IMPACT PROOF CHILE

MULTIDISCIPLINARY PROJECT - 2016/2017

IMPACT PROOF PRELIMINARY DESIGN FOR THE MARINE BIOLOGY STATION IN DICHATO

Universidad de Concepción

TU Delft

Impactproof Chile
Joining forces to protect Dichato from future impacts

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Preface

This report gives an integrated design proposal for the upgrade of the Marine Biology Station of the Universidad de Concepción (UdeC) in Dichato, Chile, hereafter referred to as EBMD. It is the result of a Multidisciplinary Project carried out by five MSc Civil Engineering students of Delft University of Technology, with specialisations Structural, Geo- and Hydraulic Engineering. The aim of such a Multidisciplinary Project, which is by no means similar to a MSc Thesis, is to integrate acquired knowledge from different master programmes and deliver a thorough analysis of the problem or situation and a resulting design. The work for this Multidisciplinary Project, including site visits and preliminary investigations, design work and reporting was carried out from November 2016 to January 2017 in Concepción, Chile, with as primary host and client UdeC.

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Finally, we would like to thank our sponsors for making this project possible and motivating us to tackle the project head-on: BouAd Adviesgroep B.V., Concreet Projectmanagment, Geobest B.V., DIMI and BMT ARGOSS.

SPONSORS

APPROACH

The following report is divided into three parts for reasons of clarity and legibility. Overall, the structure follows that of a typical design process.

Part I: Analysis outlines the problem and scope of the project, accompanied by an overview of the current in-situ hydraulic, geotechnical and structural conditions at the Estación de Biología Marina de Dichato[1] (EBMD) harbour complex. Based on required functionalities, boundary conditions, environmental and cost considerations, several alternative conceptual solutions are presented which all adhere to the Program of Requirements. Using a Multi-Criteria Analysis in combination with a cost estimate, the alternatives are evaluated with respect to each other, allowing for an optimum alternative to be carried forward in the design phase.

Part II: Design encompasses the various stages of a preliminary design. First, dimensioning is carried out based on rules of thumb, in order to facilitate design progress between disciplines. The dimensioning is given in an Appendix, The more detailed design is presented in the report and includes the design of a jetty and a breakwater, as well as a rough design for a reconfigured EBMD building and the on-site pavement. In the design of the jetty and breakwater, various programs are used to model each element, including ETABS, SWANOne, DELFT3D, BREAKWAT 3.0 and PLAXIS2D. These programs are also used in the next phase: An Extreme Impact Evaluation is conducted, in which the effects of a seismic and tsunami event as that of 2010 are simulated, and an assessment is made on the induced damage on the designed facilities. Finally, a construction timeline is suggested and a more detailed cost-breakdown is provided.

Part III: Appendices contains all relevant background information and calculations which are referenced throughout the report.

[1] Marine Biology Station of Dichato

SUMMARY

To the North of the city of Concepción in the South-West of Chile, lies the town of Dichato. It is situated on the south-eastern side of Coliumo Bay. The Universidad de Concepción (UdeC) owns a marine concession in this bay, which includes a section of beach and sea for a Marine Biology Station (EBMD), belonging to the Faculty of Natural and Oceanographic Sciences. The objective of the EBMD is to provide research and educational support in the field of marine sciences.

The EBMD concession consists of several onshore buildings with research and educational facilities, some of which are unfinished or damaged due to the Maule 2010 earthquake of magnitude M_w 8.8. Furthermore, there are remains of an old jetty for the docking of a Marine Biology vessel, which was also destroyed in 2010. Thus, UdeC is interested in redeveloping the EMBD, as there is no location for mooring of the vessel and transhipment of goods, as well as incomplete construction or use of several onshore facilities.

To solve this problem, as well as to stimulate local authority interest in funding of the redevelopment of the EMBD, a design proposal is made. First the required functions, with as primary function *mooring*, and the in-situ conditions are investigated, leading to a Program of Requirements. Five design alternatives are established and weighed in a Multi-Criteria and Cost-Benefit Analysis, which leads to the conclusion that the Traditional option is most suitable, due to vast Chilean experience with the type of design and limited costs. From the Program of Requirements it is decided to focus on the offshore aspects of the design solution, in this case the jetty and the breakwater.

The design of the jetty is carried out according to the Chilean design codes, and using the structural analysis and design program ETABS, which can incorporate seismic loading. The final design of the jetty includes a concrete deck on a steel frame, with steel piles embedded in the rock in a Marco Duplas (inclined) configuration to resist lateral loading. All elements are tested for structural soundness. The breakwater, on the other hand, is designed through a combination of wave modelling using statistical methods and DELFT3D; and a crest height and stone dimension analysis using BREAKWAT3.0. The upgrade of an existing unfinished building and the pavement are treated in lesser detail.

For all elements of the EBMD upgrade, resilience is taken into account as a primary factor in extreme impact[2] design, focussing on allowing structures to have a quick recover capacity, since it is not feasible to design coastal structures to resist impacts like large-scale earthquakes and tsunamis. The damage to the designed elements in the case of a repeat of the Maule 2010 earthquake and tsunami is analysed in an Extreme Impact Evaluation. A range of hazards, including several modes of structural failure of the jetty and breakwater, as well as relevant geohazards for the site, are classified according to level of risk. Mitigation measures are suggested as well.

[2] Earthquake and tsunami events, in the context of this project.

Finally, following a more detailed cost breakdown and a construction timeline, it is concluded that the proposed design solution is feasible within a construction time of 35 weeks and estimated costs of 450 mil CLP. The construction of the jetty and breakwater allows the EBMD to carry out its scientific and academic research safely and more efficiently, whilst also limiting damage and incorporating resilience in the event of large-scale earthquakes and tsunamis.

Impact proof preliminary design for the

Marine Biology Station in Dichato

2016/2017

PART I

ANALYSIS

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1 Introduction

1.1 Site description

1.1.1 Physiography

To the North of the city of Concepción in the South-West of Chile, lies the town of Dichato. This coastal town is part of the municipality of Tomé which in turn belongs to the Biobío Region. Dichato had around 3500 residents in the census of the year 2002 (Instituto Nacional de Estadisticas, 2005). The bay of Dichato, or Coliumo Bay as it is often called, is closed and its calm waters attract many recreational visitors in the summer season. The other main economic activity, which happens all year round, is artisanal fishing. Locals also own marine concessions or 'management areas' to carry out aquafarming of clams and seaweed. See Figure 1-1 for the location of Caleta Villarrica, the section of coast where the University of Concepción (UdeC) has a marine concession, and where fishing activity is carried out by residents of Dichato.

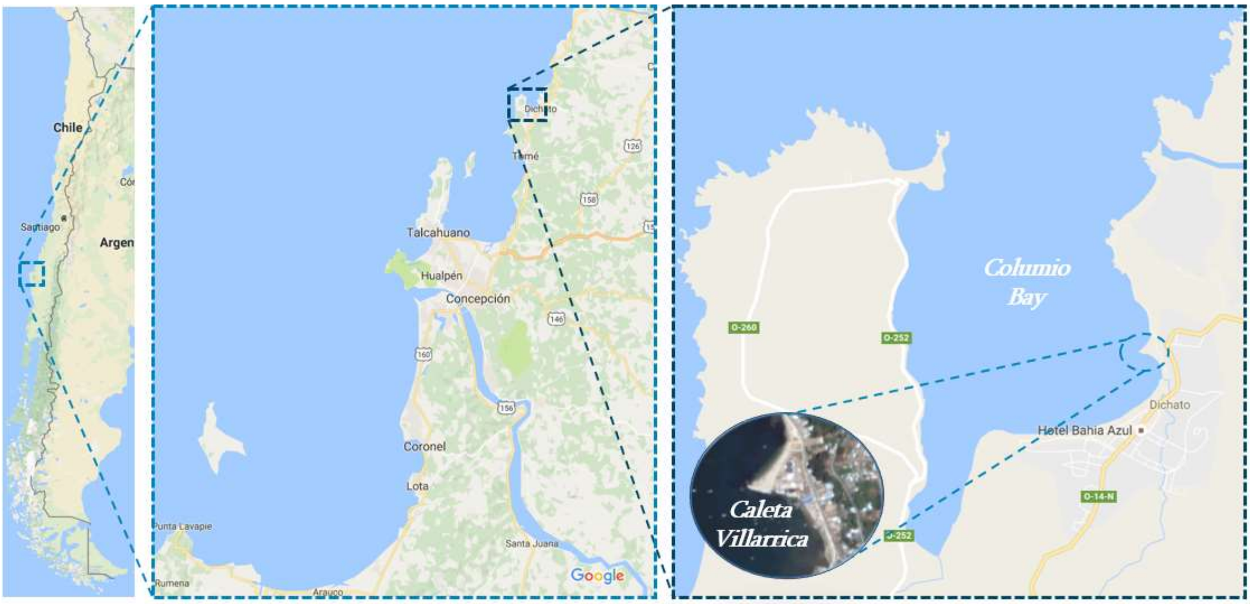


Figure 1-1: Location of Concepción, Dichato and Caleta Villarrica in Chile

1.1.1.1 Topography and infrastructure

Figure 1-2 maps the urban areas, water bodies and infrastructure in and around Coliumo Bay, with height contours to indicate the elevation in the area. The bay has width of approximately 2.7 km and a maximum depth 20 m with respect to mean sea level.

Figure 1-2: Topography, land use and infrastructure around Coliumo Bay

1.1.1.2 Climate

A cold sea current in the Pacific Ocean makes the regional climate relatively dry and helps to maintain a mild, Mediterranean climate throughout the year. Temperatures rarely exceed 30 °C or fall below 0 °C. The great majority of precipitation falls between May and October.

1.1.2 CONCESSION SITE

The University of Concepción (UdeC) owns a marine concession in Coliumo Bay, which includes a section of the beach, the shore and the sea. The marine concession and its location within the bay is shown is in Figure 1-3. Within the Lot II of the concession lies the Marine Biology Station (EBMD), which belongs to the Faculty of Natural and Oceanographic Sciences of UdeC. The objective of the EBMD is to provide research and educational support in the field of marine sciences (UdeC, 2003).

Caleta Villarrica is also marked in Figure 1-3. At this coastal protrusion, the EBMD concession included a jetty for the docking of a Marine Biology vessel called Kay Kay II. This vessel served as a floating laboratory and as storage space for breeding programs of marine species. However, in the 2010 Maule earthquake this jetty was destroyed.

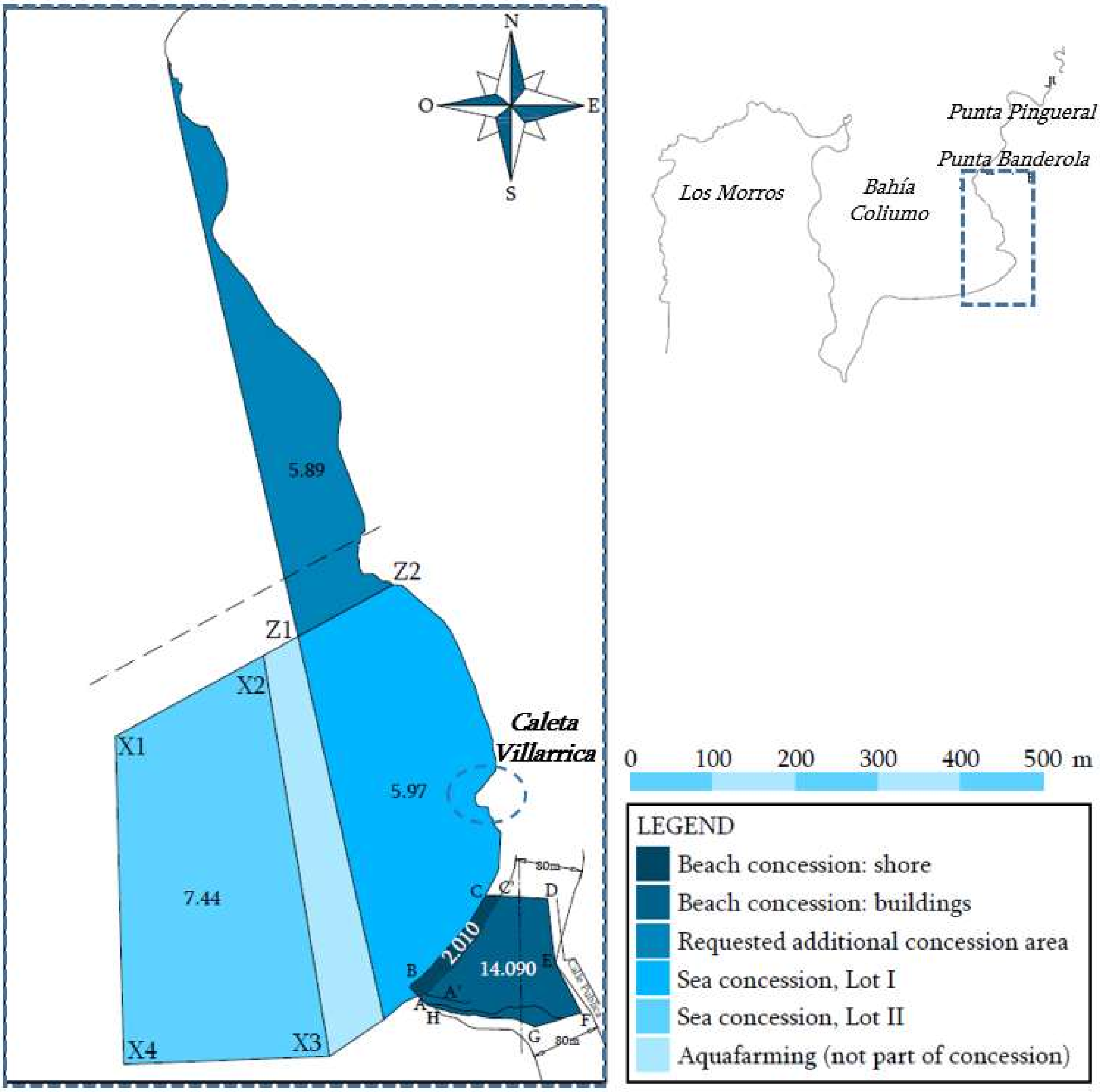


Figure 1-3: UdeC marine concession in Coliumo Bay

1.1.3 SITE HISTORY

Table 1-1 gives an overview of the changes in the appearance and features of the Villarrica harbour in the period April 2004 to January 2016. In 1957 a dock was built on steel piles embedded in concrete foundations, with a wooden platform of 340 m^2. Currently, only the piles remain, which are heavily corroded. This dock was registered in the name of the Internal Tax Services. The EBMD was inaugurated by UdeC in November of 1978. Since 2003, the UdeC marine concession exists as shown in Figure 1-3. Several projects were constructed, namely an access pier for the docking of the Kay Kay II vessel (Figure 1-4); a storage facility also serving as laboratory (green-roofed building in Figure 1-4); a concrete defence wall north of the pier; and a southern defence wall, made of masonry, which deteriorated quickly due to the severity of the elements -waves and tides in particular. Figure 1-4 shows some of these key features of the Villarrica harbour site, together with elevation contours and crosses marking SPT locations.

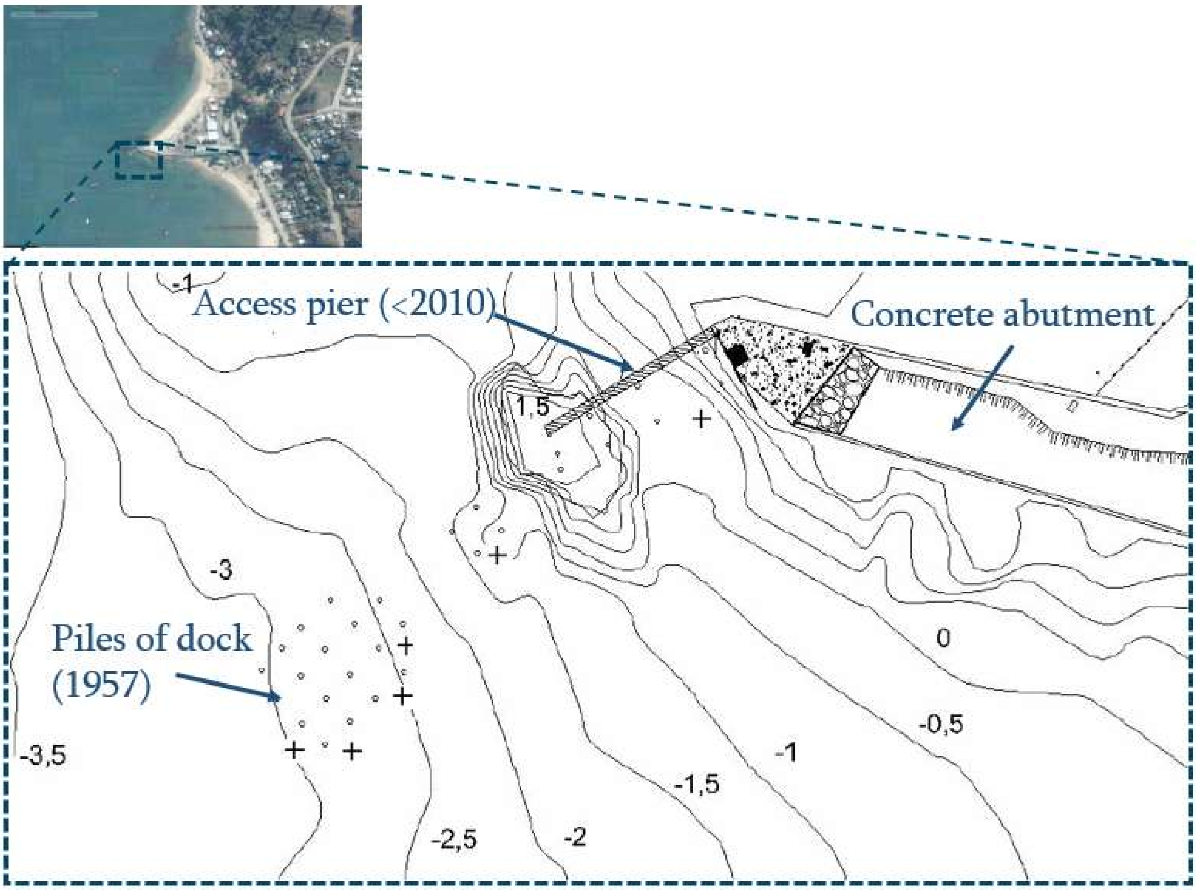


Figure 1-4: Historical maritime elements at Villarrica site

Table 1-1: Changes in Villaricca harbour site from 2004 to 2016

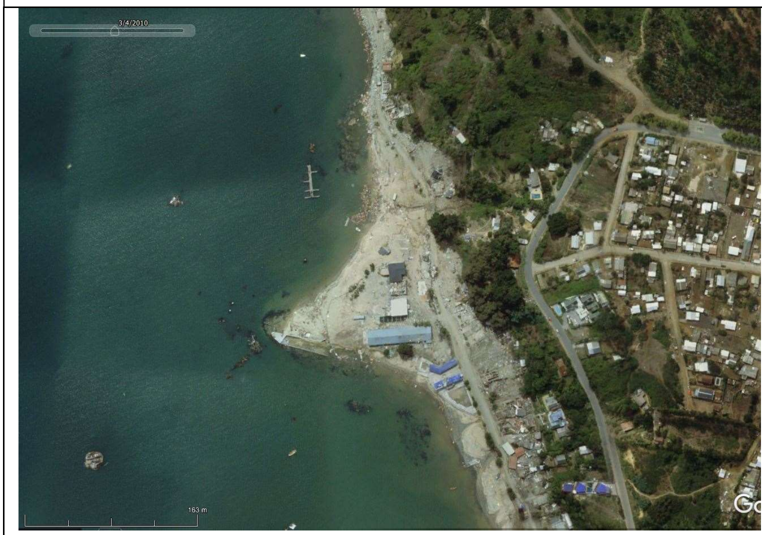
 *Figure 1-5: Caleta Villarrica in April 2004*	*April 2004* The EBMD access pier and storage facility are marked in Figure 1-5. In 2007 a sewage treatment plant was built to avoid contamination of the bay.
 *Figure 1-6: Caleta Villarrica in March 2010*	*March 2010* After the Maule earthquake and tsunami of February 27, 2010, the access pier has been destroyed and washed away completely. Houses are flushed away. The defence walls are also severely damaged. Tsunami-induced erosion is widespread. The concrete abutment remains.

Figure 1-7: Caleta Villarrica in January 2016

January 2016

Between 2011 and 2013, as part of Coastal Reconstruction Plan, buildings and roads are reconstructed and widened, respectively. A new 1 m concrete wall is constructed on the northern side of Caleta Villarrica.

1.1.3.1 Maule earthquake and tsunami of February 2010

On February 27, 2010, an earthquake of moment magnitude 8.8 struck Central Chile, with an epicentre in the Maule region. For more background information on this specific natural disaster as well as other earthquake and tsunami phenomena in Chile, see Appendix A.

The tsunami following the Maule earthquake left 80% of Dichato destroyed, severely affecting fishing and recreational activities. The wave height was between 5.3 and 7.3 m at the site of interest. Timber-framed homes and unreinforced or poorly reinforced masonry structures were particularly vulnerable. Many fishing boats washed up hundreds of metres inland. The tsunami flooded 80 hectares of the town and reached an inundation height of up to 4 m.

Reconstruction measures as part of the Coastal Reconstruction Plan (2011-2013) included widening of Dichato's main street; the creation of a 'mitigation park' as retention measure; an 800 m long coastal defence wall; and the reconstruction of 600 residences - including houses moved to elevated ground and houses built on stilts.

1.2 Problem description

1.2.1 Problem statement

From the site description and historical events emerges the following problem statement:

Currently the Marine Biology Station of UdeC cannot perform its main objectives in a safe and efficient manner, especially since the destruction of the jetty by the tsunami following the 2010 Maule earthquake. There is no location for mooring of the EBMD vessel and transhipment of goods, as well as incomplete construction or use of several onshore facilities.

1.2.2 PROJECT SCOPE

In order to meet the UdeC demand for scientific and academic support of marine sciences, the harbour complex at Villarrica must be redesigned. The UdeC marine concession marks the topographical limits of the project site. However, considering the pending expansion of the marine concession as shown in *Figure* 1-3, there is potential for enlargement of the harbour complex site. The harbour design will be performed according to Chilean design codes and standards. Reference may be made to similar projects in the Coliumo Bay and Concepción area, which are exposed to comparable natural and urban conditions. The design is limited to an exploration and evaluation of several alternative solutions, as well as a preliminary design of the solution determined to be optimum. Following the project, the design is put forward to the local Department of Public Works, who may consider elaboration of the design and provision of funds.

1.2.3 PROJECT OBJECTIVES

The primary objectives of the following interdisciplinary project are to:

- Identify all present natural and manmade features at the harbour site -including hydraulic, geotechnical and structural conditions- and consider future changes in any of these.
- Design a facility to allow the Marine Biology Station of UdeC to continue and expand its scientific and academic research projects in an efficient and safe manner.
- Design a facility of which the various elements may withstand, to an extent to be identified, the impact of earthquake loading and associated tsunami phenomena.
- Consider sustainable solutions which take into account all relevant stakeholders; full project life cycles and costs; and the surrounding environment.

1.3 REFERENCE PROJECTS

To assist as reference in the design of the jetty for the harbour complex, several jetties are visited and inspected. These visits provide guidelines on optimum structural configurations and construction methods in Chile. An unexhaustive summary of the site visits is listed below.

1.3.1 COLIUMO: TRADITIONAL JETTY

A field trip to the town of Coliumo, on the opposite side of the Bay to Dichato, a better insight in the building of simple jetties in Chile is obtained. In Coliumo, in the same bay of Coliumo, opposite of our project site, a jetty is recently build for the local fisherman. The jetty is bigger than what the requirements are for the Marine Biology Harbour complex (i.e. mooring place for several fishing boats), but nonetheless it gives a good insight.

1.3.1.1 Construction

The jetty is made in an L-shape, the concrete deck is poured in-situ, on steal H-beams. The beams are supported by steal piles, placed both straight and inclined, to absorb horizontal forces. The piles are founded on concrete slabs on the rocky seabed. A clear view of the construction discussed is given in figure Figure 1-8.

Figure 1-8: Construction detail of the deck, piles, beams and foundation slabs of the Coliumo jetty

Alongside the L-shaped deck there are several staircases in order to serve the ships at different tidal water levels. In Figure 1-9 an example at Coliumo is given. The total length of the stairs can be adjusted to the design, as well as the height to the tidal difference. Wooden piles are placed in front to serve as fenders and foundation to the seabed. Small bollards have been placed on the steel stairs, whereas bigger ones are placed on the deck. A crane to assist the ship has been placed on the deck next to the stairs.

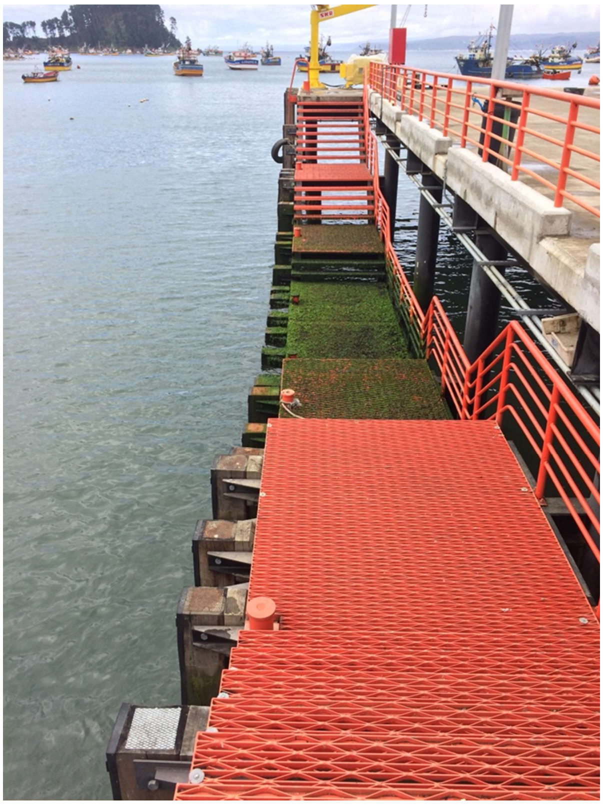


Figure 1-9: Levelled steel staircase at Coliumo jetty

1.3.1.2 Sea defence

In and around Coliumo not much sea defence is present as it is situated on the lea-side of the bay, in the shadow of a peninsula. In Figure 1-9 the peninsula is on the left side. The opening to the open sea is at the right of it, facing in North direction. In comparison to the marine biology harbour complex, Coliumo is much more sheltered from wave impact. A traditional jetty like this would need an additional sea defence in Dichato, on the opposite side of the bay.

1.3.2 BREAKWATER FOR JET SKI JETTY IN DICHATO

Although offshore sea defences are rarely seen in Chile, breakwaters are placed on occasion in small-size applications. In Dichato, for example, a breakwater has been placed on the windward side of a floating jet ski jetty, see Figure 1-10 and Figure 1-11. The original jetty (pre-2010) consisted of an extended dock of wooden piles and a concrete deck, of which only the first few metres withstood the 2010 tsunami and remain today as can be seen in Figure 1-10. The rest of the jetty has been replaced with a temporary floating one.

The breakwater, too, was destroyed by the tsunami wave, see the rubble of previous armourstone on the leeward side of the breakwater in Figure 1-11. The breakwater was reconstructed shortly after the tsunami simply by placing back the removed stones. No re-design of any of the elements in this jet ski docking facility was considered, as designing against tsunami impact is considered futile[1].

Figure 1-10: Jet ski floating jetty

[1] After interview with caretaker of jet ski docking facility, 23/12/2016.

Figure 1-11: Breakwater to the North of jet ski jetty

1.3.3 Jetty and dock failure mechanisms due to Maule 2010

The following images show jetties and other harbour facilities in Chile that were affected by the Maule 2010 earthquake and ensuing tsunami. Several failure mechanisms may be inferred from the photographs, as well as structural mitigation measures. These examples stem from a presentation given by Dr. Dechent of UdeC, on December 6th 2016.

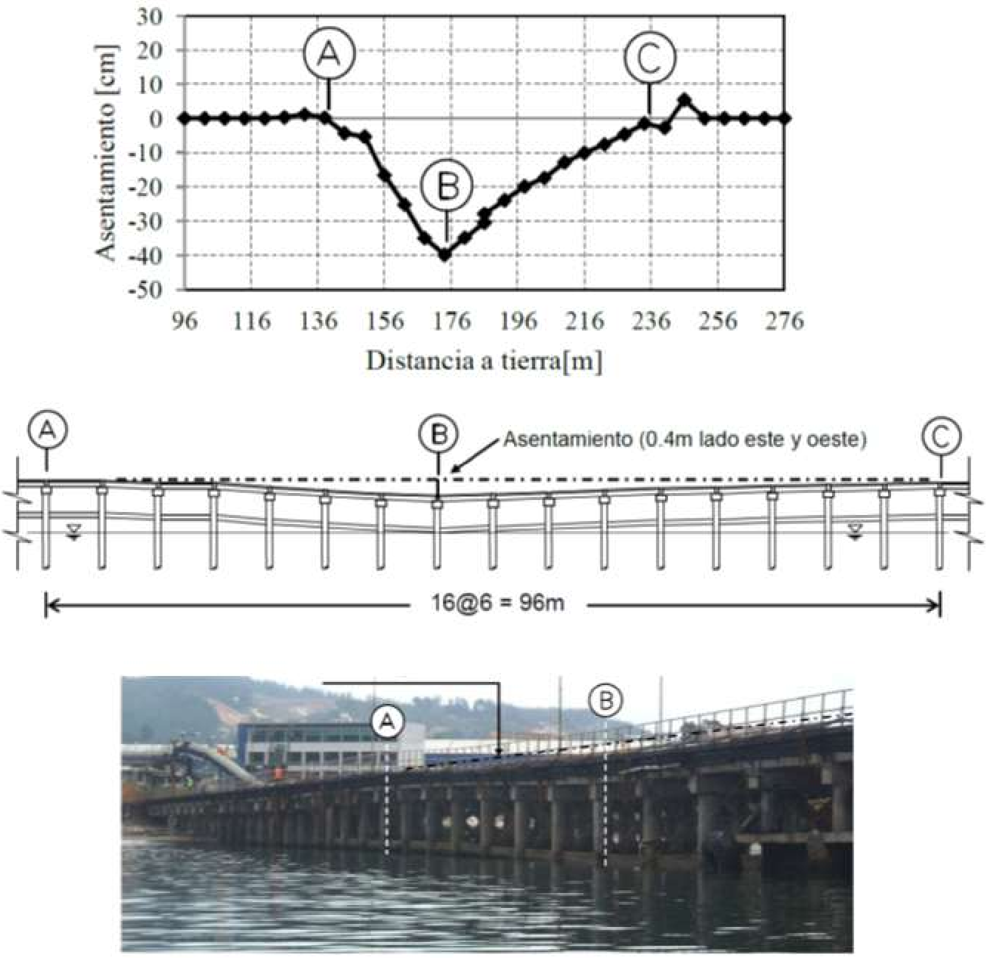


Figure 1-12: Liquefaction-induced differential settlements at Puerto Lirquén

Figure 1-13: System inclination due to lateral spreading at Muelle Jureles (left) and Muelle Coronel Sur (right)

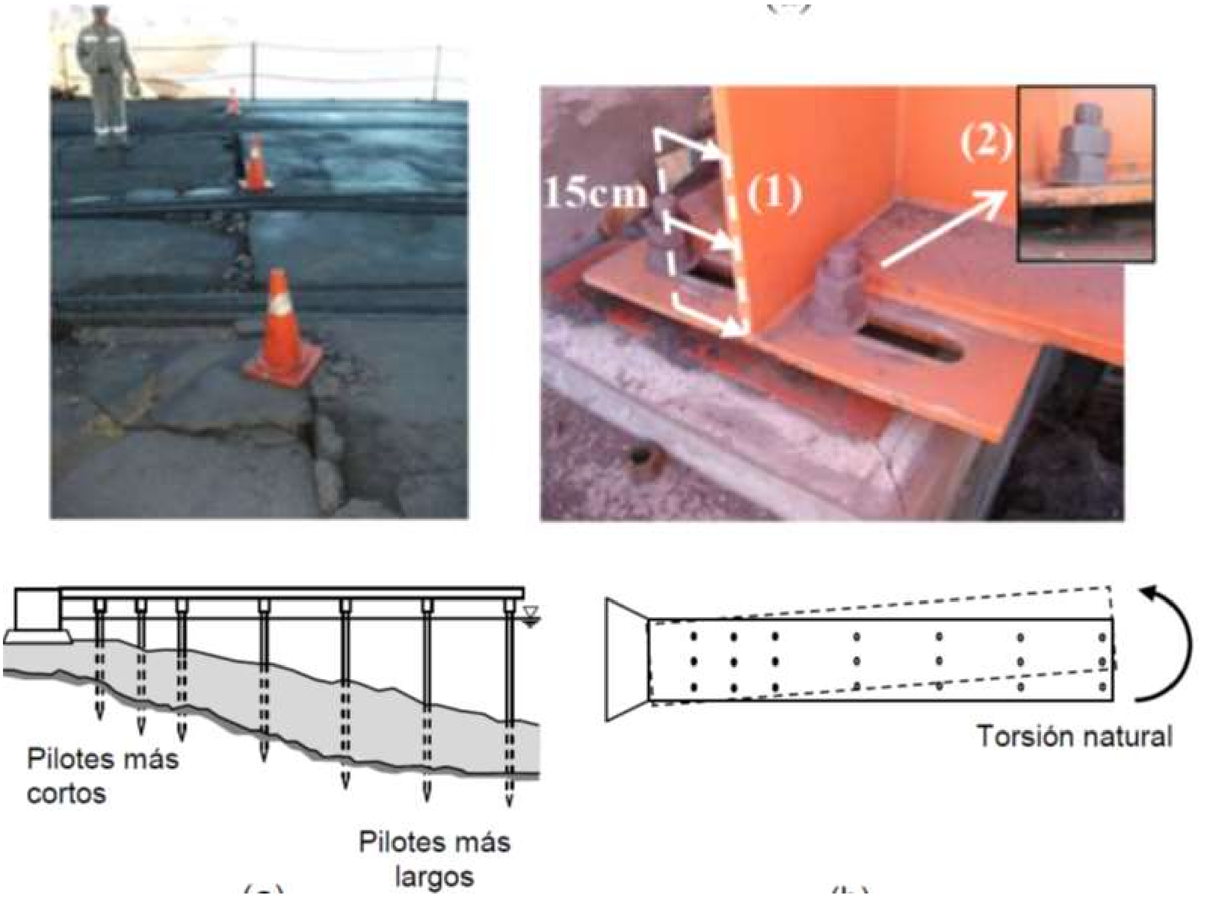


Figure 1-14: Jetty access displacement and concrete slab cracks due to torsion --a case of lateral earthquake loads and the short-pile effect at Muelle Huachipato.

2 PROBLEM ANALYSIS

To make sure the final design fulfils the needs of the EBMD, an extensive program of requirements is compiled. To achieve a complete program of requirements a method as depicted in Figure 2-1 is used. Analyses of the functions, boundary conditions, sustainability and safety requirements are carried out to make sure all needs and interests are included in the Program of Requirements. The different analyses are elaborated in the following chapter. Finally, the analyses are put together and result in the Program of Requirements. The Program of Requirements is used as basis to for design.

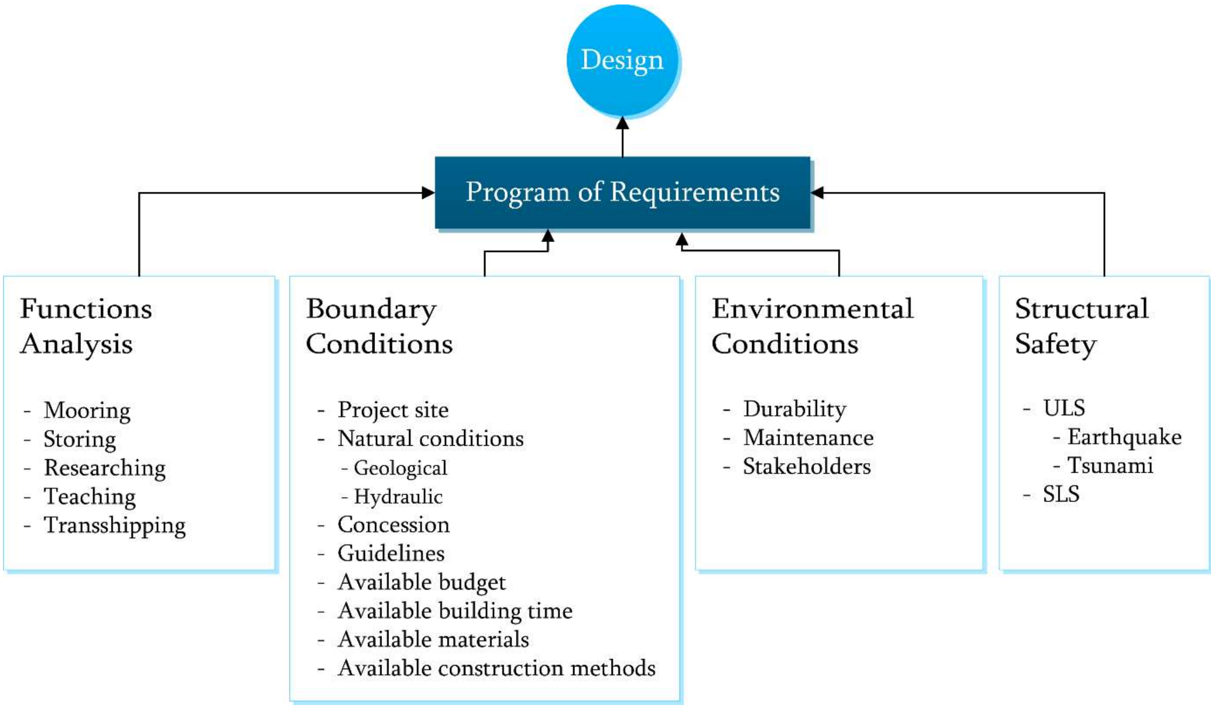


Figure 2-1: Method to achieve the Program of Requirements

2.1 FUNCTION ANALYSIS

The main functions of the Harbour Complex are: mooring; storing; researching; teaching; and transhipping. These main functions are divided into partial functions which result in the requirements to be fulfilled, as shown in *Figure 2-2*. A short elaboration of each main function follows.

2.1.1 MOORING

The first important function of the Harbour Complex is *mooring*. This function includes both the process of mooring the vessel as well as it being moored at the shore for a longer period of time. For the process of mooring a predefined annual downtime is accepted, whereas the function of being moored needs to be available constantly. Sufficient facilities must be present to make both functions possible.

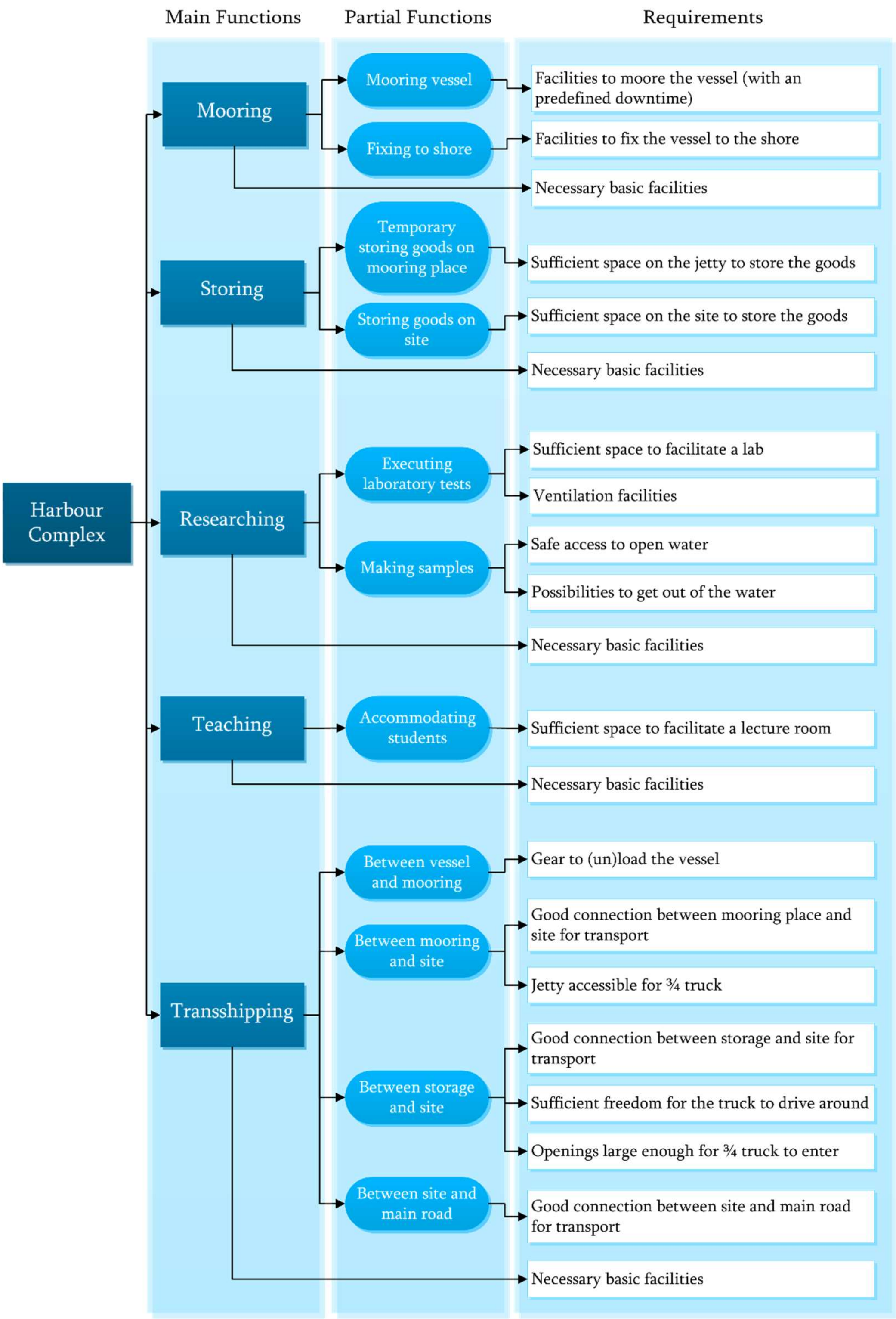


Figure 2-2: Function analysis diagram

2.1.2 STORING

Another main function is *storing*. The Harbour Complex needs to include sufficient facilities to enable storing goods both permanently and temporarily. To fulfil this need, enough storage space is required. Furthermore, facilities are necessary to protect the goods against possible damage due to environmental causes.

2.1.3 RESEARCHING

A more educational based function is *researching*. The Marine Biology Station (EBMD), to which the vessel belongs, is part of the Department of Oceanography. This means that there is need for the execution of multiple laboratory tests and sampling. The execution of laboratory tests requires sufficient space to facilitate laboratories. Furthermore, sufficient ventilation facilities are needed to guarantee good working conditions for users. The sampling requires access to the open water and possibilities to exit the water in case of accidents.

2.1.4 TEACHING

Another more educational based function is *teaching*. To teach the students, facilities are needed to accommodate all the students. This need requires sufficient space to facilitate a lecture room.

2.1.5 TRANSSHIPPING

The last main function is *transshipping*. To guarantee functioning of the harbour complex, all different parts must be connected correctly. The connection between the vessel and the mooring place requires a crane to load and unload the vessel. The connection between the mooring place and the site must contain sufficient facilities for transport and the jetty must be accessible for a ¾ truck[2]. To facilitate transshipping between the site and the storage, sufficient space must be available for the ¾ truck to drive around. On top of that, openings must be large enough for the ¾ truck to enter. Lastly, a good connection is required between the site and the main road to ensure easy transportation.

2.1.6 BASIC FACILITIES

Besides the facilities to provide for the main functions, there are also necessary basic facilities. These basic facilities are meant to provide a feasible and workable situation for fulfilling the main functions. They are not elaborated in detail in this part of the design process, but will be cited in general as depicted in Figure 2-1.

[2] Refers to light-duty trucks of GVWR (Gross vehicle weight rating) of 5000 kg, LxWxH of 6.0 x 2.5 x 2.0 m^3 (Ford F-250 as reference), with 2 axes with wheelbase 360 cm.

2.2 BOUNDARY CONDITIONS

2.2.1 GEOTECHNICAL CONDITIONS

2.2.1.1 Regional geology

The geologic map in Figure 2-3 shows the regional geology. A rock and soil type description of the geologic units relevant to the region around the Dichato harbour and Coliumo bay. Figure 2-3 also depicts the presence of two faults to the South of Dichato, running diagonally NW to SE. For the geologic history of the region, consult Appendix B.1

Table 2-1: Geologic units at and around Dichato site and Coliumo Bay, see Figure 2-3.

Unit abbreviation	Era / period	Period / epoch	Description	Associated formations at the coast
Qm	Quaternary	Pleistocene-Holocene	Coastal deposits: sands and gravels	
E1c	Cenozoic	Eocene	Paralytic continental sedimentary sequences: sandstones, shales and coal seams.	Trihueco, San José
PE1		Paleocene-Eocene	Marine and transitional sedimentary sequences: sandstones, calcareous siltstones and coal seams.	Curanilahue, Boca Lebu (Quiriquina)
Pz4b	Palaeozoic	Silurian-Carboniferous	Early Carboniferous slates, phyllites and meta-sandstones with low grade metamorphism.	
CPg		Carboniferous-Permian	Granites, granodiorites, tonalities and diorites, of hornblende and biotite, locally of muscovite. Compound batholiths and stocks.	

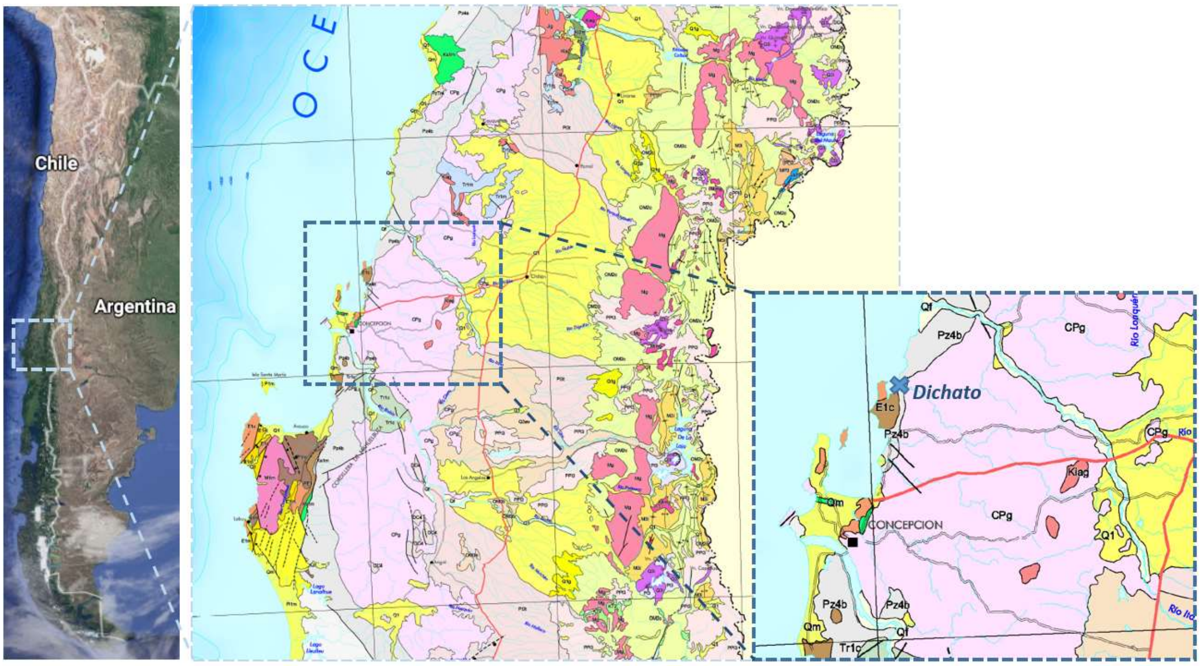


Figure 2-3: Location of scope area and geologic map (scale 1 : 100 000) (SERNAGEOMIN, 2009)

2.2.1.2 Geology at harbour complex

As part of the Lebu formation of PE1, the concession site is located in the Quiriquina Formation, which dates back to the Late Cretaceous from fossil evidence. It lies unconformably atop the metamorphic complex of Punta de Parra, of Palaeozoic age. The formation is made up of sedimentary rock of which the lithology may be divided into 4 sections: (1) basal conglomerate (2) yellow sandstone with conglomerate lenses (3) coquina layers intercalated with sandstone and (4) sand- and siltstone with sandy-calcareous concretions.

2.2.1.3 Geotechnical ground mass characterisation

In February 2010 the Soil Mechanics Laboratory carried out 6 Standard Penetration Tests (SPTs) off the coast of Caleta Villarrica, as marked by the crosses in Figure 2-4. The SPTs were performed using an ACKER MC-2 model, mounted on a raft, with 1.75 inch inside diameter tubes. Records of the SPT penetration indices, water levels, determination of index properties and the USCS classifications tests of the recovered samples are given in Appendix B.4, as well as photographs of retrieved samples. A second test was carried out onshore in November 2016: a geophysical survey using geophones and a Nakamura set-up. Please consult Appendix B.4.3 for an explanation of the survey and the results, which give an indication of the stratigraphy of the subsurface at the Marine Biology Station up to 70 m depth.

Figure 2-5 illustrates the results of the SPT, where a distinction is made between three soil or rock types according to USCS standards: SM (silty sand); ML (inorganic silt of low plasticity); and, at the rejection point of the penetration cone, rock (sandstone with silty matrix). It may be noted that the depth of the sandstone increases irregularly towards the shore, with a rock outcrop between S-3 and S-6. Also, the presence of silty sand is limited to the locations of S-1 and S-6.

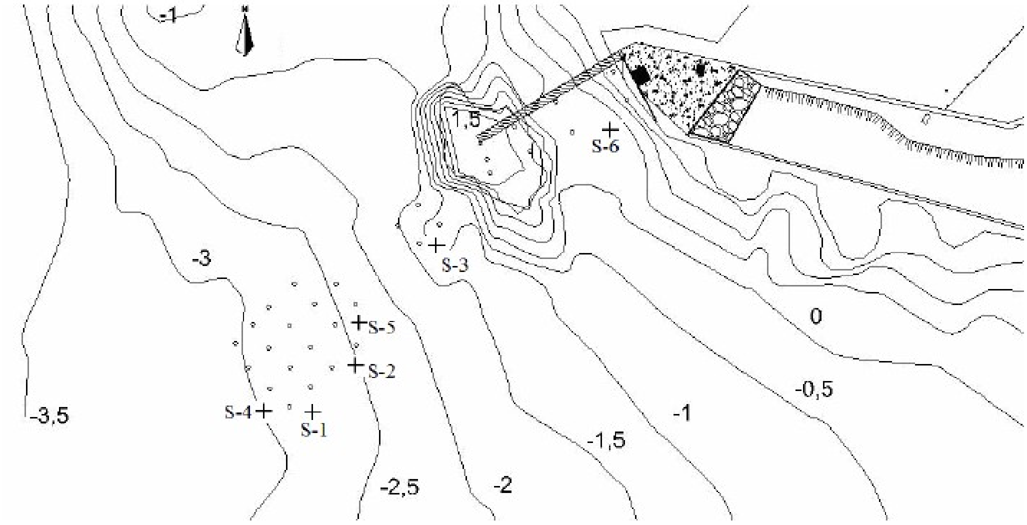


Figure 2-4: Locations of SPTs performed in February 2010. MLW (Mean Low Water level case, see Figure 2-12*)*

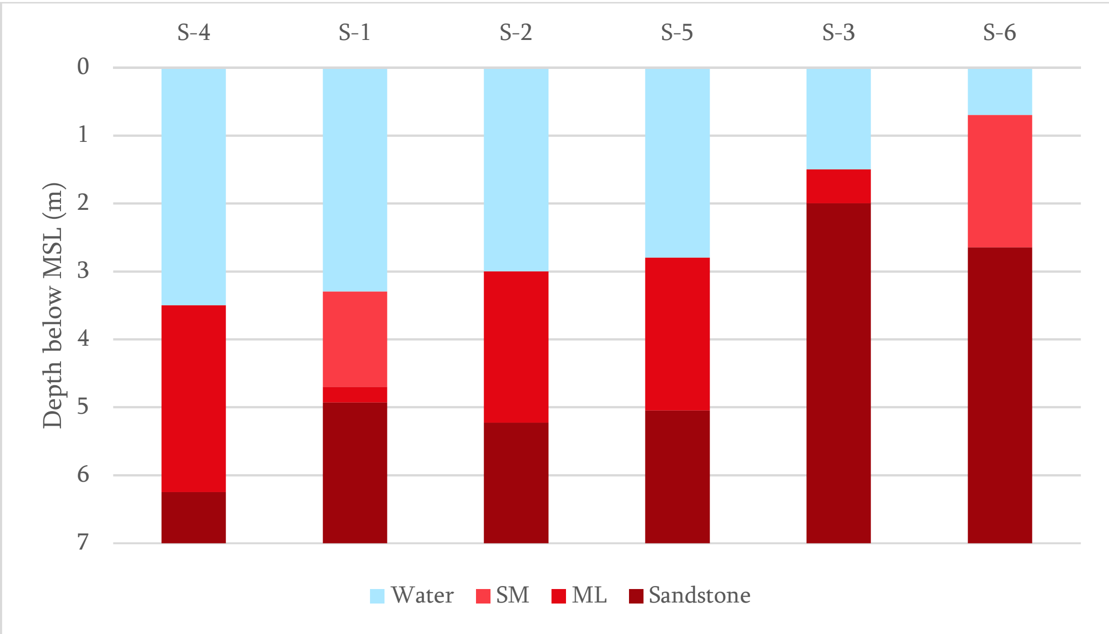


Figure 2-5: SPT results (arranged in SW-NE direction)

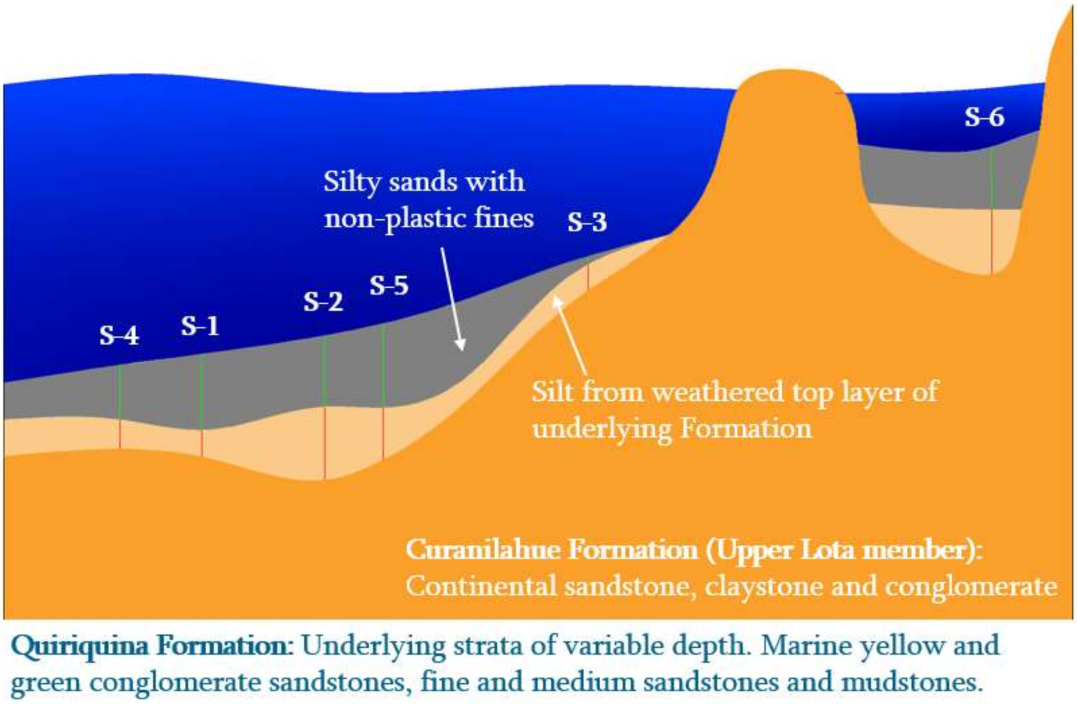


Figure 2-6: Geological cross-section at Caleta Villarrica (not to scale)

2.2.1.4 Overlying soil strata

From the SPT records, the classification and properties are derived in Table B-1 in Appendix B.4. Table B-2 gives the associated N values for each sample. The N value is a function packing density and grading of the soil, and may be linked with properties such as friction angle and safe bearing pressure, SBP, which are also given in the tables (Waltham, 2009).

2.2.1.5 Rock bed

Since the depth of the overlying soils is limited and the bearing capacity of the silty sand is quite low, foundations are preferably constructed on the underlying sandstone. This Curanilahue rock was also tested in the Soil Mechanics Laboratory of UdeC in 2010, and was found to have a uniaxial compressive strength of 102.2 kg/cm^2, or 10.0 MPa. Based on a combination of two sources (Waltham, 2009) (Price, 2008) the SBP of the sandstone lies between 1 and 2 MPa.

2.2.1.6 Failure behaviour

The Curanilahue sandstone failed with a fragile failure mode during the UCS test conducted by the Soil Mechanics Laboratory of UdeC (Sandoval Munoz, 2010). More information on failure mechanisms, both for foundations for the jetty as for foundations for onshore structures, is given in Appendix B.5.

2.2.1.7 Groundwater conditions

The groundwater table at the Marine Biology Station fluctuates around the Mean Sea Level, MSL.

2.2.1.8 Availability and evaluation of construction material

Two main types of required construction material may be identified: 1) armourstone (coarse aggregate) for use in hydraulic structures and 2) concrete aggregate for potential use in both hydraulic and onshore structures. The main objectives of hydraulic structures are volume filling; providing a foundation and filtering system; and protecting the structure against wave or current action and scouring. Natural armourstone is usually ideal in all three of these cases (Rock Manual). The tailings of quarried armourstone are often used as core material. Table 2-2 and Table 2-3 give properties and an evaluation of the relevant rock types with regards to hydraulic and other structural applications (Construction Industry Research, Information Association,et al., 2007) (Waltham, 2009).

Table 2-2: Relevant properties of available rock types when quarried

Rock type	Density (t/m^3)	Maximum size[3]	Shape
Slate	2.7-2.8	LG	Tabular
Shale	2.3-2.7	LG	Tabular
Sandstone	2.3-2.8	LG	Tabular
Granite	2.5-2.8	HG	Equant

Table 2-3: Generalised evaluation of use of available soils and rock in hydraulic structures and as construction aggregate

Rock or soil type		Evaluation as construction material		
		Armourstone / core material	Concrete aggregate	Pavement aggregate
Coastal deposits (*Qm*)		(+/-) Not suitable as armourstone due to limited size of clasts. Suitable as filter material and in core, if silt and clay content is low enough.	(++) Rounded clasts make for well-flowing concrete.	(+) Angular crushed quarry material is better than natural gravel. Coal impurities require filtering out, else may react with bitumen binders.
Schistose rocks and mudrocks	Slate (*Pz4b*)	(+/-) Not suitable as armourstone. Limited suitability as core fill due to platy block shapes and may form low permeability barriers when compacted. Mica-rich zones are much weaker than normal and require identification. Weatherability depends greatly on	(-) Largely unsuitable as pavement material, or as concrete aggregate due to inadequate strength and flaky shape of crushed material. Silicates may react with alkalis in cement and cause expansion.	

[3] CG: coarse grading, LG: light grading, HG: heavy grading, see Table 3.5 of Rock Manual (Construction Industry Research, Information Association,et al., 2007) for mass sieve percentages for each type of grading.

		quartz or clay mineral content. Dust may be an issue on construction sites		
	Shale (*E1c*)	(-) Limited suitability as armourstone due to instability when exposed (swelling, slaking). Filter zones and drains may be blocked or cemented due to weathering. Suitable as core material.	(-) When containing sulphides, the rock may attack concrete. Usually not suitable for use in concrete, for pavements.	
Sandstones	Continental (*E1c*), marine (*PE1*)	(+/-) Limited suitability as armourstone. Medium or low strength units may not produce free-draining rockfill. Quartzites can present quarrying and compacting difficulty. Rocks may lose significant amount of strength upon saturation due to decomposition of cement or matrix. Presence of interbeds of shale or claystone are hazardous.	(+) Suitable as aggregate in concrete. Silica-rich rocks may induce alkali-silica reaction in concrete (expansion and cracking).	(-) High percent wear due to abrasiveness. Softer and more porous than ideally used in pavement aggregate.
	Metamorphosed (*Pz4b*)	(+) Suitable as armourstone due to high density and strength.	(+) Generally suitable	(+) Generally suitable
***Granitic rock** (CPg)*	Fresh	(++) Very strong and durable, suitable for armourstone and filter material. Tailings may be used in core.	(++) Test how mica-rich the granite is - fine, platy particles make it unsuitable as fine aggregate. Alkali-aggregate reaction depends on age.	(++) Angular particles are highly suitable
	Weathered	(+) Can be weathered in extreme to form silty or clayey fine gravels or sands. Good for core material, but can be highly erodible if silty and may need blanketing.	(+) Generally suitable	(+) If parent rock is very coarse grained, gravels can be used as road sub-base

2.2.1.9 Foundation design

Foundations are designed according to the Chilean national code NCh433 (earthquake resistant design). See Appendix B.5 for a more detailed analysis of foundation design in Chile. At the harbour site, in terms of seismic design, the existing rock and soil types may be classified according to the Chilean soil type classification as presented in Table 2-4 (NCh2369.Of2003 (table 5.3)).

Another consideration is the location of the existing pile remains, which are steel piles with concrete 'shoes', see Figure 2-4 for locations. The old piles will be cut for construction of a new jetty, but the concrete shoes which remain at the sea floor ought to be avoided when driving new piles.

Table 2-4: Seismic classification of soils and rock on site

Soil or rock unit	Description	Estimated v_s (m/s)	Soil type
SM	Silty sand	<200	C (III)
ML	Low plasticity silt	200-400	C (III)
Sedimentary rock[4]	Weak sandstone from Curanilahue formation	300-500	B (II)
Underlying rock	Sedimentary rock from Quiriquina formation	> 500	A (I)

2.2.2 HYDRAULIC CONDITIONS

The hydraulic conditions in the Coliumo Bay are key for the hydraulic calculations of the design. Different conditions are covered in this paragraph, including the most important ones: offshore waves and bathymetry. Information from the two will be combined in a SWAN (Simulating Waves Nearshore) model, in order to gain insight in the exact wave conditions near the shore. Furthermore, water level, currents, wind, salinity and tide are covered in this chapter.

2.2.2.1 Bathymetry

A good insight in the bathymetry is of major importance as it serves as input for the modelling of the forces on construction. A detailed zoom-in is given in Figure 2-7, as also the greater depths further off from the shore are significant in making a good translation of offshore wave conditions to the nearshore wave properties. Figure 2-7 also gives the grid point where the wave and wind data were measured.

[4] Of importance: presumed bearing stratum for foundation design.

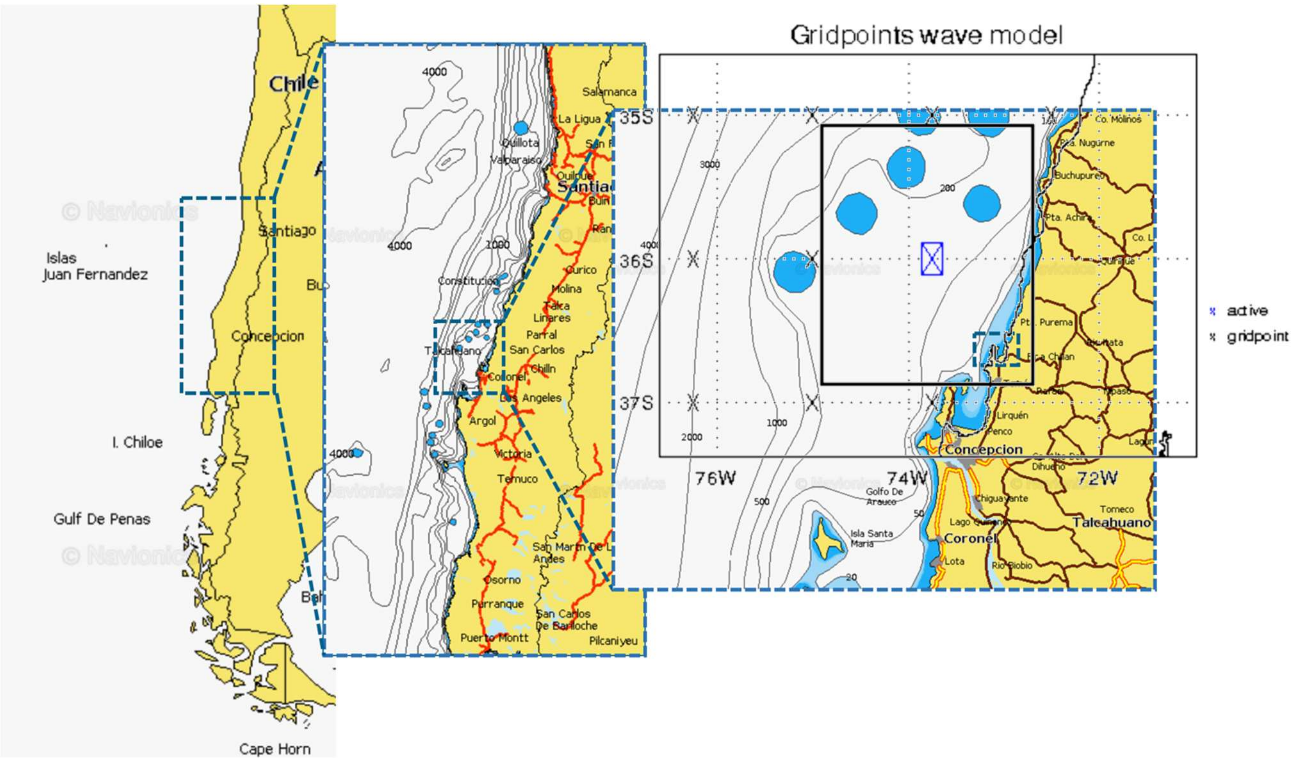


Figure 2-7: Bathymetry. Source: Navionics & WaveClimate.org

The detailed bathymetry of the Coliumo Bay is relevant for the structural design as well, as it has to be suitable for the normative vessel.

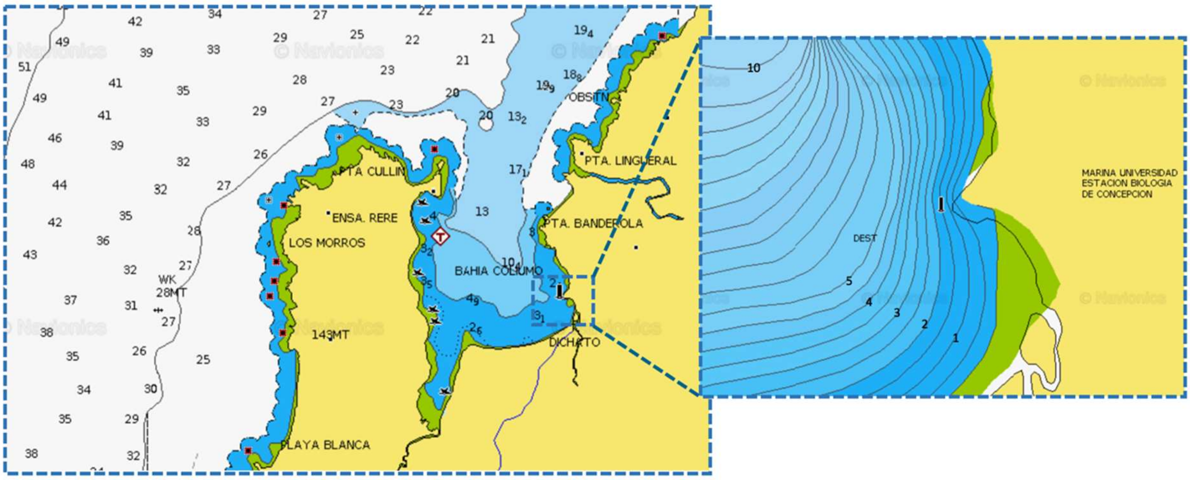
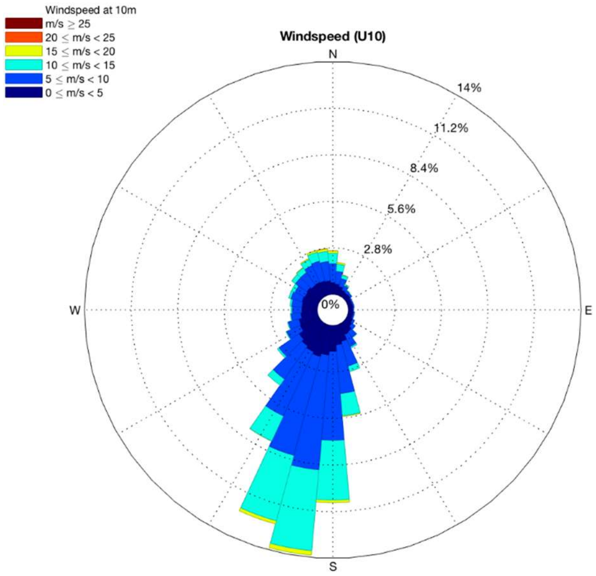


Figure 2-8: Seabed profile. Source: Navionics

2.2.2.2 Wind (u_{10})

The climate in this region is mostly influenced by anticyclone wind caused by a high pressure cell in the central pacific. In this particular area, the prevailing wind directions are south to south-south-west during summer. During winter, northern winds are more frequently occurring. This can be seen in Figure 2-9. The figure on the left describes the origin of the winds. A big peak at the bottom means from the south. The longer a line is, the more often it occurs. The colours describe the magnitude of the wind. The average maximum winds in Dichato are in the order of 15-20 m/s, both from the north and south. In the figure on the right, it is made clear that the summer winds (from the south) have a higher average value than the winter (northern) winds. Nevertheless, the storms in the winter can be of great importance for the design. The highest wind speeds will originate from the north, as occurred in August 2015. A storm with wind speeds of 35 m/s left great damage at the coast of middle Chile (Winckler, 2016).



Monthly distribution of wind speed (m/s)

lower	upper	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
00	01	1.3	2.4	2.0	3.7	3.8	3.1	3.2	3.2	2.8	1.9	1.0	1.1
01	02	3.4	4.0	4.0	5.6	6.3	4.7	5.4	5.3	4.5	4.2	2.2	2.6
02	03	4.5	5.2	5.8	8.2	9.5	8.0	8.4	8.9	7.4	5.9	4.4	4.5
03	04	5.3	6.7	8.3	9.9	11.1	10.1	9.2	9.9	10.0	8.8	6.6	5.9
04	05	6.4	7.4	9.2	10.0	11.5	10.0	11.0	10.1	10.3	9.5	8.0	7.6
05	06	7.3	8.0	9.6	10.7	10.1	10.3	10.6	9.4	10.2	10.6	9.1	7.7
06	07	9.3	9.2	10.7	9.5	9.2	9.3	9.6	10.2	10.7	11.2	9.6	8.2
07	08	8.5	9.4	9.9	9.3	9.1	9.3	8.5	9.8	10.1	11.5	10.7	9.6
08	09	10.0	10.2	9.1	8.5	8.2	7.3	7.2	8.2	9.3	10.6	10.0	10.6
09	10	9.4	9.3	9.1	7.8	5.8	7.9	6.9	7.1	7.1	8.9	10.4	11.0
10	11	9.4	8.9	7.5	6.0	4.7	5.8	5.6	5.3	6.1	7.0	9.7	9.4
11	12	8.6	7.1	6.2	5.0	3.9	4.8	5.8	4.1	4.7	4.7	7.0	8.3
12	13	6.6	5.4	4.0	3.2	2.1	3.0	3.5	2.8	3.0	2.8	5.1	6.1
13	14	4.9	3.3	2.5	1.7	1.7	2.1	2.1	2.0	1.7	1.5	3.4	4.0
14	15	3.0	2.4	1.2	0.7	1.4	1.7	1.4	1.4	1.2	0.5	1.7	1.8
15	16	1.2	0.9	0.7	0.2	0.6	0.9	0.9	0.9	0.6	0.2	0.9	1.2
16	17	0.6	0.4	0.2	0.2	0.4	0.7	0.3	0.5	0.2	0.0	0.3	0.3
17	18	0.1	0.0	0.0	0.1	0.2	0.3	0.2	0.4	0.1	0.1	0.0	0.1
18	19	0.0	0	0	0.0	0.3	0.4	0.1	0.2	0.1	0.0	0.0	0.0
19	20	0.0	0	0	0	0.1	0.2	0.1	0.1	0.0	0	0.0	0
20	21	0	0	0	0	0.1	0.1	0.0	0.1	0	0.0	0	0
21	22	0	0	0	0	0.0	0.0	0.1	0	0	0.0	0	0
22	23	0	0	0	0	0.1	0.1	0.0	0.0	0	0	0	0
23	24	0	0	0	0	0.0	0.0	0	0	0	0	0	0
24	25	0	0	0	0	0	0	0	0	0	0	0	0
total		100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Figure 2-9: Wind directions/ percentage of occurrence and intensity

2.2.2.3 Tides

The tidal system at the west coast of South America is governed by rotary movement of an amphidromical point in the South Pacific. In front of the Chilean coast the tide is dominated by a micro-tidal regime with a mean tidal spring range of up to 2 meters. This results in a semi-diurnal tidal character in the Coliumo Bay, as can be seen in Figure 2-10. A high tide and a second (lower) high tide occur in every daily cycle of 24 hours. The tides do not result in a large intertidal area in the Coliumo bay, because of the relatively steep slope of the coast. This information is important for the coastal processes.

As there is not much data available of the tide elevations in Chile, an analysis of the tidal elevation of one whole year is executed. The data is obtained from the Talcahuano wave buoy, approximately 15km south of Dichato (lat/lon -36.700846/ -73.106259). The period within data is measured is from 26-11-2015 till 22-11-2016 (IOC, 2016).

After removing some odd values the result is displayed in Figure 2-10. Clearly the semi-diurnal character of the tide can be observed, as the higher peaks and lower peaks are separated. In a more detailed plot this becomes even more clear, Figure 2-11, these are the observations of one month.

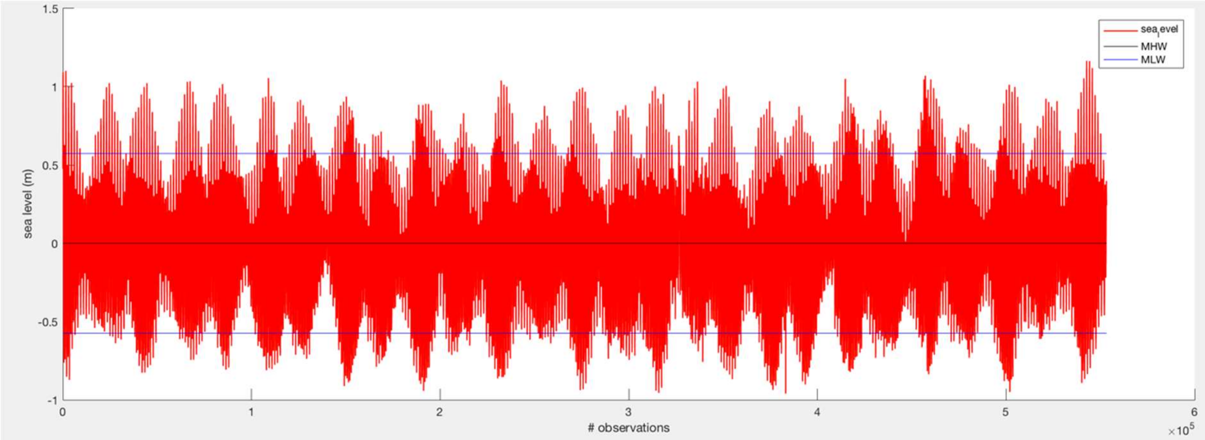


Figure 2-10: Tidal observations of 1 year from the Talcahuano buoy

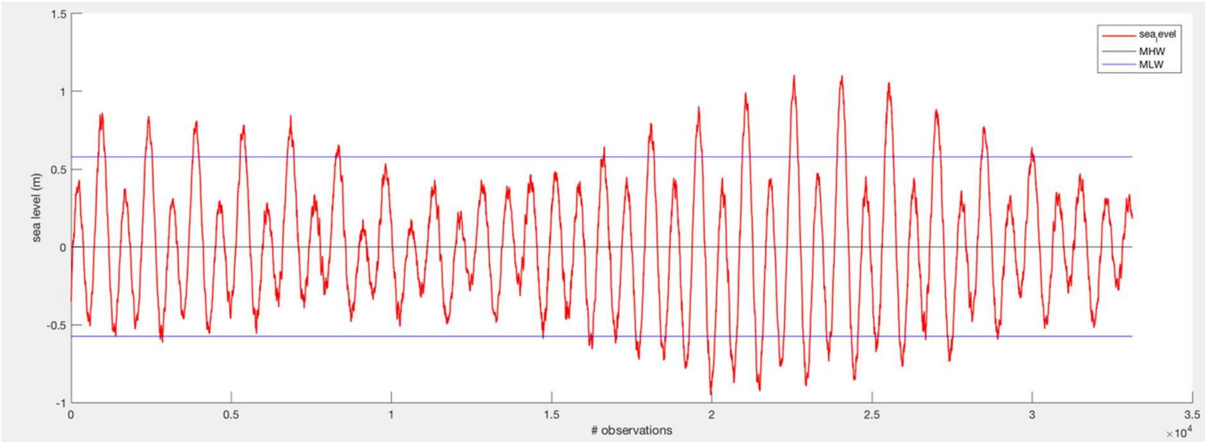


Figure 2-11: Tidal observations of 1 month from the Talcahuano buoy

From the data, a mean high water level is calculated, by calculating the maxima per tidal cycle. Those values are averaged and represent the MHW. The same is done for the mean low water. The highest all time (HAT) and lowest all time (LAT) is obtained from the data as well. In *Figure 2-12* a summary of the values is displayed. An average tidal range (ATR) of 1.15m is the result of the mean high and low water tide.

The Mean Sea Level (MSL) will be adopted as reference level during this project.

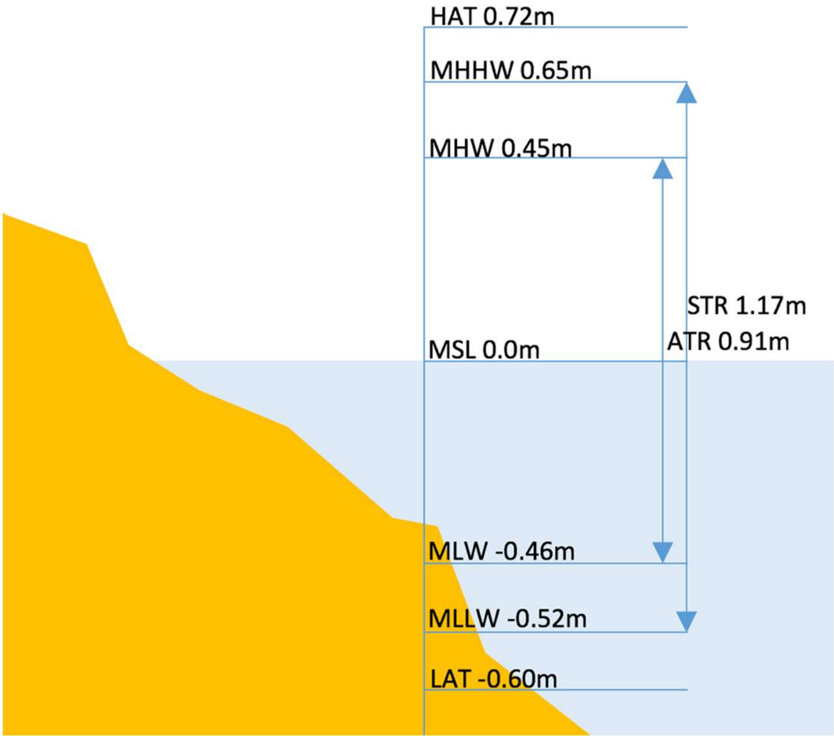


Figure 2-12: Tidal system of the Coliumo Bay
HAT = Highest All Time; MHHW= Mean Highest High Water level; MHW = Mean High Water level; MSL = Mean Sea Level; MLW = Mean Low Water level; MLLW = Meanest Lowest Low Water level; and LAT = Lowest All Time.

2.2.2.4 Waves (H_s)

The waves offshore of Dichato are mainly from two distinct directions, the south-west (SW) and the north-north-west (NNW). The NNW waves are more directly incoming into the bay of Coliumo, as can be seen in the description of the bathymetry. Nevertheless, both scenarios will be elaborated extensively.

The waves from the south-west are the prevailing waves, as can clearly be seen in *Figure 2-13*. These are mainly swell waves with their origin around the 'roaring forties' above Antarctica. The significant wave heights (H_s) are around 7 m offshore and have a peak period (T_p) of 14.5s (BMT Argoss, 2016). This is without the storm conditions from the south-west, where significant wave heights are in the order of 9m (T_p 15.5 s).

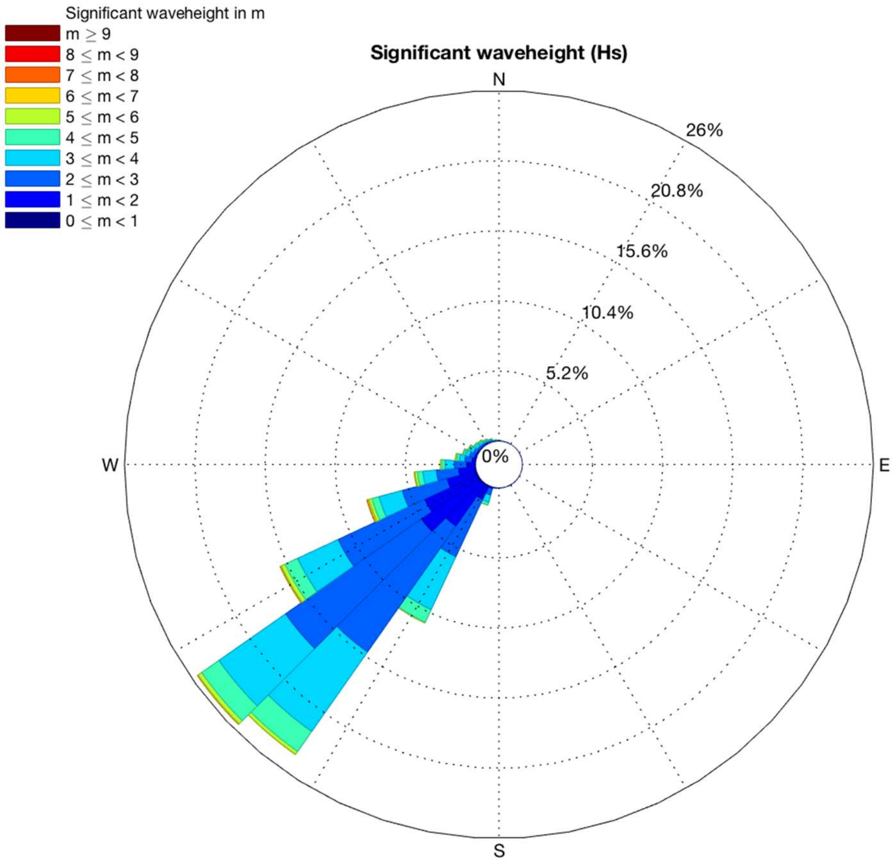


Figure 2-13: Wave heights and directions offshore at Coliumo Bay. Clearly the main direction, the SW, is visible. Note that the NW winter storms are of importance for the design as well.

The second important direction of waves is from the north-north-west. These waves are wind waves, originated from storms (mainly during the southern hemisphere winter) off the coast off Chile. A very destructive storm from this particular direction occurred in August 2015. Wind speeds (v_{10}) of 35m/s and offshore wave heights of 10.23m with a peak period (T_p) of 7s where measured along the coast. The storm is very well documented and analysed (Winckler, 2016). To give an indication of the force of the NNW storm, a picture at Valparaíso is displayed in Figure 2-14.

Figure 2-14: Winterstorm August 2015, at the port side of Valparaíso (Winckler, 2016).

An extensive analysis of the wave heights and directions is performed in Appendix H, in which the waves are plotted against their directions. A Weibull in combination with a peak-over-threshold analysis will be performed to extract the different wave heights for the different scenarios.

In Part II: Design, the different scenarios of the wave heights will be translated to on shore values using Delft3D and SWAN, both software from the TU Delft, also elaborated in Appendix H.

2.2.2.5 Currents

To get an insight in currents along the Coliumo bay, two different views can be taken. The first one is a world view, the second is on the level of the bay. The Peru-Humboldt current is the large-scale eastern boundary current, driven by the global wind patterns, more specifically, driven by the Antarctic Circumpolar current (West Wind Drift) and the trade winds in the Hadley cell. This current has an average speed of maximum 10 km/day, has a width of approximately 1000km and goes up to a depth of 500m. Figure 2-15 gives a view of the origin of the current and its position in the world circulation (Pietrzak, 2011).

The situation on bay level might be more interesting for the specific case, as currents in the bay itself would result in sand transport. Two little rivers (Estero Coliumo and Estero Dichato) end in the bay. During fieldwork, it became clear that these two small rivers will not influence the design at all. Their catchment area is fairly small and so is their discharge.

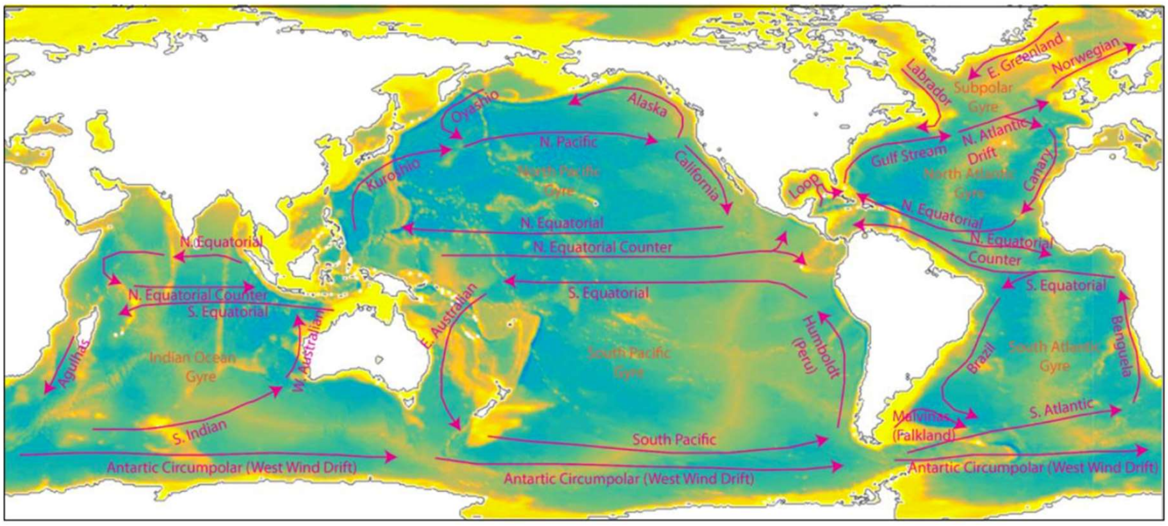


Figure 2-15: Main ocean currents. Figure generated by James Salmon using the WOCE data base.

2.2.2.6 Salinity

The salinity of the seawater near Dichato is around 34.5 PSU (Practical Salinity Unit). This equals approximately 34.5 g/kg salt. This value is somewhat lower than for instance in the Atlantic Ocean. The lower value has its origin in the melting ice of Antarctica. The density of the seawater is around 1026 kg/m3 with an average temperature of 14 °C (Pietrzak, 2011).

2.2.3 Structural conditions

2.2.3.1 Current situation

As mentioned in Chapter 1.1.3: Site History, in 1957 a dock was constructed from steel piles and a wooden deck. Nowadays, only the steel piles remain, heavily eroded, embedded in concrete foundations. The piles could be removed, but their foundations will remain and should be taken into account in new designs.

Another structural aspect to be taken into account for the jetty design is the concrete abutment as mentioned in Chapter 1.1.3. This part of the previous harbour still solidly remains and could be used as an integrated part in the design of the harbour complex. Its dimensions are indicated in Figure 2-16. The concrete abutment is strengthened on its southern side by protruding concrete elements.

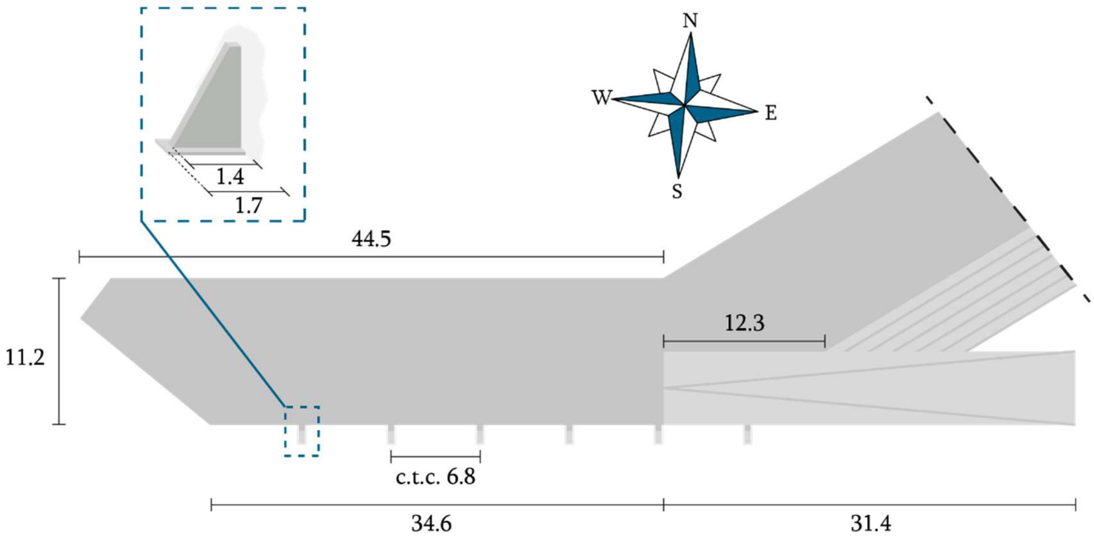


Figure 2-16: Concrete abutment (dimensions in m)

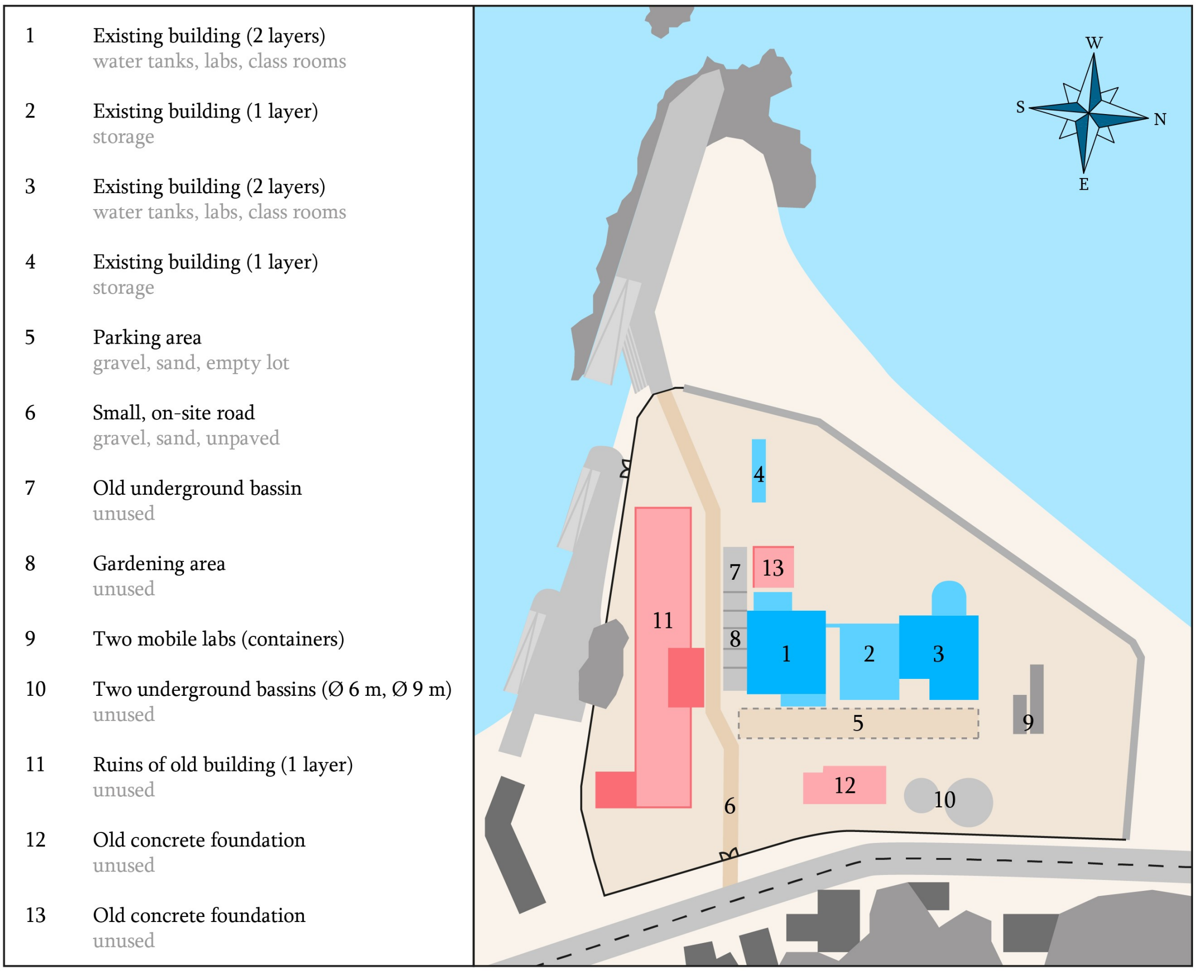


Figure 2-17: Current situation Marine Biology Station

Figure 2-17 shows the current situation of the Marine Biology Station, focusing on the onshore facilities. The buildings marked 1, 2, 3 are currently in a good condition. They have been developed after the tsunami of February 17, 2010, and are therefore in no need of structural attention. On the ground floor level, some attention should be paid to the parking area (number 5) and the small on-site road (number 6). From a structural point of view, redevelopment of the structures 11, 12 and 13 is of interest.

2.2.3.2 Potential onshore redevelopment

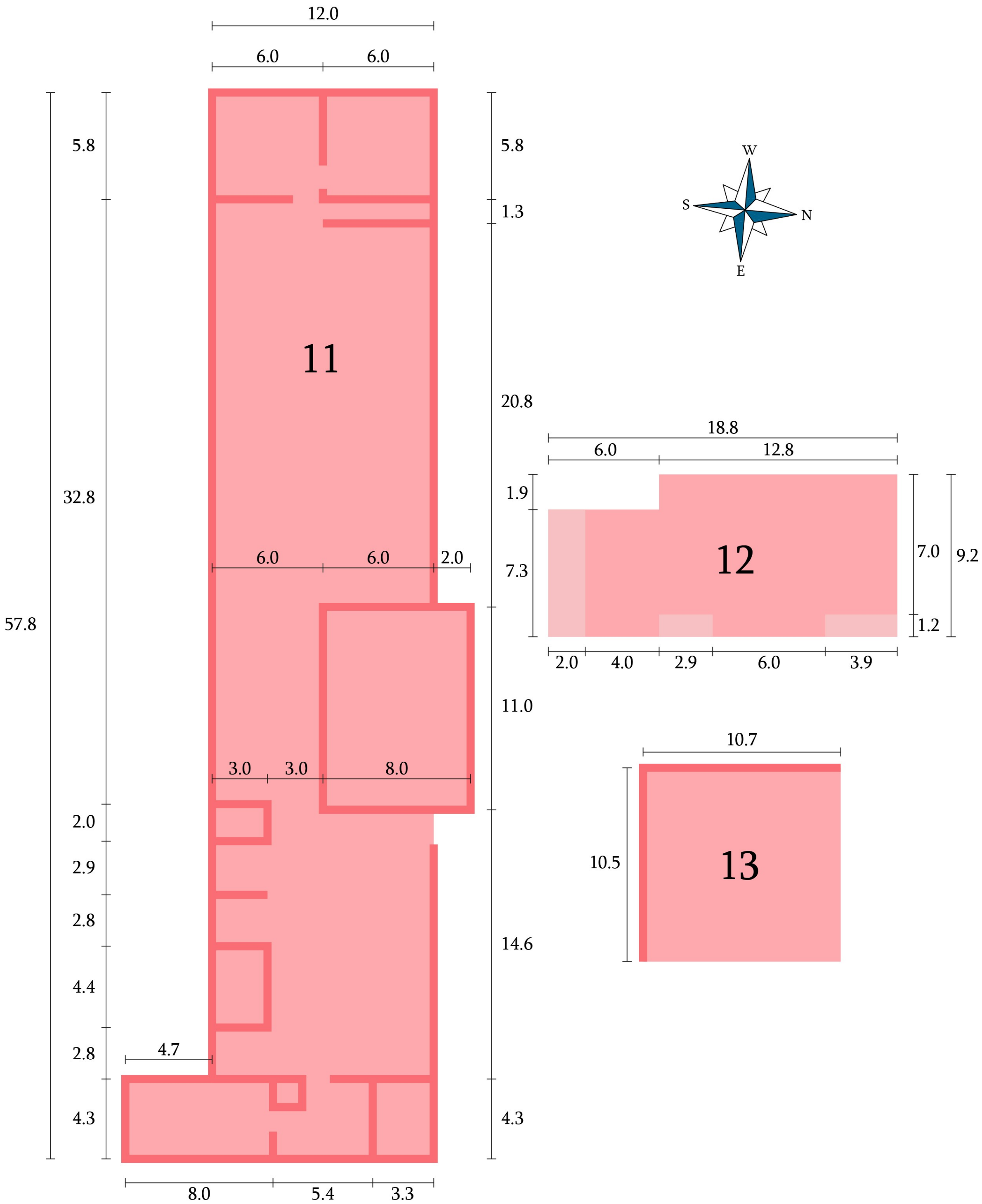


Figure 2-18: Onshore structures suited for potential structural redevelopment (dimensions in m)

The remains of the old on-shore structures numbered 11, 12, 13 have potential to be structurally redeveloped (see Figure 2-18). The largest structure, building 11, has no ceiling. Only walls remain up to a height of one building layer. The on-site operator who was spoken with during a site visit on December 1, 2016, said that building 11 requires similar use as building 1 and 3 of Figure 2-17.

Redevelopment would entail designing laboratories and research facilities on the ground floor, and constructing another level for class rooms and presentation areas.

Structure 12 only has concrete foundations remaining, used for a small cabin in the past, with a few units for nightly stays. The building has been wiped away and its original function has not been restored in the current building stock. It would be preferable if sleeping units yet again became a part of the on-site complex.

In the last structure, numbered 13, only the concrete foundations remain as well as low walls on two of the four sides. The previous function of this structure remains unknown, but it could prove a good site for additional functions as the foundations are already in-place.

2.2.3.3 Relevant standards

The Chilean standards to be taken into account during this project are listed below.

NCh1537:	analysis of the self-weight and dead/live solicitations
NCh431:	analysis of the snow solicitations
NCh427:	structures of steel
NCh430:	structures of reinforced concrete
NCh1928:	structures of reinforced masonry (ceramic bricks/concrete blocks)
NCh2123:	structures of masonry (ceramic bricks/concrete blocks) confined with beams and columns of reinforced concrete
Ordenanza General de Urbanismo y Construcciones:	structures of stone masonry confined with beams and columns of reinforced concrete
NCh1198:	structures of wood
NCh935/NCh2209:	prevention of fire in buildings
NCh3171:	load combinations
NCh433:	earthquake-resistant design of buildings
NCh2745:	analysis of buildings with seismic isolation
NCh2369:	earthquake-resistant design of industrial structures and facilities

The Chilean standards are largely based on the standards of the United States of America. The abovementioned standards are a shortlist and can be complemented by other relevant standards if required.

2.2.3.4 Building materials and systems

The Chilean standard NCh433 clearly summarizes the categories in which the building methods are divided.

1. Walls and other braced systems

The gravitational and seismic actions are resisted by walls, or by braced portals that resist seismic actions by elements that work mainly in axial direction. There is a division made between the following subcategories:

- Structural steel
- Reinforced concrete
- Reinforced concrete and confined masonry
- Timber
- Confined masonry
- Reinforced masonry (both concrete blocks and ceramic bricks)

2. Portal systems

The gravitational actions and the seismic actions in both analysis directions are resisted by portals. There is a division made between the following subcategories:

- Structural steel
- Reinforced concrete

3. Mixed systems

The gravitational and seismic loads are resisted by a combination of the previous systems.

The abovementioned subdivisions give a clear overview of the building materials that are generally applied in Chilean structures. For the final choice of building material and method(s) the available budget and the natural conditions of the location are relevant factors to be taken into account.

2.2.4 General construction methods

The construction methods which are common in the Netherlands are very well applicable in the modern-day Chile. As far as is known up to this point, every crane and piece of machinery can be transported to the construction site. On site, there is enough space to manoeuvre. For machinery on the water, the small depth near the coast must be considered as well as the tidal differences. Currents will not influence the use of this machinery. Considering the possibility of bringing in machinery commonly used in the Netherlands, all the conventional building methods are applicable.

2.2.5 Construction time

The available construction time cannot yet be discussed in detail and is not in the primary scope of the project. However, it should be noted that it is not necessary to be constructed in a specific season due to Dichato's moderate climate. This might influence the total required construction time.

2.2.6 Budget

The budget for the project cannot be easily expressed. The Universidad de Concepción is the client and financer of the project and the Ministry of Public Works is a potential financer too. However, after the earthquake and tsunami of 2010 and its devastating results, the funds for public works, such as harbours, were emptied. This means that a new process must be initiated in order for the Ministry of Public Works to reconsider the (re)development of the Harbour Complex. This is implemented in the scope of the project. The project needs to be financially feasible. Moreover, the chances of success (in this case: further development and even execution of the project) are higher if numerous alternatives can be shown. This way, the financers are granted a thorough insight in multiple alternatives and their financial (dis)advantages. Once this is achieved, actual execution of the project is possible.

2.3 Environmental conditions

2.3.1 Life cycle

2.3.1.1 Rehabilitation of existing structures

The existing structures and remnants of structures, such as the wooden piles and steel beams from the previous dock, are highly deteriorated. The piles have been corroded by tidal level changes and the aggressive marine environment. The state of the existing concrete abutment, which withstood the 2010 tsunami relatively unscathed, requires further investigation. It must be inspected for structural damage and a sample must be taken and tested in the laboratory.

2.3.1.2 Durability

The durability of the structure refers to the to-be-determined design lifetime of the structure. For Dutch coastal works the lifetime is normally 100 years. In Dichato, Chile, this value is rather high. Due to its position in an area with high risk of earthquakes, this value should be taken lower. Moreover, the uncertainty in magnitudes (and more importantly Peak Ground Acceleration) is high, which would result in unacceptable high design criteria.

2.3.1.3 Lifetime

The *Design, Construction, Operation and Conservation Manual for Maritime and Coastal Works* of Chile (2013) delineates four classes for structural lifetime as shown in Table 2-5.

Table 2-5: Definition of design life time in accordance with ISO 2394:1998

Clase	Vida útil de diseño (años)	Ejemplo
1	1 – 5	Estructuras temporales
2	25	Elementos estructurales reemplazables
3	50	Estructuras y construcciones públicas. Estructuras que no clasifiquen en la Clase 4
4	100 o más	Estructuras de tipo: conmemorativo, especiales, alta importancia y de larga escala temporal

After discussion with Professor Dechent of UdeC, the lifetime expectancy of the harbour complex is set at 25 years (class 2), due to structurally replaceable elements of the facilities to be designed and the lack of public use of the mooring facility. After this period of time major maintenance or rehabilitation is acceptable, as well as the need to (partly) redesign parts of the harbour complex. Because of the high uncertainty of the exceedance of ultimate limit state and the occurrence of the design event, a reconsideration of the safety requirement may be necessary. Furthermore, a 25-year life time is feasible as the Maritime Concession expires in 2022, after which a re-evaluation of the functioning of the EBMD and all its facilities is conducted.

2.3.1.4 Maintenance

During its 25-year design lifetime, small maintenance is accepted, as unforeseen damage is unavoidable. However, the need for maintenance should be kept to a minimum in the design of the Harbour Complex. This is important as field research shows that maintenance is not as common in Chile as it is in the Netherlands - there are often organisational and financial difficulties.

2.3.2 STAKEHOLDERS

This paragraph describes all stakeholders, including all individuals, groups or organisations that have interest or concerns with regards to the (re)development of the Harbour Complex. Three types of stakeholders are distinguished, namely internal, external and interface stakeholders. The internal stakeholders are the parties which are part of the organisation. External stakeholders are parties which are not directly part of the organisation, but which are affected by its activities. Interface stakeholders are those who act both internally and externally. The stakeholder analysis for the Harbour Complex is displayed in Figure 2-19.

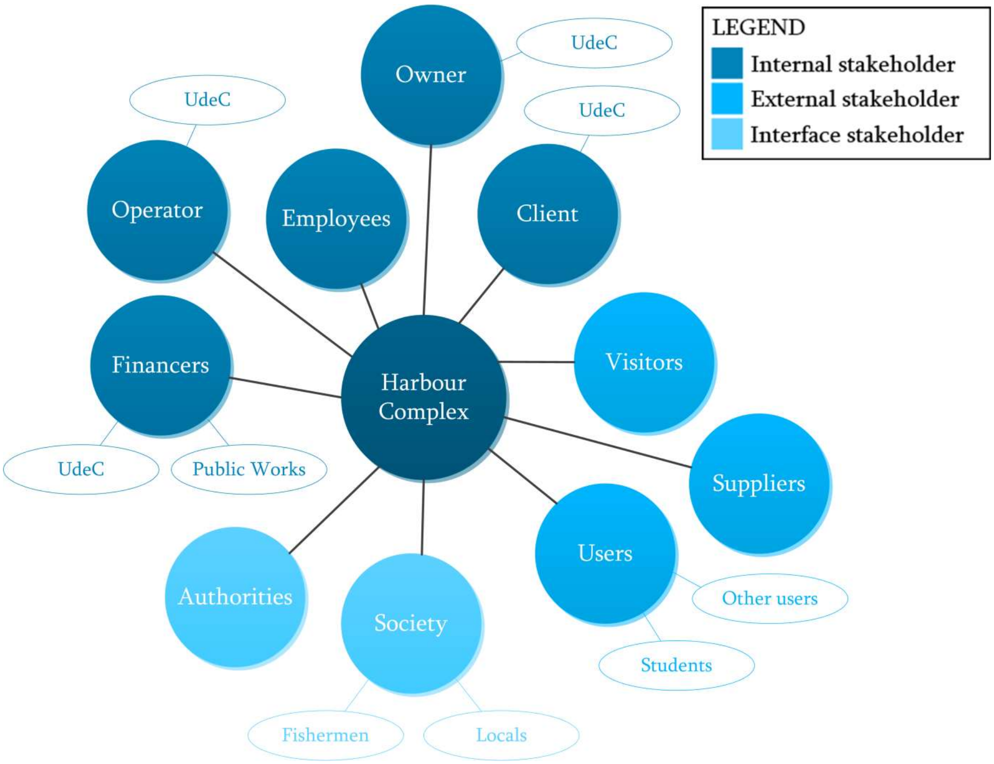


Figure 2-19: Stakeholder analysis

2.3.2.1 Internal stakeholders

The internal stakeholders include the owner, the client, the operator, financers and employees. The Harbour Complex is part of the Department of Oceanography of the Universidad de Concepción. This means that the Universidad de Concepción is the owner, the client and the operator of the Harbour Complex. The Universidad de Concepción is also responsible for the financing of the Harbour Complex, possibly together with Public Works. Unfortunately, the latter party is not yet confirmed. Another internal stakeholder is comprised of the employees. The interests and needs of the external stakeholders are summed up in Table 2-6.

Table 2-6: Overview internal stakeholders

Stakeholder	Interests and needs
Owner	· Durability · Sustainability · Good price/quality ratio
Client	· Efficient design (time, costs and functionality)
Operator	· Easy maintenance · Accessibility and clear logistics · Safety
Financers UdeC Public Works	· Proper use of their funding · Good brand awareness · Positive contribution to society
Employees	· Good and safe working conditions · Efficient functionality · Sufficient and adequate facilities

External stakeholders

The group of external stakeholders is formed by the users (including but not limited to the UdeC students), visitors and the suppliers. Accessibility of the complex plays an important role for all external stakeholders involved. The interests and needs of the external stakeholders are summed up in Table 2-7.

Table 2-7: Overview external stakeholders

Stakeholder	Interests and needs
Users Students Other users	· Accessibility of the Harbour Complex · Pleasant atmosphere and surroundings · Sufficient and adequate facilities
Visitors	· Accessibility of the Harbour Complex · Pleasant atmosphere and surroundings
Suppliers	· Accessibility and clear logistics

Interface stakeholders

The last category comprises of interface stakeholders, that are both internally and externally involved in the project. This category includes the authorities and the fishermen and other locals of this region. The interests and needs of the interface stakeholders are summed up in *Table 2-8*.

Table 2-8: Overview interface stakeholders

Stakeholder	Interests and needs
Authorities	· Design in compliance with rules and regulations · Socially responsible
Society Fishermen Locals	· No nuisance · No hindrance of local activities · Good overall appearance

2.4 Design Safety

In this investigation and design, the Chilean manner to cope with safety is adopted. The safety of the design arises from the degree of risk, associated with structural failure. Failure will occur when the limit state of the structure is reached and the structure is no longer able to fulfil its design requirements. This state may be reached due to an extreme event, and in practice is most often associated with seismic activity and associated tsunamis.

2.4.1 Extreme events

In a coastal region in Chile, like Coliumo bay, two different types of an extreme event can occur. The difference is depending on the location of the earthquake. In the case of an onshore earthquake it is less likely that a tsunami arises on sea, depending on the extent of the rupture zone. In the case of an offshore earthquake a common scenario is that the structure first endures one or more shocks. Subsequently the structure may be subjected to a tsunami, which can cause failure of an already damaged structure.

The probability of occurrence associated with extreme events is linked to the seismic gap as elaborated in Appendix A.3. Since the majority of the accumulated slip deficit (the last major earthquake in the Maule/Biobio region zone was in 1835) was filled during the 2010 earthquake, a large seismic event is unlikely to occur at this latitude in the near future (Esteban, Takagi, & Shibayama, Handbook of coastal disaster mitigation for engineers and planners, 2015).

2.4.2 RISK AND DESIGN PHILOSOPHY

Risk is defined as the damage multiplied by the probability (Jonkman, Vrijling, & Van Gelder, 2003). There are different studies which elaborate statistically the probability of occurrence of earthquakes and tsunamis. But in general, it is assumed that an earthquake and a tsunami will occur during the live time of a structure. Moreover, the general philosophy in Chile is that the probability of occurrence cannot be influenced, so the focus needs to be on the potential damage due to an extreme event to reduce the risk (Aranguiz & Martinez, 2016). This approach is different from the probabilistic Dutch method of determining risk of failure of a structure. This way a return period of a certain extreme event is not necessary to be calculated, as it is not of interest for the Chilean design.

The amount of damage can be divided in loss of life and a loss of economic value. The design needs to provide enough safety to minimize the probability of fatalities due to an extreme event. This implies that during an extreme event, the structure must withstand the earthquake so collapse of a structure does not cause fatalities. In general, it is not feasible to protect all areas from tsunami inundation, so the only option for people is to evacuate. This knowledge is deeply embedded in the Chilean society. See also: The Mapuche myth about Kay Kay and Treng Treng in Appendix A.3. This explains the high number of tourists who died during the tsunami event of 2010, whereas the number of fatalities amongst the local population was very low. For the design of the EBMD complex, this means that, concerning a tsunami, only the economic damage needs to be minimized.

To minimize the probability of fatalities due to an earthquake, structures must fulfil the legal regulations defined in the Chilean codes. In case of a tsunami, an evacuation procedure must be followed.

The resistance against the different failure mechanisms, as described in Chapter 9: *Impact and Risk Assessment,* due to both the earthquake, the tsunami and the associated economic damage, will differ from design to design. Minimizing the economic damage depends on choices made in the design process, which correlates with the initial cost. Thus, minimizing the economic damage will be seen as a design value. The evaluation of the economic damage will be based on the numerical model of the tsunami of 2010 (Aranguiz & Martinez, 2016).

Besides the earthquake and tsunami also other actions or failure mechanisms may break the adherence of the EBMD complex to the design requirements. In contrast to the total destruction caused by primary risks, in this situation failure will result in downtime of the mooring facility caused. According the wave climate in the harbour the authors of the Handbook of 'Coastal disaster mitigation for engineers and planners; advice for a harbour in a tsunami prone area, to design the wave protection against wind waves in the area. After that, at the end of the design procedure a check should be made to get insight in the effect of a tsunamis (see Chapter 9). So the usual method for the design of any wave protection is used. To design a protection against wind waves in the area an Ultimate Limit State and a Serviceability Limit State are formulated.

For the ULS a probability of failure of 10% under normal storm conditions (no tsunami conditions) is accepted. During a lifetime on 25 years. For the SLS, the downtime of the jetty is set on 5% per year. This downtime results in an acceptable non-service of 18 days a year. A more detailed description is given in Appendix H.1.3.

Some secondary risks may include the wave climate as described in Chapter 2.2.2: Hydraulic Conditions, and some minor geohazards as described in Appendix K.

2.5 Program of Requirements

The program of requirements is based on the analyses of the functions, boundary conditions, environmental conditions and safety requirements of the EBMD harbour complex. To get to a complete list of requirements, all those functions and conditions are worked out to a quantitative value. This means, for instance, for the sufficient draught, a value of ‘at least 2.25m at low tide’. Those quantitative requirements can be checked/fulfilled when designing the different parts of the harbour complex. The requirements receive a code for further reference.

Table 2-9: Program of Requirements, functions

Functions

	Origin	Code	Requirement	Value	Unit	Source
Mooring	1.1 Mooring vessel	F.1.1.1	Sufficient draught	-1.9	m	Ports&waterways 2
		F.1.1.2	Tide independent	-2.5	m+MSL	Ports&waterways 3
		F.1.1.3	Maximum wave height	0.3	m	Ports&waterways 4
	1.2 Fixing vessel	F.1.2.1	Fendersystem	8500	kg	Vessel charasteristics
		F.1.2.2	Bollard	3120	kg horizontal	Vessel charasteristics
				840	kg vertical	Vessel charasteristics
	1.3 Basic facilities	F.1.3.1	Lighting	50	lux	NEN-EN 12464-2
		F.1.3.2	Power connections	16A 220V		Vessel charasteristics
		F.1.3.3	Water connections			
		F.1.3.4	Fire fighting facilities	Berth reaching		
		F.1.3.5	Disposal water facilities			
		F.1.3.6	Rescue equipment	Safety stars Rescue buoy		
Storing	2.1 Goods on mooring	F.2.1.1	Storage	10	m2	
		F.2.1.2		5	kN/m	
	2.2 Goods on site	F.2.2.1	Dry storing	30	m2	
		F.2.2.2	Safe storing			
	2.3 Basic facilities	F.2.3.1	Lighting	100	lux	
		F.2.3.2	Power connections	32A 220V		
		F.2.3.3	Water connections			
		F.2.3.4	Air quality control	5	m3/hr/m2 fresh air	NEN 1089
Researching	3.1 Laboratory tests	F.3.1.1	Lab facilitaties	200	m2	
		F.3.1.2	Air quality control	20	m3/hr/m2 fresh air	NEN 1089
		F.3.1.3	Water quality control			
		F.3.1.4	Transport Seawater			
	3.2 making samples	F.3.2.1	Acces to open water	Mooring facility		
		F.3.2.2	Connection open water meanland			
	3.3 Basic facilities	F.3.3.1	Lighting	100	lux	
		F.3.3.2	Power connections	64A 220V		
		F.3.3.3	Water connections			
		F.3.3.4	Sanitary facilities			
Teaching	4.1 Student accomodatic	F.4.1.1	Facilitate lecturing	150	m2	
	4.2 Basic facilities	F.4.2.1	Lighting	100	lux	
		F.4.2.2	Power connections	32A 220V		
		F.4.2.3	Water connections			
		F.4.2.4	Sanitary facilities			
		F.4.2.5	Air quality control	15	m3/hr/m2 fresh air	NEN 1089
Transshipping	5.1 Vessel to mooring	F.5.1.1	(Un)loading vessel	1000	kg	
				4	m range	
	5.2 Mooring to site	F.5.2.1	Accessible for 3/4 truck	5000	kg	
				2.5	m width	
	5.3 Storage to site	F.5.3.1	Accessible for 3/4 truck	2.5	m width	
		F.5.3.2	Turning possibility truck			
	5.4 Site to main road	F.5.4.1	Connection main road			
	5.5 Basic facilities	F.5.5.1	Lightning	50	lux	
		F.5.5.2	Dewatering			

Table 2-10: Program of Requirements, boundary conditions

Boundary condition

Origin		Code	Requirement
Concession	1	BC.1	See chapter 1.1.2. Concession
Budget	2	BC.2	N.A.
Guidelines	3	BC.3	See chapter 2.2.3.3 Relevant standards
Geo	4	BC.4	Avoid existing piles
Hydraulic	5	BC.5	No restrictions
Building materials and systems	6	BC.6	Cannot re-use existing material

Table 2-11: Program of Requirements, environmental conditions

Environmental conditions

	Origin		code	Requirement
Life cycle	1.1 Existing structure		E.1.1.1	Rehabilitation
			E.1.1.2	
			E.1.1.3	
	1.2 Lifetime		E.1.2.1	25 years
			E.1.2.2	
	1.3 Durability		E.1.3.1	survive 25 years
			E.1.3.2	
	1.4 Maintenance		E.1.4.1	Minimum maintenace
			E.1.4.2	
Stakeholders	2.1 Internal			
		Owner	E.2.1.1	Durability
			E.2.1.2	Sustainability
			E.2.1.3	Good price/quality ratio
		Client	E.2.1.4	Efficient design (time, costs and functionality)
		Operator	E.2.1.5	Easy maintenance
			E.2.1.6	Accessibility and clear logistics
			E.2.1.7	Safety
		Financers	E.2.1.8	Proper use of funding
			E.2.1.9	Good brand awareness
			E.2.1.10	Positive contribution to society
		Employees	E.2.1.11	Good and safe working conditions
			E.2.1.12	Efficient functionality
			E.2.1.13	Sufficient and adequate facilities
	2.2 external			
		Users	E.2.2.1	Accessibility of the Harbour Complex
			E.2.2.2	Pleasant atmosphere and surroundings
			E.2.2.3	Sufficient and adequate facilities
		Visitors	E.2.2.4	Accessibility of the Harbour Complex
			E.2.2.5	Pleasant atmosphere and surroundings
		Suppliers	E.2.2.6	Accessibility and clear logistics
		Authorities	E.2.3.1	Design in compliance with rules and regulations
			E.2.3.2	Socially responsible
		Society	E.2.3.3	No nuisance
			E.2.3.4	No hindrance of local activities

Table 2-12: Program of Requirements, Safety Design

Safety

	Origin	code	Requirement
SLS	1.1 Permanent loads	S.1.1.1	See chapter 2.2.3.3
	1.2 Live loads	S.1.1.2	Relevant standards
	1.3 Wave climate	S.1.1.3	maximum downtime 5%
ULS	2.1 Earthquake	S.1.2.1	See chapter 2.2.3.3 Relevant standards
Damage control	3.1 Tsunami	S.1.3.2	Damage minimization (Damage factor)

3 DEVELOPMENT OF ALTERNATIVES

The development of the Dichato harbour complex involves three components: 1) a mooring facility 2) potential (re)development of onshore structures and 3) application of pavement to enable transportation from the mooring facility to other parts of the site. For each of these, alternatives may be defined as shown in Figure 3-1. The 5 options for mooring facility may be combined with 5 options for onshore structures and the 6 options for pavement, giving a total of 150 possible combinations.

However, considering the underlying focus of this project on the mooring facility as the primary part of the scope, five alternative solutions are formulated based around the 5 options for mooring facilities given in Figure 3-1. These solutions are illustrated in Figure 3-1 to Figure 3-5. Therefore, the ensuing Multi-Criteria Analysis and Cost-Benefit Analysis solely involve the mooring facilities. However, each solution carries a common theme and involves a combination with the other two aspects of onshore structures and pavement to highlight the unique selling point.

Combine all options (with or without breakwater)

Combine all options

Mooring facility	(Re) development of onshore structures	Application of pavement
Leave in current state	Leave in current state	Leave in current state
Use existing concrete abutment	Remove remains of old buildings only.	Gravel
Construct a traditional piled jetty	Reconstruct sleeping cabins on existing foundations.	Asphalt surface treatment
Construct a second perpendicular concrete abutment	Redevelop ruins of old buildings in similar fashion to existing major buildings (for educational and research purposes).	Normal concrete slabs
Construct a floating deck - jetty	Reconstruct all buildings on stilts	Thin concrete slabs
		Concrete deck on stilts as pavement

Figure 3-1: Alternatives for each of three Harbour Complex aspects

3.1 Null option (A): Apply no changes

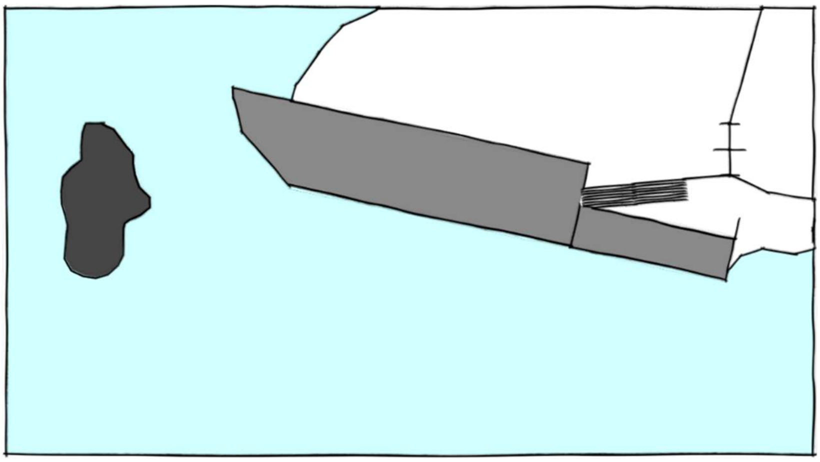


Figure 3-2: Null option (Alternative A)

For this option, the status quo is maintained. No jetty or wave protection is applied and transhipment continues to take place using a transitional smaller vessel.

3.2 Simple option (B): Re-use and upgrade

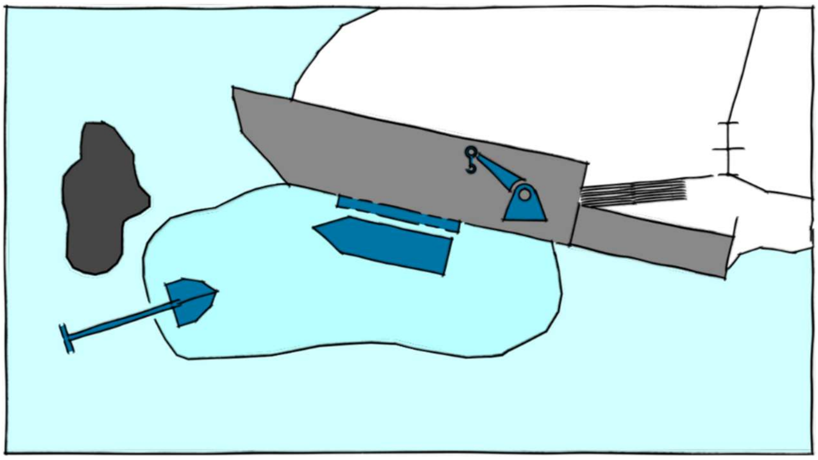


Figure 3-3: Simple option (Alternative B)

The construction time and costs are minimized for this option through the employment of the existing concrete abutment as a mooring facility. A staircase is built in front of the abutment in concrete or steel and a crane is installed on top. An area around the abutment is dredged to create sufficient depth for the vessel. Considering the mooring location at the lee-side of the Caleta Villarrica no additional wave defence system is applied. The rock outcrop as shown in Figure 3-3 might have to be removed for manoeuvrability purposes. Also, the quality of the concrete abutment needs checking and, potentially, upgrading.

3.3 TRADITIONAL OPTION (C): USE LOCAL REFERENCE PROJECTS

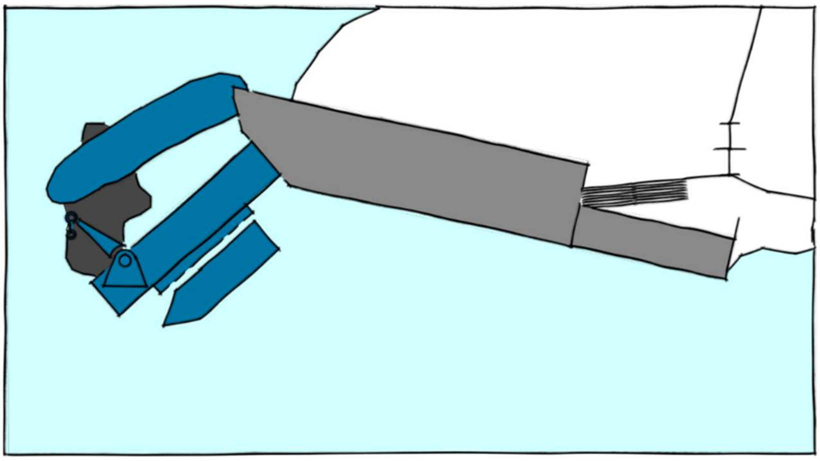


Figure 3-4: Traditional option (Alternative C)

As part of the Coastal Reconstruction Plan of 2011-2013, a new jetty was built in Coliumo, across the bay from Dichato, for docking and transhipping purposes for local fishermen. A similar concept may be applied for the EBMD jetty: a concrete deck on steel piles which are alternately inclined to resist horizontal forces from seismic or vessel impact loads. As in option B), a steel or concrete staircase is built on the lee-side of the structure and a crane is installed on the concrete deck. A rubble mount breakwater is erected on the windward side of the jetty. As in Coliumo and other harbour towns, a concrete slab road runs from the existing abutment to the onshore storage structure.

3.4 IMPACT PROOF OPTION (D): MAXIMIZE SEISMIC AND TSUNAMI RESISTANCE

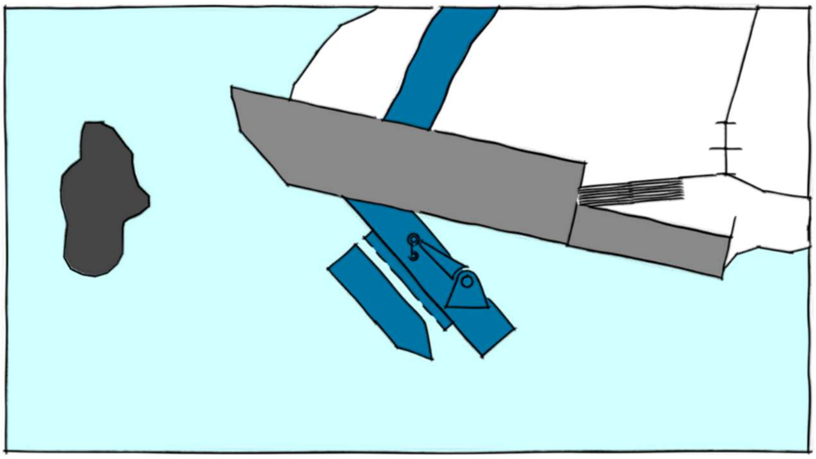


Figure 3-5: Impact proof option (Alternative D)

Considering the exceptional natural conditions to which the harbour complex may be subjected, it is important to consider a design which is as resistant as possible to seismic and tsunami loading. Taking into account the fact that the existing concrete abutment withstood the 2010 tsunami without significant damage, a second concrete abutment is constructed pointing southwards from the existing one to avoid the wave impacts of highest intensity. This second abutment serves as the mooring facility. On the windward (northern) side, an armourstone wall is erected as coastal protection.

3.5 FUTURE FLEXIBLE OPTION (E): ENABLE FUNCTIONAL AND SPATIAL FLEXIBILITY

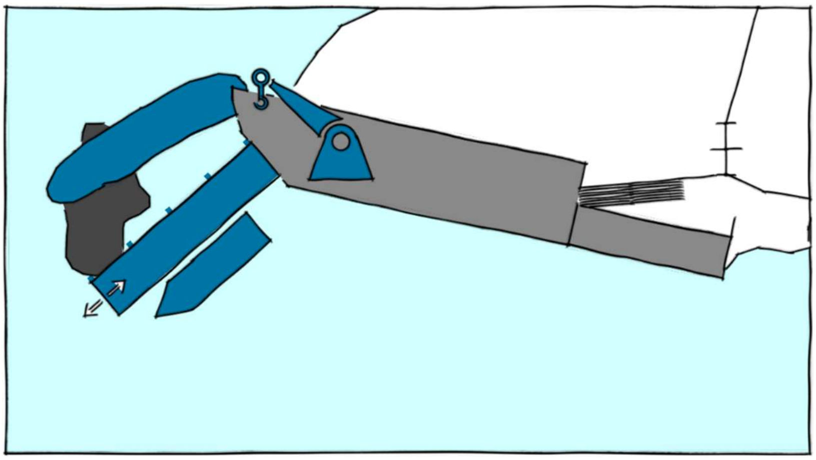


Figure 3-6: Future flexible option (Alternative E)

The Marine Biology Station may want to expand its operations in the future in terms of extent of storage and research facilities. Also, the function of the harbour as a whole could be made more adaptable to changing future conditions and requirements. To achieve this, a floating jetty deck is placed in between piles. This platform could be pulled onto the sea by the Marine Biology Station vessel in case of a tsunami, and its structural flexibility would allow it to withstand a high degree of lateral seismic loading. The rock outcrop would have to be removed, and a large crane must be installed on the concrete abutment instead of on the platform in this case.

3.6 OTHER FACILITIES: ONSHORE STRUCTURES AND PAVEMENT

The assignment of onshore structures and pavement alternative solutions is based on: knowledge of common solutions and best practice; the thematic coherence of each alternative A-E; and the wishes of the client as voiced during site visits. The several alternatives for the other on-site facilities are presented in Table 3-1.

Table 3-1: Onshore structures per alternative

	Alternative	Onshore structures	Pavement
A	Null	No upgrading is done in terms of functionality or space.	The on-site road and parking lot remains unpaved.
B	Simple	The new buildings are left untouched as they live up to good standards. The remains of other structures are removed to improve the quality of the surroundings. The only function currently absent on site are the sleeping cabins. They are to be constructed on the old foundations (see paragraph 0).	The road and parking lot are improved by strengthening the underlying layers with a surface treatment and adding a layer of gravel.
C	Traditional	The large ruins on the side of the area are redeveloped in a similar fashion to the existing on-site building stock. The remaining concrete walls on the ground floor are used as a basis for redesign, including laboratories and other research facilities, and a new floor is constructed on top for class rooms. As in the simple option, new cabins are designed on the old foundations.	The road and parking lot in this case are upgraded in a manner typical for Chilean roads: a thick concrete pavement.
D	Impact Proof	The concrete abutment is taken as a reference: this remained in place during the tsunami. Both the large building and the cabins (see paragraph 0) are reconstructed in thick concrete with limited façade openings. They contain one building layer as to reduce the impact of the potential tsunamis.	The road and parking lot are enhanced by applying a solidly anchored asphalt layer, embedded in the soil.
E	Future Flexible	The wish for flexibility is taken into account here. The remaining walls are taken down and the new buildings (both the educational building and the cabin, see paragraph 0) are constructed on stilts, high enough to decrease direct tsunami impact, and strong enough to resist the uplift force in case of a tsunami. The new buildings are designed to be flexible: easy to (dis)assemble.	The road and parking lot will be improved by using thin concrete slabs, reinforced with plastic fibres.

4 EVALUATION OF ALTERNATIVES

4.1 MULTI-CRITERIA ANALYSIS (MCA)

4.1.1 METHODOLOGY

4.1.1.1 Multi-Criteria Analysis (MCA)

A Multi-Criteria Analysis (MCA) serves as a decision-making tool for intercomparing proposed solutions based on an evaluation of multiple conflicting criteria. Due to the multitude of criteria involved which are not related to spatial considerations, an MCA is a suitable approach to generate a quantitative value for the 'benefit' of each solution. The MCA generally does not involve a monetary analysis. Rather, it is accompanied by a separate cost breakdown in Chapter 4.2. Together with the 'benefit', this figure generates a cost-benefit ratio per solution which allows the optimum design to be selected.

As a starting point for evaluation of alternative preliminary designs, it is assumed that all five solutions comply with the minimum requirements as stated in the Program of Requirements. However, the extent to which each solution fulfils these requirements may differ. Based on a brainstorm of design criteria and engineering considerations, 20 criteria are selected as relevant for comparison of the five alternatives. These criteria fall under the following headings: external interfaces, flexibility, construction, prestige, maintenance, safety and technical and organisational risks. Scores range from 1 to 4, with the following meaning: 4) favourable and no associated risk; 3) neutral and small risk; 2) unfavourable and moderate risk; and 1) bad and large risks.

4.1.1.2 Assignment of Criteria Weights

Weights are assigned to each of the MCA criteria prior to scoring the alternatives, as certain criteria weigh heavier on the level of benefit of a solution than others. A higher weight thus indicates prioritisation. The weight assignment is carried out using the *Pairwise Comparison* method, which has a statistical but heuristic underlying theory and has been proven to give a higher level of trustworthiness than a simple ranking method, for example. Please see Appendix D.2 for the weight factor determination table.

4.1.2 EXPLANATION OF ASSIGNED MCA SCORES

Table 4-1 serves as a reminder of the main features of each alternative. Please consult Chapter 3 for more details and visualisations. Table 4-2 provides a motivation of the assigned scores for each criterion per alternative. The full scoring table can be found in Appendix D.1

Table 4-1: Motivation Multi-Criteria Analysis scores

	A: Null option	B: Simple	C: Traditional	D: Impact proof	E: Future flexible
External interfaces	No added benefits for any stakeholder.	Dredging has environmental consequences. Little effect on surroundings.	Design fits surroundings.	Piles and robust stone and concrete structures cause visual pollution.	Involvement of local fishermen. Aesthetic design.
Flexibility	No extension carried out.	Limited added functions. No spatial re-organisation.	Possible extension of onshore structures.	Robust and inflexible design.	Possibility to move jetty. Functional and spatial flexibility.
Design simplicity	No design is made.	Re-use and upgrading require little engineering.	Design may be re-modelled after existing jetties in Coliumo Bay.	Highly complex wave defence design. Pile designs both offshore and onshore.	Complex jetty design. Long design time.
Construction	No construction carried out.	Minimal construction time and space required. Rock outcrop may pose risk for vessel manoeuvrability.	Average building time. Breakwater construction requires a lot of material and space.	Requires large amounts of material and equipment space. Rock outcrop removal.	Requires specialist knowledge and construction methods, but is light structure overall. Large crane on abutment.
Prestige	No added prestige for	Few visible outstanding elements.	Familiar but little prestige.	Serve as example of town protection system.	Futuristic and visually attractive elements.
Maintenance	Not a sustainable solution. Maintenance	Durable as concrete abutment survived previous tsunami.	Maintenance after extreme event is difficult.	Highly durable and robust structure, requiring minimal maintenance.	Relatively vulnerable jetty, but can be maintained onshore.
Safety	Transhipping highly unsafe. No extreme event protection.	Minimal added protection. No wave defence system.	Includes breakwater and protection against high waves (not tsunami). Some ductility for seismic loads.	Designed for extreme events, highly robust, ductile connections, piled structures and roads.	Safety during extreme event exceeds safety during normal operation due to floating platform.
Technical risk	No technical risks.	Minimal technical risk, but less robust than other options. Dredging is a risk.	Minimal technical risk: proven concepts	Complex connections between elements. Multiple piled structures.	Rarely applied solution, many unknowns.
Organisational risk	No organisational risks.	Minimal organisation, limited structural elements. Re-use of elements.	Ability to reproduce logistics and organisation from Coliumo jetty.	Requires large volumes of material, access roads must reach structures on piles.	Relatively small volumes of material, but applications need much preliminary study.

Table 4-2: Main elements of mooring facility for each option

	Null option (A)	Simple option (B)	Traditional option (C)	Impact proof option (D)	Future flexible option (E)
Elements	n/a	Stairs and fenders	Deck, piles, stairs and fenders	Blocks, sheet piles, stairs and fenders	Deck, piles and fenders
Dimensions	n/a	n/a	30 x 6 m	30 x 6 m	30 x 6 m
Foundation	n/a	n/a	Piles placed in soft rock	Sheet piles in soft rock (concrete fill)	Piles (one side only) in soft rock

4.1.3 Summary of MCA results

Table 4-3 gives the benefit score per alternative, resulting from the Multi-Criteria Analysis. The full MCA table is given in Appendix D.1. The alternatives with the highest benefit scores are Impact Proof and Future Flexible.

Table 4-3: Results of the MCA of alternatives A-E

	Null (A)	Simple (B)	Traditional (C)	Impact Proof (D)	Future Flexible (E)
Benefit	n/a	223	252	**285**	**285**

A short description of the most remarkable scores per alternative follows. The Null Option (A) does not fulfil the requirements elaborated in the Program of Requirements so this alternative is disregarded. The Simple Option (B) has a moderate score on most criteria, but has a good score on the construction aspects due to the simplicity of the processes involved. The prestige and possibilities for maintenance are low, however. The Traditional Option (C) has excellent scores on the simplicity of the design, but the score on maintenance after an extreme event is very low. For the Impact Proof Option (D), the scores on safety, prestige and possibilities for maintenance are good. On the other hand, the scores on the simplicity of the design are low. Finally, the Future Flexible Option (E) scores well on flexibility and external interfaces. However, the alternative has a high risk of design errors and the scores on project time and logistics are also slightly lower.

4.2 COST ESTIMATES

A breakdown is made of the costs associated with each alternative. As in the MCA, the focus is on the various mooring facilities i.e. the onshore structures and pavement involved in the alternatives are not considered at this point. The annual maintenance costs are assumed at 2% of the total investment costs for all options except (B)[5], for a lifetime period of 25 years.

4.2.1 COST DESCRIPTIONS

Table 4-4: Cost descriptions

Alternatives	Cost aspect	Description / Assumptions	Unit price (CLP)	Source
All	Labour	Semi-skilled labour. Includes pouring works, stair placement, installation placement	20,000 per day per labourer	ICCS (2011)
	Concrete	Based on prices in Manual de Precios Construcción Análisis (ONDAC)	80,000 per m^3	ONDAC (2013)
	Steel	Based on prices in International Construction Cost Survey (ICCS)	4,056,000 per m^3	ICCS (2011)
	Reinforcement	Assumed to be 1% of concrete volume for foundations	-see steel price-	ICCS (2011)
	Formwork	Based on volume of foundation blocks / concrete deck	6699 per m^2	ONDAC (2010)
	Installations	Based on a lump sum (same for all alternatives)	4,900,000	CYPE Ingenieros
	Crane for transhipment	Purchase price of crane with 1000 kg capacity.	1,380,000	Crane Authority (2009)
	Crane for construction phase	Rented crane. Assume same crane for all applications.	2,000,000 per day	CYPE Ingenieros
	Pouring works	Requires 5 men and one day.	100,000 per day	ICCS (2011)
	Placement of installations	Requires 5 men and a full working week.	100,000 per day	ICCS (2011)
	Placement of crane	Requires 2 men and a second crane.	2,040,000 per day	ICCS (2011)
B, C, D	Mooring stairs	Based on 1 reference steel staircase from Grainger Industrial Supply * 7	14,000,000	Grainger (2016)

[5] Dredging is required every 5 years for this option, carrying additional maintenance costs.

	Placement of mooring stairs	Requires 7 men and a crane.	2,140,000 per day	ICCS (2011)
C, E	Offshore breakwater	Based on 2100 m^3 of armour stones from rocks within vicinity	20,000 per m^3	CYPE Ingenieros
	Placing armour stones	Based on crane rent for a 20 day period.	200,000,000 per day	CYPE Ingenieros
	Foundation piles	Based on 12* 5m long steel piles of D=368mm.	302,100 per pile	ICCS (2011)
C, D	Steel beams	Assume similar beams to foundation piles, length 6m. No. of beams differs per alternative.	362,498 per beam	ICCS (2011)
B	Dredging	Onshore dredging * 2 for hydraulic applications. Includes material transport and vessel. 100 m^3 of dredging required.	73,500 per m^3	CYPE Ingenieros
C	Concrete deck	Assume 30x6x0.25 (L*W*H) m^3 with reinforcement and formwork	6,751,602 in total	ONDAC (2013) and ICCS (2011)
D	Sheet piles	72m of 5m long sheet piles	76,000 per m	CYPE Ingenieros
	Anchoring / strutting of sheet piles	Based on 20 anchors or struts	100,000 per anchor	CYPE Ingenieros
	Concrete block	Poured in situ, includes reinforcement and formwork, and a 6x6x0.25 m^3 concrete deck on steel beams near the shore.	94,346,380 in total	ONDAC (2013) and ICCS (2011)
	Onshore armour stones	Smaller than offshore breakwater at 1800 m^3	20,000 per m^3	CYPE Ingenieros
	Placing armour stones	Based on a shorter crane period of 10 days due to onshore application.	200,000,000 per day	CYPE Ingenieros
E	Floating jetty	From Chilean reference projects: 10,000 ton floating dock in Valparaiso by Sociedad Iberoamericana de Reparaciones Navales (SOCIBER)	220,000 per m^2	SOCIBER
	Fixing mechanism	To attach jetty to the 12 piles.	300,000 per pile	SOCIBER
	Crane for transhipment	Requires larger crane as it must be placed on the concrete abutment, not on the jetty. Assume normal crane cost * 2.	2,760,000	CYPE Ingenieros

4.2.2 SUMMARY OF COST ESTIMATE

The full cost breakdown per alternative may be found in Appendix D.3. Table 4-5 gives a summary. The least expensive option, at less than half the cost of Impact Proof, is the Simple option, (B).

Table 4-5: Costs estimate results

	Null (A)	Simple (B)	Traditional (C)	Impact Proof (D)	Future Flexible (E)
Cost (100.000 CLP)	0	81.0	150.8	279.2	216.3
Cost (1000 EUR)	0	117.4	217.2	404.7	313.5

4.3 COST-BENEFIT ANALYSIS

To make a substantiated choice between the different alternative, it is important to evaluate the total picture of cost and benefits per alternative. Therefor a cost-benefit analysis will be elaborated to select the optimum design alternative. In this analysis come the results of the MCA and the cost estimation together. Per solution is the benefit-cost ratio calculated by to following formula:

$$Ratio = \frac{Benefit\ score}{Costs}$$

The outcomes for the different alternatives are shown in Table 4-6 and depicted in Figure 4-1. It appears that option (B) Simple, is the optimal option to ensure the performance of the required tasks of the Marine Biology Station. Its costs are very low, and the benefit score is as high as other options.

Table 4-6: Result Cost-Benefit Analysis

	Alternative	Benefit score	Costs (1000s of EUR)	Benefit-Cost Ratio
A	Null-option	n/a	0	0
B	Simple	223	117.4	**1.90**
C	Traditional	252	217.2	1.16
D	Impact Proof	285	404.7	0.70
E	Future Flexible	285	313.5	0.91

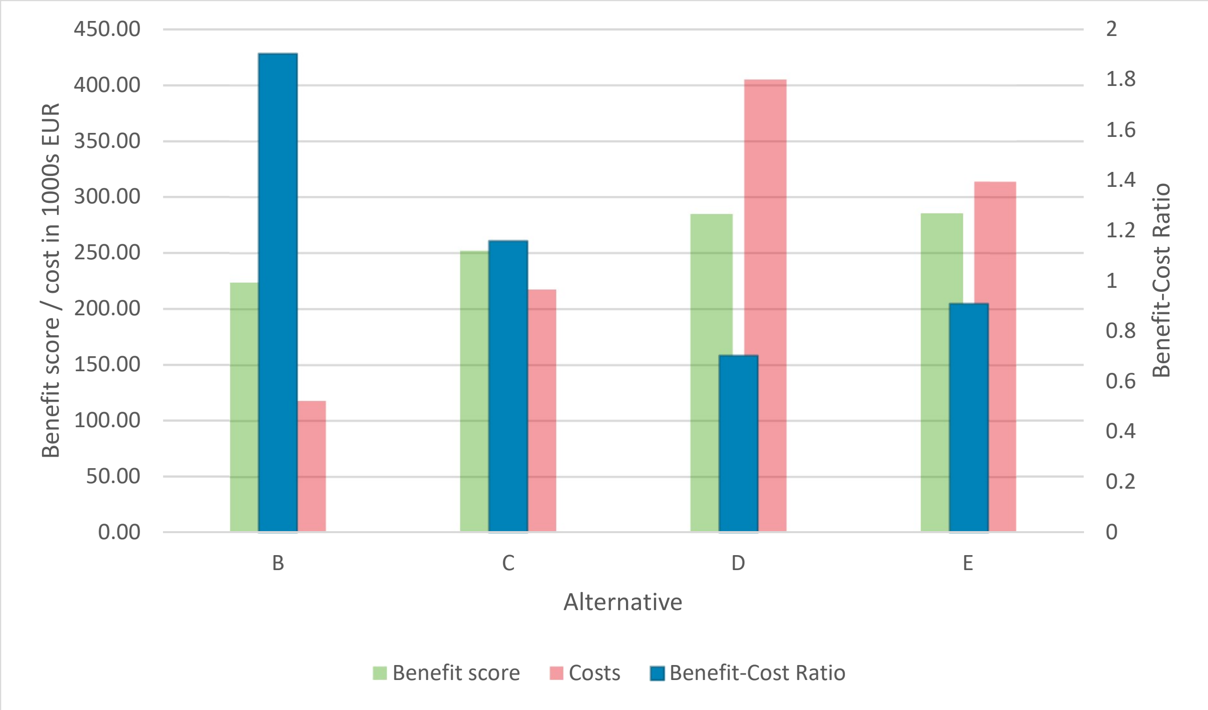


Figure 4-1: Results Cost-Benefit Analysis

4.4 Conclusions on Alternative(s) for Design

4.4.1 Hydraulic Structures: Traditional

From Figure 4-1 option (B) Simple emerges as the optimal design alternative. Whilst options (D) Impact Proof and option (E) Future Flexible have a higher benefit score, this is offset by the costs involved. Option (B) scores well despite having the lowest benefit score due to the limited costs involved in constructing this mooring facility.

However, the Simple option carries a major disadvantage; namely the required level of maintenance involved. In order for this option to work, it would be of vital importance to dredge on a regular basis in order for the vessel maintain sufficient mooring depth. Although maintenance is used as MCA criterion as well as reflected in the costs, deliberation with Chilean structural specialists leads to the conclusion that maintenance is a critical factor in design and ought to be included in the Program of Requirements –prior to multi-criteria evaluation. Therefore, it is decided to progress to the design phase with the alternative with the next-best cost-benefit ratio: The Traditional option (C).

4.4.2 Other facilities

In paragraph 3.6 we have shortly discussed the alternatives with respect to all other facilities at the harbour complex. As mentioned in chapter 3's introduction, these aspects of the redevelopment are not the focus of our project and therefore have not been taken into account in the lengthy analyses of the current chapter. However, we suggest the following improvements onshore:

a) The remains of the large building (number 11, see Figure 2-17) shall be reused and it shall be redeveloped in a similar manner to the new buildings that have already been constructed on site, as was suggested in the Traditional Alternative (see paragraph 3.3). This option is most in line with the interests expressed by the on-site operator during our location visit of 1 December 2016.
b) The foundations of the old sleeping cabin (number 12, see Figure 2-17) shall be redeveloped in order to restore the original function there, as was also suggested in the Traditional Alternative (see paragraph 3.3), in agreement with the interests expressed.
c) The other on-site foundational remains (number 13, see Figure 2-17) will not be redeveloped at this point, as no need has been expressed to do so at this point. We do suggest to refurbish and maintain this foundation to make it suitable for future expansion.
d) The on-site road and parking lot shall be paved in accordance with the Chilean guidelines for low-volume roads: granular (sub)base in combination with a protective surface treatment. This is a relatively easy, feasible and affordable option, while at the same time greatly improving the dust problems of the current road and the accessibility of the area. Thus, the value-for-price is quite good compared to more exotic options like thin concrete slabs with plastic fibres, or asphaltic options.

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