow area

The *ebb stream* will accelerate at the northwestern side of the island and pass the northeastern corner of the island at its maximum speed as well. Still, the flow will not be able to follow the sudden bend of the shoreline and an area with eddies will be formed, this time on the eastern side of the island. Moored ships are not effected by strong currents, but navigating ships will be sailing in slowly flowing water and a sudden transition from moving to non-moving water may cause navigation difficulties. Therefore, it is advised to study the manoeuvring of ships in more detail in a later phase, when more precise flow patterns have been determined.

For the port expansion the contraction of the current near the berths and the increased velocities in the northeastern corner are considered to be the most important one. Therefore, only the velocities of the flood stream are determined in this report. Furthermore, the flow velocity near the most northern quay is assumed to be equal to the flow velocity in the smallest cross-section (where the contraction is the largest).

Flow net

To estimate the currents after the construction of the peninsula, a flow net has been drawn as a first approach. It must be noted that flow nets are only valid for small scale contractions (the fluid is assumed to be frictionless) and that the values obtained with the flow nets can be only seen as an estimate for this large scale reclamation.

In the flow net (see Appendix I) approximate streamlines after the construction of the peninsula have been drawn. When perpendicular to the streamlines potential lines are drawn in the right way, small squares are formed[5]. Between two streamlines, the flow Q is assumed to be constant; there is no flow perpendicular to the flow direction. The value of Q is determined at the outer (upstream) edge of the analysed area, where the streamlines are running parallel and the current is assumed to be constant. Then from the flow net, an estimate can been made for the currents in the smallest cross-section.

Upstream, the current flowing through each square is:

[5] In the flow nets of appendix I, the squares have been transformed to rectangles, this is one of the effects of using flow nets for this large scale contraction

$$Q = B_{square} * d * u_0 = 400\ m * 3\ m * 0.25\ m/s = 300\ m^3\ /\ square$$

The squares that have a width of 400 metres upstream have contracted to rectangles with a width of approximately 50 m in the smallest cross section near the northeastern corner of the peninsula. Compared to the current upstream, now 8 times the amount of water flows through a cross-section of the same width of 400 metres. The flow velocity will not be constant over this 400 m, but over the newly dredged channel (bottom width 200 m, total width 250 m), the flow is assumed to be constant. Assuming that the depth near the corner of the peninsula is the same as in the newly dredged channel (deepest level: 13.2 m- CD, average depth: 12.11 m) (see Figure 11-3), this leads to an average flow velocity of 0.50 m/s, 2.0 times u_0.

Note: The calculation presented above is based on measurements on the maps in Appendix I. Small differences in the position of the streamlines can result in much lower or higher velocities.
For example: when only 4.5 times the amount of water flows through the channel, the velocity will be equal to 0.45 m/s or 1.78 u_0. The accuracy of the values is only limited and should be considered as a first approach.

11.1.3 Currents after construction of peninsula: calculations

The maximum and minimum flow velocity can be estimated with simple schematisations as well. As can be seen from the flow nets in Appendix I, the flood stream causes higher velocities near the quay wall, and therefore this current (average upstream flow velocity u_0 = 0.5 knot = 0.25 m/s) has been chosen for all calculations. Two different schematisations are described below:

- minimum flow velocity: all current evenly spread
- maximum flow velocity: all current through new channel

Minimum velocity: All current evenly spread, uniform flow.

The minimum velocity at the most contracted part of the peninsula is estimated as follows; all of the current, which used to flow through the non-contracted cross-section of 4050 m (Figure 11-1), will be evenly spread in the contracted cross-section (2700 m). The presence of the newly dredged channel, which gives access to the berths on the eastern side of the peninsula, is neglected here (see Figure 11-2).

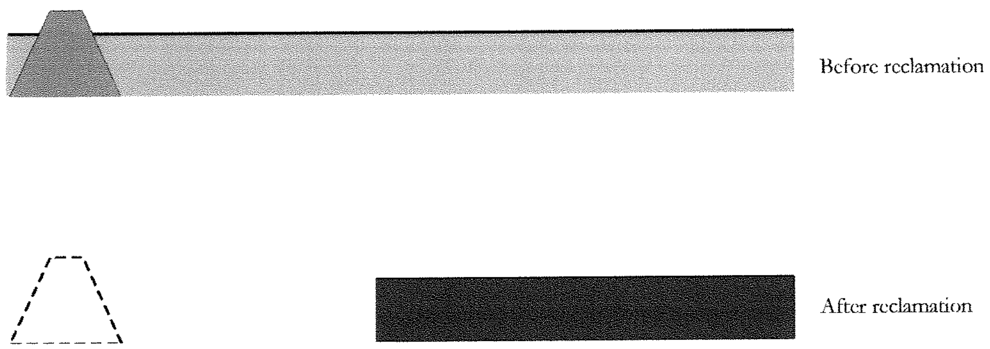


Figure 11-2 Minimum velocity : all current evely spread

Directly after construction, the velocity flow velocity becomes:

$$u_{new} = \frac{Q_0}{A_{new}} = \frac{4050\ m * 0.25\ m/s * 3.5\ m}{2700\ m * 3.5\ m} = 0.375\ m/s = 1.5 * u_0$$

With this schematisation, the minimum velocity near the most northern berth is calculated to be 1.5 * u_0 for any velocity. When an extreme flow velocity of 1.0 knot occurs, the velocity in the contraction and near the most northern berth will be 1.5 knot.

Maximum velocity: All 'extra' current through channel
The maximum velocity near the most northern berth is calculated when all of the diverted current is assumed to flow through the newly dredged access channel (bottom width 200 m, slope 1:6, see Figure 11-3).

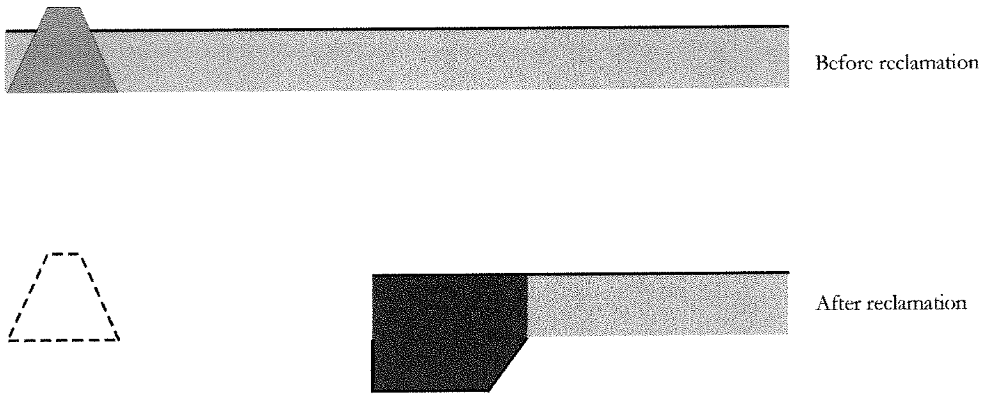


Figure 11-3 Maximum velocity: all extra current through channel

This channel is assumed to be present in the most northern part of the contraction as well. The maximum velocity becomes:

$$u_{\max, est.} = \frac{(1350+258)*3.5*0.25}{13.2*200+58.2*(13.2+3.5)/2} = \frac{1407}{3126} = 0.450 \, {}^{m}\!/_{s} = 1.80 \, u_0$$

Note: In both calculations above the influence of the existing contraction, which is caused by the shape of the coastline and the breakwater, is neglected. This influence has not been neglected in the flow net.

Range of velocities, limiting value
Above, the flow velocity near the northeastern corner has been estimated in three different ways. All of the calculations have a limited accuracy, but are useful as a first estimate. The calculations result in a range of flow velocities between 1.5 and 2.0 times u_0. A better estimate of the flow velocity can be made by measuring the velocity in a scale-model, or by carrying out a more detailed calculation with a numerical morphological and flow model like *DELFT 2D*. For this first design stage, the maximum calculated velocity of 2.0 u_0 is used as an upper limit.

In various sources limiting values for flow velocities parallel to the quay can be found [*PIANC, 1995*] & [*Thoresen, 1988*]: 3 knots (1.5 m/s). Comparing this limit to the occurring extreme velocity near the most northern berth of 2.0 u_{max} = 2.0 * 0.5 m/s = 1.0 m/s, no extra downtime is expected.
Although this velocity will not influence the mooring and berthing of the ships, it may cause erosion in the northeastern corner of the reclamation. A bottom protection will prevent uncontrolled erosion.

11.1.4 Scour protection

As mentioned above, in the accelerated flow parallel to the quay extreme flow velocities of 1.0 m/s can occur. This is not the highest velocity, which occurs near the quay, as propeller jet velocities may reach values of up to 2.5 m/s near the sea bed (ref CUR/CIRIA 169). The shore protection will be designed using this velocity.

The required diameter of the stones in the scour protection has been calculated according to the method presented in paragraph 5.2.4.1 of ref (CUR/CIRIA), using equations, developed by Shields and Isbash. First a first estimate of the stone size is made with the formulas of Isbash. Additionally a correction factor is applied for turbulence (propeller jet). After that, a more precise calculation is made with a modified Shields equation, developed by Pilarczyk (1990) to take the water depth into account.

Isbash

Approximating the stone size using Isbash, yields:

$$\Delta D_{n50} = k_t^{-2}\, 0.71 \frac{u^2}{2g} \rightarrow D_{n50} = \frac{k_t^{-2} \cdot 0.7\, u^2}{\Delta \cdot 2g} = \frac{2 \cdot 0.7 \cdot 2.5^2}{1.65 \cdot 19.62} = 0.28\, m$$

in which

k_t^{-2}:	turbulence factor:	2.0
u:	jet velocity:	2.5 m/s

Shields

Using ψ=0.03 (almost no transport of stones) and taking the above calculated 0.28 m as an estimate for D_{50}, results in a value of Λ_h=77. The final stone size is then calculated with:

$$\frac{u^2/2g}{\Delta D_{n50}} = \frac{\psi'_{cr}}{0.035} k_{sl}\, k_t^{-2}\, \Lambda'_h \frac{1}{\phi_{sc}} \rightarrow D_{n50} = \frac{u^2/2g}{\Delta} \frac{1}{k_t^{-2}\, k_{sl}\, \Lambda'_h} \frac{0.035}{\psi'_{cr}}$$

in which,

ϕ_{cr}	= 1, describes the stability of protection layer
k_{sl}	= 1, because no slope is present in the scour protection
k_t^{-2}	= 1/3, turbulence factor.
ψ'_{cr}	= 0.035, this is Shields' stability factor
Λ'_h	≈ Λ_h/33, special depth factor

Using above parameters, the stone size D_{n50} is determined to be 0.25 m, which is about equal to the value calculated with Isbash.

11.2 CONCLUSION AND RECOMMENDATIONS

Conclusion

In this chapter the flow velocity near the (most northern) quay has been estimated. The flow accelerates in the contraction to about 2 times the upstream velocity; the maximum value, which has been calculated is 1.0 m/s (or 2 knots). This is less than the maximum allowable velocity 1.5 m/s and no extra downtime of the exposed berths is expected due to currents parallel to the quay.

The scour protection is designed on an estimated propeller jet velocity near the bottom of 2.5 m/s, the nominal stone diameter is: 0.25 m

Recommendations

1. In a later stage a more detailed analysis of the currents and flow velocities after construction of the peninsula should be made. Not only tidal currents should be taken into account, but wave induced currents as well. A suitable method can be a numerical computation with a program like Delft 3D (Delft Hydraulics) or STP (Danish Hydraulic Institute)
2. The effect of waves on the seabed near the quay wall should be investigated. If necessary, the scour protection must be adapted to the conclusions of the investigation.

12.WAVE INDUCED MOTIONS OF MOORED SHIP

In a protected port ship motions at the quay are generally not a problem, if the waves do not penetrate into the port and the port basin does not have regular oscillations. This is different for an unprotected quay where ships are directly exposed to waves and currents. The waves will cause additional motions of the ship, influencing mooring and loading operations. In this chapter a first estimate is made of these wave induced motions for ship moored at the eastern side of the peninsula of alternative 3.

12.1 INTRODUCTION

A moored vessel has six modes of freedom; three rotary and three lateral modes, see Figure 12-1. Although the ship is kept near the berth moored by means of mooring lines and fenders, the motions of the ship are only slightly hindered by these "elastic" lines. The behaviour of a vessel in waves is highly non-linear and cannot be calculated in an easy way. The motions are usually numerically calculated in either the frequency domain or in the time domain. Time domain functions are much more complex and produce more detailed results, frequency domain computations give more general results in the form of a motion spectrum for each mode of freedom, which can be derived from the wave spectrum by multiplying the wave spectrum with some response function for that motion. This function is called the ROA, Response Of Amplitude, function.
Here, the motions have been calculated in the frequency domain as they provide sufficient accuracy for the preliminary design.

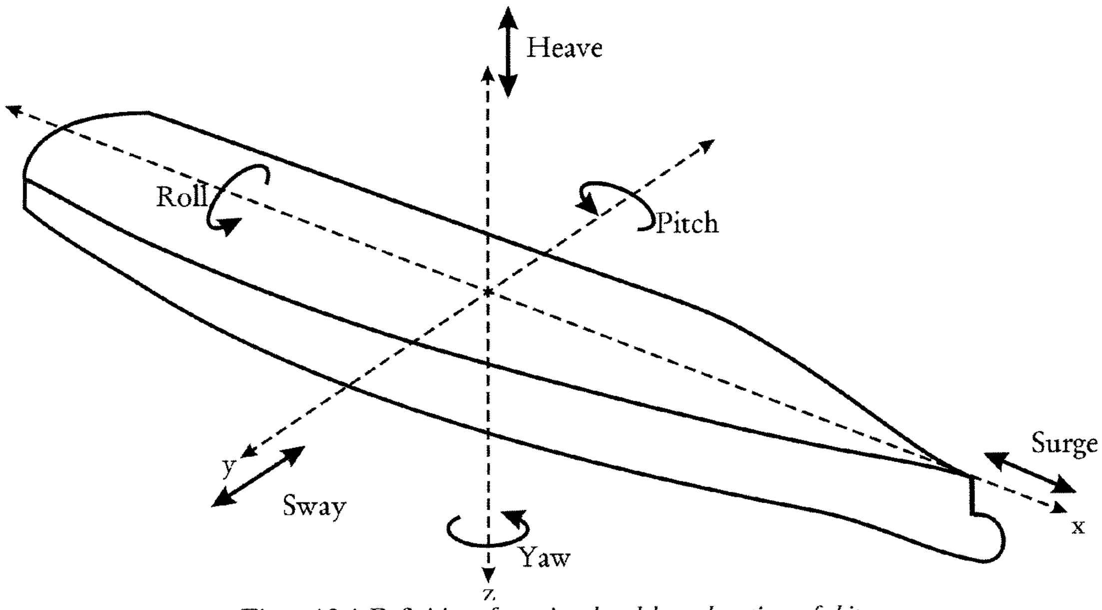


Figure 12-1 Definition of rotational and lateral motions of ship

In paragraph 12.2 first the wave climate in Buenos Aires has been transformed into several representative wave spectra, then the ROA-functions of the container ship have been calculated in paragraph 12.3 and finally an estimate has been made of the wave induced motions of a Panamax containership at the exposed quay.

12.1.1 General notes

Wave period

From various sources can be found that in general ships do not move with the same frequency as the waves they are exposed to. Ships usually move in a sub-harmonic of the wave frequency, near to the natural period of the ship. Especially low frequent waves, like swell, determine the motion of ships at the berth. When only the wave frequency is varied not all motions react to the same extent. At lower frequencies the surge motion increases a lot, while the other motions do increase, but not as much as expected. The roll rotation has its peak at higher frequencies (shorter waves) and is not that sensitive to long waves.

Wave direction

The motion of a ship is determined not only by the wave period, but by the wave direction as well. At each wave direction, the ship has a different ROA-function. To illustrate this: the roll motion of a ship will be more severe when exposed to beam waves. On the other hand, the surge motion will decrease at the same time.

Type of berthing structure

The motion of a ship at a berth depends on the type of quay structure as well. When the ship is moored at a vertical quay, waves will be completely reflected, which causes standing waves. This causes large lateral movements, but small rotational movements (and thus mooring line forces) compared to a quay structure which has a slope. At this type of structure, usually a slope above which an apron on piles has been constructed, waves will not reflect. No standing waves will occur and therefore rotational motions will increase. On the other hand, lateral motions (surge, sway and heave) and the mooring line forces will be smaller when using a slope structure.

Type of fenders & mooring lines

When fenders with a higher stiffness are used, the movements will be only slightly smaller, but the induced forces on ship and quay are much larger.
Usage of mooring lines with a high stiffness (steel lines) will cause smaller lateral movements, but much higher line forces. Rotational movements are not much affected by the choice of mooring lines.

12.2 WAVE SPECTRA IN BUENOS AIRES

The waves occurring near Buenos Aires are short waves, generated by wind. These wind-induced waves have short zero crossing periods (high frequencies) and are not very high. For example, the significant wave height with a return period of 50 years is H_s=1.95 m. From the statistical distribution of appendix C.3, for several waves, the return period has been calculated. For this purpose an average storm duration of 6 hours has been assumed.

The Return period (RP) is then calculated as follows:

$$RP = \frac{1}{events/yr} = \frac{1}{possible\ events \cdot chance\ of\ occurrence} = \frac{1}{365 \cdot 24/6 \cdot P_{wave}}$$

This leads to the results in Table 12–1 below

Wave height [m]	Peak Zero crossing period [s]	Frequency [rad/s]	Chance of occurrence	RP
H_s = 1.0	T_P = 5.0	1.26	5.46 %	4.6 days
H_s = 1.2	T_P = 5.0	1.26	1.06 %	0.06 yr
H_s = 1.4	T_P = 5.1	1.23	0.16 %	0.43 yr
H_s = 1.6	T_P = 5.7	1.10	0.01 %	6.9 yr
H_s = 1.8	T_P = 6.1	1.03	< 0.01 %	10 yr
H_s = 1.95	T_P = 6.3	0.997	< 0.01 %	50 yr

Table 12–1 Return Period for different design waves

The development of waves in the Rio de la Plata is limited by fetch length and water depth, and the spectrum of the waves will never be fully developed. Therefore a spectrum shape as determined in the Joint North Sea Wave Project (JONSWAP) project has been assumed. The following definition of a Mean JONSWAP wave spectrum has been used for limited fetch situations:

$$S_\varsigma(\omega) = \frac{320 \cdot H_s^2}{T_p^4} \cdot \omega^{-5} \cdot \exp\left\{\frac{-1950}{T_p^4} \cdot \omega^{-4}\right\} \cdot \gamma^A$$

with : $\gamma = 3.3$ (peakedness factor)

$$A = \exp\left\{-\left(\frac{\frac{\omega}{\omega_p} - 1}{\sigma\sqrt{2}}\right)^2\right\}$$

$$\omega_p = \frac{2\pi}{T_p}$$

σ = a step function of ω : if $\omega < \omega_p$ then $\sigma = 0.07$

if $\omega < \omega_p$ then $\sigma = 0.09$

This leads to a spectrum with a shape as shown in Figure 12-2

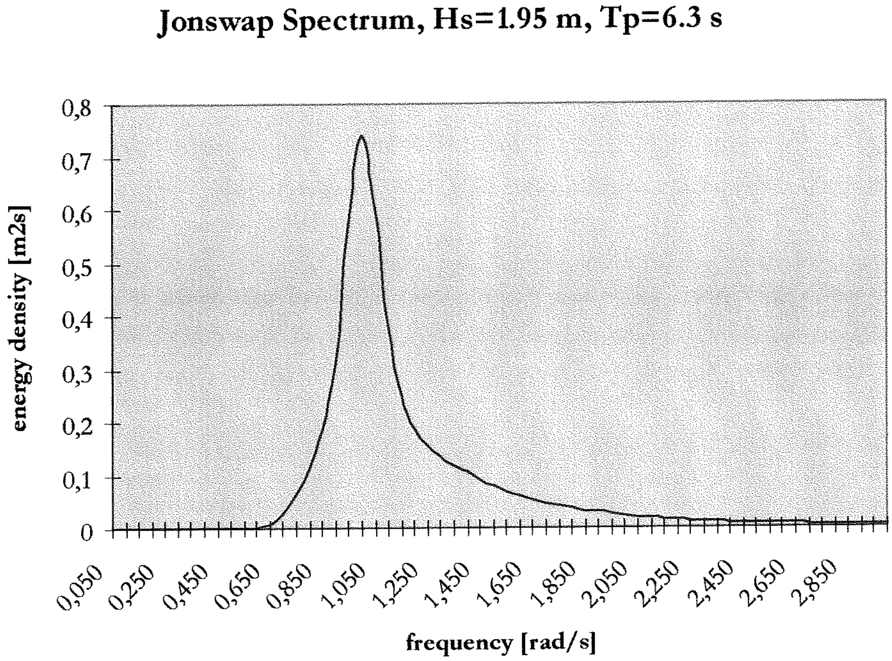


Figure 12-2 JONSWAP Spectrum for Hs=1.95 m, Tp=6.3 s (design wave of Buenos Aires)

The significant wave height can be calculated with the area under the spectrum (m_0) using the following mathematical relation:

$$H_s = 4\sqrt{m_0}$$

Charts of the different wave spectra can be found in appendix J.

12.3 Determination of Response of Amplitude (ROA) functions for Panamax containership at quay

To calculate the response of a ship to the loads, which are induced by waves, a response function must be known. The ROA function gives the relation between the wave spectrum at the berth and spectrum of the ships motions (see Figure 10-3)

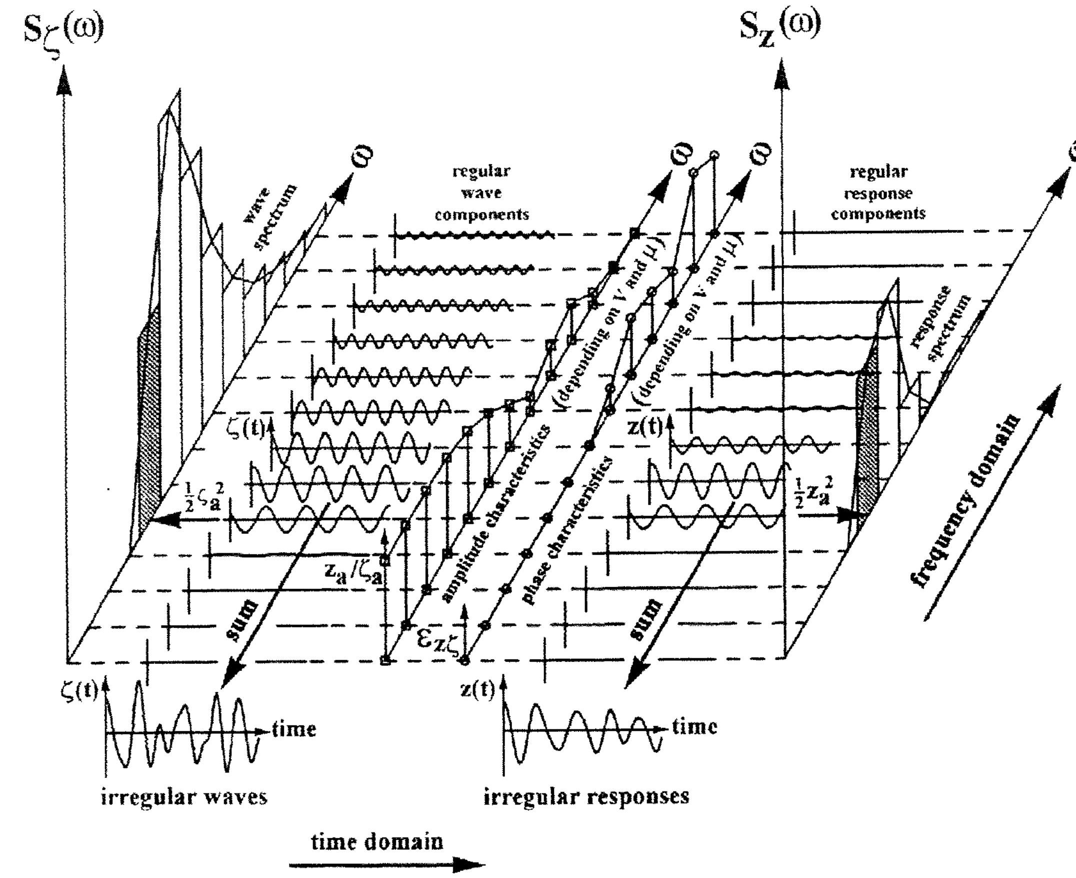


Figure 12-3 Principle of transfer of waves into responses:
Source: Journée, J.M.J., The behaviour of ships in a seaway

The calculation of this ROA-function is a non-linear problem, because of the damping characteristics of the vessel and non-linear characteristics of fenders and mooring lines. Simple linear rules of thumb do not exist yet and ROA functions are calculated by numerical models, like the one described below.

DELFRAC

The numerical calculations of the ROA functions have been made with the numerical model DELFRAC, developed by the faculty of Maritime Technology of Delft University of Technology. It uses the three-dimensional diffraction theory to compute the wave loads and motions at zero forward speed. Because the method is based on the linear potential theory, the response of the ship is assumed to be linear proportional to the wave height. Besides wave loads and motions, the resulting disturbance created by the presence of a floating body on the surrounding wave field can be computed.

It should be noted that the response is calculated in the frequency domain and not in the time domain, so historical movements of the ship are not taken into account. For preliminary calculations, the program has proven to give good results.

The quay on the eastern side of the peninsula has been modelled as a quay of two times the ship's length (600 m) with a constant depth of CD -13.5 m. It does not make a large difference that the channel stops after approximately 200 metres, because in 200 m the waves can adjust to the increased water depth. No longer quay has been used, because this does not give more precise results, but when a shorter quay had been used, the influence of reflected waves from some directions would have been neglected.

The Panamax container ship is situated 2 metres from the quay wall, which is about equal to the average size of a fender for this type of ship. Mooring lines and fender characteristics have not been taken into account, because these make little difference to the calculated motions.

After modelling, the response per metre of wave height for the Panamax container ship has been calculated for waves from the following directions: 0, 45, 90, 135 and 180 degrees off stern, see

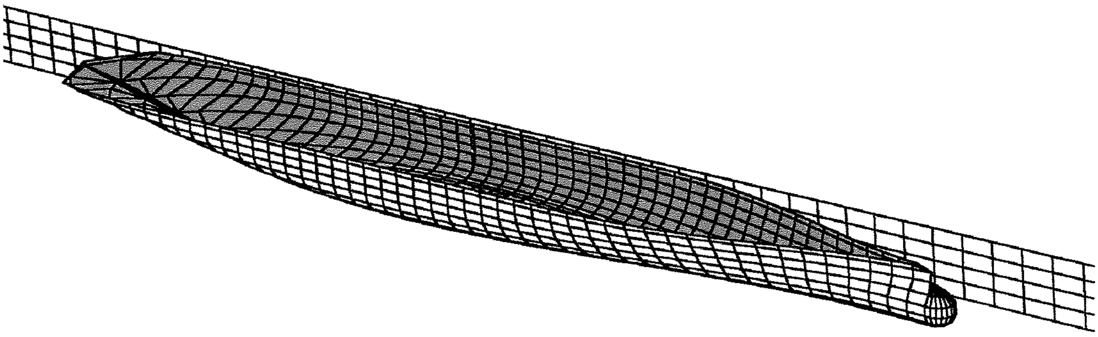


Figure 12-4 Modelled container ship at exposed quay in DELFRAC

From the graphs in Appendix J can be concluded that:

- The surge motion of the container ship depends on low frequent waves from any direction.
- Heave, sway, roll, pitch and yaw motions are affected not only by low frequent waves, but by high frequent waves as well.
- Response peaks can be found at 0.45 rad/s (14 s), 0.85 rad/s (7.4 s) and 1.05 rad/s (6 s). All motions have a more severe response to low frequencies than to high frequencies.
- The limiting wave direction of the container ship is 135 degrees off stern for all motions except surge. The response of the ship is larger than when waves come from 45 degrees, because the exposed area from 135 degrees is larger than from 45 degrees.
- Beam waves (90 degrees) cause significant roll and sway motions, the surge motion is the smallest from this direction

12.4 Ship motions at quay in various wave spectra

Now that the ROA-functions have been determined for wave from different directions, the motion spectra of the ship at the quay can be calculated.
The transfer of the wave spectrum to the motion spectrum is done with the following formula:

$$S_z(\omega) = \left|\frac{z_a}{\varsigma_a}(\omega)\right|^2 \cdot S_\varsigma(\omega) = ROA^2(\omega) \cdot S_\varsigma(\omega)$$ (Heave spectrum is determined)

Or formulated in words: *the spectrum component of the response motion at frequency* ω *is equal to the wave spectrum component at the same frequency multiplied by the square of the response function of that motion at that frequency.*

The motion amplitudes can be calculated from the motion spectrum just like the wave amplitudes above. Again this is illustrated for the heave motion:

$$\bar{z}_{a,s} = 2 \cdot RMS = 2\sqrt{m_0}$$

Note: The significant motion (total motion) is twice this value.

For the six analysed wave spectra, the significant ship motions have been calculated. In appendix J these motions have been stated and they have been compared with general accepted operational limits and safety limits, as presented in table 5-9. In Table 12–2 is shown at what wave height those limits are first reached.

	Operational Limit Reached	Safety Limit Reached
Wave height	1.40 m	1.60 m
Wave direction	90°	135°
Limiting motion	sway (0.47 m)	roll (3.3°)
Chance of occurrence	< 0.16 %	< 0.01%

Table 12–2 Limiting natural conditions for berthed Panamax containerships

From the values stated above can be concluded that Panamax containerships can safely moor at the exposed quay on the eastern side of the peninsula. When waves occur with a significant wave height H_s of 1.60 metres or more, the safety of the ships at the quay cannot be guaranteed and ships have to leave the berth. The chance of occurrence is 0.01%

To validate the assumption that at lower frequencies (longer waves, swell) operational and safety limits are reached at much smaller wave heights, for a small swell spectrum (Bretschneider shape) the motions have been computed as well. This shows that when a swell with a wave height of 0.40 metres occurs, ships cannot moor safely at the exposed side of the peninsula. Exposure to swell is not expected in Buenos Aires however, because the port lies 200 km from the ocean and swell waves have damped out before reaching the port.

12.5 CONCLUSIONS AND RECOMMENDATIONS

Conclusion

This chapter shows that in Buenos Aires Panamax containerships can berth safely at the quay on the eastern side of the island. Ships on this quay are exposed to high frequent wind induced waves and for more than 99% of the time motion safety limits are not reached. When ships are exposed to (small) swell, safety limits are reached at much smaller have heights and berthing at an exposed quay is not advisable

Recommendations

1. It is recommended to do an extensive mooring analysis in a later stage, in which drift forces and line pull forces are taken into account as well and the motions of the ship are calculated in the time-domain.
2. From the extended mooring analysis it is recommended to define a safety criterion at which ships need to leave berth or cannot enter the port. This criterion should be used in future practice and needs to be sufficiently accurate, something like an early warning system for high wind speeds from WSW to S direction (weather forecast) in combination with wave measurements.
3. There is little information on the occurrence of low frequent waves in the Rio de la Plata. As these low frequent waves are governing for ship motions at the quay, even though very small, a better investigation on the possibility of exposure to the waves is advisable.

BUENOS AIRES PORT EXPANSION 2020

PART 4:

DESIGN AND CALCULATION OF QUAY WALL

13.DESIGN PARAMETERS QUAY WALL

The expansion of the port of Buenos Aires consists of 3575 metres of quay structure with a water depth of CD -13.5 metres and a quay surface situated 4.75 metres above Chart Datum.
In the following paragraphs a conceptual design of the quay structure will be presented; a design, which is based on the principle that a quay wall should be easy to construct and fit for purpose.

13.1 DESCRIPTION OF QUAY STRUCTURE

Combiwall
The quay wall consists of a combiwall structure, tubular piles and infill sheeting, anchored by means of tie-rods and anchor wall.
The steel tubular piles act as the main soil retaining structure and carry small vertical loads and larger horizontal loads (fender, bollard and crane loads) from the quay into the subsoil. The infill sheeting, consisting of double sheet piles type Larssen, carries the earth pressure in horizontal direction to the tubular piles.
The anchors consist of steel anchor rods, which are anchored on one side to the tubular piles and on the other side to an anchor wall of steel sheet piling.

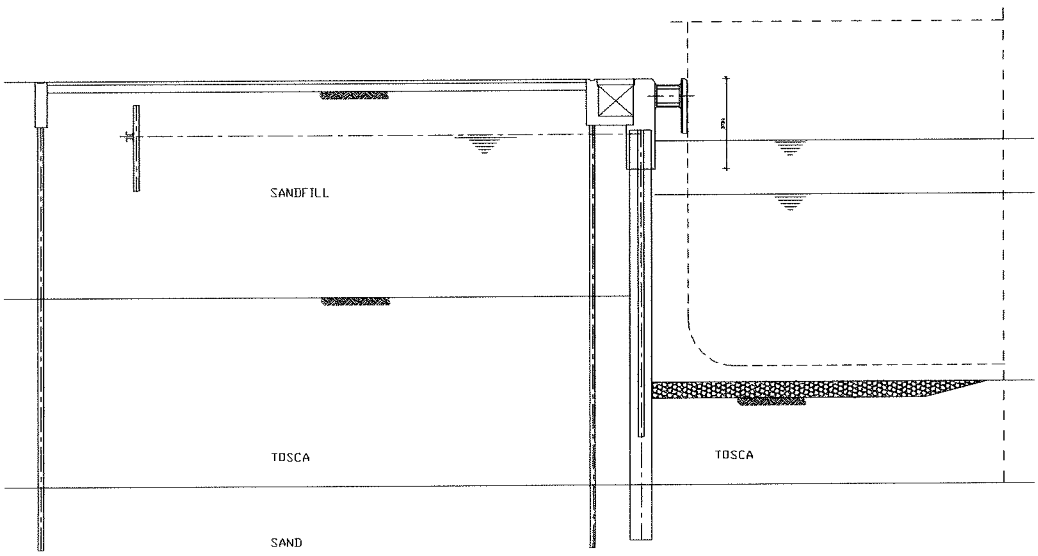


Figure 13-1 Design of quay wall

Note: The quay structure can be designed with a relieving platform as well. This reduces the horizontal pressures on the combiwall and results in a smaller profile for the combiwall, but the design of the structure becomes more complex on the other hand. The objective of the conceptual design is to produce an estimate of the costs of the quay per running meter with a margin of +/- 30 %. Use of a relieving platform will be an interesting optimisation in the detailed design stage.

Sandfill
Because no large settlements are allowed behind the quay wall (over a distance of approximately 50 m) the top layer of alluvial deposits and soft tosca will be dredged away until sufficient bearing capacity is found in the tosca layer (around CD -9 m). On top of this tosca layer backfill will be placed, which consists of dredged sand from near-shore sandpits. This backfill will be compacted, in order to reduce settlements of this layer to a minimum.

Crane beam
The front crane beam will be founded on piles because no settlements are allowed in this beam. The rear crane beam is founded on piles as well.

Paving
The paving consists of paving blocks which are positioned on top of a stabilized sand layer of about 500 m thickness. Residual settlements of the tosca layer and sandfill, which amount about 0.02 m after commissioning, have to be adjusted in the pavement.

13.2 STANDARDS

The quay wall structure has been designed using the following standards:

EAU 1990:	Recommendations of the Committee for Waterfront Structures, Harbours and Waterways
NEN 6740:	Geotechnical guidelines

When no clear statement is found in EAU 1990, BS 6349 is used as alternative.

13.3 LIFESPAN

The lifespan of the quay structure has been taken as 60 years, in accordance with BS 6349, Part 1, par.16

13.4 LOADS

The soil parameters used for the design have been described in chapter 15.
The structure is designed for loading class 2, EAU 1990.

13.4.1 Vertical live loads

Live loads according to EAU 1990, par. 5.5

Containers: full, stacked 2 high:	35 kN/m^2
Containers: full, stacked 4 high:	55 kN/m^2
General traffic:	10 kN/m^2

Near the quay wall, container stacking up to 2 high is allowed (35 kN/m^2). At the waterfront (5 metres) only traffic is allowed (10 kN/m^2).

13.4.2 Hydrostatic pressure loads

Hydrostatic pressure loads are caused by water level differences. On the land side of the quay wall the ground water level has been taken according to EAU 1990, in tidal areas: MSL + 0.3 m → GWL: CD + 1.1 m
At the Rio de la Plata side, the following design water levels have been taken:
An average value of low water level: CD - 0.50 m (MLLW - set-down of 80 cm)
An extreme low water level: CD - 3.40 m (level with return period of 100 years)

13.4.3 Bollard loads

The working load of the bollard depends on the displacement of the ship and the mooring conditions at the quay. In order to accommodate Post Panamax container ships the bollard should be dimensioned with this displacement G:
$G = LOA \times B * T * C_b * \rho_w = 282 * 40 * 13.3 * 0.65 * 10 =$ approx. 1,000,000 kN.
According to EAU 1990, Table R 12-1, the calculation value of the Line Pull Force for these ships is 1000 kN. Since ships can be influenced by currents and waves at the exposed berth this value has to be increased by 25 %, resulting in a line pull force of 1250 kN. Taking into account that slightly larger ships might enter the port, the design force for the bollard load has been chosen as 1500 kN, as is customary for new large container berths.
Computation of the coping beam, on which the bollard is mounted, on an elastic foundation shows that the 1500 kN is will be spread and results in a load of 110 kN/m1, see 14.1.2. The spacing of the bollards is equal to the normal length of a coping beam section; about 30 m.

13.4.4 Crane loads

Just behind (4 metres) the quay wall a crane beam is constructed on which the Post-Panamax container crane will operate. This crane has a maximum vertical corner load of 6000 kN, see. This load is spread over 8 wheels, resulting in a vertical wheel load of 750 kN. A horizontal force of 60 kN/m is transferred to the combiwall.

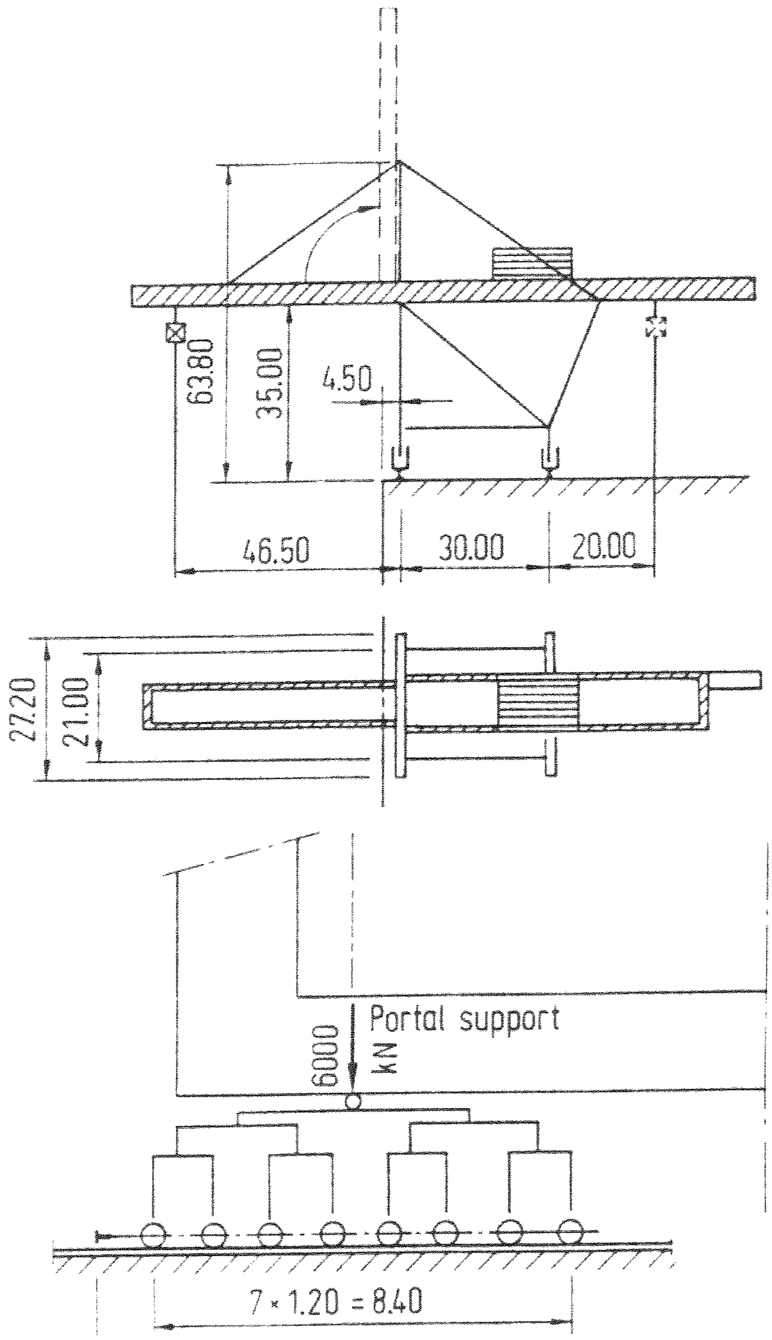


Figure 13-2Crane load according to EAU 1990

13.4.5 Fender loads

Fender loads are determined by the type of fender used to absorb the kinetic energy from the berthing ship. This kinetic energy depends on the displacement of the ship and its berthing velocity.

The transverse berthing velocity of the largest ships has been taken as 0.15 m/s, in accordance with EAU table R 40-1, moderate wind and heavy sea.

A typical value of the fender force is around 1500 kN per fender, but this depends largely on the type of fender used. Whichever fender is used, the load of the fender on the quay structure is not governing and no further computations with this force have been made.

The spacing of the fenders is maximum 0.15 * L_s = 0.15 * 100 = 15 m, L_s is the length of a regular feeder ship in the port of Buenos Aires.

13.4.6 Material, load and safety factors

The load and material factors used for the design are based on EAU 1990.
For earth parameters calculation values (reduced characteristic values) are used. Therefore representative calculations still hold a certain degree of safety. When using the calculation methods as mentioned in EAU 1990, the following factors are taken into account.

Sheet pile calculation
Earth pressures: $F_L = 1.0$
Water level difference: $F_L = 1.0$
Safety against failure of construction: $F = 1.5$, $\gamma_m = 1.5$
Stability in upper failure plane: $F = 1.5$
Stability in lower failure plane $F = 1.5$

Pile foundation
The safety factor for the pile foundation of the crane beam has been chosen as:
Crane load: $F = 2.5$
Negative friction $F = 1.0$
Positive friction $F = 1.0$

13.4.7 Load combinations

For the design of the quay wall two load combinations are distinguished:

Load combination 1: Groundwater level inside: CD +1.1 m, water level outside: CD -0.5 m (MLLW-set-down), surcharge + bollard & crane force. This load combination can be seen as a serviceability limit state and can occur several times a year

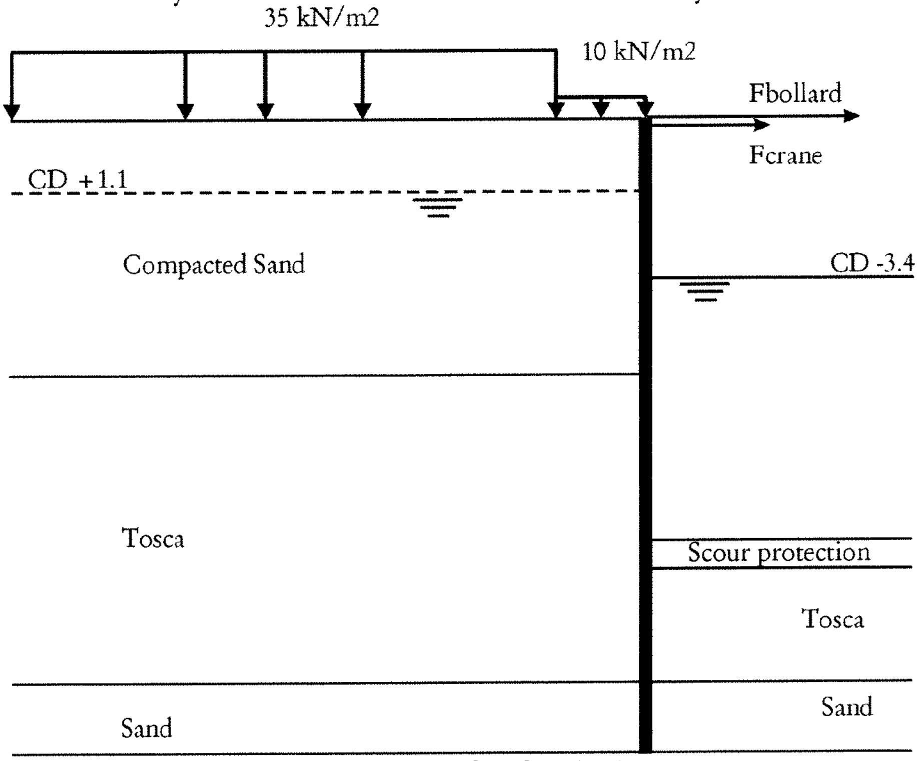


Figure 13-3 Load combination 1

Load combination 2: Groundwater level inside: CD +1.1 m, extreme water level outside: CD -3.4 m, surcharge on quay, bollard & crane force. This load combination can be seen as an ultimate limit state.

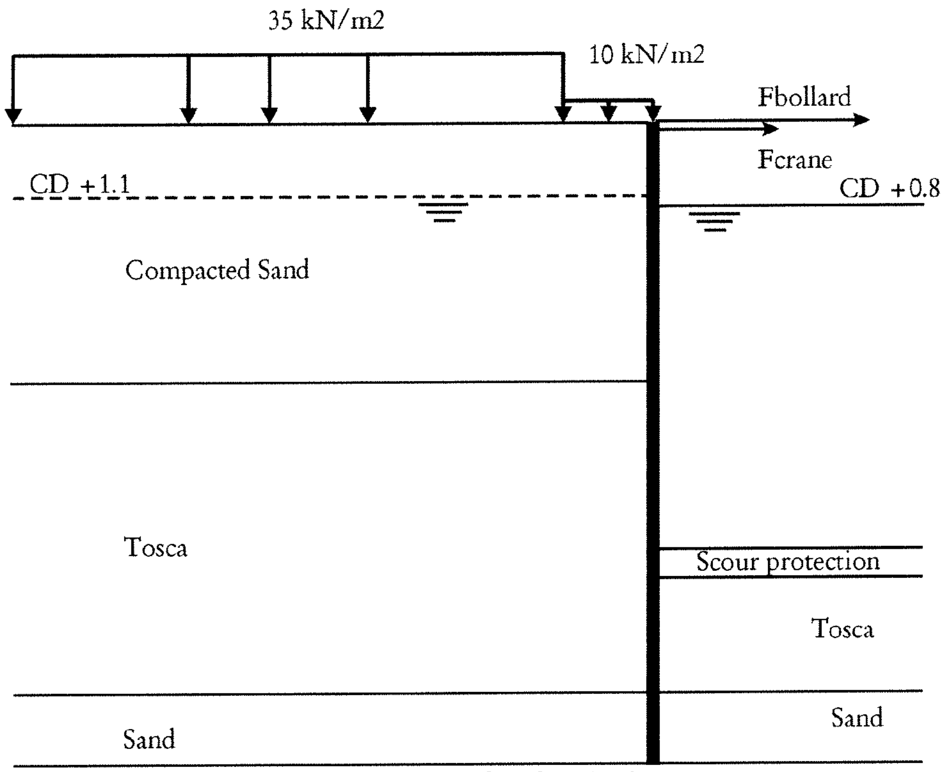


Figure 13-4 Load combination 2

Load combination 2 is governing for the strength of the combiwall and is used as Ultimate Limit State (ULS). Load combination 1 is used as Serviceability Limit State (SLS).

13.5 SOIL PARAMETERS

Most of the parameters have been taken from [AGP, *Proyecto de Ampliación de Bs.As., Obras… , 1997*]. For the fill behind the quay representative values have been chosen, source: EAU 1990.

	γ_{dry}	γ_{wet}	ϕ	c'	δ
	kN/m³	kN/m³	deg	kN/m²	deg
Fill behind quay	19	21	35	0	21.7
Tosca (drained)	18	20	29	20,	19.3
(undrained)				c_u=150 kN/m²	
Sand below tosca	19	21	35	0	23.3

Table 13–1 Representative soil properties

13.6 MATERIAL PROPERTIES

Steel:

Combiwall - tubular piles

- StE 445.7	f_y	= 445 N/mm^2
	$f_{y,calc}$	= 297 N/mm^2

Anchors, anchorwall & walings

- Fe 510 B	f_y	= 360 N/mm^2
	$f_{y,calc}$	= 240 N/mm^2

To exclude corrosion problems in the sheet piling, the concrete superstructure has been extended to CD -1 m. For the rest of the structure, corrosion problems are minor, because the quay wall is positioned in fresh water. A corrosion allowance has been taken as safety precaution, according to EAU 1990, par. 8.1.11:
Dredged Sea-bed level to CD -1 m: 0.02 mm^1/yr
This corrosion has been added to the calculated wall thickness of the tubular piles.

Concrete:

Material properties for concrete have not been mentioned here, because in this preliminary design, no calculations have been made for the concrete superstructure or crane beam. Noted is that reinforced concrete elements used in quay walls generally have a characteristic cube strength after 28 days of 45 N/mm^2.

14.CALCULATION OF QUAY WALL

In this chapter the calculation of the quay structure is presented. The results of the calculations are presented below:

Details of quay wall structure:

Tubular piles:	∅ 1420 mm, c.t.c[6]. 2670 mm, t=20 mm, length L ≈ 26.0 m, piling level CD - 24.2 m
Infill sheeting:	2 x Larssen 604, length L = 19.5 m, piling level CD - 18.0 m
Coping beam:	from CD -1 to CD +4.75
Anchor rods:	∅ 110 mm, c.t.c. 2670 mm, L = 31.5 m
Anchor wall:	Larssen 605, L = 5.5 m, piling level: CD -2.25
Waling anchor wall:	2 x thickened UNP 400, or special profile

Details of crane beam:

Foundation piles:	concrete: 450 x 450 mm, L=26.75 m, piling level CD -25.0 m

14.1 COMBIWALL CALCULATION

The main functions of the quay wall are to act as soil retaining structure and to carry horizontal and vertical forces from the quay into the subsoil and the anchoring system.
The combiwall has been checked with the method of "BLUM" and as an elastically supported sheet pile. For the calculation, the combiwall is assumed to run to the top level of the reclamation at CD +4.75. In reality the combiwall ends at CD +1.75 and on top that a coping beam is placed. The influence of this coping beam has been neglected in the sheet pile calculations, but not in the conversion of concentrated loads to line loads on the coping beam.

14.1.1 Calculation of piling level: Blum

Calculation in appendix K.2
With load combination 2: (Soil + surcharge + low water level + assumed bollard force + crane force), which is governing for the strength of the soil and the combiwall, the piling level has been calculated to be CD - 24.17 m, leading to a total length of the piles of 28.92 m and an anchor force of 769 kN/m, or 2053 kN/anchor. The required anchor rod diameter is calculated to be:

$$D_{anchor} = \sqrt{\frac{4}{\pi} \cdot \frac{F_{anchor}}{f_y / \gamma_m}} = \sqrt{\frac{4}{\pi} \cdot \frac{2053 \cdot 10^3\ N}{360/1.5\ N/mm^2}} = 104\ mm$$

As a first estimate the anchor diameter has been taken as 110 mm.

14.1.2 Spread of the bollard force

The concentrated bollard force of 1500 kN (see paragraph 13.4.3) will be spread by the coping beam on top of the combiwall. In order to determine the spreading of the bollard force, the coping beam, on which the bollard is mounted, has been modelled as a bar of approximately 30 metres of length supported by 12 springs, c.t.c. 2.67 m, see Figure 14-1.

[6] c.t.c. = center to center

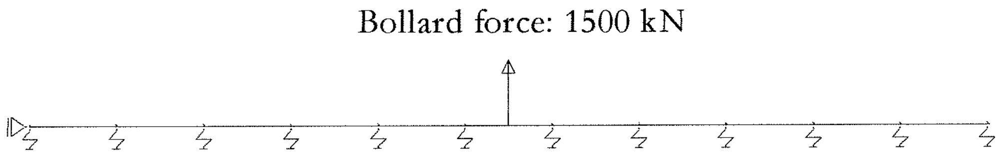


Figure 14-1 Modelled coping beam, determination of additional anchor force

The spring stiffness is determined by both the stiffness of the combiwall and the stiffness of the anchors. After each stiffness has been determined separately, the system stiffness is determined by:

$$\frac{1}{k_{system}} = \frac{1}{k_{combiwall}} + \frac{1}{k_{anchor}}$$

Stiffness of system (see Appendix K.3)
To calculate the stiffness of the combiwall, $k_{combiwall}$, a "BLUM" calculation has been carried out for load case 1 with and without bollard force. The stiffness of the combiwall perpendicular to the quay wall was calculated from the differential displacement of the top of the sheet pile wall. The stiffness of the anchors, k_{anchor}, was calculated as well:

$$k_{anchor} = \frac{E\,A}{l} = \frac{2.1 \cdot 10^{8}\ \frac{N}{mm^{2}} \cdot \left(\frac{1}{4} \cdot \pi \cdot D_{anchor}^{2}\right)}{31500\ mm}$$

The stiffness of the system was used as input for the springs in the computer program. The maximum reaction force, given as output of the computer program, was used as spread force on the combiwall. 311 kN = 116 kN/m, which is about equal to the estimated 110 kN/m. This validates the previous assumption of the spread bollard force, which has been made in the calculation of the piling level in paragraph 14.1.1. The piling level calculated there, CD -24.2 m, has been chosen for the remaining calculations.

The maximum reaction force in the calculation of appendix K.3.2 (311 kN) is almost directly transferred to the anchor. In the sheet pile calculations the bollard force has not been taken into account, but after calculating the anchor force due to soil and surcharge loads, the bollard reaction force is added to the anchor force.

14.1.3 Determination of displacements, bending moments

Calculation in Appendix K.4
Now that the piling level has been determined, the quay wall has been calculated as an elastically supported sheet pile to determine the displacements, forces and moments induced by soil pressure and surcharge load.

The results of these calculations can be found in Table 14–1 below.

Load Case	SLS	ULS
$\mathbf{M_{max}}$	1739.3 kNm/m	2937 kNm/m
$\mathbf{S_{max}}$	487.5 kN/m	671.9 kN/m
$\mathbf{u_{max}}$	54.4 mm	98.6 mm
$\mathbf{u_{top}}$	-4.1 mm	-10.8 mm
$\mathbf{F_{max,soil}}$	430.7 kN/m	679.2 kN/m
$\mathbf{F_{bollard}}$	118.4 kN/m	118.4 kN/m
$\mathbf{F_{anchor,max}}$	549.1 kN/m	797.6 kN/m
	1466.1 kN	2129.6 kN

Table 14–1 Forces and displacements of Load Cases

M:	bending moment
S:	shear force
u_{max}:	maximum displacement
u_{top}:	displacement of top of combiwall
$F_{max,soil}$:	maximum anchor force due to soil and surcharge loads
$F_{bollard}$:	maximum anchor force due to bollard load
$F_{max,anchor}$:	maximum anchor force (all loads)

The maximum bending moment in the structure amounts 2937 kNm/m.
The combiwall is checked for strength with the following formula:

$$\frac{M_{\max}}{W_{combiwall}} \leq \frac{f_y}{\gamma_m}$$

$$\frac{2937e3}{10400} = 282\frac{N}{mm^2} \leq 297\frac{N}{mm^2} \Rightarrow o.k.$$

The maximum stresses in the combiwall in ULS are: 282 N/mm^2, giving the combiwall enough safety against failure. In working conditions the maximum stresses in the sheet pile will amount to approximately 167 N/mm^2.

14.2 DESIGN OF ANCHORS AND ANCHORWALL

In this paragraph the calculations of the anchors and anchorwall have been summarized. The calculations can be found in appendix K.5 - K.8

14.2.1 Anchor rod

The maximum anchor force that results from the elastical calculation of the sheet pile in ULS amounts 797.6 kN/m, see Table 14–1. When the anchors are placed 2670 mm c.t.c., the force in each anchor becomes 2130 kN. The required anchor diameter of the anchor rod is calculated below.

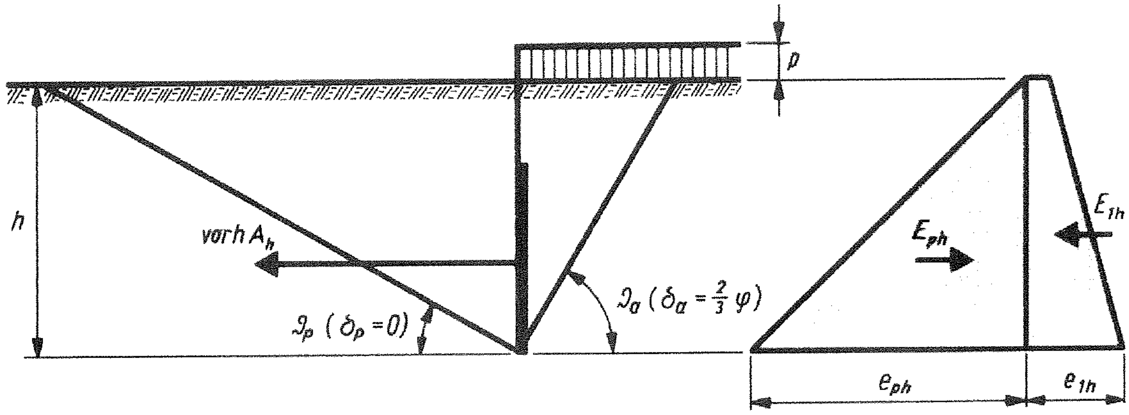
$$\frac{F_{anchor,\max}}{A_{anchor}} \leq f_y \Rightarrow D_{anchor} = \sqrt{\frac{4 \cdot F_{anchor,\max}}{\pi \cdot f_y / \gamma_m}} \Rightarrow D_{anchor} = 106\ mm$$

This shows that the chosen anchor, ∅110 mm, c.t.c. 2670 mm meets the required strength.

14.2.2 Stability of anchorage

The stability of the anchorage has been analysed for two cases:
- lower failure plane [Kranz]
- higher failure plane

The calculation of the stability in the higher failure plane has been carried out to validate the safety against failure of the anchoring soil.



$$\eta = \frac{E_{ph}}{\text{vorh } A_h + E_{1h}} \geq 1{,}5$$

Figure 14-2 Forces on anchoring soil, source: Hoesch

In appendix K.5 the forces as shown in Figure 14-2 are calculated and the calculation results that the chosen wall height of 5.5 m is sufficient to ensure the safety of the anchoring soil. The minimum distance between anchor wall and combiwall is 26.3 m. It is important that the active soil wedge from the combiwall does not overlap the passive soil wedge of the anchoring soil.

14.2.3 Kranz stability calculation

Calculation in Appendix K.6

The stability of the quay wall in the lower failure plane has been checked with the method of Kranz. The friction force of the soil has to able to withstand induced forces on the anchored combiwall (anchor force, reaction forces). See Figure 14-3. With the chosen anchor length and anchor depth, the quay wall is stable in the deeper failure plane.

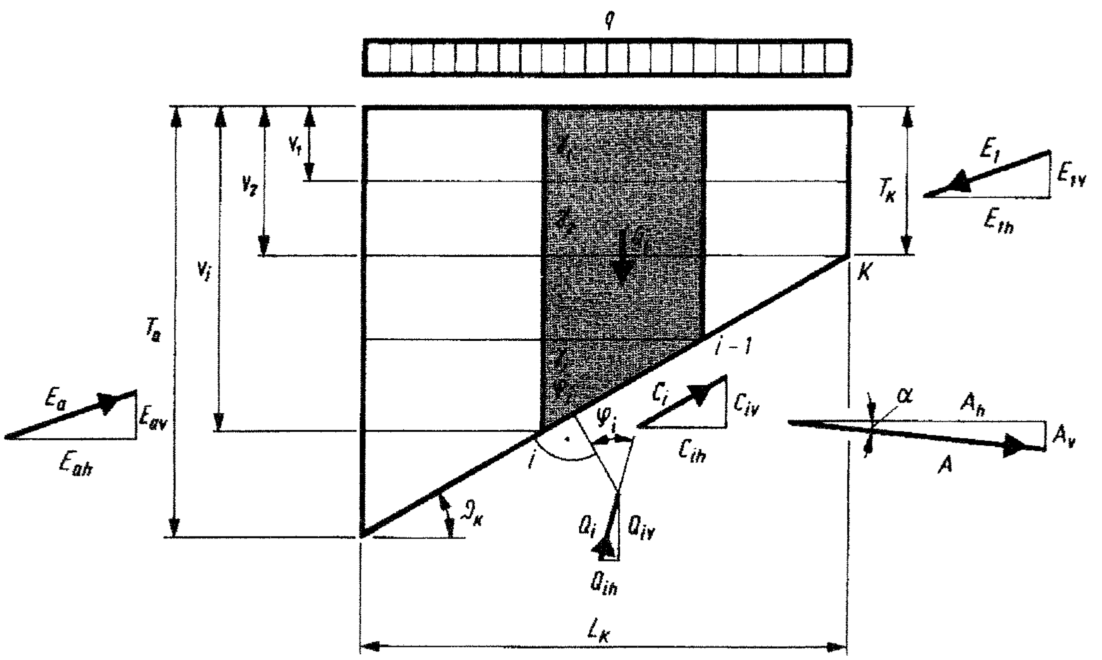


Figure 14-3 Forces on soil as used in the Kranz stability calculation, Source: Hoesch

14.2.4 Anchor wall: Sheet pile determination

Calculation in Appendix K.6.1

In paragraph 14.2.2 the height of the anchor wall has been calculated: 5.50 metres. The anchor rod is not positioned in the centre of the anchor wall, but has been placed 0.75 metres off the centre at CD + 1.25m. This induces an extra moment in the sheet pile and the maximum moment in the sheet pile is: 467.7 kNm/m (appendix K.6)

The required section modulus is then:

$$W = \frac{M}{f_y / \gamma_m} = \frac{467.7 \cdot 10^6 \ Nmm/m}{355/1.5 \cdot \dfrac{N}{mm^2}} = 1985 \cdot 10^3 mm^3 / m$$

A continuous Larssen 605 sheet pile (W = 2020 * 10^3 mm^3/m, appendix X.10) has been chosen as anchorwall.

14.2.5 Anchor wall: Calculation of waling

Calculation in Appendix K.6.2

The anchor force is transferred from the anchor rod to the sheet pile by means of a waling. The maximum moment in the waling is calculated to be:

$$M_{waling} = \frac{1}{10} \cdot q_{anchor} \cdot l^2 = \frac{1}{10} \cdot 797.6 \frac{kN}{m} \cdot (2.67 \ m)^2 = 568.6 \ kNm$$

The required section modulus is:

$$W = \frac{M}{f_y / \gamma_m} = \frac{568.6 \cdot 10^6 \ Nmm/m}{355/1.5 \cdot \dfrac{N}{mm^2}} = 2369 \cdot 10^3 mm^3/m$$

The required section modulus is just a bit larger than a double UNP 400 profile waling (W=2040 mm^3). Therefore this waling will be thickened. The total length of the quay wall is about 3600 metres, thus a special delivery of an adjusted waling can be more economical than thickening all original profiles.

14.3 DESIGN OF CRANE BEAM FOUNDATION

Calculations in Appendix K.7

14.3.1 Load determination of crane beam

The forces induced by the container crane are spread by the crane beam. The horizontal crane force (60 kN/m) is transferred to the combiwall, the vertical force is carried to the subsoil by several foundation piles. The spreading of the concentrated wheel loads of 750 kN is calculated by modelling the crane beam as an elastically supported beam, see Figure 14-4. First, the concentrated wheel loads are replaced by a line load of 526 kN/m (over a distance of 11.4 m). With *TECHNOSOFT Raamwerken* the maximum pile reaction force is calculated to be 1345 kN, appendix K.8. The design force of the foundation piles, becomes (after applying the safety factor):

$$F_{cr,pile} = 2.5 \cdot 1345 = 3363 \ kN$$

This force is carried to the subsoil by foundation piles of 0.45 x 0.45 x 26.75 m^3.

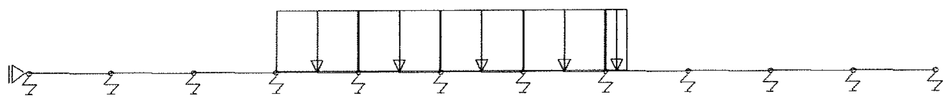


Figure 14-4 Modelled crane beam, determination of pile reaction force

14.3.2 Additional load of foundation piles: negative skin friction

When the crane beam foundation piles have been driven to their piling level, the soil around the piles will still undergo some settlements. Due to friction between the piles and the surrounding soil an additional load is introduced: the so called negative skin friction. The negative skin friction is calculated with the 'slip' method and has to be taken up by the foundation pile:

$$F_{skin,neg} = O_s \cdot h \cdot \overline{\sigma}'_v \cdot K_0 \cdot \tan\delta$$

in which	O_s:	outline of pile (4 * Diameter)
	h:	height of soil layer(s)
	σ'_v:	average soil pressure in soil layer
	$K_0 \tan\delta$:	friction factor: 0.25

The negative skin friction is calculated to be: 1787 kN and the total induced force on the foundation pile becomes: 3363 + 1787 = 5150 kN (appendix K.8)

14.3.3 Determination of pile point resistance

From the schematized CPT in appendix X.X, can be seen that around CD -21.0 m the bearing capacity increases to high values (15 MPa and up). When a pile is driven into the sand layer soon the maximum value of 15 MPa (NEN 6740) is reached for the pile point resistance. The total allowable force due to this pile point resistance is:

$$F_{pilepoint} = A_{pile} \cdot 15\,MPa = 0.45^2 \cdot 15 \cdot 10^3 = 3038\,kN$$

14.3.4 Determination of positive skin friction

Driving the foundation pile into the sand layer, positive skin friction will develop. This skin friction determines the piling level of the foundation pile, as it is calculated with the formula below:

$$F_{shaft,pos} = O_s \cdot x \cdot \overline{\sigma}'_v \cdot K_s \cdot \tan\delta$$

in which	O_s:	outline of pile (4 * Diameter)
	x:	depth of pile point in sand layer
	σ'_v:	average soil pressure in sand layer
	$K_0 \tan\delta$:	friction factor: 0.25

Solving the equation below leads to the required piling level CD -21.0 m - x m:

$$F_{cr,pile} + F_{shaft,neg} \le F_{\text{pile point}} + F_{shaft,pos}$$

This minimum depth x in the sand layer becomes 3.35 m, or safely chosen: 4.0 m, leading to a piling level of CD -25.00 m. At this depth the pile point resistance is more than 15 MPa (calculated with *Koppejan*)

Note: The level of the sand layer varies over the reclaimed area. Exact piling levels must therefore be calculated with local soil profiles.

14.4 COST DETERMINATION OF QUAY WALL

The construction costs per metre of quay wall have been calculated in this chapter by determination of the direct costs.
The direct costs are: materials, construction equipment and labour.
The indirect costs, risk and profit have been assumed to be proportional to the direct costs; about 75% of the direct costs.

Materials
The following prices have been used for the materials in the calculation below.

Component	Material costs
Steel	$ 0.75 / kg
Concrete	
Crane beam	$ 300 / m3
Coping beam	$ 325 / m3
Floor /tiles	$ 250 / m3
Foundation piles	$ 55 / m1
Paving	$ 50 / m2
Scour protection	$ 10 / m3
Fendering & bollard	10 % of direct costs

Table 14–2 Material costs of quay wall

For equipment and labour, thus the installing of the materials the following prices have been used:

Component	Installation costs
Steel	
Combiwall	25 % of material costs + pontoon
Anchors/anchorwall	25 % of material costs
Concrete	labour included in material price

Table 14–3 Installation costs quay wall

The calculation of quay wall costs is presented below

Costs Quay Wall

				Total costs	Costs per terminal
COMBIWALL					
Tubular piles combiwall	6727 kg/m	5.045	$ / m	18.036.478	1.387.421
Infillsheeting Larssen 604	1170 kg/m	878	$ / m	3.137.966	241.382
Equipment & Labour		1.481	$ / m	5.293.611	407.201
Pontoon		15.000	$ / month	480.000	36.923
TOTAL COMBIWALL				26.948.055	2.072.927
ANCHORWALL					
Anchorwall Larssen 605	765 kg/m	573	$ / m	2.049.816	157.678
Anchorrods	880 kg/m	660	$ / m	2.359.010	181.462
Equipment & Labour		308	$ / m	1.102.207	84.785
TOTAL ANCHORWALL				5.511.033	423.926
CONCRETE					
Rear crane beam	2,4 m3/m	720	$ / m		
Front crane beam	2,4 m3/m	720	$ / m		
Coping beam	8,1 m3/m	2.633	$ / m		
Floor of cable gutter	1,3 m3/m	315	$ / m		
Tiles	1,2 m3/m	288	$ / m		
TOTAL CONCRETE	15,3 m3/m	4.675	$ / m	16.713.125	1.285.625
FOUNDATION PILES					
Rear crane beam	10,02 m1/m	551	$ / m	1.969.932	151.533
Front crane beam	10,02 m1/m	551	$ / m	1.969.932	151.533
TOTAL FOUNDATION PILES				3.939.864	303.066
OTHERS					
Paving till crane beam	33 m2/m	1.650	$ / m	5.898.750	453.750
Scour protection	20 m3/m	600	$ / m	2.160.000	166.154
SUBTOTAL MATERIALS		16.992		61.170.827	4.705.448
Fendering + bollards		1.699		6.117.083	470.545
TOTAL DIRECT COSTS		18.691	$ / m	67.287.909	5.175.993
COSTS QUAY WALL		*33.000*	$ / m	*117.760.000*	*9.058.462*

Table 14–4 Quay wall cost determination

BUENOS AIRES PORT EXPANSION 2020

PART 5:

CONSTRUCTION PHASING AND FEASIBILITY

15. GEOTECHNICAL INFORMATION AND DREDGING METHOD

15.1 SOIL OVERVIEW

The soil of the Rio de la Plata consists generally of 3 layers, a soft upper layer of alluvial deposits, a layer of *tosca* and a sand layer. Not all of these layers can be dredged with the same dredging material and not all of them can be used in the reclamation. Below the three layers will be described in more detail and the appropriate dredging equipment will be mentioned.

15.1.1 Soft upper layer: alluvial deposits

Description of layer

The upper layer consists of a recent alluvial deposit, a very soft clayey silt and very soft clay. It and starts around CD -3 to -4 metres and it ends at approximately CD -7 to CD –14 m, so its thickness ranges from 4 to 10 metres. This layer is unsuitable as reclamation material because of its softness and therefore it has to be transported to an offshore dump or another location[1].

Soil Parameters:

Undrained strength c_u = 12 kPa
Undrained unit weight: 17.1 kN/m3
Drained weight: 17.5 kN/m3
Drained strength: c'=0 kPa, ϕ'=22°
Equilibrium slope: 1:6 to 1:8

Suitable dredging equipment

To dredge this material, generally a Trailing Suction Hopper Dredger (TSHD) would be used. However, hoppers cannot dredge in the shallow parts of the Rio de la Plata, because of their relatively large draft. Instead a Cutter Suction Dredger (CSD) will be used. Whenever there is sufficient draft for hoppers to operate (maintenance works / dredging of basins) hoppers will be used.

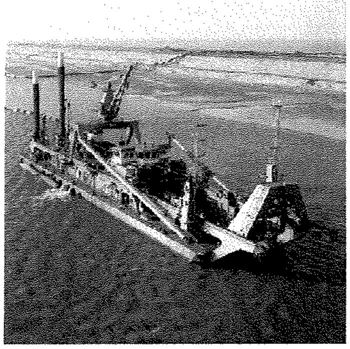


Figure 15-2 Cutter suction dredger Hector, Source: Ballast Nedam Dredging

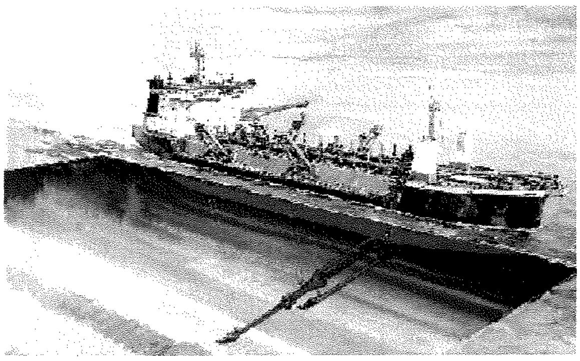


Figure 15-1 Trailing suction hopper dredger Amsterdam, Source: Ballast Nedam Dredging

[1] Either the ecological reserve or a depleted near-shore pit

15.1.2 *Tosca* layer

Description of layer

The second layer consists of *tosca*, a pampean formation of stiff clayey silt and very stiff clay, which can be cemented at some places. At the coast the formation begins near the surface (7 m -CD) and further from the coast it begins deeper (14 m -CD). The layer ends at 21 to 24 metres below CD and therefore has a thickness of 10 to 15 metres.
When the layer is left in place, it has enough bearing capacity, but when dredged the bearing capacity of this layer reduces to almost none. If the layer is used in the reclamation it will take a long time before the layer is settled and large settlements are expected. In the first phase of the port expansion, this layer will not be used on the terminal terrains.
Note: It is difficult in this stage to decide whether or not tosca is suitable as reclamation material. Additional information and geotechnical tests in a laboratory must prove the suitability in a later stage.

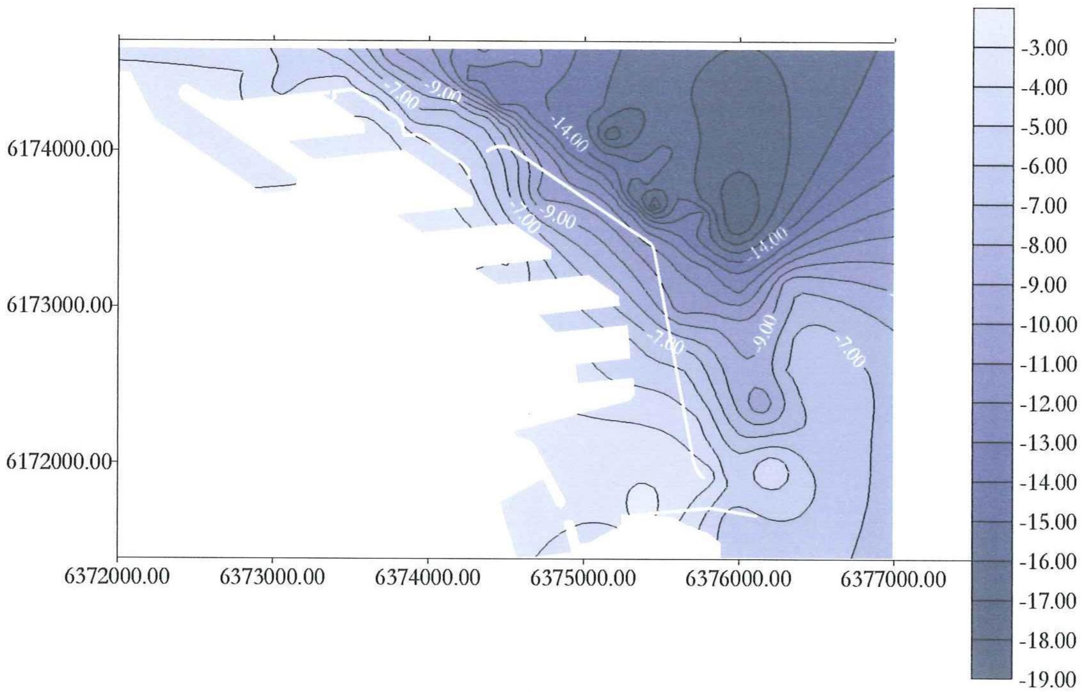


Figure 15-3 Level of tosca layer [m below CD]

Soil Parameters

Unit weight:	dry:	18 kN/m3
	saturated:	20 kN/m3
Undrained shear strength:		c_u = 100-150 kPa
Drained strength:		c' = 20 kPa, ϕ'=29°, δ=19.3°
Dredging slope:		1:3

Dredging equipment

The pampean formation cannot be dredged in large quantities by other equipment than a cutter suction dredger (CSD). A medium or large CSD can be used, depending on the distance over which the soil has to be transported and the shear strength of the *tosca*.

15.1.3 Puelche formation: sand layer

Description of layer

The third layer consists of a dense to medium dense fine sand, and begins below the *tosca* layer at a level of about 21 to 24 metres -CD. From there it continues to over a level of 45 metres -CD. This soil will be used in the reclamation

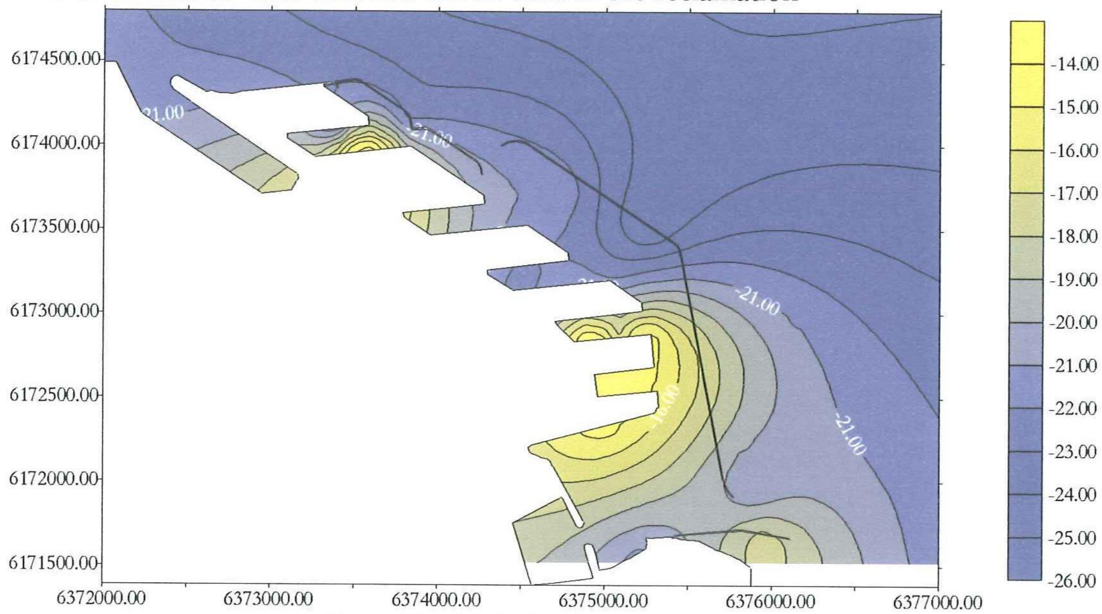


Figure 15-4 Level of sand layer [m below CD]

Soil Parameters

Unit weight:	dry:	19 kN/m3
	saturated:	21 kN/m3
Effective cohesion:		c' = 0 kPa
Effective angle of friction		$\phi' = 35°, \delta=23.3°$

Dredging slope: 1:4

Dredging equipment

The dense to medium dense fine sand may be dredged by a deep suction dredger (DSD), and will be transported by means of a (floating) pipeline.

15.1.4 Schematised soil profile

As the levels of the various soil type can vary considerably within a short distance of the port, for this report a schematised soil profile has been assumed. In this profile, the seabed is situated at CD –3.00 m, the top of the *tosca* layer at CD –7.00 m, and the top of the sand layer at CD –21.00, see left part of Figure 15-5. This soil profile has been assumed throughout all of the port, the access channels and its expansion.

Data sheets, a schematised Cone Penetration Test – diagram and soil pressure diagrams can be found in Appendix E.

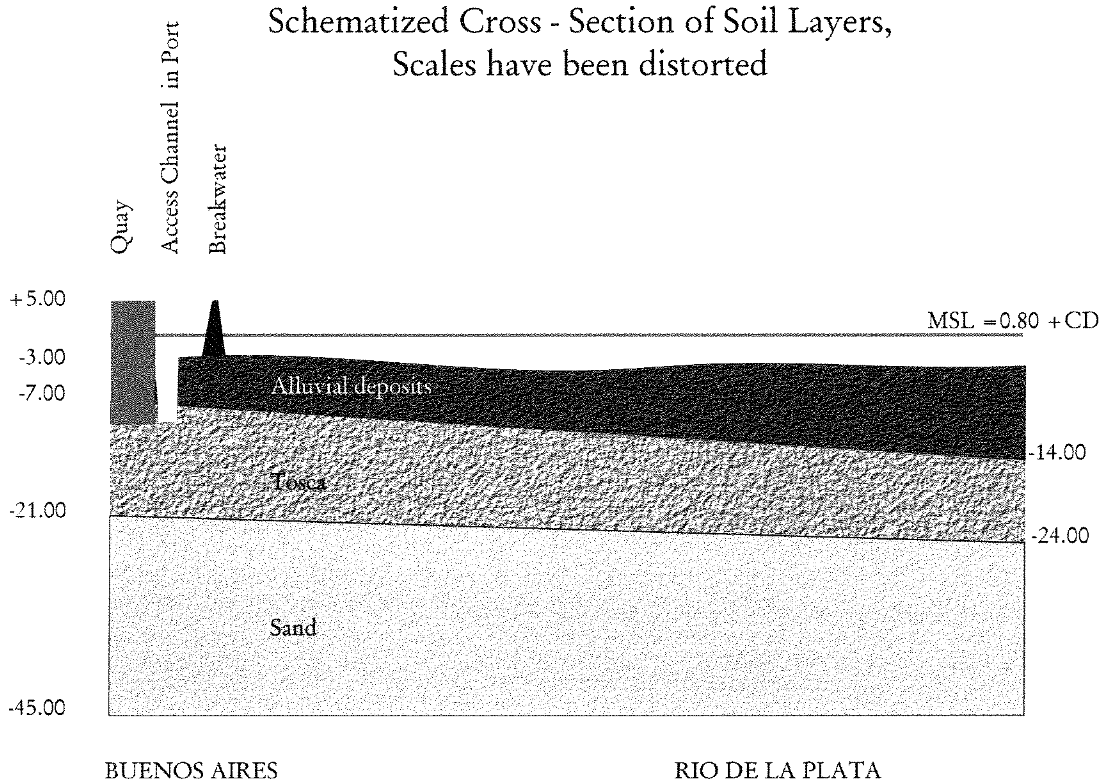


Figure 15-5 Schematized cross-section of soil layers

15.2 DREDGING METHODS

15.2.1 Capital dredging: dredging from near-shore sand pits

The sand and *tosca* will be dredged from near-shore pits. In order to reach the *tosca*, first the layer of alluvial deposits will have to be removed.

Removal of the alluvial deposits

Generally, the soft silty alluvial deposits would be dredged by TSHD's. However, the Rio de la Plata is so shallow that a TSHD could not carry out its work. Therefore a medium CSD, type Haarlem, will be used to carry out the dredging works. The material of the first pit will be dumped on an offshore location, about 30 km away, see Figure 15-6. This is done by two TSHD's, type Apollo. It is not possible to use barges instead, because these large barges do not exist, cannot unload themselves and several tugs are necessary to transport the barge over such long distances.

Thus, the material will be pumped from the CSD into a TSHD, which will sail and dump the material on the offshore dump location.
From the second pit on, the alluvial deposits will be pumped directly in the adjacent depleted sand pit, no TSHD's will be necessary anymore.

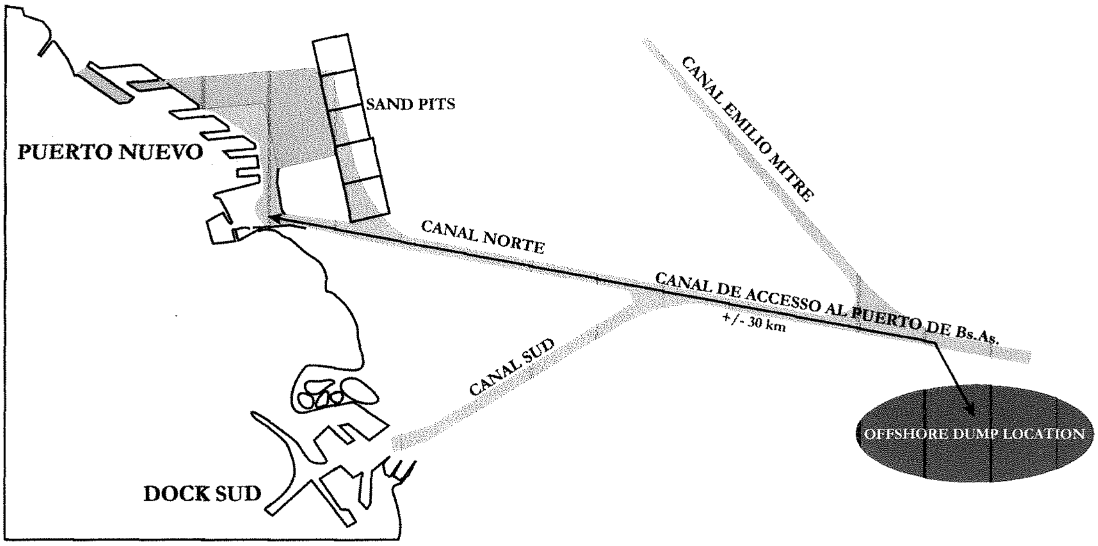


Figure 15-6 Location of sand pits and offshore dump

N.B. The material could also be used to expand the Ecological Reserve, but this is an uncertain option.

Capacities and costs

Using 2x TSHD:	capacity: 175,000 m^3/wk, costs: \$2.50 / m^3
Using CSD + 2x TSHD:	capacity: 200,000 m^3/wk, costs: \$4.50 / m^3
Using CSD	capacity: 270,000 m^3/wk, costs: \$2.00 / m^3

Tosca

When the layer of alluvial deposits is removed, the next soil layer, the *tosca* can be dredged. The *tosca* will be dredged by means of a medium (type Haarlem) or large (type Hector) CSD, which pumps the material into the reclamation area by means of pipeline transport. It depends on the transporting distance and the depth of the tosca layer whether a medium CSD is sufficient or the large CSD has to be used.

Capacities and costs:

Using a medium CSD:	capacity: 130,000 m^3/wk, costs: \$3.50 /m^3
Using a large CSD:	capacity: 230,000 m^3/wk, costs: \$3.00 /m^3

Sand

Thirdly, the sand of the lowest layer will be dredged by a DSD towards the lowest level of the sand pit (the ultimate dredging depth of the DSD). Depending on the dredging depth and transporting distance a medium DSD (Zuiderklip) or a large DSD (converted CSD Hector) will be used. Either way, the sand is transported to the reclamation area by means of a pipeline. After the first pit has been dredged to its ultimate level, it will be abandoned and a new pit will be created in almost the same way as described above; the overburden of the pit will not be transported to sea, but the depleted pit will be filled with the overburden of the second pit.

Capacities and costs
Using medium DSD: capacity: 260,000 m^3/wk, costs: \$2.00 / m^3
Using large DSD: capacity: 330,000 m^3/wk, costs: \$2.25 / m^3

When a medium DSD is used, less efficient use is made of the sand pits and for the same volume of sand in the reclamation more (or larger) pits are required. The construction of an extra sand pit is always more expensive than the usage of a larger DSD. Therefore, a large DSD will be used to dredge sand.

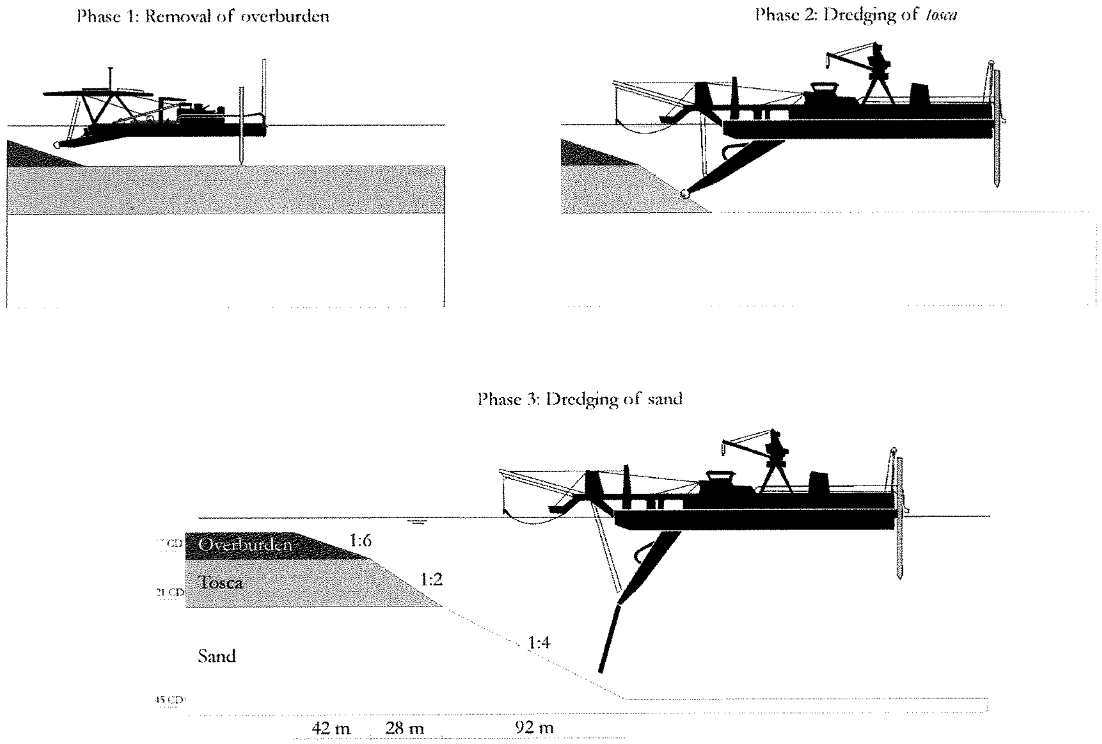


Figure 15-7 Dredging sand from near shore sand pits: working method

15.2.2 Capital dredging: dredging sand in the Rio Paraná

Another possibility is to dredge the sand directly from the bottom of the Rio Parana, approximately 100 km upstream. First the sand is dredged by trailing suction hopper dredgers (TSHD), after that the hoppers will sail to the reclamation area and finally the sand will be pumped into the reclamation area. This can be done by pipeline or by 'rainbowing' (see below)

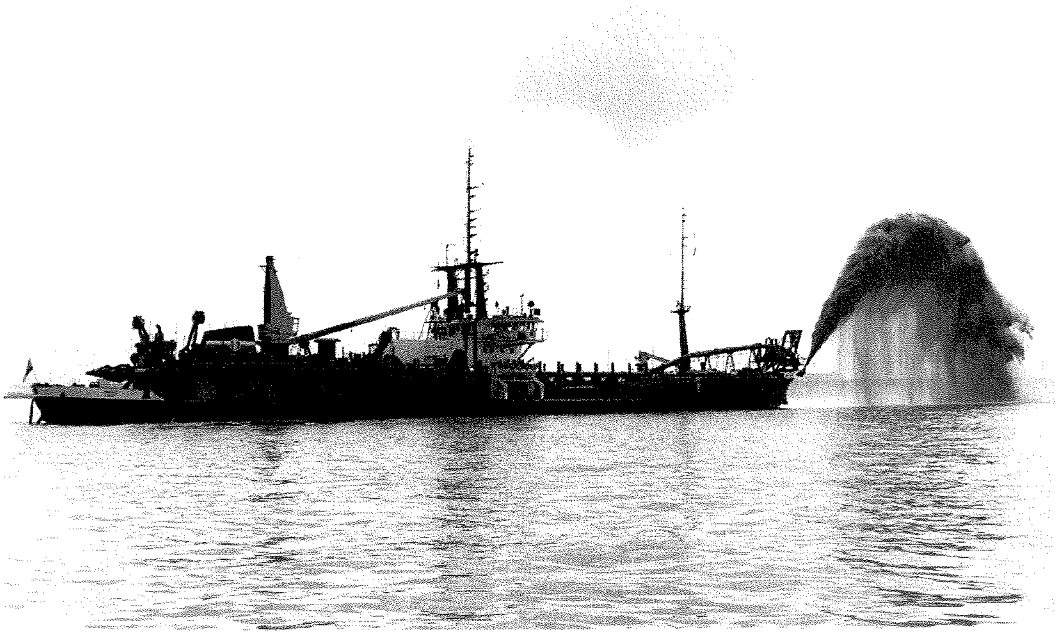


Figure 15-8 Trailing Suction Hopper Dredger, rainbowing

Costs of dredging sand at Rio Parana
When dredging the sand from the Rio Parana, several TSHD's with capacities of about 5000 ton (type Ham 311) will be used.
The capacity of such a dredger is (in this case) approximately 32,000 m^3/week. The costs of a this type of hopper dredger are approximately $260,000 / wk and this results in a price of about $8.00 per m^3 dredged sand.

15.2.3 Maintenance dredging

Maintenance dredging will be carried out, when possible, by trailing suction hopper dredgers, which dump the dredged soil offshore. For this purpose TSHD's type Apollo with capacities of about 4800 m^3 will be used. Dredging spoil from within the port cannot be dumped offshore as it might be heavily polluted.

Capacities and costs
Using medium TSHD: capacity: 75,000 – 100,000 m^3/wk, costs: $2.50/ m^3

15.3 CHOICE OF DREDGING METHOD

The choice of the dredging method is one which depends on capacity and costs. The capacity of dredging near shore sand pits is much higher than when dredging sand from the Rio Paraná (depends on the number of operated dredgers of course) To compare the costs of both dredging methods, the costs for the dredging of sand pits in the most negative scenario are compared to the price of dredging the sand from the Rio Paraná. The results are presented below:
When a large deep suction dredger is used, which can dredge to a depth of 45 metres, 42 metres of soil have to be dredged. Of these 42 metres, the first 18 metres of 'soft' material cannot be used for the reclamation and will be transported elsewhere. The other 24 metres will be used[2] for the reclamation.

[2] These values may differ, according to the location of the pit. Here average values have been taken for the levels.

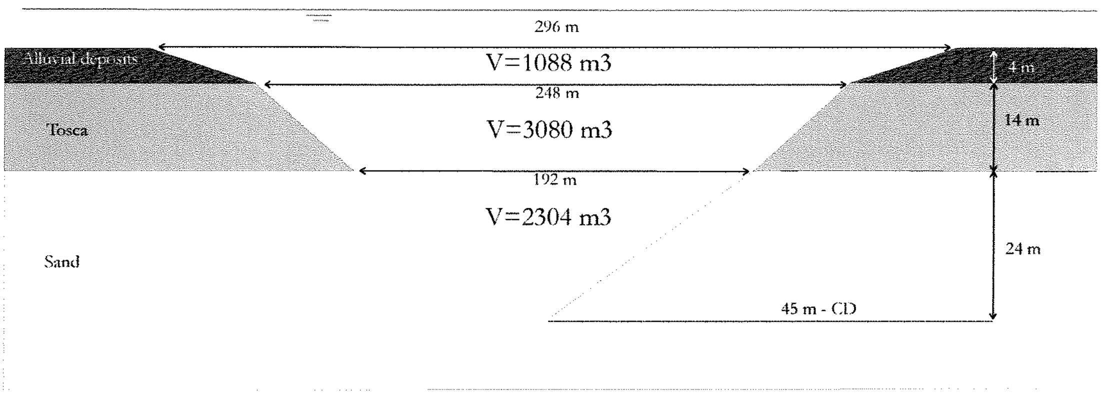


Figure 15-9 Cross section of smallest possible pit with depth of 45 m

Using the smallest possible pit with a depth of 45 metres (see Figure 15-9), and thus making the worst use of it, the average costs per m^3 will be

$$\frac{1088 \cdot \$4.5 + 3080 \cdot \$3.00 + 2304 \cdot \$2.25}{(2304)} = \$8.39 / m^3 \text{ suitable material}$$

This is only a bit more expensive than dredging sand from the Rio Parana (\$8.00/m^3). When larger and deeper pits are used and the overburden layer is pumped in to a depleted pit, the price of dredging sand and tosca from pits will become lower. For example: when the first pit has a bottom dimension at CD –45 m of 300x300 m^2, the average price per m^3 of suitable material is \$6.63, for a second pit of the same dimensions, the price becomes \$5.81/m^3.
During construction of the expansion five (or more) sand pits will be used, so only a 1/5th of the alluvial deposits will have to be transported to the offshore dump location.

15.4 EXAMPLE OF COST DETERMINATION (INDICATIVE ONLY)

Dredge Sand with Hector, a CSD converted to Deep Suction Dredger

Production:	3,000 m^3 / OH[3]
Operating hours:	110 / wk
Weekly production	330,000 m^3/wk

Costs dredger	Costs (fl/wk)	Totals (fl)
A+R	110,000	
O+r (fixed)	30,000	
O+r (variable)	50,000	
W+T 0.12/m^3	40,000	
Insurance	20,000	
Crew (export)	60,000	
Crew (local)	20,000	
Living	10,000	
Fuel	160,000	
Total costs dredger		500,000
Added Costs		
Floating pipeline	100,000	
Reclamation area	60,000	
Multicat	40,000	
Total added costs		200,000
Other costs		
Staff (1/3)	50,000	
Mobilisation	150,000	
Crew/survey/staff launch	50,000	
Total other costs		250,000
Total costs per week		950,000
Costs per m^3 dredged sand		2.88/m^3
Costs after mark-up		**4.50/m^3**
		(≅ $2.25)

[3] OH = Operating Hour

16.CONSTRUCTION PHASING

As mentioned in part 2 of this report, alternative 3 will be constructed in phases. In this chapter all the steps needed for the construction of this alternative are described.

16.1 INTRODUCTION

In the second part of this report, the medium growth scenario was adopted for the design of the alternatives of the port expansion. From the throughputs of this scenario, the required number of new terminals in Puerto Nuevo is found by dividing the throughput exceeding 1.5 million TEU by the capacity of a single terminal (300,000 TEU)

Year	Throughput Puerto Nuevo (TEU)	Required new terminals
2001	1,098,000	0
2002	1,290,000	0
2003	1,505,000	1
2004	1,705,000	1
2005	1,882,000	2
2006	2,025,000	2
2007	2,176,000	3
2008	2,337,000	3
2009	2,507,000	4
2010	2,688,000	4
2011	2,879,000	5
2012	3,081,000	6
2013	3,296,000	6
2014	3,524,000	7
2015	3,765,000	8
2016	4,021,000	9
2017	4,292,000	10
2018	4,580,000	11
2019	4,885,000	12
2020	5,208,000	13

Table 16–1 Required number of new terminals in time, medium scenario

Table 16–1 shows that in the coming 10 years only four extra terminals are required. In the 10 remaining years till 2020 nine more terminals are required. The port expansion will be constructed in 3 phases, which are described below in paragraph 16.2. After the first phase, which consists of 4 terminals, the annual capacity of Puerto Nuevo amounts 2.7 million TEU, after the second phase (another 4 terminals) 3.9 million TEU and after third phase (5 terminals) 5.4 million TEU. This capacity will be sufficient for the port of Buenos Aires to reach the following decade.

16.2 DESCRIPTION CONSTRUCTION PHASES

16.2.1 Construction phase 1

In the first phase 4 new terminals are constructed, starting at the end of Basin D and building eastwards into the Rio de la Plata. The first terminal is required around 2003, the second and third in respectively 2005 and 2007, and the fourth terminal has to be operational at the end of 2008. In this phase, a connection is made from the peninsula to the main land and the access channel to Puerto Nuevo is widened and deepened.

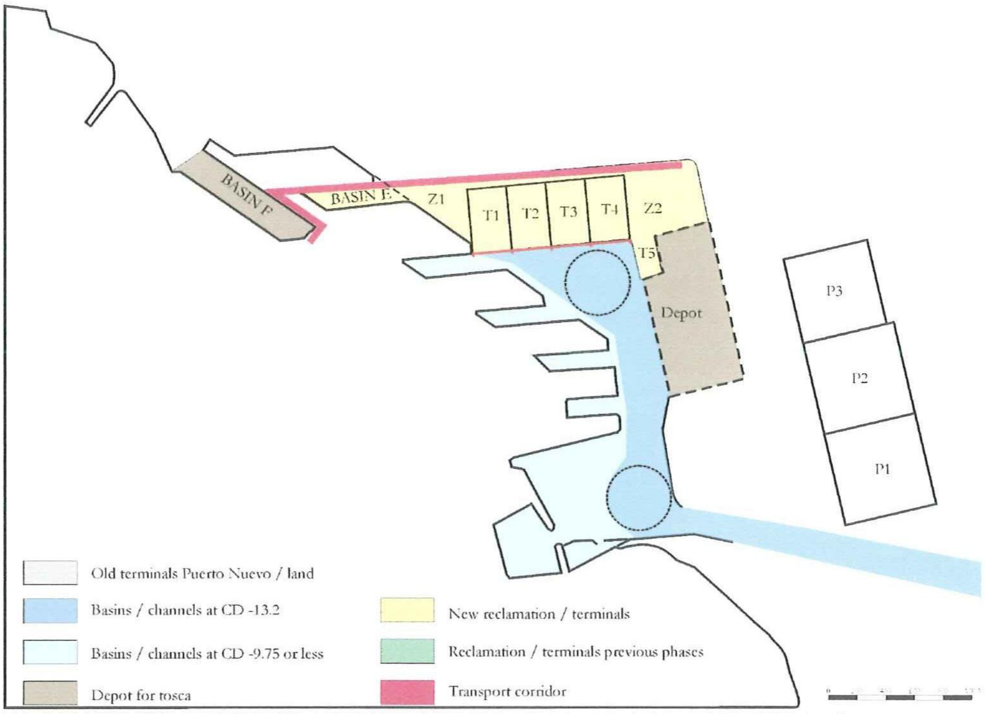


Figure 16–1 Construction phase 1: connection and 4 container terminals

Before the construction of phase 1 starts, additional soil investigation must give information whether or not *tosca* can be used as reclamation material. When it can be used, settlements will be larger and it will take a longer time before residual settlements are reduced to a minimum. In this study, *tosca* has not been chosen as reclamation material in the first phase. Under the assumption that the investigation of tosca will be positive a depot for *tosca* is constructed in the first phase, which will act as reclamation in the second or third phase. The first four terminals are required within a few years and no time is available for long settlement times during the first phase. For the first four terminals only sand is used as reclamation material, which is dredged from near shore sand pits, see Figure 16–1. Three sand pits will be used, dividing phase 1 into three sub-phases. The configuration of the sand pits is not the same as in appendix H, but the dimensions have been adapted to this construction phase. The construction sequence of phase 1a, 1b and 1c is described below according to the planning of the construction works in appendix M.

Construction phase 1a

Before the sand layer in the first sand pit is reached, 5.88 million m^3 soft material must be dredged away (1.48 million m^3 alluvial deposits and 4.40 million m^3 tosca);

Construction sequence:

- At first a depot is constructed in which *tosca* from the first sand pit is deposited, see Figure 16–1 and planning in Appendix M.
- The alluvial deposits from the first sand pit P1 are dredged by means of a medium CSD and dumped offshore by two TSHD's
- *Tosca* of sand pit 1 is dredged by a large CSD and pumped into the previously constructed depot

- With the sand from the first sand pit basin E, zones Z1 & T1 are filled and a surcharge is placed on T1. After that, T2 is filled to CD[1].
- While the deep suction dredger is reclaiming above zones, the TSHD's dredge alluvial deposits in the port and access channel.
- The widening and deepening of the access channel starts as early as possible as it will take almost two years to finish the dredging works. The alluvial deposits are dredged by 2 TSHD's and dumped offshore. The *tosca* is dredged by a CSD, pumped in another 2 TSHD's and dumped offshore.
- The construction of the northern shore protection starts just after the reclamation of zone Z1.
- After the reclamation of the various zones a working road to the new terminals is constructed.

Construction phase 1b
After sand pit 1 is depleted a new sand pit P2 on the north of sand pit 1 is created.

- The medium CSD dredges the alluvial deposits from P2 to the depleted sand pit P1.
- Both the medium and large CSD dredge tosca and pump it into P1 as well.
- Sand is dredged by the DSD into the reclamation area. First T2 is filled to its maximum level, then T3 & T4 are reclaimed. After that, sand pit P2 will be depleted.
- The construction of the quay wall for terminals 1-4 starts in this phase, at an average construction speed of 100 m per month.

Construction phase 1c

- The soft layers from the third sand pit P3 are dredged into P2
- Zone Z2 and a part of terminal 5 are reclaimed, the *tosca* depot of phase 1a and basin F are finished with a sand layer and basin E is filled up to its final level.
- The small and large breakwater will be removed during construction phase 1. These works have not been planned in appendix M, because the required amount of time and the removal speed is unknown.
- The working road is replaced by a final road
- A (temporary) shore protection is constructed on the eastern side of the peninsula.

Construction of container terminals
In the container terminals two different areas can be distinguished: the quay area and the stacking area.
The stacking area is the first to be constructed. On top of the reclaimed level (CD +4.75) a surcharge of 4.25 m is placed to speed up the settlements of the soft layers. This surcharge will remain there for approximately 5 months.

Behind the quay wall soft layers are removed until sufficient bearing capacity is found in the *tosca* layer. Then the area will be hydraulically filled to CD with a spraying pontoon. On top of this, another layer of sand is reclaimed to a level of CD +4.75. The settlements behind the quay wall will be small, because only sand is used and almost no residual settlements are expected after commissioning of the terminals. From the surcharge on the stacking area sand will be taken to flatten the quay area just before the

[1] For the location of the various zones mentioned in this paragraph, see Figure 16–1

placement of paving blocks. The quay wall itself is constructed according to the drawings in appendix L at an average construction speed of about 100 m / month. When the surcharge is removed from the stacking area and the quay wall is finished, it will take the terminal operator approximately 6 months to finish the superstructure of the terminal. The terminal operator needs to take care of at least the following actions:

- placing of Post Panamax container gantry cranes
- paving of the stacking area
- installation of yard facilities (electricity / water)
- installation of container yard cranes and other yard equipment
- construction of gates, parking lots and other buildings

When the working method of Appendix M is followed, and construction phase 1 starts at 1-1-2002, the first terminal is operational around the 1st of July 2003. Six months later, all 4 terminals are operational and both access channel and basins will have a depth of CD –13.2 m. The total construction time of phase 1 is approximately 2 years.

16.2.2 Construction phase 2

In 2004, four new container terminals will be operational and the capacity of Puerto Nuevo will increase to about 2.7 million TEU. This gives the port enough capacity to operate without problems until 2010. In 2011 a fifth new terminal must be operational, and the next construction phase will start around 2009-2010.

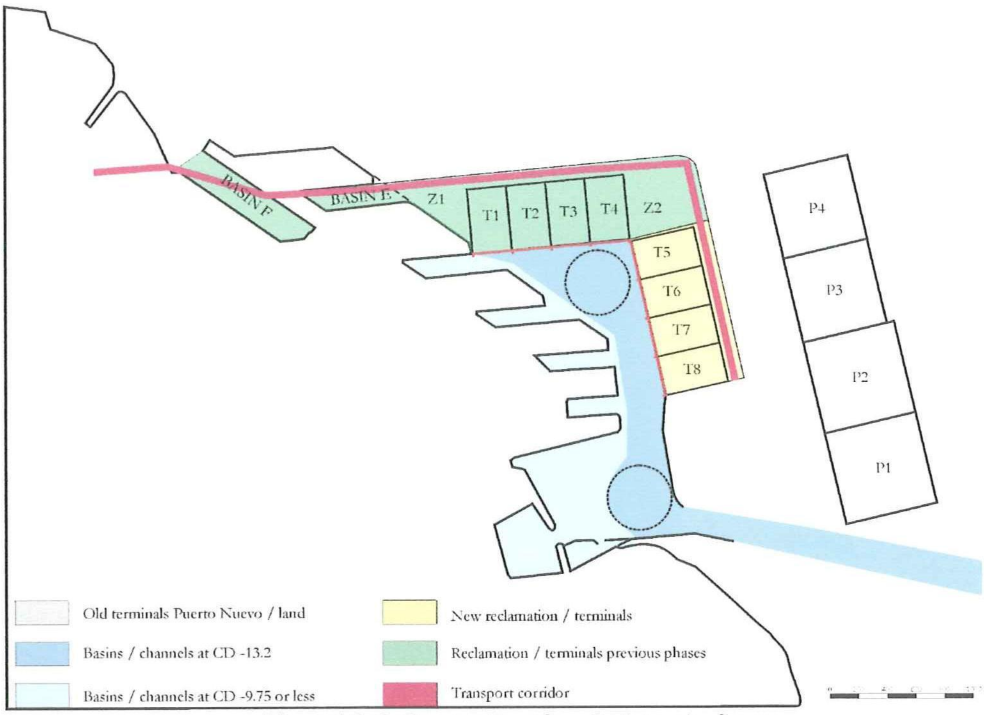


Figure 16–2 Construction phase 2: 4 terminals

The second phase consists of another four container terminals, situated at the port side of the peninsula, see Figure 16–2. The first terminal of this phase needs to be operational in 2011, the last terminal in 2015. The *tosca*, which has been deposited in the depot during the dredging of the first sand pit, has had the time to settle for a period of 9 to 13 years by then. Residual settlements are expected to be negligible after this period, because in the first construction phase the *tosca* layer is finished with a layer of sand and vertical drainage is placed. The container terminals can be constructed quite easily on the settled *tosca*. When the *tosca* layer in the depot does not have sufficient bearing capacity yet, the terminals on the eastern side of the peninsula can be constructed first. Otherwise, the *tosca* from the first sand pit has to be dumped at sea and only sand is used as reclamation material. (Extra costs: approximately 30 million US$)

16.2.3 Construction phase 3

In the last construction phase, 5 new terminals will be built on the eastern side of the peninsula, see Figure 16–3. The first terminal is required in 2016 and every year after that an additional terminal must be finished. An additional access channel must be dredged to the eastern side of the peninsula. The depleted sand pits P1 to P4 of the previous construction phases are positioned near the eastern berths, so the dredging volume of this access channel is not as large as calculated in previous chapters. The construction of phase 3 should start mid 2014 because the first terminal of this phase is required in 2016. The required sand is dredged from a fifth and maybe a sixth sand pit. (Total volume sand: approximately 6.5 million m^3)

Figure 16–3 Construction phase 3: 5 terminals and new access channel

16.3 CONCLUSIONS AND RECOMMENDATIONS

Conclusions:

- With the building time of phase 1, the construction works of the port expansion should start around 1-1-2002 in order to avoid capacity problems in the port. The urgency to act is apparent.
- The construction of phase 1 is completed around 1-1-2004, giving the port sufficient capacity up to 2010.
- The construction of near-shore sand pits at the location of the future eastern access channel introduces a construction time (and cost-) reduction of the port expansion.

Recommendations:

- Additional geotechnical information should be obtained to decide whether or not to use *tosca* in the reclamation area
- Special attention should be paid to the removal of the breakwaters. Both removal rate and -costs have to be determined to make a more detailed planning and cost estimate.

17. FEASIBILITY OF PORT EXPANSION

The feasibility of the port expansion project depends on 3 major factors:

1. Economic factors: a discounted cash flow must give more information about the financial feasibility. It determines the amount of risk and the return of investment.
2. Technical factors: the port expansion must not lead to unacceptable downtime of the terminals or waiting times of the ships calling at the port.
3. Environmental factors: economic growth must not be accompanied by destruction of environmental values.

17.1 FINANCIAL FEASIBILITY: INTRODUCTION

Although the costs of the port expansion are mostly related to the construction of new terminals, the financial feasibility of the project is determined for the whole port (old and new terminals). In practice, a central authority will be responsible for both the old and the new terminals: the construction and maintenance of all terminals and basins in Puerto Nuevo. Having a central authority prevents getting the problems of internal competition in the port, which would introduce an extra risk for investors in the new port.

To determine the financial feasibility of the port expansion a discounted cash flow calculation has been made. Cash flow (C.F.) is determined as:

C.F.= Revenues – Investments

The investments and revenues of the project are determined for each year over the estimated period of time of 20 years. Cash flow values in the future are not equal to current cash flow values and all future values must be corrected for time (discounting). For all future cash flows a discounted value (D.C.F. = discounted cash flow) is calculated using a constant discounting rate γ.

$$D.C.F._n = \frac{C.F._n}{(1+\gamma)^{n-1}}$$

in which:

n:	year from the start
$C.F._n$	cash flow value of year n
$D.C.F._n$	discounted cash flow value of year n

From this formula can be concluded that early revenues and investments are of most importance for the feasibility of the project and future investments and revenues are of less importance.
Adding up all discounted cash flow values of 20 years yields the *net present value* (N.P.V.) of the project.

$$N.P.V._{20} = \sum_{n=1}^{n=20} D.C.F_n$$

The IRR (Internal Rate of Return) is the most important standard for the return investment and is defined as the discounting percentage for which the net present value becomes zero.

$$\text{I.R.R.} = \gamma \therefore N.P.V_{20} = 0$$

17.2 CASH FLOW CALCULATION FOR ALTERNATIVE 3

17.2.1 Investments

The investments of the port expansion are determined in previous chapters and can be divided in two contributions:

- Capital construction works
- Maintenance of port infrastructure

The investments of construction phase 1 are determined in the previous chapter and can be found in appendix M. The construction costs of the second and third phase are estimated and have been stated in appendix N and Figure 17–1 below. The total required investment comes to: 671 million US$.

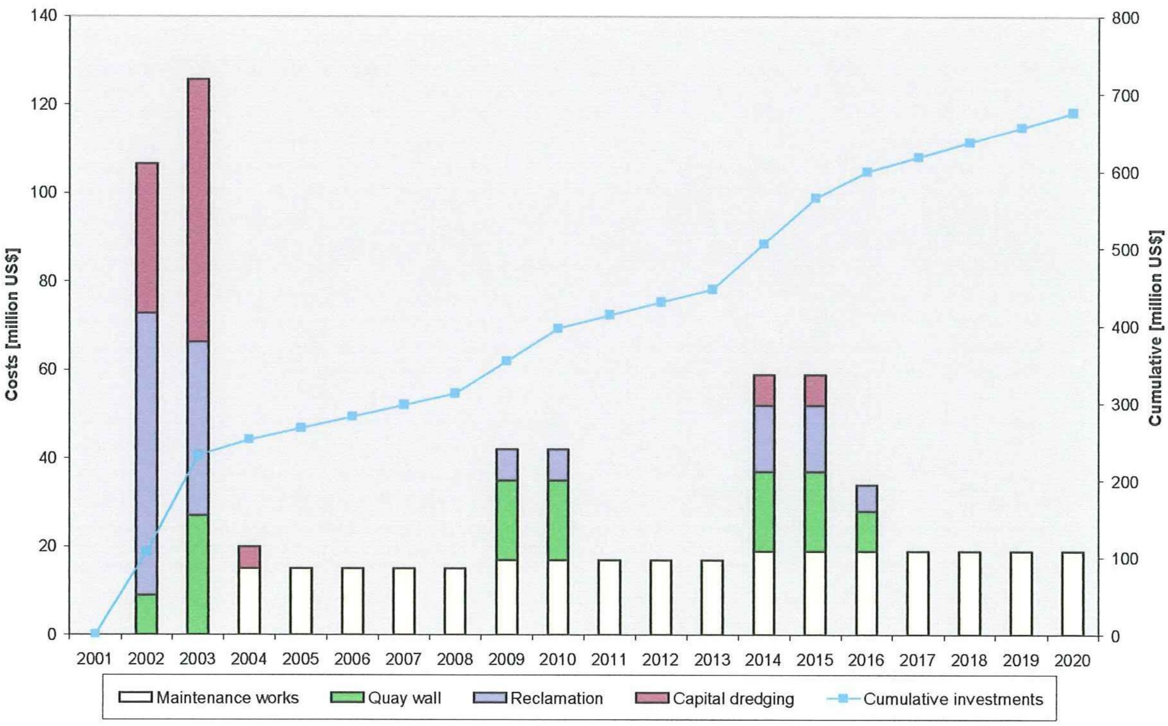


Figure 17–1 Maintenance works, capital works and cumulative investments

17.2.2 Revenues

The project revenues are divided into three main contributions, see Figure 17–2.

- Governmental contribution for the maintenance of the port
- Revenues from all port operations (both old and new terminals)
- Revenues from port charges (all ships in Puerto Nuevo)

Governmental contribution:
An amount of 15 million US$/year is the required budget for all of the concession years. This figure will account for the maintenance of the old and new port and together with the revenues from port operations, is sufficient for a financially feasible project.

Revenues from all port operations:
In the new port all terminals will be operational under the same terms. To estimate the expected revenues of port operations the present terms are adopted for all terminals. From the throughput of the medium scenario the revenues from all port operations are calculated. The present terms are:

Land lease: $ 1.00 / m^2 / month
Container charges: $ 3.00 / TEU

Revenues from port charges:
To estimate the revenues from port charges, the present toll duties of 0.30 US$ / NRT will be applied to all ships. An assumption regarding the size and number of ships, as was done in chapter 5 yields the revenues from these charges.

Port charges $0.30 / NRT

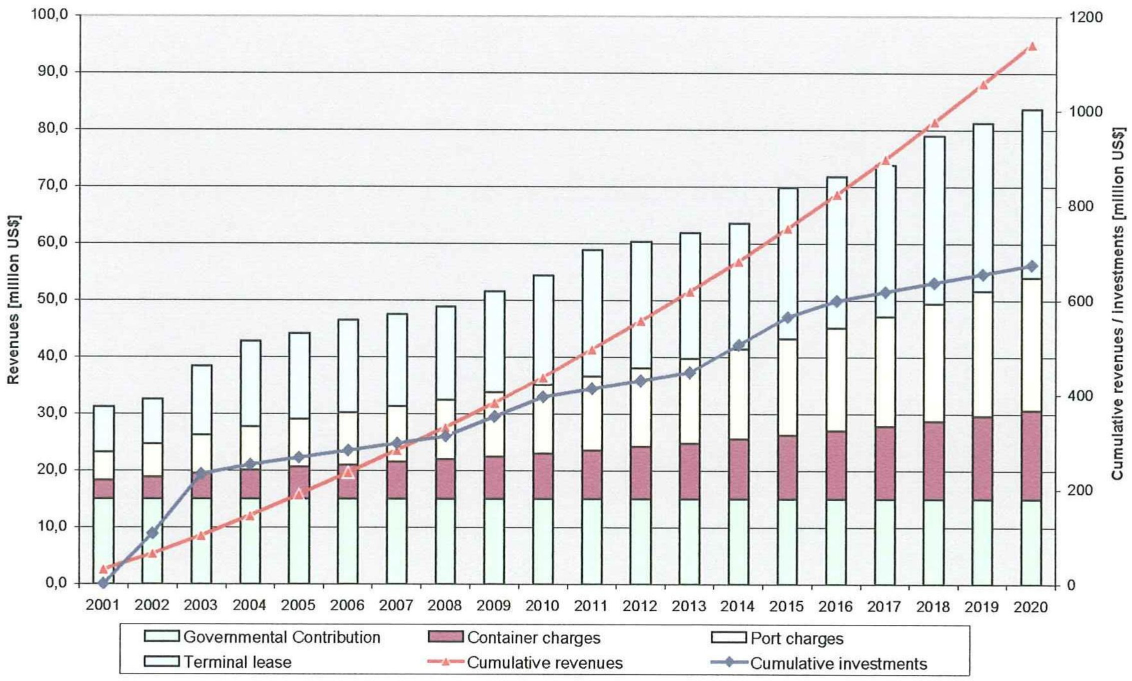


Figure 17–2 Revenues: governmental contribution, container charges, port charges and terminal lease

The cash flow calculation in Appendix N shows that the internal return rate (IRR) of the project is 21%. This is sufficient for a financially feasible project and around general accepted rates for large port expansion projects, as stated by the Worldbank and Drewry Shipping Consultants (± 18%).

17.3 ECONOMIC FEASIBILTY

Economic feasibility has many aspects. A macro-economic analysis of the impact of the port expansion, long-term strategy of Argentina's ports, the development of other ports and close cooperation with other ports have a strategic economic impact on the economy of Argentina. How important this strategic impact may be, these cannot be measured at this moment and are considered beyond the scope of this report.
In this report only growth scenarios have been developed and the position of the port in the container market has been analyzed. The conclusions concerning economic feasibility in this report will be concentrated on these aspects.
In chapters 2 and 3 has been determined that the port of Buenos Aires does not have sufficient capacity to meet the expected throughput quantities, caused by the economic growth of Argentina. Without an expansion of the port, around 2005 congestion will cause serious problems in the port and consequently shipping lines will search for other ports to deliver goods to Argentina. A port expansion is desirable as it gives Argentina the opportunity to grow without these kinds of problems. Besides that, all of the port related activities would create employment opportunities in and around the city of Buenos Aires.

17.4 TECHNICAL FEASIBILITY

The construction of the port expansion in the Rio de la Plata has some opportunities and some difficulties. The mild wave and wind climate makes it possible to construct exposed container berths and guarantees easy access conditions for the port and its new terminals. On the other hand, the position of the port of Buenos Aires and the depth of the Rio de la Plata requires the dredging of long access channels, sand is difficult to obtain and the old port operations will remain inefficient.
There are still some challenges for the port, for example: integrating existing and new terminals, regulation of the increasing traffic intensity and possible reuse of *tosca* in either the reclamation or the ecological reserve
The second and third part of the report have proven that is it possible to construct a port expansion in the Rio de la Plata, which has sufficient capacity and provides safe mooring for all sorts of ships. The project is therefore considered to be technical feasible.

17.5 ENVIRONMENTAL FEASIBILITY

The construction of a port expansion will have impact on the environment. The increased port activities will cause more pollution, proportional to the existing activities. The amount of pollution caused by container operations is not severe.
As stated in chapters 15 and 16, a large layer of soil cannot be used in the reclamation and will be transported either to an offshore dump location or the ecological reserve. However, the largest part is dumped into the depleted sand pits, not causing any additional pollution. When the dredge spoil is heavily polluted, it must undergo some sort of cleaning before it can be transported to the offshore dump / ecological reserve.
Any additional turbidity caused by the construction works will not have much impact on the environment of the Rio de la Plata, as the background level is already high.
After all, the impact on the environment is not considered to be really large and the port expansion is considered to be environmental feasible. A detailed study into environmental impact is recommended, but beyond the scope of this report.

18.CONCLUSIONS AND RECOMMENDATIONS

18.1 CONCLUSIONS

Main conclusion

An expansion of the port of Buenos Aires of 13 terminals is considered to be feasible in economical, environmental, financial and technical aspects.

Other conclusions

- Ignoring or neglecting the existing capacity problems in the port of Buenos Aires will cause serious problems for the fast increasing import and export of containerized goods, carried by ever growing container vessels. The effects for Buenos Aires and therefore for whole Argentina cannot be underestimated.
- The access channel and basins will have a dredged depth of at least CD –13.2 m
- No unacceptable downtime of the terminals is expected due to channel depth, waves, currents, berthed ship motions and traffic intensity, giving the terminals the opportunity to be operational more than 99% of the time
- Panamax container ships can berth safely at the exposed terminals on the eastern side of the peninsula without additional down-time for the terminals; no new breakwater has to be constructed for the protection of these berths. Smaller ships and feeders will berth at one of the container terminals in the sheltered port
- The construction of phase 1 in which four terminals will be built, needs to start around 1-1-2002 to avoid capacity problems in the port. After completion of this phase in 2004, the port has sufficient capacity up to 2010
- The construction of near-shore sand pits at the location of the future eastern access channel introduces a construction time (and cost-) reduction of the port expansion.

18.2 RECOMMENDATIONS:

- Additional geotechnical information should be obtained to decide whether or not to use *tosca* in the reclamation area
- Special attention should be paid to the removal of the breakwaters. Both removal rate and -costs have to be determined to make a more detailed planning and cost estimate
- The optimum quay length in relation to waiting times in the port should be determined with a traffic simulation of the port.
- A more detailed analysis of currents and flow velocities after the construction of the peninsula is highly recommended because this influences the currents near the berths, sedimentation patterns near the port (maintenance volumes) and ship manoeuvrability in the eastern access channel. Small adjustments to the design of an expansion can have large impacts.
- The effects of waves on the quay wall and the scour protection near the quay wall should be investigated, because large scour will affect the stability of the quay wall. If necessary, the scour protection must be adapted to the conclusions of this investigation
- The cooling water inlets and outlets of the power plants in Puerto Nuevo must be moved, because basin E is refilled. A design for this water supply and discharge has not been made in this report and requires special attention.

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Microsoft Encarta:	www.encarta.com
Webcosur:	www.webcosur.com

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Port of Bremerhaven:	www.bremen-ports.de
Port of Buenos Aires:	www.bairesport.gov.ar
Port of Hamburg:	www.hafen-hamburg.be
Port of Oakland	www.portofoakland.com
Port of Rotterdam:	www.port.rotterdam.nl
Port of Singapore:	www.mpa.gov.sg / www.psa.gov.sg
Port of Tanjung Pelapas:	www.ptp.com.my

Container terminals:

BACTSSA:	www.bactssa.com.ar
Eurogate:	www.eurokai.de
Exolgan Container terminal:	www.exolgan.com
Hessenatie (Antwerp)	www.hessenatie.be
TRP:	www.southamerica-business.com/trp
TPA:	www.tpa.com.ar
HHLA (Hamburg):	www.hhla.de

Stevedoring companies / ports

P&O Ports:	www.poports.com.au
Hutchison Port Holdings:	www.hph.com.hk
ICTSI :	www.ictsi.com

Shipping lines:

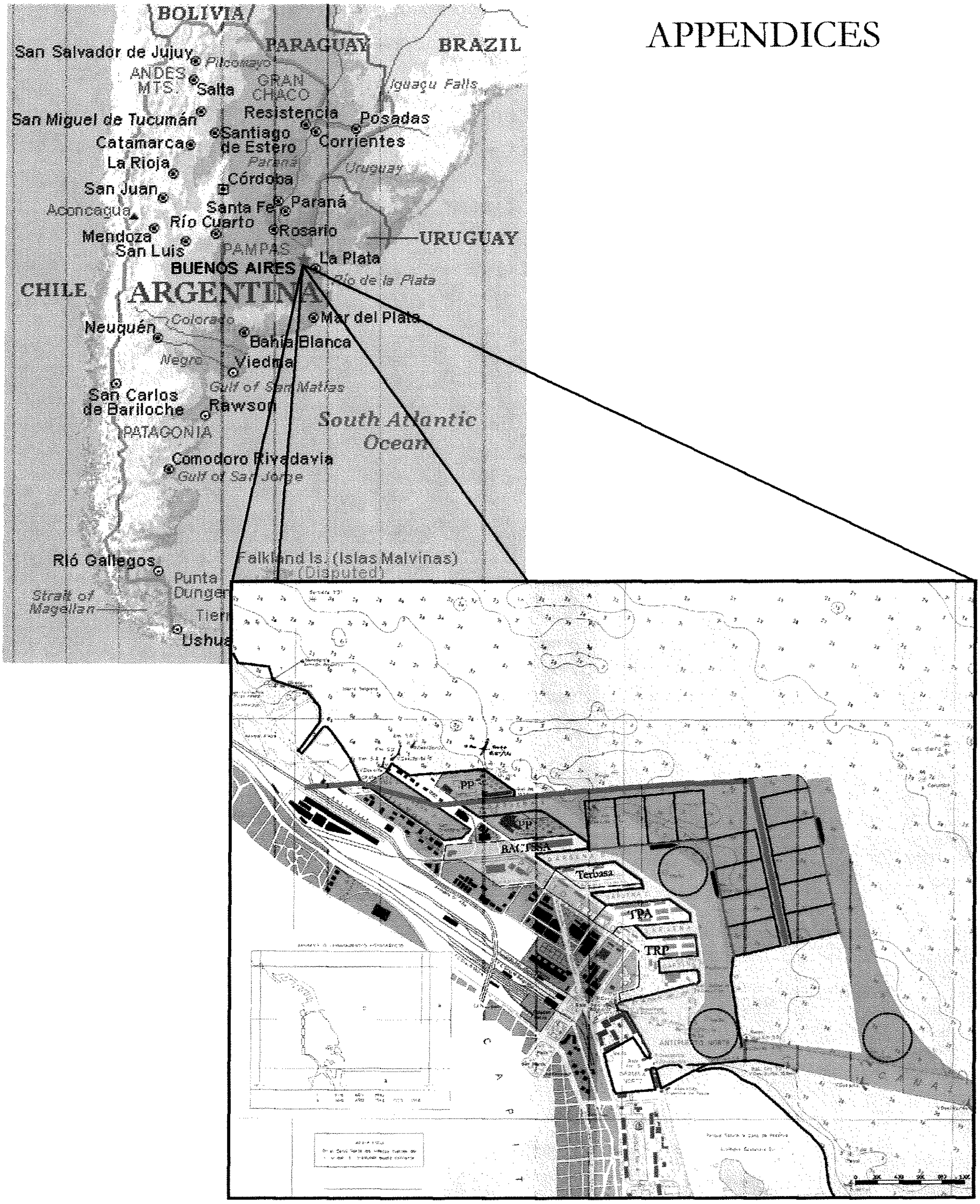
American President Lines:	www.apl.com
Evergreen:	www.evergreen-marine.com.tw
Maersk:	www.maersk.com / www.maerskline.com
Sealand:	www.sealand.com

Terminal Equipment:

Nelcon	www.nelcon.com
Liebherr	www.liebherr.com
Hyster	www.hyster.co.uk

BUENOS AIRES PORT EXPANSION 2020

APPENDICES



Appendices of final report, by R.D. van den Bosch

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TABLE OF APPENDICES

APPENDIX A. CONTAINER CARGO DATA

Appendix A.1 CONTAINER CARGO DATA FOR PUERTO NUEVO

	1995	1996	1997
Import	2,072,754	2,126,252	2,692,829
Export	1,967,102	2,027,000	2,248,997
Transit	78,341	54,763	161,812
Total	4,118,197	4,208,015	5,103,638

Table A–1 Container Throughput in Puerto Nuevo in tons, 1995-1997, Source: AGP

	1995	1996	1997	1998
Loaded	382,768	398,666	497,326	554,221
Empty	121,862	131,680	222,921	263,123
Total	504,630	530,346	720,247	818,334

Table A–2 Container throughput in Puerto Nuevo in TEU, 1995-1998, Source: AGP

Appendix A.2 REFERENCE PORTS

The data of the ports described below has been collected from:
Containerisation International Yearbook 1999 and from various Internet homepages of the ports, see literature.

Antwerp, Belgium

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
ACT (MPP)	6th	MC / RS (100)	441.000	1.865	6	15	19	-	-	-	-	236
	7th	N/A	145.500	752	3	6-15,0	2	-	-	-	-	193
Maritime	242-246	RS	100.000	720	3	11,5	3	114.702	159	11.470	38.234	139
Mexico Natie (MPP)	8-11	RS	70.000	600	4	-	4 + 1m	-	-	-	-	117
Noord Natie (MPP)	700-714	SC(23)+FL(38)	800.000	1.070	5	15,3	4 + 6q	368.000	344	4.600	73.600	748
Noordzee		SC(22)+RS(10)	550.000	1.124	5	14,5	5 + 1m	275.000	245	5.000	55.000	489
Seaport (MPP)	732-746	SC(24)	700.000	1.370	6	16,1	6 (4PP)	335.000	-	-	-	511
	1225-1231	N/A	200.000	1.360	6	18,1	2 + 1q	-	-	-	-	147
Westerlund (Ro-ro)	1207-1221	Roro	880.000	1.920	3	12	4	-	-	-	-	458
Hessenatie	716-730	SC (40)	695.000	1.220	5	15,5	6 (4 PP)	-	-	-	-	570
	851-869	SC (40)	664.000	1.180	5	14,3	8 (SPP)	1.828.817	762	13.457	182.882	563
Total			5.245.500	13.181	51			2.969.189	225	5.660	58.219	398
Average terminal depth			398	meter								
Average throughput per quay			58.219	TEU								

Bremen/Bremerhaven, Germany

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
BLG	Bremen (MPP)	SC (16), ro-ro	290.000	900	2	10-11,3	6 (5PP)	45.000	50	1.552	22.500	322,22
BLG- Bremerhaven	Eurogate	SC (72)	2.400.000	2.300	12	14	17 (7 PP)	1.655.000	720	.896	137.917	1.043,48
	Nordhafen		310.000	946	1	13	4	-	-	-	-	327,70
Others								3.219				
Total			3.000.000	4.146	15			1.703.219	411	5.677	113.548	
Average terminal depth			724	Meter								
Average throughput per quay			113.548	TEU								

Buenos Aires, Argentina

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
TRP	T 1& 2	RTG(11)+ RS.(19)	285.000	980	5	10,0	5	383.322	391	13.450	76.664	291
TPA (MPP / Ro-ro)	T 3	RTG (12)	160.000	1.345	5	10,0	2	94.623	70	5.914	18.925	119
BACTSSA	T 5	RTG (7)+Var(20)	215.000	885	4	9,45	3	242.302	274	11.270	60.575	243
Exolgan (Dock Sud)	Exolgan	RTG (12)	450.000	700	3	8-10,0	2	307.910	440	6.842	102.637	642
Total			1.110.000	3.910	17		14	1.028.157	263	9.263	60480	284
Average terminal depth			284	meter								
Average throughput per quay			60.480	TEU								

Busan, South Korea

Terminal Operator	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
Jasungdae Container Terminal	RMG (1), RTG (30). SC(14)	647.566	1.447	5	12,5	13 (3 PP)	1.808.146	1.250	27.922	361.629	448
Shinsundae Container Terminal	YGs (32)	1.038.534	1.200	4	14	11	1.452.036	1.210	13.982	363.009	865
Uam Container Terminal	RTG (10)	159.258	500	2	11	4 (PP)	345.636	691	21.703	172.818	319
Others	-	-	-	-	-	-	1.628.062	-	-	-	
Total		1.845.358	3.147	11		25	5.233.880				
Average terminal depth		586	meter								
Average throughput per quay		475.807	TEU								

Hamburg, Germany

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
HHLA	Buchardkai	SC (80x)	1.500.000	2.850	10	14,5	16 (7 PP)	1.544.000	542	10.293	154.400	526,32
	TCT	SC (30x)	320.000	920	4	13,5	6 (3 PP)	381.500	415	11.922	95.375	347,83
	Unikai	RTG / RMG	165.000	613	3	13,5	5	281.600	459	17.067	93.867	269,17
			1.985.000	4.383	17		27	2.207.100	504	11.119	129.829	452,89
EUROKAI	Eurokai	RMG (4)/ SC (45)	700.000	1.900	7	13,5-15,5	10 (5 PP)	908.044	478	12.972	129.721	368,42
Others (MPP)	Afrika	RS (11)	160.000	850	1	12	2	-	-	-	-	188,24
	HUG	RS (6)	150.000	690	3	11,6	2	-	-	-	-	217,39
Subtotal			310.000	1.540	4		4	222.333	144	7.172	55.583	201,30
Total			2.995.000	7.823	28			3.337.477	427	11.143	119.196	382,85
Average terminal depth			383	meter								
Average throughput per quay			119.196	TEU								

Manila, Philippines

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
MICT	Containers	RTG (23)	940.000	1.300	5	14,5	10	907.202	698	9.651	181.440	723
South Harbor	Cont/ GC	RTG (9)	850.000	2.210	8	10-14	5	487.244	220	5.732	60.906	385
North harbor	Pier 2	RS (10)	39.680	444	2	6	0	59.921	135	15.101	29.961	89
(feeders)	Pier 4	YG(2)	18.000	217	5	6	0	101.131	466	56.184	20.226	83
	Pier 6	N/A	18.000	217	5	6	0	8.986	41	4.992	1.797	83
	Pier 8	N/A	18.180	220	5	6	0	107.211	487	58.972	21.442	83
	Pier 10	N/A	17.845	220	5	6	0	99.982	454	56.028	19.996	82
	Pier 12	N/A	60.000	221	5	6	0	102.465	464	17.078	20.493	271
	Pier 14	N/A	17.856	222	5	6	0	115.002	518	64.405	23.000	80
	Pier 16	FL	25.452	250	4	6	0	-	-	-	-	102
	Marine	N/A	986.782	280	3	4	0	73.124	261	741	24.375	3524
Others								58.806				
Total			2.991.795	5.801	52		15	2.121.074	366	7.090	40.790	
Average terminal depth			516	meter								
Average throughput per quay			40.790	TEU								

Nagoya, Japan

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
Kinjo Pier	76-79	SC (18)	176.000	800	4	10,5	5	278.771	348	15.839	69.693	220
Tobishima Pier	90-92	N/A	170.000	620	3	10-12,0	4	243.048	392	14.297	81.016	274
	93-94	N/A	225.000	700	2	14-15	5	300.387	429	13.351	150.194	321
NCB Terminal	NCB	SC (69)	289.000	900	3	12	6	648.508	721	22.440	216.169	321
ITS	Nabeta	N/A	175.000	350	1	14	3	27.450	78	1.569	27.450	500
Total			1.035.000	3.370	13		23	1.498.164	445	14.475	115.243	307
Average terminal depth			307	Meter								
Average throughput per quay			115.243	TEU								

Oakland, United States of America

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal Depth
Sea Land	Sea Land	Chassis	265.000	699	3	12,2	4	330.000	472	12.453	110000	379
Yusen	Yusen	RTG (2) + RS (13)	162.000	274	1	12,2	2	135.000	493	8.333	135000	591
Maersk	Maersk	RS(5) + chassis	350.000	319	1	12,2	3	-	-	-	-	1097
Trans Bay	Trans Bay	RTG (10)	113.000	412	2	11,6	2	-	-	-	-	274
TraPac	TraPac	Chassis	135.000	335	1		2 (PP)	-	-	-	-	403
B&E	B&E 35/37	RMG/RTG (3)/ RS (13)	255.474	635	2	12,8	3	317.108	353	12.413	158.554	402
	B&E 38			263	1	12,2	-	-	-	-	-	0
Matson	Matson	SC (16) + RS (41)	265.000	467	2	10,7	3	-	-	-	-	567
APL	APL 60/61	RS (5)	305.000	440	2	11,3	3	-	-	-	-	693
	APL 62/63		-	390	2	11,6	3	-	-	-	-	
Charles P	Charles P	RTG (3) + RS (11)	198.000	522	2	12,2	4	-	-	-	-	379
Total			2.048.474	4.756	19			1.357.400	285	6.626	71.442	431
Average terminal depth			431	Meter								
Average throughput per quay			71.442	TEU								

Rotterdam, Netherlands

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
ECT Maasvlakte	Delta	SC (60x)	920.000	1.650	5	16	8	-	-	-	-	557,6
	Delta/Seal	RMG (28)	620.000	970	4	16,6	5	-	-	-	-	639,2
	DDE	RMG	650.000	1.030	4	16,6	7	-	-	-	-	631,1
Subtotal			2.190.000	3.650	13		27	2.752.500	754	12.568	211.731	600,0
ECT Home Terminal		SC (36)	750.000	1.700	7	10-13,0	12	1.023.000	602	13.640	146.143	441,2
Brittanie		Ro-ro	272.000	1.240	4	12,6	5	153.400	124	5.640	38.350	219,4
Uniport	Pier 7	RMG (5)	200.000	800	3	13	4	161.200	202	8.060	53.733	250,0
Handico	Handico	Front-end	43.000	100	1	6	1	59.800	598	13.907	59.800	430,0
HT Holland	HT Holland	RMG (1), RS	43.000	130	1	6,5	1	-	-	-	-	330,8
Hanno	Pier 5/6	N/A	220.000	1.500	6	13,5	4	258.384	172	11.745	43.064	146,7
Others			-	-	-	-	-	1.019.330	-	-	-	
Subtotal			1.528.000	5.470	22		27	2.675.114	489		121.596	279,3
Total			3.718.000	9.120	35		54	5.427.614	595	14.598	155.075	407,7
Average terminal depth			408	Meter								
Average throughput per quay			155.075	TEU								

Santos, Brazil

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
Tecon 1			366.000	510	2	13	-	297.000	582	8.115	148.500	718
Libra Group	T37	RTG (3), RS (10)	180.000	375	2	12	3	197.000	525	10.944	98.500	480
Others								335.486				
Total			546000	885	4	25	3	829.486	937	15.192	207.372	
Average terminal depth			617	Meter								
Average throughput per quay			207.372	TEU								

Shanghai, China

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
Shanghai -	Zhang Hua Bang	RTG (20)	303.000	783	3	12,5	6 (3 PP)	-	-	-	-	387,0
Container -	Jun Gong Lu	RTG (15x)	337.000	858	2	10,5	6	-	-	-	-	392,8
Terminals	Bao Shan	RTG (14) / RS (24)	218.000	640	3	9,4	4	1.766.000	774	20.583	220.750	340,6
Others	-	-	-	-	-	-	-	754.000	-	-	-	
Total			858.000	2.281	8			2.520.000	1.105	29.371	315.000	
Average terminal depth			376	Meter								
Average throughput per quay			315.000	TEU								

Yokohama, Japan

Terminal Operator	Terminal	Operation	Area (m2)	Quay Length [m]	Berths	Depth [m]	# cranes	TEU (1997)	TEU/m	TEU/ha	TEU / berth	Terminal depth
Honmuku Private User Terminals	a5-a6	N/A	221.000	600	2	13	3	-	-	-	-	368
	a7	N/A	81.000	250	1	13	2	-	-	-	-	324
	a8	RTG (5)	84.000	250	1	13	2	-	-	-	-	336
	d4	SC (13)	105.000	300	1	14	3	-	-	-	-	350
	d5	SC (10)	105.000	300	1	14	3	-	-	-	-	350
Subtotal			596.000	1.700	6		13	950.129	559	15.942	158.355	350
Honmuku Public-User Terminals	C5-D3	N/A	323.032	1.620	8	11-13	9	630.697	389	19.524	78.837	199
Daikoku Private User Terminals	c1-c4	RTG (16)	538.500	1.300	4	12-14	11	671.471	517	12.469	167.868	414
Daikoku Public User Terminals	1,2&9	N/A	129.706	720	2	12	5	54.392	76	4.193	27.196	180
Total			1.587.238	5.340	20		38	2.306.689	432	14.533	115.334	297
Average terminal depth			297	meter								
Average throughput per quay			115.334	TEU								

FL: Forklift truck
N/A: (Data) not available
PP: Post Panamax (containercrane)
RMG: Rail Mounted Gantry
RS: Reach-stacker
RTG: Rubber Tyres Gantry
SC: Straddle carrier
SPP: Super Post Panamax
YG: Yard Gantry

APPENDIX B. CARGO FORECAST DATA

Appendix B.1 ACTUAL THROUGHPUT FIGURES

Year	Argentina (in TEU)	Puerto Nuevo	Growth rate
1991	233,002	233,002	
1992	267,228	267,228	14.7%
1993	436,014	436,014	63.2%
1994	532,681	532,681	22.2%
1995	638,275	504,630	19.8%
1996	779,554	530,346	22.1%
1997	1,028,157	720,247	31.9%
1998	1,137,620	818,334	10.6%
Note: Figures for Argentina are figures for Puerto Nuevo and Dock Sur, other ports have negligible throughputs			

Table B–1 Container throughput in Argentina 1991-1998, Source AGP

Appendix B.2 ECONOMIC DEVELOPMENT: LOW SCENARIO

Region	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	1998-2005 growth
North America	22,957	24,485	25,493	26,417	27,373	28,321	29,293	30,281	31,283	32,306	7,821
Western Europe	34,659	38,575	41,549	44,483	47,358	50,188	53,109	56,103	59,148	62,261	23,686
Far East	44,571	47,839	49,316	51,896	54,664	57,644	60,673	63,764	66,966	70,249	22,410
South East Asia	23,260	25,595	26,549	28,247	30,481	33,258	36,394	39,683	43,128	46,741	21,146
Middle East	7,392	8,004	8,546	9,258	10,066	10,890	11,670	12,473	13,292	14,136	6,132
South America	10,331	11,478	12,524	13,628	14,706	15,844	17,008	18,202	19,436	20,705	9,227
Australasia	3,592	3,794	3,939	4,096	4,255	4,408	4,565	4,728	4,888	5,055	1,261
South Asia	3,805	4,311	4,746	5,245	5,779	6,380	7,037	7,736	8,467	9,228	4,917
Africa	5,069	5,301	5,699	6,090	6,466	6,898	7,321	7,755	8,212	8,684	3,383
Eastern Europe	807	933	1,046	1,165	1,288	1,410	1,536	1,664	1,793	1,926	993
Total	156,443	170,315	179,408	190,525	202,436	215,241	228,606	242,389	256,613	271,291	
Additional TEU			9,093	11,117	11,911	12,805	13,365	13,783	14,224	14,678	100,976

Table B–2 Forecast global container port throughput (in '000 TEU), source: Drewry Shipping Consultants Ltd

Dividing the absolute values of each region in Table B–2 by the total world volume, the regional percentages are derived. In this way can be seen which regions have larger growth rates, because their percentage of the world volume is increasing see Table B–3

Region	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005
North America	14.7%	14.4%	14.2%	13.9%	13.5%	13.2%	12.8%	12.5%	12.2%	11.9%
Western Europe	22.2%	22.6%	23.2%	23.3%	23.4%	23.3%	23.2%	23.1%	23.0%	23.0%
Far East	28.5%	28.1%	27.5%	27.2%	27.0%	26.8%	26.5%	26.3%	26.1%	25.9%
South East Asia	14.9%	15.0%	14.8%	14.8%	15.1%	15.5%	15.9%	16.4%	16.8%	17.2%
Middle East	4.7%	4.7%	4.8%	4.9%	5.0%	5.1%	5.1%	5.1%	5.2%	5.2%
South America	6.6%	6.7%	7.0%	7.2%	7.3%	7.4%	7.4%	7.5%	7.6%	7.6%
Australasia	2.3%	2.2%	2.2%	2.2%	2.1%	2.0%	2.0%	2.0%	1.9%	1.9%
South Asia	2.4%	2.5%	2.6%	2.8%	2.9%	3.0%	3.1%	3.2%	3.3%	2.4%
Africa	3.2%	3.1%	3.2%	3.2%	3.2%	3.2%	3.2%	3.2%	3.2%	3.2%
Eastern Europe	0.5%	0.6%	0.6%	0.6%	0.6%	0.7%	0.7%	0.7%	0.7%	0.7%
Total	100 %	100 %	100 %	100 %	100 %	100 %	100 %	100 %	100 %	100 %

Table B–3 Forecast global container port throughput (in percentages of world volume), source: Drewry Shipping Consultants Ltd

Region	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	1998-2005 growth
North America		6.66%	4.12%	3.62%	3.62%	3.46%	3.43%	3.37%	3.31%	3.27%	31.9%
Western Europe		11.30%	7.71%	7.06%	6.46%	5.98%	5.82%	5.64%	5.43%	5.26%	61.4%
Far East		7.33%	3.09%	5.23%	5.33%	5.45%	5.25%	5.09%	5.02%	4.90%	46.8%
South East Asia		10.04%	3.73%	6.40%	7.91%	9.11%	9.43%	9.04%	8.68%	8.38%	82.6%
Middle East		8.28%	6.77%	8.33%	8.73%	8.19%	7.16%	6.88%	6.57%	6.35%	76.6%
South America		11.10%	9.11%	8.82%	7.91%	7.74%	7.35%	7.02%	6.78%	6.53%	80.4%
Australasia		5.62%	3.82%	3.99%	3.88%	3.60%	3.56%	3.57%	3.38%	3.42%	33.2%
South Asia		13.30%	10.09%	10.51%	10.18%	10.40%	10.30%	9.93%	9.45%	8.99%	114.1%
Africa		4.58%	7.51%	6.86%	6.17%	6.68%	6.13%	5.93%	5.89%	5.75%	63.8%
Eastern Europe		15.61%	12.11%	11.38%	10.56%	9.47%	8.94%	8.33%	7.75%	7.42%	106.4%
World growth		8.87%	5.34%	6.20%	6.25%	6.33%	6.21%	6.03%	5.87%	5.72%	59.3%

Table B–4 Forecast global container port throughput growth rates, source: Drewry Shipping Consultants Ltd

As can be seen from above figures, the growth of South America is higher than the global average. In this scenario the global growth rate is assumed to be 5.72 percent per annum after 2006. As the growth rate of South America has been larger than the global average, its percentage of global throughput is getting larger as well. This percentage is assumed to reach 8.00 percent in 2020, which results in an annual growth of about 6.05 percent. Summarising and extrapolating the figures for South America yields:

Year	World throughput (in '000 TEU)	Growth rate	South America (in '000 TEU)	Percentage of global throughput	Growth rate
1996	156,443		10,331	6.60%	
1997	170,315	8.87%	11,478	6.74%	11.10%
1998	179.408	5.34%	12,524	6.98%	9.11%
1999	190,525	6.20%	13,628	7.15%	8.82%
2000	202,436	6.25%	14,706	7.26%	7.91%
2001	215,241	6.33%	15,844	7.36%	7.74%
2002	228,606	6.21%	17,008	7.44%	7.35%
2003	242,389	6.03%	18,202	7.51%	7.02%
2004	256,613	5.87%	19,436	7.57%	6.78%
2005	271,291	5.72%	20,705	7.63%	6.53%
2006	286,809	5.72%	21,960	7.66%	6.06%
2007	303,214	5.72%	23,291	7.68%	6.06%
2008	320,558	5.72%	24,703	7.71%	6.06%
2009	338,894	5.72%	26,199	7.73%	6.06%
2010	358,279	5.72%	27,786	7.76%	6.06%
2011	378,772	5.72%	29,469	7.78%	6.06%
2012	400,438	5.72%	31,254	7.80%	6.06%
2013	423,343	5.72%	33,146	7.83%	6.05%
2014	447,558	5.72%	35,153	7.85%	6.05%
2015	473,159	5.72%	37,280	7.88%	6.05%
2016	500,223	5.72%	39,536	7.90%	6.05%
2017	528,836	5.72%	41,928	7.93%	6.05%
2018	559,086	5.72%	44,465	7.95%	6.05%
2019	591,065	5.72%	47,154	7.98%	6.05%
2020	624,874	5.72%	50,006	8.00%	6.05%

Note: South America figures include Caribbean and Central America

Table B–5 Forecast South American container throughput, low scenario

According to the OKITA II [JICA 1996], Argentina will handle about 8.19 percent of the South American container traffic, but 1998 figures have proven that Argentine ports already handle about 9.10% of the region's throughput. Maintaining this percentage gives for this scenario:

Year	Argentina (in TEU)	Percentage of South American throughput
1998	1,137,620	9.08%
1999	1,240,148	9.10%
2000	1,338,246	9.10%
2001	1,441,804	9.10%
2002	1,547,728	9.10%
2003	1,656,382	9.10%
2004	1,768,676	9.10%
2005	1,884,155	9.10%
2006	1,998,375	9,10%
2007	2,119,498	9.10%
2008	2,247,938	9.10%
2009	2,384,138	9.10%
2010	2,528,563	9.10%
2011	2,681,711	9.10%
2012	2,844,105	9.10%
2013	3,016,304	9.10%
2014	3,198,896	9.10%
2015	3,392,508	9.10%
2016	3,597,803	9.10%
2017	3,815,484	9.10%
2018	4,046,296	9.10%
2019	4,291,030	9.10%
2020	4,550,522	9.10%

Table B–6 Forecast Argentine container throughput, low scenario

Appendix B.3 ECONOMIC DEVELOPMENT: MEDIUM SCENARIO

Year	Argentina (in TEU)	Growth rate
1998	1,137,620	
1999	1,274,134	12.0%
2000	1,427,031	12.0%
2001	1,598,274	12.0%
2002	1,790,067	12.0%
2003	2,004,875	12.0%
2004	2,205,363	10.0%
2005	2,381,792	8.0%
2006	2,524,699	6.0%
2007	2,676,181	6.0%
2008	2,836,752	6.0%
2009	3,006,957	6.0%
2010	3,187,375	6.0%
2011	3,378,617	6.0%
2012	3,581,334	6.0%
2013	3,796,214	6.0%
2014	4,023,987	6.0%
2015	4,265,426	6.0%
2016	4,521,352	6.0%
2017	4,792,633	6.0%
2018	5,080,191	6.0%
2019	5,385,002	6.0%
2020	5,708,102	6.0%

Table B–7 Forecast Argentine container throughput, medium scenario

Appendix B.4 ECONOMIC DEVELOPMENT: HIGH SCENARIO

in million TEU	1994	1995	1996	1997	1998	1999	2000	2005-H	2005-L	2010-H	2010-L
East Asia	**54.08**	**61384**	**69.98**	**78.34**	**89.23**	**100.05**	**111.49**	**178.82**	**161.58**	**244.01**	**201.67**
Europe	30.28	33.06	34.74	37.00	39.46	41.90	44.75	62.33	60.09	86.97	80.85
North America	20.31	21.85	22.86	24.10	25.41	26.79	28.26	36.82	35.11	48.26	43.80
Australasia/Oceania	3.20	3.46	3.67	3.88	4.08	4.28	4.50	5.75	5.64	7.35	7.22
Developed markets	**53.79**	**58.38**	**61.23**	**64.97**	**68.94**	**72.97**	**77.51**	**104.89**	**100.84**	**142.57**	**131.87**
Caribbean/C. America	5.05	5.39	5.83	6.40	7.03	7.72	8.48	13.48	12.32	21.44	17.86
South America	2.54	2.76	3.07	3.42	3.80	4.23	4.71	7.76	6.76	12.79	9.71
Middle East / Indian Subcontinent	8.02	8.58	9.31	10.16	11.04	11.97	12.94	18.81	15.79	27.40	19.40
Africa	4.06	4.66	5.14	5.61	6.11	6.60	7.13	10.97	8.67	16.88	10.55
Developing markets	**19.66**	**21.38**	**23.35**	**25.58**	**27.98**	**30.53**	**33.27**	**51.03**	**43.54**	**78.51**	**57.53**
Total	**127.53**	**141.60**	**154.57**	**168.89**	**186.15**	**203.56**	**222.26**	**334.74**	**305.96**	**465.10**	**391.06**

Table B–8 World Regions: Forecast container port throughput to 2010, source: Ocean Shipping Consultants Ltd

	1994	1995	1996	1997	1998	1999	2000	2005-H	2005-L	2010-H	2010-L
East Asia	**42.4**	**43.7**	**45.3**	**46.4**	**47.9**	**49.2**	**50.2**	**53.4**	**52.8**	**52.5**	**51.6**
Europe	23.7	23.3	22.5	21.9	21.2	20.6	20.1	18.6	19.6	18.7	20.7
North America	15.9	15.4	14.8	14.3	13.7	13.2	12.7	11.0	11.5	10.4	11.2
Australasia/Oceania	2.4	2.5	2.4	2.3	2.2	2.1	2.0	1.7	1.8	1.6	1.8
Developed markets	**42.2**	**41.2**	**39.6**	**38.5**	**37.0**	**35.9**	**34.9**	**31.3**	**33.0**	**30.7**	**33.7**
Caribbean/C. America	4.0	3.8	3.8	3.8	3.8	3.8	3.8	4.0	4.0	4.6	4.6
South America	2.0	2.0	2.0	2.0	2.0	2.1	2.1	2.3	2.2	2.7	2.5
Middle East / Indian Subcontinent	6.3	6.1	6.0	6.0	5.9	5.9	5.8	5.6	5.2	5.9	5.0
Africa	3.2	3.3	3.3	3.3	3.3	3.2	3.2	3.3	2.8	3.6	2.7
Developing markets	**15.4**	**15.1**	**15.1**	**15.1**	**15.0**	**15.0**	**15.0**	**15.2**	**14.2**	**16.9**	**14.7**
Total	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**

Table B–9 World Regions: Forecast container port throughput to 2010 (in percentages of world volume), source: Ocean Shipping Consultants Ltd

in million TEU	1994	1995	1996	1997	1998	1999	2000	2005-H	2005-L	2010-H	2010-L
Caribbean	2.96	3.15	3.44	3.75	4.08	4.45	4.85	7.46	7.00	11.48	10.09
Central America, Atlantic	1.88	2.03	2.16	2.37	2.60	2.86	3.14	5.03	4.49	8.07	6.41
Central America, Pacific	0.21	0.21	0.24	0.29	0.34	0.41	0.49	0.98	0.83	1.89	1.36
Caribbean / Central America	**5.05**	**5.39**	**5.83**	**6.40**	**7.03**	**7.72**	**8.48**	**13.48**	**12.32**	**21.44**	**17.86**
South America, Atlantic	1.53	1.69	1.86	2.06	2.30	2.55	2.84	4.65	3.99	7.61	5.61
South America, Pacific	1.01	1.07	1.21	1.35	1.51	1.68	1.88	3.12	2.77	5.18	4.10
South America	**2.54**	**2.76**	**3.07**	**3.42**	**3.80**	**4.23**	**4.71**	**7.76**	**6.76**	**12.79**	**9.71**
Total	**7.59**	**8.15**	**8.90**	**9.82**	**10.83**	**11.95**	**13.19**	**21.24**	**19.08**	**34.23**	**27.57**

Table B–10 Caribbean, Central & South America: Forecast container port throughput to 2010 (in million TEU), source: Ocean Shipping Consultants Ltd

	1994	1995	1996	1997	1998	1999	2000	2005-H	2005-L	2010-H	2010-L
Caribbean		6.42%	9.21%	9.01%	8.80%	9.07%	8.99%	8.99%	7.61%	9.00%	7.59%
Central America, Atlantic		7.98%	6.40%	9.72%	9.70%	10.00%	9.79%	9.88%	7.41%	9.92%	7.38%
Central America, Pacific		0.00%	14.29%	20.83%	17.24%	20.59%	19.51%	14.87%	11.12%	14.04%	10.38%
Caribbean / Central America		**6.73%**	**8.16%**	**9.78%**	**9.84%**	**9.82%**	**9.84%**	**9.71%**	**7.76%**	**9.73%**	**7.71%**
South America, Atlantic		10.46%	10.06%	10.75%	11.65%	10.87%	11.37%	10.36%	7.04%	10.35%	7.05%
South America, Pacific		5.94%	13.08%	11.57%	11.85%	11.26%	11.90%	10.66%	8.06%	10.67%	8.16%
South America		**8.66%**	**11.23%**	**11.40%**	**11.11%**	**11.32%**	**11.35%**	**10.50%**	**7.49%**	**10.51%**	**7.51%**
Total		**7.38%**	**9.20%**	**10.34%**	**10.29%**	**10.34%**	**10.38%**	**10.00%**	**7.66%**	**10.01%**	**7.64%**

Table B–11 Caribbean, Central & South America: Average growth rates 1994-2010, source: Ocean Shipping Consultants Ltd

Year	Throughput (in TEU)	Growth rate
1998	1,137,620	
1999	1,257,070	10.5%
2000	1,389,062	10.5%
2001	1,534,914	10.5%
2002	1,696,080	10.5%
2003	1,874,168	10.5%
2004	2,070,956	10.5%
2005	2,288,406	10.5%
2006	2,528,689	10.5%
2007	2,794,201	10.5%
2008	3,087,593	10.5%
2009	3,411,790	10.5%
2010	3,684,733	8.0%
2011	3,905,817	6.0%
2012	4,140,166	6.0%
2013	4,388,576	6.0%
2014	4,651,891	6.0%
2015	4,931,004	6.0%
2016	5,226,864	6.0%
2017	5,540,476	6.0%
2018	5,872,905	6.0%
2019	6,225,279	6.0%
2020	6,598,796	6.0%

Table B–12 Forecast Argentine container throughput, high scenario

APPENDIX C. NATURAL BOUNDARY CONDITIONS

Appendix C.1 CORRECTION ON LEVEL OF RIO DE LA PLATA AS A FUNCTION OF WIND SPEED AND DIRECTION

	wind speed in km / h								
direction	10	20	30	40	50	60	70	80	Effect
N	-18.00	-30.00	-39.00	-48.00	-60.00	-72.00	-84.00	-96.00	decrease
NNE	-9.00	-21.00	-30.00	-36.00	-45.00	-57.00	-69.00	-81.00	decrease
NE	-3.00	-6.00	-9.00	-24.00	-39.00	-57.00	-69.00	-81.00	decrease
ENE	3.00	6.00	9.00	12.00	18.00	33.00	42.00	51.00	increase
E	6.00	12.00	24.00	36.00	48.00	60.00	72.00	84.00	increase
ESE	9.00	24.00	339.00	57.00	69.00	81.00	93.00	105.00	increase
SE	21.00	39.00	57.00	72.00	87.00	102.00	114.00	126.00	increase
SSE	30.00	45.00	66.00	81.00	96.00	114.00	132.00	150.00	increase
S	18.00	36.00	57.00	66.00	76.00	99.00	108.00	120.00	increase
SSW	12.00	24.00	0.90	48.00	57.00	69.00	78.00	87.00	increase
SW	-9.00	-12.00	-18.00	-24.00	-33.00	-52.00	-51.00	-60.00	decrease
WSW	-6.00	-12.00	-18.00	-30.00	-39.00	-51.00	-60.00	-69.00	decrease
W	-9.00	-15.00	-27.00	-36.00	-45.00	-54.00	-63.00	-72.00	decrease
WNW	-12.00	-21.00	-30.00	-36.00	-45.00	-54.00	-66.00	-78.00	decrease
NW	-12.00	-21.00	-30.00	-39.00	-48.00	-60.00	-69.00	-78.00	decrease
NNW	-15.00	-24.00	-36.00	-45.00	-54.00	-63.00	-75.00	-87.00	decrease

Table C–1 Correction of Rio de la Plata level in cm as function of wind direction and wind speed, Source: Naval Hydrographic Survey

Appendix C.2 DISTRIBUTION OF WIND SPEED VERSUS WIND DIRECTION

			Wind speed										
Direction	km/h	Calm	1-5	6-10	11-15	16-20	21-25	26-30	31-35	36-40	41-45	>45	Total
	m/s		<1.7	1.7-	3.1-	4.4-	5.8-	7.2	8.6-	10.0-	11.4-	>12.5	
	knots		0-3	4-6	6-9	9-12	12-14	14-17	17-20	20-22	22-25	>25	
	Bft		0-1	2	2-3	3	4	4-5	5	5-6	6	6+	
		6.62											6.62
N			0.52	0.49	2.00	1.30	0.57	0.47	0.08	0.10	0.01	0.01	5.55
NNE			0.59	1.37	2.68	1.36	0.90	0.75	0.17	0.17	0.01	0.03	8.03
NE			0.98	1.89	2.27	1.49	0.55	0.59	0.15	0.13		0.01	8.06
ENE			0.82	1.02	1.76	0.54	0.36	0.29	0.07	0.02	0.01		4.88
E			1.11	2.26	2.48	1.77	0.63	0.56	0.19	0.13	0.01	0.01	9.13
ESE			0.56	1.25	2.37	1.69	0.95	0.95	0.38	0.26	0.06	0.05	8.52
SE			0.45	1.16	2.22	1.73	1.06	1.04	0.31	0.31		0.01	8.30
SSE			0.20	0.65	1.26	0.88	0.70	0.82	0.47	0.41	0.06	0.08	5.54
S			0.46	0.84	1.23	1.06	0.61	0.79	0.37	0.52	0.02	0.07	5.96
SSW			0.38	0.70	1.36	0.86	0.68	0.59	0.37	0.30	0.03	0.09	5.36
SW			0.59	0.81	1.07	0.95	0.46	0.57	0.29	0.22	0.02	0.03	5.00
WSW			0.33	0.42	0.82	0.38	0.46	0.40	0.20	0.17	0.03	0.05	3.23
W			0.56	0.65	0.85	0.71	0.31	0.33	0.08	0.10		0.04	3.63
WNW			0.34	0.49	0.86	0.42	0.31	0.26	0.10	0.05		0.01	2.84
NW			0.30	0.75	1.44	1.05	0.54	0.29	0.14	0.06	0.01	0.01	4.57
NNW			0.33	0.63	1.50	1.07	0.63	0.45	0.11	0.05		0.01	4.78
Total		6.62	8.51	15.38	26.15	17.24	9.71	9.15	3.47	3.00	0.26	0.51	100.00
Total accumulated		6.62	15.13	30.51	56.66	73.90	83.61	92.76	96.23	99.23	99.49	100.00	

Table C–2 Distribution of wind speed versus wind direction, Estacion Aeroparque 1981-1990.

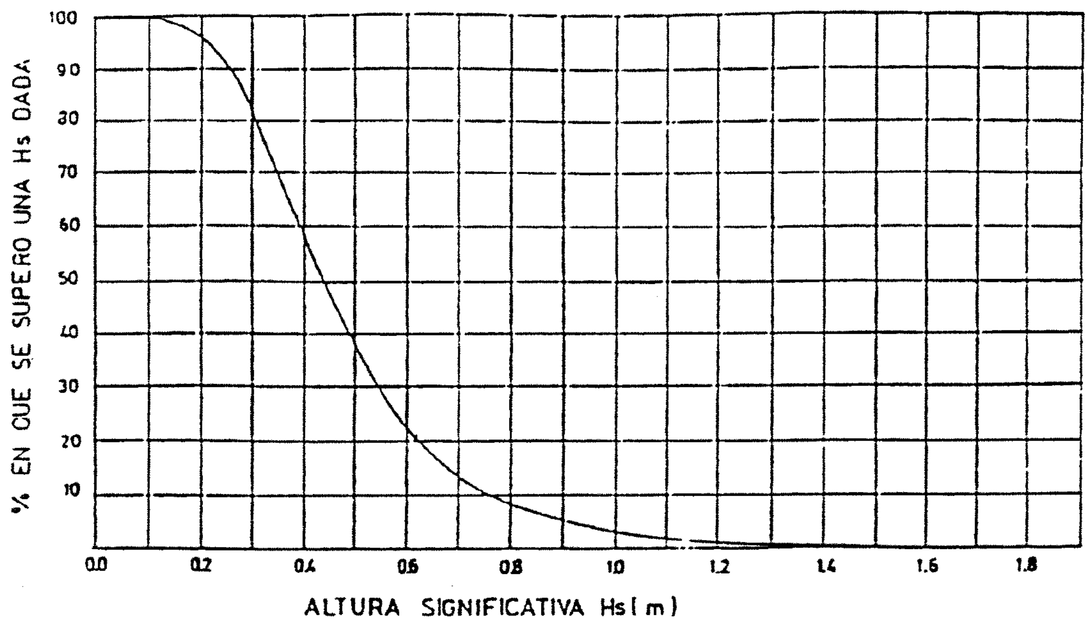
Appendix C.3 DISTRIBUTION OF WAVE HEIGHT AND ZERO CROSSING PERIODS

Source: Serman & Asociados S.A. – Consultura

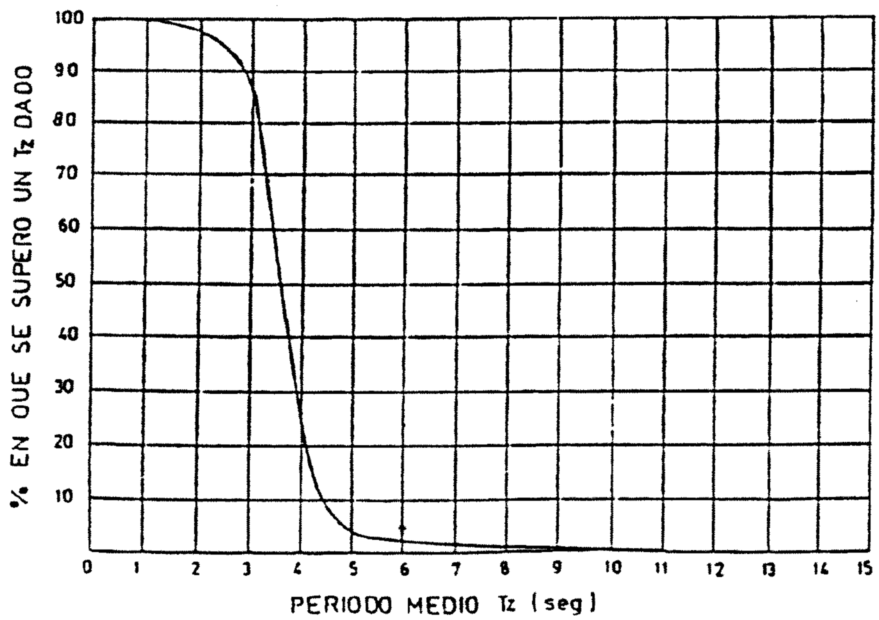
Statistical Analysis of waves near Buenos Aires,

Wave buoy at approximately 3 km off the coast.
Measuring period: August 1985 – February 1986

a) Altura Significativa de Ola



b) Período de Cruce de Cero



Ballast Nedam Dredging

Wave statistics, point I, Km 14.

Source: CEDEPOMAR – Estudio de Antecedentes del Canal Ing. Emilio Mitre

Hs[m]	N			NE			E			SE		
	Ts[s]	% Rel	% Exc.	Ts[s]	% Rel	% Exc.	Ts[s]	% Rel	% Exc.	Ts[s]	% Rel	% Exc.
0.1-0.2	1.3-1.8	2.25	13.82	1.3-1.8	2.72	12.99	1.3-1.8	1.93	12.55	1.3-1.8	0.44	12.84
0.2-0.4	1.8-2.5	3.99	11.57	1.8-2.5	3.60	10.27	1.8-2.5	3.22	10.62	1.8-2.4	1.65	12.40
0.4-0.6	2.5-3.1	5.08	7.58	2.5-3.1	2.96	6.67	2.5-3.1	2.29	7.40	2.4-3.0	1.65	10.75
0.6-0.8	3.1-3.8	2.36	2.50	3.1-3.7	2.53	3.71	3.1-3.6	2.59	5.11	3.0-3.6	2.30	9.10
0.8-1.0	3.8-4.2	0.14	0.14	3.7-4.2	1.07	1.18	3.6-4.1	1.89	2.52	3.6-4.1	2.97	6.80
1.0-1.2				4.2-4.6	0.11	0.11	4.1-4.7	0.60	0.63	4.1-4.6	2.90	3.83
1.2-1.4							4.7-5.1	0.03	0.03	4.6-5.2	0.78	0.93
1.4-1.6										5.2-5.7	0.14	0.15
1.6-1.8										5.7-6.1	0.01	0.01
Mean Hs	0.42			0.45			0.53			0.79		
Mean Ts	2.50			2.60			2.80			3.50		
% ad.	13.82			12.99			12.55			12.84		

Hs[m]	S			SW			W			NW			All directions (360°)		
	Ts[s]	% Rel	% Exc.	Ts[s]	% Rel	% Exc.	Ts[s]	% Rel	% Exc.	Ts[s]	% Rel	% Exc.	Ts[s]	% Rel	% Exc.
0.1-0.2	1.3-1.7	0.80	10.41	1.3-1.7	1.11	8.55	1.3-1.7	0.73	5.40	1.2-4.8	0.76	8.13	1.3-1.8	10.74	84.69
0.2-0.4	1.7-2.4	1.60	9.61	1.7-2.4	1.89	7.44	1.7-2.4	1.43	4.67	1.8-2.5	2.32	7.37	1.8-2.5	19.70	73.95
0.4-0.6	2.4-2.9	2.53	8.01	2.4-2.9	2.37	5.55	2.4-3.0	1.40	3.24	2.5-3.1	2.41	5.05	2.5-3.0	20.69	54.25
0.6-0.8	2.6-3.4	2.97	5.48	2.9-3.4	2.19	2.18	3.0-3.5	1.19	1.84	3.1-3.8	2.20	2.64	3.0-3.6	18.33	33.56
0.8-1.0	3.4-3.9	1.88	2.51	3.4-3.8	0.80	0.99	3.4-3.9	0.60	0.65	3.8-4.3	0.42	0.44	3.6-4.1	9.77	15.23
1.0-1.2	3.9-4.3	0.55	0.63	3.8-4.3	0.17	0.19	3.9-4.6	0.05	0.05	4.3.-4.8	0.02	0.02	4.1-4.6	4.40	5.46
1.2-1.4	4.3-4.7	0.07	0.08	4.3-4.7	0.02	0.02							4.6-5.1	0.90	1.06
1.4-1.6	4.7-5.0	0.01	0.01										5.1-5.7	0.15	0.16
1.6-1.8													5.7-6.1	0.01	0.01
Mean Hs	0.61			0.51			0.49			0.49			0.54		
Mean Ts	2.90			2.60			2.60			2.70			2.80		
% ad.	10.41			8.55			5.40			8.13			84.69		

Appendix C.4 WATER LEVELS AND FLOW VELOCITY IN ACCESS CHANNEL

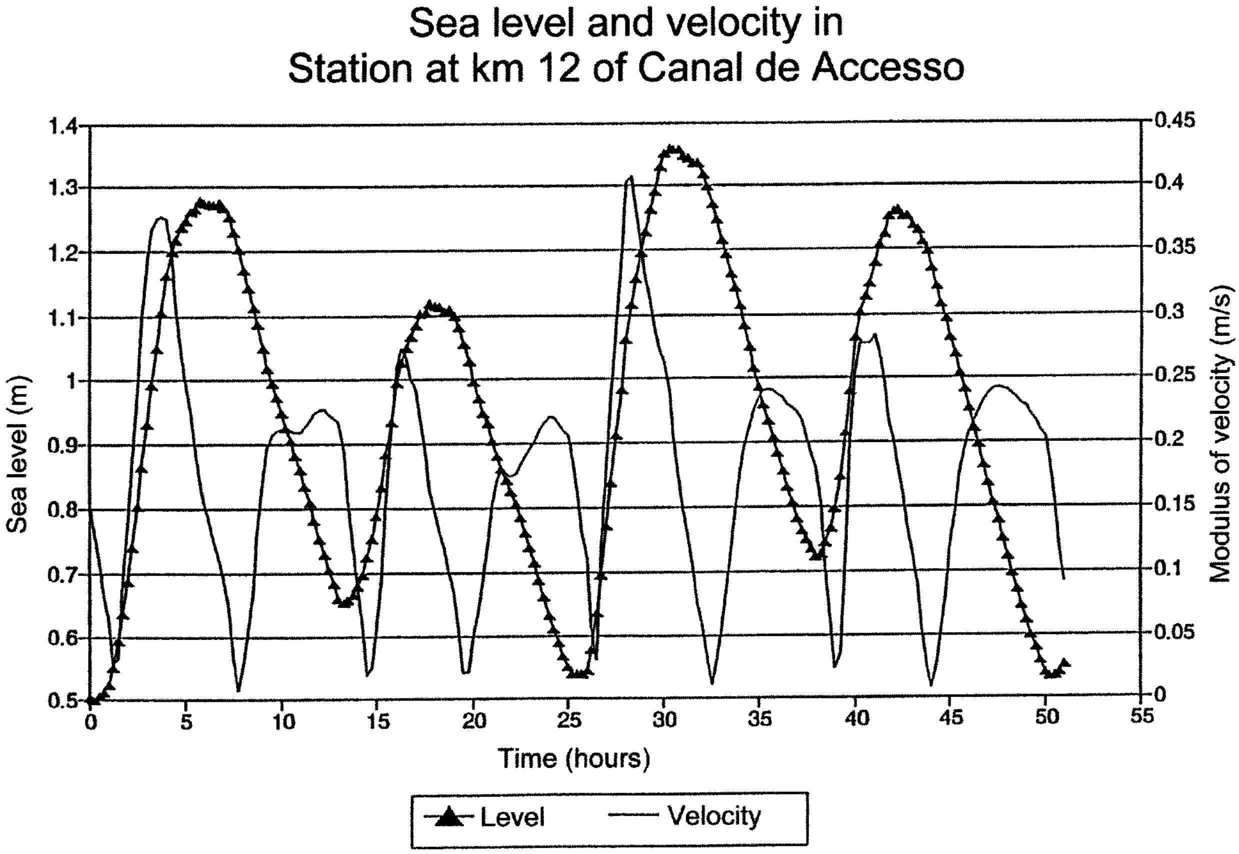
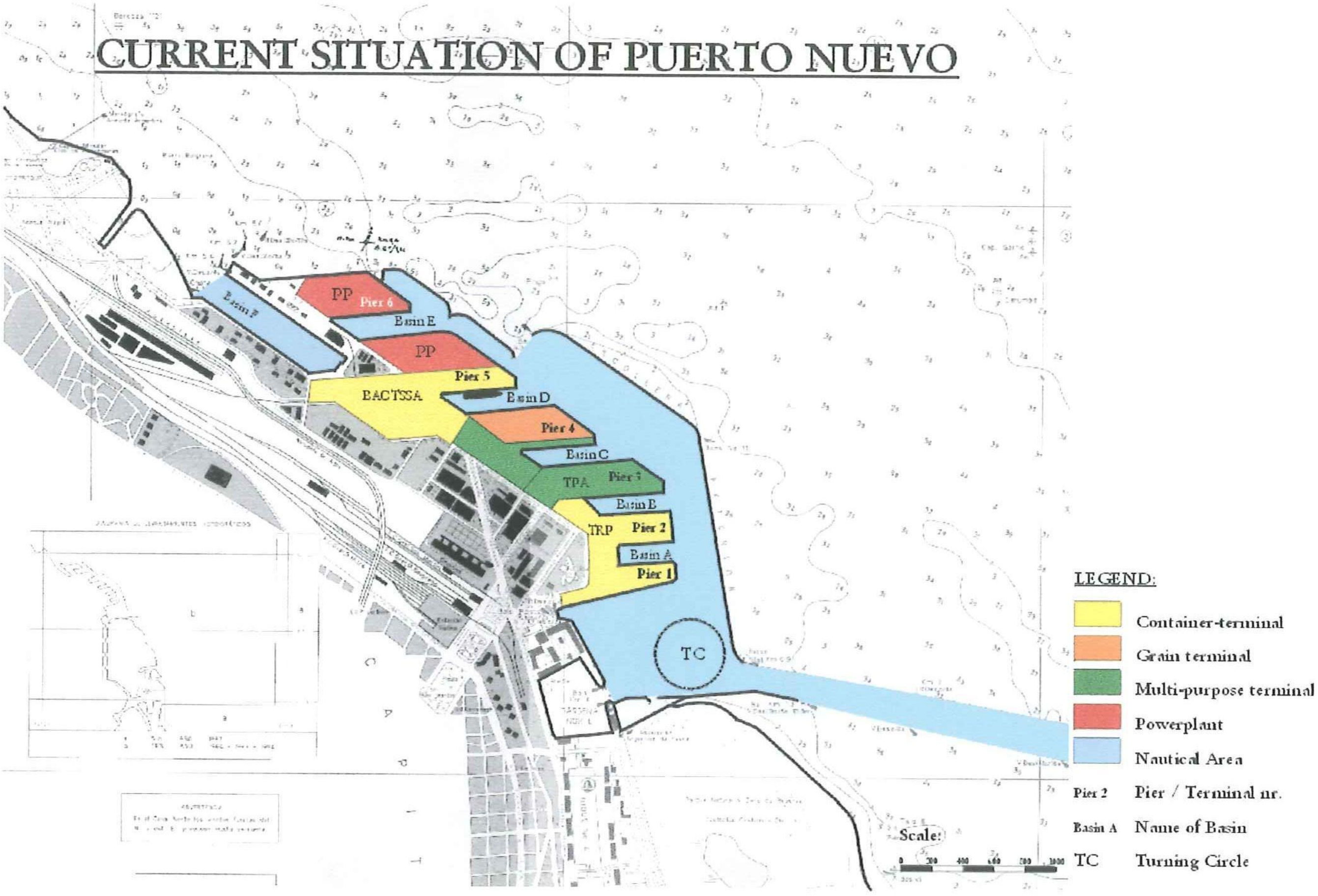


Figure C–1 Sea level and flow velocity near Buenos Aires, source: Instituto Nacional del Agua y del Ambiente (INA)

APPENDIX D. FIGURES OF CURRENT SITUATION AND EXPANSION ALTERNATIVES

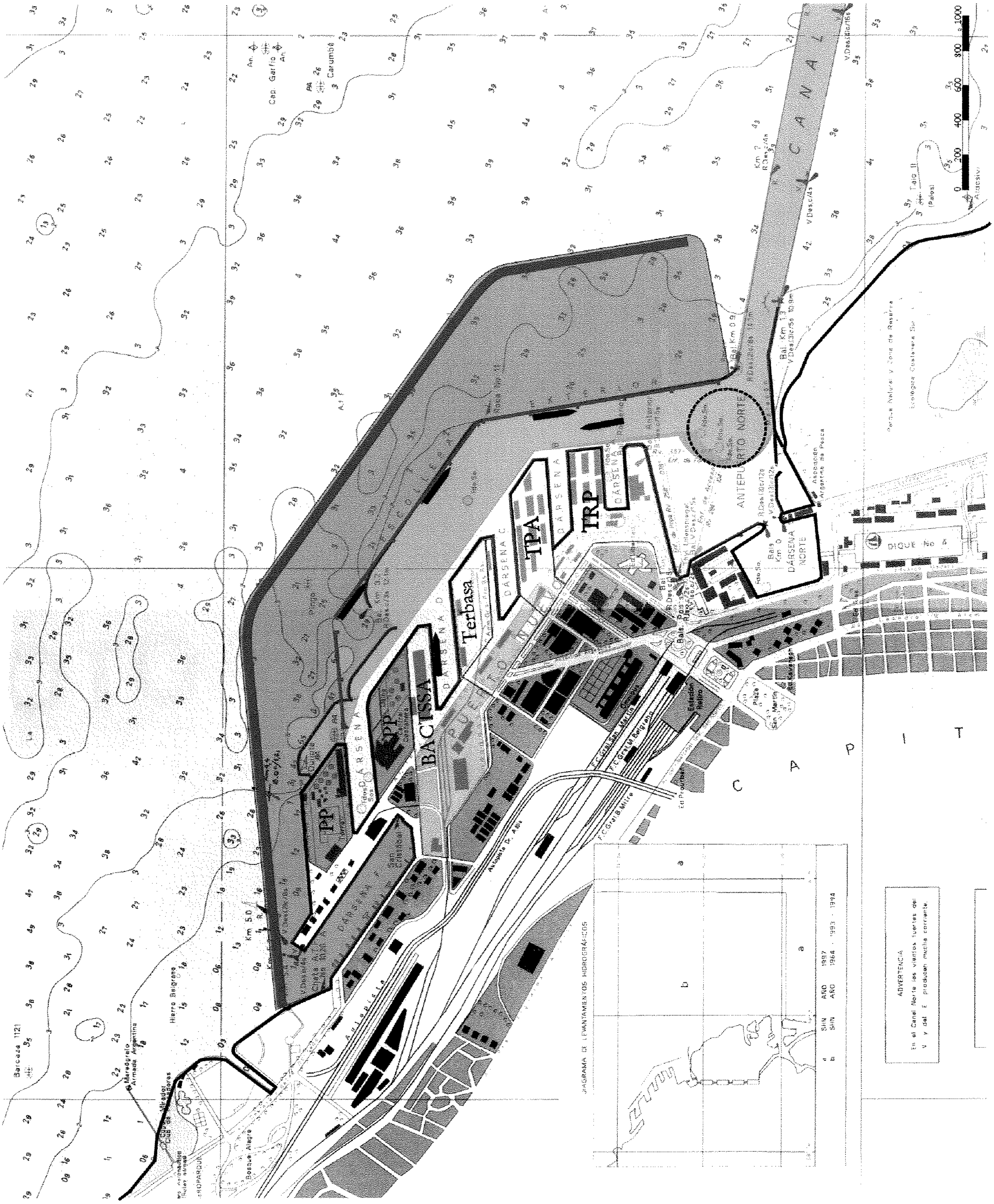
Appendix D.1 CURRENT SITUATION



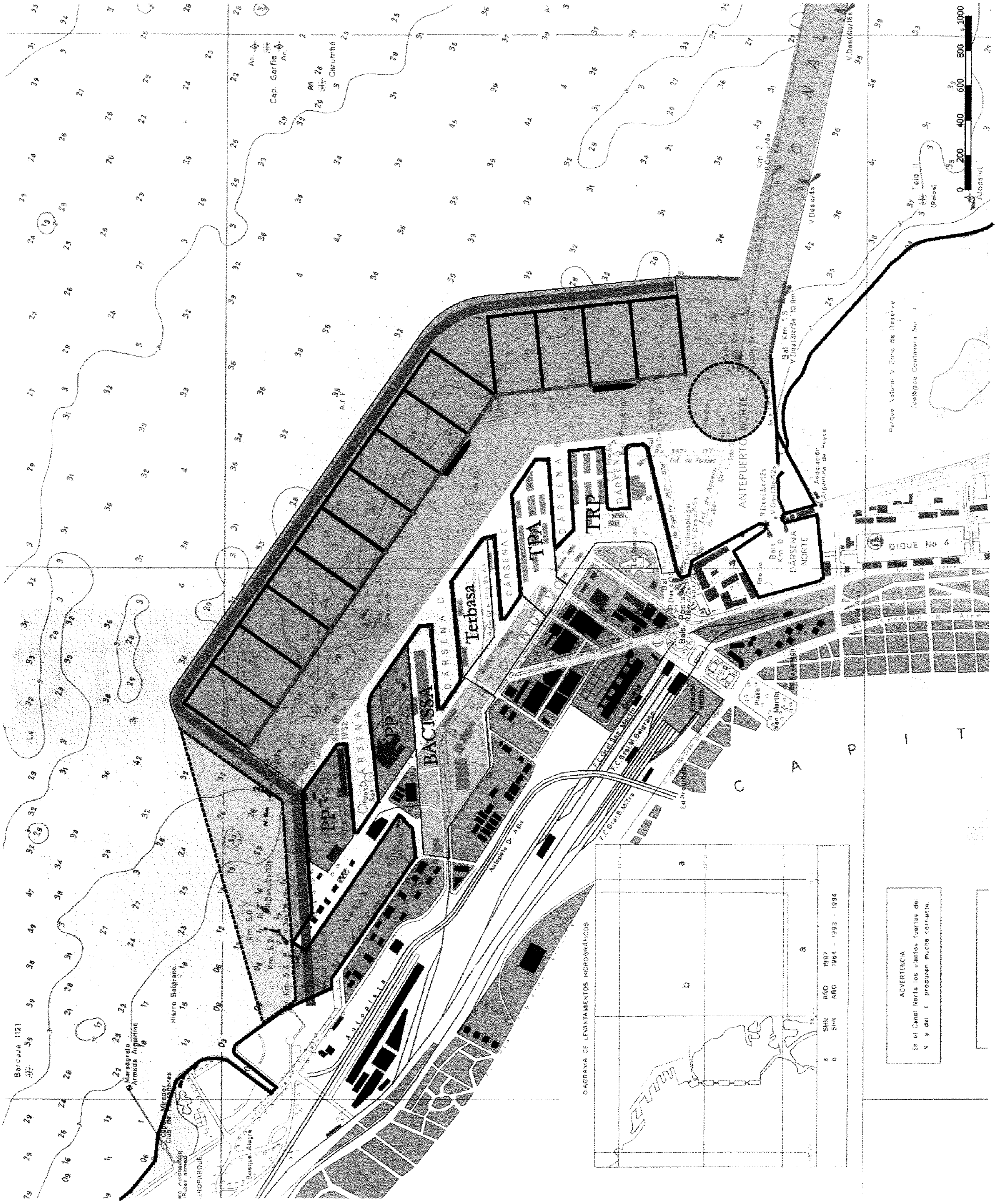
Appendix D.2 FIGURES OF ALL ALTERNATIVES

Alternative 1: AGP Proposal
Alternative 2: Modified AGP Proposal
Alternative 3: Compact Island
Alternative 4: One long quay
Alternative 5: Northern Expansion
Alternative 6: Boomerang

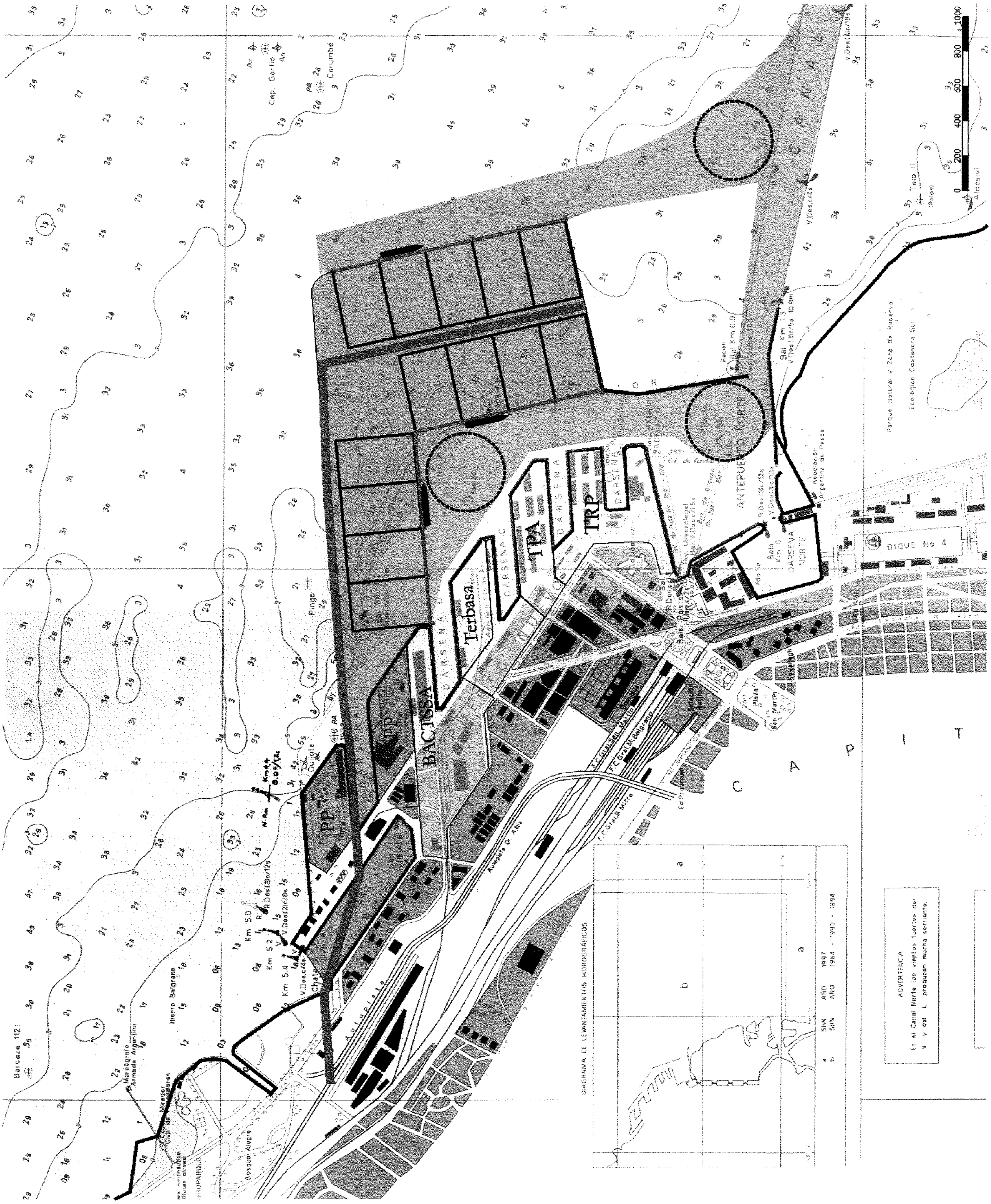
ALTERNATIVE 1: AGP Proposal



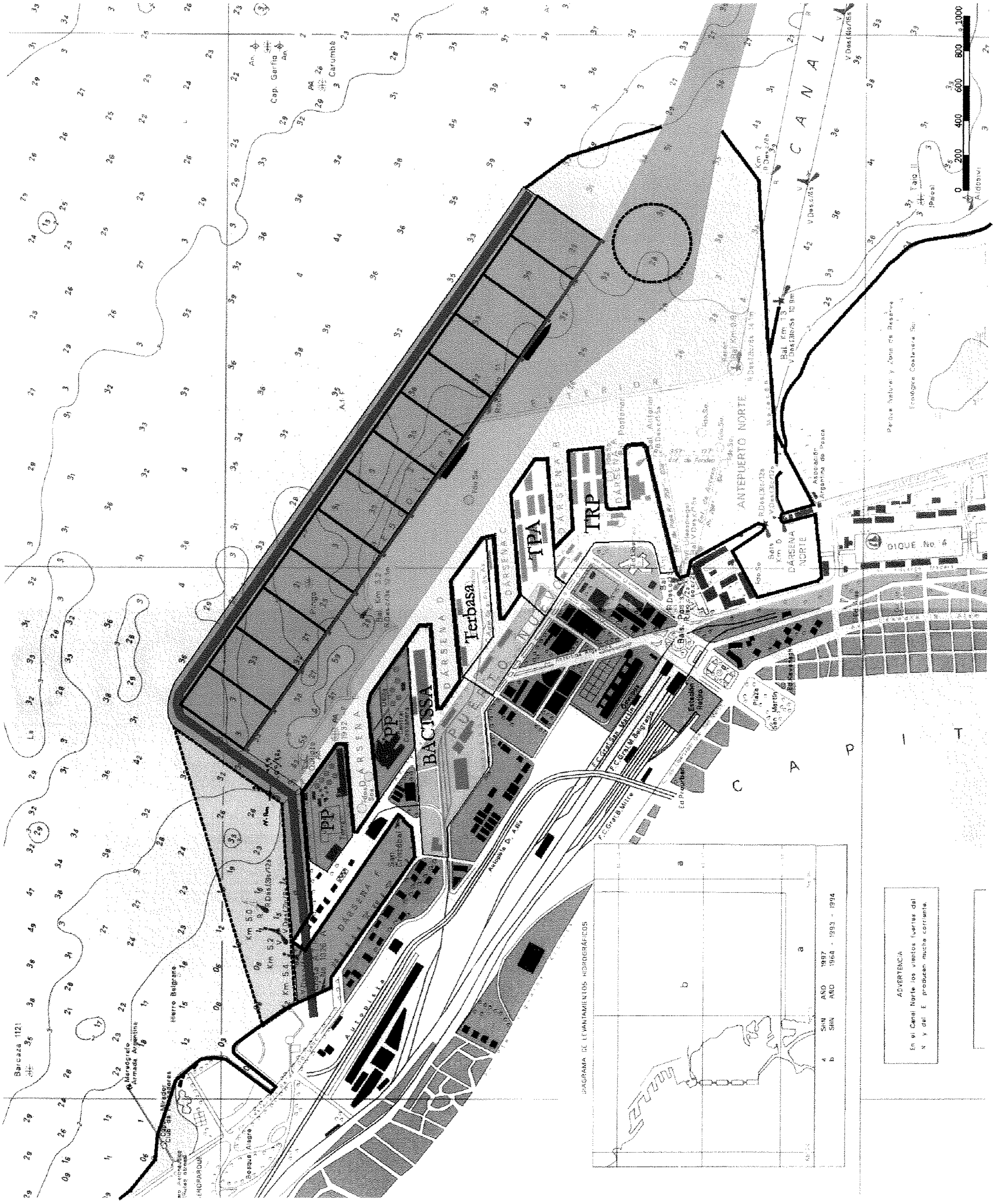
ALTERNATIVE 2: Modified AGP Proposal



ALTERNATIVE 3: Compact peninsula



ALTERNATIVE 4: One long quay



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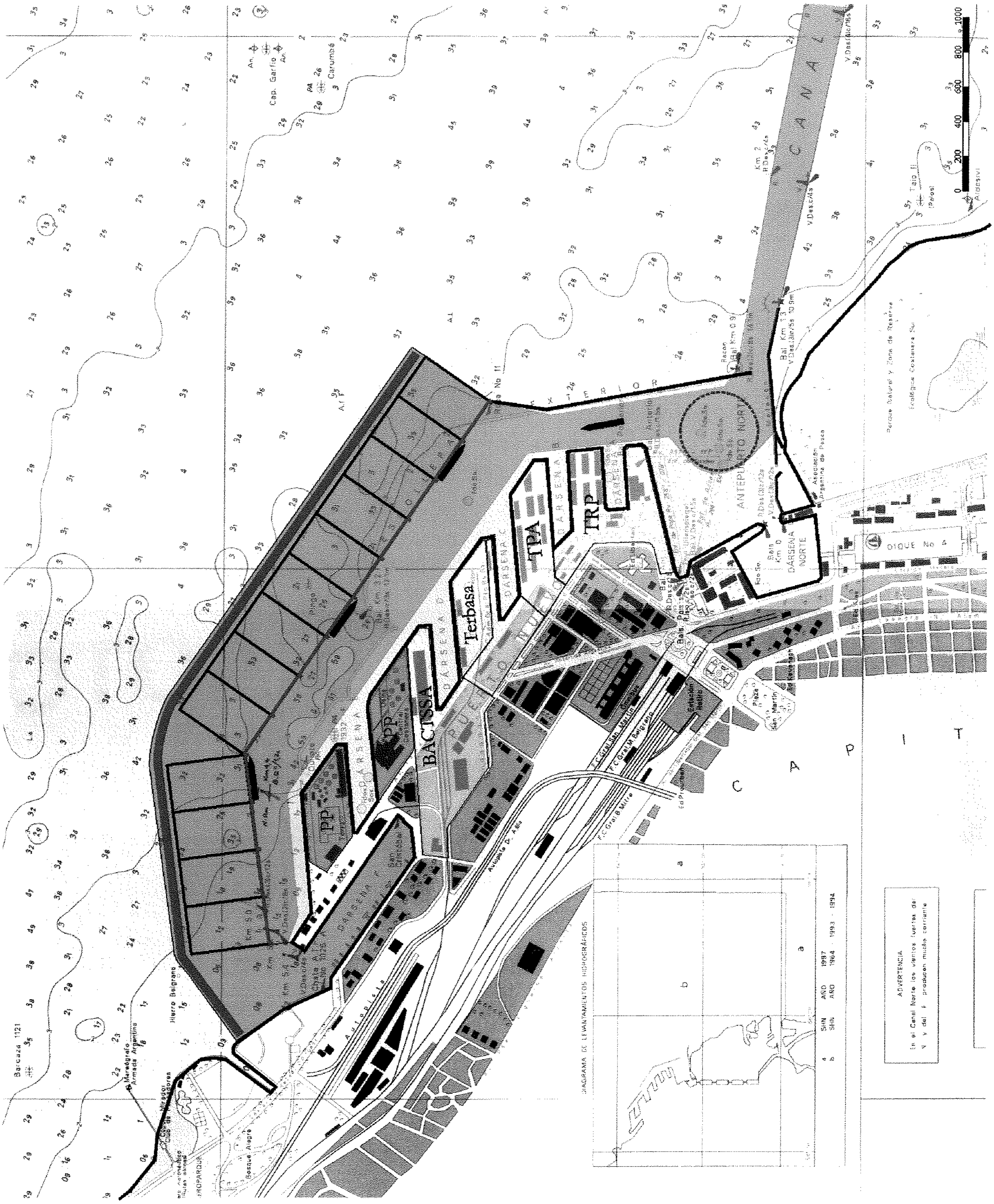
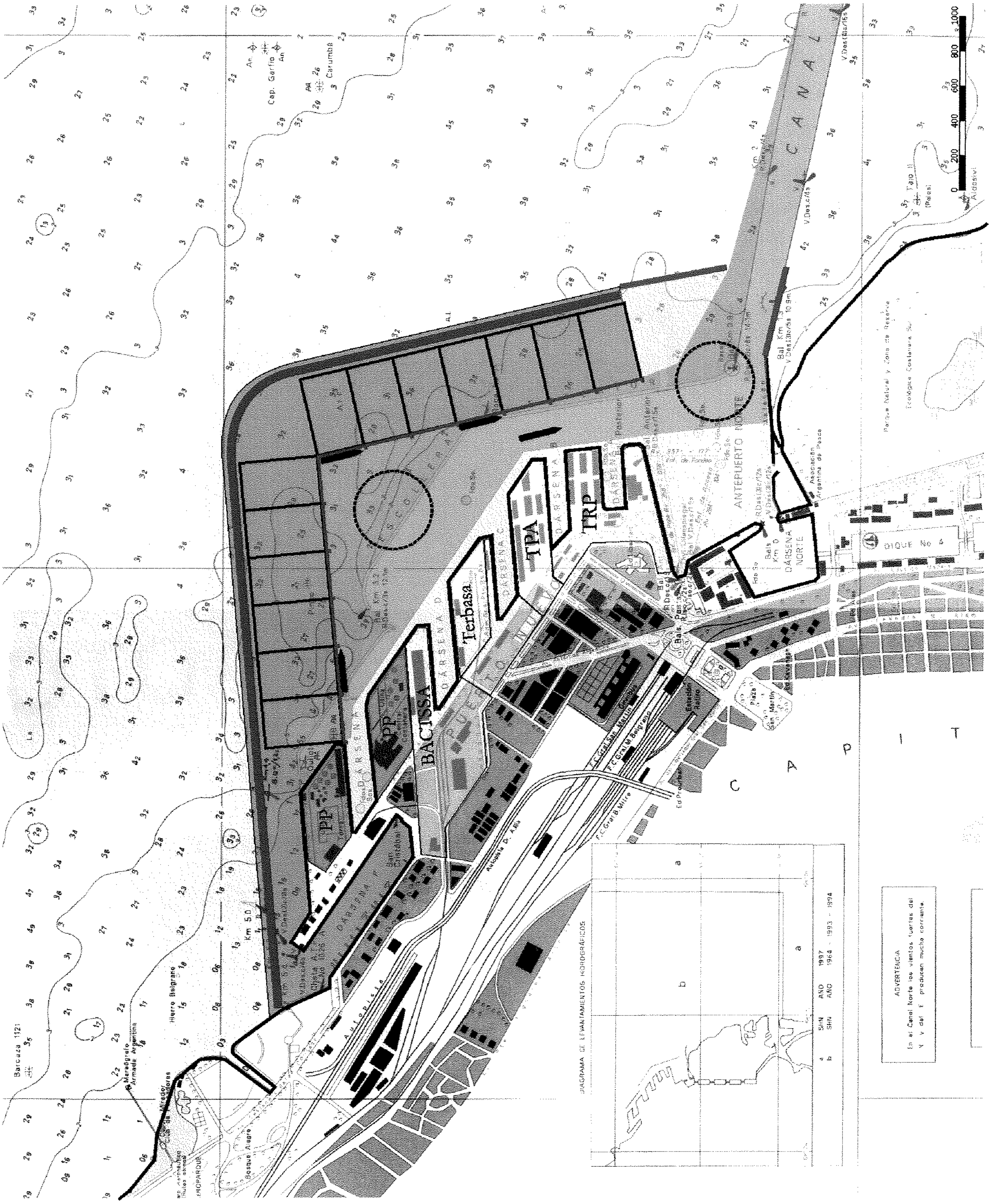
Alternative 5: Northern expansion



Alternative 6: The boomerang



Appendix D.3 DETAILS OF ALTERNATIVE 3

Ballast Nedam Dredging

APPENDIX E. GEOTECHNICAL INFORMATION

Appendix E.1 SOIL PARAMETERS:

Various sources report various values for geotechnical parameters. In Table E–1 below some ranges have been given

Soil type	Top level [m rel to CD]	γ_w [kN/m3]	γ_d [kN/m3]	φ [°]	c' [kPa]	φ_u [°]	c_u [kPa]	α
Reclaimed sand	+4.75	21.0	19.0	35	0	-	0	1:3
Alluvial deposits	-3 to -4	17.1	-	22	-	0	12	1:6 to 1:8
Tosca	-7 to -14	20.0		29	10-30	5-20	90-200	1:3 – 1:2
Sand	-21 to -24	20.0	18.0	35	0	-	-	1:4

Table E–1 Values of soil parameters in literature

For the calculations in this report the soil profile has been schematised:

Soil type	Top level [m rel to CD]	γ_w [kN/m3]	γ_d [kN/m3]	φ [°]	c' [kPa]	φ_u [°]	c_u [kPa]	α
Reclaimed sand	+4.75	21.0	19.0	35	0	-	0	1:3
Alluvial deposits	-3	17.0	-	22	-	0	12	1:6 to 1:8
Tosca	-7	20.0		29	20	0	150	1:2
Sand	-21	20.0	18.0	35	0	-	-	1:4
Note: in most of the calculations in this report, tosca has been analysed in a drained situation. As the top layer is dredged away, its parameters have not been calculated with.								

Table E–2 Used soil parameters

in which:

- γ_w: saturated unit weight
- γ_d: dry unit weight
- φ: internal angle of friction, drained
- φ_u: internal angle of friction, undrained
- c': drained shear strength
- c_u: undrained shear strength
- α: equilibrium angle

Appendix E.2 SCHEMATISED SOIL PROFILE FOR BUENOS AIRES

Source: E.C.I.S.A.: Informe Geotecnico, 1997: Sondeo IV Prima

Note: qc values have been calculated with standard relations between SPT and qc

Level [m +/- CD]	Soil type	SPT value [N]	SPT Graphical Form	qc [Mpa]	qc_recl [Mpa]	$\sigma'_{v,recl+q}$ [kPa]	$\sigma'_{v,recl}$ [kPa]	$\sigma'_{v,dredge}$ [kPa]	$\sigma'_{v,0}$ [kPa]
4,75						35,0	0		
4,00						49,3	14,3		
3,00						68,3	33,3		
2,00						87,3	52,3		
1,00						105,5	70,5		
0,00	-					116,5	81,5		
-1,00	-					127,5	92,5		
-2,00	mud	1		0,1		138,5	103,5		0
-3,00	mud	1		0,1		149,5	114,5		2,0
-4,00	mud	1		0,1		160,5	125,5		4,0
-5,00	mud	1		0,1		171,5	136,5		6,0
-6,00	mud	1		0,1		182,5	147,5		8,0
-7,00	tosca	1		0,1		193,5	158,5		10,0
-8,00	tosca	1		0,1		204,5	169,5		20,0
-9,00	tosca	2		0,1		215,5	180,5	0	30,0
-10,00	tosca	3		0,2		225,5	190,5	10,0	40,0
-11,00	tosca	18		1,4	2,70	235,5	200,5	20,0	50,0
-12,00	tosca	18		1,4	2,53	245,5	210,5	30,0	60,0
-13,00	tosca	20		1,5	2,66	255,5	220,5	40,0	70,0
-14,00	tosca	25		1,9	3,18	265,5	230,5	50,0	80,0
-15,00	tosca	50		3,8	6,13	275,5	240,5	60,0	90,0
-16,00	tosca	45		3,4	5,34	285,5	250,5	70,0	100,0
-17,00	tosca	36		2,7	4,15	295,5	260,5	80,0	110,0
-18,00	tosca	32		2,4	3,60	305,5	270,5	90,0	120,0
-19,00	tosca	26		2,0	2,86	315,5	280,5	100,0	130,0
-20,00	tosca	35		2,6	3,78	325,5	290,5	110,0	140,0
-21,00	sand	30		15,0	21,23	335,5	300,5	120,0	150,0
-22,00	sand	33		16,5	22,95	346,5	311,5	131,0	161,0
-23,00	sand	30		15,0	20,54	357,5	322,5	142,0	172,0
-24,00	sand	29		14,5	19,57	368,5	333,5	153,0	183,0
-25,00	sand	30		15,0	19,99	379,5	344,5	164,0	194,0
-26,00	sand	42		21,0	27,65	390,5	355,5	175,0	205,0
-27,00	sand	50		25,0	32,56	401,5	366,5	186,0	216,0
-28,00	sand	50		25,0	32,24	412,5	377,5	197,0	227,0
-29,00	sand	50		25,0	31,94	423,5	388,5	208,0	238,0
-30,00	sand	50		25,0	31,66	434,5	399,5	219,0	249,0
-31,00	sand	50		25,0	31,41	445,5	410,5	230,0	260,0

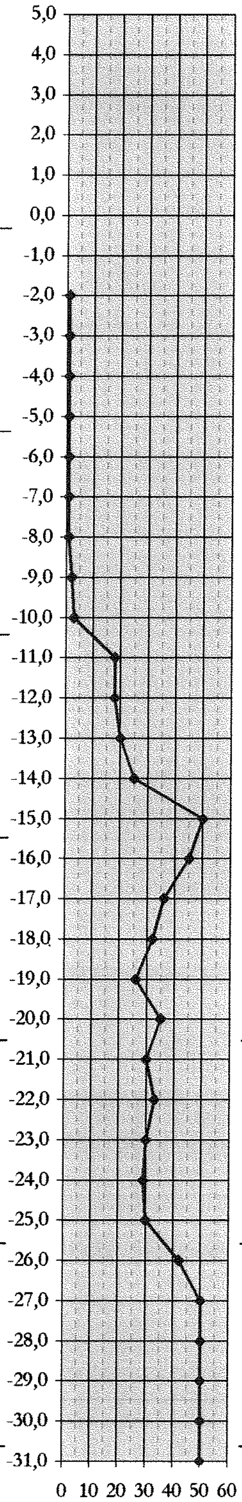


Table E–3 Schematized soil profile

Appendix E.3 GRAPHICAL CPT

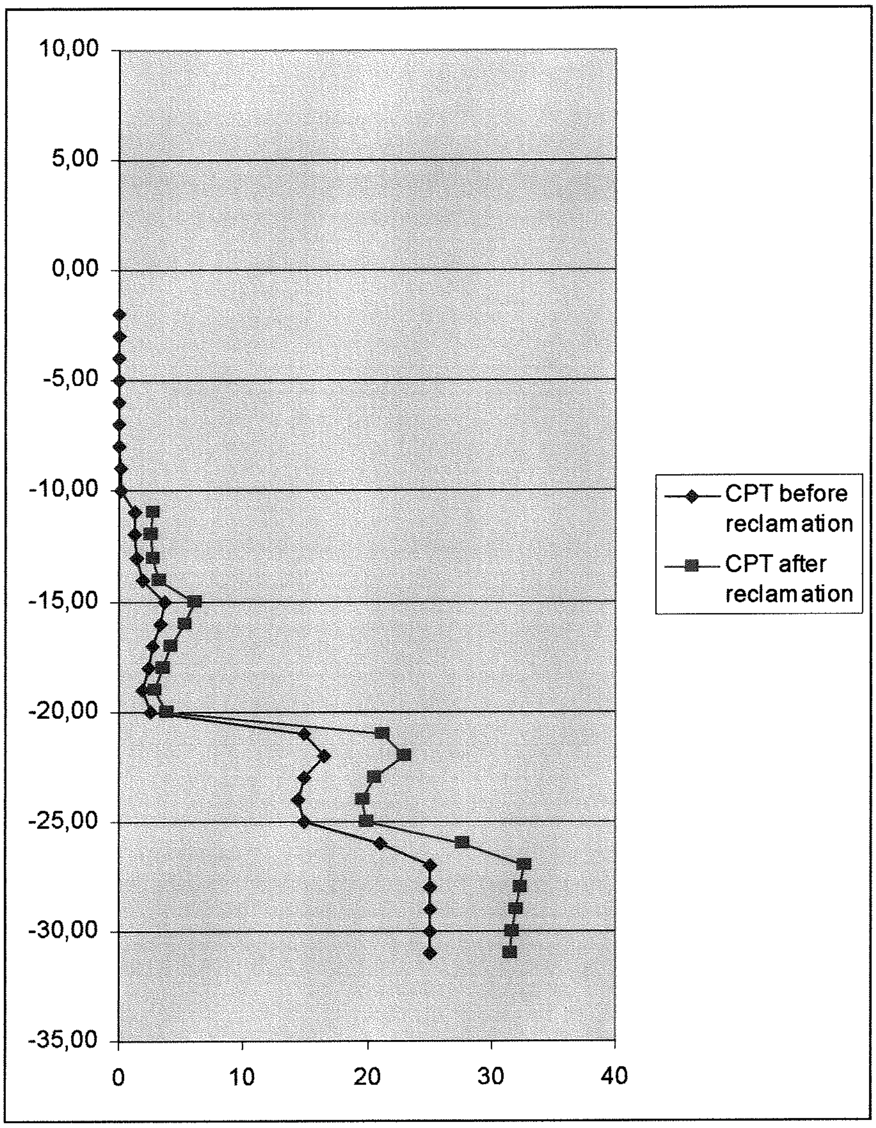


Figure E–1 Cone penetration test

values in Figure E–1 are stated in Table E–3 and been obtained by multiplying the SPT values N by a factor:

Sand: $q_c = 0.5 * N$

Clay: $q_c = cu / 15$ $cu = 4 * N$

$$q_{c,recl} = \sqrt{\frac{\sigma'_{v,recl}}{\sigma'_{v,0}}} * q_{c,0}$$

Appendix E.4 Vertical soil pressure diagram

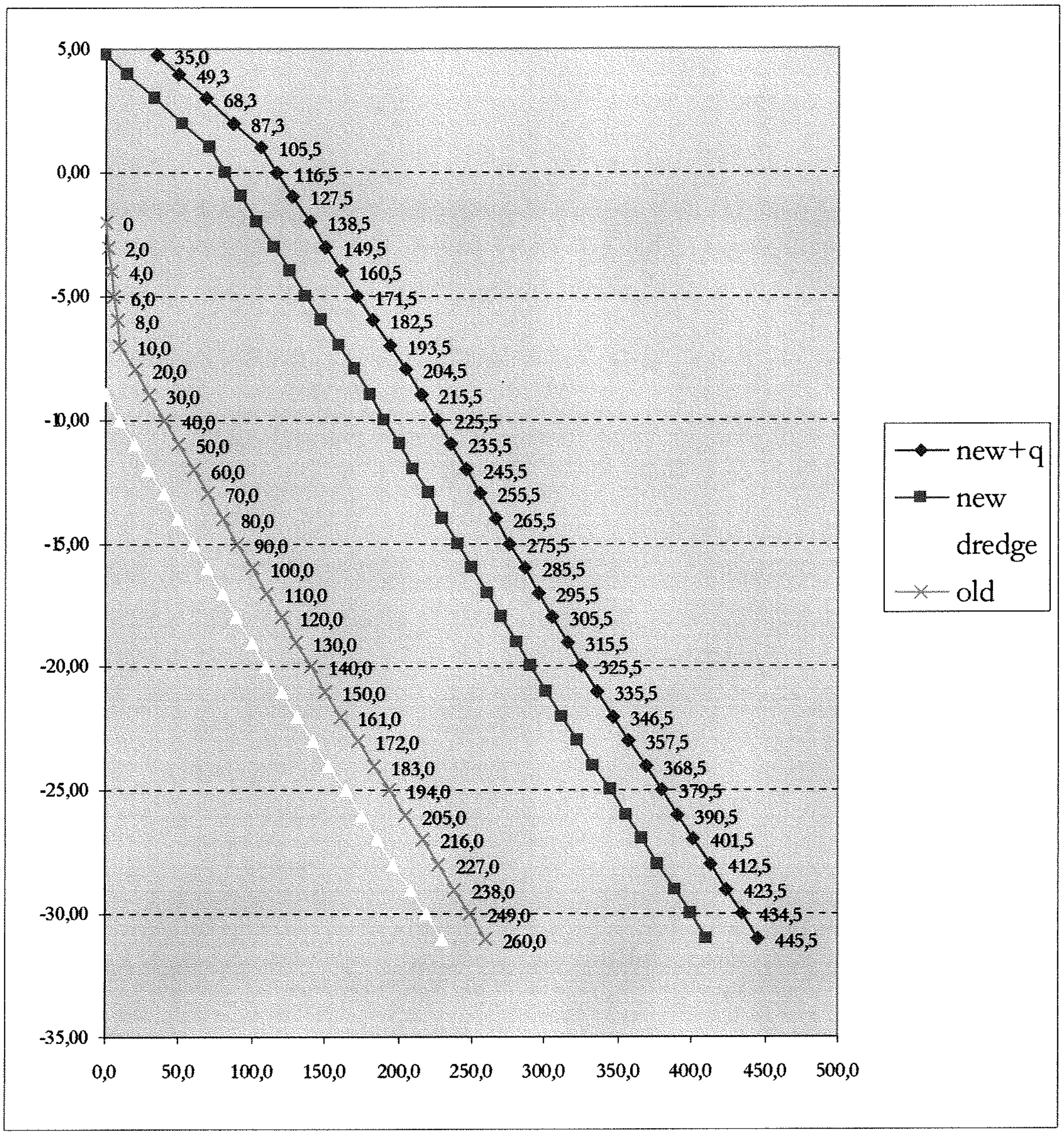


Figure E–2 Soil pressures before and after reclamation

APPENDIX F. DREDGING EQUIPMENT DATA SHEETS

Source: Dredgers of the world, 1997

Appendix F.1 APOLLO: TRAILING SUCTION HOPPER DREDGER

Apollo

GENERAL
Name Apollo
Owner Ballast Nedam Dredging
Marine Manager Ballast Nedam Bagger-en Exploitatiemaatschappij BV
Year Built 1977
Builder Verolme Scheepswerf Heusden BV
Type Trailing Suction Hopper Dredger
Classification (Notation on Area of Operation) Bureau Veritas I 3/3 E + Hopper Dredger: Deep Sea. SI: Trading area coastal waters, offshore distance not to exceed 15 miles; sailing time from Hook of Holland within 12 hours; from other ports SI approval required.
Flag Netherlands **Home Port** Zeist
Port of Registry Rotterdam **Registration** 14978 Z Rott 1977
Call Sign PCSW

MAIN DIMENSIONS
LOA 103.32 m **Breadth** 18 m **Depth** 8.65 m
GRT 5115 t **NRT** 1534 t **DWT** 6929 t
Lightweight 3321 t **Draft Dredging** 7.17 m
International Load Line 6.17 m

MACHINERY AND POWER
Total Installed Diesel Capacity 8251 kW
Propulsion Dredging Diesel **total capacity** 3972 kW

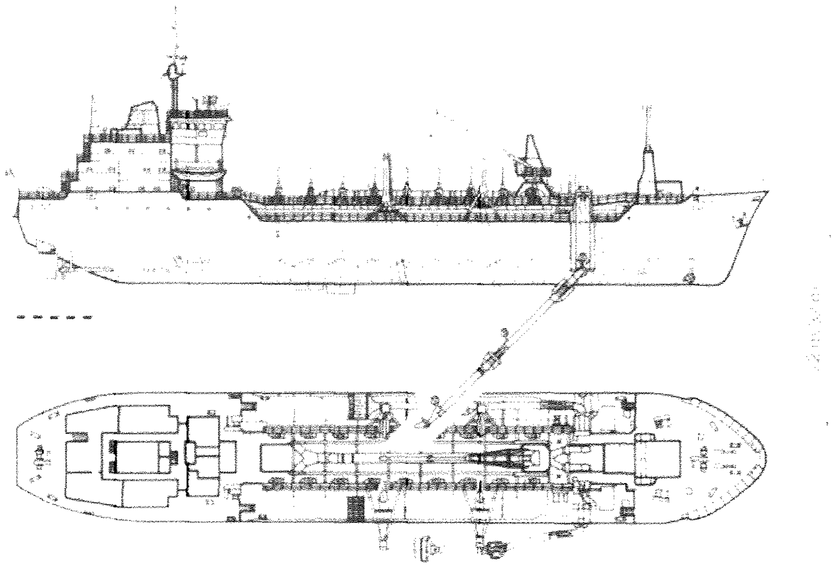
MACHINERY AND POWER (cont'd)
Propulsion Free Sailing Diesel **total capacity** 3972 kW
Bow Thruster(s) Electric **total capacity** 257 kW
Speed Loaded 12.6 knots
Speed Unloaded 12.6 knots
Bunker Gas Oil
Inboard Pump(s) 2 x Diesel **total capacity** 2648 kW
Jet Pump(s) Diesel on draghead **total capacity** 634 kW

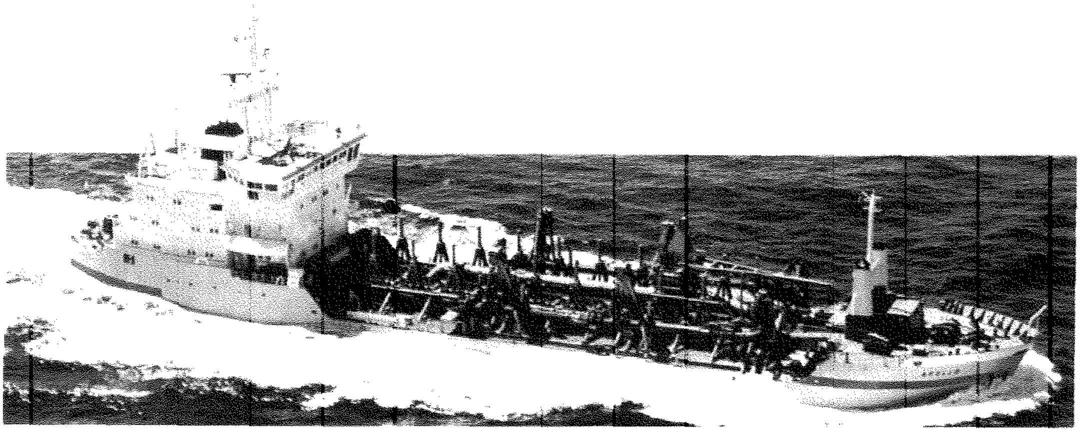
OPERATING PARAMETERS
Dredging Depth Normal 24 m
Dredging Depth Extended *34 m*

DREDGING AND DISCHARGING EQUIPMENT
Suction Pipe(s) 2 **diameter** 900 mm
Discharge System Bottom doors dumping system.
Hopper Capacity Top Coaming 4850 m3
Hopper Capacity Top Overflow 4695 m3

MOORING AND POSITIONING SYSTEMS
Mooring System 1 x stern and 2 x bow anchors.

ADDITIONAL DATA
1. 2 x diesel powered controllable pitch propellers. 2. Travelling crane.





Ballast Nedam Dredging

Appendix F.2 HAARLEM: CUTTER SUCTION DREDGER

Haarlem

GENERAL
Name Haarlem
Owner Ballast Nedam Dredging
Marine Manager Ballast Nedam Bagger-en Exploitatiemaatschappij BV
Year Built 1984
Builder Scheepswerf Stapel BV
Type Cutter Suction Dredger
Classification (Notation on Area of Operation) Bureau Veritas I 3/3 + dredger/NP. Deep Sea
Flag Netherlands **Home Port** Amsterdam
Port of Registry Amsterdam **Registration** 8708 Z Amst 1983
Call Sign PEMF

MAIN DIMENSIONS
LOA 71.65 m **Breadth** 11.85 m **Depth** 3.9 m
GRT 681 t **NRT** 204 t **Lightweight** 1160 t
Draft Dredging 2.63 m **International Load Line** 2.93 m

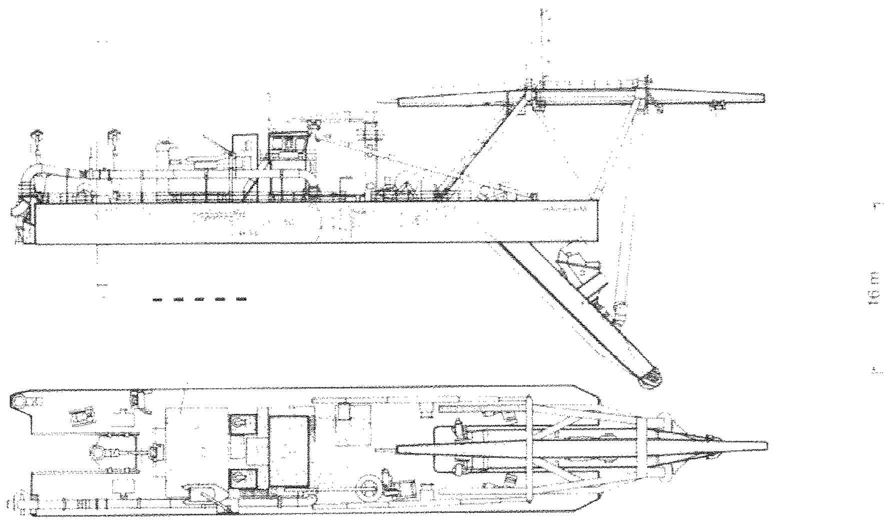
MACHINERY AND POWER
Total Installed Diesel Capacity 5054 kW
Bunker Gas Oil
Cutter Electric 736 kW
Inboard Pump(s) 2 x Diesel **total capacity** 3500 kW

OPERATING PARAMETERS
Dredging Depth Normal 16 m

DREDGING AND DISCHARGING EQUIPMENT
Suction Pipe(s) 1 **diameter** 800 mm
Discharge Pipe(s) 1 **diameter** 750 mm

MOORING AND POSITIONING SYSTEMS
Mooring System Spuds and anchor booms
Side Winch Lifting Capacity 30 t
Side Winch Speed 18 m/min.

ADDITIONAL DATA
1. Side winch maximum pulling power 45 tons



Ballast Nedam Dredging

Appendix F.3 HAM 311: TRAILING SUCTION HOPPER DREDGER

HAM 311

GENERAL
Name HAM 311
Owner Hollandsche Aanneming Maatschappij BV
Marine Manager Nederlandsche Overzee Baggermaatschappij BV
Year Built 1994
Builder I.H.C. Holland NV
Type Trailing Suction Hopper Dredger
Classification (Notation on Area of Operation) Bureau Veritas I 3/3 E + Hopper Dredger. Deep Sea. AUT-MS. Dredging within 15 miles offshore or 20 miles from port
Flag Netherlands **Home Port** Rotterdam
Port of Registry Rotterdam **Registration** 18475 Z Rott 1994
Call Sign PEME

MAIN DIMENSIONS
LOA 94 m **Breadth** 17.02 m **Depth** 7.6 m
GRT 3423 t **NRT** 1055 t **DWT** 5059 t
Lightweight 2367 t **Inland Tonnage** 5561 t
Draft Dredging 5.68 m **International Load Line** 5.06 m

MACHINERY AND POWER
Total Installed Diesel Capacity 5317 kW
Propulsion Dredging Diesel **total capacity** 1650 kW
Propulsion Free Sailing Diesel **total capacity** 2300 kW
Bow Thruster(s) Electric **total capacity** 335 kW

MACHINERY AND POWER (cont'd)
Speed Loaded 11.5 knots
Speed Unloaded 12.2 knots
Bunker Marine Diesel Oil/Gas Oil
Dredge Pump(s) Diesel **total capacity** 1545 kW
Inboard Pump(s) Diesel **total capacity** 1545 kW
Jet Pump(s) Diesel on draghead **total capacity** 890 kW
Discharge Pump(s) Diesel **total capacity** 2820 kW

OPERATING PARAMETERS
Dredging Depth Normal 27.5 m

DREDGING AND DISCHARGING EQUIPMENT
Suction Pipe(s) 1 **diameter** 900 mm
Discharge Pipe(s) 1 **diameter** 800 mm
Discharge System Van der Graaf bow coupling, diameter 0.75m Bow discharge: 2820kW, including 1545kW inboard pump power Bottom doors dumping system.
Hopper Capacity Top Coaming 3717 m3
Hopper Capacity Top Overflow 3510 m3

MOORING AND POSITIONING SYSTEMS
Mooring System 1 x stern anchor and 2 x bow anchors with chains.

ADDITIONAL DATA
1. 2 x diesel powered controllable pitch propellers. 2. Draft on inland mark: 6.25m.

Appendix F.4 HECTOR: CUTTER SUCTION DREDGER

Hector

GENERAL
Name Hector
Owner Ballast Nedam Dredging
Marine Manager Ballast Nedam Bagger-en Explöitatiemaatschappij BV
Year Built 1985
Builder Machinefabriek Vos
Type Cutter Suction Dredger
Classification (Notation on Area of Operation) Bureau Veritas I 3/3 (-) + Dredger/NP. Deep Sea.
Flag Netherlands **Home Port** Zeist
Port of Registry Rotterdam **Registration** 17334 Z Rott 1985
Call Sign PEOT

MAIN DIMENSIONS
LOA 91 m **Breadth** 16.5 m **Depth** 5.2 m
GRT 1939 t **NRT** 581 t **Lightweight** 2795 t
Draft Dredging 3.5 m
International Load Line 3.94 m

MACHINERY AND POWER
Total Installed Diesel Capacity 10821 kW
Bunker Gas Oil

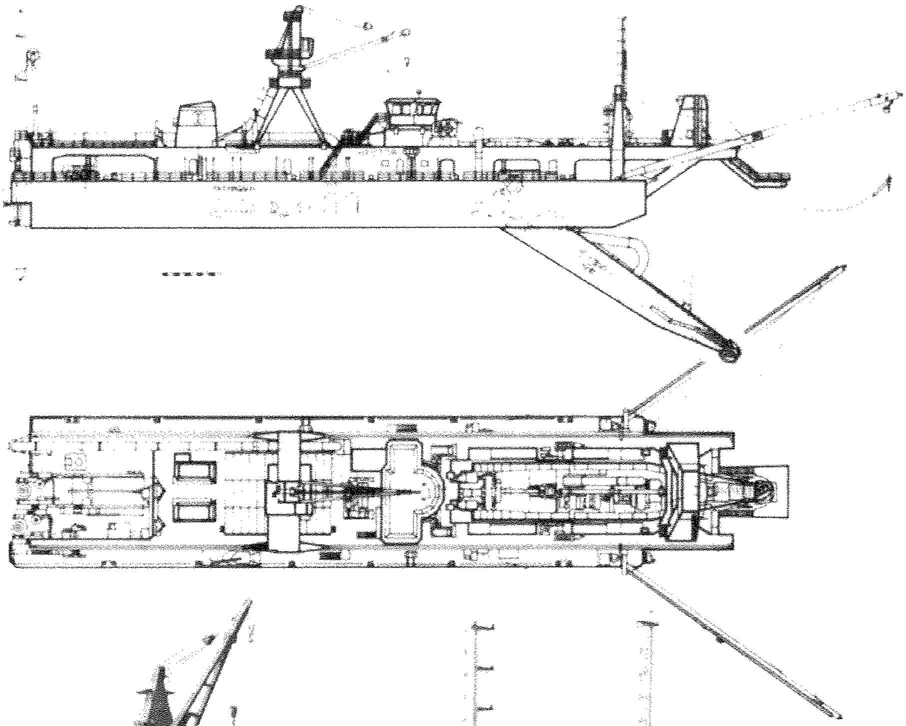
MACHINERY AND POWER (cont'd)
Cutter Electric 1117 kW
Dredge Pump(s) Electric underwater pump **total capacity** 1765 kW
Inboard Pump(s) 2 x Diesel **total capacity** 5590 kW
Jet Pump(s) Electric on draghead **total capacity** 736 kW

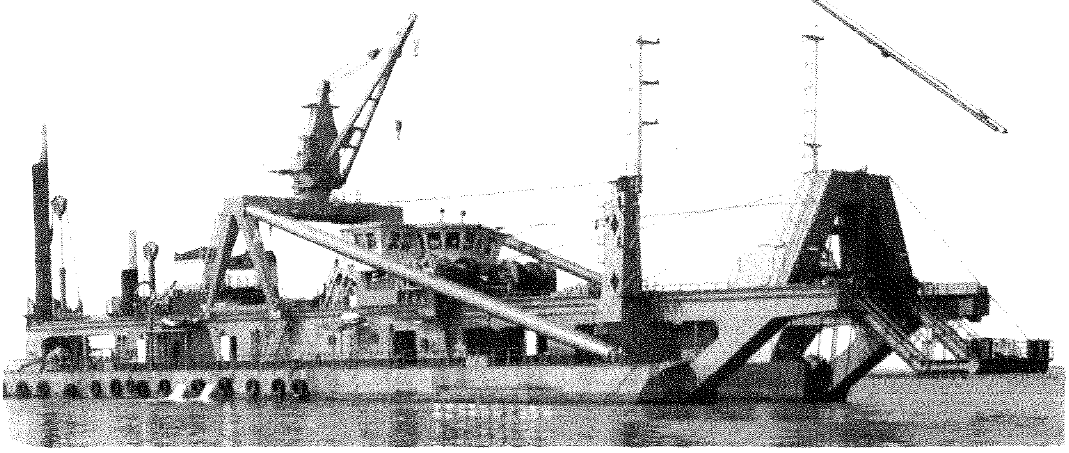
OPERATING PARAMETERS
Dredging Depth Normal 23 m
Dredging Depth Extended 28 m

DREDGING AND DISCHARGING EQUIPMENT
Suction Pipe(s) 1 **diameter** 900 mm
Discharge Pipe(s) 1 **diameter** 850 mm

MOORING AND POSITIONING SYSTEMS
Mooring System Spuds and anchor booms
Side Winch Lifting Capacity 41 t
Side Winch Speed 25 m/min

ADDITIONAL DATA
1. Side winch pulling power 65 tons maximum. 2. Travelling crane. 3. Dredging depth with suction pipe (profile pipe): 40m





Appendix F.5 ZUIDERKLIP: DEEP SUCTION DREDGER

Zuiderklip

GENERAL
Name *Zuiderklip*
Owner Royal Boskalis Westminster NV
Marine Manager Baggermaatschappij Boskalis BV
Year Built 1969
Builder Wijnands Werkendam BV
Conversion/Refit Year 1985
Type Cutter Suction Dredger
Classification (Notation on Area of Operation) Bureau Veritas I 3/3 (-) o Dredger/NP *Sheltered Waters*
Flag Netherlands **Home Port** Hardinxveld-Giessendam
Port of Registry Dordrecht **Registration** 4549 Z Dord 1971
Call Sign PIYU

MAIN DIMENSIONS
LOA 62.5 m **Breadth** 11 m **Depth** 2.7 m
GRT 364 t **NRT** 109 t **DWT** 102 t
Lightweight 600 t **Inland Tonnage** 741 t
Draft Dredging 1.8 m **International Load Line** 1.99 m

MACHINERY AND POWER
Total Installed Diesel Capacity 3998 kW
Bunker Gas Oil

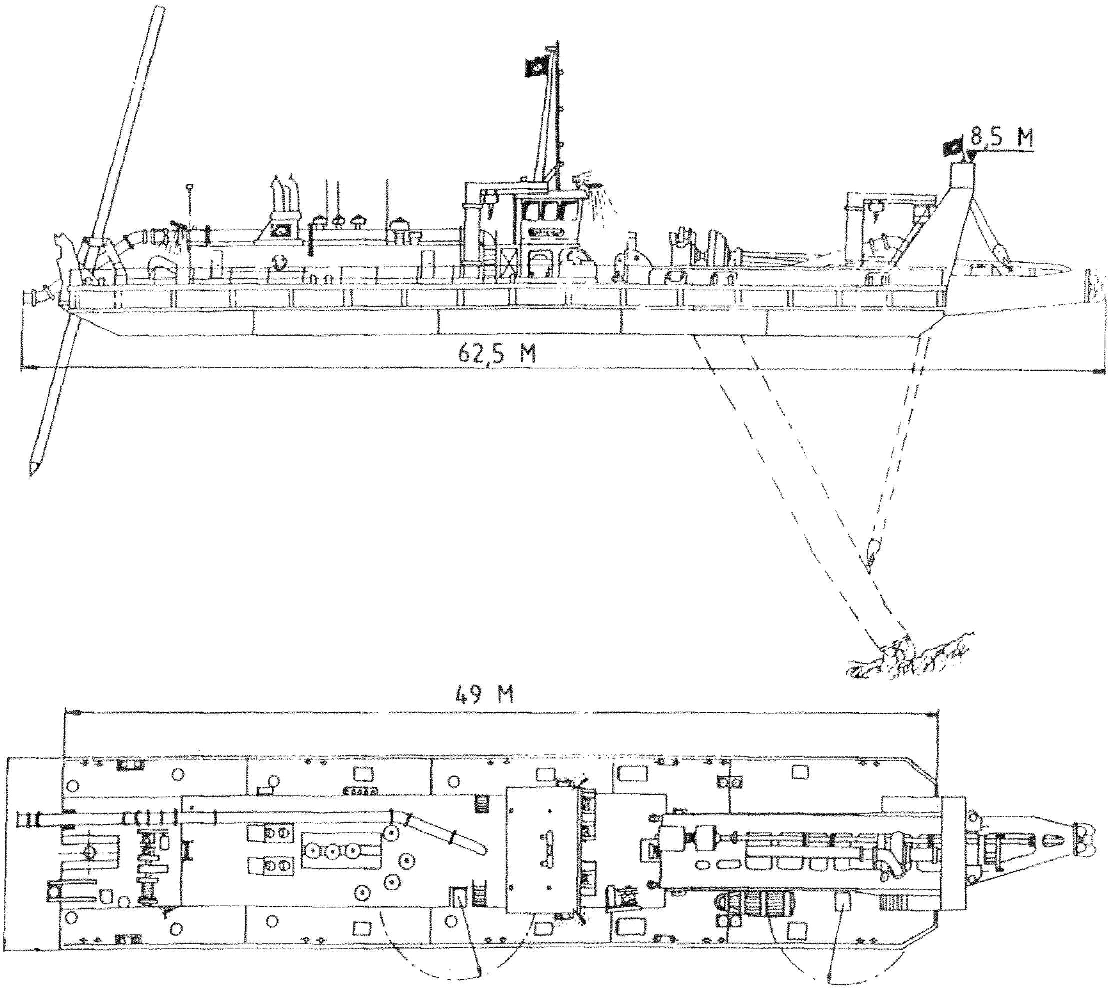
MACHINERY AND POWER (cont'd)
Cutter Electric 295 kW
Dredge Pump(s) Electric underwater pump **total capacity** 917 kW
Inboard Pump(s) Diesel **total capacity** 2312 kW

OPERATING PARAMETERS
Dredging Depth Normal 19 m

DREDGING AND DISCHARGING EQUIPMENT
Suction Pipe(s) 1 **diameter** 700 mm
Discharge Pipe(s) 1 **diameter** 600 mm

MOORING AND POSITIONING SYSTEMS
Mooring System 1 x tilting spud; 1 x fixed spud. 1 x side anchor
Side Winch Lifting Capacity 11.5 t
Side Winch Speed 17.5 m/min.

ADDITIONAL DATA
1. Refit/conversion: New engines installed. 2. 2 x fixed crane.



APPENDIX G. DREDGING CALCULATIONS

Appendix G.1 USED EQUATIONS:

Suction formula:

$$p = -\rho_w g\, z + \rho_m\, g(z - a) + \xi \cdot \frac{1}{2}\rho_m v_z^2$$ Equation G–1

- p: suction head, max vacuum
- z: suction depth
- a: depth of pump below waterline
- ξ: factor, in which losses are accumulated
- v_z: suction speed

Production:

$$Q_{sand} = Q_z \cdot C_s = v_z \cdot A_{buis} \cdot C_s$$

$$C_s = \frac{\rho_m - \rho_w}{\rho_{situ} - \rho_w} f_t$$ *Equation G–2*

f_t: transport factor (function of grain diameter d_{mf})

Appendix G.2 STANDARD VALUES FOR VARIOUS PARAMETERS:

Density of water	1000 kg /m^3
Density of overburden	1450 kg /m^3
Density of tosca	2000 kg /m^3
Density of sand in situ	2000 kg /m^3
Density of sand in hopper	1900 kg /m^3
Factor to include losses: ξ	3.5

Appendix G.3 DREDGING METHOD: NEAR-SHORE SAND PITS

All productions have been calculated as m^3 of situ material per week, as is customary. Only the limiting capacities have been presented here, pumping capacities have been checked.

Appendix G.3.1 Top layer

For the removal of the top layer, sandy silt, a medium sized cutter suction dredger[1] type Haarlem will be used in combination with two trailing suction hopper dredgers[2], type Apollo. It is not possible to use several barges, because these large barges do not exist, cannot unload themselves and several tugs are necessary to transport the barge over such a long distance. With this combination of CSD and 2 TSHD, either the suction

[1] Cutter suction dredger (CSD)

[2] Trailing suction hopper dredger (TSHD)

capacity of the CSD will be limiting, or the transport capacity of two trailing suction hopper. The pumping distance for the CSD is negligible and thus the pumping capacity of the CSD is not calculated here.

Capacity Calculation Cutter Suction Dredger Haarlem

Parameters:

Pumped volume: 2.3 m^3 /s, v_z=4.57 m/s
Depth of pump, a = 2 m
Suction depth: z = 10 m (deepest dredging depth)
Suction pipe diameter: 800 mm

From Equation F-1 can be found that $\rho_{m,max}$= 1460, but this is higher than the density of the situ material, for here an approximate density of 1230 kg/m^3 is used.
The maximum load of a dredger type Apollo: 4695 m^3, is equivalent to 2400 m^3 situ material.

Cycle time:

Loading 4695 m^3:	34 minutes
Manoeuvring of ship	08 minutes
Other activities	10 minutes
Total Cutter cycle	52 minutes

Cutter Suction Dredger Capacity: 2770 m^3/hour (situ)
Working hours: 110 / wk
Capacity: 304,700 m^3/wk

Capacity Calculation Trailing Suction Hopper Dredgers type Apollo

The TSHD are not using their pumps to dredge the material, the material is pumped into the hopper by the CSD. Therefore their capacity only depends on the sailing distance and sailing speed.
Parameters:
Sailing distance: 30 km
Sailing speed: 12,6 knots (6,3 m/s)

Determination Cycle time	
Loading	39 min
Sailing	80 min
Manoeuvring & dumping	10 min
Sailing	80 min
Manoeuvring	07 min
Total	216 min

Capacity 2x Apollo: 1333 m^3/hour
Operating Hours: 150 / wk,
Capacity: 200,000 m^3 / week

Limiting capacity is the capacity of the two TSHD, thus their capacity is the maximum weekly production.

Appendix G.3.2 Tosca Layer

The tosca layer will be dredged with medium (Haarlem) or large cutter suction dredger (Hector). The cutting and swing production are the most important, the suction capacity is not the limiting factor. Tosca, a stiffy clay, has an estimated specific cutting energy E_{sp}= 800 kJ/m^2.

Capacity calculation Haarlem

Effective cutting power $$P_{eff} = \frac{P_{cutter}}{f_p} = \frac{736\square\,\text{kW}}{1.3} = 566\,\text{kW}$$

Theoretic cutting production: $$Q_{cut} = \frac{P_{eff}}{E_{sp}} = \frac{566}{800} = 0.708\,{m^3}/{s}$$

Step length spud carriage 1.25 m

Translation speed: $$v_t = \frac{Q_{cut}}{D_{cutter} I_{step}} = 0.227\,{m}/{s}$$

Max translation speed $$v_{t,\max} = \frac{1}{0.85} \cdot v_{reep} = 0.353\,{m}/{s} > v_t$$ OK

Rotation speed of cutterhead $$v_c = \frac{n_c \cdot \pi \cdot D_{cutter}}{60} = 3.92\,{m}/{s}$$

Winch force: $$F_r = \frac{P_{cutter}}{v_c} = 187.8\,\text{kN}$$

Available winch force 300 kN OK

Dredging time: $$T_{dredging} = \frac{L_{spudcarriage}}{I_{step}} \frac{B}{v_{verhaal}} = \frac{5}{1.25} \cdot \frac{68}{(0.227 \cdot 60)} = 20\,\text{min}$$

Determination cycle time	
Dredging:	20 min
Positioning spud	2 min
Repositioning spud carriage:	2 min
Total	24 min

Dredged volume: V_{cyclus}=5*68*2.5* 0,70=595 m3

Production: $$C = \frac{V_{cycle}}{T_{cycle}} \cdot 60 \cdot f_{irr} = \frac{595}{24} \cdot 60 \cdot 0.8 = 1190\,\frac{m^3}{hour}$$

Operating hours: 110 hours/week

Production: 130,000 m3/wk

Capacity calculation Hector

Effective cutting power
$$P_{eff} = \frac{P_{cutter}}{f_p} = \frac{1176 \cdot kW}{1.3} = 905\,kW$$

Theoretic cutting production:
$$Q_{cut} = \frac{P_{eff}}{E_{sp}} = \frac{905}{800} = 1.13\,m^3/s$$

Step length spud carriage 1.5 m

Translation speed:
$$v_t = \frac{Q_{cut}}{D_{cutter} I_{step}} = 0.251\,m/s$$

Max translation speed
$$v_{t,\max} = \frac{1}{0.85} \cdot v_{swing} = 0.416\,m/s > v_t$$ OK

Rotation speed of cutterhead
$$v_c = \frac{n_c \cdot \pi \cdot D_{cutter}}{60} = 4.71\,m/s$$

Winch force:
$$F_r = \frac{P_{cutter}}{v_c} = 249.7\,kN$$

Available winch force 410 kN OK

Dredging time: $$T_{dredging} = \frac{L_{spudcarriage}}{I_{step}} \frac{B}{v_{verhaal}} = \frac{6.0}{1.50} \cdot \frac{125}{(0.251 \cdot 60)} = 33\,\text{min}$$

Determination cycle time	
Dredging:	33 min
Positioning spud	2 min
Repositioning spud carriage:	2 min
Total	37 min

Dredged volume: V_{cycle}=6*125*3.0* 0,70=1575 m3

Production: $$C = \frac{V_{cycle}}{T_{cycle}} \cdot 60 \cdot f_{irr} = \frac{1575}{37} \cdot 60 \cdot 0.8 = 2043 \frac{m^3}{hour}$$

Operating hours: 110 hours/week:

Production: 225,000 m3/wk

Appendix G.3.3 Sand layer

The sand layer will be dredged with a medium deep suction dredger (type Zuiderklip) or a large deep suction dredger (converted cutter suction dredger Hector). The limiting capacity is the suction capacity of the DSD.

Production calculation Zuiderklip

Max. dredging depth	40 m
Depth of pump, when dredging at -40	10 m
Suction pipe diameter	700 mm
Available vacuum	75 kPa
Pump discharge	2.0 m^3/s
Max. dredging density	$\rho_{m.\max} = \frac{p + \rho_w \cdot g \cdot z}{(z-a) \cdot g + \xi \cdot \frac{1}{2} \cdot v_z^2} = 1354 \frac{kg}{m^3}$
Discharge (situ)	$Q_{situ} = \frac{(\rho_m - \rho_w)}{(\rho_{situ} - \rho_w)} Q = 0.708\ m^3/s$
Gross hourly capacity	$C_{situ} = 3600 \cdot Q_{situ} = 2549\ {m^3}/{hour}$
Operating hours / week	120
Efficiency	75%
Week production	240,000 m^3

Production calculation Hector

Max. Dredging depth	45 m
Depth of pump, when dredging at -40	10 m
Suction pipe diameter	900 mm
Available vacuum	70 kPa
Pump discharge	3.3 m^3/s
Max dredging density	$\rho_{m.\max} = \frac{p + \rho_w \cdot g \cdot z}{(z-a) \cdot g + \xi \cdot \frac{1}{2} \cdot v_z^2} = 1309 \frac{kg}{m^3}$
Discharge (situ)	$Q_{situ} = \frac{(\rho_m - \rho_w)}{(\rho_{situ} - \rho_w)} Q = 1.02\ m^3/s$
Gross hourly capacity	$C_{situ} = 3600 \cdot Q_{situ} = 3924\ {m^3}/{hour}$
Operating hours / week	120
Efficiency	75%
Production	330,000 m^3/wk

Appendix G.4 DREDGE SAND AT RIO PARANA

When dredging sand (fine sand 150 - 200 μm) at the Rio Parana trailing suction hopper dredgers of the type (size) HAM 311 will be used. The sailing distance is so large that this determines the weekly production to a large extend. Therefore the hopper will be loaded to its maximum.

Production calculation HAM 311

Dredging depth	10 m
Depth of pump	2 m
Hopper capacity	3517 m^3 or 4900 ton → 2579 m^3 sand in hopper or 2323 m^3 situ
Suction / discharge pipe diameter	900 mm / 750 mm
Sailing speed unloaded / loaded	12.2 / 11.5 knots
Sailing distance	100 km
Available vacuum	70 kPa
Factor to calculate losses: ξ	3.5
Average overflow	35 %
Pump discharge	2.86 m^3/s (velocity: 4.5 m/s)
Max dredging density	$\rho_m = \frac{p + \rho_w \cdot g \cdot z}{(z-a)\cdot g + \xi \cdot \frac{1}{2} v_z^2} = 1476 \, kg/m^3$
Production (situ)	$Q_{situ} = \frac{(\rho_m - \rho_w)}{(\rho_{situ} - \rho_w)} Q = 1.42 \, m^3/s, \quad C_s = 0.48$
Production in hopper	$Q_{hopper} = 1.11 \cdot Q_{situ} = 1.58 \, m^3/s$

Dredging Time	
Dredging until hopper full	$T_{full} = \frac{2579}{2.86 * 60} = 15 \min$
Volume sand:	15*1.58*60 =1422 m^3
Dredging volume during overflow	$\frac{2579 - 1422}{1 - 0.35} = 1780 m^3$
Dredging time during overflow	18.8 min
Total Dredging time	33.8 minutes

Discharging (pumping) time	
Using the same velocity	4.5 m/s
Discharge	1.99 m^3/s
Same density as above: C_s=0.48	$\frac{2579}{1.99 * 0.48 * 60} = 45 \min$
Discharge time	45 minutes

Determination Cycle Time	
Sailing unloaded	273 minutes
Manoeuvring	8 minutes
Dredging	33.8 minutes
Sailing loaded	290 minutes
Manoeuvring + Coupling	10 minutes
Discharge	45 minutes
Uncoupling	5 minutes
Total cycle time (rounded)	665 minutes

Production/hour:	2323/(660/60) = 209 m^3
Working hours/wk :	150,
Production:	31,500 m^3/wk

APPENDIX H. DREDGING VOLUMES AND COSTS CALCULATIONS FOR ALTERNATIVES 2 AND 3

Appendix H.1 RECLAMATION

When the settlements of the alluvial deposits amount 1.50 metres, the level at which the expansion will be constructed is the design level + settlements =
4.75 + 1.50 = 6.25 m + CD. With an average seabed level of 3 m - CD, this means 9.25 metres, which have to be filled.
For both alternatives 2 and 3, which have the same area of 259 ha, the required volume of suitable material is 23,957,500 m^3.
Below two dredging costs calculations are made: When *tosca* is not used in the reclamation, this volume sand has to be dredged from sand pits. Using the dredging method as described in chapter 15 results in: 5 rectangular pits with bottom dimensions: 360 x 360 m.

	Volume (m^3)	Unit price	Price (M$)
Alluvial deposits 1st pit	1,534,791	$ 4.50	6.91
Tosca 1st pit	4,567,059	$ 4.50	20.51
Alluvial deposits 2nd – 5th pit	6,139,163	$ 2.00	12.28
Tosca 2nd – 5th pit	18,268,237	$ 3.00	54.80
Sand	23,957,500	$ 2.25	53.90
Total	54.466.750		148.45

Table H–1 Reclamation dredging calculation tosca not used, alternative 2

When *tosca* is partly used in the reclamation (the *tosca* from the 1st sand pit) only 4 sand pits are needed and the capital dredging costs of the reclamation become:

	Volume (m^3)	Unit price	Price (M$)
Alluvial deposits 1st pit	1,534,791	$ 4.50	6.91
Tosca 1st pit	4,567,059	$ 3.00	13.70
Alluvial deposits 2nd – 4th pit	4,604,372	$ 2.00	9.21
Tosca 2nd – 4th pit	13,701,178	$ 3.00	41.10
Sand	19,390,441	$ 2.25	43.63
Total	43,797,841		114.55

Table H–2 Reclamation dredging calculation tosca of 1st sand pit used, alternative 2

The possible cost reduction of 34 million US$ makes it worthwhile to check whether or not *tosca* can be used as reclamation material. For now, only *tosca* from the 1st sand pit will be used in the reclamation as it has sufficient time to settle (10 – 13 years, see chapter 16).

In order to speed up the settlements, vertical drains will be used. When placing synthetic strips every 2 metres, this results in 2900 drains per ha and costs approximately $40,000 per ha reclaimed land.

Appendix H.2 DREDGING ACCESS CHANNEL

The length of the access channel, from the entrance of the port of Buenos Aires to the split between *Canal de Accesso al Puerto de Buenos Aires* and *Canal Emilio Mitre* is 11,000 m. Only *Canal Norte* will be dredged to the new required dimension, *Canal Sud* will keep its current dimension. The current bottom dimensions of *Canal Norte* are: depth: 9.75 m and width 100 m. The new dimensions, calculated in chapter 5 are: depth: 13.2 m and width 220 m. This enlargement does not have to be made at once, but can be done in phases.

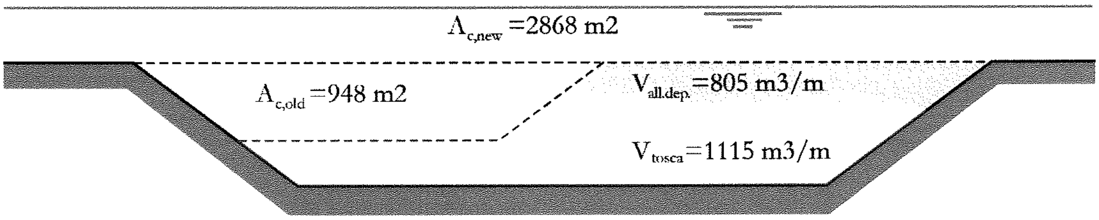


Figure H–1 Crossection of old and new access channel

From Figure H–1 can be seen that the dredging volume amounts: 1915 m^3/m
The total capital dredging volume of the access channel becomes approximately:

$$(805 + 1115) * 11{,}000 \approx 21{,}000{,}000 \text{ m}^3.$$

The top layer of alluvial deposits will be dredged by TSHDs, the *tosca* layer will be dredged by a CSD.

Soil layer	Main equipment	Dredging volume (m^3)	Costs / m^3	Costs (M$)
Alluvial deposits	2x TSHD	8,900,000	$ 2.0	17.80
Tosca	CSD + 2x TSHD	12,100,000	$ 4.5	54.45
Total		21,000,000		72.25

Table H–3 Dredging quantities and costs widening and deepening access channel

Appendix H.3 DREDGING BASINS

Appendix H.3.1 Alternative 2: Capital dredging basins

In the new port, the depth of the basins has to be adjusted. As stated in previous chapters, not all basins have a sufficient depth today and not all basins can be dredged to this level. In the following table for every basin the current depths have been given and the required capital dredging volume has been calculated. Together this gives the requires capital dredging volume and costs for deepening the basins in Puerto Nuevo

Basin Name	Area (ha)	Current depth	Future depth	Dredging volume (m3)
Basins A to E	37.8	-9.75	-9.75	-
NW-SE (95 ha)	47.5	-9.75	-13.2	1,638,750
	47.5	-4.00	-13.2	4,370,000
N-S (35,6 ha)	17.8	-9.75	-13.2	614,100
	17.8	-3.00	-13.2	1,815,600
Turning circle + access in port	26	-11.00	-13.2	572,000
Antepuerto	25	-3.00	-13.2	2,550,000
Antepuerto (not used)	50	-5.00	-	-
Total	269.4			11,560,450

Table H–4 Capital dredging volumes basins, alternative 2

The average dredging costs have been estimated to be: $3.50 / m^3
Capital dredging costs for the basins amount then: 40.46 million US $

Appendix H.3.2 Alternative 2: Maintenance volumes

Over the total nautical area used for port operations (the basins and the access channel) an average accretion of 1.5 metres per year is measured today. The same accretion is assumed to occur in the future so the maintenance volume is proportional to the nautical area. The values and costs per year can be found in the table below. Maintenance works will be done by Trailing Suction Hopper Dredgers, and unit price is estimated to be $2.00 / m^3

Area name	Area	Volume (m^3/year)	Price (M$/year)
Basins	269.4 ha	4,041,000	8.08
Access channel	242 ha	3,630,000	7.26
Total		7,671,000	15.34

Table H–5 Annual maintenance volumes and costs, alternative 2

Appendix H.3.3 Alternative 3: Capital dredging basins

Basin Name	Area (ha)	Current depth	Future depth	Dredging volume (m3)
Eastern access channel	56	-3,00	-13.2	5.712.000
Basins A to E	29	-9,75	-9,75	-
Access to new channel	29	-3,00	-13.2	2.958.000
N-S Old Port (15 ha)	9	-9,75	-13.2	310.500
	6	-3,50	-13.2	582.000
"Triangle" Old port (52 ha)	20.8	-9,75	-13.2	717.600
	31.2	-3,00	-13.2	3.182.400
Turning Circle old + access	20	-11,00	-13.2	440.000
Antepuerto	10	-8,00	-13.2	520.000
Antepuerto (not used)	42	-5,00	-	-
Total	253			14,422,500

Table H–6 Capital dredging volumes basins, alternative 3

Appendix H.3.4 Alternative 3: Maintenance volumes

Over the total nautical area used for port operations (the basins and the access channel) an average accretion of 1.5 metres per year is measured today. The same accretion is assumed to occur in the future so the maintenance volume is proportional to the nautical area. The values and costs per year can be found in the table below. Maintenance works will be done by Trailing Suction Hopper Dredgers, and unit price is estimated to be \$2.00 / m^3

Area name	Area	Volume (m^3/year)	Price (M\$/year)
Basins	197 ha	2,955,000	5.91
Eastern channel	56 ha	840,000	1.68
Access channel	242 ha	3,630,000	7.26
Total		7,425,000	14.85

Table H–7 Annual maintenance volumes and costs, alternative 3

Appendix H.4 DATA-SHEETS ALTERNATIVE 2

Alt. 2: Modified AGP-Proposal: Summary of costs

									REMARKS
QUAY AND BERTHS									
Number of berths	13								
Quay length	3.600	m	$	33.000,0	118,80	M$			Estimate
SUBTOTAL QUAY							118,80	M$	
RECLAMATION									
Total area	259	ha							
Volume (-3 to +6.25)	23.957.500	m3							Prelim. Calculation
Alluvial deposits	6.101.850	m3			20,61	M$			Prelim. Calculation
Tosca	27.402.356	m3			50,31	M$			Prelim. Calculation
Sand	19.390.441	m3	$	2,25	43,63	M$			Prelim. Calculation
Vertical drains	259	ha	$	40.000,0	10,36	M$			Prelim. Calculation
SUBTOTAL RECLAMATION							124,91	M$	
SHORE PROTECTION									
Protection Riverside	6.850	m	$	3.000,0	20,55				
SUBTOTAL SHORE PROTECTION							20,55	M$	
BREAKWATER CONSTRUCTION									
Operational costs	500	m	$	2.500,0	1,25	M$			Estimate
Material costs	500	m	$	7.500,0	3,75	M$			Estimate
SUBTOTAL BREAKWATER CONSTRUCTION							5,00	M$	
REMOVAL OF BREAKWATER									
Removal of short breakwater	1.150	m		p.m.					
Removal of large breakwater	2.815	m		p.m.					
TRANSPORT CORRIDOR									
Road	6.500	m	$	3.000,0	19,50	M$			Estimate
Railroad					p.m.				
SUBTOTAL TRANSPORT CORRIDOR							19,50	M$	
CAPITAL DREDGING BASINS + ACCESS CHANNEL									
Basins	11.560.450	m3	$	3,50	40,46	M$			Prelim. Calculation
Hopper	8.900.000	m3	$	2,00	17,80	M$			Prelim. Calculation
Cutter + Hopper	12.100.000	m3	$	4,50	54,45	M$			Prelim. Calculation
SUBTOTAL BASIN DREDGING							112,71	M$	
TOTAL CONSTRUCTION COSTS ALTERNATIVE 2							**401,47**	**M$**	
ANNUAL MAINTENANCE									
Basins	4.041.000	m3	$	2,00	8,08	M$			Prelim. Calculation
Access Channel	3.630.000	m3	$	2,00	7,26	M$			Prelim. Calculation
							15,34	M$	
Total costs incl 20 year maintenance							***708,31***	***M$***	
Costs per m2 reclamation							***273,48***	***$***	
Costs per m2 terminal							***414,22***	***$***	

CALCULATIONS ALTERNATIVE 2

BASINS	area		depth	volume	
Basins A to E	37,8	ha	-9,75	-	
NW-SE (95 ha)	47,5	ha	-9,75	1.638.750	m3
	47,5	ha	-4,00	4.370.000	m3
N-S (35,6 ha)	17,8	ha	-9,75	614.100	m3
	17,8	ha	-3,00	1.815.600	m3
Turning circle+access in port	26	ha	-11,00	572.000	m3
Antepuerto	25	ha	-3,00	2.550.000	m3
Antepuerto (not used)	50	ha	-5,00	-	
Capital dredging works					
All Basins, turning circle, channel in port				11.560.450	m3
Maintenance				4.041.000	m3

ACCESS CHANNEL

Length	11.000 m	(in port 1100 m)
Depth	13,2 m	
Width	220 m	
Current Depth	9,75 m	AGP Plans
Current Width	100 m	AGP Plans

Capital Dredging works	
Hopper	8.900.000 m3
Cutter + Hopper	12.100.000 m3
Total Capital Dredging	21.000.000 m3
Maintenance Channel	3.630.000 m3

Appendix H.5 DATA-SHEETS ALTERNATIVE 3

Alt. 3: Compact Island: Summary of costs

								REMARKS
QUAY AND BERTHS								
Number of berths	13							
Quay length in port	2.200	m	$	33.000,0		72,60 M$		Estimate
Quay length exposed	1.375	m	$	33.000,0		45,38 M$		Estimate
SUBTOTAL QUAY							117,98 M$	
RECLAMATION								
Total area	259	ha						
Volume (-3 to +6.25)	23.957.500	m3						Prelim. Calculation
Alluvial deposits	6.101.850	m3				20,61 M$		Prelim. Calculation
Tosca	18.305.550	m3				50,31 M$		Prelim. Calculation
Sand	19.390.441	m3	$	2,25		43,63 M$		Prelim. Calculation
Vertical drains	259	ha	$	40.000,0		10,36 M$		Prelim. Calculation
SUBTOTAL RECLAMATION							124,91 M$	
SHORE PROTECTION								
Protection Riverside	3.680	m	$	3.000,0		11,04		Estimate
SUBTOTAL SHORE PROTECTION							11,04 M$	
BREAKWATER CONSTRUCTION								
Operational costs	250	m	$	2.500,0		0,63 M$		Estimate
Material costs	250	m	$	7.500,0		1,88 M$		Estimate
SUBTOTAL BREAKWATER CONSTRUCTION							2,50 M$	
REMOVAL OF BREAKWATER								
Removal of short breakwater	600	m			p.m.			
Removal of large breakwater	2.815	m			p.m.			Estimate
TRANSPORT CORRIDOR								
Road	5.500	m	$	3.000,0		16,50 M$		
Railroad						p.m.		Estimate
SUBTOTAL TRANSPORT CORRIDOR							16,50 M$	
CAPITAL DREDGING BASINS + ACCESS CHANNEL								
Basins	5.752.500	m3	$	3,5		20,13 M$		Prelim. Calculation
Eastern Channel	2.656.000	m3	$	3,5		9,30 M$		Prelim. Calculation
Access Channel hopper	8.900.000	m3	$	2,0		17,80 M$		Prelim. Calculation
Access Channel cutter & hopper	12.100.000	m3	$	4,5		54,45 M$		Prelim. Calculation
SUBTOTAL BASIN DREDGING							101,68 M$	
TOTAL CONSTRUCTION COSTS ALTERNATIVE 3							**374,60 M$**	
ANNUAL MAINTENANCE								
Basin	2.955.000	m3	$	2,0		5,91 M$		Prelim. Calculation
Eastern Channel	810.000	m3	$	2,0		1,62 M$		Prelim. Calculation
Access Channel	3.630.000	m3	$	2,0		7,26 M$		Prelim. Calculation
							14,79 M$	
Total costs incl 20 year maintenance							***670,40 M$***	
Costs per m2 reclamation							***258,84 $***	
Costs per m2 terminal							***354,71 $***	

CALCULATIONS ALTERNATIVE 3

BASINS

Newly dredged channel	54	ha	-10,00	1.728.000 m3
Basins A to E	29	ha	-9,75	-
Access to new channel	29	ha	-10,00	928.000 m3
N-S Old Port (15 ha)	9	ha	-9,75	310.500 m3
	6	ha	-3,50	582.000 m3
"Triangle" Old port (52 ha)	20,8	ha	-9,75	717.600 m3
	31,2	ha	-3,00	3.182.400 m3
Turning Circle old + access	20	ha	-11,00	440.000 m3
Antepuerto	10	ha	-8,00	520.000 m3
Antepuerto (not used)	42	ha	-5,00	-

Capital dredging works			
All Basins, turning circle, channel in port	5.752.500	m3	
Eastern Channel	2.656.000	m3	
Total capital dredging works	8.408.500	m3	
Maintenance eastern channel	810.000	m3	1,5 m / year (estimate)
Maintenance basins	2.955.000	m3	1,5 m / year (estimate)

ACCESS CHANNEL

Length	11.000 m	(in port 900 m)
Depth	13,2 m	
Width	220 m	
Current Depth	9,75 m	AGP Plans
Current Width	100 m	AGP Plans

Capital Dredging works		
Hopper	8.900.000 m3	
Cutter + Hopper	12.100.000 m3	
Total Capital Dredging	21.000.000 m3	
Maintenance	3.630.000 m3	1,5 m / year (estimate)

APPENDIX I. FLOW NETS

Appendix I.1 FLOOD STREAM

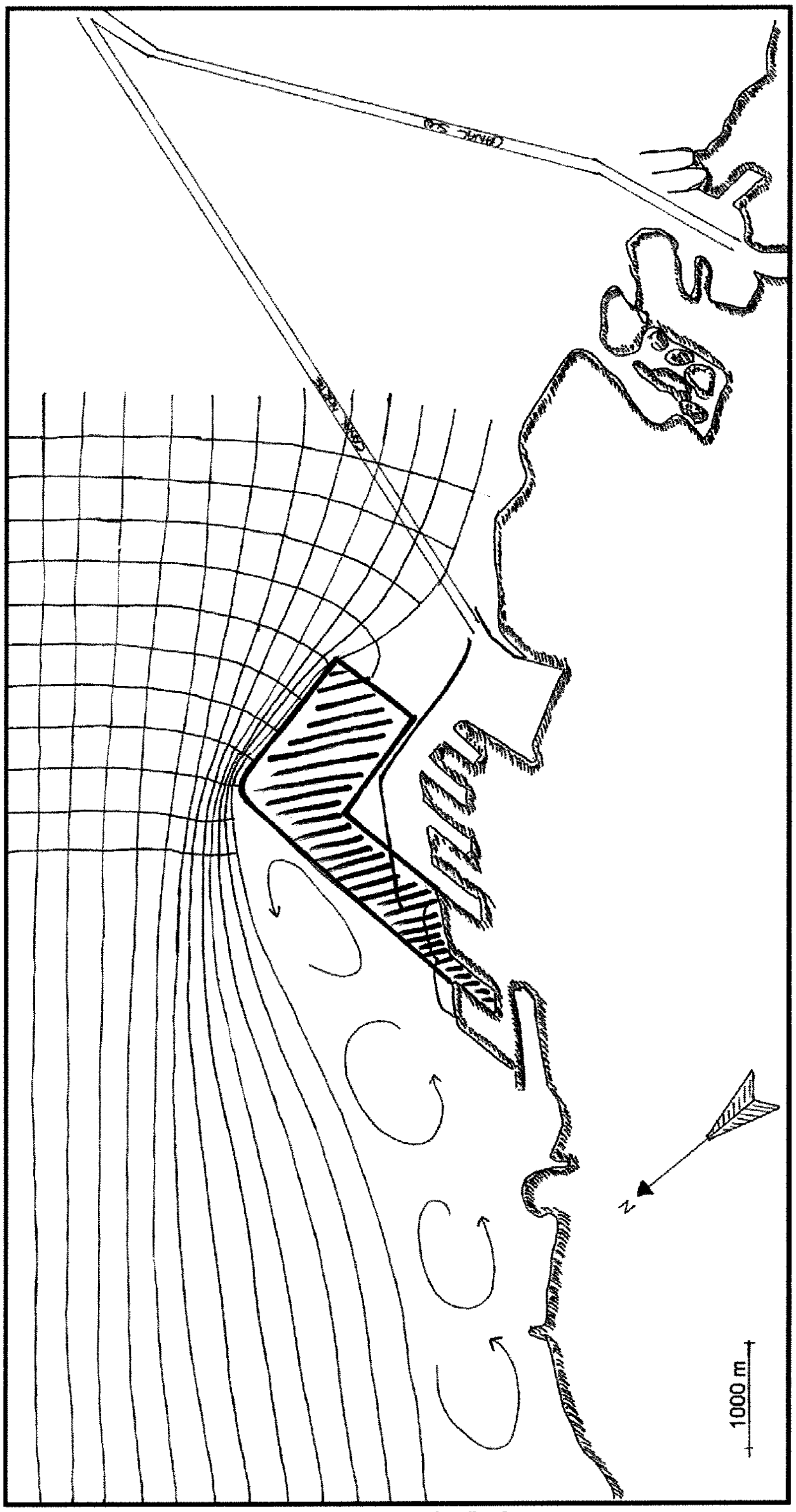


Figure I–1 Flow net for flood stream

Appendix I.2 EBB STREAM

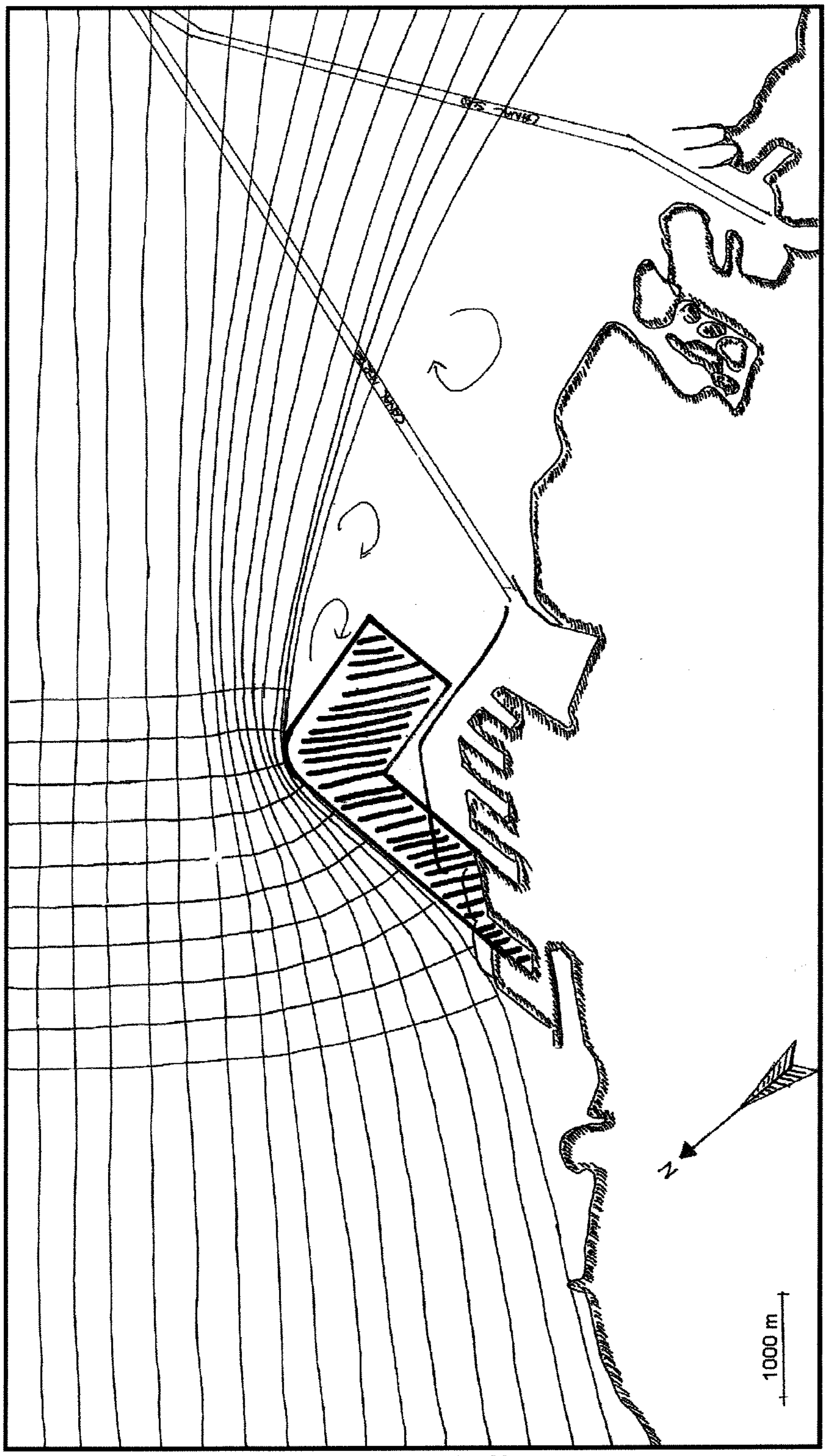
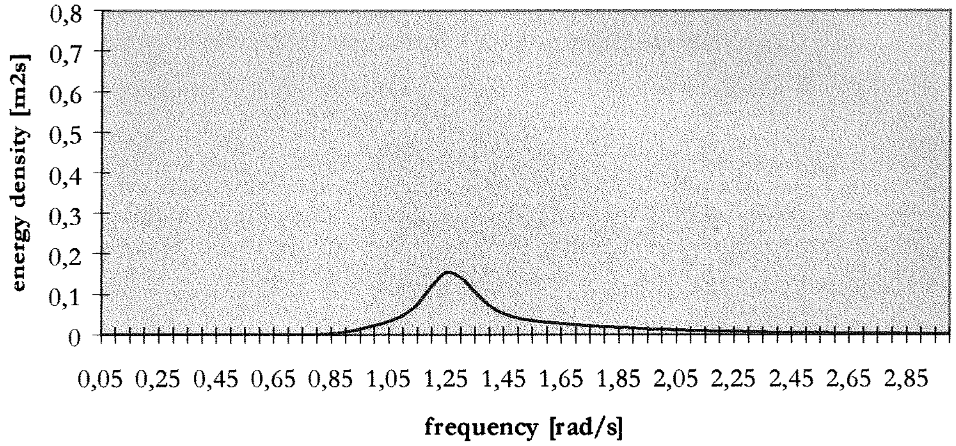


Figure I–2 Flow net for ebb stream

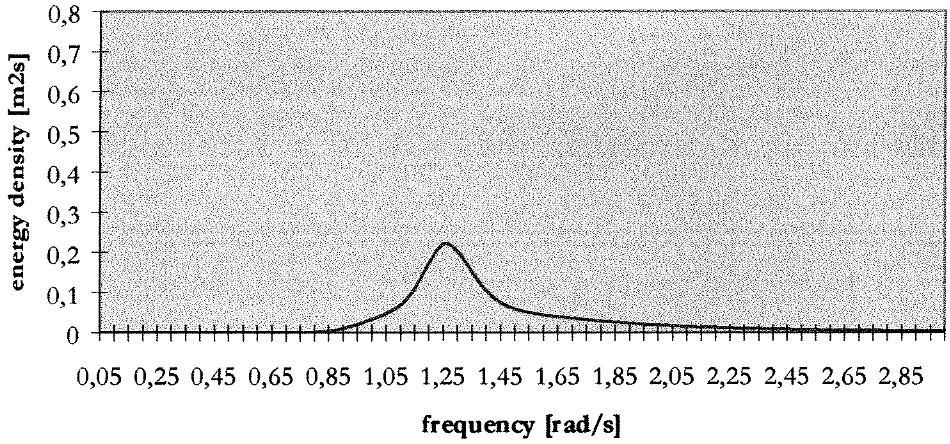
APPENDIX J. SHIP MOTIONS AT QUAY

Appendix J.1 WAVE SPECTRA

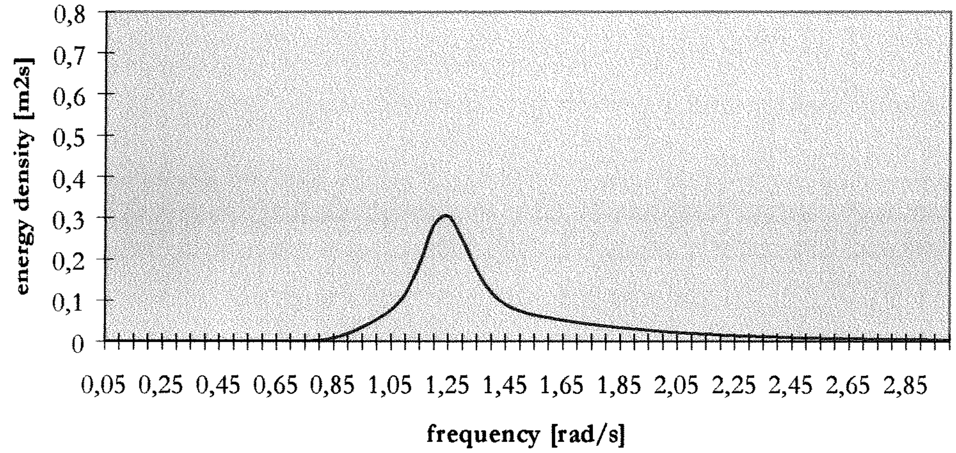
Jonswap Spectrum, Hs= 1.0 m, Tp= 5.0 s



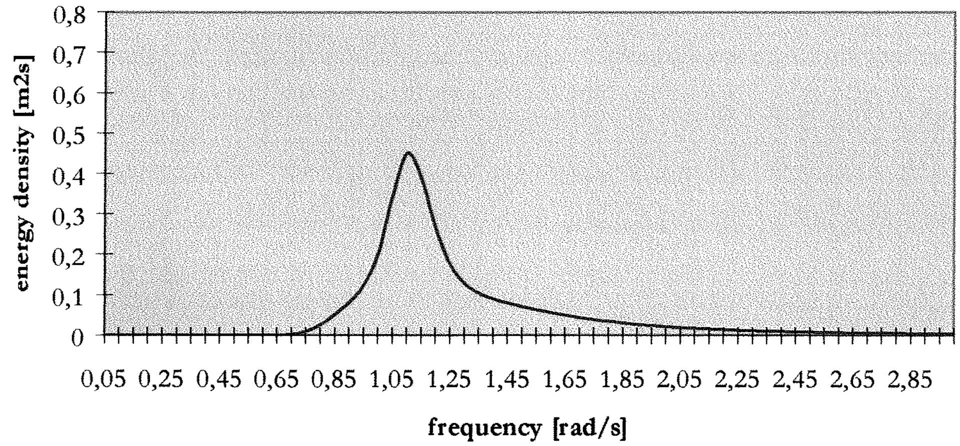
Jonswap Spectrum, Hs= 1.2 m, Tp= 5.0 s



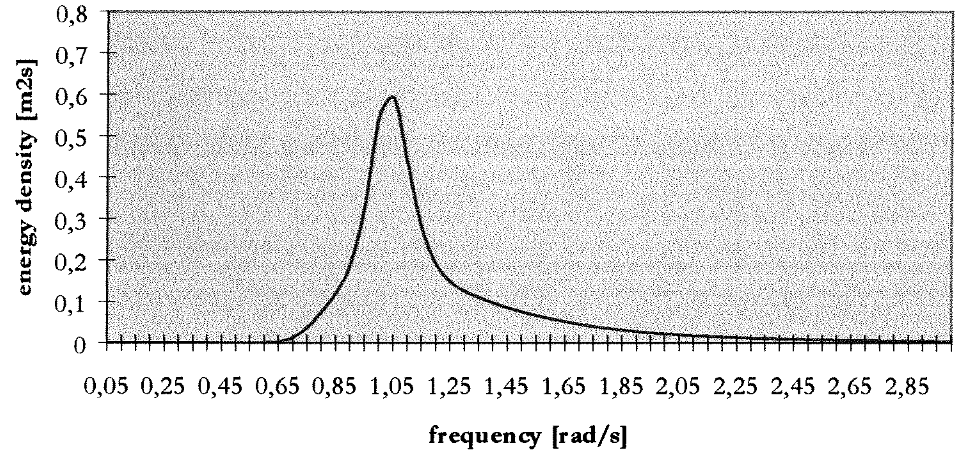
Jonswap Spectrum, Hs= 1.4 m, Tp = 5.1 s



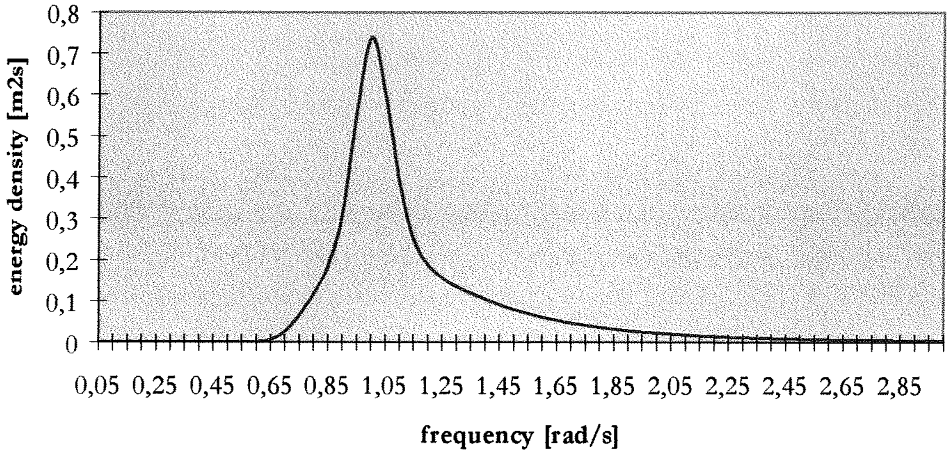
Jonswap Spectrum, Hs= 1.6 m, Tp= 5.7 s



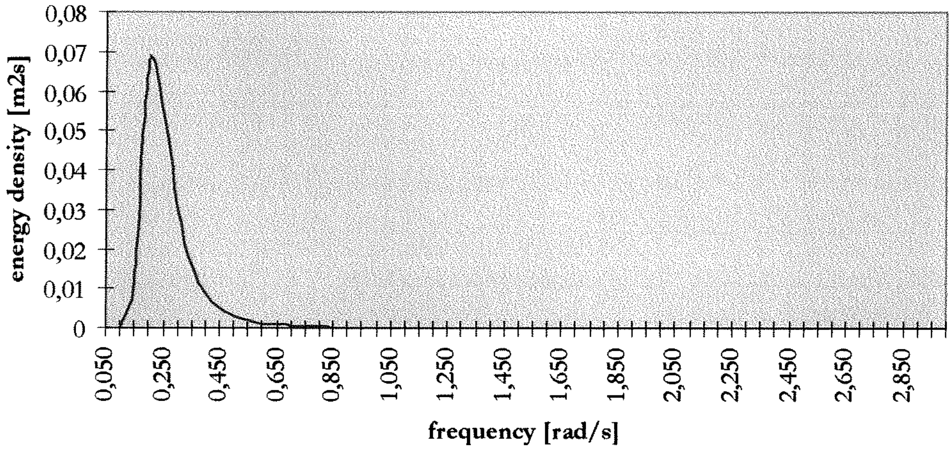
Jonswap Spectrum, Hs= 1.8 m, Tp = 6.1 s



Jonswap Spectrum, Hs= 1.95 m, Tp= 6.3 s



Bretschneider Spectrum, Hs = 0.40 m, Tp = 30 s



Appendix J.2 Response of Amplitude (ROA) functions for Panamax containership at berth for different wave directions

On the following pages:

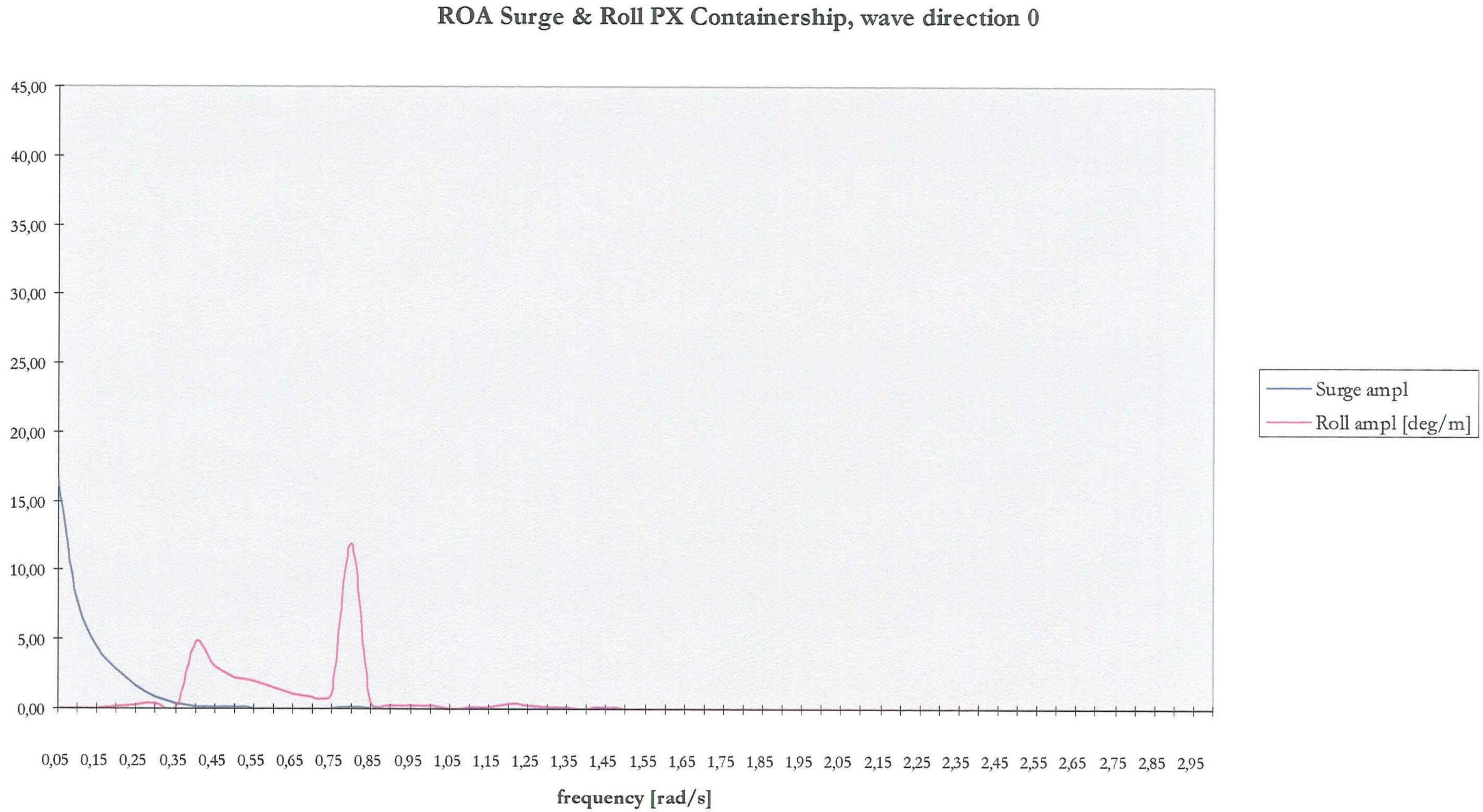
Wave direction: 0
ROA Surge & Roll
ROA Sway, Heave, Pitch and Yaw

Wave direction: 45
ROA Surge & Roll
ROA Sway, Heave, Pitch and Yaw

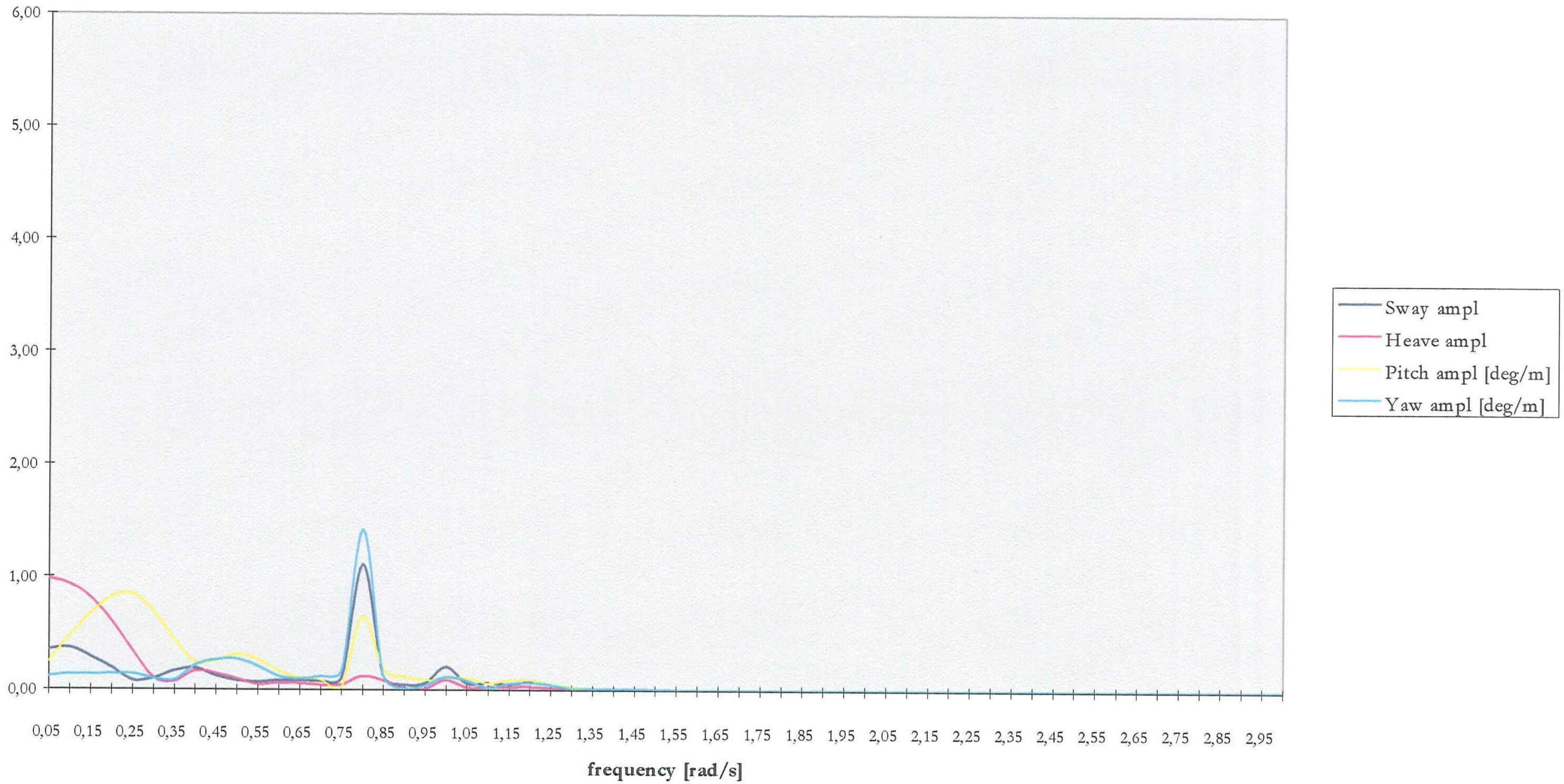
Wave direction: 90
ROA Roll
ROA Surge, Sway, Heave, Pitch and Yaw

Wave direction: 135
ROA Surge & Roll
ROA Sway, Heave, Pitch and Yaw

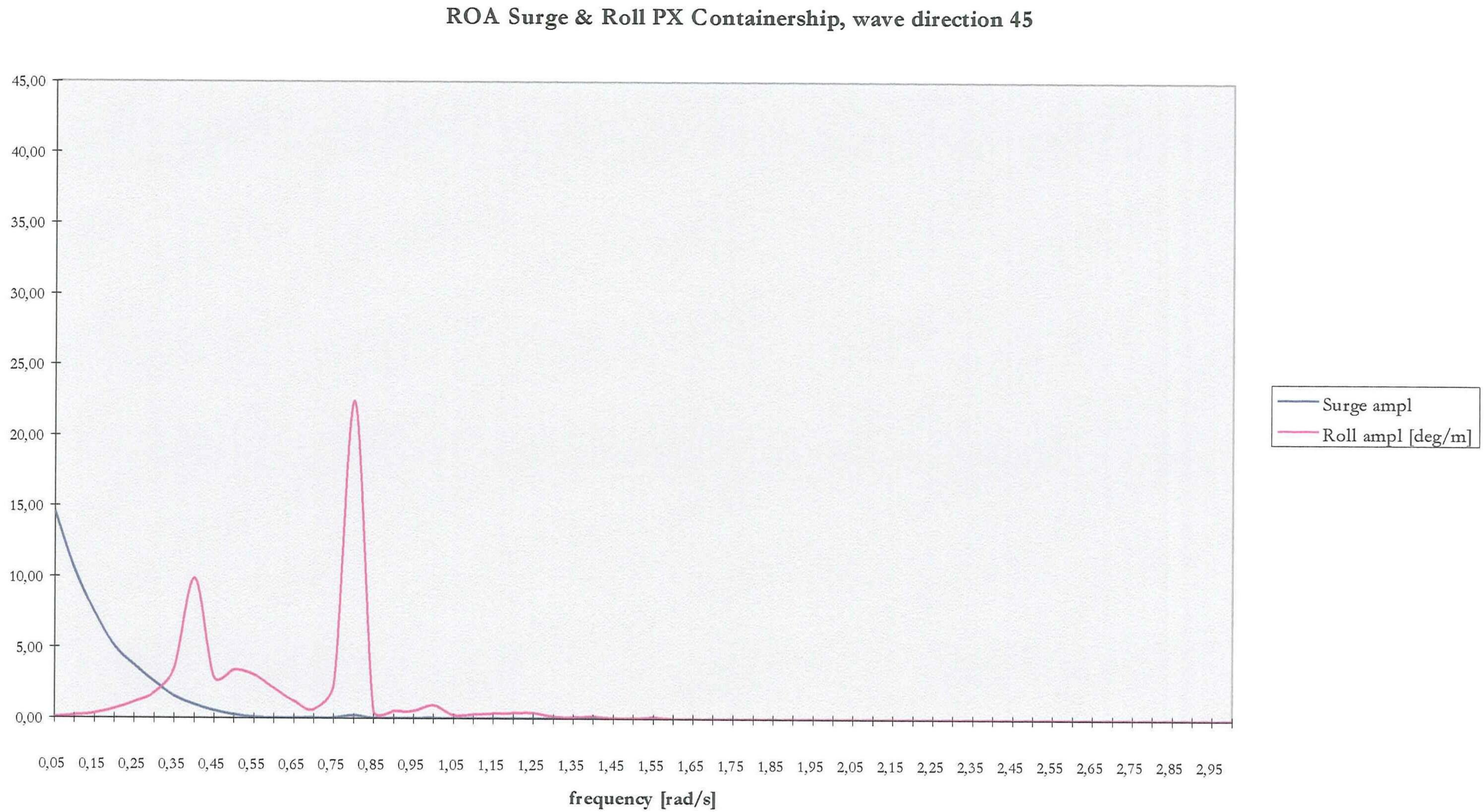
Wave direction: 180
ROA Surge & Roll
ROA Sway, Heave, Pitch and Yaw


ROA Surge & Roll PX Containership, wave direction 0
45,00
40,00
35,00
30,00
25,00
20,00
15,00
10,00
5,00
0,00
0,05 0,15 0,25 0,35 0,45 0,55 0,65 0,75 0,85 0,95 1,05 1,15 1,25 1,35 1,45 1,55 1,65 1,75 1,85 1,95 2,05 2,15 2,25 2,35 2,45 2,55 2,65 2,75 2,85 2,95
frequency [rad/s]
Surge ampl
Roll ampl [deg/m]

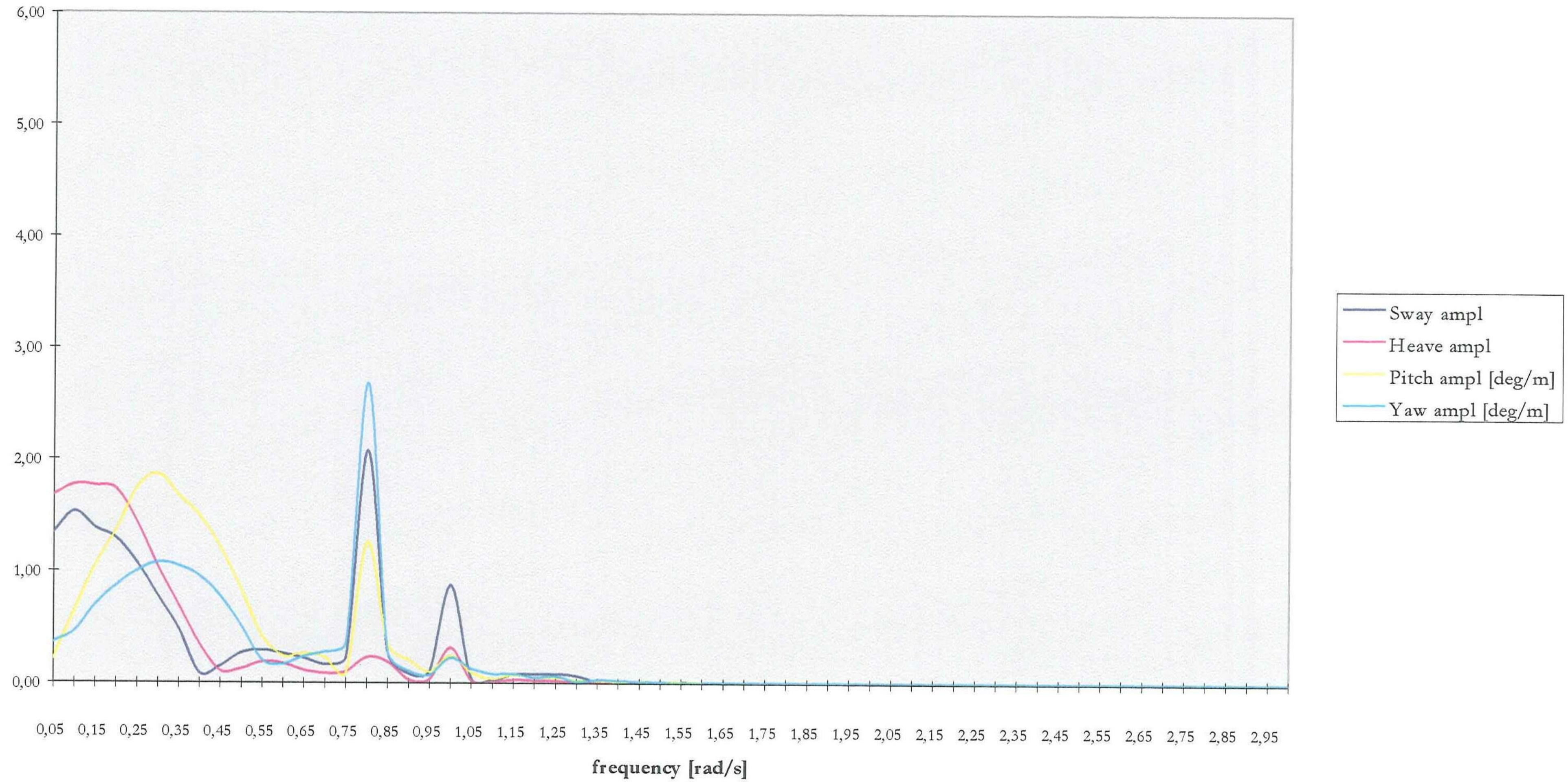
ROA Sway, Heave, Pitch and Yaw PX Containership, wave direction 0

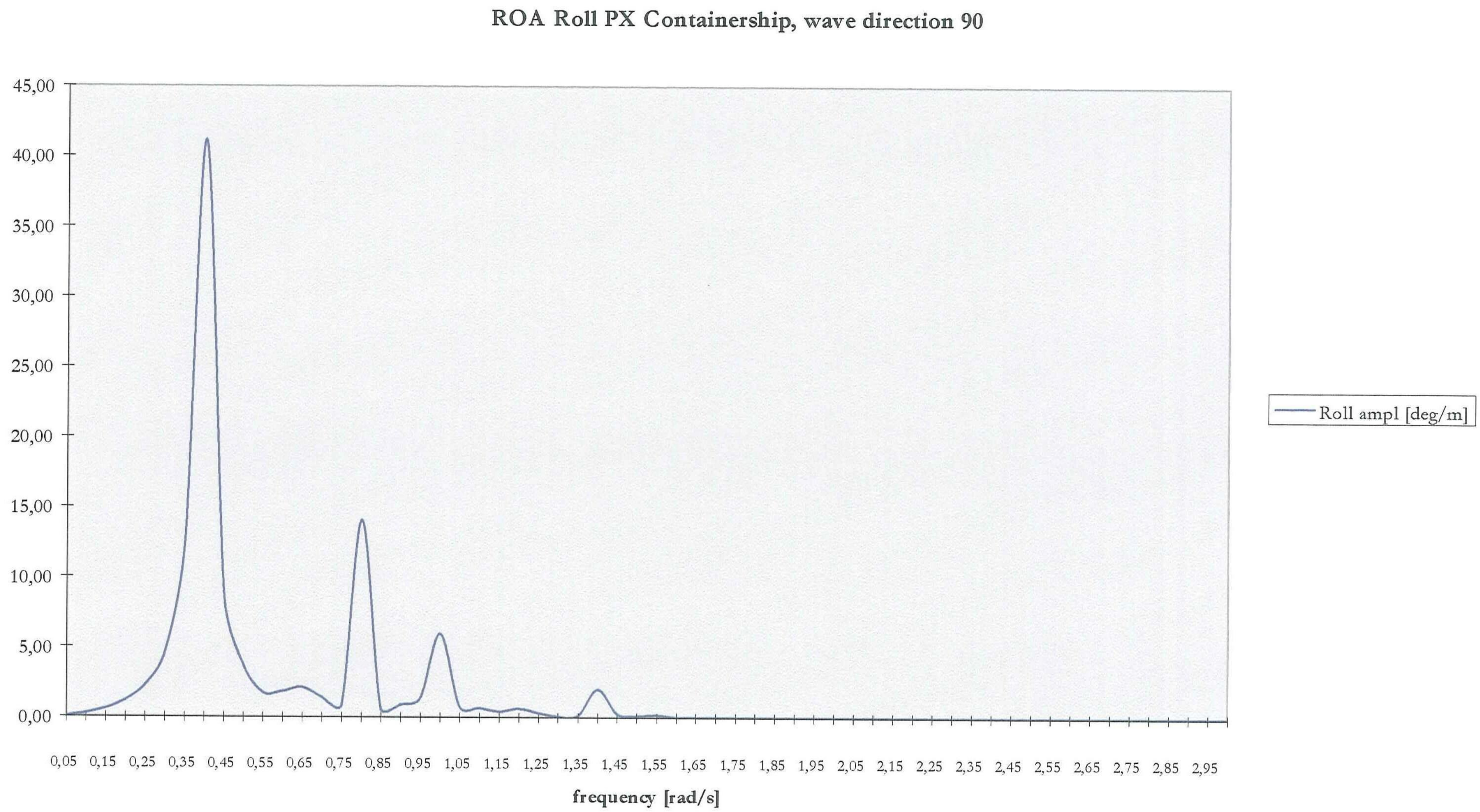


Ballast Nedam Dredging

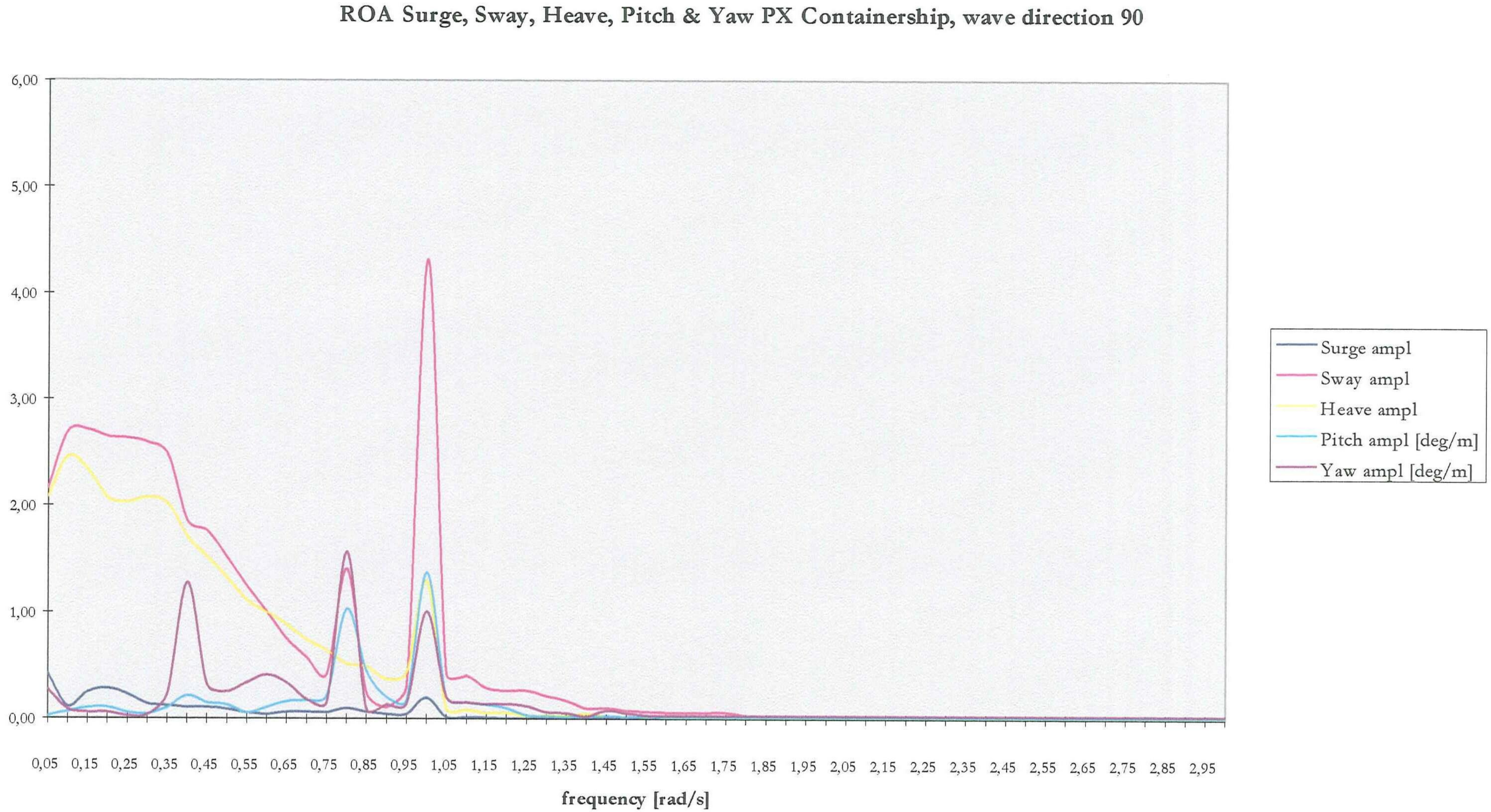

ROA Surge & Roll PX Containership, wave direction 45
45,00
40,00
35,00
30,00
25,00
20,00
15,00
10,00
5,00
0,00
0,05 0,15 0,25 0,35 0,45 0,55 0,65 0,75 0,85 0,95 1,05 1,15 1,25 1,35 1,45 1,55 1,65 1,75 1,85 1,95 2,05 2,15 2,25 2,35 2,45 2,55 2,65 2,75 2,85 2,95
frequency [rad/s]
Surge ampl
Roll ampl [deg/m]

ROA Sway, Heave, Pitch & Yaw PX Containership, wave direction 45

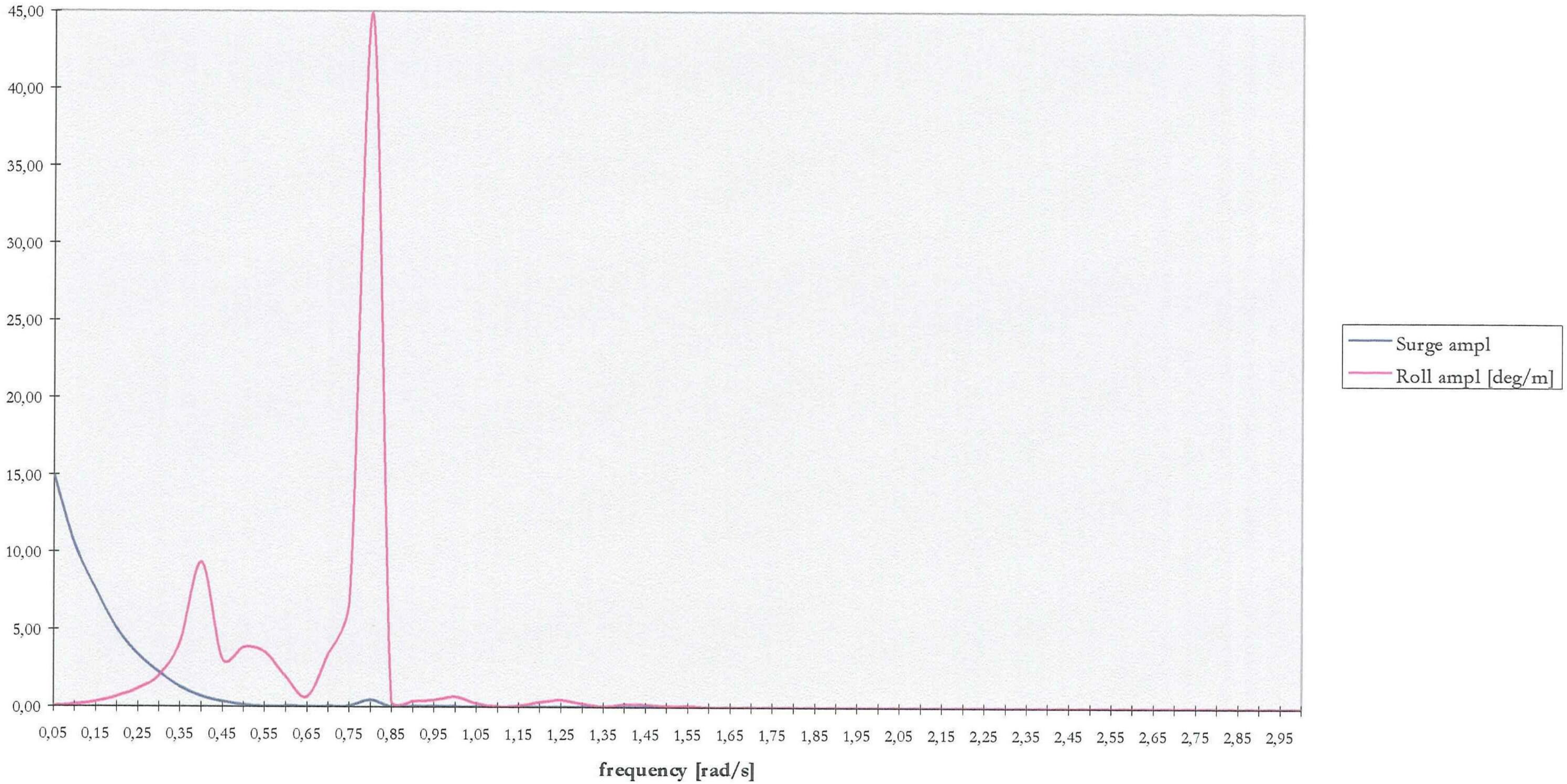




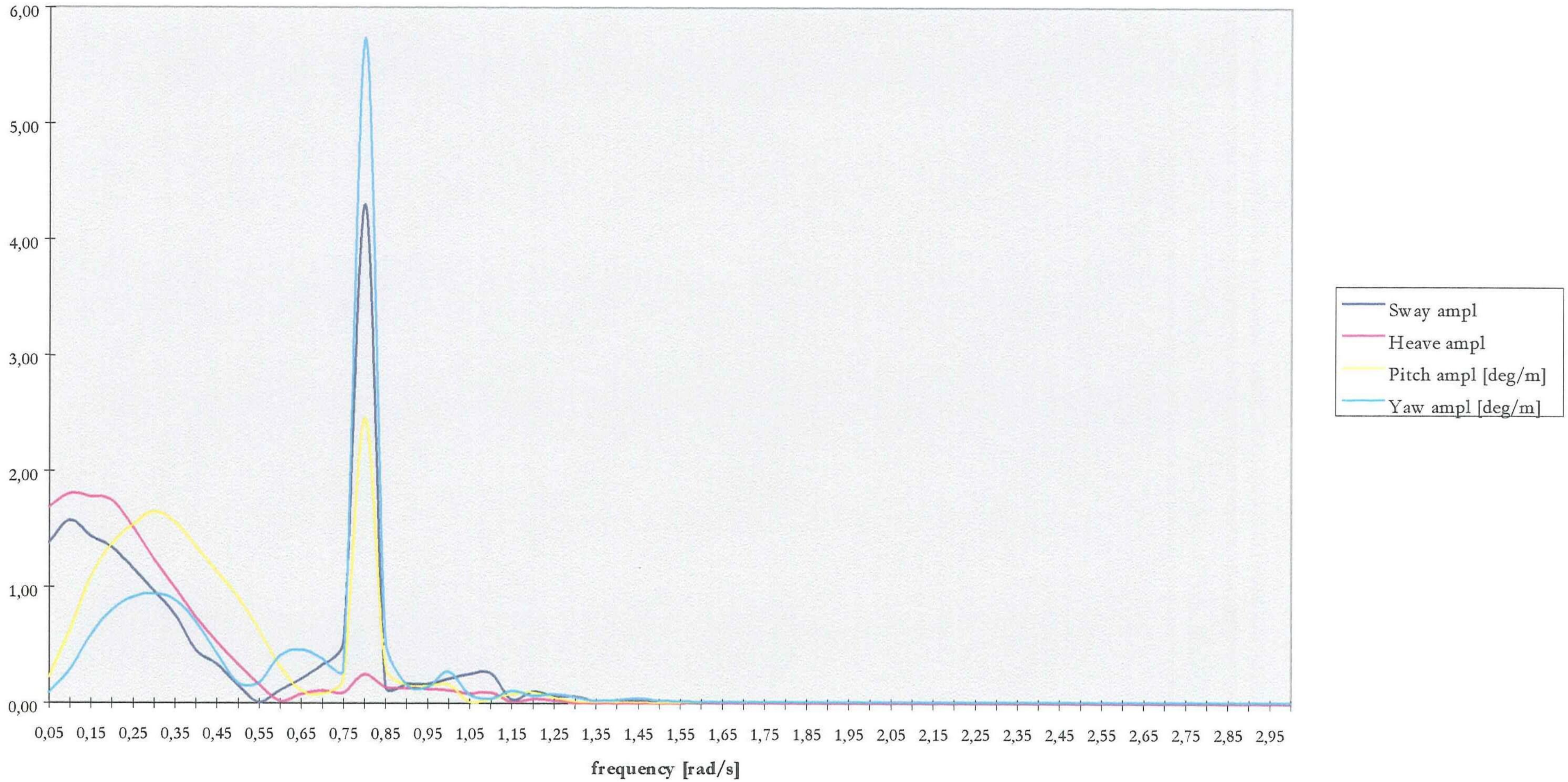
Ballast Nedam Dredging


ROA Surge, Sway, Heave, Pitch & Yaw PX Containership, wave direction 90
6,00
5,00
4,00
3,00
2,00
1,00
0,00
0,05 0,15 0,25 0,35 0,45 0,55 0,65 0,75 0,85 0,95 1,05 1,15 1,25 1,35 1,45 1,55 1,65 1,75 1,85 1,95 2,05 2,15 2,25 2,35 2,45 2,55 2,65 2,75 2,85 2,95
frequency [rad/s]
Surge ampl
Sway ampl
Heave ampl
Pitch ampl [deg/m]
Yaw ampl [deg/m]

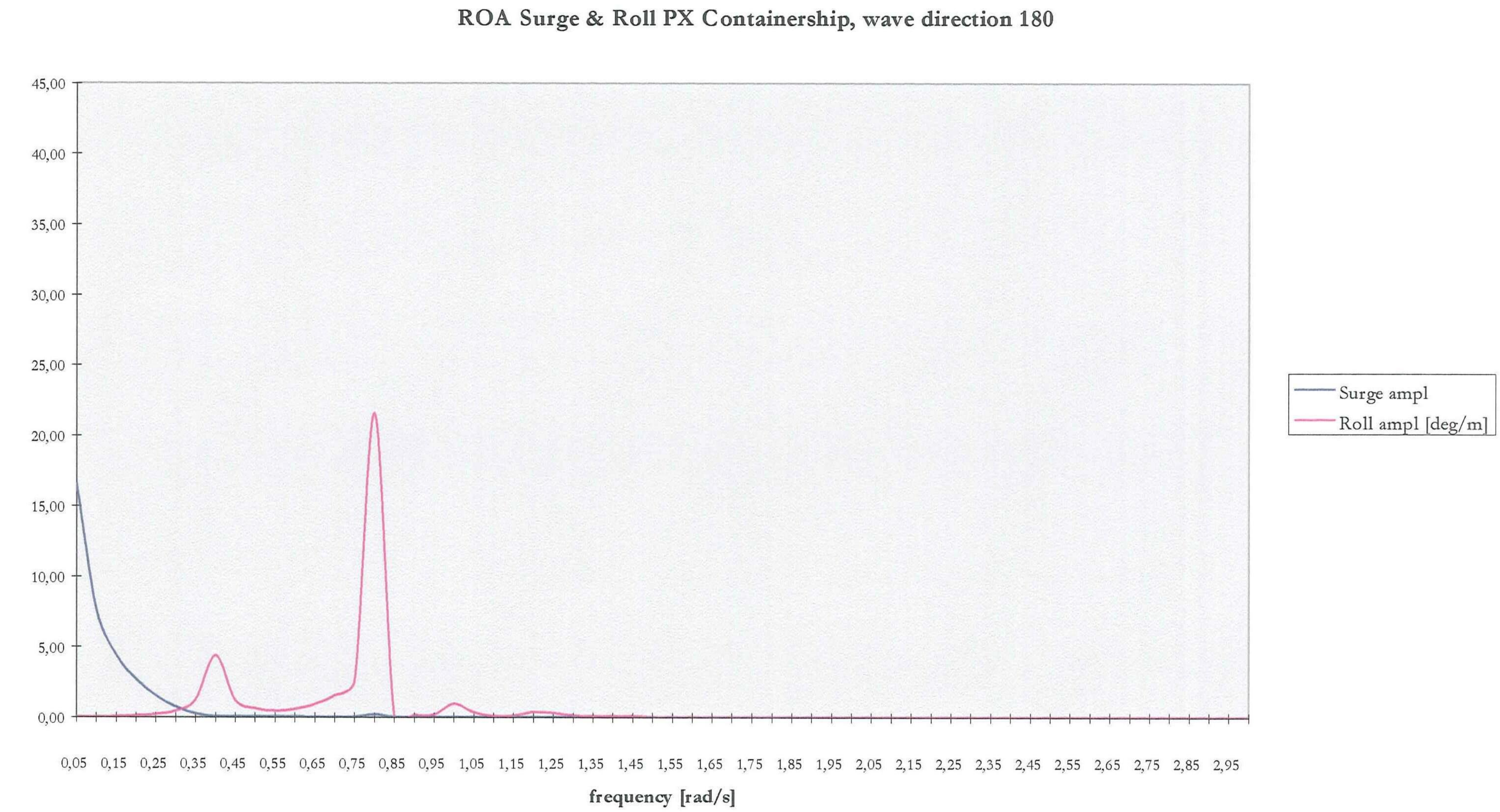
ROA Surge and Roll PX Containership, wave direction 135



ROA Sway, Heave, Pitch & Yaw PX Containership, wave direction 135

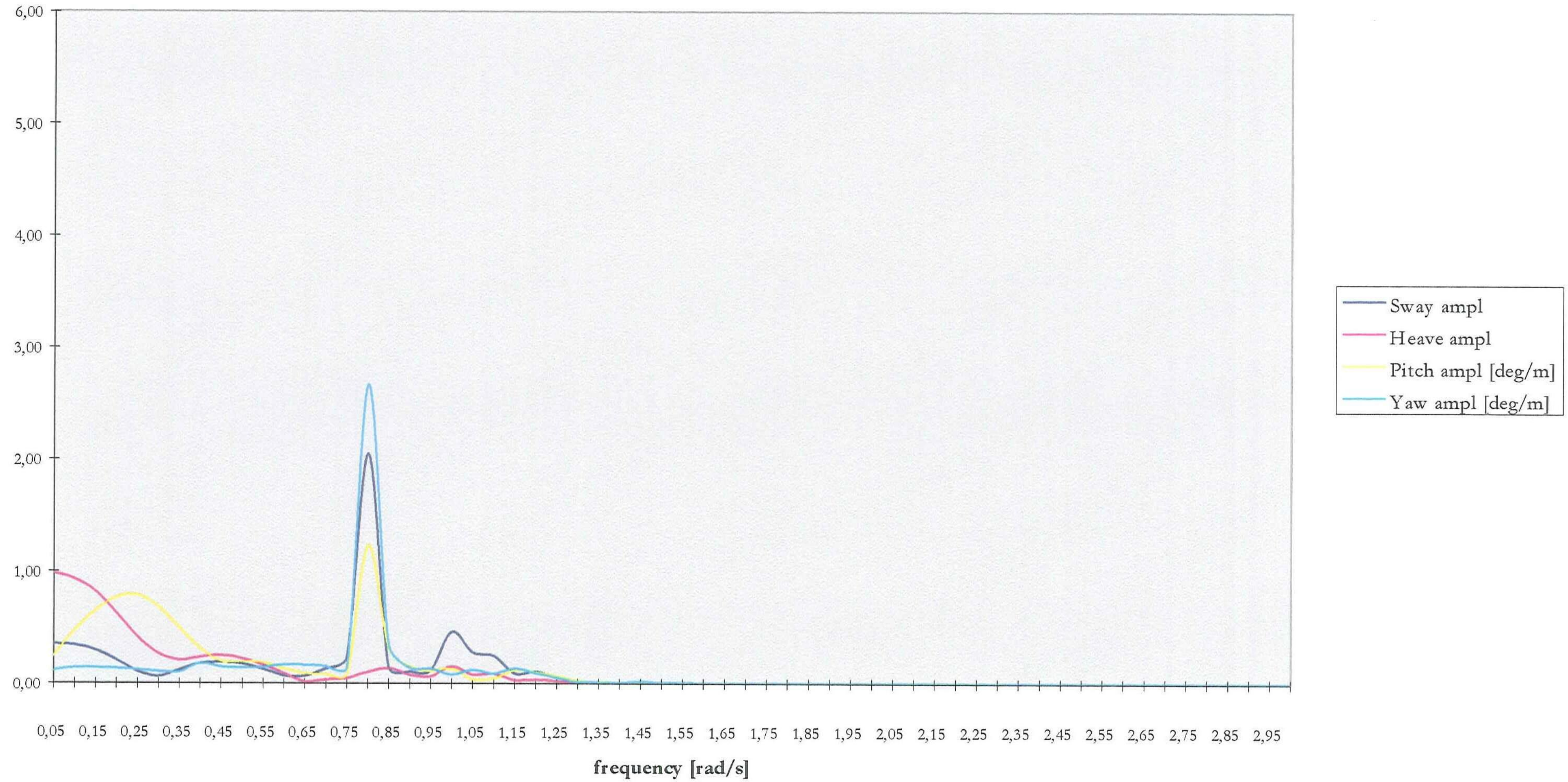


Ballast Nedam Dredging



Ballast Nedam Dredging

ROA Sway, Heave, Pitch & Yaw PX Containership, wave direction 180



Appendix J.3 MOTION AMPLITUDES FOR DIFFERENT WAVE SPECTRA

SHIP MOTIONS AT QUAY, WITH SHORT WAVES (JONSWAP SPECTRUM)

Significant Motion Amplitudes for: Hs = 1.00 m Tp = 5.0 s
Main direction: All
P_Hs 5,46%
Tot poss. storms / yr 1460
Events / yr 79,72
RP 4,58 days

	WAVE DIRECTION						MOVEMENT SMALLER THAN OPERATIONAL LIMIT						MOVEMENT SMALLER THAN SAFETY LIMIT					
	0	45	90	135	180		0	45	90	135	180	limit	0	45	90	135	180	limit
Surge	0,00	0,00	0,01	0,01	0,00	m	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,50
Sway	0,03	0,07	0,31	0,07	0,02	m	yes	yes	yes	yes	yes	0,40	yes	yes	yes	yes	yes	1,00
Heave	0,01	0,02	0,09	0,05	0,03	m	yes	yes	yes	yes	yes	0,23	yes	yes	yes	yes	yes	0,30
Roll	0,16	0,31	0,53	0,57	0,29	deg	yes	yes	yes	yes	yes	1,50	yes	yes	yes	yes	yes	3,00
Pitch	0,03	0,03	0,10	0,04	0,03	deg	yes	yes	yes	yes	yes	0,75	yes	yes	yes	yes	yes	1,25
Yaw	0,02	0,04	0,08	0,08	0,04	deg	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,75

Significant Motion Amplitudes for: Hs = 1.20 m Tp = 5.0 s
Main direction: All
P_Hs 1,06%
Tot poss. storms / yr 1460
Events / yr 15,48
RP 0,06 yrs

	WAVE DIRECTION						MOVEMENT SMALLER THAN OPERATIONAL LIMIT						MOVEMENT SMALLER THAN SAFETY LIMIT					
	0	45	90	135	180		0	45	90	135	180	limit	0	45	90	135	180	limit
Surge	0,00	0,01	0,02	0,01	0,00	m	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,50
Sway	0,03	0,08	0,37	0,08	0,07	m	yes	yes	yes	yes	yes	0,40	yes	yes	yes	yes	yes	1,00
Heave	0,01	0,03	0,11	0,02	0,02	m	yes	yes	yes	yes	yes	0,23	yes	yes	yes	yes	yes	0,30
Roll	0,20	0,37	0,64	0,68	0,35	deg	yes	yes	yes	yes	yes	1,50	yes	yes	yes	yes	yes	3,00
Pitch	0,03	0,04	0,12	0,05	0,04	deg	yes	yes	yes	yes	yes	0,75	yes	yes	yes	yes	yes	1,25
Yaw	0,03	0,05	0,10	0,09	0,05	deg	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,75

Significant Motion Amplitudes for: Hs = 1.40 m Tp = 5.1 s
Main direction: E-SW
P_Hs 0,16%
Tot poss. storms / yr 1460
Events / yr 2,34
RP 0,43 yrs

	WAVE DIRECTION						MOVEMENT SMALLER THAN OPERATIONAL LIMIT						MOVEMENT SMALLER THAN SAFETY LIMIT					
	0	45	90	135	180		0	45	90	135	180	limit	0	45	90	135	180	limit
Surge	0,00	0,01	0,02	0,01	0,01	m	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,50
Sway	0,04	0,11	0,47	0,12	0,09	m	yes	yes	no	yes	yes	0,40	yes	yes	yes	yes	yes	1,00
Heave	0,01	0,04	0,15	0,03	0,03	m	yes	yes	yes	yes	yes	0,23	yes	yes	yes	yes	yes	0,30
Roll	0,29	0,54	0,80	1,02	0,51	deg	yes	yes	yes	yes	yes	1,50	yes	yes	yes	yes	yes	3,00
Pitch	0,04	0,05	0,15	0,07	0,05	deg	yes	yes	yes	yes	yes	0,75	yes	yes	yes	yes	yes	1,25
Yaw	0,04	0,07	0,13	0,14	0,08	deg	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,75

Significant Motion Amplitudes for: Hs = 1.60 m Tp = 5.7 s
Main direction: SE-S
Pwav 0,01%
Tot poss. storms / yr 1460
Events / yr 0,15
RP 6,85 yrs

	WAVE DIRECTION						MOVEMENT SMALLER THAN OPERATIONAL LIMIT						MOVEMENT SMALLER THAN SAFETY LIMIT					
	0	45	90	135	180		0	45	90	135	180	limit	0	45	90	135	180	limit
Surge	0,01	0,02	0,04	0,03	0,02	m	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,50
Sway	0,10	0,24	0,90	0,33	0,21	m	yes	yes	no	yes	yes	0,40	yes	yes	yes	yes	yes	1,00
Heave	0,02	0,07	0,28	0,05	0,05	m	yes	yes	no	yes	yes	0,23	yes	yes	yes	yes	yes	0,30
Roll	0,87	1,64	1,64	3,26	1,58	deg	yes	no	no	no	no	1,50	yes	yes	yes	no	yes	3,00
Pitch	0,07	0,12	0,30	0,19	0,11	deg	yes	yes	yes	yes	yes	0,75	yes	yes	yes	yes	yes	1,25
Yaw	0,11	0,21	0,25	0,42	0,21	deg	yes	yes	no	no	yes	0,25	yes	yes	yes	yes	yes	0,75

Significant Motion Amplitudes for: Hs = 1.80 m Tp = 6.1 s
Main direction: SE
P_Hs 0,0068%
Tot poss. storms / yr 1460
Events/yr 0,1
RP 10 yrs

	WAVE DIRECTION						MOVEMENT SMALLER THAN OPERATIONAL LIMIT						MOVEMENT SMALLER THAN SAFETY LIMIT					
	0	45	90	135	180		0	45	90	135	180	limit	0	45	90	135	180	limit
Surge	0,02	0,03	0,07	0,05	0,03	m	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,50
Sway	0,16	0,39	1,46	0,55	0,32	m	yes	yes	no	no	yes	0,40	yes	yes	no	yes	yes	1,00
Heave	0,04	0,11	0,47	0,08	0,07	m	yes	yes	no	yes	yes	0,23	yes	yes	no	yes	yes	0,30
Roll	1,46	2,76	2,68	5,49	2,65	deg	yes	no	no	no	no	1,50	yes	yes	yes	no	yes	3,00
Pitch	0,11	0,19	0,49	0,31	0,17	deg	yes	yes	yes	yes	yes	0,75	yes	yes	yes	yes	yes	1,25
Yaw	0,18	0,34	0,40	0,71	0,34	deg	yes	no	no	no	no	0,25	yes	yes	yes	yes	yes	0,75

Significant Motion Amplitudes for: Hs = 1.95 m Tp = 6.3 s
Main direction: SE

P_Hs	0,0014%
Tot poss. storms / yr	1460
Events/yr	0,02
RP	50 yrs

	WAVE DIRECTION						MOVEMENT SMALLER THAN OPERATIONAL LIMIT						MOVEMENT SMALLER THAN SAFETY LIMIT					
	0	45	90	135	180		0	45	90	135	180	limit	0	45	90	135	180	limit
Surge	0,02	0,04	0,08	0,07	0,03	m	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,50
Sway	0,19	0,47	1,69	0,68	0,38	m	yes	no	no	no	yes	0,40	yes	yes	no	yes	yes	1,00
Heave	0,04	0,13	0,55	0,09	0,08	m	yes	yes	no	yes	yes	0,23	yes	yes	no	yes	yes	0,30
Roll	1,84	3,47	3,22	6,92	3,34	deg	no	no	no	no	no	1,50	yes	no	no	no	no	3,00
Pitch	0,13	0,24	0,57	0,39	0,21	deg	yes	yes	yes	yes	yes	0,75	yes	yes	yes	yes	yes	1,25
Yaw	0,23	0,43	0,47	0,89	0,42	deg	yes	no	no	no	no	0,25	yes	yes	yes	no	yes	0,75

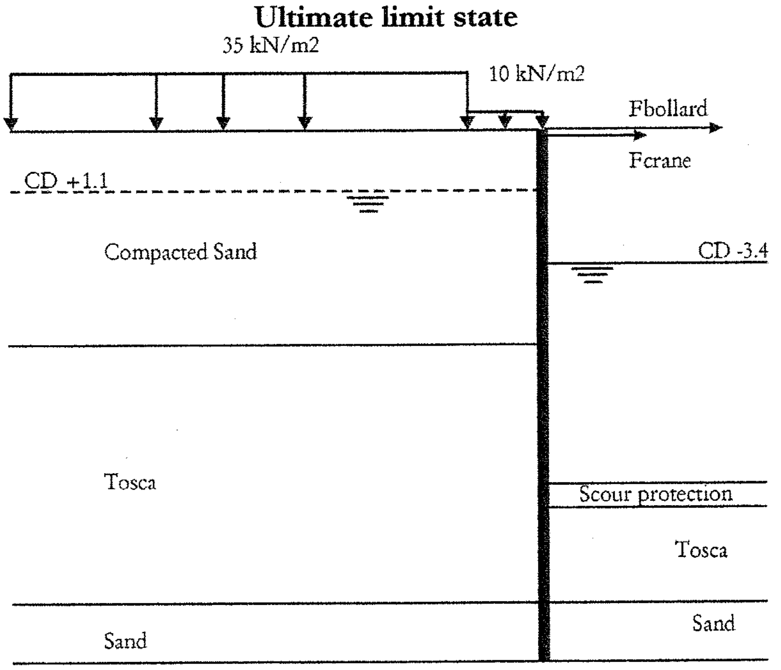
SHIP MOTIONS AT QUAY IN SWELL (AS AN EXAMPLE, NOT OCCURRING IN BUENOS AIRES)

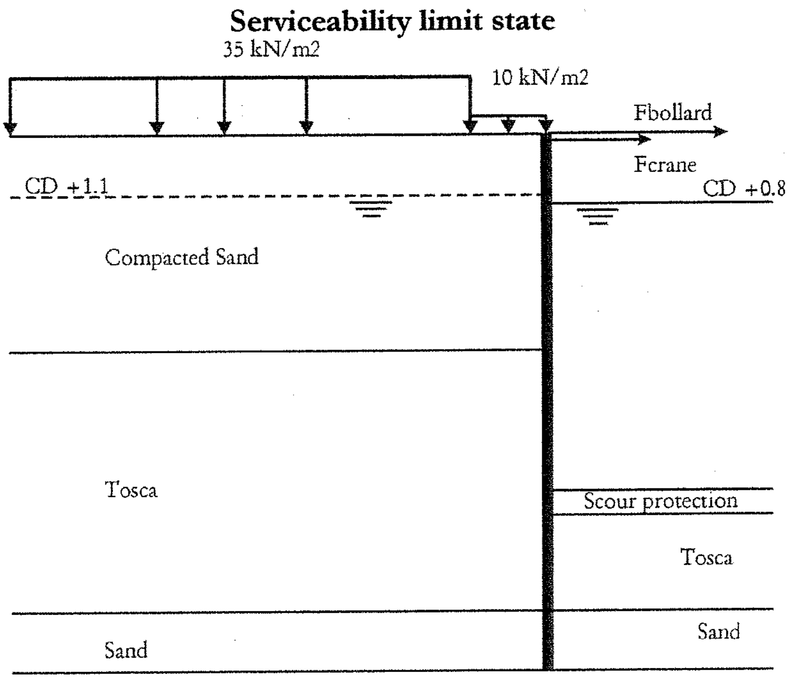
Significant Motion Amplitudes for: Hs = 0.40 m Tp = 30 s
Swell: Small swell from any direction

	WAVE DIRECTION						MOVEMENT SMALLER THAN OPERATIONAL LIMIT						MOVEMENT SMALLER THAN SAFETY LIMIT					
	0	45	90	135	180		0	45	90	135	180	limit	0	45	90	135	180	limit
Surge	0,43	0,81	0,04	0,79	0,41	m	no	no	yes	no	no	0,25	yes	no	yes	no	yes	0,50
Sway	0,03	0,21	0,50	0,23	0,04	m	yes	yes	no	yes	yes	0,40	yes	yes	yes	yes	yes	1,00
Heave	0,09	0,28	0,40	0,29	0,10	m	yes	no	no	no	yes	0,23	yes	yes	no	yes	yes	0,30
Roll	0,25	0,54	1,91	0,63	0,26	deg	yes	yes	no	yes	yes	1,50	yes	yes	yes	yes	yes	3,00
Pitch	0,15	0,31	0,02	0,29	0,14	deg	yes	yes	yes	yes	yes	0,75	yes	yes	yes	yes	yes	1,25
Yaw	0,03	0,19	0,06	0,17	0,03	deg	yes	yes	yes	yes	yes	0,25	yes	yes	yes	yes	yes	0,75

APPENDIX K. QUAY WALL CALCULATIONS

Appendix K.1 LOAD CASES





Appendix K.2 BLUM CALCULATION:PILING LEVEL

BALLAST NEDAM ENGINEERING B.V. RELEASE : 4.00

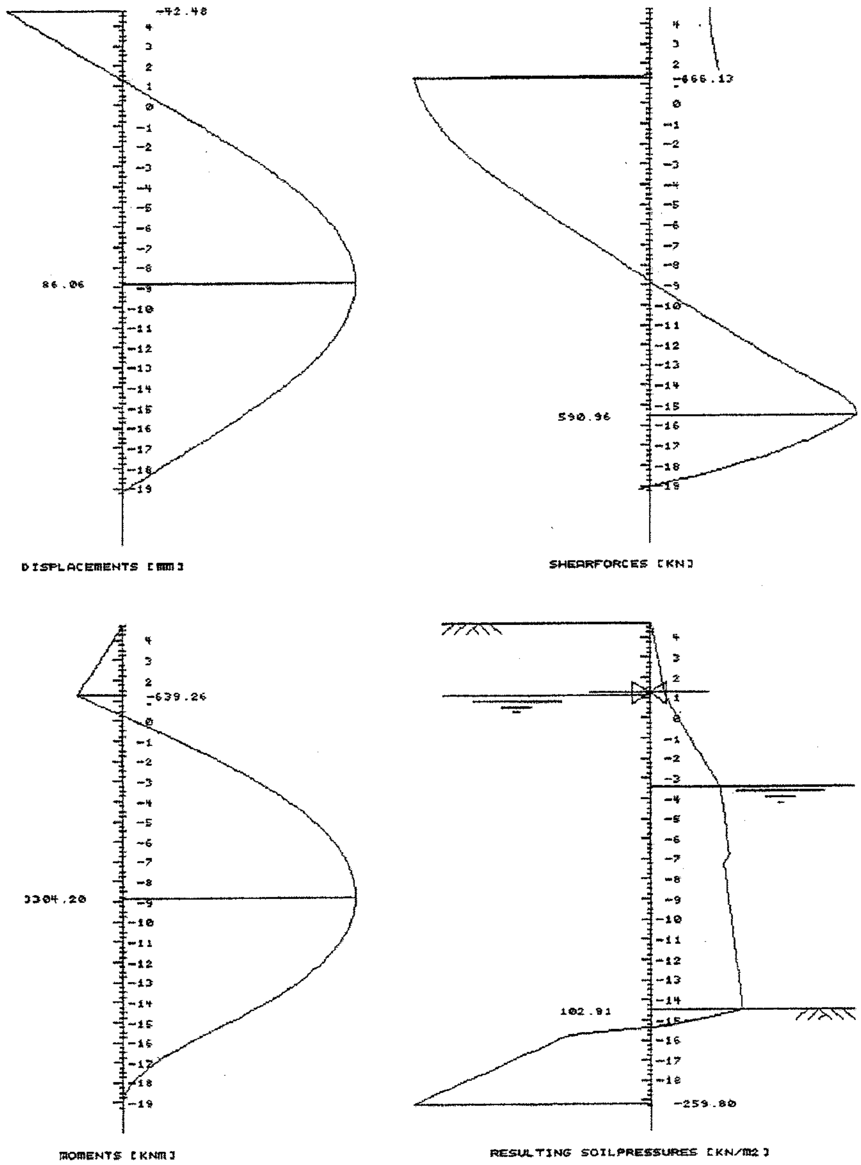
TECHNOSOFT : DAMWANDEN BLUM-BEREKENING 27 MAR100

PROJECT : BA PORT EXPANSION, QUAY WALL [REPRESENTATIVE VALUES]

SUBJECT : BLUM

PHASENR. : 1

FREE ENDED ANCHORED



ANCHORLEVEL : 1.250 FA= -870.10

BALLAST NEDAM ENGINEERING B.V. RELEASE : 4.00

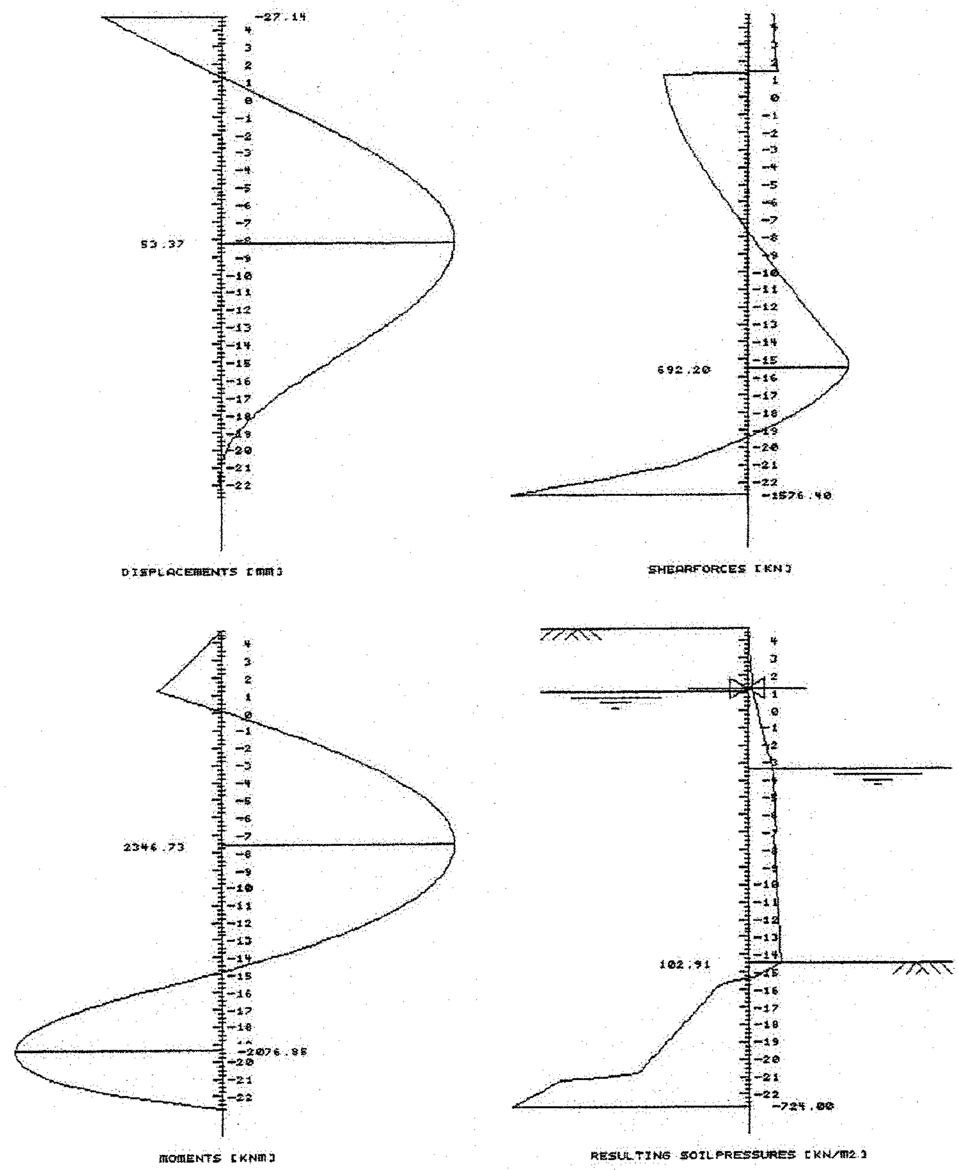
TECHNOSOFT : DAMWANDEN BLUM-BEREKENING 27 MAR100

PROJECT : BA PORT EXPANSION, QUAY WALL [REPRESENTATIVE VALUES]

SUBJECT : BLUM

PHASENR. : 1

FIXED ENDED ANCHORE



ANCHORLEVEL: 1.250 FA= -769.86

Appendix K.3 STIFFNESS OF SYSTEM

Appendix K.3.1 Blum calculation

$kN := 1000 \cdot newton$ $\quad E := 2.1 \cdot 10^8 \frac{kN}{m^2}$ $\quad F := 110 \cdot \frac{kN}{m}$

Sheetpile stiffness

The sheetpile stiffness is calculated as follows:
First an BLUM calculation of the sheetpile is made in TECHNOSOFT "Damwanden" without the bollard force and the displacement of the top op the sheetpile is noted.
Then a calculation is made with the bollard force. The difference between the displacements is equal to the force divided by the stiffness of the sheetpile.
The stiffness of the anchor is not taken into account, because a BLUM calculation is based zero movement of the anchor. (Infinite stiffness)

$u_{top0} := -20.17 \cdot mm$ $\quad u_{topbollard} := -14.52 \cdot mm$ $\quad ctc := 2.67 \cdot m$

$\Delta u := u_{topbollard} - u_{top0}$ $\quad \Delta u = 5.65 \, mm$

$k_{sheetpile} := \frac{F}{\Delta u} \cdot ctc$ $\quad k_{sheetpile} = 5.198 \cdot 10^4 \frac{kN}{m}$

Anchor stiffness

The stiffness of the anchor depends on the length and the cross section of the anchor:

$D_{anchor} := 110 \cdot mm$ $\quad l_{anchor} := 31.5 \cdot m$

$A_{anchor} := \frac{1}{4} \cdot \pi \cdot D_{anchor}^2$ $\quad A_{anchor} = 9.503 \cdot 10^{-3} \, m^2$

$k_{anchor} := \frac{E \cdot A_{anchor}}{l_{anchor}}$ $\quad k_{anchor} = 6.336 \cdot 10^4 \frac{kN}{m}$

System stiffness

$a := \frac{1}{k_{anchor}} + \frac{1}{k_{sheetpile}}$ $\quad k_{system} := \frac{1}{a}$ $\quad k_{system} = 2.855 \cdot 10^4 \frac{kN}{m}$

when this stiffness is used in the program TECHNOSOFT RAAMWERKEN, the maximum reaction force in the anchor is calculated to be 311 kN or 116 kN/m. This is about equal to the chosed 110 kN/m

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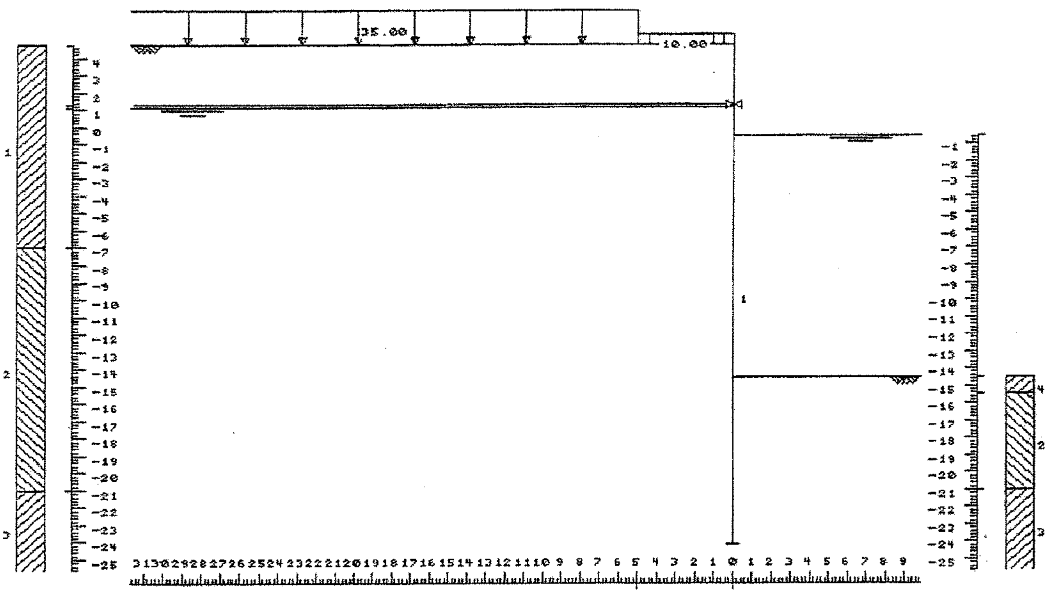
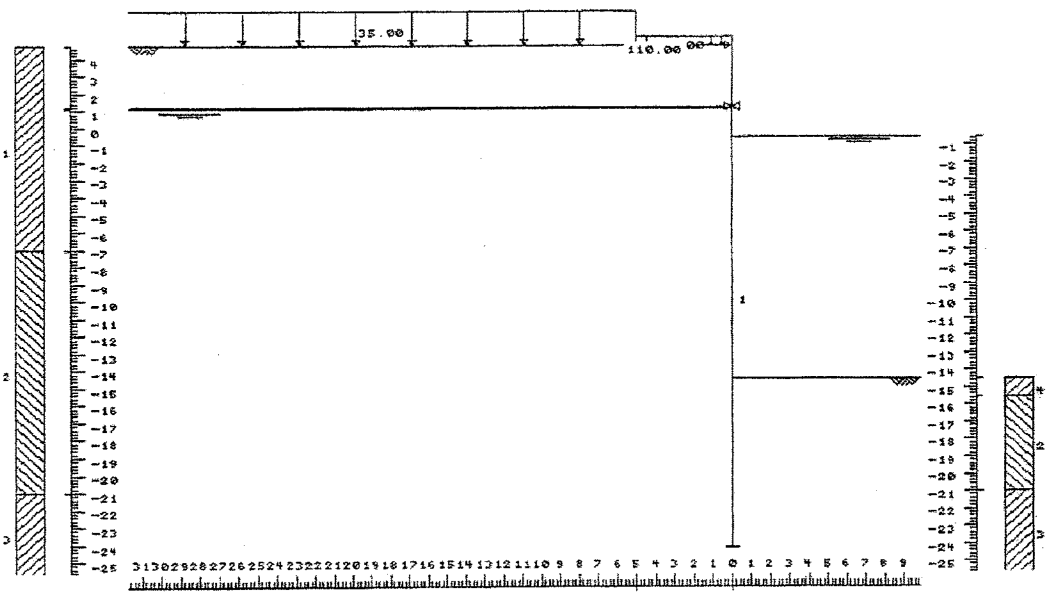
TECHNOSOFT / GRAFISCHE INVOERKONTROLE (DAMWANDEN) REL: 4.00 27 MAR XX



CONSTR. : 1 SLS WITHOUT BOLLARD FORCE



CONSTR. : 2 SLS WITH BOLLARD FORCE

BALLAST NEDAM ENGINEERING B.V. RELEASE : 4.00

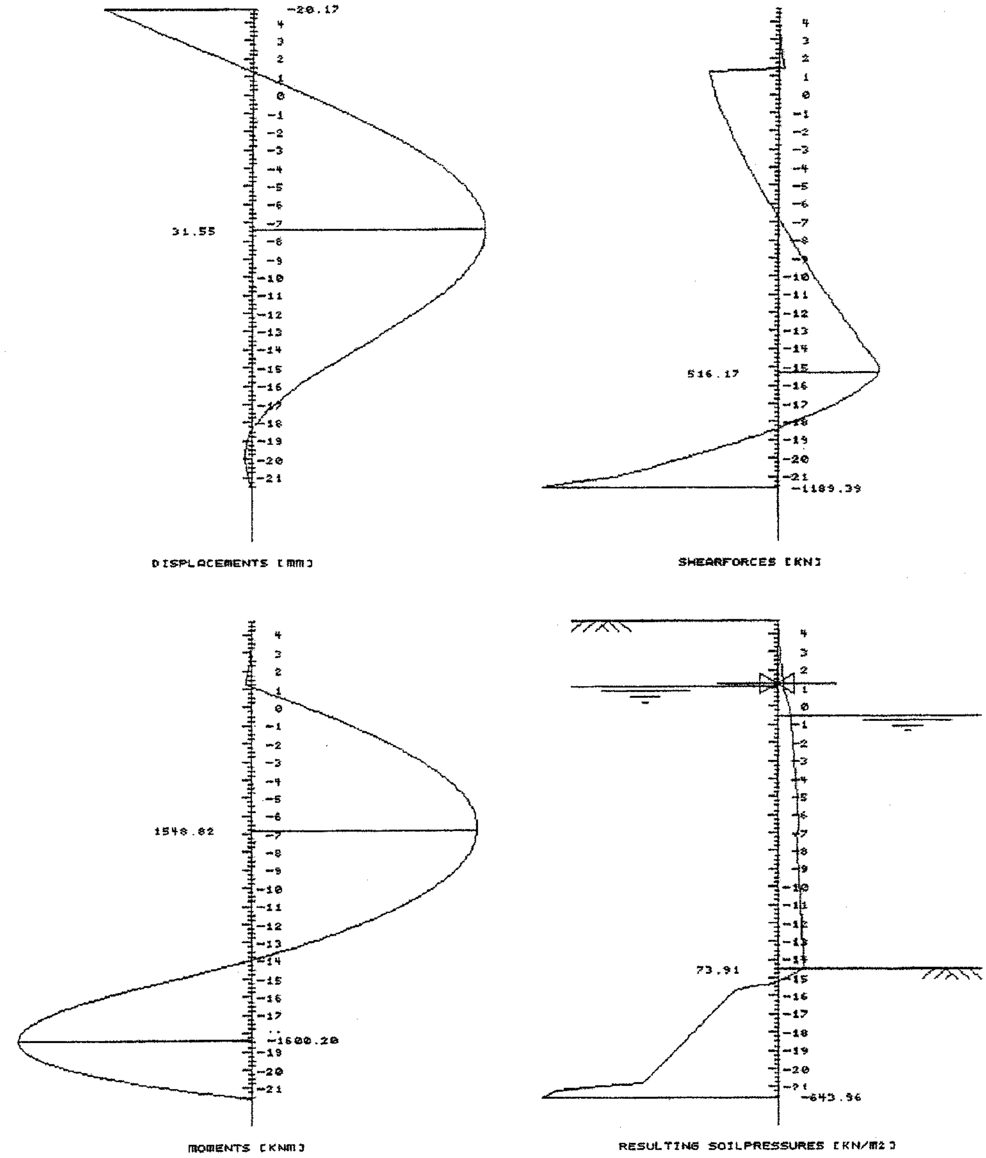
TECHNOSOFT : DAMWANDEN BLUM-BEREKENING 27 MAR100

PROJECT : BA PORT EXPANSION, QUAY WALL [REPRESENTATIVE VALUES]

SUBJECT : BOLLARD FORCE DETERMINATION

PHASENR. : 1

FIXED ENDED ANCHORE



ANCHORLEVEL: 1.250 FA= -385.49

Ballast Nedam Dredging

BALLAST NEDAM ENGINEERING B.V. RELEASE : 4.00

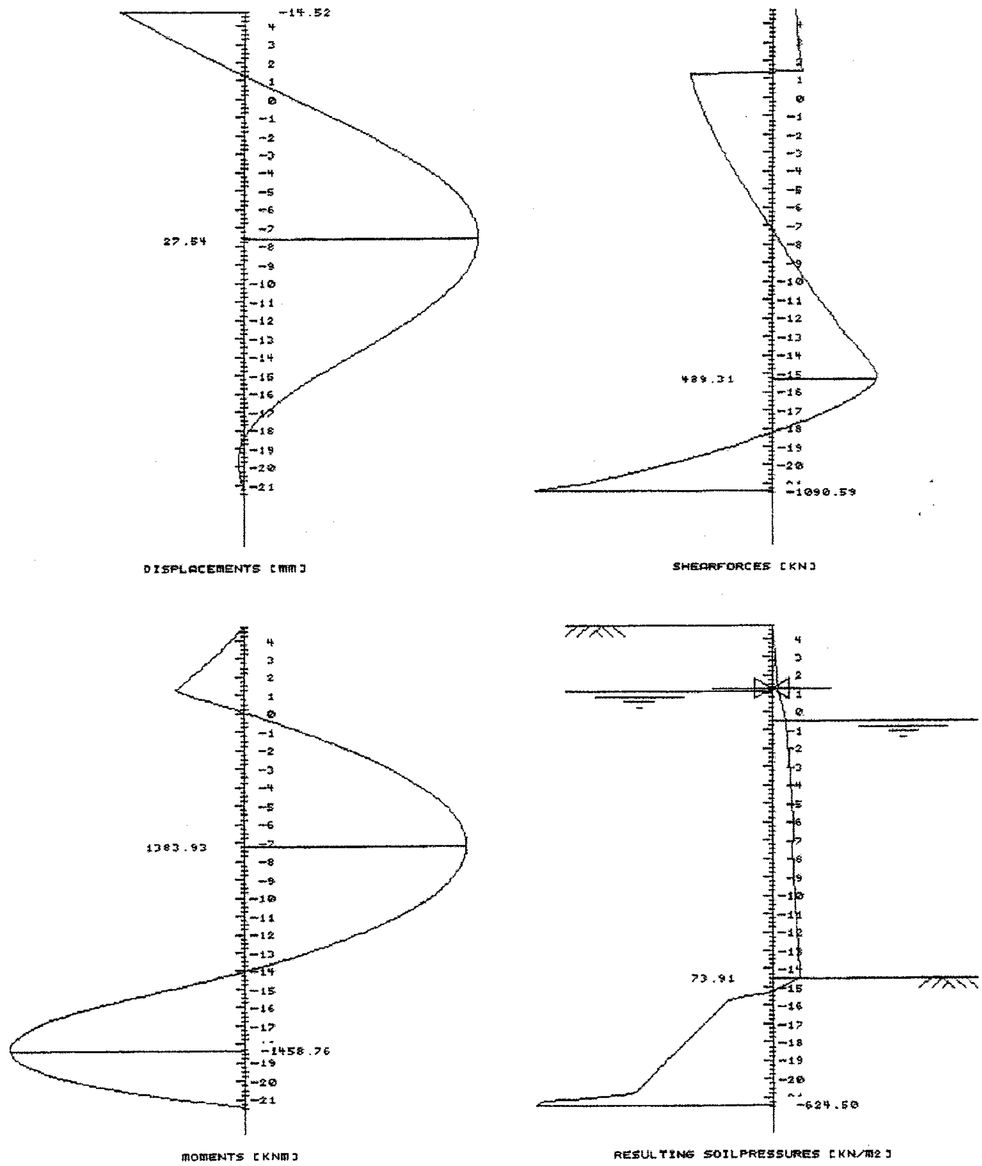
TECHNOSOFT : DAMWANDEN BLUM-BEREKENING 27 MAR100

PROJECT : BA PORT EXPANSION, QUAY WALL [REPRESENTATIVE VALUES]

SUBJECT : BOLLARD FORCE DETERMINATION

PHASENR. : 2

FIXED ENDED ANCHORE



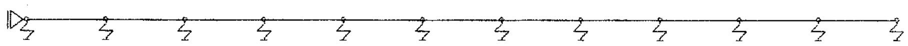
ANCHORLEVEL: 1.250 FA= -522.35

Ballast Nedam Dredging

Appendix K.3.2 Technosoft Results: Bollard Force

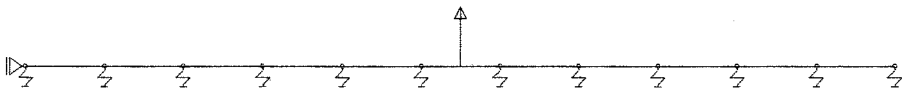
Theory for structural analysis: Geometrical linear.

GEOMETRY



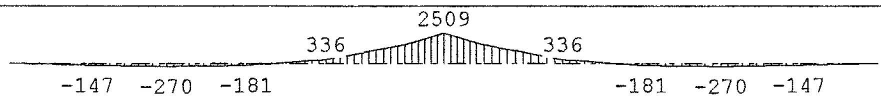
LOADINGS

LCa:1 Bollard force: 1500 kN



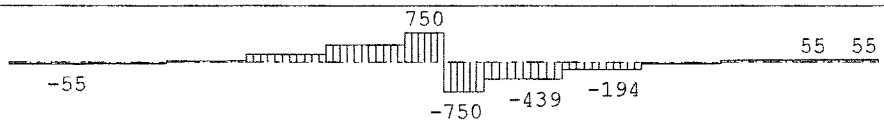
MOMENTS

LCa:1 Bollard force: 1500 kN



SHEAR FORCES

LCa:1 Bollard force: 1500 kN

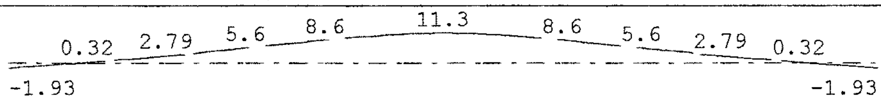


NORMAL FORCES

LCa:1 Bollard force: 1500 kN

TRANSLATIONS [mm]

LCa:1 Bollard force: 1500 kN



REACTIONS

LCa:1 Bollard force: 1500 kN

Nd.	X	Z	M
1	-0.00	55.21	
2		-9.18	
3		-79.64	
4		-160.05	
5		-245.09	
6		-311.26	
7		-311.26	
8		-245.09	
9		-160.05	
10		-79.64	
11		-9.18	
12		55.21	
	0.00	-1500.00	: Sum of the reactions
	0.00	1500.00	: Sum of the loads

Appendix K.4 ELASTICALLY SUPPORTED SHEET PILE CALCULATIONS

BALLAST NEDAM ENGINEERING B.V. RELEASE : 4.00

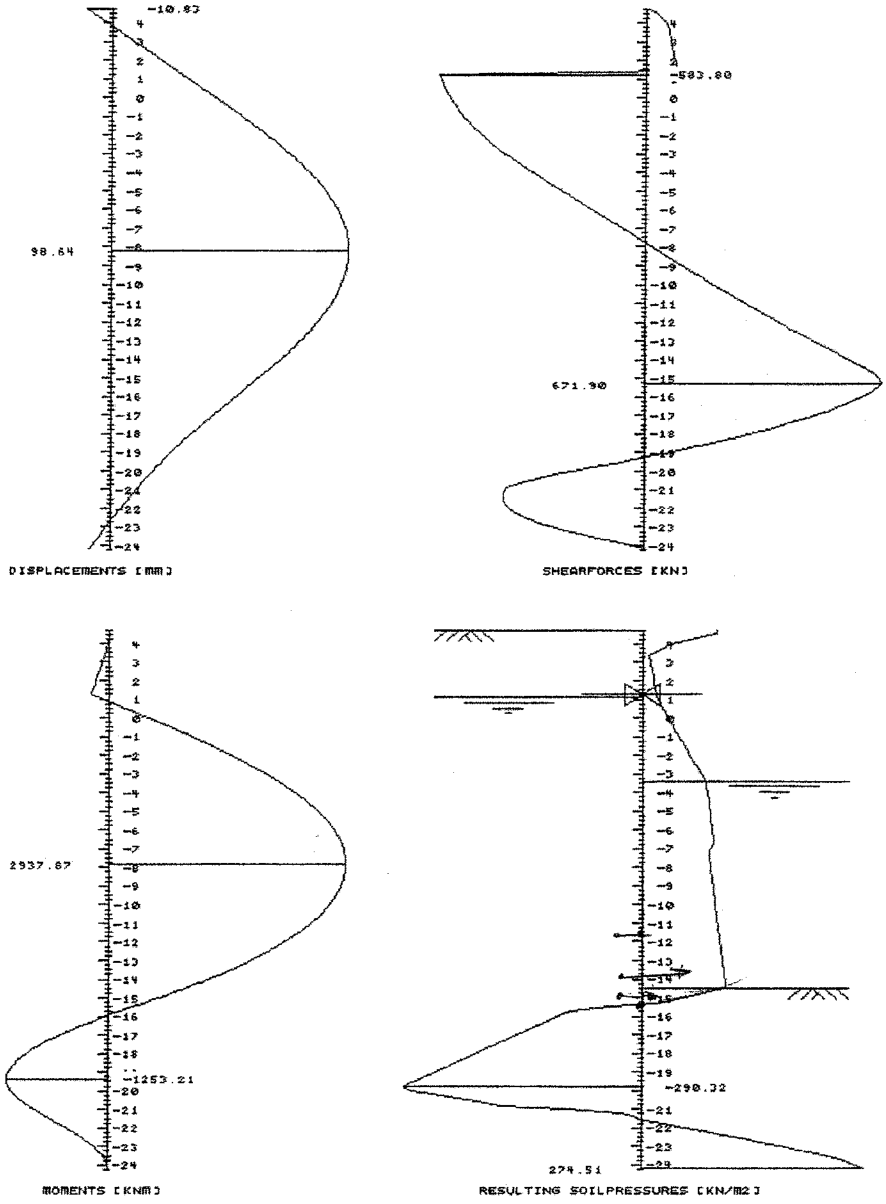
TECHNOSOFT : DAMWANDEN ELASTISCHE BEREKENING 31 MAR100

PROJECT : BA PORT EXPANSION, QUAY WALL [REPRESENTATIVE VALUES] PHASENR. : 2

SUBJECT : TUBULAR PILE 1420, T=18

ANCHOR LEV. : 1.25

FORCE : -679.24



Appendix K.5 STABILITY OF QUAY WALL

Appendix K.5.1 Upper failure plane stability anchor wall

Buenos Aires Port Expansion, with surcharge q=35 kN/m2

$kN := 1000 \cdot newton$

$D_{anchorrod} := 1.25\ m$ [m + CD]

$H_{anchorwall} := 5.5 \cdot m$

$D_{anchor} := D_{anchorrod} - (H_{anchorwall} - 2 \cdot m)$ $D_{anchor} = -2.25\ m$

$F_{anchor} := 797.6 \cdot \frac{kN}{m}$

$v := \begin{bmatrix} 3.65 \cdot m \\ 4.75 \cdot m - D_{anchor} \end{bmatrix}$ $v = \begin{bmatrix} 3.65 \\ 7 \end{bmatrix} m$

Determination of horizontal ground forces on anchorwall

$i := 0 .. 1$

$\gamma_{dr} := \begin{bmatrix} 19 \\ 11 \end{bmatrix} \cdot \frac{kN}{m^3}$ $\phi := \begin{bmatrix} 35 \\ 35 \end{bmatrix} \cdot deg$ $c' := \begin{bmatrix} 0 \\ 0 \end{bmatrix} \cdot \frac{kN}{m^2}$ $q := 35 \cdot \frac{kN}{m^2}$

$\alpha := 0$ $\beta := 0$ $\delta a := \frac{2}{3} \cdot \phi$ $\delta a = \begin{bmatrix} 23.333 \\ 23.333 \end{bmatrix} \circ deg$

$$K_{a_i} := \frac{\cos(\phi_i + \alpha)^2}{\cos(\alpha)^2 \cdot \left(1 + \sqrt{\frac{\sin(\phi_i + \delta a_i) \cdot \sin(\phi_i - \beta)}{\cos(\alpha - \delta a_i) \cdot \cos(\alpha - \beta)}}\right)^2} \qquad K_a = \begin{bmatrix} 0.224 \\ 0.224 \end{bmatrix}$$

$$K_{p_i} := \frac{\cos(\phi_i - \alpha)^2}{\cos(\alpha)^2 \cdot \left(1 - \sqrt{\frac{\sin(\phi_i) \cdot \sin(\phi_i - \beta)}{\cos(\alpha) \cdot \cos(\alpha - \beta)}}\right)^2} \qquad K_p = \begin{bmatrix} 3.69 \\ 3.69 \end{bmatrix}$$

Determination of safety factor F

η is defined as the ratio of available anchor force and actual anchorforce.

$$F_{anchor} := 679.24 \cdot \frac{kN}{m}$$

$$F_{bollard} := \frac{310 \cdot kN}{2.67 \cdot m} \qquad F_{bollard} = 116.105 \, \frac{kN}{m}$$

$$F_{amax} := F_{bollard} + F_{anchor} \qquad F_{amax} = 795.345 \, \frac{kN}{m}$$

$$\eta := \frac{\Sigma H}{F_{amax}} \qquad \eta = 2.105$$

safe := if(η > 1.5, "yes" , "no") safe = "yes"

The total force in each anchor rod will become:

$$F_{rod} := 2.67 \cdot m \cdot F_{amax} \qquad F_{rod} = 2.124 \cdot 10^3 \, kN$$

The required steel area using steel quality Fe 510 B:

$$\sigma_{steel} := \frac{360}{1.5} \cdot \frac{N}{mm^2}$$

$$A_{steel} := \frac{F_{rod}}{\sigma_{steel}} \qquad A_{steel} = 8.848 \cdot 10^3 \, mm^2$$

$$D_{req} := \sqrt{4 \cdot \frac{A_{steel}}{\pi}} \qquad D_{req} = 106.141 \, mm$$

Appendix K.5.2 Kranz stability calculation

Buenos Aires Port Expansion, with surcharge q=35 kN/m2
Determination of angle between deepest zero-force point and lowest point of anchor wall

$D_{anchor} := 2.75 \cdot m$

[m below CD]

$D_{sheetpile} := 19.25 \cdot m$

$L_{anchor} := 31.5 \cdot m$

$\tan\zeta_k := \dfrac{D_{sheetpile} - D_{anchor}}{L_{anchor}}$ $\quad \tan\zeta_k = 0.524$

$\zeta_k := \operatorname{atan}(\tan\zeta_k)$ $\quad \zeta_k = 27.646 \cdot deg$

$kN := 1000 \cdot newton$

Determination of soil weights above failure plane:

v distance to ground surface
g' weight of (submerged) soil
q surcharge
G soil weights

$v := \begin{bmatrix} 3.65 \\ 5.5 \\ 11.75 \\ 24 \end{bmatrix} \cdot m \qquad \gamma := \begin{bmatrix} 19 \\ 21-10 \\ 21-10 \\ 20-10 \end{bmatrix} \cdot \dfrac{kN}{m^3} \qquad q := 35 \cdot \dfrac{kN}{m^2}$

$G1 := \dfrac{v_2 - v_1}{\tan(\zeta_k)} \cdot \left[v_0 \cdot \gamma_0 + (v_1 - v_0) \cdot \gamma_1 + \dfrac{(v_2 - v_1) \cdot \gamma_2}{2} \right]$ $\quad G1 = 1.48 \cdot 10^3 \cdot \dfrac{kN}{m}$

$G2 := \dfrac{v_3 - v_2}{\tan(\zeta_k)} \cdot \left[v_0 \cdot \gamma_0 + (v_1 - v_0) \cdot \gamma_1 + (v_2 - v_1) \cdot \gamma_2 + \dfrac{(v_3 - v_2) \cdot \gamma_3}{2} + q \right]$ $\quad G2 = 5.957 \cdot 10^3 \cdot \dfrac{kN}{m}$

Determination of horizontal and vertical ground forces on ground body

Sheetpile: Eah & Eav

Anchorwall: E1h & E1v

γ_{dr} : voluminal mass of soil (dry)
f: angle of repose
Ka: factor for active ground pressure
γ_g: vertical ground pressure at sheetpile
$\gamma_{i := 0..3}$ ertical ground pressure at anchorwall

$$\gamma_{dr} := \begin{bmatrix} 19 \\ 11 \\ 11 \\ 10 \end{bmatrix} \cdot \frac{kN}{m^3} \qquad \phi := \begin{bmatrix} 35 \\ 35 \\ 35 \\ 29 \end{bmatrix} \cdot deg \qquad c' := \begin{bmatrix} 0 \\ 0 \\ 0 \\ 20 \end{bmatrix} \cdot \frac{kN}{m^2}$$

$\alpha := 0 \qquad \beta := 0$

$$\delta := \frac{2}{3} \cdot \phi \qquad \delta = \begin{bmatrix} 23.333 \\ 23.333 \\ 23.333 \\ 19.333 \end{bmatrix} \circ deg$$

$$K_{a_i} := \frac{\cos(\phi_i + \alpha)^2}{\cos(\alpha)^2 \cdot \left(1 + \sqrt{\frac{\sin(\phi_i + \delta_i) \cdot \sin(\phi_i - \beta)}{\cos(\alpha - \delta_i) \cdot \cos(\alpha - \beta)}}\right)^2} \qquad K_a = \begin{bmatrix} 0.224 \\ 0.224 \\ 0.224 \\ 0.292 \end{bmatrix}$$

Groundforces Sheetpile wall [Combiwall]

$$q = 35 \cdot \frac{kN}{m^2}$$

$$\gamma_g := \begin{bmatrix} q \\ q + v_0 \cdot \gamma_{dr_0} \\ q + v_0 \cdot \gamma_{dr_0} + (v_1 - v_0) \cdot \gamma_{dr_1} \\ q + v_0 \cdot \gamma_{dr_0} + (v_1 - v_0) \cdot \gamma_{dr_1} + (v_2 - v_1) \cdot \gamma_{dr_2} \\ q + v_0 \cdot \gamma_{dr_0} + (v_1 - v_0) \cdot \gamma_{dr_1} + (v_2 - v_1) \cdot \gamma_{dr_2} + (v_3 - v_2) \cdot \gamma_{dr_3} \end{bmatrix} \qquad \gamma_g = \begin{bmatrix} 35 \\ 104.35 \\ 124.7 \\ 193.45 \\ 315.95 \end{bmatrix} \frac{kN}{m^2}$$

$$E_{ah1} := (v_0 - 0 \cdot m) \cdot \left(\frac{\gamma_{g_0} \cdot K_{a_0} + \gamma_{g_1} \cdot K_{a_0} - 4 \cdot c'_0 \cdot \sqrt{K_{a_0}} \cdot \cos\left(\frac{2}{3} \cdot \phi_0\right)}{2} \right) \qquad E_{ah1} = 57.073 \frac{kN}{m}$$

$$E_{ah2} := (v_1 - v_0) \cdot \left(\frac{\gamma_{g_1} \cdot K_{a_1} + \gamma_{g_2} \cdot K_{a_1} - 4 \cdot c'_1 \cdot \sqrt{K_{a_1}} \cdot \cos\left(\frac{2}{3} \cdot \phi_1\right)}{2} \right) \qquad E_{ah2} = 47.548 \frac{kN}{m}$$

$$E_{ah3} := (v_2 - v_1) \cdot \left(\frac{\gamma_{g_2} \cdot K_{a_2} + \gamma_{g_3} \cdot K_{a_2} - 4 \cdot c'_2 \cdot \sqrt{K_{a_2}} \cdot \cos\left(\frac{2}{3} \cdot \phi_2\right)}{2} \right) \qquad E_{ah3} = 223.123 \frac{kN}{m}$$

$$E_{ah4} := (v_3 - v_2) \cdot \left(\frac{\gamma_{g_3} \cdot K_{a_3} + \gamma_{g_4} \cdot K_{a_3} - 4 \cdot c'_3 \cdot \sqrt{K_{a_3}} \cdot \cos\left(\frac{2}{3} \cdot \phi_3\right)}{2} \right) \qquad E_{ah4} = 660.274 \frac{kN}{m}$$

$$E_{av} := \begin{bmatrix} E_{ah1} \cdot \tan\left(\frac{2}{3} \cdot \phi_0\right) \\ E_{ah2} \cdot \tan\left(\frac{2}{3} \cdot \phi_1\right) \\ E_{ah3} \cdot \tan\left(\frac{2}{3} \cdot \phi_2\right) \\ E_{ah4} \cdot \tan\left(\frac{2}{3} \cdot \phi_3\right) \end{bmatrix} \qquad E_{av} = \begin{bmatrix} 24.619 \\ 20.51 \\ 96.246 \\ 231.656 \end{bmatrix} \frac{kN}{m}$$

$$\Sigma E_{ah} := E_{ah1} + E_{ah2} + E_{ah3} + E_{ah4} \qquad \Sigma E_{ah} = 988.019 \frac{kN}{m}$$

$$\Sigma E_{av} := E_{av_0} + E_{av_1} + E_{av_2} + E_{av_3} \qquad \Sigma E_{av} = 373.031 \frac{kN}{m}$$

Determination of ground forces on anchorwall

$$\gamma_{g1} := \begin{bmatrix} q \\ q + v_0 \cdot \gamma_{dr_0} \\ q + v_0 \cdot \gamma_{dr_0} + (v_1 - v_0) \cdot \gamma_{dr_1} \end{bmatrix} \qquad \gamma_{g1} = \begin{bmatrix} 35 \\ 104.35 \\ 124.7 \end{bmatrix} \frac{kN}{m^2}$$

$$E_{1h} := (v_0) \cdot \left(\frac{\gamma_{g1_0} \cdot K_{a_0} + \gamma_{g1_1} \cdot K_{a_0} - 4 \cdot c'_0 \cdot \sqrt{K_{a_0}} \cdot \cos\left(\frac{2}{3} \cdot \phi_0\right)}{2} \right) \qquad E_{1h} = 57.073 \frac{kN}{m}$$

$$E_{2h} := (v_1 - v_0) \cdot \left(\frac{\gamma_{g1_1} \cdot K_{a_1} + \gamma_{g1_2} \cdot K_{a_1} - 4 \cdot c'_1 \cdot \sqrt{K_{a_1}} \cdot \cos\left(\frac{2}{3} \cdot \phi_1\right)}{2} \right) \qquad E_{2h} = 47.548 \frac{kN}{m}$$

$$E_{1v} := \begin{bmatrix} E_{1h} \cdot \tan\left(\frac{2}{3} \cdot \phi_0\right) \\ E_{2h} \cdot \tan\left(\frac{2}{3} \cdot \phi_1\right) \end{bmatrix} \qquad E_{1v} = \begin{bmatrix} 24.619 \\ 20.51 \end{bmatrix} \frac{kN}{m}$$

$$\Sigma E_{1h} := E_{1h} + E_{2h} \qquad \Sigma E_{1h} = 104.622 \frac{kN}{m}$$

$$\Sigma E_{1v} := E_{1v_0} + E_{1v_1} \qquad \Sigma E_{1v} = 45.129 \frac{kN}{m}$$

Cohesive forces in failure plane

$$c := \begin{bmatrix} 0 \\ 20 \end{bmatrix} \cdot \frac{kN}{m^2} \qquad C_v := \begin{bmatrix} c_0 \cdot (v_2 - v_1) \\ c_1 \cdot (v_3 - v_2) \end{bmatrix} \qquad C_h := \begin{bmatrix} \dfrac{C_{v_0}}{\tan(\zeta_k)} \\ \dfrac{C_{v_1}}{\tan(\zeta_k)} \end{bmatrix}$$

$$C_v = \begin{bmatrix} 0 \\ 245 \end{bmatrix} \frac{kN}{m} \qquad C_h = \begin{bmatrix} 0 \\ 467.727 \end{bmatrix} \frac{kN}{m}$$

$$\Sigma C_v := C_{v_0} + C_{v_1} \qquad \Sigma C_v = 245 \frac{kN}{m}$$

Determination of reaction forces Q

In the lower failure plane reaction forces Q are present, which (when failure occurs) work at an angle δ perpendicular to the failure plane

- Ground near sheetpile: Eiv = + Eav & Eih = +Eah
- Ground between sheetpile and anchorwall: Eiv=Eav=0
- Ground near anchor wall: Eiv = - Eiv & Eih = -Eih

Angle of anchor: α_1 $\quad \alpha_1 := 0\cdot deg$

$$Q_{1h} := \frac{\sin(\phi_2 - \zeta_k)}{\cos(\phi_2 - \zeta_k - \alpha_1)} \cdot \left[\left(G1 - C_{v_0} - -\Sigma E_{1v}\right)\cdot\cos(\alpha_1) - \left(\frac{C_{v_0}}{\tan(\zeta_k)} + -\Sigma E_{1h}\right)\cdot\sin(\alpha_1)\right]$$

$$Q_{1h} = 196.892 \frac{kN}{m}$$

$$Q_{2h} := \frac{\sin(\phi_3 - \zeta_k)}{\cos(\phi_3 - \zeta_k - \alpha_1)} \cdot \left[\left(G2 - C_{v_1} - \Sigma E_{av}\right)\cdot\cos(\alpha_1) - \left(\frac{C_{v_1}}{\tan(\zeta_k)} + \Sigma E_{ah}\right)\cdot\sin(\alpha_1)\right]$$

$$Q_{2h} = 126.183 \frac{kN}{m}$$

$$\Sigma Q_h := Q_{1h} + Q_{2h} \qquad \Sigma Q_h = 323.076 \frac{kN}{m}$$

$$\Sigma E_{ah} = 988.019 \frac{kN}{m} \qquad \Sigma E_{1h} = 104.622 \frac{kN}{m} \qquad \frac{\Sigma C_v}{\tan(\zeta_k)} = 467.727 \frac{kN}{m}$$

$$\Sigma H := \Sigma E_{ah} - \Sigma E_{1h} + \Sigma Q_h + \frac{\Sigma C_v}{\tan(\zeta_k)} \qquad \Sigma H = 1.674\cdot 10^3 \frac{kN}{m}$$

ACTIVE EARTH FORCES + ANCHOR FORCE

$$\gamma_{ga} := \begin{bmatrix} q \\ q + v_0 \cdot \gamma_{dr_0} \\ q + v_0 \cdot \gamma_{dr_0} + (v_1 - v_0) \cdot \gamma_{dr_1} \end{bmatrix} \qquad \gamma_{ga} = \begin{bmatrix} 35 \\ 104.35 \\ 141.2 \end{bmatrix} \frac{kN}{m^2}$$

$$E_{ah1} := (v_0 - 0 \cdot m) \cdot \left(\frac{\gamma_{ga_0} \cdot K_{a_0} + \gamma_{ga_1} \cdot K_{a_0} - 4 \cdot c'_0 \cdot \sqrt{K_{a_0}} \cdot \cos\left(\frac{2}{3} \cdot \phi_0\right)}{2} \right) \qquad E_{ah1} = 57.073 \frac{kN}{m}$$

$$E_{ah2} := (v_1 - v_0) \cdot \left(\frac{\gamma_{ga_1} \cdot K_{a_1} + \gamma_{ga_2} \cdot K_{a_1} - 4 \cdot c'_1 \cdot \sqrt{K_{a_1}} \cdot \cos\left(\frac{2}{3} \cdot \phi_1\right)}{2} \right) \qquad E_{ah2} = 92.303 \frac{kN}{m}$$

$$\Sigma E_{ah} := E_{ah1} + E_{ah2} + F_{anchor} \qquad \Sigma E_{ah} = 946.977 \frac{kN}{m}$$

PASSIVE EARTH FORCES

$$\gamma_{gp} := \begin{bmatrix} 0 \\ v_0 \cdot \gamma_{dr_0} \\ v_0 \cdot \gamma_{dr_0} + (v_1 - v_0) \cdot \gamma_{dr_1} \end{bmatrix} \qquad \gamma_{gp} = \begin{bmatrix} 0 \\ 69.35 \\ 106.2 \end{bmatrix} \frac{kN}{m^2}$$

$$E_{ph1} := (v_0 - 0 \cdot m) \cdot \left(\frac{\gamma_{gp_0} \cdot K_{p_0} + \gamma_{gp_1} \cdot K_{p_0} - 4 \cdot c'_0 \cdot \sqrt{K_{p_0}} \cdot \cos\left(\frac{2}{3} \cdot \phi_0\right)}{2} \right) \qquad E_{ph1} = 467.042 \frac{kN}{m}$$

$$E_{ph2} := (v_1 - v_0) \cdot \left(\frac{\gamma_{gp_1} \cdot K_{p_1} + \gamma_{gp_2} \cdot K_{p_1} - 4 \cdot c'_1 \cdot \sqrt{K_{p_1}} \cdot \cos\left(\frac{2}{3} \cdot \phi_1\right)}{2} \right) \qquad E_{ph2} = 1.085 \cdot 10^3 \frac{kN}{m}$$

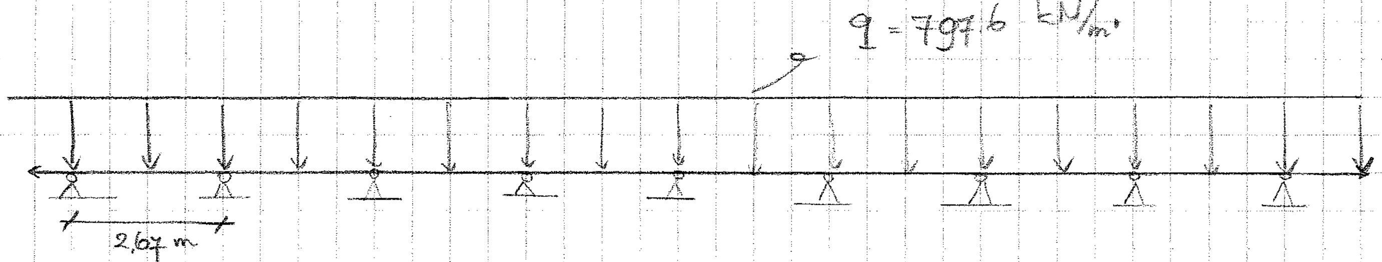
$$\Sigma E_{ph} := E_{ph1} + E_{ph2} \qquad \Sigma E_{ph} = 1.552 \cdot 10^3 \frac{kN}{m}$$

SAFETY: $\eta := \frac{\Sigma E_{ph}}{\Sigma E_{ah}}$ $\qquad \eta = 1.639$

Ballast Nedam

Buenos Aires Port Expans.
29.03.00

WALING DESIGN.



$M_{max} = \frac{1}{10} \cdot q \cdot l^2$

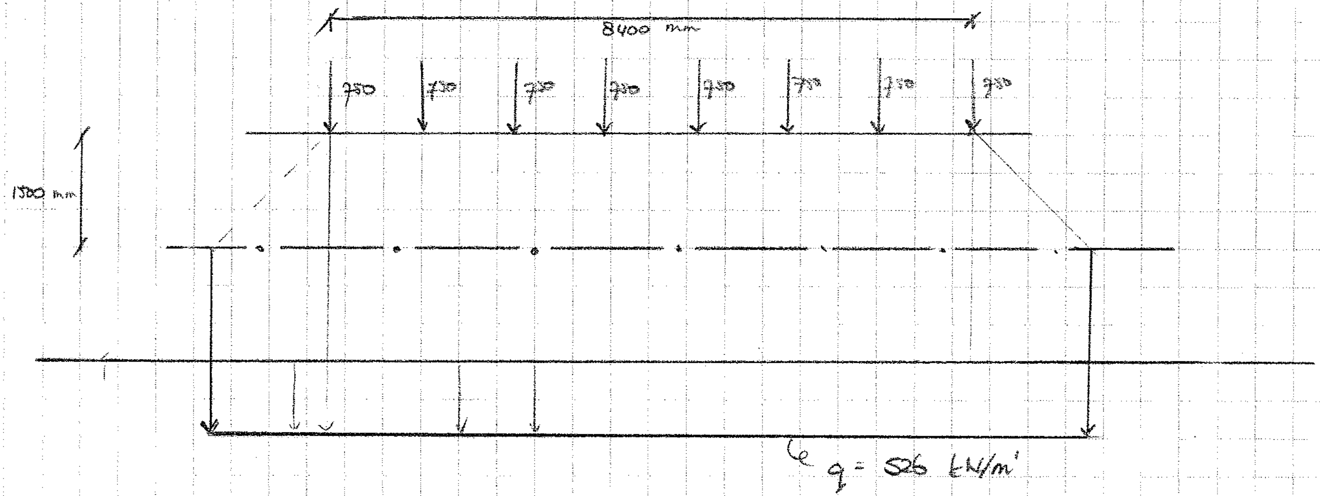
$l = 2{,}67$ m
$q = 797.6$ kN/m' } → $M_{max} = 568.6$ kNm

$W_{req} = \frac{M_{max}}{\sigma_y} = \frac{568.6 \cdot 10^6 \text{ Nmm}}{355/1.5}$ $W_{req} = 2369 \cdot 10^3 \text{ mm}^3$

Chosen profile: UNP 400, thickened $W = 2 \times 1020 + \text{thickening} \approx 2400 \cdot 10^3 \text{ mm}^3$

CRANE BEAM : PILE FOUNDATION :

The concentrated load of 6000 kN is calculated to spread over 11.4



by the concrete crane beam. This beam is founded on concrete piles c.t.c. 2.67 m.

From a calculation, where this crane beam is modelled as an elastically founded beam is found that the maximum reaction force in the piles is 1345 kN.

With safety : $F_{pile} = 1345 \times 2.5 = 3346$ kN.

The maximum pile level bearing capacity is 15 MPa.

As a first estimate the dimensions of the pile are :

$$A_{pile} = \frac{F_{pile}}{15\ Mpa} \Longrightarrow D_{pile} = 472\ mm, \text{ say } \emptyset\ 450\ mm$$

CRANE BEAM : PILE FOUNDATION

From the schematised CPT for Buenos Aires can be found that the piling level for the pile should (at least) be CD − 22 m. Because of the settlements of the tosca and sand layer, negative shaft friction must be added to the force from the crane beam

Determination of shaft friction

- The negative shaft friction has been determined with the "slip"- method:

$$F_{s,nk} = O_s \cdot h \cdot \bar{\sigma}'_v \cdot k_0 \tan\delta$$

with $k_0 \tan\delta = 0{,}25$

$O_s = 4 * 0.450 \quad 1{,}80 \text{ m}$

$\bar{\sigma}'_v = 189{,}1 = \text{kN/m}^2$

$$F_{s,nk} = 21.0 \cdot 1.80 \cdot 189.1 \cdot 0.25 = 1787 \text{ kN}$$

$$F_{tot} \downarrow = 5133 \text{ kN}$$

- The positive shaft friction is:

$$F_{s,pf} = O_s \cdot x \cdot \sigma'_v \cdot k_s \cdot \tan\delta$$

with $k_s = 2.0$

$\tan\delta = \tan(3/4 \cdot 35°) = 0{,}493$

$\sigma'_v = 333.5 + \frac{x * 11}{2}$ $\quad x =$ distance in sand layer

$$F_{s,pf} = 1{,}8 * (333.5 + 5.5\,x) \cdot x \cdot 0{,}986 = 591{,}9\,x + 9{,}76\,x^2$$

CRANE BEAM : PILE FOUNDATION

$F_{pile\,level} = 15\ MPa * 450^2 = 3038\ kN$

$F_{tot}\uparrow = 3038 + 591.9\,x + 9.76\,x^2$

Determining of pile level : CD - 21 - x ⇒

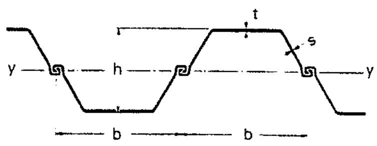
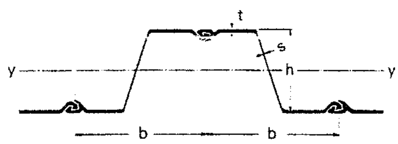
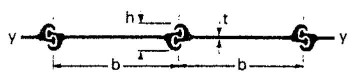
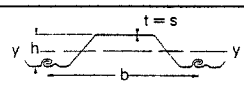
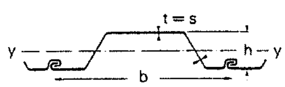
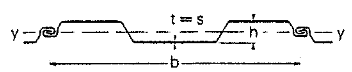
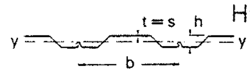
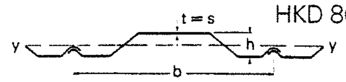
$F_{tot}\downarrow = F_{tot}\uparrow$

$5133 = 3038 + 591.9 \cdot x + 9.76 \cdot x^2 \implies x = 3{,}35\ m \implies 4\ m.$

$F_{tot}\downarrow = 5133\ kN$

$F_{tot}\uparrow = 3038 + 2524 = 5562\ kN$ ⇒ ok.

Piling level : CD - 21 - 4 m : CD - 25.0 m.

Profil	Profilbreite	Wandhöhe	Rückendicke	Stegdicke	Umfang	Eigenlast		Widerstandsmoment	Profilform
	b	h	t	s	Abwicklung	kg/m		W_y [1])	
	mm	mm	mm	mm	cm/m Wand	Einzelbohle	kg/m² Wand	cm³/m Wand	Lieferlängen der Larssen Profile, Hoesch Profile und Union Flachprofile bis 36 m.
Larssen Profile									
Larssen 601		310	7,5	6,4	258	46,3	77	745	
Larssen 602		310	8,2	8	258	53,4	89	830	
Larssen 603		310	9,7	8,2	260	64,8	108	1200	
Larssen 603 K		310	10	9	260	68,1	113	1240	
Larssen 604 [2])		380	10,5	9	282	74,5	124	1620	
Larssen 605	600	420	12,5	9	290	83,5	139	2020	
Larssen 605 K		420	12,2	10	290	86,7	144	2030	
Larssen 606		435	15,6	9,2	293	94,4	157	2500	
Larssen 606 K		435	15,6	10	293	97,5	162	2540	
Larssen 607		435	21,5	9,8	293	114,4	191	3200	
Larssen 607 K		435	21,5	10	293	115,2	192	3220	
Larssen 20		220	7	6	250	39,5	79	600	
Larssen 21		220	8,2	8	250	47,5	95	700	
Larssen 22		340	10	9	280	61	122	1250	
Larssen 23	500	420	11,5	10	315	77,5	155	2000	
Larssen 24		420	15,6	10	315	87,5	175	2500	
Larssen 24/12		420	15,6	12	315	92,7	185	2550	
Larssen 25		420	20	11,5	311	103	206	3040	
Larssen 31 [3])	450	150	9,5	9,5	230	45	100	460	
Larssen 32 [3])	450	250	10,5	10,5	250	54,9	122	850	
Larssen III	400	247	14,2	9,2	285	62	155	1350	
Larssen 43	500	420	12	12	280	83	166	1660	
Larssen 430 [4])	708	750			396		235	6450	
Hoesch Profile									
Hoesch 12	575	260	9,5	9,5	259	61,5	107	1140	
Hoesch 95 [3])		190	8	8	240	49,9	95	750	
Hoesch 116 [3])		250	9,3	9	253	60,9	116	1200	
Hoesch 122 [3])		190	11	10,7	240	64,1	122	940	
Hoesch 134	525	300	10	9,5	274	70,4	134	1700	
Hoesch 155		300	12,8	9,8	267	81,4	155	2000	
Hoesch 175		340	14	10	299	91,9	175	2600	
Hoesch 215		340	18,8	12	291	113	215	3150	
Union Flachprofile									
FL 409	400	86	9	–	215	54	135	120	
FL 412			12	–		61	152		
FL 512	500	88	12	–	255	70,5	141	90	
Hoesch Leichtprofile									
HL 1	450	80	4,5	4,5	230	20,2	45	140	 HL 1, HL 2, HL 2/7
HL 2	600	130	6	6	238	37,8	63	338	
HL 2/7	600	131	7	7	239	45	75	388	
HL 3	700	150	8	8	243	61,5	88	540	 HL 3
Hoesch Tafelprofile									
HT 45 [3])			4,5	4,5		45	45	159	
HT 50 [3])	1000	90	5	5	227	50	50	175	
HT 60 [3])			6	6		60	60	208	
HT 70 [3])			7	7		70	70	240	
Hoesch Kanaldielen									
HKD 400/6	400	50	6	6	240	22,1	55	102	 HKD 400/6
HKD 800	800	100	8	8	237	59	73	273	 HKD 800

[1]) Die Widerstandsmomente der Larssen Profile bedingen eine Verriegelung der im Werk zusammengezogenen Schlösser zur Aufnahme der Schubkräfte.

[2]) Liefertermin noch offen.

[3]) Walzung/Lieferung nur auf Anfrage.

[4]) Wandform aus Larssen 43.

APPENDIX L. QUAY WALL DRAWINGS (AUTOCAD)

In the following pages are shown:

Appendix L.1 QUAY WALL: GENERAL DESIGN AND LEVELS

Appendix L.2 QUAY WALL: CONSTRUCTION PHASING

APPENDIX M:

CONSTRUCTION SEQUENCE ALTERNATIVE 3; PHASE 1

Activity / Work Area	Main equipment	Dump Area	Dredge/ borrow area	Production rate	Quantity	Unit price	Req Time	Costs
Phase Ia:								
1. Construction of depot dike				1000 m / month				5.000.000
2. Dredge all. deposits pit 1	CSD 1 & 2x TSHD 1	Offshore	P1	200,000 m3 / wk	1.480.000	4,5	7,4	6.660.000
3. Dredge tosca pit 1	CSD 1 & CSD 2	Offshore	P1	230,000 - 360,000 m3/wk	4.400.000	3	1,2	13.200.000
4. Dredge all deposits quay 1-4	2x TSHD 1	Offshore		200,000 m3/wk	300.000	2,5	1,5	750.000
5. Reclam. basin E to CD	DSD 1	Basin E	P1	330,000 m3/wk	700.000	2,25	2,1	1.575.000
6. Reclam. Z1 (triangle) to CD	DSD 1	Z1	P1	330,000 m3/wk	500.000	2,25	1,5	1.125.000
7. Reclam. T1 & T2 to CD	DSD 1	T1 / T2	P1	330,000 m3/wk	900.000	2,25	2,7	2.025.000
8. Reclam. T1 to +6.25	DSD 1	T1	P1	330,000 m3/wk	950.000	2,25	2,9	2.137.500
9. Surcharge on T1 (to +9.00)	DSD 1	T1	P1	330,000 m3/wk	300.000	2,25	0,9	675.000
10. Reclam. Z1 (triangle) to +6.25	DSD 1	Z1	P1	330,000 m3/wk	1.000.000	2,25	3,0	2.250.000
11. Reclam. basin E to +4.00	DSD 1	Basin E	P1	330,000 m3/wk	350.000	2,25	1,1	787.500
12. Start shore protection			Z1	2500 t / day = 150 m / month	2.500	3.000	72 wk	7.500.000
13. Construction of work road					3.300	3000		9.900.000
Phase Ib:								
1. Dredge all. deposits pit 2	CSD 1	P1	P2	270,000 m3/wk	1.480.000	2	5,5	2.960.000
2. Dredge tosca pit 2	CSD 1 & 2	P1	P2	230,000 - 360,000 m3/wk	4.400.000	3	12,2	13.200.000
3. Reclam. T2 to +6.25	DSD 1	T2	P2	330,000 m3/wk	950.000	2,25	2,9	2.137.500
4. Reclam. T3 + T4 to CD	DSD 1	T3 / T4	P2	330,000 m3/wk	900.000	2,25	2,7	2.025.000
5. Reclam. T3 +T4 to +6,25	DSD 1	T3 / T4	P2	330,000 m3/wk	1.900.000	2,25	5,8	4.275.000
6. Surcharge T1 to T2	Trucks	T2	T1					1.000.000
7. Reclam. part of rectangle to +6.25	DSD 1	Z2	P2	330,000 m3/wk	650.000	2,25	2,0	1.462.500
8. Surcharge on T 3 & 4 (to +9.00)	DSD 1	T3 / T4	P2	330,000 m3/wk	600.000	2,25	1,8	1.350.000
Phase Ic:								
1. Dredge all. deposits pit 3	CSD 1	P2	P3	270,000 m3/wk	1.260.000	2	4,7	2.520.000
2. Dredge tosca pit 3	CSD 1 & 2	P2	P3	230,000 - 360,000 m3/wk	3.700.000	3	10,3	11.100.000
3. Reclam. Z2 to +6.25	DSD 1	Z2	P3	330,000 m3/wk	1.250.000	2,25	3,8	2.812.500
4. Reclam. T5 to +6.25	DSD 1	T5	P3	330,000 m3/wk	500.000	2,25	1,5	1.125.000
5. Sand layer on depot and basin F	DSD 1	Various	P3	330,000 m3/wk	1.000.000	2,25	3,0	2.250.000
6. Surcharge T2 to Darsena E	Trucks / Bulldozers							1.000.000
8. Surcharge T3&4 to rectangle & T5	Trucks / Bulldozers		P3	330,000 m3/wk				1.000.000
9. Start eastern (temporary) shore protection								
Quay wall:								
Construction quay wall T1				100 m / month	275	32500	12,0	8.937.500
Construction quay wall T2				100 m / month	275	32500	12,0	8.937.500
Construction quay wall T3				100 m / month	275	32500	12,0	8.937.500
Construction quay wall T4				100 m / month	275	32500	12,0	8.937.500
Dredging of Basins & Access channel								
Dredge of access channel (hoppers)	2x TSHD 1	Offshore	AC	150,000 - 200,000 m3/wk	10.600.000	2,5	60,6	26.500.000
Dredge of access channel (cutters)	CSD 3 + 2x TSHD 2	Offshore	AC	200,000 m3/wk	10.400.000	4,5	52,0	46.800.000
Dredge of basins (hopper)	2x TSHD 1	Offshore	PN	150,000 - 200,000 m3/wk	2.320.000	2,5	13,3	5.800.000
Dredge of basins (cutter)	CSD 3 + 2x TSHD 1	Offshore	PN	200,000 m3/wk	3.480.000	4,5	17,4	15.660.000

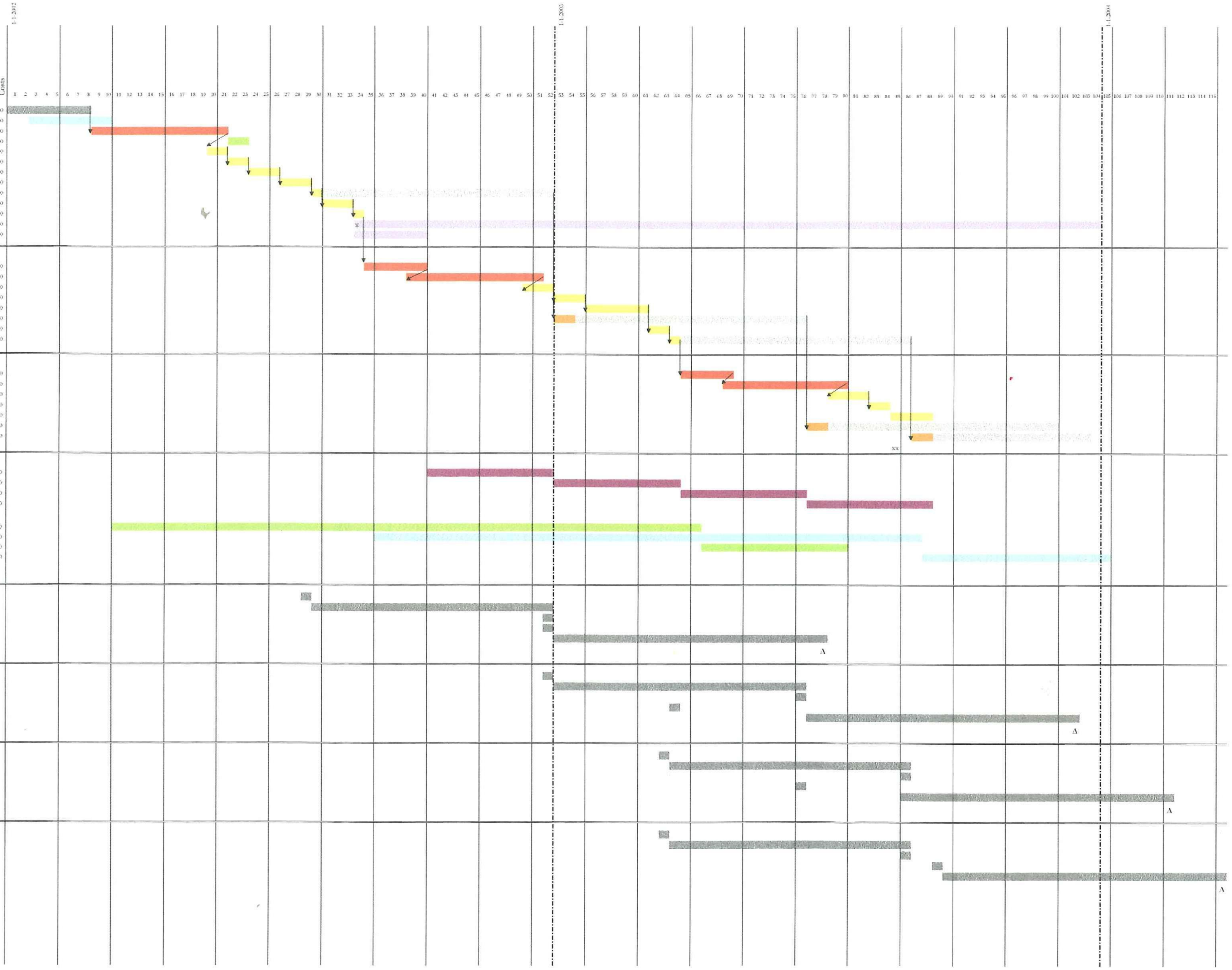
PLANNING GROUPED BY TERMINAL

Terminal 1:
Terrain at CD +6.250
Surcharge on terrain
Settlements done
Quay wall ready
Paving , cranes etc
Terminal area ready

Terminal 2:
Terrain at CD +6.250
Surcharge on terrain
Settlements done
Quay wall ready
Paving , cranes etc
Terminal area ready

Terminal 3:
Terrain at CD +6.250
Surcharge on terrain
Settlements done
Quay wall ready
Paving, cranes, etc.
Terminal area ready

Terminal 4:
Terrain at CD +6.250
Surcharge on terrain
Settlements done
Quay wall ready
Paving, cranes, etc.
Terminal area ready



AC = Access channel
CSD 1 = Cutter Suction Dredger type Haarlem
CSD 2 = Cutter Suction Dredger type Hector
CSD 3 = Cutter Suction Dredger type Hector
DSD 1 = Deep Suction Dredger type Hector
PN = Puerto Nuevo
TSHD 1= Trailing Suction Hopper Dredger type Apollo
TSHD 2= Trailing Suction Hopper Dredger type Apollo

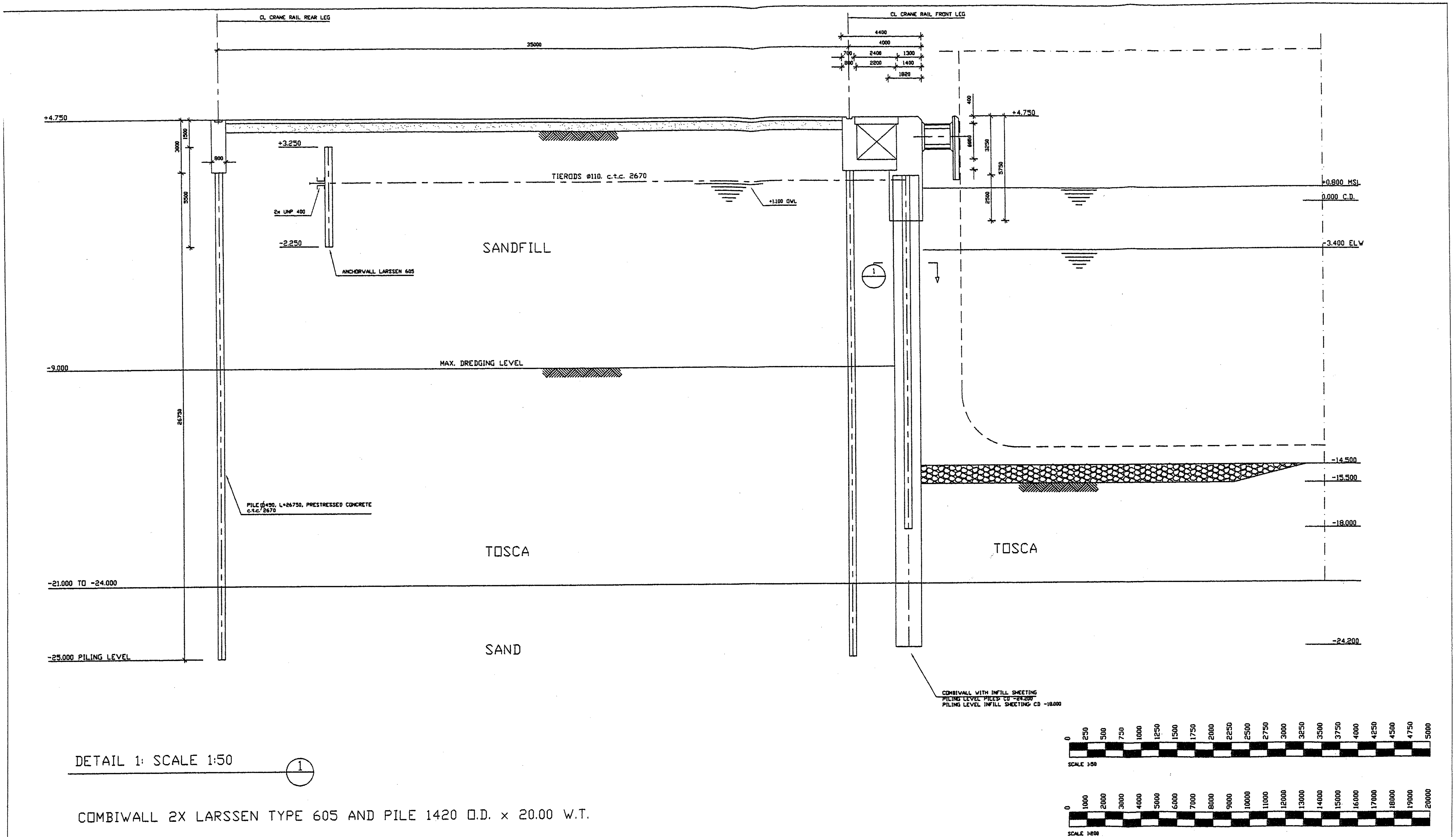
APPENDIX N. CASH FLOW CALCULATION

Discounting percentage: 15 %

	INVESTMENTS						REVENUES						CASH FLOW		
YEAR	Reclamation	Capital dredging	Quay wall	Maint.	Total costs / year	Cum. Investments	Container charges	Port charges	Terminal lease	Gov. contr.	Total	Cum. Revenues	Cash Flow	Disc. Cash Flow	Cum. Disc Cash Flow
2001	0	0	0	0	0	0	3.3	4.9	7.92	15.0	31.2	31	31.2	31.2	31.2
2002	63.9	33.7	9	0	106.6	107	3.9	5.8	7.92	15.0	32.6	64	-74.0	-64.4	-33.2
2003	39.4	59.2	27	0	125.6	232	4.5	6.8	12.1	15.0	38.4	102	-87.2	-66.0	-99.1
2004	0	5	0	15	22	252	5.1	7.7	15.1	15.0	42.9	145	22.9	15.0	-84.1
2005	0	0	0	15	17	267	5.6	8.5	15.1	15.0	44.2	189	29.2	16.7	-67.4
2006	0	0	0	15	17	282	6.1	9.1	16.3	15.0	46.4	236	31.4	15.6	-51.8
2007	0	0	0	15	17	297	6.5	9.8	16.3	15.0	47.6	283	32.6	14.1	-37.7
2008	0	0	0	15	17	312	7.0	10.5	16.3	15.0	48.8	332	33.8	12.7	-25.0
2009	7	0	18	17	42	354	7.5	11.3	17.7	15.0	51.5	384	9.5	3.1	-21.9
2010	7	0	18	17	42	396	8.1	12.1	19.2	15.0	54.4	438	12.4	3.5	-18.4
2011	0	0	0	17	17	413	8.6	13.0	22.2	15.0	58.8	497	41.8	10.3	-8.1
2012	0	0	0	17	17	420	9.2	13.9	22.2	15.0	60.3	557	43.3	9.3	1.3
2013	0	0	0	17	17	447	9.9	14.8	22.2	15.0	61.9	619	44.9	8.4	9.7
2014	15	7	18	19	57	506	10.6	15.9	22.2	15.0	63.6	683	4.6	0.8	10.4
2015	15	7	18	19	57	565	11.3	16.9	26.7	15.0	69.9	752	10.9	1.5	11.9
2016	6	0	9	19	32	599	12.1	18.1	26.7	15.0	71.8	824	37.8	4.6	16.6
2017	0	0	0	19	17	618	12.9	19.3	26.7	15.0	73.8	898	54.8	5.9	22.5
2018	0	0	0	19	17	637	13.7	20.6	29.6	15.0	79.0	977	60.0	5.6	28.0
2019	0	0	0	19	17	656	14.7	22.0	29.6	15.0	81.3	1058	62.3	5.0	33.1
2020	0	0	0	19	17	675	15.6	23.4	29.6	15.0	83.7	1142	64.7	4.5	37.6
Total	153.3	111.9	117	289	675.2		176.2	264.4	401.4	300	1142		470.8	37.6	
IRR															***21%***
NPV															***37.6***

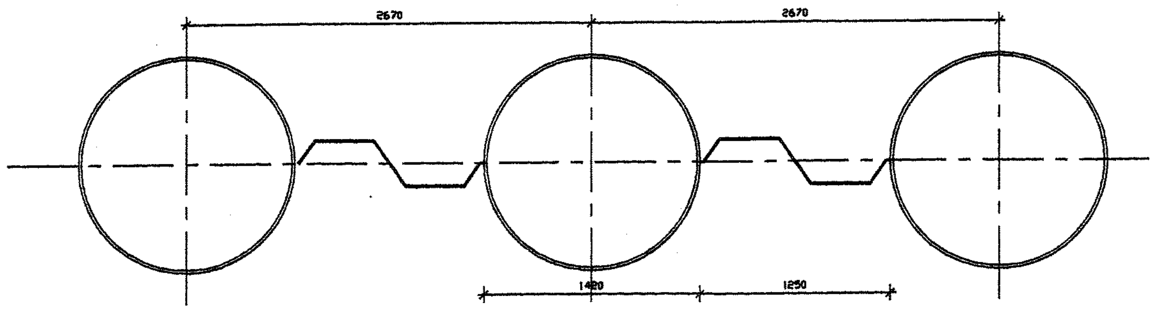
Table N-1 Cash flow calculation for disounting percentage of 15%

Ballast Nedam Dredging



DETAIL 1: SCALE 1:50

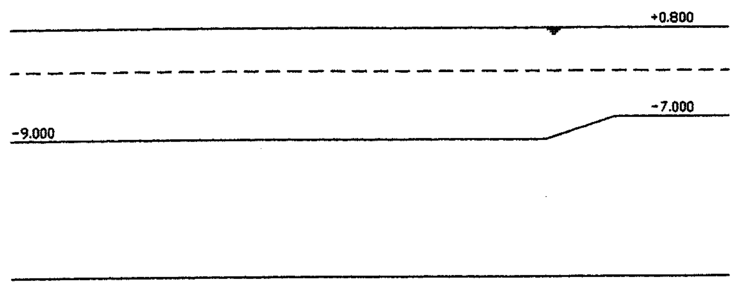
COMBIWALL 2X LARSSEN TYPE 605 AND PILE 1420 O.D. x 20.00 W.T.



NOTE: DIMENSIONS ARE IN MM, LEVELS IN M REL. TO CD

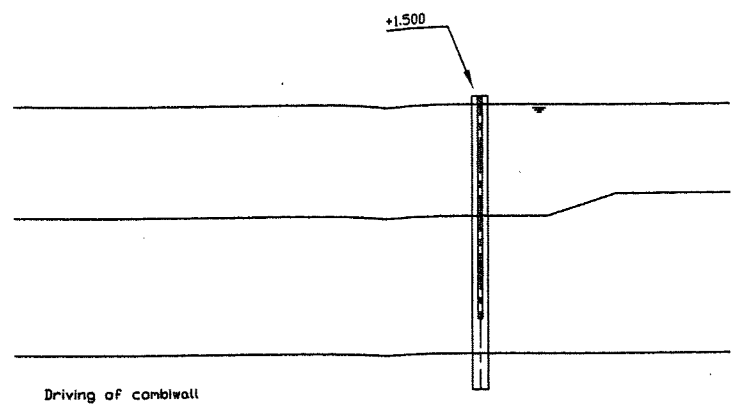
Project	BUENOS AIRES PORT EXPANSION 2020 CONTAINER TERMINALS
Section	QUAY WALL GENERAL DESIGN AND LEVELS
Date: 20-4-00	By: RBO
Paper size: A3	Plot scale: 1:200 / 1:50

PHASE 1



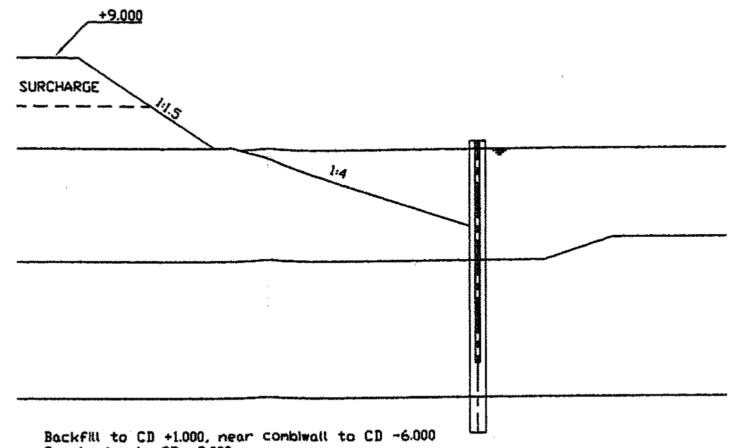
Dredging seabed to CD -9.0 (tosca)

PHASE 2



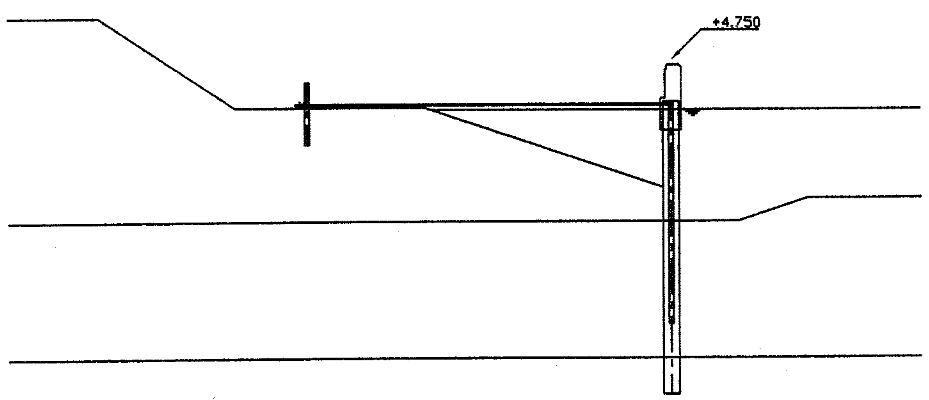
Driving of combiwall

PHASE 3



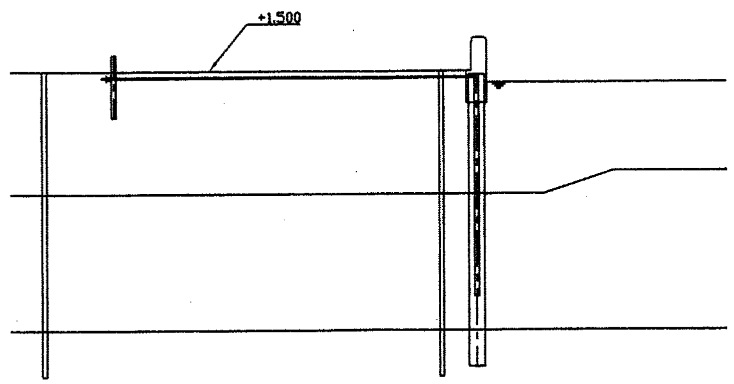
Backfill to CD +1.000, near combiwall to CD -6.000
Surcharhe to CD +9.000

PHASE 4



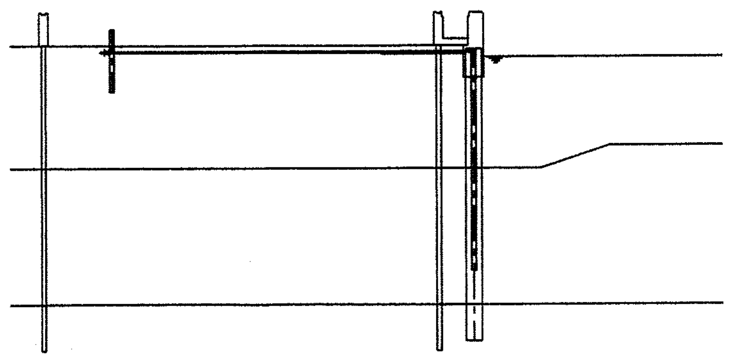
Removal of surcharge near quay wall
Driving anchorwall
Anchoring combiwall
Construction of coping beam

PHASE 5



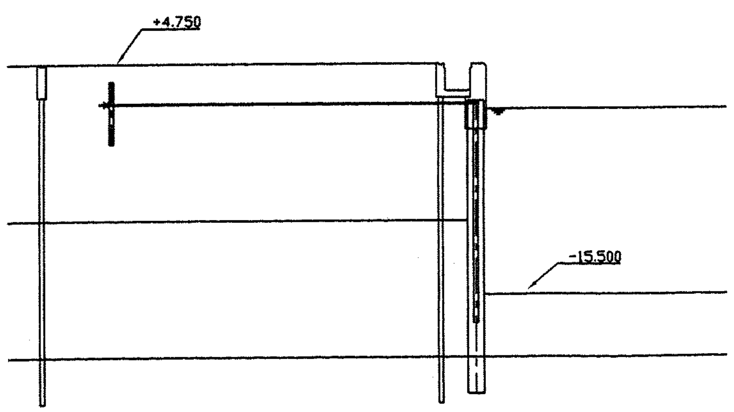
Backfill to CD +1.5
Driving of crane foundation piles

PHASE 6



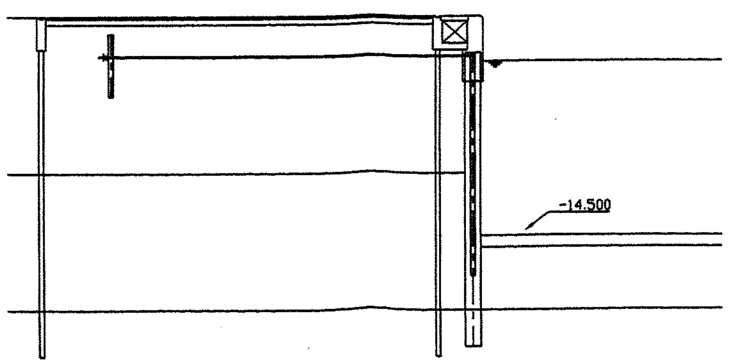
Construction of rear crane beam
Construction of cable gutter, connection with coping beam
Construction of front cranebeam

PHASE 7



Backfill to CD +4.75
Dredging to CD -15.50 at waterfront

PHASE 8



Finishing backfill with stabilized sand
Placement of paving blocks on stabilised sand
Placement of concrete tiles on cable gutter
Placement of scourprotection

Project	BUENOS AIRES PORT EXPANSION 2020 CONTAINER TERMINALS		
Section	QUAY WALL CONSTRUCTION PHASING		
Date	20-4-00	By	RBO
Paper size	A3	Plot scale	