

## Structural Shape study Al Ghubaiba Ferry Terminal

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## Preface

This report describes the result of my Master's thesis project. This project is the completion of my MSc study in Building Engineering at Delft Technical University, faculty Civil Engineering and Geosciences. The subject of the Master's thesis is a study into a structural solution in reinforced concrete for the roof of the Al Ghubaiba ferry terminal. This thesis subject was obtained from Royal Haskoning. The research was performed at Corsmit in Rijswijk, subsidiary of Royal Haskoning, from September 2007 to October 2008.

I would like to thank all members of my graduation committee for their contribution to this report, and for their comments and advices during our meetings. I would also like to thank Mr A. de Santis (Architect at the Dubai office of Royal Haskoning Architects) for sharing his ideas concerning the Dubai ferry creek terminal project. Next, I would like to thank Mr. M. Hendriks (researcher at the Structural Mechanics section) for his support on the used finite element models. Furthermore I would like to thank Mr. H. Hooft who checked parts of this report on spelling and grammar errors.

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## Abstract

The city of Dubai has grown extensively in the last decades. The growth of Dubai's infrastructure has not kept pace with the growth of the city itself, which results in traffic jams and overcrowding of public transport facilities. To make the Dubai area (city and suburbs) more accessible, a ferry network is being developed to provide transportation facilities for the 26 million tourist and commuters who crowd the city annually. The first part of the ferry network that will be built consists of one line with 4 terminals. The Al Ghubaiba terminal is the largest of the four terminals; it holds a key position in the city centre and needs to become a real landmark. The free-formed design for the Al Ghubaiba terminal made by Royal Haskoning Architects combines modern architecture with efficient spatial planning. Creating a structural solution that underlines the desired architectural appearance is considered to be a challenge. The main goal of this Master's thesis is to analyze a structural solution in reinforced concrete for the roof structure of the Al Ghubaiba Ferry Terminal according to the initial design concept made by Royal Haskoning Architects.

The architect's design concept defines the roof of the Al Ghubaiba terminal as a canopy that is created by carefully lifting the existing quay. The public area hereby continues smoothly over the terminal roof, creating a elevated square. To accentuate this design concept, a slender roof structure is required which contains a transparent façade along the creek site and has a column-free floor plan.

The type of load carrying mechanism present in a thin concrete structure is highly dependent on its shape, slenderness, loading and support conditions. For a large span roof in reinforced concrete a good shape is of vital importance. A good shape in this sense has an internal flow of forces which is primary based on membrane forces and does not suffer from buckling instability. The terminal roof has a complex geometry which contains parts of beam, plate, arch and shell action. These load carrying mechanisms are analyzed to review their efficiency and to determine the relation of the geometry of a structure to its internal flow of forces. The knowledge obtained in this analysis is used to optimize the structural geometry of the terminal roof.

A simplified finite element model is made in DIANA to analyze the structural behaviour of the roof structure. An analysis of the initial roof structure reveals that the roof structure contains a large amount of bending and is critical to buckling. A viable structural solution which makes use of this initial roof geometry and is column free is for this reason problematic. Based on calculation results a roof geometry is suggested which has a more preferable structural behaviour.

Regarding structural mechanics it is known that designs with an utmost slenderness can be obtained if a geometry approaches the funicular shape of its governing loading. The property of a funicular shape is that, for a specific loading condition, only membrane forces are used to transfer the external load. A method is developed to find a roof geometry which approaches the funicular shape of its typical loading condition. This method uses the initial structural geometry and combines this with its displacement field to obtain an improved shape.

For slender structures like these, buckling behaviour is an important subject. The initial roof design lacks adequate safety against buckling. By increasing the amount of curvature the roof structure will become more stiff and stable.

To further analyze the preferred roof geometry, a more complex computational model is made in DIANA. This model contains more elaborate loading conditions, support conditions and material properties and can be used to perform unity checks. By synchronizing the cross section of the roof with the internal forces present a structural design is obtained which contains sufficient strength, stiffness and stability.

This Master's thesis provides a structural solution in reinforced concrete for the roof of the Al Ghubaiba ferry terminal. The recommend roof geometry complies with the initial design concept made by Royal Haskoning Architects. However, a higher roof geometry is required to make a column free roof structure in reinforced concrete possible.



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## Introduction

This Master's thesis project comprises a structural shape study for the Dubai Creek ferry terminals, designed by Royal Haskoning Architects. Royal Haskoning has made several free form designs for the various terminals along the creek of Dubai. In this project the focus lies on the analysis of a roof structure in reinforced concrete that complies with the initial architectural concept.

Finding a roof geometry with a favourable structural behaviour is in the project's current design phase considered more important than dimensioning on a detailed level. A full in-depth structural analysis is therefore not included in this project.

In the following introduction this graduation study will be discussed in more detail.



## 1.1 Specifications of this graduation study

At the moment when this thesis was initiated, the preliminary layout of the Dubai Creek ferry network had just been finished. The ferry network consists of one line with four terminals at key locations along the creek of Dubai. The first sketches of the different terminal designs show very elegant buildings.

The Al Ghubaiba terminal is the main building in the ferry network. It has the largest capacity and needs to become an identifiable landmark for the city. The design for the Al Ghubaiba terminal made by Royal Haskoning combines modern architecture with efficient spatial planning. Due to the scale of the building combined with the interesting desired architectural appearance, working out a structural solution of the Al Ghubaiba terminal is considered the most challenging.

Royal Haskoning has to complete the various phases in the design process in such a pace that there is no time to conduct a thorough structural shape study. Leaving out this structural shape study has resulted in a rather traditional structural design. In this structural design columns and beams are used as load carrying elements instead of using the roof itself as a load-bearing structure. The chance of creating an structure that fully supports the architectural concept and makes use of the typical shape is thereby left out. This study uses the initial design concept and tries to come up with a structural solution in reinforced concrete. This solution makes use of the structural abilities of the intended typical shape, hereby creating a terminal with a column free floor plan. This approach should lead to a structural system that endorses the architectural view.



The initial design is used as starting point of this project. Developments and new insights with regard to the initial design, formed after the initiation of this thesis project, have not been incorporated into this project.

It is mainly the design of the roof construction which gives this building its iconic aesthetics. Other aspects of the terminal like the floating pontoons and the underground connection to the subway are, however interesting, not studied further in this report.

#### **1.2 Problem definition**

Royal Haskoning Architects has developed a design concept for the Al Ghubaiba location which comprises a terminal partly built on reclaimed land. The design concept, presents the quay along the creek as a carpet-like slab, gently lifted from the ground, providing a canopy for the terminal and extending the public area to an elevated square. This graduation project studies a structural solution in reinforced concrete that complies with this design concept. See Figure 1-1. By using reinforced concrete it is thought that a structure can be made which uses the structural abilities of the intended shape. The final structural design should contain a column free floor plan and a highly transparent façade at the creek site of the terminal, thereby accentuating the idea of a lifted quay.

Creating a new design for the roof structure of the Al Ghubaiba terminal which deviates from the initial design concept is not thought to be an objective of this graduation project. The use of structural materials other than reinforced concrete is also not considered in this report.

Subjects which have to be worked out when designing a shell-like concrete structure are:

- Geometry
- Dimensioning
- Building method
- Interaction with foundation
- Detailing and finishing
- Logistics

This graduation project covers the top two of these subjects.



#### 1.3 Thesis goals

The main goal of this Master's thesis is to analyze a structural solution in reinforced concrete for the roof structure of the Al Ghubaiba Ferry Terminal according to the initial design concept made by Royal Haskoning Architects.

The first part of this report studies a suitable geometry for the roof structure. A study will be performed to find out of the intended roof geometry is suitable for a structure in reinforced concrete. When necessary, a recommendation for a modified roof geometry will be presented to obtain a favourable structural behaviour. The goal is to find a roof geometry which endorses the architectural view and makes a structure in reinforced concrete possible.

The obtained roof geometry should have adequate strength, stiffness and stability. The goal is to create practical design calculations which have sufficient accuracy to make sure that the roof structure contains enough strength, stiffness and stability.

Apart from the geometry of the roof structure itself, the way that it is obtained is found relevant. The generation of the roof geometry as well as the structural calculations will be performed using computational models. Not many applications are freely available which are fit to analyse the geometry and structural behaviour of a shell-like structure and are able to come up with a improved geometry. A goal is to derive computational methods to modify the geometry of a shell-like structure in a way that its structural behaviour improves.

#### 1.4 Framework of this report

This report starts with a view on the City of Dubai and the reasons for the initiation of the Dubai Ferry Creek project. After discussing the purpose of the ferry project the specific terms and conditions for a modern public transport building are discussed in Chapter 3. Chapter 3 also analyzes the "Lifted Quay" design concept made by Royal Haskoning Architects, and the way this is worked out. Chapter 3 results in a simplified geometry which will be used as a basis in the structural shape study. The internal force distribution in the final roof structure will contain parts of beam, plate, arch and shell-action.

These basic load carrying mechanisms are first studied separately in Chapter 4. Chapter 5 describes how by using different computational modelling techniques the geometry of the terminal roof is modified in a way that improves the structural behaviour. Chapter 6 describes the calculations performed to make sure that adequate strength, stiffness and stability is present in the designed roof structure.

The results obtained and the methods used during this study are evaluated in Chapter 7. The findings of this Master's thesis research are stated in Chapter 8 as conclusions and recommendations for further research.

Background information and specifications of the modelling stages and calculations can be found in the appendices.

There are multiple methods to describe a curve and a surface. Appendix A gives a brief introduction into the field of curve and surface geometry used for design purposes. During this study, the results of computational models are used to modify a structural geometry. Appendix B describes a comparison between structural applications performed to find out which is most suitable for this intended typical use.

The computational models used in this study are built in a specific way to make them more useful for design purposes. Appendix C shows how the computational models is built. Appendix D shows how the geometry of the design can be modified by using the results of computational models. The specifications of the calculations are described in Appendix E. The results of all the computational models used during this project can be found in Appendix F. The contact information of Jasper Janssen and the members of the graduation committee can be found in Appendix G.







## The context Dubai

Before discussing the technical starting points of this project it is useful to study the motives of the Dubai Ferry Creek project. Dubai is an extraordinary city which has grown extensively during the last decades. At the moment multiple residential, leisure and commercial project are being developed. This unique city states specific demands to its public transport system. This chapter describes why public transport with Ferries is a ideal addition to the existing public transport network.

Section 2.1 give a short review of Dubai's history and gives an overview of the developments in the near future which will have their influence on the city. Section 2.2 reviews the current state of Dubai's public transport system and compares this to other metropolises.

The new ferry project will start as one ferry line containing four terminals. The terminal locations are analyzed in Section 2.3. This section also reveals the preliminary designs made by Royal Haskoning for each of the terminals.



#### 2.1 A fast growing city

Dubai is the capital of the emirate Dubai, one of the seven emirates that constitute the United Arab Emirates (UAE). The United Arab Emirates is a Middle Eastern federation of seven states situated in Southwest Asia on the Persian Gulf, see Figure 2-1. Dubai lies directly within the Arabian Desert, a land that mainly consists of a sandy desert. Dubai covers an area of 4,114 km<sup>2</sup> and has a population of about 1,4 million. The city of Dubai is established in the early 18th century by the Al Abu Falasa clan. The town's geographical location attracted traders and merchants from all over the region. For many years now, Dubai attracts foreign traders by very low trade tax rates.

The discovery of oil, in the early 1960's, led to a massive influx of foreign workers, mainly Indians and Pakistanis, into the city. As a result, between 1968 to 1975 the city's population grew, according to some estimates, by over 300%.

The Persian Gulf War of 1990 had a huge impact on the city. Economically, Dubai banks experienced a massive withdrawal of funds due to uncertain political conditions in the region. During the course of the 1990s, however, many foreign trading communities, first from Kuwait, during the Persian Gulf War, and later from Bahrain, during the Shia unrest, moved their businesses to Dubai. Large increases in oil prices after the Persian Gulf war

encouraged Dubai to continue to focus on free trade and tourism. The success of the Jebel Ali free zone allowed the city to replicate its model to develop clusters of new free zones, including Dubai Internet City, Dubai Media City and Dubai Maritime City. The construction of Burj Al Arab, the world's tallest freestanding hotel, as well as the creation of new residential developments, were used to market Dubai for purposes of tourism. Since 2002, the city has seen an increase in private real estate investment in recreating Dubai's skyline with such projects as The Palm Islands and Burj Dubai.

At the moment less than 3% of Dubai's revenues come from petroleum and natural gas. A majority of the emirate's revenues are from the Jebel Ali free zone authority (JAFZA) and, increasingly, from tourism and other service businesses. Dubai has attracted world-wide attention through innovative real estate projects and sports events.

As shown in Figure 2-2, Dubai is situated around a creek that runs from the Persian Gulf through the city. The eastern section of the city forms the locality of Deira and is flanked by the emirate of Sharjah in the east and the town of Al Aweer in the south. The Dubai International Airport is located south of Deira. The western section forms the locality of Bur Dubai and is bordered almost entirely by the emirate of Abu Dhabi. Much of Dubai's real estate boom has been concentrated to the west of this region, on the Jumeirah coastal belt and along



Figure 2-1: The middle east, Image from geogdata.csun.edu



Sheikh Zayed Road (E11). Port Rashid, Jebel Ali, Burj Al Arab and theme based free zone clusters such as Business Bay are all located in this section.

In order to increase Dubai's tourism, multiple artificial islands are being constructed at the moment along the coast of Dubai and in the Persian Gulf. The islands will have a large number of residential, leisure and entertainment centres.

The developments that are under construction at the moment are:, see Figure 2-3

- The World, a man-made archipelago of 300 islands in the shape of a world map.
- The Palm islands, the three largest artificial islands in the world. The islands are The Palm Jumeirah, The Palm Jebel and The Palm Deira.

The Dubai Waterfront is a proposed project and should become the largest waterfront and the largest man-made development in the world. The project is a conglomeration of canals and artificial islands. It will occupy the last remaining Persian Gulf coastline of Dubai. The vision of the project is to create a world-class destination for residents, visitors and businesses in the world's fast growing city.(Dubia Waterfront, 2005)



Figure 2-3: Developments along the coast of Dubai, Image from (Royal Haskoning, 2007)



Figure 2-2: The city of Dubai, Image from travel-to-Dubai.com





#### 2.2 The public transport network

The extensive growth of the city of Dubai comes with a downside which is typical for every major city, a large amount of traffic. The total traffic network of Dubai hasn't grown in the same pace as the city itself and therefore has become massively overcrowded. To make Dubai more attainable an extended public transport network is required. The existing public transport network of Dubai only consists of a bus system that services 69 routes. When a comparison is made between Dubai and other metropolises like New York and Hong Kong, one can conclude two things:

The first difference can be found in the urban density of the cities. Where New York and Hong Kong are concentric, with one main city centre where the rest of the city is built around Dubai is much more a linear city that spreads out along the coastline. This linearity results in big distances between the city centre and the newest residential areas.

The second difference between the different cities is the density of their public transport networks. See Figure 2-4 and Figure 2-5. Both New York and Hong Kong have multiple modes of public transport, like subway and busses, which together form a dense grid trough the entire city. The public



Figure 2-4: New York subway network, Image from worldpress.com

transport networks, of New York and Hong Kong are with their great number of subways and busses much more evolved compared to Dubai.

From 2005 the need for a safer and smoother roads and an enhanced public transport network has been high on the government's agenda. Therefore the Road and Transport Authority (RTA) was formed. The RTA is responsible for planning and providing the requirements of transport, road and traffic in the Emirates of Dubai, and between Dubai and other Emirates of the UAE, neighbouring countries. (RTA, 2007) To meet the demands of its growing population the RTA is working on many public transportation projects. These new ways of transport should ease the congestion of Dubai's road network and keep the city attainable.

At the moment a subway project costing \$3.89 billion is under construction. The subway system is expected to be partially operational by 2009 and fully operational by 2012. The subway will comprise two lines: one from Rashidiya to the main city centre and another line from the airport to Jebel Ali, see Figure 2-6. The new subway will contain 70 kilometres of track and 43 stations, 33 above ground and ten underground. When these lines are finished and transport by subway proves to be a success two additional lines will be built. Seven monorails are also slated to be constructed to help feed the subway system, connecting various places such as Dubailand, Palm Jumeirah, et al, to the main track.

Multiple key points in the city of Dubai are located along the creek and therefore are accessible by water. The typical layout of the city therefore allows Public transport by water to be a valuable



Figure 2-5: Hong Kong subway network, Image from johomaps.com



addition to the total public transport network. Public transport by water is also ideal with respect to all the artificial island project along the seashore. To find out how public transport by water would fit best in the existing infrastructure with maximal possible interchange to the bus and future subway network, an urbanism plan was made by Royal Haskoning. This plan recommends public transport by water in the form of a ferry network. The initial ferry network will contain four terminals in the Dubai creek and yearly provide transportation facilities for 26 million tourists and commuters. This network can in the future be extended with additional terminals along the creek the seashore and the artificial islands.

#### 2.3 The ferry locations

As a start for this new layer in the total public transport network one ferry line with 4 terminals is designed. As shown in Figure 2-8, the terminal locations are:

- Gold Souk
- Al Ghubaiba
- Union Square
- Deira City centre



Figure 2-8: The terminal locations, Image from (Royal Haskoning, 2007)



Figure 2-6: Dubai subway network, Image from (Royal Haskoning, 2007)



Figure 2-7: The ferry transport network, Image from (Royal Haskoning, 2007)





#### 2.3.1 Gold Souk

Gold Souk is the terminal location that lies nearest to the Persian Gulf. When the artificial island projects are finished, Gold Souk can become the first transport hub of the city centre. To maintain the existing quay, shown in Figure 2-9, the terminal will be built on reclaimed land in the creek. This location is at the narrow part of the creek, so the projection into the water has to be kept to a minimum, respecting the navigation channel with a width of 90 meters. Gould Souk terminal will be connected to the nearby bus station, car park and the wider Gold Souk area by means of an existing pedestrian underpass and a new covered walkway. The Gold Souk terminal will be designed to accommodate two Creek vessels (max 160 passengers) concurrently at berth. The initial design Royal Haskoning made for the Gold Souk terminal is presented in Figure 2-10.



Figure 2-9: Terminal location Gold Souk, Image from (Royal Haskoning, 2007)



Figure 2-10: Preliminary design Gold Souk terminal, Image from (Royal Haskoning, 2007)

#### 2.3.2 Al Ghubaiba

Al Ghubaiba is the largest of the four terminals, has a key position in the centre of Dubai and needs to become a real landmark. Al Ghubaiba lies in a bend of the creek with the heritage project and the popular fish-market next door, see Figure 2-12. The station lies in the preferred location to work as hub for the ferry system. Here the creek route and the coastal line will be integrated and connected to other modes of transport such as subway and the international bus services. Next to the typical facilities that all the other terminals will have as well, Al Ghubaiba will provide space for the operations control and administration centre for the whole ferry system. The position of the terminal respects the important views over the creek from the fish-market. Al Ghubaiba will be designed to accommodate two Creek vessels and two Coastal vessels concurrently at berth. The initial design Royal Haskoning made for the Al Ghubaiba terminal is presented in Figure 2-11. Just as the Gould Souk terminal, the Al Ghubaiba terminal will be build on reclaimed land.



Figure 2-12: Al Ghubaiba, Image from (Royal Haskoning, 2007)







#### 2.3.3 **Union Square**

Union Square lies at the wide part of the creek at a urban location near to a subway station, see Figure 2-13. Union Square is a station that will be built totally in the water on a piled platform in the creek. The terminal will be linked to other modes of transport (bus and subway) by means of an elevated walkway. The Union Square terminal will be designed to accommodate two Creek vessels (max 160 passengers) concurrently at berth. The initial design Royal Haskoning made for the Union Square terminal is presented in Figure 2-14.



Figure 2-13: Terminal location Union square, Image from (Royal Haskoning, 2007)



Figure 2-14: Preliminary design Union Square terminal, Image from (Royal Haskoning, 2007)

#### 2.3.4 **Deira City Centre**

The Deira City Centre station will be a temporary floating structure located next to the temporary floating bridge. The terminal will be connected to Deira City Center and its subway station by a temporary foot bridge. After removal of the floating bridge, which blocks the ferry line from going further in the creek, the station will probably be converted to a permanent structure, possibly linked to the new permanent bridge. The Deira City Center terminal will be designed to accommodate one Creek vessel at berth. The initial design Royal Haskoning made for the Deira City Centre terminal is presented in Figure 2-16.



Figure 2-15: Terminal location Deira City Centre, Image from (Royal Haskoning, 2007)



Figure 2-16: Preliminary design Deira City Centre terminal, Image from (Royal Haskoning, 2007)

#### 2.3.5 **Terminal to work out**

At the moment of this thesis initiation, the design for the Al Ghubaiba terminal has been developed to the most detailed level. It has the largest capacity and needs to become an identifiable landmark for the city. The design for Al Ghubaiba terminal made by Royal Haskoning combines modern architecture with efficient spatial planning. Due to the scale of the building combined with the interesting desired architectural appearance, working out a structural solution of the Al Ghubaiba terminal is considered the most challenging.





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## **Terminal design**

In Chapter 2 it was stated that the new ferry line is required to ease the congestion of Dubai's road network and to keep the city attainable. The initial ferry line which consists of four terminals should yearly provide transportation facilities for 26 million tourist and commuters. This magnitude of capacity stated high demands on the design and layout of the ferry terminal. The way transport building should be designed in the way they are used nowadays is described in Section 3.1.

Besides a high capacity, a terminal building should function as a landmark for the transport company, thereby inviting passengers to make use of its transportation facilities. This landmark function is obtained by a interesting design concept created by a design team of Royal Haskoning. Section 3.4 reviews the design concept and the way a worked out design should look like to comply with this concept.

Like often in the building industry, concessions have been made during the elaboration of the initial design. Section 3.5 reviews the final design, worked out by Royal Haskoning. This worked design consists of a slender, fluent and double curved geometry as required by the design concept but has due to a rather traditional construction not the desired transparent façade. To obtain this transparency a column free structure in reinforced concrete will be analyzed. The way the initial geometry is describes is impractical for a parametric shape study. The roof geometry is redescribed in Section 3.6.



## 3.1 Ferry terminals and other public buildings

## 3.1.1 What makes a good public transport building

Designers of terminal and other public transport buildings are confronted with multiple specific challenges. Primarily a building for public transport is a place where a large number of travellers should be "handled" in an efficient way. During rush-hour, many thousands of people pass through the public transport buildings. This makes high demands on the arrangement of both the station and its surroundings. Arrivals and departures of various means of transportation should connect as well as possible.

The time people have to spend at a station is shorter when the frequency of transport increases. The facilities on a station should be adapted to this. A full-blown restaurant is often not necessary, some small cafeterias selling fast food are sufficient to provide the commuter with its snack or coffee.

Given the fact nowadays the traveller uses a modern public transport building completely differently compared to some years ago, this station should have a different architecture. There is no need for cathedral-like buildings with extensive facilities. People arrive with the intension to depart again as soon as possible. The length of their stay is limited. An aspect that requires more attention these days is the social safety and the prevention of vandalism. However, a public transport building also functions as a landmark for the transport company. With its design, colour and interior the building should invite the passengers to make use of its transportation facilities. Considered in this manner, a public transport building constitutes an important part of the "marketing mix" of elements the public transporter uses to compete with other means of transport. (Lansink, 2001)

A good public transport building has a clear and transpired interior. People who arrive at the building must directly see where they need to go without have to use all kinds of maps or follow pictograms. Examples of public transport buildings where transparency and efficiency are well combined are the Nils Ericson Bus terminal in Gotenborg, the International Port Terminal in Yokohama and the TWA Terminal at John F Kennedy International Airport in New York.

## 3.1.2 The Nils Ericson Bus-terminal (Göteborg, Sweden, 1996)

For the design of the terminal, a clear and transparent interior is very important. A existing project where these aspects can found is the Nils Ericson Bus-terminal. The project, a terminal for the central bus-station of Göteborg is the result of a design competition in 1988.

The building is very clear and bright, and makes an important contribution to the city's planning. Travellers can spend their stopover in the glazed waiting-room, which provides equable internal conditions in Göthenborg's climate (gentle for Sweden, but wet, windy and harsh compared to



Figure 3-1: The Nils Ericson Bus-terminal Image from (Miles, 1997)



Figure 3-2: The terminals interior, Image from (Miles, 1997)



most of the rest of Europe).

One of the architects' central concerns has been to give the great length of the building (in a way it could be endless) a sense of human scale and place. The basic rhythm of the building is of course given by the 6.6m bay structure, with the western trusses bearing onto massive in-situ concrete plinths that are projected outside to give a horizontal rhythm that orders the bus stances, and internally clearly demarcates the boarding gates. The eastern trusses rest on much less obtrusive supports. On this side, the length is broken up by inserting small shop units in the middle of the structural bays. The shops are grouped in banks, and between these are the three eastern entrances.

The bus-terminal resembles that of an airport, and this is not entirely a bad thing, for it keeps the pollution and unpleasantness of big engines as far away as possible from the intimate human world. The problems of most airports are confusion and formlessness. The clarity of the Nils Ericson bus station avoids this, yet it also escapes the monotony and institutionalism that curse many big public buildings. (Miles, 1997)

#### 3.1.3 International Terminal Port (Yokohama, Japan, 2003)

One of the world's largest ferry terminals is the Yokohama International Port Terminal which is situated along the Osanbashi pier. The terminal can accommodate up to four ships at a time. Just like the Dubai creek ferry terminal buildings, this project is a combination of a public square and a

public transport building. The design was made by Foreign Office Architects who won the 1995 competition with their concept of a self-supporting steel structure, built like a ship, that would integrate the flow of passengers with public gathering places into a seamless whole.

The design is extraordinarily complex, spaces and surfaces are woven together and flow continuously from one end of the 400m long building to the other. Ramps link the different levels and blur the divisions between enclosed space, the cantilevered decks, and the undulating roof promenade. The structure is built from prefabricated sections of fireresistant steel plates that are folded like origami and backed by stiffened girders to form an integral structure-skin and provide clear spans of up to 30m. Floorboards of ipe, a dense Brazilian hardwood, flow through walls of glass that are stabilized with glass fins. The consistent use of steel, wood, and glass ties the whole structure together.



Figure 3-3: The terminals interior, Image from (Webb, 1999)



Figure 3-4: International Port Terminal, Image from (Webb, 1999)





Visitors can drive into the first floor parking area or walk into the arrivals and departures hall from the top of the entry ramp. Ships dock on either side of the pier and board or disembark their passengers through walkways into the customs and immigration area.

It was the inspiration of the architects to develop the promenade as a major public amenity: a place where locals could stroll out into the harbour and look back at their city. Anticipating that cruise ship traffic would be insufficient to make full use of the complex, they designed it as an infrastructure that could be used for markets, expositions, and group activities of every size. (Webb, 1999)

#### 3.1.4 TWA Terminal at John F. Kennedy International Airport (New York, USA, 1961)

Built in 1956-1961 for what was then Idlewild (now JFK) Airport in Queens, the TWA terminal was one of famed designer Eero Saarinen's last works, he died in 1960. The building's undulating shape was meant to evoke the excitement of high speed flight. Its curvilinear forms were used inside and out and even the terminal's smallest interior details were designed to harmonize with the curving "gull winged" shell.

The roof is not one piece, but four. Each section is balanced on two legs, with one end touching the remaining sections at the roof's centre, while the opposite end projects out into space. It must be stated that the cantilevered wings are not supported by the glass canopy below. The design consists essentially of four interconnecting barrel vaults of slightly different shapes, supported on four Y-shaped columns. Together, these vaults make a vast concrete shell, 15 meters high and 95

meters long, which makes a huge umbrella over the passenger areas. The bands of skylights that separate and articulate the four vaults increase the sense of airiness and lightness. (Duncan, 2002)

To fashion such a concrete structure 50 years ago, balancing each enormous barrel-shaped vault of concrete on just two legs was a remarkable feat. Building the forms to pour the concrete must have been an accomplishment in itself.

Since its completion in 1962, the TWA Terminal at John F. Kennedy International Airport has served as an icon of both modern air travel and modern design. But its daring gull-winged construction, a reinforced concrete sculpture that tested the limits of its material and of what modernism could be. was the source of its distinction as well as downfall. The building's stand-alone, sinewy form made it difficult to adapt it to the rapidly modernizing airline industry. Larger airplanes, increased flow and automobile traffic, passenger computerized ticketing, handicapped accessibility, and security screening are just a few of the challenges that Terminal 5 (as it's officially known) could not meet without serious alteration.

In December 2005, JetBlue, which occupies the adjacent Terminal 6, began construction of an expanded terminal facility, which will utilize the



Figure 3-5: Interior, Image from (Duncan, 2002)



Figure 3-6: TWA Terminal at John F. Kennedy International Airport, Image from (Duncan, 2002)





front portion of Saarinen's Terminal 5 as an entry point. The peripheral air-side parts of Terminal 5 have been demolished to make space for a mostly new terminal, which will have 26 gates and is expected to be complete by 2008. (Schwartz, 2005)

#### 3.2 The design team

The total design stage, from the urbanism masterplan for the choice of the terminal locations to the architectural and structural detailing of the final design is performed by a designteam led by Syb van Breda, an architect from the Dubai office of Royal Haskoning Architects. Some other projects designed by this design team are, The Wheel of Dubai, a 180 meters high rotating hotel / conference centre, shown in Figure 3-7 and the 6th Crossing, a 1100 meter spanning bridge for car and subway traffic in Dubai shown in Figure 3-8.



Figure 3-7: The Wheel of Dubai, Image from (Royal Haskoning Architecten, 2008)



Figure 3-8: 6th Creek Crossing, Image from (Royal Haskoning Architecten, 2008)





#### 3.3 The terminal concept

The Dubai Creek ferry terminals are designed to have an iconic aesthetic, readily identifiable in the cityscape as an element of urban infrastructure, and easily repeatable, almost like a piece of furniture for Dubai. The terminals themselves are to fit carefully in their respective environments, ranging from a heritage site at Al Ghubaiba, to a free standing position in the middle of the Creek at Union Square. Upon approach by patrons and commuters, the terminal will have an "airportstyle" arrival and departures lounge tone or appearance. The ferry project is based on the terminal concept as shown in Figure 3-11. The ferry's and waterbuses that will use the terminal are presented in Figure 3-9 and Figure 3-10. The Berthing of the ferries is by means of a floating pontoon to accommodate tidal differences in the Dubai Creek. The pontoons are connected to the terminal building proper by means of a hinging gangway. Both floating pontoon and gangway are in the open air. Shelter against the sun is provided by an overhanging canopy. The terminal building itself is a partly build on reclaimed land. Each terminal is unique in appearance and form, reflecting the different circumstances at each location, but conceived with the similar main elements like waiting areas, floating pontoons and overhanging roof. The dynamic roof forms of the terminals are a visual clue for patrons to identify a public transport facility. (Royal Haskoning, 2007)

## 3.4 The design concept: The lifted Quay

The Al Ghubaiba ferry terminal will be the largest of the four terminals and because of its key position in the centre of Dubai needs to become a real landmark. However, because of the heritage project and the popular fish-market next door there is no space along the quay. To solve this problem a design concept, shown in Figure 3-13, was proposed which combines a public transport building with a public square. The first rough sketches show a kind of carpet-like slab that is gently lifted from the ground providing a canopy for the terminal. In this way, the roof of the terminal becomes a extended public square which can be used as a viewing point. By making holes in the terminal deck daylight is brought into the terminal and at the same time an interaction is made between the people up and under the terminal deck.

To ensure that the terminal design coincides with



Figure 3-9: Ferry, Image from (Royal Haskoning, 2007)



Figure 3-10: Water bus, Image from (Royal Haskoning, 2007)





the lifted quay concept the shape has to meet the following demands:

- The transition between the existing quay and the new terminal roof has to be smooth and needs to have the same appearance.
- When looking from the Creek side, the terminal roof has to resemble a slender continuous slab.
- The roof of the terminal must be low enough that it does no conflict with the view of the posterior buildings.
- The roof must be accessible for people and therefore be not to steep.
- The terminal roof provides its own structural system and does not require any additional supports like columns or walls.
- The façade of the terminal needs to be highly transparent.
- Skylights should be made, not only to let daylight into the terminal and creating a spectacular shadow pattern but also to create an interaction between pedestrians

#### up and underneath the terminal deck.

The combination of a building with a public square is not shown often but there are some examples. The International Port Terminal in Yokohama, earlier discussed in Section 3.1.3 is a good example of a building with a public square. Two other projects which combine a building with a public square are the Maritime Youth House and the Culture House, both standing in Copenhagen and both designed by BIG Architects.



Figure 3-12: Sketched plan-view of terminal deck, Image from (Royal Haskoning, 2007)



Figure 3-13: Sketch of Al Ghubaiba terminal by A.de Santis, Royal Haskoning Architects, 2007





### 3.4.1 Maritime Youth House (Copenhagen, Denmark, 2002)

The architectural concept where a public square is combined with a building underneath is seen in more projects. An architect firm who uses this combination often is BIG-Architects from Denmark.

An example of one of their projects where a public square and a building are combined is the Maritime Youth House in Copenhagen, Denmark.

In this project two clients had to share the facilities, a sail club and a youth house, and their requirements where conflicting. The youth house wanted outdoor space for the kids to play and the sail club needed most of the site to moor their boats. The building is a result of these contradictory demands. The entire site is laid out with a wooden deck. The deck is elevated to allow for boat storage underneath while still letting kids run an play above on the curving and winding landscape.(BIG Architects, 2007) The geometry of the surface is created by laying a cartesian grid over the entire site placing a flat surface on the grid and then push and pull the surface on some of the grid-points. This way a surface is created that is build up of a mesh with flat faces. When the structural system of this design is studied, a rather traditional system of columns and beams is found. The vertical loads are transferred by bending trough the flat wooden panels into the wooden columns. For the transfer of horizontal forces and stability all the flat faces are coupled and stabilized by the walls of the building.



Figure 3-14: Development of the surface, Image from (BIG Architects, 2007)



Figure 3-15:Maritime Youth House in Copenhagen, Denmark, Image from (BIG Architects, 2007)



Figure 3-16: A building combined with a public square, Image from (BIG Architects, 2007)



## 3.4.2 Culture House (Copenhagen, Denmark, 2007)

Another project by BIG Architects where a building and a public square are combined is the Culture house situated in the inner harbour of Copenhagen which is designed at the moment. This site calls for a building that embraces the public activities on the harbour front. The roof has a classic saddle shape that rises in the north-western and the southeastern of the 2500m2 site.

The roof creates two public spaces, one grand exhibition space under the vault roof and one fully accessible public plaza on the double curved roof. (BIG Architects, 2007)

From structural perspective the saddle shape is a proven concept. In the double curved surface a convex line and concave can be distinguished. The convex line running between the two low points works as an arch (compression) an the concave line between the two high points works as a cable (tension). The side beams transfer the forces form the surface to the support. For this shape only two supports at the low points of the surface are required. The columns under the high point are only required to prevent the saddle from turning over and can therefore be relative slender.



Figure 3-17: Classical saddle surface, Image from (BIG Architects, 2007)



Figure 3-18: Two public spaces, Image from (BIG Architects, 2007)



#### 3.5 The worked out design

With the earlier described design requirements and the lifted quay design concept Royal Haskoning has made a preliminary design for the Al Ghubaiba terminal. The first design that was presented to the Client, the RTA, is shown in Figure 3-19.

#### 3.5.1 Interior

In Figure 3-20 a floor plan is shown of the initial terminal design. One can distinguish the main entrance, waiting areas, the hinging gangways and the floating pontoon. Figure 3-21 shows that the terminal-roof span until the line of the floating pontoon.

On the artist impression shown in Figure 3-22 one can see that the architect has managed to create the airport like interior that is desired by the client. The circle pattern shown in the artist impressions combines a modern roof finish with a number of skylights. The skylights let daylight into the terminal, create a spectacular shadow pattern and create an interaction between the pedestrians up and underneath the terminal deck. For the circle pattern a design created by the Belgium mathematician Jos Leys, shown in Figure 3-23 is used. The reason this specific pattern is chosen is because of fact that the curved edges of the terminal deck match with the internal lines of the pattern as shown in Figure 3-23.

#### 3.5.2 **Roof geometry**

The geometry of the initial design is build out of an surface of revolution. This surface is generated by rotating the curve shown below called 'the meridian' around a vertical axis over an angle of  $\alpha$  = 78°. See Figure 3-24 [b]. The meridian is build out of two circular arcs, one concave arc at the down part and a convex arc in the top see Figure 3-24 [c].



Figure 3-19: Artist impression of the initial terminal design, Image from (Royal Haskoning, 2007)









By trimming the surface of revolution by a smooth curve a shape is created that connects the sides of the terminal deck smooth to the quay. The entrance of the terminal at the quay side is created by cutting a second arc out of the surface. Multiple round holes in surface let some sunlight trough into the terminal.

When this shape is compared to the design requirements the following statements can be made:

- The overall shape gives the idea that the quay runs over fluently in the deck of the terminal.
- The back of the terminal follows the shape of the quay, crucial when for an extended square on reclaimed land.
- The height of the terminal is 7 meters, high enough from the inside to create and airport-style interior and low enough to keep the respect the view over the creek from the fish market.
- The dimension of the terminal is adequate to place all the required facilities within the building and accommodate two Creek vessels and two Coastal vessels.

#### 3.5.3 Structural system

Although the terminal roof has a futuristic appearance, the structural system which was designed by Royal Haskoning is rather traditional. The main structural system, shown in is formed by steel beam spanning between support beams in the façade at the creek side and at the quayside. The support beams bear on steel columns in the façade. Between the steel beams a 200mm thick composite floor system type Comflor 100 is used. From the façade to the cantilever, the roof is made from a corrugated steel decking in order to save weight. This part of the terminal roof is not accessible as a

#### viewing point.

With the choice for a traditional structural system with steel beams and façade-columns the wish for a roof with a slender continuous appearance is met. However, the façade columns conflict with the initial design concept and make a highly transparent façade impossible.

However if for, the designed surface of revolution a more complex structural system was used, where the roof provides its own structural system and façade columns can be omitted other problems



Figure 3-21: Cross-section, Image from (Royal Haskoning, 2007)



Figure 3-22: Artist impression of the terminal interior, Image from (Royal Haskoning, 2007)



Figure 3-23: Circle pattern, Image from (Leys, 2005)



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arise.

- The height of the terminal of 7 meters is with respect to the length of about 150 meters very low. Question arise if enough membrane forces can be generated to work as a shell or arch structure, when this is not the case the roof will act as a slab.
- The amount of curvature along the free edge is very low, which will probably result in an unfavourable buckling behaviour.
- The size of the entrance seems quite large, resulting in an unequal length of supports at each side of the terminal.

By adapting the parameter configuration of the initial geometry, the problems as stated above can probably be solved. However the relation between the geometry of the initial surface of revolution and its shape-parameters are not very clear. When for instance the height of the structure needs to be increased one has to find a total new configuration

of lengths and radii. For this reason it was chosen to describe the column free part of the roof as a surface of revolution with a horizontal axis instead of a vertical one. This new basic shape has a limited amount of design parameters which can easily be adjusted and give good control over the geometry.



Figure 3-25: Structural roof plan, Image from Royal Haskoning, 2008



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## 3.6 The modified roof geometry: the orange peel

The shape that is used as a starting point for the design is the kind of orange peel shape shown in Figure 3-27 [a]. This shape can be obtained by making two slices in a torus. A torus is a surface of revolution with a circular arc as generator. The reason this shape is used is because it can be easily described by parameters which can be studied one at the time.

Figure 3-27 [b] shows a cross section of the torus with all its geometric parameters. The curvature of the surface is determined by the two torus radii  $R_{torus}$  and  $R_{generator}$ . The dimensions of the shape is determined by the combination of the angle and the orientation of the cutting planes and the orientation of the torus itself.

The global dimensions of the terminal deck are known (150 x 38 m), the geometrical parameters are therefore converted to design parameters as shown in Figure 3-27 [c]. It was chosen to use a symmetrical surface resulting in  $\alpha = \beta = 45^{\circ}$ . The height of the shape which under geometrical parameters was a combination of R<sub>torus</sub> and e<sub>v</sub> is now formulated in a single design parameter h. The difference in height between quay and creek side of the deck can be adapted by the tilt of the torus.

The influence of each individual parameter on the global flow of forces is discussed in Section 5.2.

At the line where the torus section and the existing quay meet, the quay itself will be bended upwards. The existing quay functions as a supporting edge for the terminal roof and gives the terminal its smooth carpet-like appearance which is desired by the design concept. See Figure 3-28. The entrances of the building are formed by two holes in position of the lifted quay. The holes of the entrances are not located in the area which is formed from the torus.



Figure 3-28: Transverse cross section with the lifted quay



Figure 3-29: The orange Peel and the lifted quay.



Figure 3-27: The Orange peel, perspective [a], geometrical parameters [b], design parameters [c]

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# Chapter **4**

#### Load carrying mechanisms - Beam action - Arch action - Plate action Shell action Moulding Knowledge Relation shape and Distinguish type of structural behaviour carrying mechanism Modify geometry Initial roof geometry Modified roof geometry Combination of LC mechanisms Combination of LC mechanisms Undesired structural behaviour Optimized structural behaviour

Figure 4-1: Study separate load carrying mechanisms

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## Form and force

Multiple roof geometries can be designed that comply with the design concept as stated in Section 4.3. All these geometries will have a different internal stress distribution. By analyzing the internal stress distribution of a complex roof structure a combination of internal carrying mechanisms can be distinguished.

During this chapter not the design itself will be discussed but only the pure forms of the internal carrying mechanisms present in the structure. By this approach, qualitative knowledge about the relation between the geometry of a structure and its structural behaviour is obtained. Knowledge which leads to a set of geometric modifications which can be used to solve a certain undesirable structural behaviour of the typical internal carrying mechanism present.

For the total roof structure, which is a complex geometry, the knowledge obtained in this chapter helps to distinguish which type of internal carrying mechanisms are present. The set of geometric improvements can be used to improve the structural behaviour of the total roof structure, see Figure 4-1.

. Analysis results which initiate a geometrical modification are:

- Internal stresses which exceed the maximal design stress
- Deformation which exceeds the limits for a proper detailing
- A low buckling value which indicated a lack of sufficient stability

After applying the geometric improvements a structural behaviour should be obtained where all the analysis results comply with the prescribed structural demands.

#### 4.1 Structural systems

Several structural systems can be used when designing a roof structure. Span length is unquestionably a crucial determinant in selecting a structural response for a given situation. The importance of the structural span is evident from noting that design moments for uniformly distributed loads are proportional to the square of the length of the span. Doubling the span length, for example, increases the design moments by a factor 4; quadrupling the span lengths increases design moments by a factor of 16. Member sizes, of course, depend closely on the magnitude of the design moment present. The appropriateness of a particular structural system is dependent on this as well. For this reason it is useful to review how several different structural systems provide an internal resisting moment to balance the external applied moment.

The shape of the terminal deck, which is a part of a torus, earlier described in Section 3.6, is no textbook example of a basic structural system. The shape of the terminal deck can be seen as a combination of the structural elements presented in Figure 4-2. It is expected that the way this structure provides an internal resisting moment is a combination of the load carrying mechanisms related to these basic structural elements.

Note that the basic system for all the structural elements presented in Figure 4-2 is the same. A couple is formed between compression and tension

zones whose magnitude exactly equals the applied moment. For a given moment the magnitude of the internal forces developed in the compression and tension zones depends directly on the magnitude of the moment arm present. The deeper the structure, the greater the moment arm, the lower are the tensile and compressive internal forces present.

Because it is expected that a combination of the moment carrying systems is present, each of these systems is studied individually to find out how the shape the structure influences the moment capacity. Section 4.2 starts with bending in a single direction like in a beam. In section 0 structures which transfer load by bending in two directions, like plates and beam grids are studied to find out how a typical shape of these structures can improve their moment capacity. Obviously, if structural depth were always increased in response to the increased design moments associated with longer spans, internal forces could be kept at a reasonable level. This is essentially what an arch or shell does. These structures are usually relatively deep and inherently provide a very high large internal moment arm. The forces in the resisting couple can thus be fairly small with the structure still capable of providing a very large internal resisting moment. In Section 4.4 classical mechanics is used to find relations between rise, span and thickness for stable arches. Section 4.5 studies shell structures. All the partial studies in this chapter are made on a qualitative basis, where it is tried to use as simple models as possible.



Figure 4-2: Classification of structural elements, Image from (Schodek, 2001)



#### 4.2 Beam action

#### 4.2.1 Beams in buildings

Few buildings are build that do not make use of beams. When used to form the primary structural system in a building, beam elements are most typically used in a hierarchical arrangement. Planar surface elements (e.g., decking or planks) have limited span capacities and are therefore typically supported by longer-span secondary members to form a two-level system. These members are, in turn, supported by collector-beams to form a three level system. Loads acting on the surface are first picked up by the surface members, then transferred to the secondary beams, which transfer them to the collectors or supports. See Figure 4-3.

The amount of load carried by each member thus progressively increases. This increase in loading, coupled with an increase in length, typically leads to a progressive increase in member size or depth.

The use of hierarchical arranged elements can lead to more efficient use of building material. The total weight of a multiple level arranged structure will be smaller than for a single level one. However, the use of multiple levels will have effect on the depth of the total structure.

The actual stresses developed in a beam depend on the amount and distribution of material in the cross section of the member. Basically, the larger the beam, the smaller the stresses. The way this material is organized in space, however is important. A thin rectangular member that is placed on its side cannot carry anywhere near the same amount of load as an equivalent sized member placed so that its maximum dept is in line with the applied loads.

Support conditions are particularly important. A member that has its ends restrained is much stiffer than one whose ends are free to rotate. A member

with fixed ends, for example, can carry a concentrated load at midspan twice that of a similar beam with unrestrained ends.

#### 4.2.2 Basic stress distribution

External loadings on a beam produce a set of internal forces, related stresses and deformations. For equilibrium to obtain, a set of internal forces must be developed in the structure whose net effect is to produce a rotational moment equal in magnitude but opposite in sense to the external bending moment and a vertical force equal and opposite to the external shear force. These internal resisting forces are generated through the development of internal bending and shear stresses.

At any cross section of the beam, fibres in the upper portion of the beam are shortened and those in the lower portion elongated by the action of the external bending moment. If the beam is constructed of a material which is linearly elastic, the stresses produced by bending are directly proportional to the deformations present. The stresses induced by the load are maximum at the outer fibres of the beam and decrease linearly to zero at the neutral line, see Figure 4-4. The tensile stresses, associated with the elongations in beam fibres and the compressive stresses with contractions are together typically referred to as bending stresses.



Figure 4-3: Beams in buildings, Image from (Schodek, 2001)



Figure 4-4: Basic load carrying mechanism in a beam, Image from (Schodek, 2001)



One of the most interesting aspects of beam analysis is the way bending and shearing stresses interact. In a beam, the result of the interaction between bending and shearing stresses is to produce a set of resultant tensile and compressive stresses, typically called principal stresses, that act in different directions from either the bending or shear stresses individually.

In a cantilevered beam, the stresses acting on several typical elements are illustrated in Figure 4-6. Note that for an element at the neutral axis of the beam [c], where bending stresses are zero, only shear stresses exist. These stresses can be resolved into equivalent principle tensile and compressive stresses acting at 45° angles to the neutral axis. At the extreme surfaces of the beam [a], an element carries only bending stresses, since shear stresses are zero. Thus, the principle stresses are aligned with the neutral axis. For an intermediate element subject to both shear and bending stresses [b], the principle stresses have an inclination depending on the relative magnitudes of the shear and bending stresses.

#### **4.2.3 Stability: Lateral torsional buckling** Consider a thin, deep beam illustrated in Figure 4-5.



Figure 4-5: Lateral buckling in beams, Image from (Williams, et al., 2000)

Application of a load may cause lateral buckling in the beam, and failure will occur before the full strength of the section can be utilized. Lateral buckling occurs because of the compressive forces developed in the upper region of the beam coupled with insufficient stability of the beam in the lateral direction. Beam structures should always be checked on their vulnerability for lateral torsional buckling. Prevention of lateral buckling can be provided in two primary ways, one by using transverse bracing, or two by increasing the lateral stiffness of the beam.



Figure 4-6: Principle stresses in a cantilevered beam, Image from (Schodek, 2001)




#### 4.2.4 Design of beams structures

In both (Schodek, 2001) and (Williams, et al., 2000) can be found that the approximate dimension of the cross section of a beam member in relation with the span is the following:

Like described earlier, the exact required construction height for this specific project is dependent on the specific cross section, the width between the members, the support conditions and the load distribution.

When for instance a series of adjacent beams as shown in Figure 4-7 is studied spanning a round area with diameter *I*. All the beams have a full rectangular cross section and their own weight is the dominant load case.

The loading on the beams is:

 $\begin{aligned} q_d &= \gamma_{per} \cdot \rho \cdot g \cdot b \cdot h + \gamma_{var} \cdot q_{var} \approx \gamma_{des} \cdot \rho \cdot g \cdot b \cdot h \\ &= P \cdot b \cdot h \end{aligned}$ 

Where

-	$q_d$	= Design load
-	$q_{var}$	= Variable load
-	h	= Beam height
-	$\gamma_{per}$	= Safety factor, permanent loads
-	$\gamma_{var}$	= Safety factor, variable loads
-	$\gamma_{des}$	= Design load factor
-	ρ	= Density
-	g	= Gravitational Acceleration
-	h	= Beam height
-	b	= Beam width

 $P = \text{Load factor} = \gamma_{des} \cdot \rho \cdot g$ 

The beams have pinned supports, the bending moment for the longest beam at midspan is equal to:

$$M_d = \frac{q_d \cdot l^2}{8} \approx \frac{P \cdot b \cdot h \cdot l^2}{8}$$

When a beam, made of an elastic material, is loaded by this bending moment a beam height is required of:

$$W_{rec} = \frac{h_{rec}^2 \cdot b}{6} = \frac{M_d}{\sigma_c} \approx \frac{P \cdot b \cdot h \cdot l^2}{8 \cdot \sigma_c}$$
$$h_{rec} \approx = \frac{3 \cdot P \cdot l^2}{4 \cdot \sigma_c}$$

Where:

-  $W_{rec}$  = Required section modulus -  $\sigma_c$  = Maximal allowable stress

The required height for beams with clamped supports is:

$$h_{rec} \approx \frac{P \cdot l^2}{2 \cdot \sigma_c}$$

The obtained required beam height is a direct function of the span. This means that for large spans, very high beams are required. This required beam height does not comply with the achievable slender structure. There are however multiple options which can lead to a thinner structure.

A numerical example of beams spanning a circular hole is presented below. The outcome will mainly be used as a reference in the next sections.

- Span = 20m
- Cross section [bxh] = 1000x500mm
- Load = 12,5kN/m<sup>1</sup>
- Hinged support conditions
- Linear elastic material E=27.000N/mm<sup>2</sup>

The maximal bending moment, at midspan of the central beam is equal to:

$$M_y = \frac{q_d \cdot l^2}{8} = \frac{12,5 \cdot 20^2}{8} = 625kNm$$

The deflection of the central beam is equal to:

$$U_z = \frac{5 \cdot q_d \cdot l^4}{384 \cdot E \cdot I} = \frac{5 \cdot 12, 5 \cdot 20e^{34} \cdot 12}{384 \cdot 2,7e4 \cdot 500^3 \cdot 1000} = 92,6mm$$

In Section 4.3.1 combinations from longitudinal and transverse beam systems will be analyzed further.



Figure 4-7: Adjacent beams

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## 4.2.5 Optimizing the geometry of a beam

This section considers the shaping of a beam as a way of improving the overall efficiency of a beam. Dead weight is the governing load for the terminal deck, so by removing material from the cross section the required construction height will be less.

The moment of inertia (I) and the section modulus (W) are of primary importance in beam design. A common design objective is to provide the required I or W for a beam carrying a loading with a crosssectional configuration that has the smallest possible area, and therefore weight. The basic principle for minimizing the area for a fixed moment of inertia is contained in the definition of the moment of inertia, which is  $I = y^2 dA$ . The contribution of a given element of area (dA) to the total moment of inertia of a section depends on the square of the distance of this elemental area from the neutral axis of the section. This would lead one to expect that the best way to organize material in space is to remove it as far as practically possible from the neutral axis from the section (make a deep section with most material at the extremities). It will be recalled that maximal shearing stresses always occur at the neutral axis of the beam, therefore the web of the beam requires a certain minimal thickness to prevent is from shear failure. In addition, there are lines of principle stresses which cross the beam in the middle. The principle compressive stress could cause the web to buckle locally when it is too thin.



Figure 4-8: I section on two supports



Figure 4-9: Bear (Schodek, 2001)

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When a beam is analyzed with a web thickness of ¼ the beam width and a flange thickness of 1/10 the beam height, maximal bending moment is equal to:

$$q_{d} = P \cdot A = P \cdot b \cdot h - \frac{3}{4} b \cdot \frac{8}{10} h = P \cdot \frac{2}{5} b \cdot h$$
$$M_{d} = \frac{1}{8} q_{d} \cdot l^{2} = \frac{1}{20} \cdot P \cdot b \cdot h \cdot l^{2}$$

The moment of inertia is:

$$I = \frac{1}{12}b \cdot h^3 - \frac{1}{12} \cdot \frac{3}{4}b \cdot \left(\frac{8}{10}h\right)^3 = \frac{77}{1500}b \cdot h^3$$

The required beam height becomes:

$$\sigma_c = \frac{M_d \cdot h}{2 \cdot l} = \frac{75 \cdot P \cdot l^2}{154 \cdot h}$$
$$h_{rec} = \frac{75 \cdot P \cdot l^2}{\sigma \cdot 154} < \frac{3 \cdot P \cdot l^2}{4 \cdot \sigma_c}$$

The approach of using a more effective cross sections results in a reduction of construction height of 35%. This reduction is just an example, the exact gained construction height by using a more efficient cross section can further improve if a more detailed study is performed to all the specific parameters.

When looking at the moment distribution of a beam, loaded by its own weight, it is clear that the most structural height is required around midspan. By varying the shape of the member along its axis it is possible to provide a better fit between the characteristics of a beam and the shear and moments which typically vary along the length of a beam.

Towards the supports, the bending moment and therefore the required cross section becomes smaller. By reducing the size of the cross section, the total bending moment is reduced.

#### 4.2.6 Design recommendations

The following design recommendations can be concluded from studying bending in one direction as the internal carrying mechanism.

- Large bending moments will result in heavy members, therefore it is crucial to keep the amount of bending low.
- Prevent lateral torsional buckling by using transverse bracing, or by using members





with sufficient stiffness in the lateral direction

- Make use of efficient cross sections, deep section with most material at the extremities.
- Adjust the member size with respect to bending moment present at each specific location.

By applying these design recommendations to the design at places where bending in one direction is a dominant type of carrying mechanism, the internal stresses can be kept at a low level.



#### 4.3 **Plate action**

#### 4.3.1 **Beam grid**

Consider the simple crossed beam system supported on four sides shown in Figure 4-11. As long as the beams are identical, the load will be equally divided along both beams. As the beams are not identical, a greater portion of the load will be carried by the stiffer member. The key to analyzing a grid structure of this type is to recognize that a state of deflection compatibility must exist at each point of connection in a crossed-beam system.

An interesting aspect of the grid is the twisting of the members. As a member deflects, its ends tend to rotate. This tendency to rotate causes torsion to develop in the perpendicular members. At the same time these members provide a torsional resistance to the end rotations of the long member. The long member is, in effect, stiffened by the torsional restraint offered by the perpendicular members.

Note that if the beams were simply crossed and non-rigidly attached at intersection points, the bending rotation of one member would not cause twisting in the other. The consequent loss in overall rigidity due to the loss in torsional resistance associated with the twisting action would cause greater deflections to occur in a non-rigidly



Figure 4-10: Beam grid

connected system than in a rigidly interconnected grid.

To find out how the load is divided by the beams in two directions, two beam-models are studied with the finite element program Scia Esa PT. One orthogonal beam grid with hinged and one with fixed connection between the crossing members. Both beam models span the same round area with diameter a of 20meters as used in Section 4.2. The used cross sections, material, loading and support conditions are equal to the numerical example discussed in Section 4.2. The results of the two models, are presented in Figure 4-12and Figure 4-13 (only a quarter of the grids are shown).

For the grid with hinged connections between the crossing members al the load is transferred by only shear and bending forces. The maximal bending moment at midspan is 526,2kNm, 16% lower than for a single beam. The maximal deflection at midspan is 76,6mm, 17% less than the single beam discussed in Section 4.2.

For the grid with fixed connections between the crossing members the load is transferred by shear, bending and torsional forces. The maximal bending moment at midspan is 479,3kNm, 23% lower than for a single beam. The maximal torsional forces are found near the edge of the beam grid between the orthogonal quadrants. The maximal torsional moment is 154,7kNm. The maximal deflection at midspan is 71,1mm, 23% less than the single beam discussed in Section 4.2.

#### 4.3.2 Two way spanning floor

In the previous paragraphs the load transfer trough a simple beam, and a beam grid is studied. Now we consider a plate with similar dimensions as the beam grid analyzed before. An inspection of the probable deflected shape reveals maximal





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deflection at the middle of the free edge and decrease towards the supported edges. Internal moments can be expected to vary accordingly.

The difference between a plate and a beamgrid is the way each structure provides balancing internal moments. In a grid the internal moment is provided by couples formed by compression and tension forces concentrated in the beams. In a plate the internal forces are formed by a continuous line of couples in the upper and the lower surface of the plate. The moment distribution in the beamgrid is very approximately equal to the beam spacing times the average moment per unit length at that location. The approximation described above improves as the grid mesh becomes finer and finer.

The internal forces in the plate are shown in Figure 4-14 (only quarter of a plate is shown). When the plate and the beamgrid are compared in a



Figure 4-13: Beam grid with hinged connections, My [top], Mx [middle], Uz [bottom]



quantitative manner the obtained results for the beam grid should be divide by its beam spacing, in this case 2 meter.

The maximal bending moment at midspan is 251,4 kNm, a slightly higher value than the quantitative value for the beam grid with fixed connections. The higher value has to with the fact that the moment distribution for the beam grid shows a magnitude for the total beam width, where the plate shows a maximal bending moment at one single point.

The obtained deflection for the plate structure at midspan is 28,9mm, it has to be stated that the total plate contains twice the cross sectional area of the beam grids. The deflection for the beamgrid with fixed connections dived by 2 is 37,0mm, 22% more than the plate structure.



Figure 4-12: Beam grid with fixed connections, My [top], Mx [middle], Uz [bottom]







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## 4.3.3 **Optimizing plate structures**

The governing loadcase for the terminal deck is its dead weight. By reducing the required building material the dead weight will become smaller, resulting in lower internal forces. One way of reducing the amount of building material is by changing the plate thickness with respect to the bending moments. A second way would be to use a more efficient cross section, this can be done by making holes in the plate around the neutral axis and thereby saving weight without a significant reduction of stiffness.

A two way spanning floor system which is common used in the building industry and is based on the use of efficient cross sections with low weight and high stiffness is the BubbleDeck floor system.

The BubbleDeck floor system comprises a biaxial carrying hollow slab in which plastic balls serves the purpose of eliminating concrete that has no carrying effect. Figure 4-16 shows a semi precast element of the BubbleDeck system which will be filled with concrete at the building site. The producer of the BubbleDeck claims that, compared to a solid slabs with similar strength, its system is 33% lighter, and has 10% greater stiffness. (BubbleDeck, 2004)

A final way to optimize a plate structure is based on the pattern shown in Figure 4-15. A detailed analysis of the elastic stress distribution present in a plate reveals the presence of lines of principle stress. These lines, often called isostatics, are directions along which the torsional shear stresses are zero. When analyzing again the torsional moment distribution of the beamgrid with fixed connections it become clear how the isostatics pattern of the plate can be useful during shape design. Where the principle moments and the beams have similar direction, no torsional moments exist.

Some designers have devised plates that are ribbed in a matter that intended to reflect these isostatic lines. These structures are, of course, expensive to construct, but their designers claim that these expenses are not excessive and that material savings compensate for any added construction cost.

It has to be stated that this way of optimizing plate structures is undoubtedly interesting but is curious from the viewport of classical theory in that they present the physical manifestation of a model of a stress distribution in a elastic material that is usually constructed in an inelastic material (concrete). That the ribs are indeed lines of principle stress is not argued, but a self-fulfilling prophesy may be present in placing stiffer ribs along these lines.





Figure 4-16: The BubbleDeck floor system, Image from (BubbleDeck, 2004)



## PIERE LUIGI NERVI

Piere Luigi Nervi (1891-1979) was an Italian engineer and architect and is renowned for his brilliance as a structural engineer and his novel use of reinforced concrete. Nervi thought that intuition as much as mathematics should be used in design, especially with thin shelled structures. He borrowed from both roman and Renaissance architecture to create aesthetically pleasing structures, yet applied structural aspects as ribbing and vaulting often based on nature. This was to improve the structural strength and eliminate the need for columns. He succeeded in turning engineering into an art by taking simple geometry and using sophisticated prefabrication to find direct design solutions in his buildings. One of Nervi's most famous buildings is the Palazzetto dello sport in Rome that was build for the 1960 Olympic games. It is interesting to see how the shape of foundation and the support structure of the dome are based on the horizontal pressure from the dome together with the vertical forces of the different stories. Also interesting is the prefabricated ribbed floor structure Nervi designed for the Woolfactory in Rome, where the shape of the curved ribs is based on the isostatics pattern. (Polonyi, 1989)



Figure 4-18: Woolfactory in Rome, Image from (Polonyi, 1989)



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## 4.3.4 Proposed structure: a grid stiffened slab

The alternative type of beamgrid proposed by the architect is shaped by the counter volumes of a circle pattern. This beamgrid resembles in some way the use of a levelled arranged structure and the use a more efficient cross section. This alternative beamgrid configuration, shown in the architectural render of Figure 4-19, is from aesthetic point of view favourable.

The choice for a hexagonal packing, as shown in Figure 4-20 is from structural point favourable over a square packing because it behaves more homogeneous in every direction and is has more weight reduction when equal beam widths are concerned.

The obtained structure will behave like a grid build



Figure 4-19: Terminal interior, Image from (Royal Haskoning, 2007)



Square Packing Two directional Grid Figure 4-20: Circle packing

Hexagonal Packing Three directional Grid



out of beams with a T-shaped cross section. This type structure is not as efficient as for instance the bubble deck system but is preferred because of esthetical reasons.

## 4.3.5 Design recommendations

The following design recommendations can be concluded from studying bending in multiple directions as the internal carrying mechanism.

- Try to keep the amount of torsional moments low.
- Ribbed floor where the shape of the ribs is based on the isostatics pattern is a good way to save construction material.
- The stress distribution in a full plate structure can be of value when designing a efficient beamgrid or ribbed floor.

By applying these design recommendations to the design at places where bending in multiple directions is a dominant type of carrying mechanism, the internal stresses can be kept at a low level.





## 4.4 Arch action

The relation between the structural systems analyzed above is the fact that they transfer the loading by bending and shear stresses. In this chapter structures will be analyzed which transfer load by mainly axial forces. A cable subjected to external load will obviously deform in a way dependent on the magnitude and the location of the external forces. The form acquired is often called the funicular shape of the cable (the term funicular is derived from the Latin word for "rope"). Only tension forces will be developed in the cable. Inverting the structural form obtained will yield a new structure that is exactly analogous to the cable structure except that compression rather than tension forces are developed. Since the forms of both tensile cable and compressive arch structures are derived are related to the notion of a loaded hanging rope, they are collectively referred to as funicular structures.

4.4.1 General principles of funicular shapes

Of fundamental importance in the study of arches and cables is a knowledge of what exact curve or series of straight-line segments defines the funicular shape for a given loading. A cable of constant cross section carrying only its own dead weight will naturally deform into a catenary shape, see Figure 4-23. A cable carrying a load that is uniformly distributed along the horizontal projection of the cable, will deform into a parabola. Cables carrying concentrated point load (ignoring the dead weight of the cable itself) deform into a series of straight-line segments.

The magnitude of forces developed in the arch or cable, are dependent on the relative height or dept of the funicular shape in relation with its length as well as the magnitude and location of the applied loads. The greater the rise of an arch or sag of a cable, the smaller are the internal forces developed in the structure, and vice versa. Reactive forces depend on these same parameters and have both vertical and horizontal thrusts which must be resisted by the foundation or some other element such as a compressive strut or tie rod.

#### 4.4.2 Design of arch structures

The internal forces of a parabolic, two hinged, tied arch under an equally distributed load as presented in Figure 4-22 is:

$$R_A = R_B = \frac{PL}{2}$$
,  $F_{tie} = \frac{PL^2}{8f}k$ ,  $M_c = \frac{PL^2}{8}(1-k)$ 

In which the relative tie-stiffness k can be found by:

$$k = \frac{1}{1 - v}, \qquad v = \frac{15}{8} \cdot \frac{\beta}{f^2}, \qquad \beta = \frac{EI_{arch}}{E_{tie}A_{tie}}$$

Where:

-	$R_{a,b}$	= Reaction force
-	Р	= Load
-	L	= Span
-	f	= Arch depth
-	k	= Relative stiffness
-	β	= Tie/arch stiffness ratio
-	υ	= Poission's ratio
-	Е	= Modulus of elasticity
-	А	= Cross-sectional area

For a high tie-stiffness  $E_{tie}A_{tie}$ , it can be found that the bending moment in the top of the arch,  $M_c$ , under a equally distributed load go to 0 where for a low tie-stiffness the bending moment at the top of



Figure 4-22: Parabolic, two hinged, tied arch



Figure 4-23: Cable under dead weight [L], uniform distributed load [M], point load [R], Image from (Schodek, 2001)

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the arch will go to  $M_c = \frac{PL^2}{8}$ , the value that can also be found for beams on two supports.

The tie-force  $F_{tie}$  like described above is a function of the initial arch-height f. Because the horizontal reaction-force H leads to a elongation in the tie, the supports will move outwards, deforming the arch and changing the internal forces. This increase of deformation is called the 2<sup>nd</sup> order displacement. When the 1<sup>st</sup> order displacements are to large no deformed state of equilibrium is found, the arch suffers from snap-trough buckling.

## 4.4.3 Numerical example

To compare the load carrying mechanism of arches with a bending beam, a numerical example similar as the one earlier used in Section 4.2.4 will be used. A circular hole with a diameter of 20 meters is considered. To cover this hole adjacent arches are places as presented in Figure 4-24. Both a circular and a parabolic arch are studies, both with an height of 5 meters. The arches are studied with a model build with the structural analysis program Scia ESA PT. The properties of arches are:

- Span = 20m
- Height = 5m
- Cross section [bxh] = 1000x500mm
- Load = 12,5kN/m<sup>1</sup> ground surface
- Hinged support conditions
- Linear elastic material E=27.000N/mm<sup>2</sup>

A summary of the internal forces of the beam earlier discussed in Section 4.2.4 and the two arches is presented in Table 4-1.

Value	Unit	Straight beam	Circular arch	Parabolic arch
N <sub>max</sub>	[kN]	0.0	172.4	176.6
M <sub>y;max</sub>	[kNm]	625.0	28.1	1.1
U <sub>z;max</sub>	[mm]	92.7	0.95	0.22
$\sigma_{vm;max}$	[N/mm <sup>2</sup> ]	15	0.99	0.35

Table 4-1: Internal forces for beam and arches





Figure 4-24: Adjacent arches

The difference between the obtained internal forces for the beam and the two arches are significant. Where the beam transfers all the loading, the arches transfer the load primarily by axial forces. The Von Mises stresses and the deflection U<sub>7</sub> reveal that the transfer of load trough axial forces is much more efficient than trough bending. Comparing the circular and the parabolic arch reveals the importance of a good shape. The circular arch may transfer most of the load trough axial forces, however the Von Mises stresses are still about three times higher than for the parabolic arch. The parabolic arch has a shape which is equal to the funicular line of the load which acts on it and therefore theoretically does not transfer any loading trough bending.

## 4.4.4 Stability: snap trough buckling

For buckling analysis, it is useful to distinguish two basic types of arches: high arches and flat arches. High arches are those of which the centreline of the arch may be considered incompressible, and flat arches are those of which its shortening is important. Flat arches may be assumed to fail in a symmetric mode, whose basic characteristic is the shortening of the centreline of the arch. This contrasts the behaviour of high arches, for which shortening is negligible, making asymmetry of bending the paramount feature. Analysis of high arches, on the other hand, must include the curvature term  $\frac{w}{R^{2y}}$  which is negligible for flat arches (since R is large).

First a low arch which is vulnerable for snap-trough instability will be considered.

The principle of snap-trough buckling can be well explained by a two-bar truss, also known as the von Mises truss as shown in Figure 4-25a. The two bars are elastic, characterized by axial stiffness  $\frac{EA \cos q}{L}$ , and the Euler load of each bar is assumed to be so large that the bars never buckle.

This system can be analyzed on a equilibrium and a energetic approach. A way to study the von Mises truss on a bases of equilibrium, used by Timoshenko and Gere (1961), is to apply equilibrium conditions to a primary system in which one hinge is allowed to slide horizontally and then restore compatibility by introducing the statically indeterminate horizontal thrust in the arch.





The energetic approach studies the potential energy within the system. The potential energy can be found by subtracting the virtual work of the load P from the shortening energy in the elastic bars.

Figure 4-25b shows the equilibrium path of the system. When the von Mises truss in loaded in a load-controlled manner first the stable equilibrium path 0-1 is followed, until the critical state point 1 is reached. When P is further increased, the system becomes unstable and snaps trough from point 1 to point 3, as indicated by the dashed line in Figure 4-25b. When the load is further increased, the stable path 3-4 is followed.

Just like the von Mises truss, the stability of flat arches can also be studied by a equilibrium or a energetic approach. The potential energy in a two hinged flat arch is the sum of the bending, shortening and virtual work. This can be expressed by:(Bazant, et al., 1991)

$$\Pi = \int_0^l \frac{1}{2} EI(\Delta k)^2 dx + \frac{1}{2} EA\left(\frac{\Delta l}{l}\right)^2 l - \int_0^l p(z_0 - z) dx$$

Where:

- Π = Potential energy
- EI = Bending stiffness
- $\Delta k$  = Rotation
- EA = Axial stiffness
- L = Span /2
- p = Load
- z = Truss height
- f = Arch depth

When the snap-trough behaviour of flat arches is studied on bases of equilibrium of the deformed shape it is assumed that for a sinusoidal arch, as presented in Figure 4-26, the deformed shape can also be expressed with a sine-curve.

Its load p, the initial shape  $w_0$  and final shape w can

be expressed by:

$$p = p_0 \sin \frac{\pi x}{l}, \quad w_0 = \frac{f}{l} \sin \frac{\pi x}{l}, \quad w = q_0 \sin \frac{\pi x}{l}$$

Figure 4-26: Parabolic pinned arch

These and the horizontal trust H are connected by the usual equation for a beam with end load H and lateral load p,

$$EI \cdot \frac{d^4(w - w_0)}{dx^4} + h \cdot \frac{d^2w}{dx^2} = -p$$

The shortening of the arch is given by:

$$\frac{Hl}{EA} = \frac{1}{2} \int_0^l \left[ \left( \frac{dw_0}{dx} \right)^2 - \left( \frac{dw}{dx} \right)^2 \right] dx$$

From which:

$$H = \frac{\pi^2 E A}{4} f^2$$

Substituting for w,  $w_0$ , p and H, and equating coefficients of corresponding terms, the following equations for the deformed shape  $B_1$  is obtained:

$$B_1^3 - B_1(\lambda_1^2 - 1) = \lambda_1 - R$$

Where,

$$B_1 = \frac{h_{cr} \cdot \sqrt{3}}{d}, \qquad \lambda_1 = \frac{h \cdot \sqrt{3}}{d}, \qquad R = \frac{\sqrt{432} \cdot p}{\pi^4 \cdot E \cdot d^4}$$
$$h = \frac{f}{L}; \quad d = \frac{t}{L}$$

In this equation the slenderness  $\lambda_1$  is proportional



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to the initial rise f and the arch thickness t, the loadparameter R to the load and the deformed shape B<sub>1</sub> to the central rise when loaded. Figure 4-27 shows this relation graphically. The typical value for the load parameter R is found by substituting the bending stiffness of a cross section with the moment of inertia of a rectangular cross section.

At the point M where  $q=q_0$ , the arch is about to snap through, the dimensionless load parameter R at this moment is equal to: (see Figure 4-28)

$$R_{cr} = \lambda + \sqrt{\frac{4(\lambda^2 - 1)^3}{27}}$$

snap-trough buckling, Besides which is the dominant buckling mode for low arches, asymmetrical buckling can be governing as well.

#### 4.4.5 Stability: asymmetrical buckling



Figure 4-27:  $\lambda_1$ , R and B<sub>1</sub>



Vulnerability for asymmetrical buckling is studied for a circular arch loaded perpendicular to its surface, in (Vandepitte, 1981) can be found that the critical buckling load is:

$$p_{cr} = \frac{EI}{r^3} \left( \frac{\pi^2}{\beta^2} - 1 \right)$$

Where  $\beta$  is the half midpoint angle and r is the radius. See Figure 4-29.



Figure 4-29: Circular arch

 $\beta$  can be expressed as  $\beta \cong \frac{4f}{l} = 4h$  and r as  $r \simeq \frac{l^2}{8f} = \frac{l}{8h}$ . When the arch height to span ratio h and the arch thickness to span ratio d are substituted one gets:

$$p_{cr} = \frac{128Eh^3d^3}{3} \left(\frac{\pi^2}{16h^2} - 1\right) \cong \frac{8\pi^2 Ehd^3}{3}$$

The critical buckling-load for asymmetrical buckling can also be determined for the sinusoidal arch studied before. In the analytical method used for symmetrical buckling it is assumed that the line of deflection is a halve sine-curve. By using additional sine curves with smaller periods, Ashwell (1962) proves that arches can act in three different ways. See Table 4-2 and Figure 4-30.

All the obtained values are valid for a sinusoidal distributed load, the corresponding critical values for a uniformly distributed load can be obtained by multiplying by the first term of the Fourier series,  $\pi$  / 4. The obtained solution is not exact but has an error of less than 0,5%. (Bazant, et al., 1991)





The achievable  $\lambda$ -values will in practice most of the time be larger than  $\sqrt{5,5}$ . This means that asymmetrical buckling will be the governing buckling mode.

For the terminal-roof we are not looking for a critical load, but for a required thickness and rise that are able to carry a certain variable design-loading.

To determine the required thickness a spreadsheet is made which first calculates the critical loading for a certain height and thickness ratio. When the dead weight for the used arch thickness is subtracted from the critical load, one finds the critical variable load. When changing the arch-thickness one can find for each height ratio, a critical variable load that is equal to the variable design load. See Figure 4-31.



Figure 4-31: Find the value d for a fixed variable loading

Figure 4-33 is a result of the above described method. The used input is:

- E-modulus =  $10000 \text{ N/mm}^2$
- Span = 150m
- Density deck = 25kN/m<sup>3</sup>,  $\gamma_m$ = 1,3
- Variable load =  $5kN/m^2$ ,  $\gamma_m = 1,5$

The desire was to make a spreadsheet which is totally independent of arch dimensions. However, the span is required to determine the weight of the structure, the thickness ratio D has to multiplied by the arch length, and a material density to get a load that can be compared with the critical buckling load.

Figure 4-33 shows that for lower arches a larger thickness is required to resist the variable design load. It can be found that for arches with an h-ratio of larger than 20,4, no solution can be found. This is because the weight of the deck increases faster than the gained loading capacity for the thicker arch. For arches with a h-ratio of 15, an thickness of span / 38 is required and for arches with a h-ratio of 20 this value becomes span / 29.

It has to be stated that a full slab is taken into



igure 4-30: Snap-trough a	nd Asymmetrical buckling
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	<u>Slenderness</u>	Critical load parameter	<u>Explanation</u>
	$\lambda \leq 1$	-/-	The arch is so flat that it will not buckle but will bend continuously, similar to a straight beam.
	$1 < \lambda \leq \sqrt{5,5}$	$R_{cr} = \lambda + \sqrt{\frac{4}{27}(\lambda^2 - 1)^3}$	Single sine buckling mode is governing, the arch will suffer from symmetrical snap-trough buckling.
	$\lambda \geq \sqrt{5,5}$	$R_{cr} = \lambda_1 + 3\sqrt{\left({\lambda_1}^2 - 4\right)}$	Double sine buckling mode is governing, the arch will suffer from asymmetrical buckling.
Table 4-2: Critical load par	ameters		

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account, solutions that can save weight, like the use of more economic cross sections are not taken into account. Tie stiffness is also not considered, the studied arches react therefore too stiff. A way to take the deformation of the tie into account is by using a fictive axial arch stiffness. The idea behind this method is a change of length  $\Delta L$  of either the arch (compression) or the tie (tension) have the same effect on the deformation and therefore the buckling instability of the arch. For low arches the normal force in the tie and the arch will be in the same order. By using a fictive arch stiffness, the elongation of the tie is added to the shortening of the arch. See Figure 4-32. This assumption is only valid for small deformation and for low arches, where the difference in length between the arch and tie is small. The fictive arch stiffness can be obtained by adding the reciprocal values of the axial stiffness of both the arch and the tie. This method gives a good indication of the importance of tie stiffness.

The same principle can be used to study arch buckling for an arch that has been shortened due to temperature difference. The change of length due to a drop in temperature can be rewritten to a fictive axial arch stiffness which shortens the same amount of length.

On the basis of Figure 4-33 it is possible to find a stable arch that requires the least amount of material. Because both the length and the thickness of a stable arch are known it possible to determine the required amount of material. However, the difference in length between low-rise arches, compared to the required thickness differs not enough to come to an optimum. The arch that

requires the least amount of material is much higher and thinner than the ones studied above. The choice for the required arch height should therefore not be made on basis of least material.

#### 4.4.6 **Design recommendations**

The following design recommendations can be concluded from studying arch action as the internal carrying mechanism.

- The stiffness of the supports is of crucial importance for arches.
- Base the shape of the arch on the funicular line of the governing load present.
- By increasing the height of the arch the internal forces will decrease.
- For high arches, the asymmetrical buckling mode can become governing.
- For low arches, the symmetrical snaptrough buckling mode can become governing.

By applying these design recommendations to the design at places where arch action is a dominant type of carrying mechanism, the internal stresses can be kept at a low level.



Figure 4-32: Model a tie by using a fictive E-modulus





#### 4.5 Shell action

#### 4.5.1 Structural mechanics of shell structures

Classical domes have a thickness-to-radius ratio of 1:50, eggshells can have a thickness-to-radius ratio of 1:100, modern concrete shell domes can be built to a astounding ratio of 1:800. Constructed with small quantities of simple, inexpensive, low-tech concrete and wire mesh, these structures are safe as well as beautiful. With a faith in geometry that the ancient Pythagoreans would have appreciated, practitioners have come to the counterintuitive realization that the strength of shells results from their shape, not their mass. In a structural sense they embody the famous architectural dictum "Less is more. (Robbin, 1996)

A shell is a thin, rigid, three-dimensional structural form taken by the enclosure of a volume bounded by a curved surface. (Schodek, 2001) A shell surface may assume virtually any shape. Common forms include rotational surfaces generated by the



Figure 4-35: Stress resultants and load components on a shell element, Image from (Blaauwendraad, 2003)

rotation about an axis; translational surfaces generated by sliding one plane curve over another plane curve; ruled surfaces generated by sliding two ends of a line segment on two individual plane curves; and a wide variety of complex surfaces formed by combinations of rotational, translational and ruled surfaces. The shape used however, however, need not be restricted to those easily described in mathematical terms. Free-form shapes may prove a viable solution to many structural problems. Construction considerations, however, may limit the range of form options.

Shell structures have a few unique properties, which makes them interesting for designers and structural engineers. Shells can display elegance and lightness if designed correctly. With a minimum of material, large spans can be made. A shell can be recognized by its small thickness to span ratio. What makes this possible is the principle of membrane action, which is unique for shell structures. The basic assumption of membrane theory is that in a distributed loaded thin shell only pure membrane stress fields are developed. In this stress field, only normal and in-plane shear stresses are developed, which are uniformly distributed over the cross section. Bending stresses are negligible small compared to the in-plane stresses. Due to the initial curvature a shell can resist in-plane forces as well as out-of-plane loads by membrane action. (Blaauwendraad, 2003)

A good way to envision the behaviour of any shell surface under the action of load is to think it as

analogous to a membrane, a surface element so (a) (c) (b) (d) (e)

Figure 4-34:Support conditions and membrane disturbances, Image from (Blaauwendraad, 2003)







thin that only tension forces can be developed. A soap bubble and a thin sheet of rubber are examples of membranes. A membrane carrying a load normal to its surface deforms into a threedimensional curve and carries the load by in-plane tension forces that are developed in the surface of the membrane. The basic load-carrying action of a rigid shell of a similar geometry is analogous to that produced in an inverted membrane. Of primary importance is the existence of two sets of internal forces in the surface of a membrane that act in perpendicular directions. Also is the existence of a type of tangential shearing stress which is developed within the membrane surface (which is associated with the twist normally present in the surface), which also helps carry the applied load. (Schodek, 2001)

Three dimensional forms may also be made of assemblies of short, rigid bars. These structures are not, strictly speaking, shell structures, since they are not surface elements. Still their structural behaviour can be conceptualized as being quite similar to continuous surface shells which the stresses normally present in a continuous surface are concentrated into individual members.

However, in some cases membrane theory does not satisfy equilibrium and/or the displacement requirements anymore and bending theory is needed. Disturbance of membrane behaviour occurs when:

- boundary conditions and deformation constraints are not compatible with the requirements of a pure membrane stress field (Figure 4-34 b&c)
- the shell is loaded by a concentrated load (Figure 4-34 d)
- a change in shell geometry occurs (Figure 4-34 e)

To resist the forces that disturb the membrane behaviour of the shell, additional structural elements are needed. In a lot of shell structures ribs and/or edge beams are added, resulting in a structure where membrane action and bending behaviour is combined to resist load. True shells where the loads are resisted by membrane behaviour only are not seen very often. So if the definition of a shell depends on whether loads are transferred through membrane action or not only, probably a lot of structures which are considered to be shells are in fact not.

Structures which transfer all their load trough membrane action can be much thinner than structures which transfer their load trough bending. Keeping this in mind, shell structures with a high ratio of membrane action can be called more efficient. In the next chapter a few analytical full efficient shells are discussed for different specific types of loading.

## 4.5.2 Analytical solution for well known shell structures Hyperbolic paraboloid shells

The behaviour of shells having a rules surface may be envisioned by looking at the nature of the curvatures formed by the straight-line generators. If the edge conditions are of the type which can offer restraint (foundations or very stiff edge beam), an arch-like action will exist in regions of convex curvatures and a cable-like action in regions of concave curvature.

Membrane forces are expressed on a unit width basis for an arch and cable-like actions noted, 45° to the edges. Under a uniform loading, the forces at the top of any arch or cable-strip is:

**″**⊔Delft

Jasper Janssen



ROYAL HASKONING

$$C_x = T_x = \frac{w_s L_x^2}{8d_x}$$

Where  $L_x$  is the span of the strip,  $d_x$  is its instantaneous height, and w<sub>s</sub> is the load carried by the cable or arch strip. Since arch and cable strips cross one another, the normal distributed load w present at a point is half-shared between the crossing strips. Expressing  $d_x$  in terms of  $L_x$  and  $L_x$  in terms of sides "a" and "b" for a hyperbolic paraboloid surface of maximum height h, the forces become:

$$C = \frac{-wab}{2h}$$
 and  $T = \frac{wab}{2h}$ 

When the envisioned strips terminate at the edges of the plate, they exert forces vertically and horizontally. An arch strip meets a cable strip a 90° angle, 45° to the shell edge. Vertical components balance each other. The component of the horizontal trusts perpendicular to the shell edge also balance one another. There is, however, a resultant edge shear force directed along the edge of the plate. Edge forces are approximately given by:

wab



Figure 4-38: Force equilibrium in a spherical shell, Image from (Schodek, 2001)



Figure 4-37: Meridional and hoop forces in a spherical shell, Image from (Schodek, 2001)

These edge shears accumulate along the edge of the shell against restraint points into large total edge forces:

$$F_{total} = \frac{wa^2b}{2h}$$

Depending on the location of the free and constrained points, these total edge forces may be in either tension or compression. Large edge beams may be required to carry these edge forces.

#### Spherical shells

The internal forces and stresses in axi-symmetrical shells uniformly loaded can be found quite easily through application of the basic equations of equilibrium. Consider the dome segment illustrated in Figure 4-38. Assume that the loading is a uniform gravity load distributed on the surface of the shell. If the total load of all such loads acting downward is denoted as W and the meridional in-plane internal force per unit length present in the shell surface as  $N_{\varphi}$ , a consideration of equilibrium yield the following:

$$N_{\phi} = \frac{W}{2\pi R \sin^2 \phi}$$

Where w is the load per unit area of shell surface acting downward.

The hoop forces that act in the circumferential or latitudinal direction are typically denoted as  $N_{\boldsymbol{\theta}}$  and also expressed in terms of a force per unit length, and can be found by considering equilibrium in the transverse direction. From membrane analysis it can be found that the in-plane forces that act perpendicularly to one another are related by the general expression:

 $\left. \frac{1}{2} \right|_{\mathsf{Chapter 4: Form and force}}$ 



$$p_{r} = \frac{N_{\phi}}{r_{1}} + \frac{N_{\phi}}{r_{2}} \text{ or } N_{\phi}$$
  
=  $r_{2} (w \cos \phi) - {\binom{r_{2}}{r_{1}}} N_{\phi}$ 

In a sphere,  $r_1 = r_2 = R$  and  $p_r = w \cos \phi$ , substituting this we find:

$$N_{\theta} = Rw\left(-\frac{1}{1+\cos\phi}+\cos\phi\right)$$

The distribution of meridional and hoop forces can be found by simply plotting the equations for the two forces. Figure As is evident, the meridional forces are always in compression while the hoop forces undergo a transition at an angle of 51°5' as measured from the perpendicular. Shells cut off above this only develop compression stresses in their surfaces while deeper shells can develop tension stress in the hoop direction.

The above presented solution is worked out according to the membrane theory. In this theory a state of equilibrium is found by using only axial forces. It has to be stated that the spherical shell described above will deform a bit due to the uniform distributed loading. This deformation will cause bending moments. In (Flugge, 1962) the analytical solution is presented for a spherical shell under a equally distributed loading according to the bending theory. The analytical solution for this elementary shape which incorporates multiple Bessel functions is mathematical so complex that it will not be discussed further. It can only be stated that for more complex shape where not only axial forces but also bending becomes important analytical solutions are almost unattainable and FE-

programs are required to obtain accurate results.

#### 4.5.3 Numerical example

To compare shell action with other type of load carrying mechanisms numerical example is used. Assume the round area with 20 meter diameter as used earlier. To cover this hole a shell with a height of 5,0 meters is placed over it. See Figure 4-40. The shell structure is studied with the structural analysis program Scia ESA PT. The shell properties are:

- Span = 20m
- Height = 5m
- Thickness = 500mm
- = 12,5kN/m<sup>1</sup> ground surface Load
- Hinged support conditions
- Linear elastic material E=27.000N/mm<sup>2</sup>



Figure 4-40: Shell structure [partly shown for visibility]

A summary of obtained the internal forces is presented in Figure 4-39.

The difference between the obtained internal forces for the plate and the shell structure is significant. Where the plate transfers all the loading by bending, the shell transfer the load primarily by axial forces. The Von Mises stresses ( $\sigma_{vm;max}$  =  $0,22N/mm^2$  for the shell) and the deflection ( $U_{z;max}$ = 0,12mm for the shell) reveal that the transfer of







load trough axial forces is much more efficient than trough bending. Comparing the shell with the arches, earlier discussed in Section 4.4.3, reveals that the global shape for a shell is not as critical as for an arch. If the shape of an arch deviates from the funicular line of its loading bending moments, which come with high stresses, are required to create equilibrium. For a shell the distribution of axial hoop forces creates equilibrium, even if the global shape of a shell deviates from the funicular shape of its loading.

The reason for the difference in internal forces for the numerical example as stated above and the analytical solution for circular shells presented in Section 4.5.2 has to do with the difference in support conditions. The analytical solution for circular shells presented in Section 4.5.2 assumes a ideal membrane support as shown in Figure 4-34. For the numerical example pinned supports are used.

#### 4.5.4 In-extensional deformation

The extreme slenderness of a shell structure incorporates big stiffness differences. There is the high stiffness due to membrane and shear on one hand and a low bending stiffness on the other hand allowing in-extensional deformation. An inextensional deformation is a deformation in which only bending occurs while membrane extension and contraction do not occur. Shells that have zero Gaussian curvature over large areas or unstiffened free edges are susceptible to in-extensional deformation. To prevent buckling and large deformation, a shell needs to be designed such that in-extensional deformation cannot occur. To study which in-extensional deformations can be possible for a designed geometry one can use a scaled down physical model or the smallest natural frequencies and associated normal modes presented by a FEprogram.

#### 4.5.5 Stability of shells, analytical solution

The buckling behaviour of shells is described by an eight order differential equation

$$\begin{split} & \frac{Et^3}{12(1-v^2)} \nabla^2 \nabla^2 \nabla^2 \nabla^2 \cdot u_z + E \cdot t \cdot \Gamma^2 \cdot u_z \\ & = \nabla^2 \nabla^2 p_z - n_{xx} \cdot u_{z,xx} - 2 \cdot n_{xy} \cdot u_{z,xy} - n_{yy} \cdot u_{z,yy} \end{split}$$

Where:

- $u_z$  is the displacement perpendicular to the shell surface
- $p_z$  is the loading perpendicular to the surface

$$\nabla^{2} = \frac{\partial^{2}()}{\partial x^{2}} + \frac{\partial^{2}()}{\partial y^{2}}$$
$$\Gamma^{2} = k_{x} \cdot \frac{\partial^{2}()}{\partial y^{2}} - 2 \cdot k_{xy} \frac{\partial^{2}()}{\partial x \cdot \partial y} + k_{y} \cdot \frac{\partial^{2}()}{\partial x^{2}}$$

The x and y direction are often not linear but are plotted on the surface of the shell. The differential equation can be solved analytically for elementary elementary shell shapes and loading. (Blaauwendraad, 2003)

Shells can, similar to arches, see Section 4.5.4., become instable and vulnerable for snap-trough buckling. The approximate critical buckling-load data for a shallow spherical dome is presented by Ashwell in (Ashwell, 1962). The used parameters are:

Load parameter 
$$R = \frac{p}{16Ed^4}(1-v^2)$$
  
Geometrical parameter  $\lambda^2 = \frac{a^2}{tr}\sqrt{12(1-v^2)}$   
 $\approx \frac{4h}{d}\sqrt{3(1-v^2)}$   
t  
t  
h=f/L  
d=t/L

Figure 4-41: Spherical shell

For  $\lambda \leq 2,08$  the dome is so flat that it will not buckle but deforms continuously.

For  $\lambda > 2,08$  the dome is vulnerable for snaptrough buckling for a  $\lambda$  of up to about 6, above  $\lambda = 6$ the dome will buckle probably of the finite disturbance type. See for approximate data Figure 4-42.







1962)

It has to be stated that for concrete shell structures, the above described analytical solutions are practically unattainable because of non linear material behaviour and imperfect shapes.

#### 4.5.6 **Imperfection sensitivity**

It was already shown that the initial geometry of a shell structure is crucial to obtain a desirable. A slight variation of the geometry may drastically change the structural response. This delicate characteristic can be further underlined by two classical examples. Gould pointed out in (Gould, 1985) a complete sign change of the compressive hoop forces of a hyperbolic cooling tower under dead load when a small geometrical imperfection in form of a local axi-symmetric bulge with a maximum deviation of 50 percent of the wall thickness with respect to the perfect geometry occurs. This observation is verified by a linear elastic finite element analysis in Figure 9 where this small, hardly visible deviation leads to tension in hoop direction. Thus it is mandatory to include this possibility in the layout of the steel reinforcement of the shell. Buckling of thin cylindrical shells under uniform axial compression is а "classical" phenomenon intensively investigated bv experiments in the past; see the experimental results of a thin Mylar cylinder tested by M. **Esslinger in Figure 4-44** 

A phenomena that can be recognized from the load displacement diagram is that the theoretical buckling load of the perfect shell is by far not reached due to small geometrical imperfections. Imperfections include dents, residual stresses, temperature stresses, in-homogeneities, creep, shrinkage, eccentricity of loading and first order deformations. Not only compressed cylinders but also bent cylinders, and radially compressed domes are very sensitive for imperfections. Hyppars are not sensitive to imperfections.

To incorporate this phenomena into shell design often the following procedure is used. First, the critical loading is computed by using formulas or a finite element program. Then this loading is reduced by a factor that accounts for imperfection sensitivity, often called the "knock down factor". The result needs to be larger than the design loading.

Finite element programs can compute critical load factors and the associated buckling modes. The real critical load is represented by the smallest load factor because a shell will buckle at the first opportunity it gets. If the second smallest buckling



Figure 4-43: Geometrical deviation of initial geometry for cooling tower, Image from (Ramm, et al., 2005)



. . . . .

load is very close (say within 2%) to the smallest buckling load it is expected that the structure is highly sensitive to imperfections. This is because the interaction of buckling modes gives a strong softening response after the critical state. Hoogenboom states in (Hoogenboom, 2006) that shells that are sensitive to imperfections the maximum load factor might be as small as 1/6 of the critical load factor.

## 4.5.7 Stability of thin concrete shells based on the extensive practical experience of Heinz Isler

When discussing the stability of thin concrete shells, one has to mention one name: Heinz Isler. Heinz Isler (born 1926) is a Swiss engineer whose greatest contribution to the history of structural design were in the area of concrete shell constructions. After experimenting with pneumatic forms to create shell shapes, he discovered what he came to believe was the best method for making these forms, the hanging-membrane reversed. During his career, Isler has designed numerous thin concrete shell structure, including the Deitingen Service Station (Figure 4-45) and the Heimberg Indoor Tennis Centre (Figure 4-46). (Princton University Art Museum, 2003)

For his extensive practical experience, Isler deduced some rules as to how the risk of instability in this concrete shells structures can be reduced.

For its primary static tasks a shell must have:

- A good shape
- Sufficient curvature at every point
- Equilibrium at the supports (ties etc.)

- Correct edges
- Reasonable deformation
- Good protection against corrosion

If these conditions are fulfilled in all load cases, for both short and more extensive periods a thin shell structure must also be safe against all sorts of instabilities:

- Snap-trough buckling
- Local bucking in the surface
- Buckling of edges
- Torsion of segments (wind-wheel effects)

To check the general behaviour of a shell concerning stability, Isler uses the following private empirical general buckling formula:

$$P_k = \alpha \cdot \beta \cdot \gamma \cdot E \cdot \left(\frac{t}{r}\right)^x \ge S \cdot P_{eff}$$

P <sub>k</sub>	Critical buckling load
α	Number between 0,2 and 1,2
β	Practical reduction factor (inaccuracies
	etc.) about 0,3
γ	Form specific factor for every shape
Е	Modulus of elasticity short and long period
t	Shell thickness
r	Local radius of curvature
х	Power between ca. 2 and 3, dependent on
	the form, cylinder = 3, sphere = 2
S	Safety factor
P <sub>eff</sub>	Real load

This formula proves that the main influence on stability comes from the design, namely:

Three factors depend on the form or shape which is chosen:  $\gamma$ , r and the most powerful: x, the power of



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t/r. Cylindrical parts of a shell have about 30 times less buckling resistance than spherical ones because the power x becomes 2,5 up to 3 instead of 2,0 with a sphere. Two factors depend on right or false dimensions: shell-thickness t and curvature r both also chosen in the design. And on factor deals with the loads: Peff, also here design decides on the buckling safety: whether one estimates the loads correctly or insufficient.

## Correlation of Form and Stability

Sufficient curvature, sufficient shell thickness and sufficient hardness (E-module) of the material are of great importance, to avoid overall and local surface buckling. Also exact forming, accurate pouring, vibrating, smoothening, good curing watering protection against evaporation.

Free edges can be stiffened by edge members, by additional transverse curvature, by transverse counter curvature, or by avoiding compression in free edge zones.

## Creep and its influence on Stability

Creep is the slow increase of deformations under loads which do not increase. By creep a shallow arch or shell can get flatter and flatter until the angle of curvature becomes zero. At this moment even infinite reactions (normal forces) could no longer hold any perpendicular force. The only reserve lies in the bending stiffness of the buckling area. In practice, creep is in a way equivalent to a reduction of the modulus of elasticity E. Extensive creep is a direct consequence of low quality of concrete and creep becomes additionally dangerous in shells of big initial deformations.

## Small Deformations Desirable

Figure 4-48 shows a shell with a high amount of deformations. The causes were a non ideal shape (segment of a sphere and too flat) and low quality of concrete (pumped, little vibration, too liquid) The deformations of 4 years creep were 300% bigger than the elastic deformations of the first days. These results are not good, although there is not yet an actual danger, because the shell is thick enough.

Isler uses the amount of deformations as a measure of quality of a shell building. He tries to achieve the smallest possible, because they increase the safety of stability. Small deformations do not cost anything. By perfect form giving in the design, accuracy in the formwork and careful pouring of the concrete the desired small deformations can be achieved. (Isler, 1982)

## The influence on E by Cracking

A problem typical for concrete structures is cracking. Cracks appear where the tensional strength of the concrete is exceeded. Sometimes the cracks appear after months of use, sometimes years later. Cracking means infiltration of water, this can lead to damages by ice and corrosion. This does affect the stability indirectly. But stability is affected directly by the increased deformations resulting from cracking. In the shell-buckling formula it again means a substantial reduction of the modulus of elasticity E. A reduction according to Isler to less than 50%.

#### 4.5.8 **Ribbed shell structures**

Besides the shells with a full cross-section, multiple grid shells or ribbed shells are designed in the past.



Figure 4-45: Deitingen Service Station, Image from (StructuraE, 1998)



Figure 4-46: Heimberg Indoor Tennis Centre, Image from (StructuraE, 1998)



A good example of a ribbed dome is the Pallazo Dello Sport, designed by Pierre Luigi Nervi. Nervi has designed the grid pattern of its Pallazo Dello Sport in a way that the stiff beams are placed under an angle with the meridional forces as shown in Figure 4-47. The reason for this can be that the required dimensions for ribs when placed in line with the meridional and hoop-direction are not alike. A second reason for this grid pattern is that a pattern in line with the meridional and hoop-forces is not torsional stiff and can suffer from the wind wheel effect, a buckling mode where the crown of the dome starts to rotate in its horizontal plane. See It has to be stated that Nervi was both an architect and an engineer, in his designs one can see his knowledge of structural mechanics but on the other hand did he chose for structures which are more aesthetic than the mechanical optima.

## 4.5.9 Design recommendations

The following design recommendations can be concluded from studying shell action as the internal carrying mechanism.

- The shape of the structure is of crucial importance for shells, the most efficient shapes can be found by inversion of hanging membrane models.
- Sufficient curvature at every point
- Equilibrium at the supports (ties etc.)
- Stiffened edges or edges with additional transverse curvature to avoid buckling
- Shells with small initial deformation are less vulnerable for creep

By applying these design recommendations to the design at places where arch action is a dominant type of carrying mechanism, the internal stresses can be kept at a low level and the deformations can remain small.



Figure 4-48: ENSE Ski School Chamonix, Image from (StructuraE, 1998)



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Image from (Polonyi, 1989)

#### The Orange peel 4.6

#### 4.6.1 The geometry

The basic structure described above consists of multiple geometrical parameters which can be adjusted to change the shape and thereby the effect the internal stress distribution of the structure. As a starting point for this study a model with the following specifications is used.

## Geometrical properties

- Global length L = 150 m
- Height f = 20 m
- Width w = 38 m
- $\alpha = \beta = 45^{\circ}$
- $R_{main} = R_{transverse} = 150 m$
- $L_{cutoff} = 9 m$
- Thickness T = 1,0 m Tilt angle = 0°

#### Material properties

- Youngs modulus  $E = 10.000 \text{ N/mm}^2$
- Poisson ratio v = 0.2
- Density  $\rho = 2400 \text{ kg/m}^3$

#### Support conditions

- Clamped
- No edge beam

## Loading conditions

Dead weight

#### 4.6.2 **Internal stress distribution**

Based on the geometry of the structure it is expected that the main type of moment carrying mechanism will be arch action along the free edge.

A first estimate of the internal forces can be made when the structure is simplified like an arch with a thickness of 1meter as presented in

By 
$$F_c = \frac{q \cdot l^2}{8 \cdot f} = \frac{24 \cdot 150^2}{8 \cdot 20} = 3375kN$$
  
 $R_v = \frac{q \cdot l}{2} = \frac{24 \cdot 150}{2} = 1800kN$   
 $R = \sqrt{F_c^2 + R_v^2} = \sqrt{3375^2 + 1800^2} = 3825kN$   
**54**

$$\sigma_c = \frac{F_c}{A} = \frac{3375E3}{1E6} = 3,4N/mm^2$$
$$\sigma_R = \frac{R}{A} = \frac{3825E3}{1E6} = 3,8N/mm^2$$

A first estimate for the deformation can be obtained if the arch is analyzed as a two bar truss.

$$\Delta L = \frac{\sigma_{av.} \cdot L}{2 \cdot E} = \frac{3.6 \cdot 150E3}{2 \cdot 10E3} = 27mm$$
$$\Delta f = f - \sqrt{\left(\sqrt{\left(\frac{L}{2}\right)^2 + f^2} - \Delta L\right)^2 - \left(\frac{L}{2}\right)^2}$$
$$\Delta f = 20E3 \cdot \sqrt{\left(\sqrt{\left(\frac{150E3}{2}\right)^2 + 20E3^2 \cdot 27}\right)^2 \cdot \left(\frac{150E3}{2}\right)^2}$$

Δf=105mm

It has to be stated that the obtained deformation is found by a rather rough simplification where the geometry is simplified and only extensional deformation is taken into account, deformation by bending is not considered.

A more proper way to internal stress distribution of



Figure 4-49: Used geometrical properties for initial geometry



Figure 4-50: Simplified arch structure



Figure 4-51: Simplified truss structure





**″**⊔Delft

Jasper Janssen

the structure is by means of a finite element analysis. This analysis is performed with the program DIANA. In DIANA a model is made with the geometrical, materials, support and loading properties as described above and analyzed with a physical and geometrical linear calculation.

The stress distribution found by the model complies with the expected idea that arch action is main type of moment carrying mechanism. The model reveals a maximal deflection of 157mm at the middle of the free edge and decrease towards the supported edges. The magnitude of the obtained deformation is larger than for the manual calculation but this was expected because of the neglected bending deformation. The following actions can be distinguished when analyzing the distribution of the principle normal forces, see Figure 4-52.

a. Arch action between the two short supported edges, the angle of the arch is somewhat inclined towards the free edge. The magnitude of the arch forces differs little from the forces obtained by the



Figure 4-52: Principal normal forces N1 (left) and N2 (right)

manual calculation.

- b. Compressive forces in line with the two short supported edges due to the fact that the arch forces [a] are spreading from the supports.
- Tensile forces transverse towards the c. middle of the long supported edge due to the inclination of the primary arch forces [a].

Besides the arch action also bending is found as an important type of moment carrying mechanism. The principle positive bending moments at midspan [a] and the negative bending moments near the supports [b] comply with the deflected shape of the structure. The main arch action is mainly present along the free edge, in the centre of the deck, where normal forces are less dominant, bending moments in the transverse direction are present [c]. Based on the total moment distribution it can be stated that the amount of bending stresses is too high.

The large difference between the internal forces in



the roof structures reveals that the used equal material distribution is rather inefficient.

## 4.6.3 Buckling behaviour

In section 4.4 it was already stated that structure which internal moment carrying mechanism is primary based on axial (compressive) forces can become critical for buckling. The buckling behaviour of the initial structure is studied with an Euler stability analysis. An Euler stability analysis gives information about `linearized stability' of a structure. It tells whether solutions from linear elastic analysis are stable or whether small disturbances to those solutions exist, requiring no extra external energy. This type of stability analysis does not allow for any physical nonlinearities (e.g. cracking), geometrical nonlinear (e.g. large deformation) effects are only partly taken into account. However, often it is a relatively simple and effective method to get a fair impression of a structure's buckling behaviour.

When the linear Euler stability analysis is performed for the terminal structure loaded by its own weight, the three lowest buckling-modes as presented in Figure 4-54 are obtained.

The obtained bucking values present the factor which the loading has to be multiplied by before the structure will suffer from that particular buckling mode. The first buckling mode has a buckling value of 2,11 times the dead weight of the structure, this seems an adequate level of safety, but it's not!

The obtained buckling value suggests that the

structure will collapse under a load of 2,11 times the dead weight of the structure. However, the obtained values are valid for a idealized geometry. The 'real structure', which contains local imperfections will become unstable under a load which is lower than the obtained buckling load. The second buckling mode has a buckling value of 2,26, a difference of 7% with the first, indicating that the structure is not extremely sensitive for imperfections.

More advanced buckling analysis which includes imperfect geometry and nonlinear material properties are highly complex, not practical during the design stage and therefore lies outside the scope of this project. To ensure in the design stage that the structure has enough buckling capacity a higher buckling value should be aspired.

Based on the statements presented by Hoogenboom in (Hoogenboom, 2006) and earlier discussed in Section 4.5.6, the following buckling value is aspired:

B.V.>6 for structures that reveal to be highly sensitive for imperfection, this is where the second smallest buckling load is very close (say within 2%) to the smallest buckling load.

B.V.>5 for structures that have a normal sensitivity for imperfections, this is where second smallest buckling load is over 4% larger than the smallest buckling load.

for the second s

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It has to be stated that literature does not give statements about precise buckling values which



should be aspired during the design stage of specific structures.

Analysing the lower buckling modes of the initial geometry, one finds that the largest amplitude of the deformed shape is found in free edge. This type of instability could be expected when comparing the initial geometry to one of the shells designed by Heinz Isler, which all have additional curvature along the free edges. By increasing the stiffness of the free edge, the amount of energy that is required before it buckles will be larger. In chapter 6 the designed geometry will be adapted in a way that the buckling value will be improved.

## 4.7 Conclusion

A roof design can be based on different structural systems. Every structural system has its specific properties and field of application. The Orange Peel shape contains parts of an arch, dome, floor and cantilever. It was therefore expected that the internal stress distribution of this shape was based on a combination of internal carrying mechanisms. For this reason, the various moment carrying mechanisms are studied separate to obtain the relation between the geometry of a structure and the transfer of forces and buckling behaviour.

First bending is studied, the internal carrying mechanism that is used by both beam and floorelements. The internal carrying mechanism for bending is a couple formed by internal forces in the upper and bottom part of a cross section with the same magnitude of an external applied moment. Bending can be distinguished by a significant difference in internal stress in the upper and bottom part of a cross section. Heavy loads transferred by bending require a high amount of construction depth. When a slender structure is obliged, the amount of load transferred by bending should be kept low. When dead weight of a structure is dominant, an efficient type of cross section can be used to reduce material and thereby loading. Longitudinal shaping of a beam with respect to the external bending moments is another way to reduce the amount of building material. The use of slender elements can lead to instability, this can be solved by using a transverse bracing or using element with sufficient lateral stiffness.

In three or four side supported plate and grid structures, plate action can be distinguished. Plate action is a combination of bending and torsional shear forces. Just as for beam action is the internal carrying mechanism based on internal forces. Therefore a large construction depth is required to transfer significant load. A the shape of a grid structure based on the isostatics pattern of a plate requires no torsional forces and therefore requires less material than a ordinary grid structure.

Arch structures are usually relatively deep and inherently provide a very high large internal moment arm. The forces in the resisting couple can thus be fairly small with the structure still capable of providing a very large internal resisting moment. Arch action will cause horizontal forces in the support structure, supports with a high stiffness are therefore crucial. The amount of bending in an arch can be kept at a minimum if the arch shape is based on the funicular line of the governing load present. high arch can become vulnerable А for asymmetrical buckling, for a low arch snap trough buckling can become the governing type of instability. By reducing the loading on an arch or by increasing its stiffness, a higher buckling value can be obtained.

Shells have an internal carrying mechanism which is, when designed properly, totally based on axial forces. However the relation between the geometry and internal forces of a shell is highly complex. The most efficient shapes for shell structures can be found by inversion of hanging membrane models. A shell structure should have sufficient curvature at every point. Support conditions are, similar to arches, of crucial importance for shell structures. Shell structures can, when not designed properly, become vulnerable for local buckling. Stiffening the free edges of a shell structure by increasing the transverse curvature increases the buckling value.

In the previous sections a circular hole with a diameter of 20 meter was covered using different structural elements to find out the efficiency the different types of load carrying mechanisms. The differences in internal forces during these examples where significant and proved that for the design of a slender structure the amount of beam and plate action should be kept at a minimum. Arch and shell action are proven to be much more



efficient. Shell action has next to this the advantage that it has a 2-dimentional force flow. The advantage of a 2-dimention force flow is that non ideal shape-loading combinations will not lead to high internal stresses.

A study into the structural behaviour of the designed Orange peel shape reveals that the main type of moment carrying mechanism is arch action. Besides arch action the structure contains a relative high amount of bending. Some undesirable results for the first configuration of the geometry are.

- High amount of deformation due to bending
- The structure is vulnerable for buckling of the free edge
- Large differences between internal forces

In chapter 6 the designed geometry will be adapted to improve the structural behaviour.





# Shape Study

In Section 4, a global shape is described which complies with the preferred design concept for the ferry terminal. A first FE-calculation revealed that the preferred geometry has an undesirable structural behaviour:

This chapter reviews how by modifying the initial geometry of the roof structure the structural behaviour can be improved. In Section 5.1, different methods are reviewed which can be used to modify the geometry of a design in a way its structural behaviour improves.

Several computational form-finding applications are currently available which can be used for the design of cable-net, membrane and shell structures. By using a computational soap film model as an example, the possibilities and impossibilities of these applications are reviewed in Section 2.

Section 5.4 discusses a method developed to convert the bending moments of the initial structure into axial forces. This method uses the initial deformation of the structure to find an optimized geometry.

Section 5.5 discusses how the geometry of the shape can be modified in a way that the buckling behaviour is improved.

Finally, Section 5.6 studies the required thickness on a more detailed level which is required to fulfil all the structural demands.



## 5.1 Optimization methods

In Section 3.6 the "Orange Peel" geometry is introduced as a starting point for the design of a self-bearing roof structure. This geometry, analysed in Section 4.6, shows the following undesirable structural behaviour.

- Large deformation due to bending
- Low critical buckling load
- Large differences between internal forces

In this chapter, the geometry of the designed roof structure will be modified in order to improve its structural behaviour.

As shown in Figure 5-1 multiple design methods can be used to find a shape with a more favourable internal force distribution. Al these methods have their own limited field of application.

The relation between shape and flow of internal forces can be highly complex. To gain insight into this relationship, a parametric shape study will be performed. A parameter study defines systematic the effect of shape parameters. This lowintelligence method studies the difference in internal forces after changing one of the shape



parameters which describe a specific geometry. Performing a parameter study improves the insight into the structural behaviour but has a limited solution-space. The best attainable solution will be a parametric setting within the prescribed boundaries of the initial geometry. In case of the Orange Peel shape a surface of revolution with circular meridian. The solution space can be enlarged by increasing the number of shape parameter. By using for example an elliptic or parabolic meridian new parameters are obtained which can be adjusted. The disadvantage of this approach is that, for complicated geometry's, it becomes highly labour intensive.

Another way to find a geometry with a preferred internal stress distribution is by using physical models. Two types of physical models that are sometimes used by structural engineers are soapfilm models and hanging models.

Soap-film models, as presented in Figure 5-2, use the natural property of a soap-film to find a minimal energy surface between preset (closed) boundaries. Soap film models can be used for the design of membrane structures and pneumatic structures. The shape obtained by a soap film model will contain the minimal amount of tension for a non-loaded situation. This shape is not an optimum for a loaded shell-like structure. Therefore soap film models are not useful for the design of a shell-like roof structure.

Hanging models are the physical equivalent of the membrane theory and are used to find the funicular shape of a specific type of loading. A hanging model can be made out of fabric, ropes or cables and is formed by gravity (its own weight). Hanging models can be used for the design of cable net, arch and shell structures.

Simple hanging models where used in the 19<sup>th</sup> century. A more complex and better known example is the hanging model of the Colonia Guelli church in Colonia Guell (Spain) of Antonio Gaudi,



Figure 5-2: Soap film model, Image from (Coenders, 2006)



presented in Figure 5-3. Gaudi used the hanging model to find the funicular lines for the roofs of its church. Little bags on the model contains weights which represent the weight of the real building elements.

Physical models are helpful for studying shapes, but in practice they have disadvantages. It is for instance very hard to measure the shape directly of a physical model. The use of physical models is also limited because only geometries can be obtained that are based on a single load carrying mechanism. (for instance membrane forces) Geometries where the structural system is based on a combination of load carrying mechanisms, (membrane forces + bending), which is the case for the roof design of the terminal, cannot be obtained by physical models. Because of this, physical models will not be used during the shape study of this project.

To overcome the compatibility problems with physical models, some computational models exist which are based on the same principles as physical models. Oasys GSA 8.2, a structural analysis program created by ARUP, incorporates a soap film and force density module that can be used for the design of funicular and minimal energy shapes. Use of these models has benefits in comparison with physical models. By using computational soap film models as an example, the use of the current available computational form finding models is reviewed in Section **2** 

Physical and computational form finding models generate a ideal geometry for a specific load carrying mechanism. A initial shape which requires a combination of carrying mechanisms will not lead to a convergent solution.

It is also possible to use the results of a structural analysis to optimize a shape. With this iterative approach, which can be performed either by hand or computer, it is possible to improve the structural behaviour for structures with any type of moment carrying mechanism. For this project, the results of an finite element analysis are use twice to optimize the geometry with respect to its structural behaviour. The amount of bending moments in the roof structure is reduced by adapting the initial geometry using the structures initial displacement field. This method which is an example of direct result based shape modification is discussed in Section 5.4.

In Section 4.6.3 it was already stated that the roof structure is vulnerable for buckling. The geometry is therefore modified by hand in a way that more energy is required for the governing buckling mode to occur. This is discussed in Section 5.5. The final step in this form finding study is an optimization on a more detailed level. Thickness and stiffness of the structure are hereby synchronized with the internal stress distribution present.



Figure 5-3: Hanging model of the Colonia Guelli church, Image from bridgey.de





## 5.2 Parametric shape study

One parametric configuration of the Orange Peel shape was analyzed in Section 4.6. In this section a parametric shape study is performed to define the effect of each shape parameter.

The Orange Peel shape, described in Section 4.6, consists of multiple geometrical parameters which can be adjusted to change the geometry and thereby effect the flow of forces of the structure. In this section, the relation between each individual parameter and the internal stress distribution is studied further. As a starting point for this study a model with the following parameter configuration is used. See Figure 5-4.

## Geometrical properties

- Global length L = 150 m
- Height f = 20 m
- Width w = 38 m
- $\alpha = \beta = 45^{\circ}$
- $R_{main} = R_{transverse} = 150 m$
- $L_{cutoff} = 9 m$
- Thickness T = 1,0 m
- Tilt angle = 0°



Figure 5-4: Used geometrical properties for initial design

#### Material properties

- Youngs modulus E = 10.000 N/mm<sup>2</sup>
- Poisson ratio v = 0,2
- Density  $\rho = 2400 \text{ kg/m}^3$

#### Support conditions

- Clamped
- No edge beam

#### Loading conditions

Dead weight

Difference in support conditions, tilt, height, transverse curvature and the use of an edge beam will all effect the internal stress distribution. The aim is that by looking at each of these single parameters a configuration is found that has the most favourable structural properties.

It has to be stated that all the results are based on geometric linear calculations and linear material properties. The quantitative value of the results are therefore not very accurate, however the result difference between models gives valuable qualitative information. In Chapter 6, more accurate models will be used to improve the accuracy of the results.

## 5.2.1 Clamped vs. Pinned constraints

In Section 4.5 it was already stated that support conditions have big influence on the internal forces of a shell structure. To study the influence of different support conditions on the flow of forces in Orange Peel shape a model with an identical



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geometry and pinned constraints is made.

The model with pinned constraints reveals a maximal deflection of 170mm at the middle of the free edge, 8% more compared to the 157mm for the model with clamped constraints. Figure 5-5 shows the maximal principle normal forces of a structure with clamped [left] and pinned [right] support conditions. The model with clamped constraints has a more uniform distribution of arch action along the surface.

The mayor differences between the two models becomes visible when analyzing the principle moment distribution, presented in Figure 5-5 [R]. By allowing the supports to rotate the bending moments near the edges disappear. The moments transverse to the arch action are increased by about 60% and shifted towards the support.

It has to be stated that the choice for typical support conditions can not only be made on behalf of the internal distribution of forces. The support conditions are dependent of the way the foundation of the structure is detailed and build. Detailing and building the supports of a concrete deck in a way it is free to rotate is difficult. The reduction of deformation and bending moments in combination a buildable support detail gives the preference to a foundation which reacts as clamped. The effect of using a support structure which reacts not perfectly stiff is studied further in Section 6.1.3.

#### 5.2.2 Choosing the right tilt

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The tilt of the geometry determines the amount of overhang from the line of supports. Besides the difference in overhang and appearance the amount of tilt has impact on the internal force distribution. The amount of tilt is gradually increased from the initial structure, which has no tilt to 5°,10°,15° and 20°, see Figure 5-6.

According to the design requirements stated in Chapter 3, both the outer two options, 0° and 20° tilt, are not desired. The structure with no tilt is to high at the quayside and therefore requires too much adjustment of the existing guay, the structure which is tilted by 20° is so steep that it cannot be used as an accessible public square. The 5 different models reveal no significant changes in the flow of internal forces for tilted structure compared to the initial one. From the design requirements mentioned before, a tilt of 10° is chosen.

#### 5.2.3 **Transverse curvature**

In this section the influence of different transverse curvature in relation with the internal force distribution is studied. The basic structure is a part of a sphere, has the same curvature in all directions ( $R_{torus}$ =  $R_{generator}$  =150m). By projecting the basic structure on a torus instead of on a sphere one can influence two radii. To study the relation between the transverse curvature and the internal stress distribution models are made with  $R_{generator} = 150m$ (initial geometry), 120m, 100m and 75m.

As displayed in Figure 5-7, models with more transverse curvature show larger displacements. The fact that more curvature leads to larger displacements seems to differ with what would be expected in relation with shell action. However, when the results are studied more in dept one can come up with the following explanation. The force distribution in the deck is mainly based on arch action instead of shell action. By increasing the transverse curvature the depth of the arch shape in position of the free edge is in fact reduced. In Section 4.4 is was already stated that the stiffness of a flat arch is less than that of a steeper one. The free edge which becomes more flat by increasing the transverse curvature will deform more and attract less normal forces. The main arch forces will pass through the centre of the deck, where there is more dept. See Figure 5-8.

The initial idea of improving the stress distribution by adding additional curvature is thereby refuted. Because arch action is governing, it is more important to give the free edge more height than to



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increase the transverse curvature.

#### 5.2.4 **Construction height**

In Section 4.6 was found that the flow of internal forces in the roof structure is mainly based on arch action. A study into the structural mechanics of arches, described earlier in section 4.4, arch depth reveals to be an important factor. It was found that a high arch loaded by it s own weight has lower internal forces than a low arch. For this reasons it might be expected that a high roof shape will be favourable over a lower roof shape. However the design requirements described in chapter 3 state that from aesthetic point of view and from the fact that the deck will be used as an public square a lower roof shape has the preference.

The internal force distribution is analyzed for geometries with a height of f= 20m (initial geometry), f = 15m, f = 12m and f = 10m. The deflected shape of the two most extreme geometries is presented in Figure 5-10.

By analyzing the principal force distribution it can be stated that the type of load carrying mechanism remains and that the magnitude of forces almost linearly increases by a decreased height. The deflection increases approximately quadratic for

lower deck shapes.

The difference between structural, functional and aesthetical preferences makes it hard to pick a appropriate height. When a high deck shape is chosen, the structural demands will probably be met but the deck is not as functional as it can be. When a low deck shape is chosen, it is possible that in a later stage, when a more complex analysis will be performed the structure cannot be optimized to a level that all the structural demands will be met.

In Section 4.4 was found that for arches loaded by their own weight a height to span ratio of 1/12 is a reasonable ratio. This comes down to a height of 12m. When later in the dimension phase it is revealed that not all the structural demands can be met, the height of the structure should be increased.

It has to be stated that the height of the basic shape is not the same as the vertical distance between the top of the deck and the level of the existing quay. Because the basic shape is somewhat cutoff at the sides, the total deck is a bit tilted and the position of the lifted quay is not yet determined the real level of the top of the deck cannot be determined.

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Figure 5-7: Displacement for structures with R<sub>generator</sub> = 150 [L] and 75 meter [R] (Deflection multiplied by 25)



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Figure 5-8: Minimal principle normal forces for R<sub>generator</sub> = 150 [L] and 75 meter [R].

#### 5.2.5 The use of an edge beam

In Section 4.5 was stated that shell action, with axial forces in two directions, is more efficient than arch action, with axial forces in only one direction. It was also stated that some shell structures, like for instance hyperbolic paraboloid shells, make use of edge beams as support. Based on these statements the question arises how the internal stress distribution will be affected if an edge beam is introduced. It can be expected that the additional stiffness of the edge beam will mobilize additional membrane forces in the transverse direction which will have a positive effect on the deflection. To study the effectiveness of an edge beam, a model with an edge beam of 1 x 1 meters is compared with a second model containing an edge beam of 0.25 x 4 meters. The chosen dimensions are used because they have the same weight and the additional bending stiffness of the high beam, which is 16 times the stiffness of the square beam, is large enough to measure the efficiency.

When analyzing the results between the model with a normal and a high edge beam the difference is poor. The deflection for a roof with a large edge beam decrease only 5%, from 416 to 396 mm, where the terminal has gained a high edge element which conflicts with the desired slender appearance. Because of the low efficiency of an edge beam it will not be used.

#### 5.2.6 **Optimized parameter configuration**

After studying the internal stress distribution for models with different support conditions, tilt, height, transverse curvature and an edge beam the following parameter configuration is found.

Geometrical properties

- Global length L = 150 m
- Height f = 12 m



- $L_{cutoff} = 9 m$
- Tilt angle = 10°

Support conditions

- Clamped
- No edge beam

Afterwards can be stated that adapted parameter configuration is mainly based on design requirements instead of structural preferences. The undesirable structural behaviour that was found for the initial parameter configuration thereby remains the following.

- High amount of deformation due to bending
- The structure is vulnerable for buckling
- Large differences between internal forces



Figure 5-9: Optimized parameter configuration



Figure 5-10: Displacement for structures with differing height (Deflection multiplied by 25)





#### Soap film modelling 5.3

A few computational form-finding applications are currently available which can be used for the design of cable-net, membrane and shell structures. By using computational soap film models as an example the possibilities and impossibilities of these applications are reviewed.

With the soap film application in Oasys GSA 8.2 it is possible to generate geometries with an equilibrium between specific loading and internal membrane forces. When an inverted uniform gravity load is applied on the model instead of a uni-directional pressure load which is common for soap films a geometry is found that has more similarities to physical inverted hanging clothes models than with physical soap film models.

#### 5.3.1 Test, square plate

To learn how the soap film application works, a preliminary model of a simple square plate is made. This sheet contains the following elements: (See Figure 5-11)

- Edge bars
- A mesh of Quad 4 elements
- Spacers in 2 directions



Figure 5-11: Elements in soap film model

The structural properties like stiffness and weight of the beams and the mesh are not used in the form finding process. The spacers are required to keep the mesh equally distributed over the surface. The axial stiffness of the bars and mesh can be given to the stiffeners. The edge bars and the mesh elements are given soap film properties (prescribed internal tensile force), required to balance the external and internal forces.

By playing with the models external load and internal tensile forces different shapes of equilibrium can be obtained. The analysis is performed by a solver based on the dynamic relaxation. Dynamic relaxation is a method which lets the structure relax to a equilibrium situation. The velocity of the displacement converges to zero and the stiffness of the structure increases. The advantage of this method over the well-known stiffness matrix is that this method does not have to solve matrices which take a lot of calculation power.

When the simple square sheet model is given soap film properties (prescribed internal tensile force) without any external loading, the analysis finds a flat minimal energy surface. See Figure 5-12 (B). When besides soap film properties also external loads are applied to the mesh corner points, one finds an 3D surface. See Figure 5-12 (C).

By changing the axial stiffness of the spacers one can influence the shape of the generated surface. It has to be stated that by using flexible beam or mesh not only the shape but also the position of the load repositions.



Figure 5-12: (A) unloaded model, (B) with soap film properties, (C) with soap film properties and external load



Figure 5-13: (A) Stiff elements, (B) Flexible edge beams, (C) Flexible mesh

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# 5.3.2 Soap film modelling on the terminal design

To use soap film modelling for the roof structure of the terminal a mesh is required which follows the layout of the design. The mesh-coordinates for this mesh are created with an parametric model in Excel, shown Figure 5-14.

When the mesh grid was finished all the quad elements, bars, spacers and supports where modelled in GSA, also with the help of Excel.

After the first test runs it becomes clear how limited the use of this form finding application is. The program can only find solutions based on membrane forces. The only way to obtain a shape with the desired overhanging free edge is by fixing it in a preset position, see Figure 5-15. The obtained shape after fixing the free edge and making gaps for the entrances is shown in Figure 5-16. Fixing the free edge leads to no existing support reactions along the free edge and therefore a mayor difference between the model and the real structure. The obtained geometry is therefore not for the development of a new design.

Besides soap film application which use a dynamic relaxation solver to find an optimized shape, other computational form finding applications exist. Other applications use for instance a force density method to obtain shapes of equilibrium. As far as known are all these applications used for cable net, membrane and proper shell structures, all based on pure membrane forces. Designs which have a structural system which is based on a combination of axial and bending forces cannot make use of the form finding application which are currently available in the construction industry. Therefore another method is required to improve the structural behaviour of the Orange Peel design.







Figure 5-15: Model before soap film analysis





# 5.4 Improve internal stress distribution

A way to reduce the amount of bending is by using the results from an initial geometry as direct input for changes in shape. An important term within this subject is the funicular line or the funicular plane for three dimensional structures.

For statically determinate, two dimensional, frames it is known that by means of a familiar funicular line the bending moment distribution can easily be defined. By the contrary it can be stated that the bending moment distribution can be a useful input to define the funicular line and so the shape of the structure with minimal bending moments can be found.

This approach can be presented by the irregular arch-like geometry in Figure 5-17. It is known that for a uniform distributed load the funicular line is a parabola which intersects the three hinges. The magnitude of the bending moments is equal to the distance between the geometry and the funicular line multiplied by the compressive force present at that location.

For a three dimensional structure this approach is a bit more difficult because bending moments act in varying directions. The shape of the structure can therefore be adapted according to for instance bending moments in the X, Y or principle directions, M1 and M2. Another way to attain a more funicular structure is by adapt a geometry according to its deflection.

#### 5.4.1 The deformation method

Using the deformation as input for geometry modifications is not done very often. As far as known no literature This method can be explained with the thought experiment shown in Figure 5-18.

A circular arch under a equally distributed loading will deform away from the funicular line, as shown at the left of Figure 5-18. When inverting this deflection, and adding that to the initial shape, a new shape is obtained lies closer to the funicular line than the original structure. In case of the circular arch a new shape is found that approaches a parabola, the funicular line for the equally distributed loading.

To find out if this method can be used for other geometries a simple irregular frame build up out of straight beam is studied. The initial geometry of the frame is presented by the full blue line in Figure 5-19. The deflection of the frame is obtained with a linear elastic calculation performed with DIANA. The dead weight of the beams cause the frame to deflect, presented by the dotted blue line in Figure 5-19. It has to be stated that for this method not the qualitative amount of the deflection is of importance but more the direction of displacement and the amount of deflection at a certain point with respect to the overall deformation. Geometric and material properties used for this example cause the frame to deflect with significant magnitude, for frames which deflect less a deflection factor  $\gamma$  can be introduced to get noticeable results. Inverting the deflection gives the dotted red line in Figure 5-19. The geometry obtained by the inverted



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deflection will in general be higher than the initial geometry. In Section 0 it was already stated that higher frame are more favourable with respect to internal forces. Comparing the internal force distribution between the two geometries is therefore not accurate. To compensate the increase of height, all the new found Z-coordinates are multiplied by a factor  $\alpha$  which stands for:

$$\alpha = \frac{Z_{max\,;initial}}{Z_{max\,;inv.deflect}}$$

Or in words, the total height of the initial structure divided by the height of the structure obtained by inverting the deflection. Applying the factor  $\alpha$  gives the full red line in Figure 5-19. The X-coordinates of the new geometry are the ones found by inverting the deflection. The obtained geometry is a bit more smooth then the initial one.

Figure 5-20 shows that when the deflection method is applied on the obtained geometry a new geometry is obtained that is even more smooth and starts to approach the funicular line.

When applying the deformation method multiple

times in succession one has to chose the deflection factor y each step in a way that solution approaches the funicular line. A small y will bring about positive geometric changes but with small magnitude where a large y will change the structure by an amount the funicular line is passed and an even less favourable geometry is obtained. In general we find that for every time the geometry is adapted the magnitude of the deformation factor y will decrease, indicating that the difference between the obtained geometry and the ideal funicular geometry has become smaller.

#### 5.4.2 Using the deflection method

Before applying the deformation method to the deck structure that was designed by the parametric shape study the allowable geometric changes have to be determined. DIANA presents for all the mesh nodes a deformations in the three main directions of the global coordinate system. Just like in the simplified model described above, the deformation in the vertical Z-direction will be multiplied by the deformation factor  $\gamma$  to determine the amount of deflection and the factor  $\alpha$  to keep the total height of the adapted geometry equal to the initial one.



Figure 5-19: Change of geometry according to deflection method



Figure 5-20: Irregular frame adjusted with the deformation method







The deflection in the X-direction, perpendicular to the line of symmetry, is multiplied only by the deformation factor y. The deformations in Ydirection, parallel to the line of symmetry are not taken into account. The reason for this is that the cantilever length in the middle of the free edge is larger for the deflected shape as for the initial one. The deformation method would therefore make the length of the cantilever smaller. It is clear that reducing the amount of cantilever length will have positive influence on the internal stress distribution but it is an unwanted change in geometry. During the parametric shape study all the geometries are modelled with clamped supports. This type of support condition results does not allow any rotation along the supports. The deformation method only can adapt the inclination at the supports if pinned supports are used.

When applying the deformation method three times sequential the deformed shapes as presented in Figure 5-21 are obtained. The obtained deflection for the 3<sup>rd</sup> iteration is equal to the one obtained after the second iteration. This means that a converged solution is found.

One can see that between the initial and the first adapted geometry the point where the largest deformation is present is shifted from the centre to the side of the deck. This indicates that the deformation factor  $\gamma$  is chosen a bit too large. However because the magnitude of the total deformation of the adapted geometry is smaller than the initial one, the adapted geometry is more favourable. The deflected shape of the 2<sup>nd</sup> adapted geometry is very smooth and does not contain any local points with increased deformation. The deformation factor  $\gamma$  that needs to be used for a 3<sup>rd</sup>

adapted shape is so small that the changes in deformation and internal stress geometry, distribution are negligible. The total reduction between the initial shape with a deflection of 457mm and the adapted geometry of 264mm is 42%.

The reason that the deformation of the adapted geometries does not converge to 0 is because of axial deformation. When a model was used that would not been able to deform axial it would be easier to see that the deformation method adapts shapes in a way that the amount of deformation by bending is minimized. Such a method could for the same reason make use of a fixed deformation factor y because the linear relationship between the deformation of a structure and the eccentricity of that same structure with its funicular plane.

After adapting the shape it is interesting to see how the internal stress distribution is changed with respect to the initial geometry. For this purpose a model is made with the geometry found by the deformation method and the clamped supports as used during the parametric shape study.

When analyzing the contour plots which belong to the principle moments and normal forces for both the initial as for the adapted geometry one can see the following:

- The bending moments along the supported edges are not changed worth mentioning
- The bending moments in the field are reduced significant
- The magnitude of arch forces, presented in



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the field of the N2 contour plot, is reduced.

Based on experiences described above the following statements can be made:

- From the found deformation and internal stress distribution it can be stated that by using the deformation method, the structural behaviour of the adapted geometry favourable over the initial one.
- When used for non-irregular structures, the deformation method requires only a

few adaption cycles to come to a adequate result.

Because the deformation method can be • adjusted in multiple directions it gives a lot of control for geometric changes.

## 5.5 Buckling behaviour

#### 5.5.1 **Buckling behaviour of the initial** structure

With a linear Euler stability analysis, the buckling



Figure 5-22: M1, initial geometry [left], adapted geometry [right]



Figure 5-23: M2, initial geometry [left], adapted geometry [right]



Figure 5-24: N1, initial geometry [left], adapted geometry [right]



Figure 5-25: N2, initial geometry [left], adapted geometry [right]





behaviour of geometry, obtained by the deflection method, loaded by its own weight, is analysed. The first buckling modes are presented in Figure 4-54.

The obtained buckling factor of 2,75 is much lower than the aspired value of 5 to 6. This means that the design has to be modified in order to improve its buckling capacity.

#### Improve the buckling behaviour 5.5.2

Properties of the roof structure which have a positive influence on the buckling behaviour are a decrease of loading or an increase of stiffness. For the roof structure this can be translated into:

- Decrease of weight (loading)
- Increase of more efficient cross section (loading + stiffness)
- Use a material with a higher modulus of elasticity (stiffness)
- Use an edge beam (stiffness)
- Change of geometry (stiffness)

Table 5-1 presents the effect of modifications to the structural model with respect to the buckling behaviour.

Model	Description	Buckling value	Relative
Initial model		2,75	100%
Weight	Divide material density by factor 2	5,50	200%
Thickness	Increase thickness by factor 2	5,39	196%
Modulus of elasticity	Increase modulus of elasticity by factor 2	4,97	181%

Use of an edge beam	Include an edge beam of 3x1 meter	3,13	114%
Curved edge 1	Curve free edge 2 meters up	3,97	144%
Curved edge 2	Curve free edge 2 meters up more steep	4,05	147%
Curved edge 3	Curve free edge 2 meters down	3,49	127%

Table 5-1: Possible improvements buckling behaviour

The decrease of weight (loading) and the increase of stiffness are methods to improve the buckling behaviour of a structure, but it can be questioned how this should be obtained. More interesting is to see how by changing the geometry of a structure the buckling behaviour is improved.

Analysing the lower buckling modes of the initial geometry, one finds that the largest amplitude of the deformed shape is found in free edge. By increasing the stiffness of the free edge the position of the largest amplitude will move from the free edge to the centre of the field, resulting in a buckling mode which requires much more energy than the initial structure.

Making the free edge stiffer can be done by adding and edge beam along the free edge of by giving the free edge a transverse curvature. Adding a edge beam of 3x1 meters along the free edge improves the buckling value with 13%, from 2,75 to 3,13. The positive effect of the edge beam is limited.

To study the effect of the curved edge three different curves where used, all with a height of 2 meters and a width of 5 meters, see Figure 5-28.



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The first curve gently rises upward, the second rises in quadratic speed upward and the third is curved downward.

The geometry of the terminal roof including a curved edge was obtained manually using the deformed geometry of a specific loading. This approach to modify the geometry is described in Appendix B-2.

The use of a curved free edge is with a buckling factor up to 4,05, an increase of 47% for the second curve much more effective than an edge beam. The main difference between the use of an edge beam and a curved edge is that for a curved edge the effective width of the edge is much larger and that the weight is smaller. Comparing the two upward directed curved reveal no great differences. The difference between the upward and downward directed edges is larger. The upward directed edge (concave) in contrast with the deck itself (convex) contains the most curvature, acts the stiffest and therefore has the highest buckling value. See Figure 5-27.

During the dimensioning stage, described in Chapter 6, the analysis is improved to obtain more accurate results. If during this stage the buckling value for the design load remains under 6, the structure should be modified in a way that it becomes stiffer or lighter.



Figure 5-27: Upward bend edge vs. Downward bend edge.



Figure 5-28: Curved edges 1 (left), 2 (middle), 3 (right)



# 5.6 Detailed optimization: saving weight.

After designing the global shape of the terminal deck it is time to optimize the structure on a more detailed level. Stiffness and weight reduction are the key-words in the design of the roof structure. During the research on different forms of load carrying mechanisms, three main principles where found to improve the efficiency of a structure knowing:

- 1.) Shape a structural element over its length in a way that material is saved where the internal forces are low [Figure 5-29]
- 2.) The use of more efficient cross sections, save material nearby the neutral axis [Figure 5-30]
- 3.) Make use of a levelled arranged structure [Figure 5-31]

The effectiveness of each principle is dependent on the stress distribution present in the structure. If in a certain cross section only axial stress are present, only the amount of cross sectional area is required to meet a prescribed maximal stress level. The shape of the cross section and therefore the stiffness is of no importance. For a cross section where bending stresses are present the moment of inertia (stiffness) of a cross section determines the magnitude of the maximal stresses. Efficient cross sections with high stiffness are also helpful on behalf of buckling stability. Cross sections with large stiffness require more energy to deform and are therefore less vulnerable to buckling.

Where longitudinal shapeing is concerned, the thickness of the structure is synchronized with the internal forces present. The internal forces used to determine the required thickness are the maximal principal normal force [N dl] and the maximal principal moment [M\_dl] for a structure loaded by



Figure 5-29: Longitudinal shaping (Schodek, 2001)



Figure 5-30: Efficient cross sections (Schodek, 2001)

its own weight. At this point of the form finding proces all the internal forces are obtained from a highly simplified design model which neglects the following aspects:

- Loadcases other than dead load
- Non stiff support conditions
- Creep and shinkage of concrete
- Geometrical 2<sup>nd</sup> order effects

Because of this it is shozen to use design loads which are equal to the internal forces under dead load multiplied by 2.0. The required thickness is determined by:

$$\sigma_c = \frac{2|N|}{A} + \frac{2|M|}{W} = \frac{2|N|}{t} + \frac{12 \cdot |M|}{t^2} \rightarrow$$
$$t_{req.} = \frac{24 \cdot |M|}{-2|N| + \sqrt{4N^2 + 48 \cdot |M| \cdot |\sigma_c|}}$$

The formula only contains only absolute magnitudes because several combinations of positive and negative internal forces lead to improper roots.

When using a maximal compressive stress of  $\sigma_c$  = 39N/mm<sup>2</sup> (B65) the required thickness as presented in Table 5-2 is obtained, See Figure 5-32.

Section [-]	N_dl [N/mm <sup>1</sup> ]	M_dl [Nmm/mm <sup>1</sup> ]	T_req. [mm]	T_used [mm]
1	-6000	2.00E+05	446	500
2	-5000	4.00E+05	502	600
3	-4000	3.00E+05	432	500
4	1000	2.20E+06	849	900
5	-7000	1.00E+05	430	500
6	-6000	2.00E+05	446	500
7	-3000	2.00E+05	337	400
8	-2000	8.00E+05	550	600
9	-7000	8.00E+05	707	800

Table 5-2: Required thickness



Figure 5-31: Levelled arrangement (Schodek, 2001)







Figure 5-32: Required thickness

By using longitudinal shaping, a required thickness is obtained which is fully synchronized with the internal forces present. The thickness and thereby loading is reduced at every part of the structure. From section 5.5 can be concluded that the buckling behaviour of a structure will improve when the load is reduces, however the thinner, less stiff cross section will have a negative effect on the structures buckling behaviour. In Chapter 7 the buckling behaviour of the structure is studied further.

The use of the grid stiffened grid as proposed in Section 4.3 does, when the internal stress distribution is concerned, has no benefits. The loads are mainly distributed by axial forces, therefore only cross sectional area is required. If the dimensioning phase reveals that additional stiffness is required to transfer bending moments due to additional load combinations or to improve the buckling behaviour, the use of the stiffened grid will be reconsidered.

## 5.7 Conclusion

During the parametric shape study, each of the design parameters of the Orange Peel shape is studied separate to find how it reflects both the structural behaviour and the design requirements. When the support conditions are compared it is shown that the bending moments for a pinned structure are 60% larger than for a clamped variant. Because the use of proper pinned detailing is objectionable it is chosen to use a foundation that acts as a clamped constrained. When the tilt of the geometry is concerned, the design requirements are found to be more important than the minor difference of internal forces. Chosen is a geometry with a tilt of 10 degrees. The used radius in the

transverse direction is 150 meters. This value was not based on the amount of curvature but more by the height of the structure in position of the free edge. The height of the total structure is a critical dimension for this design. A high structure contains low internal forces but also obstructs the view of the posterior buildings and the use of the roof as viewing point, where a low building contains very large internal forces. Chosen is a height of 12 meters. The use of an edge beam has no positive effect on the internal stress distribution.

The undesirable structural behaviour that was found for the initial parameter configuration thereby remained the following.

- Large deformation due to bending
- Low critical buckling load
- Large differences between internal forces

The amount of bending in the structure reduces if the geometry approaches the funicular plane of the governing load. A handful of structural analysis packages are available which can adapt a structure in a way that it equals the funicular plane of its governing load. These applications have been proven to be useful for the design of cable-net, membrane and proper shell structures. However they cannot be used for structures where membrane action and bending are combined.

For the initial design the use of existing form finding applications was found not possible. Therefore a method is developed which can approach the funicular plane of a structures loading without losing the control over its shape. In this method the geometry of a initial structure is modified by means of its own displacement field. By using the deflection method a geometry can be found after a few iterations where the amount of bending moments is significant reduced and converted into axial forces.

A first analysis of the 'Orange Peel' shape revealed that mostly its free edge was highly sensitive for buckling. In general it can be stated that the buckling behaviour of a structure improves by reducing the amount of loading (own weight) or by increasing its stiffness.

Studying the lowest buckling modes of the initial geometry and the existing structures of amongst



other Heinz Isler revealed that by applying more transverse curvature along the free edge would improve the structures buckling behaviour. After comparing 3 types of curved free edges a geometry was obtained which had a critical buckling load which was 47% higher than for the initial structure.

Already in Chapter 4 was stated that by synchronizing a structures depth with its internal forces a lighter and more slender structure can be obtained. After studying the internal forces present, the required construction depth could locally be reduced by 60%.

The proposed roof geometry is presented in blue in Figure 5-34. The figure presents the roofgeometry and the way the proposed roof shape can be blended to the existing quay. The proposed roof geometry and architectural render presented in the same figure diviate significant. It is herefore questionable if the obtained shape still complys with the initial design concept.

Is had to be stated that the results of the different form finding methods have effect on each other. For the deflection method a geometry is obtained for a thickness, and thereby load distribution, which is assumed constant. With longitudinal shaping, the thickness of the structure and the internal forces present are synchronized, hereby changing the load distribution and the optimal shape.

To find the shape where the load distribution, funicular shape and internal forces all comply, a iterative approach is required where all the form finding steps are used in a loop and performed several times. See Figure 5-33.

The reason that at this point no iterative form finding approach is used is because it is highly time consuming and a quasi accuracy is obtained which does not comply with the accuracy level of the



Figure 5-33: Iterative form finding approach





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design model itself. During the dimenisioning phase the obtained geometry will be annlazyed further using calculation which include the following aspects:

- Loadcases other than dead load
- Non stiff support conditions
- Creep and shinkage of concrete
- Geometrical 2<sup>nd</sup> order effects

When the calculation perfomed during the dimentioning phase point out that the obtained geometry does not comply with the internal force distribution, an iterative approach can be used to find a more favorable combination of geometry and internal forces.





# Dimensioning

In the previous chapter attention is given to the geometry of the roof structure. The aim was to find a shape which, given the design requirements, has the most favourable structural properties. The obtained shape is a result of a qualitative study, where the difference in structural behaviour is analyzed for each typical change in shape. During this process use is made of highly simplified models. Multiple properties of double-curved thin concrete structures are not incorporated in these design models. The quantitative results, magnitude of all forces; stresses; deformations etc. are therefore still to inaccurate.

This chapter describes how a more accurate insight into the behaviour of the structure is obtained. By using more advanced modelling techniques a study is performed to find out if the designed structure can withstand all the forces working on it. Hereby it has to be stated that this study is performed at a design level, more detailed calculations which results in a full detailed design including for instance required reinforcement lie outside the scope of this report.

Many changes have been made to come to a model which describes the structure with sufficient accuracy and has adequate strength stiffness and stability. The input and results of the final model are presented in Appendix F.



## 6.1 Increase calculation accuracy

#### 6.1.1 **Geometric non linear calculation**

All the previous FE- models analyzed the internal stress distribution of the roof structure by means of a linear elastic calculation. Hereby assuming the initial geometry of the structure. However, the internal stress distribution changes when the structure deforms. For the terminal roof the initial deformations found by linear calculation are, compared to the initial height of the structure, small. Therefore it is not expected that the use of a geometric non linear calculation, which takes this slightly deformed structure into account, will differ significant from the linear calculation. The difference between a geometrical linear and nonlinear calculation is further described in Appendix E.

#### 6.1.2 **Elaborate load cases**

During the design stage of this project only the dead weight of the structure was taken into account. This was done because deadweight was thought to be the dominant load case and to keep the amount of results limited. During the dimensioning part of this project more load cases and load combinations will be distinguished. First a study is performed to find out how the roof structure responds to a single type of action like for instance lateral loads, asymmetric loads or imposed deformations. Loads like wind and temperature are than composed as a combination of these actions in a way the behaviour of the load is described best. See Figure 6-1. The different actions that will be considered are:

- 1. Dead weight
- 2. Uniform distributed load
- 3. Surface load
- 4. Imposed extension
- 5. Imposed bending

The load cases that will be composed from these load types are:



- LC 1: Dead weight
- LC 2 & 3: Variable load by people e.a.
- LC 4,5 & 6 Wind load .
- LC 7 & 8 Temperature load

From the load cases stated above, combinations are generated which will be used during the total dimensioning phase. Because a large number of load cases and combinations will have effect on the calculation time each of the load cases will be analyzed once to find out if its contribution to the total result is significant or that it can be neglected.

#### LC 1: Dead weight

The dead weight of the structure is the dominant type of load. This load case is used during all the past design stages. The internal stress distribution for the terminal roof loaded by its own weight is clarified in the parametric shape study, Section 5.2. Even after the global optimization of the shape with the defection method and the detailed optimization the total deflection remains 165mm. (See Appendix F)

#### LC 2 & 3: Variable load

Load on the terminal roof imposed by for instance people works in the same direction as the dead load of the structure itself. This load case is modelled as a uniform distributed load acting in the vertical direction on the total surface of the structure. The stress distribution is quite similar with respect to the dead weight load case. However, because the load distribution differs from the dead weight load case, the deflected shape for this case is less smooth. This has to do that the shape is optimized for the dead weight load distribution instead of uniform distributed load. The maximal deflection found for a load  $q_{vl} = 2,5 \text{kN/m}^2$ is 33mm. (See Appendix F)

The roof geometry is optimized for its own weight, which is a symmetrical load case. To find out how





sensitive the model is against asymmetric loads, the structure is loaded only half. Applying a asymmetric load  $q_{vl} = 2,5 \text{kN/m}^2$  results in a deformation of 40mm. (See Appendix F)

#### LC 4,5 &6: Wind loads

The authorities of Dubai oblige that the magnitude of the wind pressure is obtained according to the British standard, which uses a complex method. In this design phase wind loads are therefore modelled according to the simplified method of the Eurocode 1. According to this code, wind load on a structure can be modelled by distributed forces perpendicular to the surface which can be obtained by:

$$W_e = q_{ref} \cdot C_e(z_e) \cdot C_{pe}$$

In which:

 $W_e$  = The total wind pressure in kN/m<sup>2</sup>

 $q_{ref}$  = The mean average mean wind velocity pressure:  $q_{ref} = \frac{\rho}{2} \cdot v_{ref;0}^2$ 

$$\rho$$
 = Density of air = 1,25 kg/m<sup>3</sup>

 $v_{ref;0}$  = Reference wind speed = 30m/s

$$q_{ref} = \frac{\rho}{2} \cdot v_{ref;0}^2 = \frac{1,25}{2} \cdot 30^2 = 5,6kN/m^2$$

 $C_e$  = The exposure coefficient which takes into account the effect of terrain roughness, topography and height above ground:  $C_e(z) = 1,5$ 

Load case 4 describes a wind load from the side. The external pressure coefficients  $C_{pe}$  for wind from the side are obtained from a standardized  $C_{pe}$ scheme which is available for domes. The exact  $C_{pe}$ factors for domes are presented in Figure 6-2 [left]. To simplify the input the  $C_{pe}$  are made linear as in Figure 6-2 [middle]. The total wind pressure  $W_e$  in kN/m<sup>2</sup> becomes equal to the values as presented in Figure 6-2 [right].

Load case 5 and 6 describe a wind load from the quay side [LC5] and from the creek side [LC6]. The wind pressure for both load cases is presented in Figure 6-4.

Wind load from the side will cause the asymmetric deformed shape presented in Figure 6-5 [left]. From studying arches is was thought that asymmetric load in the lateral direction would have a significant influence on the behaviour of the structure, however because of the relative low magnitude of the load, and the specific shape of the structure which has far greater lateral stiffness compared to an arch, the maximal deflection is only 33mm. (directed upwards) (See Appendix F)



Figure 6-2: Cpe-factors; Simplified Cpe-factors; Total wind pressure from side (load case 4)



Figure 6-3: Cpe-factors; Simplified Cpe-factors; Total wind pressure quay side (load case 5)



Figure 6-4: Cpe-factors; Simplified Cpe-factors; Total wind pressure creek side (load case 6)





The wind load from both the quay and the creek side has a tendency to lift the structure. Because the roof is so stiff in its own plane, there is hardly no deformation in line with the loads. See Figure 6-5 [middle and right]. The deformation found for wind acting from the creek side is 23mm and for wind acting from the quayside is 12mm. These values is so small that they will never be governing.

#### LC 7 & 8: Temperature loads

The effect of temperature can described by a imposed (partly) extension of the cross section. To get a grip on the response of the structure this is modelled in two ways, a total change in temperature and a gradient in temperature between the top and the bottom of the roof structure. Increasing the temperature over the total cross section causes the material to expand, see Figure 6-6[a]. The centre of the structure can deform and therefore has almost no internal stresses, the supports however remain clamped causing bending and therefore stresses in the deformed shape. An increase in temperature of 25°C over the total cross section results in a lift of 60mm.

When the total cross section is loaded by a temperature gradient, the topside of the cross section is subjected to an temperature increase and therefore expands, where the bottom side of the cross section is subjected to an temperature decrease and therefore shortens. This causes the cross section to bend, see Figure 6-6[b]. The maximal found deformation for a gradient of 25° over the total cross section is only 25mm and the stress levels are about half the magnitude as for the

temperature load over the total cross section.

Now the question rises how the temperature differences in Dubai can be modelled accurate. Figure 6-5 shows the monthly averages of the minimal and maximal temperatures measured daily in Dubai.

The difference of the mean daily temperatures between January and July, 35°-18°= 17°C will be used to model the difference trough out the year, [LC7]. This will be modelled by a increase in temperature over the total cross section. For the daily temperature difference, a magnitude of 41°-29°=12°C is used. The theoretical distribution for a daily temperature fluctuation, [LC8], is described as presented in Figure 6-8. This distribution cannot be modelled by a combination by a combination imposed extension and imposed bending. A more simplified way to describe the daily temperature difference can be found in the Dutch standard of concrete bridges, the NEN 6723: Voorschiften beton - Bruggen (VBB 1995) - Constructieve eisen en rekenmethoden. In this standard daily temperature differences are modelled by a temperature gradient over the top 300mm of the cross section. See Figure 6-8. This local gradient is however not supported by the element type used in the FE-model. The maximal stress for a cross section which cannot deform under an increase of temperature of 12°C is:

$$\sigma_t = E \cdot \alpha \cdot \Delta T = 1 \cdot 10^4 \cdot 1.2 \cdot 10^{-6} \cdot 12$$
$$= 1.44N/mm^2$$

If the cross section is free to deform, equilibrium conditions apply and the internal stress distribution according to Figure 6-9 will be obtained.



Figure 6-5: Deformation under wind loading, Left [LC4], Middle [LC5], Right [LC6] multiplied by 200 X

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The magnitude of the temperature gradient and the obtained stresses are low and will therefore be not included in the load combinations.

#### Load combinations

The load combinations that will be considered are presented in Table 6-1. The magnitude of the load factors is obtained from the NEN 6702: Technische grondslagen voor bouwconstructies – TGB 1990-Belastingen en vervormingen. It has to be stated that for a detailed analysis more load cases and combinations have to be considered.

со	Description	Magnitude
CO1	Dead load [LC1]	P=1,35 x q <sub>dl</sub>
CO2	Dead load [LC1] + Variable load [LC2]	$P=1,2 \times q_{dl} + 1,5 \times q_{var}$
CO3	Dead load [LC1] + Variable load [LC3]	$P=1,2 \times q_{dl} + 1,5 \times q_{var}$
CO4	Dead load [LC1] + Wind [LC4]	$P=1,2 \times q_{dl} + 1,5 \times q_{var}$
CO5	Dead load [LC1] + Wind [LC5]	P=1,2 x q <sub>dl</sub> + 1,5 x q <sub>w</sub>
CO6	Dead load [LC1] + Wind [LC6]	P=1,2 x q <sub>dl</sub> + 1,5 x q <sub>w</sub>
CO7	Dead load [LC1] + Wind [LC7]	P=1,2 x q <sub>dl</sub> + 1,5 x q <sub>w</sub>
CO8	Dead load [LC1] + Temperature [LC8]	P=1,2 x q <sub>dl</sub> + 1,5 x q <sub>temp</sub>

Table 6-1: Load combinations

The results, maximal deformations and internal Dubai: temperatures (°C)



Figure 6-7: Temperature data, Data source: www.allmetsat.com



stresses, can be found in Appendix F.

#### 6.1.3 Foundation stiffness

In the proceeding models clamped supports were used. Although the support structure of the terminal deck itself is considered to be out of the scope of this report it is not realistic to assume that it will not deform. A more realistic assumption is to model the foundation as a support which is fixed in the Z-direction and has elastic supports for the translation in X and Y direction and all the possible rotations. Figure 6-10 shows the translation part of the supports. The foundation stiffness will be chosen so that the maximal deformation of the supports in the XY plane will be 10mm. This magnitude of deformation is common used and based on experience. This same deformation will be a design requirement for the foundation.

The maximal found reaction force in one the nodes of the mesh located at the supports is 1.17e+7N/mm<sup>1</sup>, the required foundation stiffness is therefore: (See dimensioning model in Appendix F)

$$k_f = \frac{F_{react}}{\Delta L} = \frac{1.17e7}{10} = 1.17e6$$
N/mm

After applying the calculated stiffness, the

deformation is not the expected 10mm but only 4,6mm. The reason for this is that the used reaction force is a local maximum value. The non-stiff foundation spreads the reaction forces more uniform over the line of support. The required foundation stiffness is obtained with an iterative approach. After three cycles we find a deformation of 9,7mm for a foundation stiffness of 7.5e+5N/mm<sup>1</sup>.

For the rotation part of the support conditions a comparable approach is used. The maximal allowable rotation of the support is set at 2 mrad.

The required rotation stiffness for this allowable rotation is 1.6e+12 N/rad.

#### 6.1.4 **Material behaviour**

The complete form finding process is passed trough, assuming linear elastic material behaviour with a modulus of elasticity of 10.000 N/mm<sup>2</sup> and a Poisson ratio of 0,2. These properties are often used as indication for cracked concrete but can vary by a large amount under different circumstances. Structure like these are highly static undetermined, therefore stiff differences throughout the structure have influence on the distribution of internal forces. To increase the accuracy of the results it is required to incorporate a more accurate material behaviour. Not only cracking but also creep should be taken into account.

The relation between creep deformation and the elastic deformation is given by the creep function  $\phi(t,t_c)$ 

It holds:

$$\varepsilon_{cc}(t) = \phi(t,t_c) \cdot \varepsilon_{ce}$$

According to article 6.1.5 of NEN 6720, the creep function can be written as:

$$\phi(t, t_c) = k_c \cdot k_d \cdot k_b \cdot k_h \cdot k_t \neq \phi_{max}$$



Figure 6-10: Foundation translation stiffness

Where

$\varepsilon_{cc}(t)$	=	total creep deformation
E <sub>ce</sub>	=	elastic deformation of concrete
$k_c$	=	the influence of relative humidity,
	= 1,9	for a humidity of 60-85%
$k_d$	=	the influence of the age $t_c$ at the
		time of loading, as well as the
		strength class of the cement,
	= 0,7	for cement strength class 52.5
and		a concrete age of 28 days.
$k_b$	=	the influence of the strength class
		of the concrete,
	= 0,7	for B65
$k_h$	=	the influence of the geometry of
		the cross-section,
	= 0,7	for thickness > 400 mm
$k_t$	=	the influence of the time of
		loading t,
	= 1,0	for $t \to \infty$
$t_c$	=	the age of the concrete at loading
ť	=	standing time of the load
$\phi_{max}$	=	the maximal design value of the
		creep coefficient,
	= 1,6	

By using the parameters as described above a maximal creep coefficient of  $\phi(\infty) = 1.9 \cdot 0.7 \cdot$  $0,7 \cdot 0,7 \cdot 1,0 = 0,65$  is obtained.

Several simplified methods are available for the calculation of the shrinkage and creep behaviour of structures, a number of which are:

- The effective modulus method .
- The method of Trost
- The method of Dischinger

In this report only the first method will be used.

The internal stress distribution of the geometry obtained in Chapter 5 in combination with the support conditions as derived in Section 6.2.3 and loaded by CO3 according to Section 6.2.2 is presented in Figure 6-14.

The sections 1,2,5,6 and 9 contain primarily compressive stresses. This indicates that the cross section of the structure will not be cracked.

For cross-sections where only compressive stresses are present the effective modulus of elasticity can according to article 6.1.5 of NEN 6720 the be obtained by:





$$E' = \frac{E_c}{1 + 0.75 \cdot \phi_{\infty}} = \frac{38500}{1 + 0.75 \cdot 0.65} = 25882 \, N/mm^2$$

The sections 4 and 8 contain primarily bending stresses. This indicates that the cross section of the structure will be partly cracked.

If a significant amount of bending is present, a fictive linear stiffness can be obtained for the partly cracked cross section. This fictive stiffness is obtained by using a M-K spreadsheet, often used for the design of columns.

An indication for the required amount of reinforcement in a partly cracked cross section can be determined by:

$$A_s/m = \frac{\sigma_{top}}{2f_s} \cdot \frac{\sigma_{top}}{\left(\sigma_{top} - \sigma_{bottom}\right)} \cdot t \cdot 1000$$

This formula derives a required amount of reinforcement based on the area of the stress diagram presented in Figure 6-11. The formula assumes a tensile stress in the top surface.

For section 4, the following internal stresses are obtained by the FE-model:

$$\sigma_{top} = 7.2N/mm^2$$
  
$$\sigma_{bottom} = -5.3N/mm^2$$

The required amount of reinforcement, based on the area in the stress diagram is:

$$\frac{A_s}{m} = \frac{7.2}{2 \cdot 435} \cdot \frac{7.2}{(7.2 + 5.3)} \cdot 900 \cdot 1000$$
$$= 3972 mm^2/m$$

The fictive modulus of elasticity obtained by the M-K spreadsheet is:

 $E_{t=0} = 9133 \text{N/mm}^2$ ;  $E_{t=\infty} = 9028 \text{N/mm}^2$ 

For the output of the M-K spreadsheet reference is made to Appendix C.

For section 3,7 and 8 which consist of both bending and compressive stresses the M-K spreadsheet reveals that the stiffness hardly reduces compared to the fully compressed cross section. This has mainly to do with the tensile capacity of B65, which is  $5N/mm^2$ .

### 6.1.5 The used cross section

In Chapter 5 it was shown the buckling behaviour of a structure in general is influenced by 3 things, geometry load and stiffness. A favourable geometry with respect to buckling is obtained by giving additional curvature to the a curved edge.

Because during the form finding stage it was already known that support conditions, loading conditions and material stiffness would change during this dimensioning phase the obtained buckling values where only compared on a qualitative basis.

In the proceeding sections, the accuracy of the support conditions, loading conditions and material stiffness have been improved, thereby making the quantitative value of the buckling factor more interesting.

When the linear Euler stability analysis is performed for the terminal structure loaded by load combination CO2, dead weight + variable loading, the buckling modes as presented in Figure 4-54 are obtained.

The obtained buckling factor of 3,70 for the lowest buckling mode is lower than the desired value as stated in Section 5.5. The difference between the critical buckling factor of two lowest buckling modes differs 2%, therefore is a buckling value of 6 desired. The buckling safety is insufficient and therefore unacceptable.

From the lower buckling modes can be concluded that most of the curvature in the buckled shapes is found in the centre of the field, where the cross section is thin. Adding additional stiffness at this location would have maximal positive effect on the structure's buckling behaviour. A way additional stiffness can be obtained with increasing the





construction depth is by using the proposed grid stiffened slab, earlier discussed in Section 4.3.

The thickness of the full top layer is set to 200mm, which is strong enough to span a circular hole of three meters, which is the maximal hole size. For the ribbed bottom layer a circle pattern with a  $L_2/L_1 = 0.8/1 = 0.8$  is chosen, see Figure 6-12.

For the sections 1,3,5,6 and 7 which required a full cross section of 400 or 500mm, a grid stiffened slab is used with a slab thickness of 200mm and a grid thickness of 600mm. Section, 3, which required a full cross section of 600 mm, a grid thickness of 800mm is used. See Figure 6-13.

In DIANA, the program used to analyse the structural models, the terminal deck is build like a shell and not a solid. This means that holes or ribs cannot be placed in the shell. This problem is solved by using a shell build up out of multiple layers which in total behave similar to the grid stiffened slab. How a slab element is modelled with properties similar to the grid stiffened slab can be found in Appendix B.

#### **6.2** Strength

In section 5.4 the thickness of the structure was determined in a way that the stresses in the design model did not exceed the maximal allowable compressive stress of 39N/mm<sup>2</sup>. The loading used in the design model had a magnitude of twice the dead weight of the structure. In the previous section, the accuracy of the FE-model has been improved from a design model, used to shape the geometry of the structure, to a preliminary calculation model, used to perform unity checks.

The governing load case for the maximal compressive stress reveals to be CO3: dead weight + unsymmetrical variable load. Figure 6-14 shows the internal stresses in the global axis system on each side of the roof structure. More elaborate results are presented in Appendix F. The maximal compressive stress in the structure has a magnitude of -15,6N/mm<sup>2</sup>, the allowable material stress of 39N/mm<sup>2</sup> is never exceeded. Therefore it can be stated that thickness determined in Sections 5.6 is adequate to withstand all the working forces.







Figure 6-13: Final cross sections



Figure 6-14: Stresses CO3



## 6.3 Stiffness

The initial deformation found is 99mm. This value is obtained by using a geometric non linear calculation method, taking only the dead weight of the structure as loading and using a modulus of elasticity that is not reduced on behalf of creek deformation. The obtained deformation is equal to 1/1520 times the span of 150m.

More interesting is to find out how much the initial deformation increases when the load is increased and the material has been exposed to creep. The total deformation of the structure loaded by the load combinations described in Section 6.1.2 and a reduced modulus of elasticity is 247mm, a difference of 148mm or 149% with the initial deformation.

Compared with the long span the obtained deformations are low. Nevertheless requires all the façade detailing special attention to be able to deform. When these values are compared with the perceptions of Isler it can be stated that the obtained ration between initial and total deformation is acceptable. Isler strives to the smallest deformations possible. Further he states that a difference between initial and long term deformations of about 100 to 150% is fair. Hereby it has to be stated that if the shells of Isler, with small initial deformations, where analyzed in this way comparable values would be found. The differences between initial and total deformation in real life where besides the shape mainly the effect of low quality of concrete.

## 6.4 Stability: Buckling behaviour

After performing a linear Euler stability analysis of the structure with cross sections as presented in Figure 6-13 and loaded by load combination CO2, the buckling factor reveals to be 6,43 for the lowest buckling mode. The buckling value of the second lowest buckling mode is 6,53, a difference of 1,5%, which indicates that the structure will be highly sensitive for imperfections. However the buckling value of 6,43 is high enough to make sure that adequate buckling safety is present.

## 6.5 Conclusion

During this Section, more advanced models were used to study the behaviour of structure. Hereby is studied how the structure responds to loads other than its own weight. From this it can be concluded that, like was assumed during the form finding stage, dead weight is the dominant load case and it is therefore correct that the shape was optimized for this load. Next to this, material and foundation properties were described in more detail in order to obtain more accurate results.

By using a geometric non linear calculation, were the deformed state of the structure is used for defining the internal carrying mechanism, the behaviour of the structure is determined more accurate. However, because of the structures small initial deformations, the 2<sup>nd</sup> order effect is small.

Although all calculations in this Section were performed at a design level and more detailed calculations are required during a detailed design it can be stated that the designed structure can withstand all the concerned loads and has an acceptable deformation.





# **Discussion**

In the past chapters a roof design is obtained which is recommended for a structure in reinforced concrete. When this final roof design is reviewed together with the way that it is obtained, the following questions can asked:

- Does the obtained design comply with the initial architectural concept?
- Is the designed structure safe?
- Has the chosen "design method" been the right one?
- What is the contribution of all the steps in the form finding process?

All these questions will be treated per subject. The obtained design will be analyzed first to find out if it meets the stated requirements and maybe even can be further improved. Secondly, the used "design method" including all the form finding steps which were used to obtain the final design will be reviewed. Finally the accuracy of all the obtained results will be reviewed.





# 7.1 The obtained design

Does the proposed structural solution fulfil all the structural, functional and aesthetic requirements?

### 7.1.1 Summary of the design problem

To give a proper answer to this question it is useful to redefine the starting points of this project.

The Al Ghubaiba is a densely built-up area. The only suitable location for a ferry terminal is on reclaimed land. The free-formed design for the Al Ghubaiba terminal made by Royal Haskoning Architects combines modern architecture with efficient spatial planning. The architect's design concept defines the roof of the Al Ghubaiba terminal as a canopy that is created by carefully lifting the existing quay. The public area hereby continues smoothly over the terminal roof, creating a elevated square. To accentuate this design concept, a slender roof structure is required which contains a transparent façade along the creek site and has a column-free floor plan.

### 7.1.2 Obtained results

The proposed roof structure has a fluent geometry. When the roof structure is smoothly connected to the existing quay it can become public accessible. This innovative spatial arrangement enlarges the public domain in the densely build-up Al Ghubaiba area.

The height, required for a column-free floor plan, is with 15m significantly larger than in the original concept. The roof of the terminal is therefore steeper and less easily accessible. The larger terminal height will limit the view of the posterior buildings towards the creek.

The suggested roof geometry allows for a column free floor plan and does not require any main structural elements in the façade along the creek side. This façade, although not worked out in this report, can be highly transparent. A clear and transparent interior, both highly important for a public transport building, is hereby ensured.

The higher degree of transparency in this design is seen as its utmost point of value.

Summarizing these conclusions it is questionable if the proposed roof geometry still complies with the lifted quay design concept as stated in Section 3.4. The fleunt geometry which allows for a collumnfree floorplan and a highly transparent façade comply with the initial design concept where the larger requiered height is less favorable.

# 7.1.3 Applying the structural shape study results outside this specific project

This graduation report discusses a structural shape study specific for the Al Ghubaiba ferry terminal in Dubai however, the obtained results can also be used outside the scope of this specific project.

The obtained results of this graduation report are the most useful during the preliminary design stage of a project which contains a similar basic shape. Examples of buildings which make use of a similar basic shape can be found in the buildings that will be built for the Dubai ferry terminal project in the future. Plans exist to expand the ferry network in the future to locations further down the creek and to the artificial islands along the coast. This elaborated ferry network requires many new terminal buildings. All these terminals will have about the same appearance but get their own specific design and dimensions. The results of this study prove that for terminals with smaller dimensions, slender self bearing concrete structures are attainable without the need to make them higher than functionally necessary. The obtained minimal ratio between the free height and free span for this type of structure is about 1/12. For a functional desired free height of 6-7 meters, which is common for this type of public transport buildings, a span of 80 meters would be preferable.

The study which was performed to the separate load carrying mechanisms is not project specific. The obtained design recommendations can be used for many types of structures which make use of a combination of different structural systems.

The stability of arches is worked out elaborately in this report. Analytical solutions that are obtained during the years using different approaches are combined and completed with relations valuable for design aspects.

By modifying the geometry of a shell-like structure in a way that in approaches the funicular shape of its load the amount of bending moments can be reduced. The deformation method can be useful to





obtain this improved geometry for shell-like structures. The deformation method can be adjusted for each direction of freedom and is therefore useful for various types of geometries.

This report gives an elaborate overview of how an imaginary deformed geometry and some simple coordinate transformations can be used to generate free-form shapes which may prove to be viable solutions for many structural problems.

The batch files used to build a structural analysis model can be used for all types of geometries that need to be analysed. The batch files transforms a geometry described by an IGES file type to a model which can be used for a structural analysis in DIANA. Appendix D provides detailed information how these batch files can be made and used.

During this project use is made of two structural software applications, Scia ESA PT and DIANA. The suitability of these applications during shape design is reviewed. As stated in Appendix B, both structural software applications have a limited field of application. The obtained knowledge into the suitability of different structural applications during shape design is valuable for other projects than the Al Ghubaiba Ferry Creek Terminal.

## 7.2 Accuracy of results

*Is the accuracy of the obtained results adequate for the current design stage?* 

### 7.2.1 The type of design model

Highly simplified models are desired during the design stage of a project. A good design model should be easy to make, has a short calculation time and requires a minimal accuracy of say, 80%. For structures where load is transferred by a combination of axial forces (arch and shell-action) and bending (beam and plate action) the availability of these simple design models is limited. Simple models which only consider arch action or bending cannot be combined and grid models built using bars instead of surface elements are at least as time consuming as the used shell model. Studying pure shell structures revealed that the change in structural behaviour for a small variation in shape, slenderness, load or support conditions is large. This explains the choice for the used design model which is not as simplified as would be expected.

#### 7.2.2 The obtained accuracy

It may be clear that in a later stage this design requires more advanced calculations to describe the complex behaviour of this structure. The question arises whether with the obtained results like stresses, deformations and buckling values there is sufficient confidence that the designed structure is realizable.

The design model contains 18 sections with individual geometrical (thickness and cross section) and physical (material) properties. This approach leads to local fluctuations in the result pattern between the different sections. On a local scale the obtained forces and stresses will have a rather poor accuracy but for a general scale can be stated that the network of sections is sufficiently dense to provide a good insight into the general interplay of forces. Additional accuracy can be obtained by defining the physical and geometrical properties in more detail.

From FE-models with a coarse mesh, like used in the design model, is known that the obtained deformations are underestimated compared to the reality. See for this subject also the performed benchmark study described in Appendix D. From this benchmark study can be obtained that the accuracy of FE-models differs per geometry. For the used design model, which contains 732 elements can be stated that the obtained deformation lies within 2% of its theoretical value. From this point of view it is not required to use a denser mesh.

This type of structures is multiple statically indeterminate, therefore deviations in stiffness have a mayor effect on the distribution of forces. The stiffness of reinforced concrete is significantly higher for material loaded in compression than for bending or tension. This physical non-linear behaviour of concrete, which differs per direction, is only partly taken into account. To model this behaviour more accurately, FE-elements are required which use the non-linear stress stain relation of concrete in multiple directions.

In position of a grid stiffened slab, the structure is modelled as a layered cross section with modified properties to obtain a similar general behaviour. When this structure is worked out further the use of 3D-solid elements is desired mainly in this

Chapter 7: Discussion



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position. The local stress concentration in the ribs of the grid can be studied more accurately with these 3D-solid elements.

To gain insight into the buckling behaviour of the structure, a linear Euler buckling calculation is performed. This calculation is performed to find out if the stability of the structure is critical. With the finally obtained critical buckling load of 6,43 in combination with design guidelines as presented by Hoogenboom in (Hoogenboom, 2006) can be stated with sufficient confidence that buckling safety is adequate. However it must be stated that for normal building practice a critical buckling load of about 10 is desired. The structure's buckling behaviour should therefore be observed with special attention in succeeding design phases.

### 7.2.3 The used computational model

The used computational model contains a relatively coarse mesh in combination with linear material properties. The large advantage of this is the calculation time, which is less than a minute and thereby extremely fast. This fast calculation time gives the opportunity to analyse many different configurations of shape, thickness and stiffness in a short amount of time. Adapting the material properties in consequence of the obtained internal forces (iterative process) is a time consuming operation which should be improved. The material properties are now determined manually or with help of a M-K program. A scripted cooperation between these two programs would lead to a considerable time-saving.

## 7.3 The form finding method

Has the method to come to this solution been correct?

# 7.3.1 The use of analysis results to find an improved roof geometry

For this project it was clear from the first moment that the distribution of external loads would be done by a combination of axial and bending forces. Structures of this type which also need to be very slender cannot be realized without a good shape. This good shape can be obtained, definitely for complex geometries, only by methods as used in this project. At this moment the cooperation between computational modellers and analyzers is rather limited. Where some of the used methods to improve a geometry needed to be developed and validated the design process was highly time consuming. When the cooperation between computational modellers and analysers will be further improved in the future, the design methods become faster enabling the study of other initial shapes. If more initial shapes can be studied in a short amount of time, the process becomes more design minded. Implementing new form finding applications as the deflection method in structural analysis packages would hereby be a valuable additive.

# 7.3.2 Study load carrying mechanisms separately

Studying different types of load carrying mechanisms separately had two main goals. First, it was used to obtain knowledge used to analyse and distinguish complex results. This goal was met although a lot of trivial information was obtained. The second reason to study moment carrying mechanisms separately was to create some kind of toolbox with way to modify a geometry in a way its structural behaviour improves. The obtained set of geometric modifications where in itself already known, however it was valuable to collect them by category.

### 7.3.3 Parameter shape study

The parameter shape study seems to have been of not much use. The information obtained by varying the parameters of the Orange Peel shape is rather trivial. To make the parametric shape study more valuable a basic shape should be used which can be modified in two directions. From this point of view a translational surface is more serviceable than the used surface of revolution

## 7.3.4 Deflection method

The deflection method was invented to find a shape with a low amount of bending deformation. After the use of the deformation method, the total deformation was decreased by 42%. The amount of bending moments in the structure decreased by 60%. With these results can be stated that the deformation method does what it was designed for. For this project only two iterations were required to come to a converged solution. Due to the fact that multiple programs were used sequentially for each iterative step, the method is still highly labour intensive and therefore time consuming. This could



be improved when all the steps could be connected by a script. Apart from the fact that the deflection method was time consuming it also made the use of DIANA's graphical pre-processor not longer possible. This meant that all the subsequent form finding steps could only be performed with the use of DIANA's input file. The use of an input file instead of a graphical interface led to a needlessly high abstraction level and increased the number of mistakes. A second downside of losing the possibility of using a post processor is that some simple changes in the mesh, like for for instance a local mesh refinenment, ask for a hole new model.

#### 7.3.5 Increasing the structures buckling behaviour

By modifying the geometry steps have been taken to give the design an adequate level of buckling safety. After studying the lowest buckling modes of the initial geometry and the existing structures of amongst other Heinz Isler applying more curvature along the free edge was a logical step. The geometry with the curved free edge had indeed a critical buckling load which was 47% higher than for the initial structure. The method which was used to obtain a smoothly curved free edge is described in Appendix D. This method is highly labour intensive, the use of a 3D modeller like Rhino which interacts with the structural analyser, in this case DIANA, would be valuable.

#### 7.3.6 Modifying physical and geometrical properties

By dividing the total roof into 18 sections it became possible to adapt physical and geometrical properties locally. Hereby a facetted model is obtained which gives proper results on a general scale. Applying layered elements in position of the grid stiffened slab areas works fast. A small downside of these elements is that the internal forces can only be presented per layer instead of for the total cross section.







# **Conclusions** and recommendations

In this Master's thesis, a structural shape study has been performed to obtain a structural solution in reinforced concrete for the Al Ghubaiba ferry Terminal.

The findings of this Master's thesis research will be stated here as conclusions and recommendations for further research.



## 8.1 Conclusions

### 8.1.1 The proposed structure

- This document provides a structural solution in reinforced concrete for the Al Ghubaiba ferry terminal. The main goal of this graduation project is hereby partly achieved.
- The initial geometry of the roof structure needs to be modified to come to viable structural solution. The required height for a feasible column free structure, is with 15m significantly larger than in the original design. This makes the proposed roof structure less easily accessible and therefore less functional as an extended public square. It is herefore questionable if the proposed structure still complies with the initial design concept made by Royal Haskoning.
- The reinforced concrete roof structure is analysed on a general scale. The main aspects like buckling and varying load cases are hereby considered. Additional calculations on a more detailed level are required before the roof structure can be built. However, according to the results of the performed design calculations can be stated that the roof structure contains sufficient strength, stiffness and stability.
- The designed roof structure allows for a column free floor plan. The façade along the creek side does not require any main structural elements. A clear and transparent interior is hereby ensured.

#### 8.1.2 Computational design model

- The soap film form-finding application of GSA can only find a shape of equilibrium based on membrane forces. This application is unfit to study structural shapes which have no funicular geometry.
- The structural software application Scia ESA PT is unsuitable for the analysis of a shell-like structure with complex geometry. The lack of insight into the methods the program uses to obtain its results and the insufficient ways to describe a surface make the application

unfit to analyse design models of shell-like structures

- The structural software application DIANA is suitable for the analysis of shell like structures with complex geometries. DIANA may not have a user-friendly interface but its capabilities to read graphical files and spreadsheets make the program fit to analyse design models of shell-like structures.
- The deformation method is able to approach the geometry with minimal bending deformation for a shell-like structure.
- The use of a imaginary deformed geometry and some simple coordinate transformations can be used to generate free-form shapes which may prove to be viable solutions for many structural problems.

### 8.1.3 Miscellaneous

- The categorized set of geometric modifications obtained by analyzing specific load carrying mechanisms can be used to improve the structural behaviour of a complex design.
- The results for models with only a gravity load or a more elaborate load combination do not deviate significant. To base the structures shape on a dead weight load distribution has proven to be both practical and effective.
- Analytical solutions exist which provide the critical buckling load of an arch. In this report a relation is obtained between the height, span and thickness of a stable arch. The obtained relation can be used as starting point for the design of arch-like structures.
- An iterative design process is needed to obtain a design where geometry, required thickness and load distribution are all optimized in relation to each other.



## 8.2 Recommendations

### 8.2.1 The proposed structure

- The foundation of the roof structure has a large influence on the flow of forces. The influence of partial deformation on the flow of forces in the roof structure should be analysed further.
- The behaviour of the roof structure is analysed for eight different load cases. However, more elaborate studies are required to obtain full insight into the structural behaviour under differing load combinations.
- Proper detailing is crucial to give the roof structure its desired appearance. The connection between the roof and the existing quay and the detail of the curved edge need to be worked out in cooperation with the architect.

#### 8.2.2 Computational modelling

- Further testing of the deformation method is advisable. A method to determine the largest possible deformation factor γ would further reduce the amount of iterative steps required to obtain a converged result.
- Three separate programs are required to obtain and analyze an improved geometry during each iterative step of the deformation method. This approach makes the deformation method highly labour intensive, time consuming and therefore not ideal for shape design. By the use of scripting the deformation method can be automised and therefore become a useful form finding tool in structural software applications.
- By using an imaginary deformed geometry and some simple coordinate transformations the structural analysis application DIANA can be used for shape design. This type of software is commonly used for other purposes, therefore the possibility to export an obtained geometry and use it in other applications is limited. A method which converts the numerical geometrical information used by DIANA into a graphical representation which can

be read by a modelling application would make DIANA more serviceable during shape design.

 A design tool which uses an iterative approach to find a design where geometry, required thickness and load distribution are all optimized in relation to each other would be very valuable during the design stage of a shell like structure.



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Appendix

## Curve and Surface geometry

# A.1 Surfaces used for shape design

Describing the geometry of surfaces is a wide field of techniques. Many techniques are available for defining and describing surfaces in many forms. Often used techniques in the field of architecture and structures are: (Coenders, 2006)

- Geometrical functions
- Generated surfaces
- NURBS: Non-uniform rational B-Splines

Curves and surfaces can be derived directly from geometrical functions. Often a closed form or parametric form is used. Often simple functions can be used to create seemingly complex geometrical structures. See Figure A-1.



Figure A-1: Geometrical function, Image from ocw.mit.edu.

Surfaces of revolution are generated by the revolution of a plane curve, called the meridional curve, about an axis, called the axis of revolution.

See Figure A-2. The results of revolution-developed surfaces are: conical shells, circular domes, paraboloids, ellipsoids of revolution, hyperboloids of revolution of one sheet, and others. In the special case of cylindrical and conical surfaces, the meridional curve consists of a line segment. The parametric description of the general form of a surface of revolution is:

 $x1 (u,v) = \emptyset (v) \cos u$  $x2 (u,v) = \emptyset (v) \sin u$  $x3 (u,v) = \psi (v)$ 



Figure A-2: Surface of revolution, Image from Mathworld.Wolfram.com

Surfaces of translation are generated by sliding a plane curve along another plane curve, while keeping the orientation of the sliding curve constant. The latter curve, on which the original curve slides, is called the generator of the surface. Translating any spatial curve (generatrix) against another random spatial curve (directrix) will create a spatial surface as presented in Figure A-3. In the special case in which the generator is a straight line, the resulting surface is called a cylindrical surface.



Figure A-3: Surface of translation, Image from Mathworks.com

Ruled surfaces are generated by sliding each end of a straight line on their own generating curve, while remaining the straight line parallel to a prescribed direction or plane. The generated straight line is not





necessarily at right angles to the planes containing the generating director curves. The parametric description of the general form of a rules surface is:

 $xi(u,v) = bi(u) + v\delta i(u)$ 

A special type of a geometrical function definition which is often used for structures, are NURBS. NURBS, 'Non-Uniform Rational B-Splines', are mathematical representations of n-D geometry that can accurately describe any shape from a simple 2-D line, circle arc, or curve to the most complex 3-D organic free-form surface or solid. Often people tend to believe that NURBS are 'random' curves, without any mathematical description. The unique geometrical description of a NURB curve is discussed in the following section.

#### A.2 **Curves used for shape design**

Back in the days before computers, architects, engineers, and artists would draw their designs for buildings, roads, machine parts, and the like by using pencil, paper, and various drafting tools. These tools included rulers and T-squares for drawing straight lines, compasses for drawing circles and circular arcs, and triangles and protractors for making precise angles.

Of course, a lot of interesting-shaped objects couldn't be drawn with just these simple tools, because they had curved parts that weren't just circles or ellipses. Often, a curve was needed that went smoothly through a number of predetermined points.

As a solution one often would employ long, thin, flexible strips of wood, plastic, or metal, called splines. The splines were held in place with lead weights, as shown in Figure A-5. The resulting curves were smooth, and varied in curvature depending on the position of the ducks. As computers were introduced into the design process, the physical properties of such splines were investigated so that they could be modelled





mathematically on the computer.

Bezier curves are named after their inventor, Dr. Pierre Bezier. Bezier was an engineer with the Renault car company and set out in the early 1960's to develop a curve formulation which would lend itself to shape design. A type of Bezier curves normally used for computer aided design is the Rational Cubic Bezier curve. A cubic Bezier curve is determined by four control points, Po(xo,yo),  $P_1(x_1,y_1)$   $P_2(x_2,y_2)$   $P_3(x_3,y_3)$ , and is defined by the parametric equations:

$$x = x_0(1-t)^3 + 3x_1t (1-t)^2 + 3x_2t^2 (1-t)^2 + 3x_2t^2 (1-t)^2 + x_2t^2 (1-t)^2 + x_2$$

 $y = y_0(1-t)^3 + 3y_1t(1-t)^2 + 3y_2t^2(1-t)^2 + 3y_2t^$ t + v3t3

Where  $0 \le t \le 1$ . Notice that curve starts at P<sub>0</sub> and ends at P3 and the curve is tangent to P0-P1 and P<sub>3</sub>–P<sub>2</sub>. These properties make Bezier curves an intuitively meaningful means for describing freeform shapes. (Stewart, 2003)

For quadratic Bezier curves one can construct intermediate points Q and P such that as t varies from 0 to 1:

- Points Q<sub>0</sub>, Q<sub>1</sub> and Q<sub>2</sub> vary from P<sub>0</sub> to P<sub>3</sub> and describe a linear Bezier curve.
- Point Ro and R1 vary from Q0 to Q2 and describe a quadratic Bezier curve
- Point B(t) varies from Ro to R1 and describes a cubic Bezier curve.



Figure A-4: Construction of a Bezier curve

For higher-order curves one needs correspondingly more intermediate points.

The rational Bezier adds adjustable weights to provide closer approximations to arbitrary shapes.

Another curve often used for shape design is the NURB curve. One of the advantages of NURB curves



is that they offer a way to represent arbitrary shapes while maintaining mathematical exactness and resolution independence. Among their useful properties are the following: (Rhinoceros, 2007)

- NURB curves can represent virtually any desired shape, from points, straight lines, and poly lines to conic sections (circles, ellipses, parabolas, and hyperbolas) to free-form curves with arbitrary shapes.
- A set of control points and knots, which guide the curve's shape, can be directly manipulated to control its smoothness and curvature.
- NURB curves can represent very complex shapes with remarkably little data.

NURB curves can be used as a tool to design and control the shapes of three-dimensional surfaces, for purposes such as:

- surfaces of revolution (rotating a twodimensional curve around an axis in threedimensional space)
- extruding (translating a curve along a curved path)
- trimming (cutting away part of a NURB surface, using NURB curves to specify the cut)

A NURB curve can be seen as a parametric function,

with respect to t, where t is a parameter representing time. When evaluating this function at a number of values of t, one gets a series of (x, y) pairs that can be used to plot the curve, as shown in Figure A-6.

One of the key characteristics of NURB curves is that their shape is determined by the positions of a set of points called *control points*, like the ones labelled Bi in Figure A-7.

The second curve in Figure A-7 is the same curve, but with one of the control points (B7) moved a bit. Notice that the curve's shape isn't changed throughout its entire length, but only in a small neighbourhood near the changed control point. This is a very desirable property, since it allows to make localized changes by moving individual control points, without affecting the overall shape of the curve. Each control point influences the part of the curve nearest to it but has little or no effect on parts of the curve that are farther away. At any time t, the particle's position will be a weighted average of all the control points, but with the points closer to the particle carrying more weight than those farther away. The function which determines how strongly control point Bi influences the curve at time t, is called the basis function for that control point.

Figure A-8 shows a typical example of what a basis function might look like: it has its maximum effect at some definite point in time and tapers off smoothly as it gets farther away from that point.





Figure A-8: Basis function for a control point

Since each control point has its own basis function, a NURB curve with, say, five control points will have five such functions, each covering some region of the curve (that is, some interval of time). By giving an exact specification to the basic functions we can get certain desirable effects.

A number of transformations can be applied to a NURBS object. For instance, if some curve is defined using a certain degree and N control points, the same curve can be expressed using the same degree and N+1 control points. In the process a number of control points change position and a knot is inserted in the knot vector. These manipulations are used extensively during interactive design. When adding a control point, the shape of the curve should stay the same, forming the starting point for further adjustments. A number of these operations are discussed below. (Schneider, 1996)

#### Knot insertion

As the term suggests, knot insertion inserts a knot into the knot vector. If the degree of the curve is n, then n - 1 control points are replaced by n new ones. The shape of the curve stays the same. A knot can be inserted multiple times, up to the maximum multiplicity of the knot.

#### Knot removal

Knot removal is the reverse of knot insertion. Its purpose is to remove knots and the associated control points in order to get a more compact representation. Obviously, this is not always possible while retaining the exact shape of the curve. In practice, a tolerance in the accuracy is used to determine whether a knot can be removed.

#### Degree elevation

A NURBS curve of a particular degree can always be represented by a NURBS curve of higher degree. This is frequently used when combining separate NURBS curve or when unifying adjacent curves. In the process, the different curves should be brought to the same degree, usually the maximum degree of the set of curves. The process is known as degree elevation.

The relation between a Bézier curve and a NURB curve is that Bézier curves can be viewed as a subset of NURB curves. Any Bézier curve can be represented by a particular type of NURB curve, having half its knots at one end and half at the other. The converse, however, isn't true: an arbitrary NURB curve can't, in general, be represented by a single Bézier curve. In fact, it generally requires several Beziers to represent a single NURB curve: one for each distinct segment of the curve, as defined by its knot vector. Recall that each segment of a NURB curve is affected by some subset of the control points. If at each segment knots are added at both ends, generating a new set of control points each time, until each end has a number of knots equal to the order of the curve, the result will be a Bézier representation of that particular segment. When this is done for each segment, the series of Bézier curves, taken together, look exactly like the original NURB curve.

Appendix A: Curve and Surface geometry





Appendiy B

# A suitable analysis application

#### **B.1** Functional requirements

Many computational analysis applications exist to study structural problems. All these applications have their own limited field of application. Computational analysis applications can in general be divided in three groups namely, 2D, 2,5D and 3D programs. See Figure B-1. 2D programs can analyze 1D bar-elements in a 2D space. 2,5D programs can analyze 2D plane-elements in a 3D space and 3D programs can analyze solids. Most of the calculations performed in the construction industry have a low to medium level of complexity and can be solved by 2D and 2.5D applications. Because more complex models take more time to work with, there has to be a specific reason to use them. Complex geometry, non-linear behaviour or the overall scale can be reasons to choose a more complex model.

During this thesis a computational analysis application is desired which is able to study structural design models of shell-like structures. The requirements for a design model are:

- Fast to build
- Can describe each type of geometry
- Short calculation time

The accuracy of results is for a design model of less importance all dough the obtained results may not deviate more than  $\pm 20\%$  of the exact values. The additional requirements for shell like structures is that the application needs to make use of mesh elements with adequate complexity to describe the behaviour of a double curved shell. Finally the application should give full insight into the method the model uses to obtain its results. Both Scia ESA PT and DIANA are studied to find out which application meets these requirements best.

#### B.2 Scia ESA PT

Scia ESA PT is a 2.5D element program that is mainly used in the construction industry. The strength of the program is that is easy to use, is very fast and can check structure direct on a specific national design code. The program contains a build-in pre-processor for modelling purposes, a calculation section to analysis the models, and a post-processor to present the results.

#### B.2.1 Pre-processing

The geometry of a structure can be modelled in the graphical interface or by importing a DXF file.

For simple beam and plate structures the graphical interface contains sufficient modelling tools. For more complex geometries like double curved shells the graphical interface lacks the required modelling tools present in a proper computational modelling application.

By importing graphic DXF files lines can imported and used to generate beam, plate or shell elements. ESA is able to import straight lines, circular arcs, elliptic arcs and quadratic splines. Parabolas and rational cubic Bézier curves are supported by the program itself but cannot be imported.

The different shell surfaces that can be modelled in ESA are restricted to geometries which are fully described by their boundaries. This means that not all surfaces can be described. Splitting the surface in smaller parts is a way to describe a surface more precise but diminishes the speed of modelling.

Where mesh generation is concerned ESA uses a build-in mesh generation algorithm that can be controlled by a set of parameters. However, the control over the generated mesh is limited.





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#### **B.2.2** Calculation

Scia ESA PT uses flat mesh elements for its analysis. There are some doubts about the correctness and accuracy of these flat mesh elements when they are used to model double curved surfaces.

It is very hard to find all the analysis procedures and mesh properties the program uses to perform its calculations. This lack of insight causes that the program suffers from a 'black box' problem. The lack of insight results in a lack of confidence in the results.

A benchmark study is performed to check the correctness and accuracy of ESA PT. In the first part of this study three shell models are made. For each of these models the displacements found by the finite element program are compared with the reference results. The number of mesh elements that are required to find a certain accuracy of results give some information about the level of intelligence of the used mesh elements. The benchmark that are performed are:

# $E = 3 \ 10^6 \ \text{N/mm}^2$ $z \quad v = 0.3$ $a = 300 \ \text{mm}$ $t = 3 \ \text{mm}$







Geometry: A cylinder closed on both ends by a diaphragm that is loaded by opposite forces. See Figure B-2 The deformation is measured directly under the forces. The reference deformation is 1,8248mm.

In Figure B-3 the deformation and the result accuracy is plotted for different average mesh sizes.

A guideline for this benchmark test states that about 1000 elements are needed to obtain an accuracy of 1%. ESA PT needs over 8000 elements to come to an accuracy of 1,8%.

#### Benchmark test 2

Geometry: A hemisphere that is loaded by opposite forces as presented in Figure B-4. The deformation is measured directly under the forces in y-direction. The reference deformation is 92,4mm. A guideline for this benchmark test states that approximately 200 elements should be needed to obtain 1% accuracy.









**T**UDelft

#### Benchmark 3

Geometry: A hemisphere with an opening that is loaded by opposite forces as shown in Figure B-6.. The deformation is measured directly under the forces in y-direction. The reference deformation is 93,5mm. A guideline for this benchmark test states that approximately 100 should be needed to obtain 1% accuracy.

The fault magnitude of a linear element is in the order of the mesh size h. Squared or cubic elements have fault magnitude in the order of  $h^2$  or  $h^3$ . The fault can be determined by the formula:

$$0 = \frac{u - \tilde{u}}{u} \cdot 100\%$$

Where u is the reference displacement and  $\tilde{u}$  is the found displacement.

The linear relation between the average mesh size and the displacements found for benchmark 1 give the idea that ESA PT works with an linear, relative simple, type of mesh elements. The final deformation for a model can be predicted by the formulas:







Linear elements:  $O(h) \rightarrow u = 2\tilde{u}_2 - \tilde{u}_1$ 

Square elements:  $O(h^2) \rightarrow u = \frac{3}{4}\tilde{u}_2 - \frac{1}{3}\tilde{u}_1$ 

Cubic elements: 
$$O(h^3) \rightarrow u = \frac{8}{7}\tilde{u}_2 - \frac{1}{7}\tilde{u}_1$$

Where u is the final displacement,  $\tilde{u}_1$  is the found displacement with a certain mesh and  $\tilde{u}_2$  is the found displacement with a mesh twice as fine as  $\tilde{u}_1$ . (Hoogenboom, 2007)

When all these formulas are used and compared to the reference displacement of 1.8248mm for benchmark 1 it can be found that the formula for linear elements fits best. Benchmarks 2 and 3 show no linear relation between the mesh size and the displacement and therefore do not comply with the assumption of the use of linear elements.

The accuracy of results that are produced by ESA PT are maybe pour compared to guidelines but are sufficient for the design models used in this thesis. Because of the discrepancy between the different benchmarks the question which type of mesh elements ESA PT uses remains unanswered. The lack of insight remains also after the benchmark study, therefore full confidence in the results is not obtained.

#### **B.2.3 Post-processing**

ESA has a power full post processing interface. Internal forces, stresses and deformations can be presented graphical and in tabular form. Basic result magnitude with respect to a global or a local axis system. Principle moments and normal forces can be presented by means of contour plots or vector plots.

#### B.3 Diana

Diana is a multi-purpose finite element program (three-dimensional and nonlinear) with extensive material, element and procedure libraries. Diana is used for complex projects and has proven to produce accurate results. Diana itself is just a solver of a system of equations of a finite element program. Diana can be operated by pre-processors, or batch-commands. The information about the different mesh elements and analysis procedures is widely available.



#### **B.3.1 Pre-processing**

DIANA is a program that is mostly used for analysis instead of design. When the geometry often changes during the design phase the use of batch commands can save a lot of time. All the steps in the structural analysis, from building the geometry to making the graphics of the results can be made by batch commands. Another way to make quick geometry changes is by importing the geometry data from a 3D modeller. Curves or even meshes that are build in programs like Rhinoceros or Maya can be imported to Diana with IGES files. Because the IGES format contains all the information of a shape, the Diana model is very precise. The relation with Rhinoceros provides a lot of control over the surface

#### **B.3.2** Calculation

DIANA provides a large number of methods to analyse structural problems. All the methods the program uses to obtain its results are well documented. This gives confidence in the results that are obtained.

#### **B.3.3** Post-processing

iDIANA can be used as a post processor to present internal forces, stresses and deformations graphical. Basic result magnitude with respect to a global or a local axis system. Principle moments and normal forces can be presented by means of contour plots or vector plots. Excel can be used to present the analysis results in tabular form.

#### **B.4** Conclusion

Both the programs have some strong and some weak points when they are compared on modelling speed, shape possibilities and outcome accuracy. During the design phase it is wise to make a distinction between simplified models and the analysis of the total geometry. For simplified models that are analyzed linear ESA PT can be well used, but for non-linear analysis or full geometric models it is wise to use Diana. This conclusion could have been made faster when the experiences about the shortcomings of ESA PT where taken more serious.





Appendix C

## The Orange Peel design model

#### C.1 Finite element models

During the shape study and the subsequent optimization cycles, use is made of computational models based on the finite element method. The finite element method is a numerical computer analysis and is, today, widely available in advanced analysis applications. Basically, the finite element method is a numerical approximation method which involves the construction and solution of a matrix system from partial differential equations which describe the behaviour of an idealized structure. The name 'finite element' is derived from the spatial discretization of a continuous domain, represented by a mesh of elements, polygonal regions limited by nodes, in which the behaviour of the structure is captured. The finite element method is well suited to computer implementation as constructing the (large) systems of linear equations and subsequent performing a solution can be automised.

The implementation of the model in computational software is often aided by pre-processors offered in combination with the finite element program. The finite element model consists out of the discretized geometry of the structure by a mesh, the physical properties and the loading and boundary conditions (such as supports). Mesh generation usually is done by computational software which is part of a pre-processor. Furthermore, a postprocessor presents the results of the analysis by graphical methods such as a contour plot. In this graduation project, the finite element analysis program DIANA is used, developed by Dutch company TNO Delft. The program is equipped with iDIANA pre- and postprocessor.

The modelling can be divided into geometrical modelling and physical modelling. The geometrical modelling deals with the shape of the structure which is discretized by a mesh. Furthermore, the boundary conditions and external applied loads are modelled here. The physical modelling deals with the material properties and characteristics.

During the design stages of this project, a great number of geometries are designed, analysed and modified. This way of working, which is significantly different from an approach where a fixed geometry is studied and dimensioned, calls for specific use of computational models. This section describes the way the design model for the Orange Peel was constructed. A detailed description of the Orange Peel shape can be found in Section 3.6. The geometric modifications that have been made in the various form finding phases and the way this had influence on the design model in described in Appendix D.

#### C.2 Geometrical modelling

The implementation of a structural domain into a finite element program starts with the definition and discretization of the geometry. As the primary objective of a model is to realistically replicate the important parameters and features of the real design. The geometry of the computational model must approach the geometry of a designed shape in the most optimal way. Hereby, it must be mentioned that there is no such thing as a perfect numerical model. The model always contains numerical irregularities and, thus, is imperfect. Furthermore, as mesh generation is part of geometrical modelling, the discretization of the geometry must result in a correctly described structural response to external loads. After the geometry and mesh, the material and physical properties must be defined and at the end, the boundary and loading conditions. The DIANA topdown approach means that the geometry is defined first and then the mesh. To define the geometry knowledge of computer aided design is required. For the discretization the user needs knowledge of applied mechanics and finite elements in particular.

#### C.2.1 Geometry

The geometry of the orange peel is created in a computer aided design programs and then imported by the pre-processor of DIANA. This





method is required because the computer aided design programs foresee in more advanced design aids than the pre-processors. However, problems arise when converting the imported model as in general the programs are based on different codes.

The geometrical modelling for the Orange Peel shape is done with the computer aided design program Rhinoceros. The designed surface model in Rhinoceros was made by the intersection to two planes and a torus. See Figure C-1. By converting the designed surface to a IGES-file, which can be imported in iDIANA, the problem arose that not the trimmed surface was translated but a combination of the full surface of revolution and the trimming curves. To obviate this problem and to make sure the trimmed surface geometry was exported adequately, a line grid following the designed surface was made in Rhinoceros. By assuring the density of the line grid, the difference between the designed geometry in Rhinoceros and iDIANA could be kept at a acceptable level.

The line grid is designed in a way that all the faces between the lines are smooth triangles and rectangles. The line grid is obtained by combining two half grids. For the outer parts of the deck a



Figure C-2: plane intersections side view [a], top view [b], line grid outer parts [c] total line grid [d]

triangular grid, obtained by the intersections of planes adjacent to the surface edges is used. See Figure C-2 [a&b]. For the central part of the deck a rectangular grid obtained by a division strategy of the edges is used. See Figure C-2 [c]. The use of this specific line grid, which generation is timeconsuming, seems a bit awkward but the benefits arise during the generation of the mesh.

It has to be stated that on account of visibility issues, a extra coarse mesh is shown in Figure C-2.

In iDIANA, all the faces of the line grid were filled up with surface elements. The generation of surface elements led to flipped local coordinate systems between adjacent surfaces, therefore а transformation was performed to get all the positive local Z-axis in the side of the generated shape. The definition of the geometry is performed in the Design Environment. The model is parametric as lines, surfaces and bodies can be defined from points and transforms (transformation of an original part to a new part). The nodal points of the geometry are stored in a data file in terms of a global coordinate system.

#### **C.2.2 Mesh Generation**

A finite element mesh that correctly describes the structural behaviour of a structure is as complicated as it is important. Obviously, as the mesh includes





elements, defined as the ratio of the length and the width of the element, must not be too large. A large aspect ratio leads to convergence problems and has a bad accuracy. To correctly and efficiently describe the boundary, a sufficient number of nodes must be present at the edges.

iDIANA contains a mesh quality test to evaluate the generated mesh. In general, the mesh quality test evaluates the shape of the element with respect to a theoretical ideal. The criteria involved, such as element angles, warping, aspect ratio, and mid-node position, depends on the selected element type. The mesh can be generated by hand (for simple problems) or by a mesh generator.

Before the mesh can be generated a generic element type must be chosen. A generic element only describes the shape of the element and the number of nodes. With the mesh technique and the generic elements, the density of the mesh can be controlled in various ways. DIANA offers the division of lines, surfaces and bodies or to specify the length of an element.

For meshing complex geometries iDIANA offers multiple meshing algorithms and generators. However, because the line grid that was imported to describe the models geometry consist out of smooth triangular and rectangular faces, the mesh in every face can be generated easily with a division function.

#### C.2.3 Element Types

For the discretization of the structural domain, DIANA offers a wide variety of (generic) element types to specify the mesh. The program has a library with almost 200 elements. Each element is programmed for a particular problem. For structural analysis, the elements can be divided into line elements, surface elements, volume elements and the interface elements which are used for contact boundary problems. The choice for a particular element type largely depends on the nature of the structural problem. Theoretically, for the most optimal description of a double-curved shell like structure (a volume), volume elements are to be chosen. However, volume elements are of little practical use as they have the tendency to produce very large systems of equations, leading to exceptional long computational time. Their usage is restricted to relatively small structural elements or parts of a larger structure. Fortunately, the threedimensional thin shell problem can be reduced to a two-dimensional surface problem by applying the thin shell assumptions. Hence, the shell can be modelled by using two-dimensional surface elements. The two-dimensional surface elements offered by DIANA are based on isoparametric degenerated-solid approach (by introduction of the thin shell assumptions) which means that the stress component normal to the shell surface is equal to zero (plane stress assumption) and that a normal remains straight after deformation.

The elements are named after their usage. For example, the surface element QU8-CQ40S, where QU8 stands for an 8 noded quadrilateral shaped element and CQ40 specifies the belonging to a curved quadrilateral with 40 degrees of freedom. The S at the end stands for a regular shell element. The boundaries of the element are pointed out by nodes. The vertex nodes indicate the corners of the element and, depending on the type of element that is chosen, there are zero, one or more interior points at the element edge or even midnode inside the element. Aforementioned, every node has 6 degrees of freedom, 3 translational and 3 rotational, and just as many corresponding load vectors. However, in the case of surface elements, the out-of-plane rotation is not included. Hence, there are (5 x 8 =) 40 degrees of freedom in a QU8 element. This can be seen in Figure C-3.

The number of nodes determines the order of the shape functions. Normally, 6 noded triangular or 8 noded quadrilateral elements with quadratic shape functions are appropriate for shell analysis. During the parametric shape study, use is made of , triangular TR6 CT30S elements, rectangular QU8-CQ40S elements and BE3 CL18B beam elements.



Figure C-3: QU8-CQ40S Curved shell element and its degrees of freedom







### mouncations

Adapting geometry

**D.1** 

During the shape study, geometric changes are processed by the computer aided design program. Every new geometry is imported in a new structural model, the additional geometric and physical modelling steps are formulated in a batch file. A batch file is a input data file in a text-format which can be produced and modified with any convenient text editor, for instance NOTEPAD. In a batch file consists of a list of subsequent commands that otherwise have to prompted one at the time. During the shape study stage one batch file was made that included the following:

- Importing the graphic file
- Generation of the surface elements
- Arrange the local coordinate systems
- Choosing the type of mesh-elements
- The division of the surface elements for mesh generation
- Generation of the mesh
- Specifying material properties
- Specifying physical properties to the mesh
- Generation of the loading conditions
- Generation of the support conditions



Figure D-1: Used modelling and analysis scheme during the shape study

By using this approach one can create a new geometry in a modelling application and by running a single script convert it to a full structural model ready to be analysed. This batchfile is discussed in more detail in Appendix D.2.1 To analyse the model made in iDIANA, a input file for DIANA is exported. This input file is a text file that contains all the physical and geometric information that is required for the calculation. The input file is further discussed in Appendix D.2.2. Another file that is imported in DIANA is the commandfile. This file tells DIANA how to analyze the model. The results from the analysis can be exported trough tabulated output or an output file which can be used by iDIANA for graphic output. See Figure D-1. The commandfile is discussed in more detail in Appendix D.2.3

#### **D.1.1 Deformation method**

By the use of the deformation method, geometric changes are made according to the obtained mesh deformation of the initial structure under loading. The main difference here is that for the shape study a geometry is made and adjusted with a design program as a basis for a generated mesh and that for the deformation method geometric changes will be made to the mesh itself. This means that the





Appendix D: Geometric modification



input file where the mesh has to be adjusted instead of the geometry in the pre-processor. The initial input file with the position of all the mesh nodes and the tabulated output file with the deformation of the mesh nodes serve as data for a coordinates transformation in Excel. This leads to a new input file with a adjusted mesh shape that can be analyzed in DIANA. In this way several subsequent iterations can be made and presented in a graphic way by iDIANA. See Figure D-2.

#### D.1.2 Improve buckling behaviour

The free edge is curved improve the structures buckling. The number of meshnodes along the free edge is so large that adapting their position by hand was no option. To overcome this problem the initial geometry was loaded in a way that the deformed shape was equal to the desired geometric modification. In case of the curved free edge the initial model was adapted the following, see Figure D-5.

- The E-modulus along the free edge was decreased to concentrate all the deformation.
- The free edge was loaded by a bending moment around the free edge.
- Additional supports were added in at 5 meter from the free edge to maintain the position of the mesh

#### D.1.3 Dimensioning model

During the local optimizing and the dimensioning stage not the geometry itself has to be changed but manly the physical and material properties. To be able to make local adjustments to the input properties, the surface of the deck is divided into



Figure D-5: Creating a new geometry by the use of an imaginary deformed structure

sectors as shown in Figure D-3. By using different sectors it becomes possible to model elements with different thickness and stiffness. To model more accurate boundary conditions, horizontal translation and rotation springs are used which simulate the behaviour of an actual foundation.



Figure D-3: Advanced structural model

Modifications during the local optimizing and the dimensioning stage are directly done in the inputfile. The inputfile consists of tables and parameters. The tables can be imported into excel, modified, exported to a space-separated textfile (.prn) and paste into the original inputfile. The parameters of the inputfile can be directly modified by using Notepad. See Figure D-4.



Figure D-4: Modify inputfile

#### D.1.4 Modelling the grid stiffened slab

In DIANA, the terminal deck is build like a shell and not a solid. This means that their cannot be placed holes or ribs in the shell as in a solid model would be possible. This problem is solved by using a shell build up out of multiple layers. The top layer will react as the solid part of the roof, the bottom layer will be given properties which describe the beamgrid behaviour. Instead of making direct holes or ribs, the bottom layer is built of a material with a lower density. Also the other material properties have to be considered, a layer with holes is of course not as strong as layer without holes.



In it is shown how the material properties change, or which properties that change and which are the same. The density and the modules of elasticity are the properties that will have a lower input value in the FE-program. For the density the ratio between the grid area and the full area is used. For the module of elasticity the ratio between the stiff area, where the primary reinforcement will be placed, and the full area is used. The found stresses in the bottom layer will have to be increased by the same factor the module of elasticity is decreased.

To study the effectiveness of the ribbed bottom layer a circle pattern with a  $L_2/L_1 = 0.8/1 = 0.8$  is chosen. The density constants  $c_1$  and  $c_2$  as shown in Figure D-7 and Figure D-8 results in:

$$c_1 = \frac{Area_{full}}{Area_{grid}} = 2,38$$
;  $c_2 = \frac{Area_{full}}{Area_{stiff}} = 2,78$ 

$$\rho^* = \frac{\rho}{c_1}$$

$$L = L^*; \quad \Delta L = \Delta L^*; \quad F = F^*$$

$$\Delta L = F \cdot k = F \cdot \frac{L}{EA}$$

$$E \cdot A = E^* \cdot A^*$$

$$A^* = c_2 \cdot A; \quad E^* = \frac{E}{c_2}$$

$$\sigma^* = \frac{F^*}{A^*} = \frac{F^*}{c_2 \cdot A} \implies \sigma = c_2 \cdot \sigma^*$$

^

A: Area of material (without holes)
E: Module of elasticity
L: Length
F: Force
σ: Stress
k: Spring constant
c<sub>1</sub> : Area density constant
c<sub>2</sub> : Stiff area density constant
ΔL: Axial deformation



**Figure D-6: Fictive material properties** 







#### D.2 Scripts and batchfiles

When a large number of geometries need to be analyzed it pays off to use batchfiles and scripting to atomize part of the process. In this section, the used batchfiles and inputfules used to model, modify and analyze the finite element models are discussed.

#### D.2.1 The batchfile used for pre-processing

All the commands below form together the batchfile that is created for the generation of the structural design model of the Orange Peel basic design. All the commands are written in Plain text Format.

The commands below import the graphical IGES file with the name mesh.igs, stored in the same folder as the batch file into iDIANA. The subsequent commands state that the DIANA input file (.dat) will get the name Mesh and determine precision that has to be used while importing the IGES file.

```
UTILITY READ IGES mesh.igs
mesh
.1
N
Y
.5E-1
25
Y
FEMGEN mesh
```

The commands below state that the general settings for DIANA will be set to a structural problem in a 3D environment and determine the Units that will be used.

```
PROPERTY FE-PROG DIANA STRUCT_3D
yes
UTILITY SETUP OPTIONS GRAPHICS-W ROTATIONS MODEL
UTILITY SETUP UNITS LENGTH MILLIMETER
UTILITY SETUP UNITS MASS KILOGRAM
UTILITY SETUP UNITS FORCE NEWTON
UTILITY SETUP UNITS TIME SECOND
UTILITY SETUP UNITS TEMPERATURE KELVIN
```

The next command tells iDIANA to create surface elements between all the lines from the imported IGES model.

GEOMETRY SURFACE AUTOMATIC

The generation of surface elements leads to flipped local coordinate systems between adjacent surfaces, by the next command a transformation is performed to get all the positive local Z-axis in the same side of the generated shape.

```
GEOMETRY FLIP FOCAL ALL
geometry flip all
DRAWING DISPLAY
```

The model contains of multiple sets of lines that are used for the positioning of the supports the loads and the edge beam. The next commands generate the sets for the line of symmetry ,setname top, and the outside line of support, setname bottom, by selecting all lines with specific coordinates.

```
CONSTRUCT SET OPEN bottom
CONSTRUCT SET APPEND LINES LIMITS VMIN 65000
CONSTRUCT SET CLOSE
CONSTRUCT SET OPEN top
CONSTRUCT SET APPEND LINES LIMITS VMAX 1000
CONSTRUCT SET CLOSE
CONSTRUCT SET OPEN side
```







view geometry all

The sets for the beam along the free edge and the line of support at the quay side cannot be selected by coordinates. The watch this command stops the batchfile from processing and allows the user to select the lines manual.

!WATCH THIS 0# 1 CONSTRUCT SET APPEND LINES CURSOR RIGHT 0# 1 CONSTRUCT SET APPEND LINES CURSOR RIGHT CONSTRUCT SET CLOSE CONSTRUCT SET OPEN BEAM CONSTRUCT SET APPEND LINES CURSOR LEFT @# 1 CONSTRUCT SET APPEND LINES CURSOR left Q# 1 CONSTRUCT SET CLOSE CONSTRUCT SET OPEN surface CONSTRUCT SET APPEND ALL CONSTRUCT SET REMOVE beam ALL CONSTRUCT SET CLOSE DRAWING DISPLAY

The Mesh division command states that that all the sides of the surface elements will be divided in 2, thereby creating 3 points on each surface element. Each surface elements has therefore the same size as a mesh element. The Meshing type commands choose the type of mesh elements that will be used during the mesh generation.

MESHING DIVISION LINE ALL 2 MESHING TYPES BEAM BE3 CL18B MESHING TYPES 3SIDES TR6 CT30S MESHING TYPES 4SIDES QU8 CQ40S MESHING GENERATE VIEW OPTIONS SHRINK MESH .8 VIEW MESH CURRENT

The next command contain the physical and the material properties of the mesh.

PROPERTY MATERIAL MA1 ELASTIC ISOTROP 10000 0.2 PROPERTY MATERIAL MA1 MASS DENSITY 2.4E-6 0 PROPERTY PHYSICAL PH1 GEOMETRY CURVSHEL THREGULR 1000 PROPERTY PHYSICAL PH2 GEOMETRY BEAM CLASSIII PREDEFIN RECTAN 1000 1000 PROPERTY ATTACH ALL MA1 PROPERTY ATTACH Surface PH1 PROPERTY ATTACH BEAM PH2

The next command applies a gravitational load on the mesh elements.

PROPERTY LOADS GRAVITY ALL -9.8 Z

The next commands specify the boundary conditions.

```
PROPERTY BOUNDARY CONSTRAINT top YSYMM
PROPERTY BOUNDARY CONSTRAINT bottom all
PROPERTY BOUNDARY CONSTRAINT side all
```

The next commands change the view of the model in a way that it becomes easy to confirm if the generation of the design model has worked.



```
EYE ANGLE -95 -130
EYE FRAME
LABEL MESH CONSTRNT ALL
VIEW MESH CURRENT
LABEL MESH AXES ALL Z
LABEL MESH CONSTRNT ALL VIOLET
@# 1
```

If the model is generated adequate iDIANA creates a DIANA input file ,.dat, with the Utility Write command.

```
UTILITY WRITE DIANA
Yes
```

Finally iDIANA is close and DIANA is started to analyze the generated design model.

```
File close
Yes
ANALYSE MESH
```

#### D.2.2 The Command file [.com]

To properties for the analysis of the design model in DIANA can be specified manually of with a commandfile, a text file with .doc extension. The commandfile shown below starts with reading the input file. The Linsta command determines the type of calculation and stands for Linear Static. After that all the specific results that are required, like support reactions, deformations and internal stresses are specified.

```
*FILOS
INITIA
*INPUT
READ FILE "mesh.dat"
*END
*LINSTA
BEGIN OUTPUT
  FILE "mesh"
 DISPLA TOTAL TRANSL GLOBAL
 FORCE REACTI GLOBAL
 FORCE EXTERN GLOBAL
 STRESS TOTAL CAUCHY GLOBAL NODES
 STRESS TOTAL FORCE LOCAL NODES
 STRESS TOTAL MOMENT LOCAL
                            NODES
 STRESS TOTAL DISFOR LOCAL
                            NODES
 STRESS TOTAL DISMOM LOCAL
                            NODES
 STRAIN TOTAL GREEN GLOBAL NODES
END OUTPUT
BEGIN OUTPUT
 BEGIN SELECT
  BEGIN NODES 1-200
 END NODES
 END SELECT
 STRESS TOTAL CAUCHY PRINCI
END OUTPUT
*END
```

#### D.2.3 The batchfile used for post-processing [.dat]

The analysis results are presented in the iDIANA post-processor. A batchfile can be used to generate pictures of the model. In the scripted lines below can be read how the properties of the vectors that are displayed can be adjusted. After this the camera position is specified. Finally a results type is chosen and a picture format is generated.

FEMVIEW MESH





```
UTILITY SETUP ROTATIONS EYE
VIEW OPTIONS EDGES OUTLINE
PRESENT VECTORS FACTOR .2E-2
PRESENT OPTIONS VECTORS 2D-HEADS
PRESENT OPTIONS VECTORS MODULATE 10
PRESENT OPTIONS VECTORS LINES NORMAL
EYE ROTATE TO 0 0 90
EYE FRAME
EYE ZOOM FACTOR .5
RESULTS NODAL FBX....G RESFBX
UTILITY SETUP COLOUR INVERT
!WATCH THIS
0# 1
ΕΖΙ
UTILITY SETUP OPTIONS GRAPHICS-W AUTOREDRAW ON
UTILITY SETUP PLOTTER FORMAT HARDCOPY BMP
DRAWING SAVE PLOTFILE support.bmp
ves
VIEW MESH
VIEW OPTIONS EDGES OUTLINE
RESULTS ELEMENT EL.MXX.L MXX
RESULTS CALCULATE P-STRESS P1 2DSORT
PRESENT VECTORS VIEWMODE
EYE FRAME
UTILITY SETUP OPTIONS
DRAWING SAVE PLOTFILE PM1.bmp
Yes
UTILITY SETUP COLOUR INVERT
*END
```

#### D.2.4 The Diana input file [.dat]

DIANA input file is generated from the structural model made in the iDIANA pre-processor. The input file contains all the raw data DIANA needs to perform its calculation. The file begins with the models type and the used units.

```
FEMGEN MODEL : MESH
ANALYSIS TYPE : Structural 3D
'UNITS'
LENGTH MM
TIME SEC
TEMPER KELVIN
FORCE N
```

The geometric information in the iDIANA pre-processor is presented as a list of mesh point coordinates. The mesh elements are created by connecting the mesh points. The second term in the connectivity line present the type of mesh element. More information about the used mesh elements can be found in Appendix C.2.3





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CONNECTIVITY 1 CQ40L 84 284 282 285 283 286 86 89 2 CQ40L 83 87 84 89 86 90 85 88 . 1285 SP1RO 1886 1286 SP1RO 1889

The material and the geometric properties of the mesh elements are specified below. Hereby has to be stated that the translation and rotation springs used as support elements are in the input file considered as elements with a material (stiffness) and geometrical information (direction).

```
MATERIALS
 / 52
      16 . . . 82 /
                       LAYERS
                                2 1
 / 56 233 . . . 311 /
                        1
 / 469-490 959-980/ 1
 / 981-1286/ 6
GEOMETRY
 / 52 16 . . . 55
                    /
                         1
 / 301 302 . . . 22 /
                          2
 / 1032-1082 1185-1235/
                         11
 / 1083-1133 1236-1286/ 12
'MATERIALS'
               2.5882E+04
   1 YOUNG
     POISON
               2.0E-01
               2.4E-06
     DENSIT
               12.0E-06
     THERMX
               0.93E+04
   2 YOUNG
               2.0E-01
     POISON
               1.01E-06
     DENSIT
               12.0E-06
     THERMX
.
   5 SPRING
               0.9E+6
   6 SPRING
               6.1E13
'GEOMETRY'
   1 THICK
               0.80E+03
     LAYER
               0.75 0.25
               0.60E+03
   2 THICK
               0.500E+03
                             0.50E+03
   3 RECTAN
.
  10 AXIS
              0.10E+01
                         0.00E+00
                                   0.00E+00
  11 AXIS
              0.00E+00
                         0.10E+01
                                   0.00E+00
  12 AXIS
              0.10E+01
                        0.00E+00
                                  0.00E+00
```

The groups presented below are a list of mesh points that can be used for presentation purposes during the post processing phase. Most of the used supports are translation and rotation springs which are by the input file considered as mesh elements. The vertical support of the structure is of a fixed type. The list under the `supports' section are mesh coordinates which are fixed for translation in the global `3' direction.

'GROUPS'



```
NODES

1 BOTTOM / 48 720 721 725 727 731 732 /

2 TOP / 165 166 . . . 895 /

3 SIDE / 173 175 . . . 944 /

'SUPPORTS'

/ 48 173 . . 1889 / TR 3
```

The model consists of 6 load cases and 5 load combinations. Case 1 is the gravity load that works in the global 3 direction. Case 2 and 3 present the variable load by people e.a., works in the global 3 direction. Load case 4 presents the load by wind from the side and works perpendicular to the mesh surface. Load case 6 presents the load by temperature.

```
'LOADS'
CASE 1
WEIGHT
3
       -9.81
CASE 2
ELEMEN
 / 1-366 491-856/
       FACE
       FORCE
                 -2.50E-03
       DIRECT
                   3
CASE 3
ELEMEN
 / 1-366/
       FACE
       FORCE
                 -2.50E-03
       DIRECT
                   3
CASE 4
ELEMEN
 / 52 16 . . . 260 /
       FACE
                 -1.90E-03
       FORCE
       DIRELM
                   Ζ
•
  573 496 . . . 844 /
/
       FACE
       FORCE
                 -0.67E-03
       DIRELM
                   Ζ
CASE 6
ELEMEN
 / 1-366 469-856 959-980/
      TEMPER 17. 0. 0.
COMBIN
  1
      1 1.2
                2 1.5
  2
      1 1.2
                3 1.5
  3
      1 1.2
                4 1.5
  4
         1.2
                   1.5
      1
                6
  5
      1
         1.0
```

Appendix D: Geometric modification

The directions below present the axis system used in the model, in this case a normal Cartesian global axis system is used.

```
'DIRECTIONS'

1 1.00E+00 0.00E+00 0.00E+00

2 0.00E+00 1.00E+00 0.00E+00

3 0.00E+00 0.00E+00 1.00E+00
'END'
```



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# E Calculation specifications

For this project, where the finite element method is not only used for dimensioning but also to model a geometry, a analysis type is required that is fast and gives good insight. It is not so much the quantity of the results that are important but more how the results change when the geometry of the structure is adapted. During the parametric shape study and the global shape optimization use is made of a geometric and physic linear analysis type. This means that the change in both geometry and physical properties (material) during an increase of loading are not taken into account.

# E.1 Geometric non linear calculation

All the FE- models used in the form finding stage analyzed the internal stress distribution of the roof structure by means of a linear elastic calculation. Hereby assuming the initial geometry of the structure. However, the internal stress distribution changes when the structure deforms. This phenomena can be made clear when analyzing the Von Mises truss which was used before when analyzing the buckling behaviour of arches. As an example, the truss shown in Figure E-1 is used. The parameters of the truss are chosen in a way the structure deforms noticeable (2,72mm,  $\approx$ 1/4h) and so the 2<sup>nd</sup> order effect is made clear. In the graph of Figure E-2 can be seen how the internal forces relate to the loading when analyzed by a geometric linear and non linear calculation. For the geometric non linear calculation the force increases accelerating because the calculation takes in mind the deformed (lower) geometry of the truss. This example makes clear that for a structure with a significant deformation, a linear elastic calculation gives too optimistic results.

#### E.2 Buckling analysis

The buckling behaviour of the roof structure is studied with an Euler stability analysis. An Euler stability analysis gives information about 'linearized stability' of a structure. It tells whether solutions from linear elastic analysis are stable or whether small disturbances to those solutions exist, requiring no extra external energy. This type of stability analysis does not allow for any physical nonlinearities, geometrical nonlinear (i.e., large deformation) effects are only partly taken into account. However, often it is a relatively simple and effective method to get a fair impression of a structure's buckling modes.

In (TNO Delft, 2007) an example can be found where the theory of the linear Euler stability analysis is illustrated. This example consider the Von Mises truss, a simple arch structure made from two bars loaded by a force F as shown in Figure E-3, which was used earlier.



A critical vertical displacement  $u_{crit}$ , corresponding with a critical force  $F_{crit}$ , is searched. By examining the equilibrium condition of the deformed state in more detail where load *F* is in equilibrium with the normal forces *N* in the bars as shown in Figure E-3 the following expression can be derived.

$$2\frac{A_0}{l_0}\left(\left(\frac{Ed^2}{l_0^2}\right) - \left(2\frac{Edu}{l_0^2} - \frac{Eu^2}{l_0^2}\right) + \tau\right) = 0$$

Where:

- A<sub>0</sub> is the initial cross-section of the bars
- *I*<sub>0</sub>. denotes the initial length
- E denotes Young's modulus (the bars are assumed to be isotropic)
- τ is the second Piola-Kirchhoff stress

Derivation of this expression can be found in (TNO Delft, 2007).

The first term in the left-hand-side is the linear stiffness; the second term is referred to as the stiffness due to the initial displacements and the last term is the geometrical stress-stiffness of the structure. For *Euler stability analysis* the following approximations are made:

- 1. Small displacements, i.e.,  $u^2$  is neglected.
- 2.  $\tau \approx \sigma$  where  $\sigma$  is the Cauchy stress following from a geometrically linear analysis:

These assumptions lead to a more simple expression:

$$2\frac{A_0Ed}{l_0^3}(d-3u) = 0$$

It can be seen that applying an external load  $F_{crit}$  such that  $u_{crit} = 1/3d$  leads to instability.

It should be realized that due to the approximations the critical load  $F_{crit}$  as obtained from an Euler stability analysis may be in error. In order to demonstrate this, the exact solution of the stability problem will now be derived without applying any approximations:

$$\frac{EA_0}{l_0^3}(c^2 - l_0^2 + 3(d - u)^2) = 0$$

This equation is satisfied if:

$$u_{crit} = d\left(1 + \frac{1}{\sqrt{3}}\right) \text{ or } u_{crit} = d\left(1 - \frac{1}{\sqrt{3}}\right)$$

Thus, exactly solving the stability problem renders two solutions. The most critical of both is approximately 0.42*d* whereas the linear buckling analysis rendered 0.33*d*. (TNO Delft, 2007)

In case of the roof structure, the deformations stay low, therefore is the linear Euler stability analysis an effective method to get a fair impression of a structure's buckling behaviour.









# **Analysis Results**











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