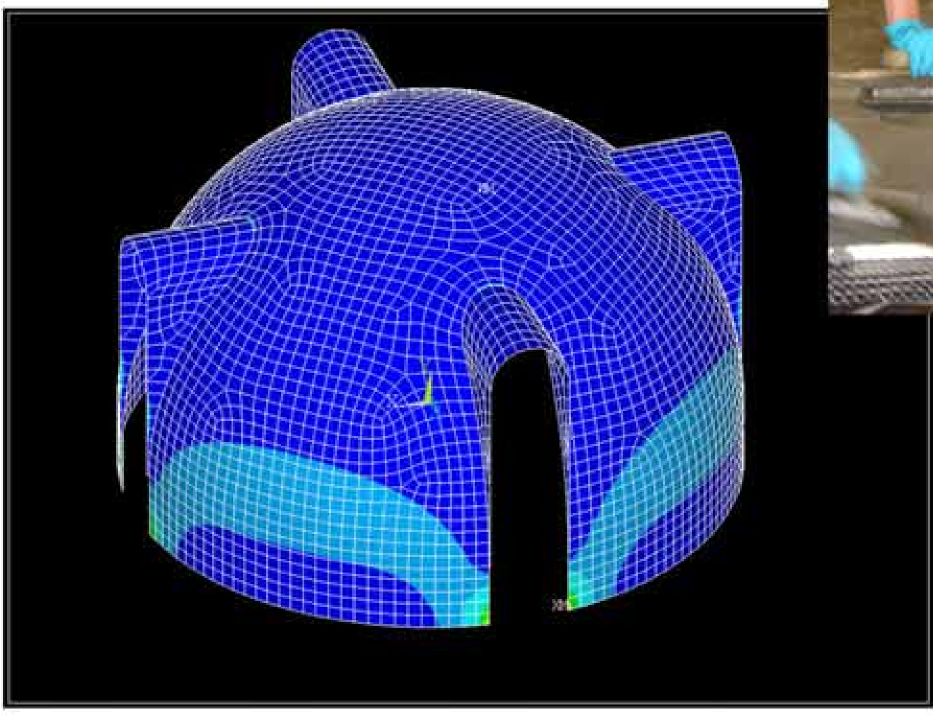
Optimization of Dome Housing in Sri Lanka

Master's Thesis Carli Hammer



Solid House Foundation

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Preface

This report is the product of my Master's Thesis on the optimization of dome housing in Sri Lanka for the Solid House Foundation. The Master's Thesis project was performed at the Structural Design Lab, which is part of the Building and Structural Engineering department of Delft University of Technology, faculty of Civil Engineering and Geosciences. Figures are by the author unless a reference is provided in the caption.

I would like to thank my graduation committee, Prof.ir.L.A.G.Wagemans, Dr.ir.E.Schlangen, Dr.ir.P.C.J.Hoogeboom, Ing Wim J.H.Stroecken for their contribution. Also, I would like to thank the other graduates of the SDL in room 0.72, who were always willing to discuss the problems that I came across. All members of the SHF Denktank, thanks for all your input. Tomas Viguurs and Arjan Spit your information has been very usefull. Petra van Hennik thanks for the introduction on the subject and Frank Braam thanks for your information about pneus. Ron Mulder and collegues of the Stevinlab., thanks for your help on the experiment, it was fun! Pieter, thanks for your support and patience.

Carli Hammer
Delft, September 2006

Summary

The Solid House Foundation uses inflatable hemisperical formwork to build concrete dome dwellings. In this thesis a study is made of possible optimization of this building concept. Main occasion is the increasing price of rebar and the bad availability of rebar in most regions where the SHF is active. As SHF is now involved in a large housing project in Sri Lanka, a first focus is on this region.

To have an idea of threads and opportunities in dome building a literature study was made on dome shapes in nature, domes in other cultures, the history of concrete shells built with inflatable formwork and of domes in general. As a result several form-related possibilities were identified that could reduce the tension stress in the shell and thus the amount of reinforcement needed.

Also, research on alternative materials for dome building was done. This resulted in several options of which ferrocement was considered the most suitable. The latter from both a cost point of view as from the fact that there is a lot of experience with this easy applicable material in Asia. For the current design half of the material turns out to be used for the foundation of the dome. After some calculations could be concluded that this heavy foundation was required to anchor the uplifting forces of the inflatable formwork. Consequently research has been done on alternative anchorage of the form, resulting in ideas for formwork that does not need anchorage at all. The research phase was rounded off with a study of the climatic circumstances in Sri Lanka. Matching building responses to the climate were studied and applied on dome designs.

The conclusions drawn from a structural analyses in the finite element program ANSYS have resulted in a proposal for alternative material use in combination with the currently applied formwork. A design for a ferrocement shell has been made and an experiment is carried out.

However to improve issues such as the heavy foundation and the dependency on electricity, a different design of the formwork is required. Therefore possibilities for an alternative design of the formwork are studied and evaluated.

Contents

INTRODUCTION

Chapter 1

Introduction

The dome shaped dwellings of the Solid House Foundation (or SHF) are constructed with reusable pneumatic formwork. This concept originates in Texas, where the Monolithic Dome Institute trains people to build there own dome. More about the SHF and this construction technique can be read in chapter 2 and section 4.3.

Figure 1.1: SHF in Bolivia [SHF, 2005]

In the East of Sri Lanka lies the village of Inspector Eatham, which SHF is currently helping to rebuild, see chapter 3. After the tsunami hit in December 2004, about 12,000 houses were damaged in the eastern part of Sri Lanka. This has resulted in an extreme shortage of skilled labor and an increase in price of labor and building materials.

In this Master's Thesis the SHF's building concept is studied for use in Sri Lanka. Can cost reductions be made by changes in the shape of the dome or in material use? Is it possible to simplify the building process? How can building physics be improved? Can local materials replace part of the currently used materials for more sustainable development of the domes? The Solid House Foundation identified the reinforcement responsible for a substantial part of the building costs, besides rebar are less readily available than materials as sand and cement. Therefor initial focus is on the reduction or replacement of steel reinforcement of the domes.

This report has been divided in four sections. The section 'Introduction' provides important background information about the Solid House Foundation and circumstances in Sri Lanka. The section 'Research' represents an overview of the first part of this thesis; information was collected on dome construction in general, alternative materials and climatic responsive design. Consequently stresses in both the shell and the foundation of the current Solid Houses caused by different load cases were analysed, see section 'Analyses'.

As a result of the research and analyses, alternatives were developed. The section 'Alternatives' consists of two chapters: Chapter 10 focuses on an alternative material, while making no changes to the application of the inflatable formwork nor to the formwork itself. However to improve issues such as the heavy foundation and the dependency on electricity, a different design of the formwork is required. Therefore possibilities for an alternative design of the formwork are discussed in chapter 11.

1.1 Objectives

The objective of this Master's Thesis is

'Optimization of SHF's Dome Housing for Sri Lanka';

using minimum costs and maximum process simplicity to build sustainable domes.

The scope will be limited to domeshells built with inflatable formworks.

Note that satisfactory results imply alternatives with improvement in sustainability, process simplicity and(/or) building costs. The implications of this objective are explained in the overview below.

Sustainability	*Simplicity*	*Minimum Costs*
Independent further development without support possible	Building process that demands demands a minimum of skilled labor	Building materials
Fire resistance	Ease of reparation	Labor
Flexible (alterations and extensions)	Easy to copy without much instruction	Transport
Availability of building materials		Machinery
Lasting 50 years		

Chapter 2

Solid House Foundation

2.1 Objective

"The Solid House Foundation's (SHF) objective is to create, conduct and facilitate housing projects in poor communities. The intention is to enable local inhabitants' associations to independently prepare and build successful housing projects. The construction is therefore done in a fairly simple, inexpensive and sustainable manner." [SHF, 2005]

Figure 2.1: Logo SHF

Founded in November 2003, the Solid House Foundation (SHF) aims at contributing to an enduring development of social housing projects. Key issues are the sustained development and social grounding. Therefore, the SHF works directly with the target group and encourages local participation in projects by stimulating people to take on responsibilities in different areas, such as the project and financial management, but also by performing labor. The number of houses built is not the only measurement of success to SHF, even more important is the quality of the transferred knowledge. The SHF aims at increasing people's independence and building their capacities. Besides building durable and solid houses, SHF is especially interested in the development of the community in a broader sense. To achieve this, SHF does not only focus on housing, but also on education, economic development, health care, safety and community building. 'By actively involving the target group in our projects, we want to achieve our practical goal of providing accommodation, as well as our social aim of increasing skills and stimulating community life. '

2.2 Building concept

Concrete domes, or 'SolidHouses', are constructed with pneumatic formwork. This concept originates in Texas, where the Monolithic Dome Institue (MDI) trains people to build there own down.

The pneumatic formwork is produced by BingFo [1] in the Netherlands and transported to the location. Other materials are purchased locally. After the reinforced concrete foundation ring and floor have hardened, the formwork is fastened to the foundation. The formwork is inflated using blower fans (and a generator). Rebar (diameter 10 mm) are tied around the balloon in meridional and circumferential direction, resulting in a framework of maximum 25 cm spacing. The vertical rebar embedded in the foundation ring are attached to the steel reinforcing of the dome itself.

Pieces of plywood or board are used to make the formwork for window and door openings. Additional rebar are added. Concrete is applied in two layers of about 3 to 4 cm each, applying the next layer when the concrete is hard to the touch. The layers can be hand applied or 'flipped' on by using a small mason trowel.

After the exterior concrete has cured enough to stand, the formwork is deflated and removed. Usually the concrete will be strong enough in 24 hours after the last coat. The exposed rebar on the inside of the dome are brushed and then coated with a layer of about 1,5 cm of concrete.

The inside is coated with plaster. The outside should be coated with an elastomeric coating to help with solar reflectance and to protect against possible leaks from hair line cracks.

The domes are hemispheres, some elevated on a cylinder of a meter. The hemispheres have a diameter of 6, 9 or 12 meter. Due to the shape, ring tension forces occur in the lower part of the spheres and reinforcement is necessary. Openings in the shell interrupt the tension rings. These interruptions have to be compensated for by extra reinforcement below and above the openings. More information on stresses in shells in section 8.1.

[1] www.bingfo.nl

Figure 2.2: Building process SolidHouse and MDI [MDI, 2005]

2.3 Projects

The first project of the SHF was completed in 2004. It took place in Bolivia, where a group of young 'shoeshine boys' completed 8 homes in El Alto (near La Paz). Sofar in Kenia domes have been built for the Owiti Orphanage in Kisumu by the Australian organisation 'Youth International'. In cooperation with the local housing union the SHF is planning to use the inflatable formwork of these domes to build more domes in Nairobi as soon as more funds are available.

After the mass destruction left by the tsunami in Asia the Solid House Foundation started to focus on this region. In Aceh the SHF is starting a building project in cooperation with a local organization called Yayasan Mamamia. In Sri Lanka the SHF is currently rebuilding a village with funding of the Red Cross. As this is both the most active project at the moment of writing as well as a large long term project, this thesis will focus on Sri Lanka.

Problems experienced in Bolivia had mainly to do with the air pressure of the formwork. To build the SolidHouses the formwork needs to be inflated for at least three consequent days. As air leaked at the base, the pump needs to be quite powerful as well. It turned out to be difficult to find a reliable and powerful pump. It is recommended to use a back-up pump for the next project.

Figure 2.3: Building SolidHouses in La Paz, Bolivia [SHF, 2005]

Figure 2.4: SolidHouse of the Owiti Orphanage in Kenia [SHF, 2005]

	MDI		SHF Bolivia	SHF/MDI
	Proportions	6 meter dome	6 meter dome	
foundation				
cement	1 bag		2250 kg	
sand	4 buckets		6,00 m3	
aggregate	6 buckets		4,50 m3	
water	2 buckets			
rebar	unknown		480,77 kg	
steel kg/m3			82 kg/m3	
dome				
cement	1 bag		2450 kg	
sand	5 buckets		6,00 m3	
rock	1 buckets		5,10 m3	
water	2 buckets			
aggregate	unknown		519,23 kg	
steel kg/m3			138 kg/m3	
total 6 meter dome		(64 batches)		
cement		2880 kg	4700 kg	1,6
sand		6 m3	12 m3	1,9
aggregate		4 m3	10 m3	2,7
rebar		567 kg	1000 kg	1,8

Table 2.1: Proportions MDI and SHF

Problems with the concrete quality occurred due to the extreme climatic circumstances, which resulted in extra coating material, an increase in shell thickness from 8 to 10 cm and an increase in rebar diameter of 8 to 10 mm. In the evaluation report the importance of the right mix and the treatment during setting of the concrete are stressed. Also requirements for the coverage of the rebar is given: a minimum of 2,5 cm on the outside and 1,5 cm on the inside.

Including sanitary, electricity, paint and coating the domes in Bolivia had building costs of 2.400 respectively 5.300 euro for domes with a diameter of 6 respectively 9 meter.

The costs are relatively high per SolidHouse as the project is small, and the investment in the formwork is relatively high. Normally between 75 and 125 domes can be produced with one pneumatic form.

In table 2.1 the proportions given by the Monolithic Dome Institute (MDI) are compared to the use of material by the SHF in Bolivia. The SHF used considerably more material, probably due to the thicker shell, but less cement per cubic meter concrete. The MDI calculates about 6 cubic meters concrete per dome of 6 m diameter. The difference in amount of coarse material is unclear as there is no detailed information about the size of the shingles used by SHF in Bolivia. MDI prescribes a maximum aggregate diameter of 25 mm for the floor and 10 mm for the dome.

Chapter 3

Sri Lanka

Sri Lanka, an island in the Indian Ocean, lies between 5 ° and 10 ° north of the equator and between the eastern longitudes 79 ° and 82 °. From North to South it has a maximum length of 435 km and at its widest point it measures 225 km, giving it a land area of 65,600 sq.km. Sri Lanka is mostly flat land that gently rises to a mountainous region, central and south. The highest mountain rises to 2,518 m. Sri Lanka has a population of about 20 million people.

Figure 3.1: Map of Sri Lanka [Climate, 2005]

3.1 Situation

The project location is at Inspector Eatham, Pottuvil DS, Ampara District, Eastern Province, Sri Lanka. Inspector Eatham is located off the main road just outside of Pottuvil, about 1,5 km inland.

The about 50 acres of land are slightly elevated. Vegetation consists of large bushes and some trees. There is a large water tank as well as some smaller ones. Large parts of the land are dry and barren outside the rain season. Smaller parts are already being used for cultivation (mainly paddy), cattle farming and brick making. There are some existing buildings like a community center, an unfinished temple and some remains of old buildings on the site.

The 280 Tamil families have been living and using the land at Inspector Eatham for many years. Starting in 1983 they have been suffering from the war and in 1990 this forced them to leave the area. They moved to the nearby Tamil community of Komari. In 1993 they returned to Inspector Eatham but in 2001 they were forced to move again by the lack of water and security issues. This time they moved to Kotukal, a nearby area close to the sea.

The tsunami on December 26th 2004 destroyed the complete community at Kotukal. Since most families were staying in mud and clay brick houses not much remained after the tsunami. The families do not want to return to Kotukal out of fear for the sea and it is their wish to return to their old land at Inspector Eatham.

Currently most families are staying with friends and relatives in the area of Inspector Eatham or in improvised shelters nearby. Some families decided to leave the area for good and moved to other parts of the country. An estimated 150 families are entitled to use land in the project area and some have started building improvised shelters scattered over the area. Most people do not actually live there yet because of the lack of water and the danger of elephants at night.

An average family consists of 4 family members, adding up to 600 persons for the estimated 150 families. Among these families there are many widows who lost their husband during the war and over two third is under 35 years old. Most families earn their income as day laborers (so called 'coolly') with some exceptions that do so with farming, fishing and self-employment. The average income is 3,300 RS (27 euro) per month per family.

Figure 3.2: Tsunami hit area of Kotugal [SHF, 2005]

Figure 3.3: Shelters around the project area Inspector Eatham [SHF, 2005]

Month	Average Sunlight Hours	Temperature				Discomfort from heat and humidity	Relative Humidity		Average Precipitation (mm)	Wet Days (+0.25 mm)
		Average		Record						
		Min	Max	Min	Max		am	pm		
Jan	7	24	27	18	33	Medium	79	78	173	10
Feb	9	24	28	19	36	Medium	72	70	66	4
March	9	24	29	19	38	High	72	70	48	4
April	9	26	32	19	39	High	69	68	58	5
May	8	26	33	19	40	High	67	61	69	5
June	7	26	33	22	39	High	65	54	28	2
July	7	26	33	21	38	High	65	53	51	3
Aug	7	25	33	21	39	High	65	56	107	6
Sept	8	25	33	21	39	High	65	61	107	6
Oct	6	24	31	21	39	High	72	69	221	13
Nov	6	24	29	19	36	High	80	78	358	17
Dec	6	24	27	19	33	Medium	80	79	363	16
average	7.4	25	31	20	37		71	66	137	8

Table 3.1: Climate Trincomalee [Climate, 2005]

3.2 Climate

Sri Lanka's position between 5 ° and 10 ° north latitude endows the country with a warm climate, moderated by ocean winds. Climate is tropical with high humidity and year-round temperatures averaging 27 to 28 °C. Pottuvil is situated in the 'dry zone' of Sri Lanka's South-East coast, which receives between 1200 and 1900 mm of rain annually. Much of the rain falls from October to March, during the rest of the year there is very little precipitation. [1]

In October and November periodic squalls occur and sometimes tropical cyclones bring overcast skies and rains to this part of the island. From December to March, monsoon winds come from the northeast, bringing moisture from the Bay of Bengal.

Average temperatures in the shade are:

- 28 – 30 °C during November, December and January
- 30 – 32 °C during February, March, September and October
- 32 – 36 °C during April, May, June, July and August

Day and night temperatures may vary by 4 to 7 °C.

Trincomalee lies in the 'dry zone' as well, 138 km north of Pottuvil. Average weather conditions are shown in figure 3.4 and table 3.1.

[1] Used sources for this section see [Climate, 2005].

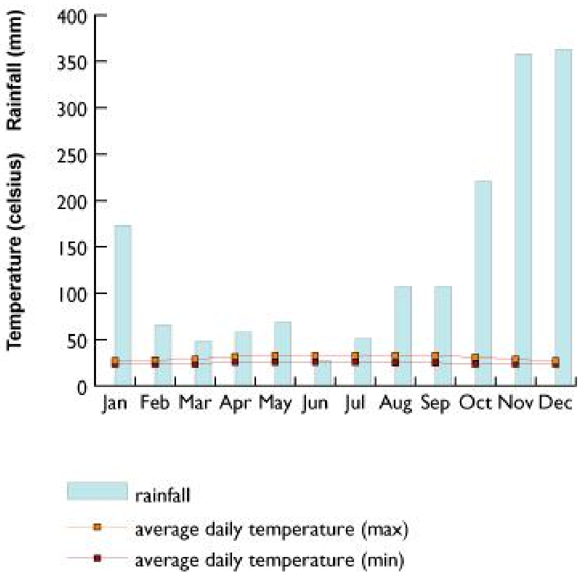


Figure 3.4: Climate Trincomalee [Climate, 2005]

As shown in figure 3.5 the probability of an earthquake in Sri Lanka is very low [Seismic, 2006].

The wind is predominantly onshore wind (east wind). As typhoons occur in the North and East of Sri Lanka special attention is paid to wind loading, even for small structures. There are about two cyclone warnings every year and once every six or seven years a cyclone actually hits the coastline. The last serious event happened in 1978, when a cyclone killed approximately 1000 people. Therefore wind speed for normative wind loads in the North and Eastern province (up to 50 km inland) is based on cyclonic conditions, being 49 m/s [2]. This results in a design wind pressure of $1,2\ kN/m^2$ and a total windload of $3,4\ kN$ on a dome of 6 meter diameter [3]

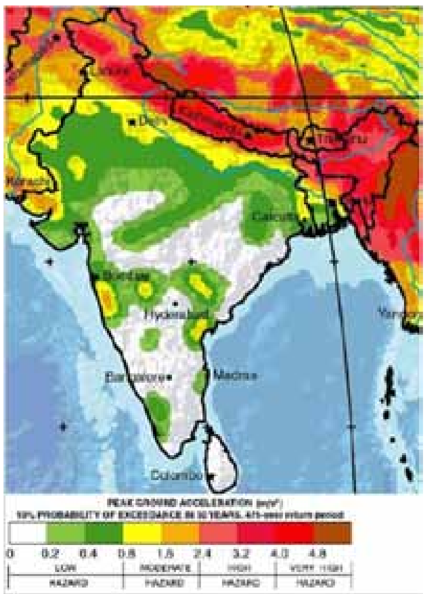


Figure 3.5: Chance of earthquakes [KNMI, 2006]

[2]Source: Prof.M.T.R. Jayasinghe, Professor of Civil Engineering at the Department of Civil Engineering at Moratuwa University, Sri Lanka

[3]In Sri Lanka the British Standards are used. These calculations are based on the Indian Standards (IS: 875), which is based on the British Standards.

Figure 3.6: Discussing functional design with inhabitants [Viguurs]

3.3 Housing

3.3.1 Functional requirements

The SHF is currently discussing functional requirements with the inhabitants of Inspector Eatham. The architects of SHF's 'denktank' have designed a flexible standard system of short walls inside the dome on ground level to which families can add walls themselves to their personal needs, see figure 3.7. The short walls are parallel to the circumference and can be used to make a dividing wall of cubboards. If the short walls are not used for a dividing wall, they do not divide the space either. Another idea of the denktank is to introduce a first level, creating extra square meters for example for sleeping. In figure 3.7 two alternatives for the first level can be seen. As the demands of a Sri Lankan family are not clear yet, changes are to be expected. The cooking unit for example might be leftout, because Sri Lankans prefer to do their cooking outside. There is no certainty about the introduction of a sewage system either. Important to note is that compared with the western world Sri Lankans spend much more time outside.

3.3.2 Available materials

After the tsunami hit in December 2004, about 12.000 houses were damaged in the eastern part of Sri Lanka. Many charity organizations are active in the area, but as they have no experience with habitat projects all housing is tendered on the local construction market. This has resulted in an extreme shortage of skilled labor and an increase in price of labor and building materials. As especially steel prices have increased considerably during the last years and as rebar are not always readily available, it would be interesting to reduce the amount of steel used in the domes.

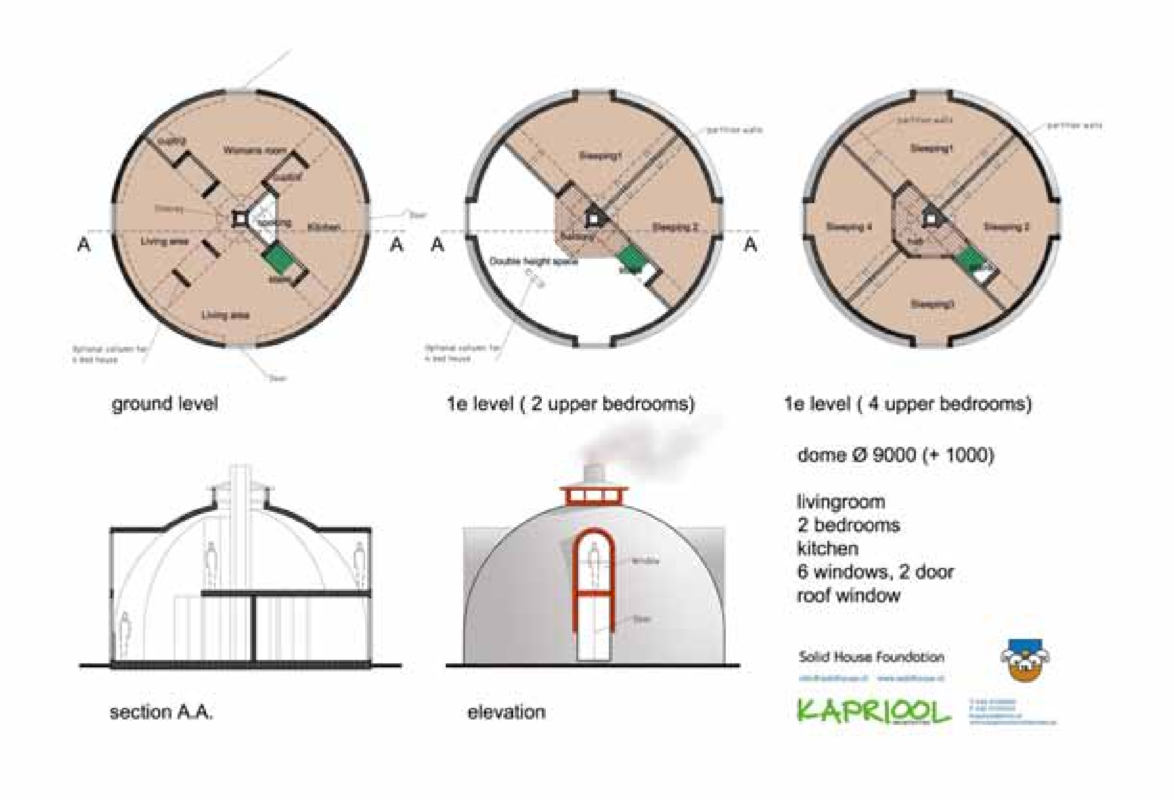


Figure 3.7: Flexible system, 'Denktank' SHF

With the help of Rik Lurinks, a student Civil Engineering who is doing an internship in Pottuvil, and Thomas Viguurs who is working for the SHF in Sri Lanka, the list below was made.

Building materials in Sri Lanka

- Sand
- Cement blocks
- Cement (nowadays partly imported from Japan)
- Rebar
- Mesh
- Reed
- Rice grass
- Palm leaves
- Corrugated sheets
- Wood
- Bamboo

Wood is relatively scarce and expensive in Sri Lanka. Available wood has mostly been imported from Indonesia or logged illegally on the island. Bamboo does grow on the western part of the Island, but not a lot and the quality is inferior to the Indonesian bamboo.

RESEARCH

Information collected on domes, materials and climate responsive design by studying various sources (see 'References').

Chapter 4

Literature Study

Domes have been built all over the world. In Europe mostly for religious buildings, in Asia and Africa they have also been used in residences offices and public buildings. In hot climates the structures provides better natural climatic control because of their height in the middle of the rooms where the light, warm air gathers and can be easily discharged through openings. An important advantage of domes is that they need less building material for the same volume enclosed.

A literature study has been carried out on domes and related subjects, in order to have a better idea of potential problems and potential alternatives for i.e. shape and material. The study is focused on the history of domes in general, inspiration from nature, the history of concrete shells built with pneumatic formwork and alter architecture.

4.1 Domes, Examples from History

In order to build domes without making use of a material that can be subjected to tension, several methods have been developed to reduce these tensions rings.

Domes made of stones held together without mortar appeared in south-east Italy, especially Valle D'Itria, sometime between 2,000 and 1,000 B.C. and proliferated in the 15th century. The conical shape and the heavy walls of these stone 'trullo domes' prevent tension to develop.

Figure 4.1: Trullo domes

In the pantheon (115-125 A.D.), probably the most famous dome in the world, ring tension forces were reduced in two ways. The first being the loading of the lower part of the hemisphere with high mass material and thereby keeping pressure in the structure. The second method used was the reduction of the dead load in the upper part, by reducing the shell's thickness, using lower mass concrete, using cassettes and leaving the uppermost part open. The relative thickness of the dome is reduced from 5.9 m at the base to nearly 5 feet 1.5 m at the top [Moore, 2006]. Moreover the ring of this oculus is effective in properly distributing the compression forces at this point. In other domes the Romans poured pots in the shell to reduce its weight [Patzelt, 1972].

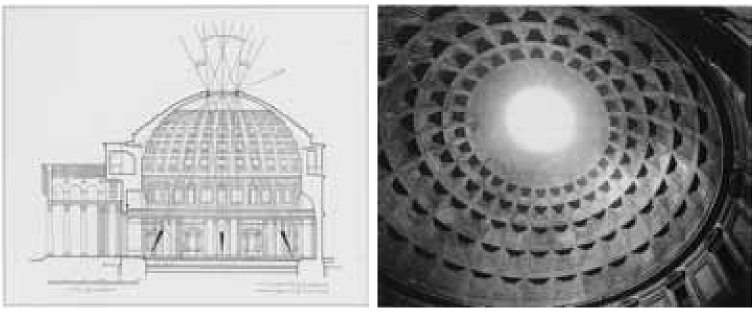


Figure 4.2: Pantheon, Rome [Portoghesi, 2000]

For the Hagia Sophia, built 417 years later in 532 AD, a different method was adopted. The lower part of the sphere was left out. The outward thrust force at the bottom was absorbed by the surrounding construction of other hemispheres. In the same way buttresses were used in the Middle Ages.

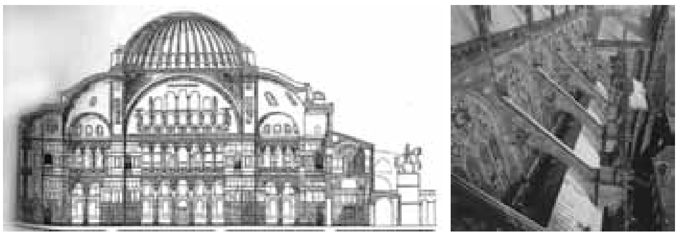


Figure 4.3: left: Hagia Sophia in Istanbul [Oosterhoff, 2002] right: Rampant arches of Strassbourg Cathedral [Portoghesi, 2000]

Brunellesci introduced a sort of tension anchorage in the dome of Florence , which was built from 1420 to 1436. As in that time wrought iron could not be made in sufficient sizes, he used oaken elements. At about a quarter of the height of the dome he designed a ring of 24 oaken elements, spanning from rib to rib, that were connected by oaken wedges. At the base of the dome he did use iron anchors to connect limestone rocks [Oosterhoff, 2002].

Figure 4.4: Adobe, production and construction [Auroville, 2006]

From another point of view the dome of Brunellesschi was bright engineering as well. Instead of one massive shell, he constructed two shells that he connected with horizontal rings and vertical ribs, creating a very stiff and strong dome.

In countries with very hot and dry climates, domes are commonly constructed with earth or adobe. Adobe soil has clay and sand in such proportions that when mixed into mud then dried out it forms a brick or a wall. The best adobe soil will have between 15% and 30% clay in it to bind the material together, with the rest being mostly sand or larger aggregate. An adobe brick is made of adobe soil and is sun cured on the ground.

Figure 4.5: Dome Florence

The architect Hassan Fathy, born in Egypt in 1899, devoted himself to housing the poor in developing nations, especially rural Egypt. Fathy revived ancient design methods and materials. He trained local inhabitants to make their own materials and build their own buildings, especially adobe domes. His designs incorporate windtowers and traditional courtyard forms to provide passive cooling [Steele, 1997] [Fathy, 1973].

By changing the shape of the cross-section to a parabolic-like curve and by using high mass walls, ring tension forces are reduced to a minimum. In these areas with a high range of diurnal temperatures they provide optimal climatic control because of there inherent thermal mass and their height. Another example is the domes of the Mousgoum in Cameroon.

Gaudi used these parabolic-like curves in his designs as well. He based the shape of his masonry vaults and domes on hanging models. In other words he used the shape of an inversed rotated catenary (which is different from a parabole). The basis of the hanging model experiment is that one characteristic load case is used to generate the final shape by large deflections of a given membrane. In 1748 the inverted catenary was already used by Giovanni Poleni to compare the shape with Michaelangelo's design of St. Peter's in Rome (see figure 4.6). In many cases, heavy edge beams can be avoided by this method, to yield naturally shaped shells with free edges. This principal was used very successfully and economically in many practical applications by Heinz Isler [Hilliges et al., 1992]. An other architect who experimented with hanging models is Frei Otto [Otto and Others, 1982]. The procedure is powerful but has some drawbacks. For example, it is not possible to find a compromise when different load cases are dominant.

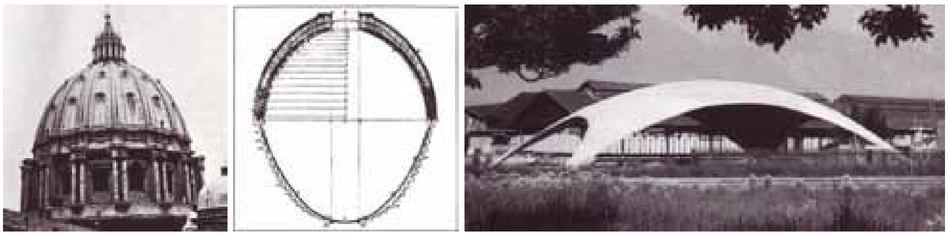


Figure 4.6: St.Pieter in Rome and it's catenary model, shell based on hanging model by Isler [Otto and Others, 1982]

A subdivided icosahedral hemisphere, the structural framework of light steel bars, first built in 1922 in Germany by Dr. Walter Bauersfeld. The frame was covered with a thin layer of concrete, based upon the thickness ratio of an egg shell to its diameter [Patzelt, 1972]. This was the world's first thin shell concrete dome, the building technique later furthered in construction of large structures by Pier Luigi Nervi of Italy and Felix Candela of Mexico. Some thirty years after the Jena dome and considerable European development of domes and thin shell construction, Buckminster Fuller patented the same subdivided icosahedron principle in 1954 and built a variety of what he termed geodesic domes in the US.

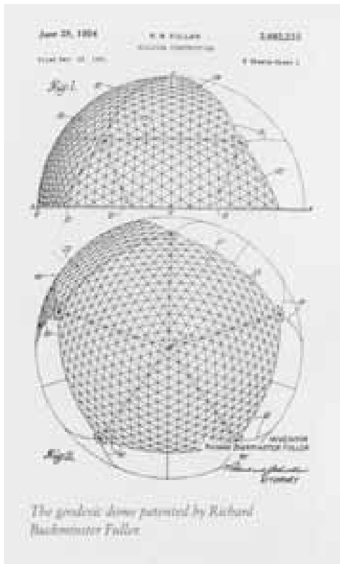


Figure 4.7: Patent Fuller [Portoghesi, 2000]

4.2 Nature

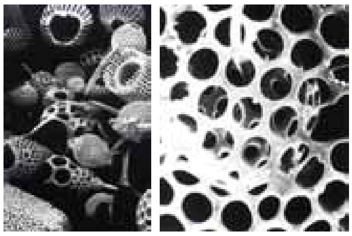


Figure 4.8: Radiolarien [Otto and Others, 1982]

The geodesic dome consists of a frame of triangles. In nature frames of hexagons are more common, for example in honeycombs. Simple organisms like radiolarians are especially interesting because of their dome shaped structures. The circular openings (for soft tissue) are surrounded by a more or less hexagonal frame. In a hexagonal frame the joints are stiff and fewer bars are needed. In case of an icosahedral hemisphere the connections can be flexible, the deformations are smaller. The domes of the Eden project in Cornwall consist mainly of hexagons, but pentagons and triangles were used as well.

However, caution must be taken when using examples from nature for building purposes, as structures in nature are mostly optimized for a far more complex and different set of parameters than the ones used for engineering purposes. Therefore it is important to know (at least a part) of the parameter that lead to a specific form in nature, so that useful principles can be defined [Vogel, 2000].

The generated model of a sea urchin shell is shown in figure 4.9 and was the subject of biomechanical study [Philippi and Nachtigall, 1996] [Bletzinger and E.Ramm, 1993]. A thicker shell has a higher load-carrying capacity than a thinner one and numerous investigations have shown that sea urchins living in turbulent water have thicker shells than echinoids living in calm water. Shell growth, however, is closely correlated with energy expenditure. The energy budget of a sea urchin is limited and must be exploited effectively. The expedient use of material and possession of a shell designed to minimize energy requirements have been shown in a FE analysis. The model always reacts positively to the introduction of ribs. In other words the introduction of ribs is a very material-efficient way to stiffen the shell.

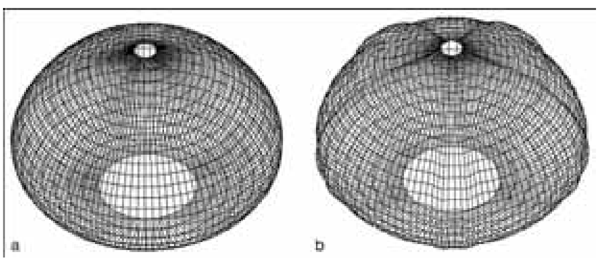


Figure 4.9: FEM analyses sea urchin shell model [Philippi and Nachtigall, 1996]

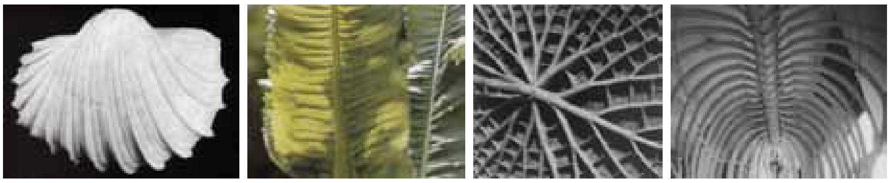


Figure 4.10: Ribs providing stiffness in nature: seashell [Patzelt, 1972], leaf of unknown plant, leaf of Victoria Regia and an elephant skeleton [Portoghesi, 2000]

Figure 4.11: Ribs in building engineering: Amiens kathedral [Portoghesi, 2000] and Nervi's Getti Wool Factory [Otto and Others, 1982]

Ribs for stiffening can also be found in i.e. plants, like the leaf of a Victoria Regia. In Roman architecture ribs were already in use. Brick arches were inserted into large vaulted roofs made of concrete. These arches were either built at groins of a transept or used to divide the hemispheric cupola into panels, discharging the weight onto the groins in order to insert large windows.

In the Late Gothic architecture of Northern Europe and England, the ribbing system that had simplified building techniques by taking the weight off the system now acquired distinct expressive traits, and the infinite variety of different patterns became the central theme of compositional research for almost two centuries [Portoghesi, 2000].

Ribbing also caught the attention of the masters of reinforced concrete. Anatole de Baudot, Perret, Maillart, Nervi, Torroja, Candela, Morandi and more recently Calatrava have all interpreted it individually according to their own sensibilities.

"During the 19th or 20th centuries, the engineer who did his best to design reinforced or suspended beams, found that some of this best ideas had, so to speak, been anticipated long ago by the bone structures of gigantic reptiles and large mammals."

Wentworth D'Arcy Thompson[1]

Thompson is referring to the optimization of material in bones. He noticed the efficient cylindrical cross-section; torsion stiff and most of the material at

[1] [Portoghesi, 2000]

the outside to take the bending forces. He also studied the fine lattice-work of bone material in human bones under compression. He realized that these actually form patterns of perpendicular compression and tension-lines. The lines show strong similarities with so-called 'Michell'-structures. Michell structures are structures designed to transmit load from specified points of application to supports using a minimum weight of linear elements. Luigi Nervi used this principle for his designs, for example for the design of the Getti Wool Factory (1953), see figure 4.2. Nervi oriented concrete ribs in direction and magnitude of the primary stress trajectories [Stach, 2002] [Patzelt, 1972] [Dumans, 2005].

4.3 Historical Development of Concrete Shells built with Pneumatic Formwork

Petra van Hennik has thoroughly studied the historical development of pneumatic formwork for her Master's thesis [Hennik, 2005]. In this section an abstract is given of her findings concerning the historical development of concrete shells built with pneumatic formwork. Note that all domes mentioned make use of shotcrete or gunnite[2].

4.3.1 Concrete applied on Outside Formwork

The building concept that the SHF uses originates in California. Wallace Neff was the first to use pneumatic formwork for the construction of reinforced concrete domes in 1942. He later developed the concept for mass housing projects in West Africa, Egypt and Brazil in the 1940's and 1950's.

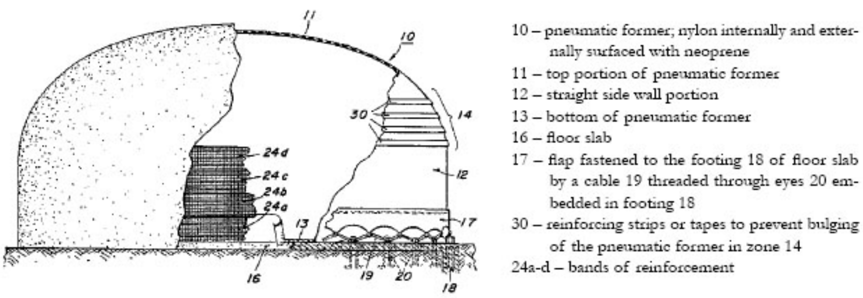


Figure 4.12: Patent drawing of Neff's building system [Hennik, 2005]

Other architects that used the system are Haim and Raphael Heifetz (Israel, 1964), Isler (Switzerland, 1977) and the MRF (2001). Each introduced slight changes to the design.

Neff domes had straight walls that he tightly wrapped with wire mesh reinforcement before inflating the formwork completely. In that way he prestressed the vertical walls and prevented cracking through ringtension. A circular foundation ring provided for anchorage of the balloon. To reduce deformation due to the weight of fresh concrete (figure 4.13), he locally reinforced the formwork and started with a thin layer of fast-setting and light concrete on the outer surface of the inflated form as a preliminary structure.

Haim and Raphael Heifetz coped with the swelling of their hemispherical formwork (figure 4.13) by using a higher air pressure. While Neff worked with an air pressure of 0,5 to 2,0 kN/m^2, Heifetz used 4,0 - 10,0 kN/m^2. To counteract the larger uplifting forces he designed several anchoring system, of which the

[2]Shotcrete is applied with a pneumatically driven pump. Concrete that is either site-mixed or delivered from a plant is poured into a large hopper, from where it's pumped through a hose and sprayed by high air pressure (700-800 N/mm^2) and air volume (around 28,3 liter/min.) against the formwork. Gunnite differs from shotcrete in that the ingredients are put separately into the gunnite truck, and then pumped through hoses, where they mix at the nozzle.

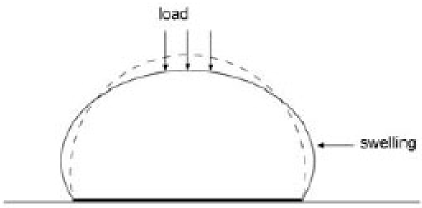


Figure 4.13: Deformation due to the weight of fresh concrete [Hennik, 2005]

Figure 4.14: Heifetz, left: Applying shotcrete on inflated formwork, right: Villas in Ramat Hasharon, Israel [Hennik, 2005]

most successful is shown in figure 4.15. Radially arranged trusses counteract the forces of the pneumatic structure (see also chapter 9). Because of the closed circuit of forces the foundation only needs to support the light and strong dome. This results in a light foundation and possibilities of building these domes on relatively light supports. Heifetz also introduced an automatic pressure control system to prevent cracks due to fluctuation in air pressure (i.e. through changes in temperature) during setting of the concrete.

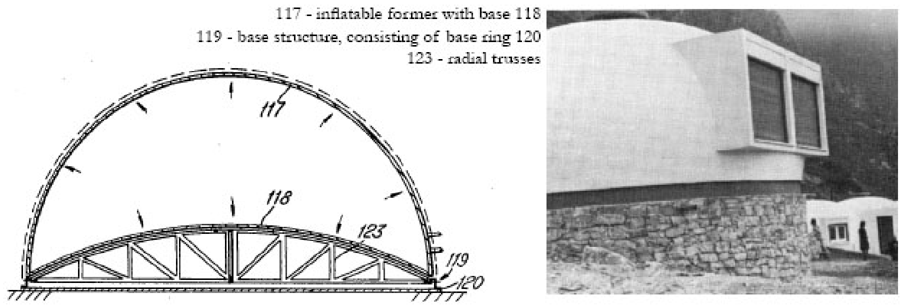


Figure 4.15: Heifetz, left: Support by trusses [Heifetz, 1971], right: House constructed with radial trusses on circular wall

In 1977 the Swiss engineer Heinz Isler used an almost full sphere instead of a half sphere, consequently no major anchoring was needed. After a design for earthquake resistant housing in Iran, Isler worked on a standard range of 'balloon shells' for living spaces, studio workshops, clubhouses and motel rooms with a diameter of ca. 7 m (figure 4.16).

4.3.2 Concrete applied Between Membranes

The Italian architect Dr. Dante Bini introduced a new working procedure in 1965. Expandable steel mesh reinforcements are lain out and concrete is poured on ground level, before inflating the form to a pressure of 3.66 - 5.27 kN/m^2. The steel springs and an external membrane hold the concrete and maintain uniform thickness, see figure 4.19. Consequently no scaffolding is needed to spray the concrete. Also, the reinforcement members are not only meridians and latitude circles, but have several directions. Shell dimensions ranged from 12 to 40 m in diameter at the base, which was not necessarily circular. He also developed 'Minishells'; 8 m x 8 m or 10 m x 10 m square-based, monolithic, reinforced concrete shell structures.

Figure 4.16: Isler, prototype for housing project in Iran and 'balloon shells' in Ponthierry, used as work studios. [Hennik, 2005]

Figure 4.17: left: Binishell on rectangular base (Randwick Girls High School, New South Wales, Australia). right: Binishell on circular base (The North Narrabeen Primary School, New South Wales, Australia). [Hennik, 2005]

Figure 4.18: Isler's dome houses [Hennik, 2005]

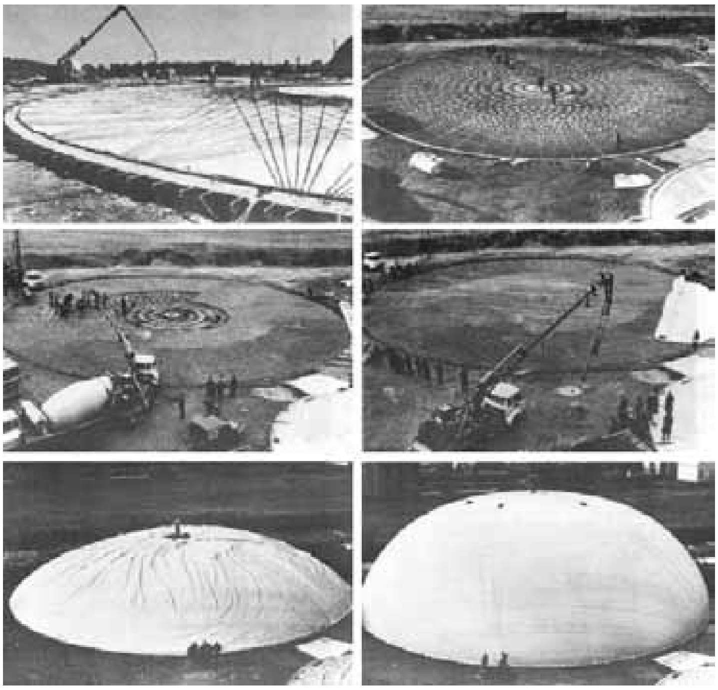


Figure 4.19: Bini's construction method [Hennik, 2005]

4.3.3 Concrete applied on Inside Formwork

In the sixties the Californian architect Lloyd Turner invented a building system which allows spraying of the inside of the pneumatic form, enabling construction under all weather conditions.

Figure 4.20: MDI, the formwork is inflated on top of a reinforced ring beam foundation, polyurethane foam is applied on the inside and reinforcement is hung onto the foam layer before the inside is shotcreted. [Hennik, 2005]

After the membrane is tightened to the foundation ring, ventilators inflate the pneumatic form and keep it a low pressure of ca 0.5 to 1.0 kN/m^2. Subsequently, the inside of the inflated membrane is sprayed with polyurethane (PU) foam in several layers with a total thickness of 7.6 to 15.2 cm. The PU-foam hardens into a self-supporting dome, which can carry the reinforcement and concrete layers. Hangers are attached to this PU-foam to attach the reinforcement rods to, see figure 4.20. After all the reinforcement is hung onto the PU-foam, steel fiber concrete is sprayed in successive layers on the interior of the PU-foam until it reaches a total thickness of ca. 6.4 cm. The successive spraying of the layers prevents deforming of the PU-foam layer together with the pneumatic membrane. When the concrete is hardened, the formwork membrane is removed. If necessary a tension ring is positioned around the lower portion of the dome to hold the walls preventing an outward deformation under the weight of the upper structure.

The Monolithic Dome Institute (MDI) in Italy, Texas, has built domes with a similar principle that can reach a diameter up to 300 meter if ribs are added. Three brothers - David, Barry and Randy South - built and patented the first Monolithic Dome in 1975. The Monolithic Dome has several improved features, such as an improved way of determining the depth of the foam layer during spraying, and stronger hangers. Furthermore, several alternative concrete mixtures and construction variations are studied at the center in Texas. The Monolithic Dome Institute experimented with steel fibers as primary reinforcement. They concluded that it does not work in thin shells; the experimental shells developed cracks or failed completely. Reinforcement bars add the best tension strength. Moreover, the extra costs for the steel fibers will pay for hanging the reinforcement bars.

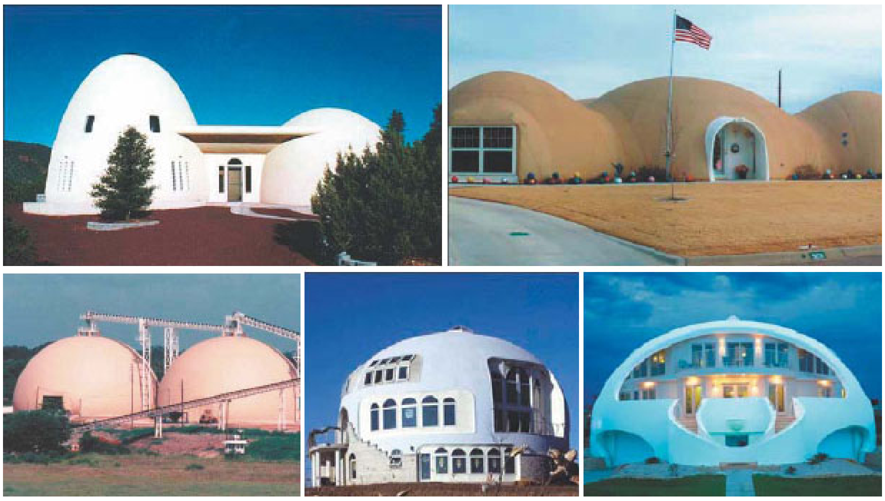


Figure 4.21: Domes constructed by MDI; in Sedona (Arizona), Shamrock (Texas), storage tanks in Port of Florida(Texas), Manitowoc (Wisconsin), Pensecola beach (Florida) [Hennik, 2005]

Monolithic domes are built in various shapes, see figure 4.22. The basic shapes are part of a sphere or an ellipsoid. Shapes that are more complex consist of cutting domes or domes connected with flat or curved parts. However, these irregular shapes do complicate design and fabrication. For instance, when a small dome is attached to a large dome, the membrane of the large dome is more tensioned, and thus more stretched. The large dome tends to overpower and pull the small dome, causing small changes in shapes without causing problems.

Apart from these domes , they also developed 'Ecoshells' for low cost housing in hot climates, garages and storages(figure 4.23. These are sprayed on the exterior of the formwork, like the domes of Neff. The formwork is inflated to 1.5 kN/m^2.

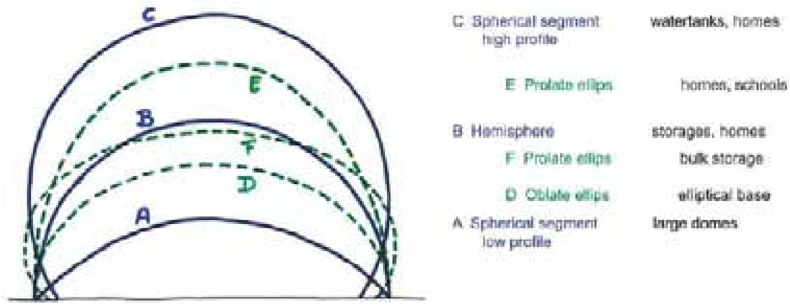


Figure 4.22: Basic shapes used by MDI, with their uses [Hennik, 2005]

Figure 4.23: Construction of an 'Ecoshell' [Hennik, 2005]

More than 1400 domes are engineered by the consulting engineer of MDI for churches, offices, storages, schools, theaters, water tanks, and supermarkets. The MDI provides training as well. The Solid House Foundation built their first project in Bolivia with formwork and training of the MDI (EcoShells).

Figure 4.24: Pirs, possible openings in domes [Hennik, 2005]

In 1988 the French company PIRS started building domes, using the same method as the MDI. The only major difference is that PIRS applies Vethane foam instead of PU-foam. Once the PIRS domes have attained their final solidity, practically any kind of opening is possible; skylights, enormous doorways and windows of any shape or size can be cut(figure 4.24). PIRS uses part of spheres and combinations of them. The dimensions of the PIRS domes vary in diameter from 14 m to 60.4 m and in height from 5.5 m to 37 m.

Figure 4.25: Pirs, left: Leisure park in Poland, right: Public Aquarium in Cabries, France. [Hennik, 2005]

4.4 Alter Architecture

Like alter-globalisation, "alter architecture" envisages a different way of conceiving of the built environment, one that takes into account the constraints linked to modern society as well as the need to protect the environment and the characteristics of the site in which it develops (climate, lifestyle, etc.). Alter architecture is about architectural forms from all over the world, from diverse nature, whether urban or rural, permanent or temporary, created by architects or not, of different form, but essentially the same in substance. They have in common their links to cultural and constructive traditions, respecting their built or natural environment and having recourse to recyclable materials that consume little energy and cause little pollution [Exhibition, 2006].

Figure 4.26: Mousgoum Dome in Cameroon [Exhibition, 2006]

In this chapter a few examples of alter architecture in relation to materials or domes are given, that might contribute to ideas for improvement of the SHF domes.

Part of alter architecture is vernacular architecture, which means 'built by non-academically trained builders'. True vernacular architecture is most apparent in the third world where indigenous populations produce their own shelter based on traditions of using locally available materials. Examples are the domes of the Mousgoum in Cameroon(figure 4.26), the dwellings of the Dogo in Mali or the structures of the M'Zab in South Algeria. Note the use of thick walls to absorb the weight of the roof and regulate temperatures.

But vernacular architecture can be found in the western world as well. In the seventies and eighties a lot of people started to experiment with building themselves. In books like 'Shelter' [Kahn, 1973] one could find ideas and recommendations for building homes. A considerable part of the book was dedicated to building domes. Around that time the geodesic domes were very popular. People built geodesic domes with plywood, recycle materials and ferrocement. 'Dome communities' were built, for example 'Drop City' on the outskirts of Trinidad , Colombo in 1967. Domes were built of all kind of recycle materials, especially panels chopped out of car tops. In 1969 an experimental high school was set up in the Santa Cruz mountains. The students lived in self-built plywood domes. Sealing the domes from water turned out to be the biggest problem. Some domes were covered by asphalt shingles to keep the water out.

Nowadays 'vernacular builders' in the western world especially focus on the use of natural materials and old techniques. Organizations like 'Green Home Building' and 'Planetary Renewal' advice people on their websites on all kind of materials and techniques, especially building with earth based materials [Ver-

Figure 4.27: From left to right: Maquette by Kalberer, dome by Hubbel, stacked wood structures of Delaroziere [Exhibition, 2006].

Figure 4.28: Low-cost housing in Ricaurte Colombia,2004 [Exhibition, 2006]

nacular, 2006]. Examples are adobe [3], cob [4], wattle and daub [5], strawbale, Compressed Stabilized EarthBlocks [6] and rammed earth [7].

Alter architecture by architects is mostly focused on the use of natural and waste materials. Marcel Kalberer was inspired by the Mesopotamiers who built their houses of reed about 5.000 years ago. Since 1984 he designs 'living' structures of braided willow twigs. The sculpter James Hubbel builds domes out self-made bricks, steel from dumped reinforced concrete and recycled glass. The architect Delaroziere is looking for possibilities built structures made of stacked rest wood [Exhibition, 2006].

Most interesting in the scope of this Master's thesis are the architects that aim at solutions for housing in third world countries.

Simon Velez (Colombia) has developed assembling techniques with cement-filled guadua bamboo. He is using his knowledge of building with bamboo for the development of earthquake resistant low-cost housing in Colombia, saving about 45% on building costs.

[3] Adobe soil has between 15 and 30% clay. It can be mixed into mud and applied directly to form a wall or it can be dried out to form bricks.

[4] Cob is adobe mixed with straw.

[5] Wattle and daub is cob or adobe applied on a wooden framework.

[6] CSEBs, bricks made by compacting earth in a form.

[7] Rammed earth is compacted earth.

Since 1980 Michael Reynolds designs according to the principle of the 'autonomic house'. He set up the foundation 'Earthship' [Earthship, 2006], which experiments with his ideas in the desert of Taos in New Mexico. An 'Earthship' incorporates many 'green' principles; utilizing recycled and low embodied energy materials, passive solar heating and cooling, photovoltaic power system, catchwater, solar hot water, gray water and black water treatment systems. For example water is recycled 3 times and bearing walls are made of discarded car tires filled with compacted earth or mortared aluminum cans. After the tsunami of December '04 he used these principles to construct a house out of waste materials in a severely hit area of the Andaman Islands. In the Netherlands the Foundation Owaze [Owaze, 2006] propagates his ideas.

The Iranian-American architect Nader Khalili believes that housing needs for refugees can be addressed by earth construction. After extensive research into vernacular earth building methods in Iran, followed by detailed prototyping, he has developed the sandbag or 'superadobe' system (1982). The prototypes have not only received California building permits but have also met the requirements of the United Nations High Commissioner for Refugees (UNHCR) for emergency housing. Both the UNHCR and the United Nations Development Programme have chosen to apply the system, which they used in 1995 to provide temporary shelters for a flood of refugees coming into Iran from Iraq.

The basic construction technique involves filling sandbags with earth and laying them in courses in a circular plan. The circular courses are corbelled near the top to form a dome. Barbed wire is laid between courses to prevent the sandbags from shifting and to provide earthquake resistance. The system is particularly suitable for providing temporary shelter because it is cheap and allows buildings to be quickly erected by hand by the occupants themselves with a minimum of training. Each shelter comprises one major domed space with some ancillary spaces for cooking and sanitary services. Incremental additions such as ovens and animal shelters can also be made to provide a more permanent status and the technology can also be used for both buildings and infrastructure such as roads, kerbs, retaining walls and landscaping elements. In 1991 Khalili founded the California Institute of Earth Art and Architecture [Calearth, 2006], a non-profit research and educational organization which teaches people how to build homes, schools, and other buildings using Khalili's methods. 'Vernacular builders' have applied his technique with burlap sacks or rice sacks filled with vermiculite, perlite, rice husk and other materials [Hart, 2006] [Rice-hulls, 2006].

Figure 4.29: Reynolds, Andaman Island project and other 'earthships' [Earthship, 2006]

Figure 4.30: Dome built with superadobe system of Khalili [Calearth, 2006]

Chapter 5

Form

As a result of the literature study several options were identified to alter (and potentially optimize) the shape of the shell. These options are grouped into 3 categories, being 'curvature', 'mass' and 'texture'. The zero alternative is a shell with uniform thickness and hemispherical shape.

5.1 Curvature

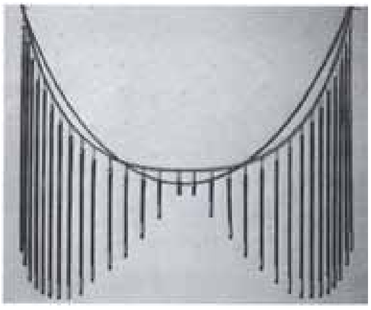


Figure 5.1: Model 'ideal' dome cross section [Minke, 2000]

The ideal cross section for a dome (of constant thickness) under dead load is that which creates only compressive forces going downwards (meridional). This means a form that creates neither tensile nor compressive ring forces. If the cross-section has the shape of a catenary, compressive ring forces will occur. This might be disadvantageous if openings have to be cut into the dome, or if it is a dome of large span. To create this 'ideal' cross section a chain is loaded by weights that proportionally represent the decreasing area from the base to the apex of a hemisphere. The model can be seen in figure 5.1.

However, since the ideal form is not spherical, its segments have a slightly different area than those of a hemisphere. Therefore, this procedure has to be considered a first approximation. Greater accuracy can be achieved by successive iterations, substituting the actual changing radii of curvature of the segments

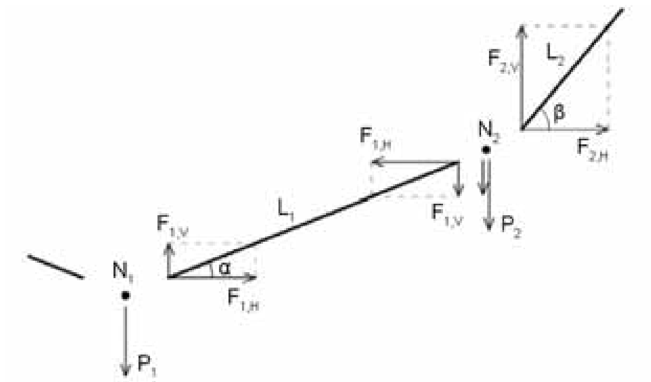


Figure 5.2: Model used to produce plots of hanging models in Excel

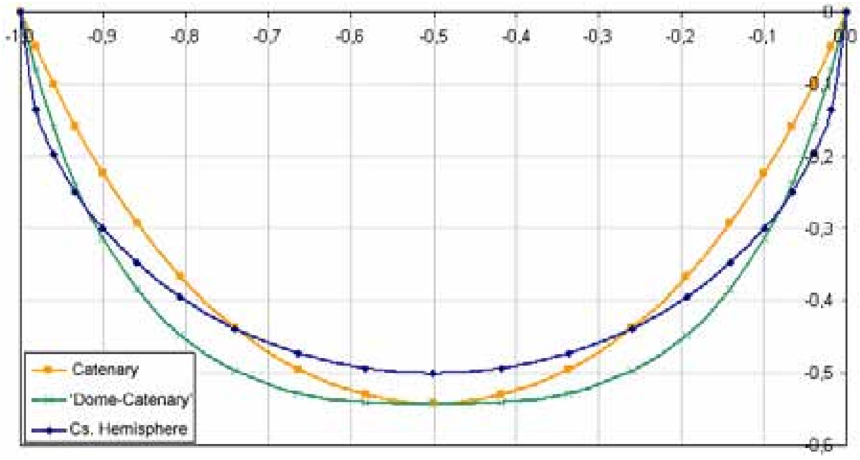


Figure 5.3: Plot of a catenary, a semicircle and the cros-section of a dome of constant thickness under dead load that creates only meridional compressive forces (no ring forces), source: author

measured from the model and adjusting loads according to the surface areas of the segments thus calculated. One could also start with an ellipse instead of a hemisphere as a basis.

The model shown in figure is used to plot a catenary in Excel. The angle α, the angle between the first element left of the center, determines the horizontal force in the cable, which is constant over the length of the cable and therefore influences it's final form. The span and length of the cable and the magnitude and the number of loads can be changed. To approximate the form of the 'ideal' cros-section as mentioned above, the load on each element should be proportional to the average circumference of the dome in this section. The circumference is proportional to the radius, which is determined by the horizontal position on the cross section of the dome. Therefore the nodes are loaded proportional to their horizontal position. The curve of this 'dome-catenary'is compared to the cross section of a hemisphere and a catenary in figure 5.3.

As the cross section of the hemisphere is outside the 'dome-catenary' at the base, ring tension forces occur in this part of the hemisphere. Correspondingly compressive ring forces occur in the top part of the hemisphere. The two curves intersect at an angle of 38° to the horizontal (measuring from the center of the hemisphere's cross section). The catenary has a much steeper curve and remains completely inside the 'dome-catenary'. Consequently a dome with an equivalent cross section would be completely under compression.

Nevertheless, when unequally distributed loads occur in addition, as is the case with snow or wind loads, the initial coincidence between the structural task and the form is no longer given. Resulting bending stresses have to be taken into account, a minimum amount of rebar may still be needed.

From a structural point of view it would be interesting to experiment with the curvature of the dome. A change in curvature can significantly decrease the amount of rebar needed.

5.2 Mass

Another way to decrease ring tension forces is varying the mass of the shell over the cross-section. For adobe dome building this principle is adopted and Romans applied it for the construction of the Pantheon (see section 4.1). Mass is added to the lower part of the cross section and mass is reduced in the upper part of the dome's shell. This can be achieved in different ways (see figure 5.4):

- Decrease shell thickness in the direction of the apex
- Impose extra weight on the lower part of the shell
- Use a more dense material for the lower part respectively less dense material for the upper part of the dome
- Make an opening in the top of the dome

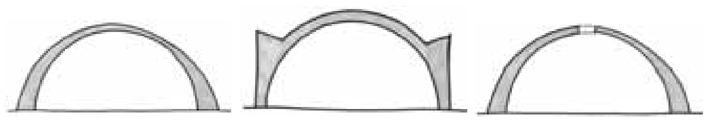


Figure 5.4: Varying mass shell over the cross section

5.3 Texture

The pneumatic formwork is made of material that does not stretch in either way and can be adjusted to produce other shape than a smooth hemisphere. In section 5.1 the curvature was varied, but also the formwork could be given a texture to achieve alternative shapes. Tension forces might be concentrated at specific part of the cross section and reinforcement could be applied more efficiently.

As can be read in section 4.2, both in nature and structural design ribs are used to provide stiffness to a structure. Forces are concentrated in ribs and consequently material can be saved in other parts of the structure. It would be interesting to study the influence of ribs on the structural behavior of the shell. Examples are shown in figure 5.5.

Figure 5.5: Giving the shell a different texture

5.4 Combination

These different categories could be combined as well. For example a texture introduced on the upper part of the formwork could reduce the mass in this part of the cross section (see figure 5.6). Or a different curvature could be combined with a decreasing shell thickness in the direction of the apex. Even surrounding forms could be added to the texture to provide for extra mass (see Hagia Sophia in section 4.1). Examples are shown in figure 5.6.

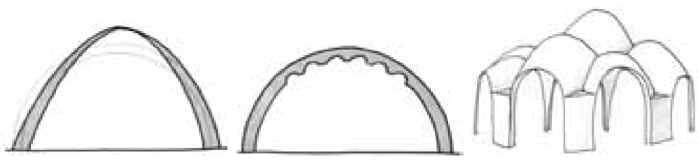


Figure 5.6: Combinations of the different categories

In figure 5.7 an overview is given of examples of the three categories and their combinations.

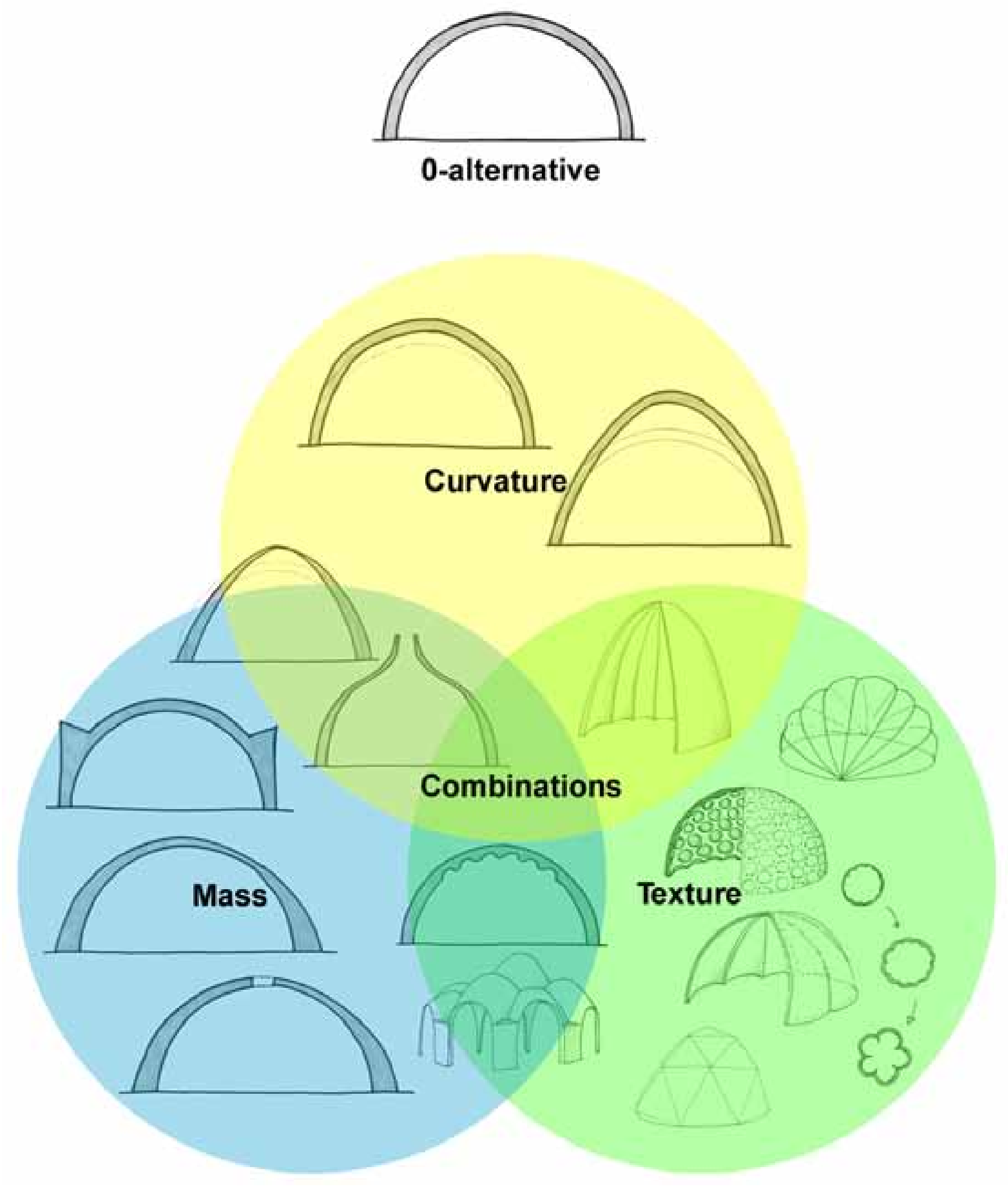


Figure 5.7: Overview of form alternatives

Chapter 6

Materials

Several materials have been studied for application in Sri Lanka. In this chapter several options are discussed to conclude with an advice for the time being. The selection is based on availability, costs, durability, simplicity, reliability and maintenance level.

Earth based materials : Earth based materials as mentioned in section 4.1 and 4.4, i.e. adobe, are easy to use, available in Sri Lanka and very cheap. Unfortunately these massive materials loose their cooling properties in a tropical climate as temperatures stay high at night. More important however, these materials need to be adequately protected against excessive weather. Straw-clay blocks for example are vulnerable to moisture penetration, cob needs to be protected against moisture as well. Concerning adobe: 'In a wet climate a good roof and a foundation to keep the adobe off the ground should be provided for. Sometimes adobe is stabilized with a small amount of cement or asphalt emulsion added to keep it intact where it will be subject to excessive weather. Some adobe buildings have been plastered with Portland cement on the outside in an attempt to protect the adobe, but this practice has led to failures when moisture finds a way through a crack in the cement and then can't readily evaporate.' [Wilson, 2006]
Considering the monsoon rains in Sri Lanka and the fact that domes are open to all of the weather, earth based materials are not considered a fruitful alternative for dome construction in Sri Lanka.

Organic materials : Housing on stilts of lightweight materials like bamboo and wood would suit the climate very well. However, wood and bamboo are either imported or logged illegally and therefore expensive and scarce in Sri Lanka. For help organizations the possibility of illegal logging alone is enough to avoid the use of wood. Leaves of i.e. palms and rice are readily available to make walls and roofs. From a building physics point of view these materials are ideal as the air can blow through, but the leaves have to be replaced quite often as they are not stable in rough weather and prone to attack by insects. Also fire safety has to be taken into account as the area is subject to regular riots. The SHF and its stakeholders care for a durable, low-maintenance solution for the village. Altogether a dome construction completely made out of organic materials is not further studied as an option for the time being.

Natural fiber composites use organic materials in combination with a polymer resin[1]. Jute and coir based composites have been developed as substitutes for plywood and medium density fibre boards. Panel and flush doors have also been developed out of these composite boards especially for low-cost housing needs. Other product development activities include usage of sisal fibre based composites as panel and roofing sheets. As coir is abundant in Sri Lanka it is interesting to look further into this material. A study can be found in section 6.1.

Cement based materials : Reinforced concrete, the nulconcept, has proved to be durable and simple enough for this application. Reduction of reinforcement would create considerable cost and time -savings as prices for steel rods are relatively high and application of the steel rods is the most complicated and time consuming part of the building process. To reduce the amount of primary reinforcement, fiber reinforced concrete might be an option. Another option might be ferrocement, as wire mesh is probably cheaper and more easy to apply. These two are discussed in section 6.2 and 6.3.

Also considered is the use of lightweight concrete. Lightweight concrete, weighing from 560 to 1850 kg per cubic meter, has been used over 50 years. The compressive strength is not as high as ordinary concrete. Among its advantages are less need for structural steel reinforcement, smaller foundation requirements and better fire resistance. A disadvantage is it's higher costs and it may shrink more upon drying. Lightweight concrete may be made by using lightweight aggregates, or by the use of foaming agents, such as aluminum powder, which generates gas while the concrete is still plastic. Natural lightweight aggregates include pumice, scoria, volcanic cinders, tuff, and diatomite. Lightweight aggregate can also be produced by heating clay, shale, slate, diatomaceous shale, perlite, obsidian, and vermiculite. Industrial cinders and blast-furnace slag that has been specially cooled can also be used.

[1]Strictly spoken natural fiber reinforced concrete is a natural fiber composite as well

6.1 Natural Fiber Composites

Composites are hybrid materials made of a polymer resin reinforced by fibres, combining the high mechanical and physical performance of the fibres and the appearance, bonding and physical properties of polymers. Due to the wide variety of available manufacturing processes, each resulting in their own characteristic products, the design possibilities are numerous. Consequently, a composite product and its manufacturing process can be chosen to best fit the environment in which the products will be made and used. Besides the technical feasibility, manufacturing of composites becomes also financially feasible when using domestically grown natural fibres in combination with simple manufacturing processes. Potential products are roofing panels, fluid containers, bridges and small boats [van Rijswijk et al., 2003].

Due to an occurence of a wide variety of natural fibres in the country, Indian researchers have directed efforts for quite some time indeveloping innovative natural fiber composites for various applications. An example is the wide range of wood substitutes developed for the housing and construction sector [TIFAC, 2006].

Three kinds of natural fibers can be identified:

- Fruit fibers, such as cotton and coir (coconut fiber)
- Bast fibers, such as jute, flax, ramie and hemp
- Leaf fibers, such as sisal

A project that underlines the potential for natural fiber composites for use in dome construction is the construction of water tanks in Guatemala. Early 90's, the Centre of Lightweight Structures TUD-TNO[2], The Netherlands, executed a United Nations project in Guatemala for the local manufacturing of primitive composites. Starting point formed Guatemala's enormous jute resources. With trucks water-soluble resin-powder (modified urea-formaldehyde) was transported to remote areas, where rainwater was used to impregnate the locally manufactured jute textiles, see figure 6.1. Large PVC balloons where used as mandrel to manufacture water tanks and latrines. After the project, production of these products for local use continued successfully.

The resin used for the project in Guatamala is normally used in combination with wood fibers, i.e. for joists. Some properties of standard resins based on melamine and ureumformaldehyde according to the 'Stork Lijmen':

- 'Can be supplied in powder or in fluid form by i.e. BASF'
- 'A hardener should be added (available in powder or liquid)'
- 'After mixing these with water the resin can be used for two hours'

[2] Concerning natural fibre composites, the faculty participated in the Biolicht project where applications for trucks, trailers and busses were developed. As a result of that project the Delft University manufactured many prototypes with flax fibre composites, among them a catamaran, an automotive roof panel and sandwich structures.

Figure 6.1: Construction of water tanks with biocomposite in Guatamala [van Rijswijk et al., 2003]

- 'The woodfibers and resin should be bonded by high pressure'
- 'The hardened resin is not sensitive to sunlight, heat resistant to 80 °C and not harmful to health and environment'
- 'In fluid form contact with the skin can cause irritation, therefore it is advised to protect the hands with vaseline and to wash them regularly'
- 'High bending strength and stiffness'

According to research done in i.e.India [TIFAC, 2006] the use of natural fibers instead of wooden fibers with a resin should not be considered a problem. However it may be necessary to treat the fibers in order to reduce water absorbtion. Also circumstances such as high temperatures and humidity influence the quality of the product. Therefore it is very difficult to determine properties like the Young Modulus. Further research and tests on site would be necessary.

Bonding by high pressure is only necessary if high surface quality is needed and short fibers are used (i.e. in case of formica). By using woven mats of natural fibers less pressure is needed as a high level of cohesion is already present. The resin could be applied on the mats by a roller. An option for improved compaction is to cure while using a vacuum. After covering the formwork and the treated mats with a foil, a vacuum is applied. Excess air is removed and the atmospheric pressure exerts pressure to compact the composite.

Another manufacturing process is resin injection. The mats are placed on the formwork and covered by a foil. A tube connects the space between formwork and foil with a supply of liquid resin, which is transferred through the mould through vacuum pressure, imprecnating the fibers. After curing the foil is removed.

Formaldehyde is considered toxic and should be used in the open air. Nowadays UF and MF resins are available that emit very little or no formaldehyde. If used in combination with a reusable inflatable formwork the formwork will need a teflon coating and treatment to avoid bonding with the resin.

Rik Brouwer of Delft University of Technology (Structures & Materials Laboratory and the Centre of Lightweight Structures TUD-TNO) has done research

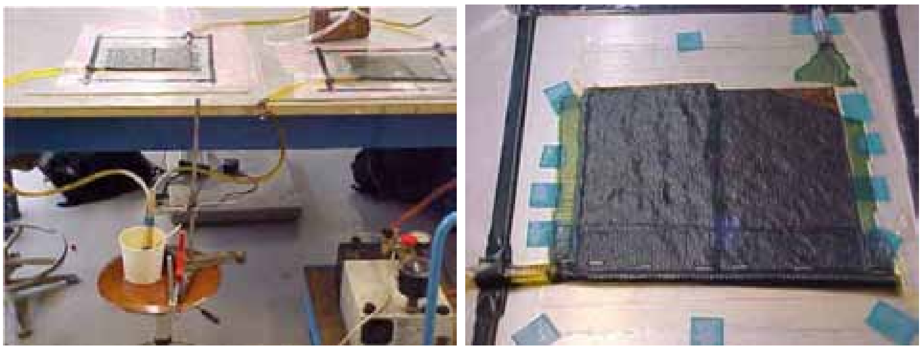


Figure 6.2: The setup of the vacuum injection technology at the laboratory of the TU Delft. The second picture shows an almost complete impregnated coir mat. The resin is injected with polyester resin from the bottom left corner, while vacuum is applied at the top right corner. In that corner a small triangle of the mat is still dry, which can be seen by a lighter brown color. [Brouwer et al., 2003]

on composite applications using coir fibres in Sri Lanka. The background of the research that was carried out is the awareness that the demand of coir and coir products is slowly decreasing and that other profitable markets have to be found for this commodity in Sri Lanka [3]. The best way to bring the existing coir industry to a higher level is the development of new coir products with higher added value. One possible technology that could fulfil this goal is the use of coir fibre in composite components.

Among other techniques vacuum injection technology was tested with polyester resin on coir mats (figure 6.2). The flexural strength varied from 29 MPa 47 MPa and flexural stiffness of 2,91GPa 2,99 GPa for coir fibre loading between 20% and 40% and different fibre treatments. A model boat hull (scale 1:3) was manufactured using the vacuum injection technology, see figure 6.3. The vacuum applied 1 bar pressure on the whole surface of the product. The coir mat turned out to be very suitable, it had a good permeability and it allowed an easy placement into the mould. Impregnation with polyester was easy, partially based on a natural good adhesion between cellulose and polyester. This was also observed during the manufacture of small samples.

[3] Sri Lanka is the single largest supplier of coir fiber to the world market and together with India accounts for almost 90% of global coir exports. Although Sri Lanka has traditionally been the lead exporter of coir fiber and pith, India holds the dominant position in terms of revenue generated by the industry, given the higher value-added component of its coir exports. In view of the relatively small size of its domestic market, the production of raw fiber and related goods in Sri Lanka are almost exclusively driven by external demand (in contrast to India which has a large domestic market). Although global coir production since the early 1990s has grown by approximately 6% per annum, the industry continues to be threatened by synthetics, stagnating world coir prices and the poor and declining profitability of small mills, which form the basis of the coir industry in Sri Lanka. Future industry success therefore lies in improved product quality and consistency, as well as the expansion of existing markets and development of commercialized new applications for coir that involve in-country value addition [USAID, 2006].

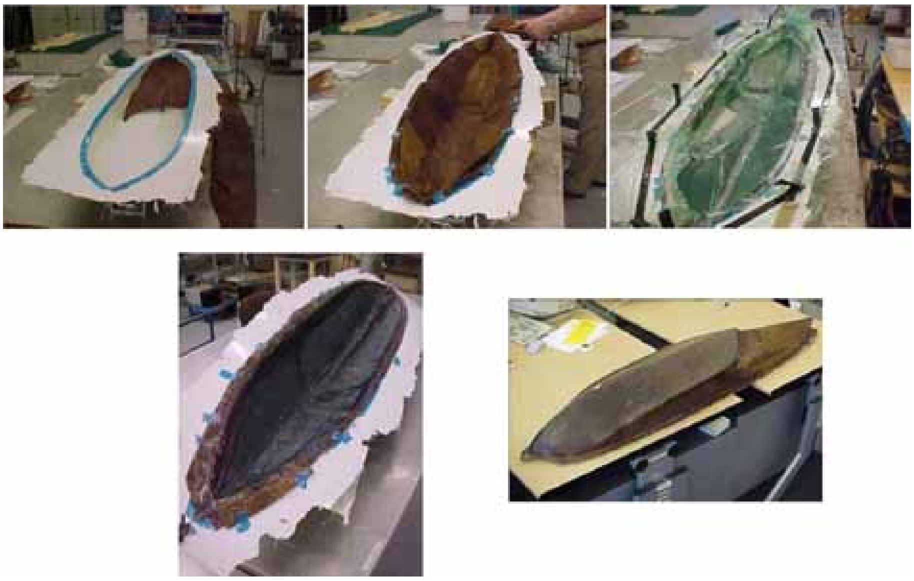


Figure 6.3: After the dry coir mats are placed into the mould, the material is bagged, the package is sucked vacuum and the resin is let in. The last pictures show the resulting product. [Brouwer et al., 2003]

However, whether this material is suitable for dome construction asks for more research and experiments. Not only on the load bearing capacity and the durability of the material, but also whether it can create a house that is comfortable to live in. It might be a very interesting option for application in (temporary) shelters as it can be built very fast with local materials.

6.2 Fiber reinforced concrete

Fibers are typically not used in concrete as a direct replacement for conventional reinforcing steel applications where flexure and tensile forces are predominant. In terms of strength, conventional reinforcement is more efficient, and the discontinuous nature of fibers precludes their use in applications where their tensile resistance would be needed to ensure structural integrity. Fibers do not cause an appreciable increase in the tensile strength of concrete, unless used at very high dosages, but they do provide some residual post-cracking strength which gives a less brittle and less vulnerable material [Massicotte and P.H.Bischoff, 2000].

Unfortunately fibers can not always be relied upon with a great deal of confidence, since behavior depends so much on the type and dosage of the fiber used, interaction of the fiber with the concrete matrix, as well as orientation and dispersion of the fibers within the member being considered for design. As a result experience and knowledge is a provision for successful application [Massicotte and P.H.Bischoff, 2000].

Several types of fibers can be defined:

- natural fibres
- steel fibres
- glas fibers
- synthetic fibers (i.e. polypropylene)

From several points of view the application of 'high-tech' fibers such as glas, steel and synthetic fibers is not fit for use in Sri Lanka:

simplicity: Professionals are needed for succesful application as the quality is very dependent on the application process.

reliability: Behavior depends on many different aspects, making it unreliable as a replacement for conventional reinforcing in Sri Lanka.

structural: Fibers do not cause an appreciable increase in the tensile strength, for ring tension forces conventional reinforcing would still be necessary.

availability: Compacting of fiber reinforced concrete is important. Currently the concrete for the SolidHouses is not compacted and applied by hand. If fiber concrete of a considerable quality is desired, the mixture is preferably shotcreted on the formwork. Besides, some fibers (i.e.steel fibers) make the mixture very uncomfortable to handle by bare hands. However this increases the energy demand at the building site as well as the initial investment.

Therefore concrete reinforced by low-tech, natural fibers such as sisal, bamboo, wood and coir (coconut husk) were studied.

Asbestos cement was the first industrialized construction material, which as reinforced with natural fibers (the mineral asbestos) and used on a large scale. Due to its exceptional mechanical behavior and low cost it is until today standing out among the construction materials. However, asbestos is considered to be a potential health hazard to humans and animals, mainly during extraction and handling, causing cancer and fibrosis [Ghavami, 2001].

Although natural fibers exist in abundance and are readily available at low cost, they have many inherent weaknesses such as low elastic modulus, high water absorption, susceptibility to fungal and insect attack, lack of durability and variability of properties among fibers of the same type [Swamy, 1990].

The lack of durability is mainly caused by the fact that natural fibers are chemically decomposed in the alkaline environment of the cement matrix, resulting in brittle composite which has reduced capacity to cracking [Hussin and Zakaria, 1990]. In a tropical environment, detoriation is even faster due to greater humidity and higher temperature [Guimaraes, 1990]. To stop or slow down the embrittlement process the alkalinity of the pore water of the cement matrix has to be reduced. This can be achieved by replacing part of the Portland cement with a highly active pozzolana such as silica fume, fly ash or rice husk ash [Hussin and Zakaria, 1990].

Several projects deploying natural fibers for roofing sheets have been carried out quite successfully in Sri Lanka. Dr P.Stroeven of the TU Delft has supervised a range of projects on this subject in Sri Lanka [Blom et al., 1999]. On the long term the sheets do loose their strength, which is not a problem as they are not subject to large forces and can be replaced easily. However the latter is not the case when applying fiber reinforced mortar for a monolithic dome.

The differences in strength and mode of failure will generally be accentuated if the fibres fail to provide any reinforcing effect. A larger product like a sheet is thus more likely to give trouble, crack and fail suddenly than a geometrically smaller product like a tile. This partly explains why a high incidence of cracking and considerable performance deficiencies have been reported by users of natural fiber reinforced sheets and tiles, limiting useful life of these products to no more than 2 to 4 years. Extensive surveys have shown clearly that it is not the technology or the product that failed but the inherent destructive effect on the fibres of the alkalinity of the cement, particularly in tropical envirionments, and the lack of adequate standards of roof construction and installations that were responsible for these failures.

The results of flexural tests of cocunut fibre reinfoced thin cement sheets can be found in table 6.1. Unreinforced concrete having a tensile strength of $1,8\ N/mm^2$, an increase in flexural strength is clearly shown. However this is not even near to the tensile strength of steel, being 350 N/mm^2 for FeB400. Therefore fibres can increase the cracking strengh, but can not be take tensile stresses like steel reinforced conrete concrete can.

Table 1. Flexural strength of 500 x 100 x 10 mm plates

Fibre content	Ave. self weight (Kg.)	Ave. density ($\times 10^3$ Kg/m^3)	Flexural strength (N/mm^2) 28 days	3 months	6 months	1 year
Fp1 – 1%	1·05	2·18	4·86	3·31	4·33	3·25
Fp2 – 2%	1·00	2·05	5·23	4·41	2·36	3·20
Fp3 – 3%	1·05	2·01	6·33	5·18	4·85	5·12
Fp4 – 4%	1·09	2·19	6·37	7·20	5·30	8·03
Fp5 – 5%	1·11	2·21	4·43	6·98	4·00	7·13
Fp6 – 6%	1·12	2·25	5·35	5·46	6·50	4·84
Control – 0%	1·08	2·17	4·33	4·08	3·32	3·63

Table 6.1: Flexural strength test coconut-fiber-reinforced thin cement sheets [Hussin and Zakaria, 1990]

Summarizing:

simplicity: Because of the vulnerability of the fibers more care has to be taken to use the right mix proportions and to include a component that reduces the alkalinity of the environment. However the process is relatively simple.

reliability: Variability of properties among fibers of the same type reduce the reliability significantly. Therefore fiber reinforced concrete is better fit for smaller products like tiles.

structural: The fibers contribute to the flexural strength to the extend of cracking strength. A disadvantage is the low elastic modulus and high water absorption of natural fiber reinforced concrete.

availability: Good, natural fibers exist in abundance and are readily available at low cost in Sri Lanka.

costs: Very low.

durability: The alkaline environment reduces the lifetime of the fibers significantly, especially in tropical climates like Sri Lanka's. Also, fungal and insect attack threaten the durability of the fibers.

From this summary is concluded that natural fiber reinforced concrete is not suitable for application in a monolithic dome shell subject to tensile stresses and open to all weather that is expected to have a lifetime longer than ten years.

6.3 Ferrocement

Figure 6.4: Ferrocement Geodesic Dome in Sri Lanka [geocities, 2006]

In 1848 Jean Louis Lambot of France constructed several rowing boats, plant pots, seats and other items from a material he called 'fercicement'. Lambot construction consisted of a mesh or grid reinforcement made up of two layers of small diameter iron bars at right angels to one another. The structure was then plastered with a cement mortar which only thinly covered the reinforcement. Several people adopted his methods, of whom Pier Luigi Nervi is the most famous. Nervi's development evolved from the fundamental concept behind reinforced concrete that concrete can withstand large strains in the neighborhood of the reinforcement, the magnitude of the strains depending on the distribution and subdivision of the reinforcement throughout the concrete. Extending this concept was the observation that the elasticity of a reinforced concrete member increases in proportion to the distribution and subdivision of reinforcement throughout the mass. Nervi noted that "the material obtained in this way had very little in common with normal reinforced concrete, possessing as it did, the mechanical characteristics of a completely homogenous material". The composite exhibited two important properties: extreme elasticity and a great resistance to cracking, which, together with its elimination of the need for formwork, has a potential for mass production. Nervi chose to describe the method of construction as 'ferrocemento' [Robles, 1985].

Meanwhile ferrocement boats and houses have been constructed all over Asia. In Indonesia numerous spherical domes for mosques have been constructed with ferrocement. The National Building Research Organization Sri Lanka even trains people in ferro cement technology for production of low-cost building materials and products [NBRO, 2006].

Ferrocement can be considered as a special form of reinforced concrete, however, it exhibits behavior so different from conventional reinforced concrete in performance, strength and potential applications that it is classified as a separate material. Ferrocement uses wire mesh, rather than heavy rods or bars, as the primary part of its metal reinforcement. Sand rather than a mixture of sand and stone in graded sizes is used in its concrete mix [Abercrombie, 1977]. Water to cement ratios commonly used in ferrocement production vary between 0.34 and 0.55, by weight.

Skeletal steel, as the name implies, is generally used for making the framework of the structure upon which layers of mesh are laid. Both the longitudinal and transverse rods are evenly distributed and shaped to form. The rods are spaced as widely as possible up to 30 cm apart. Steel rods of different kinds are used in ferrocement construction. Their strength, surface finish, protective coating and size affect their performance as reinforcing members of the composite. In general, mild steel rods are used for both longitudinal and transverse directions.

In some cases high tensile rods and prestressed wires and strands are used. Rod sizes varies from 4.20 mm to 9.5 mm, whereas 6.25 mm is most common [Robles, 1985].

The wire mesh generally consists of thin wires, either woven or welded into a mesh, but the main requirement is that is must be easily handled. The function of the wire mesh and reinforcing rod in the first instance is to act as a lath providing the form and to support the mortar in its green state. In the hardened state its function is to absorb the tensile stresses on the structure which the mortar would not be able to withstand. The mechanical behavior of ferrocement is highly dependent upon the type, quantity, orientation and strength properties of the mesh and reinforcing rod [Robles, 1985].

It should be pointed out that the material used in the third world is not ferrocement in the traditional sense (as used by Nervi) which should require a large number of meshes. For economy, the amount of mesh reinforcement is reduced to only 2 layers. The result is, in effect, an under-reinforced form of ferrocement [Tatsa, 1994].

In general, ferrocement structures need no protection unless it is subjected to strong chemical attack that might damage the structural integrity of its components. A plastered surface can take a good paint coating. In terrestrial structures, ordinary paint is applied on the surface to enhance the appearance [Robles, 1985]. The ideal ferrocement shell is a shell which is mainly subjected to compressive membrane forces and little flexure under different loading conditions [Wieland, 1985].

The greatest factors leading to the acceptance of ferrocement are:

- Its basic raw materials are readily available in most countries
- It can be fabricated into any desired shape and adapted to environmental and traditional custom of the country
- The skills for ferrocement construction can be acquired easily
- Heavy plants and machinery are not involved in ferrocement construction
- It can be easily repaired and no maintenance is required
- It is suitable for mass production and construction on self-help basis
- Superior crack control to conventional reinforced conrete

Another valuable characteristic of ferrocement is that it may eliminate the need for separate layers of waterproofing. Indeed, at the new Sydney Opera House, architect Jrn Utzon's famous sail-shaped roofs (built of conventional reinforced concrete) have been covered with tile-surfaced panels of ferrocement which serve as waterproofing of the concrete below [Abercrombie, 1977].

6.4 Compressed Stabilised Earth Blocks

For additional straight walls in the domes and for additional (non-dome)buildings the use of Compressed Stabilised Earth Blocks is considered. Blocks or bricks could also be used to build domes and vaults, but this kind of masonry would ask for skilled labor. Besides the formwork would be superfluous.

The soil for a compressed earth block (CEB) is slightly moistened, poured into a steel press (with or without stabiliser) and then compressed either with a manual or motorized press. CEB can be compressed in many different shapes and sizes. The input of soil stabilization allowed people to build higher with thinner walls, which have a much better compressive strength and water resistance. With cement stabilization, the blocks must be cured for four weeks after manufacturing. After this, they can dry freely and be used like common bricks with a soil cement stabilized mortar. Since the early days, compressed earth blocks are most of the time stabilised. Therefore, they are called Compressed Stabilised Earth Blocks (CSEB).

Figure 6.5: Auram hollow interlocking Compressed Stabilized Earth Block 295 [Auroville, 2006]

Not every soil is suitable for earth construction and CSEB in particular. But with some knowledge and experience many soils can be used for producing CSEB. Topsoil and organic soils must not be used. Identifying the properties of a soil is essential to perform, at the end, good quality products. Some simple sensitive analysis can be performed after a short training. A good soil for CSEB is more sandy than clayey, it has proportions as shown in figure 6.6.

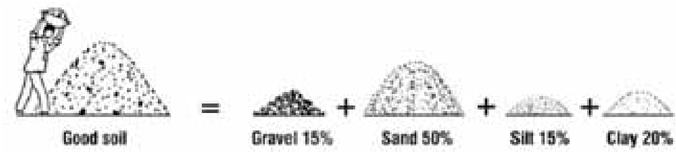


Figure 6.6: Proportions needed to produce CSEBs [Auroville, 2006]

Many stabilizers can be used. Cement and lime are the most common ones. Others, like chemicals, resins or natural products can be used as well. The selection of a stabilizer will depend upon the soil quality and the project requirements. Cement will be preferable for sandy soils and to achieve quickly a higher strength. Lime will be rather used for very clayey soil, but will take a longer time to harden and to give strong blocks. The average stabilizer proportion is rather low, being minimum 3% and averagely 5% in case of cement stabilisation and minimum 2% and averagely 6% in case of lime stabilisation.

Dry compressive strength at 28days (+10% after 1 year + 20% after 2 years)	4 to 6 Mpa = 40 to 60 Kg/cm^2
Wet compressive strength at 28 days (after 3 days immersion)	2 to 3 Mpa = 20 to 30 Kg/cm^2
Dry bending strength (at 28 days)	0.5 to 1 Mpa = 5 to 10 Kg/cm^2
Dry shear strength (at 28 days)	0.4 to 0.6 Mpa = 4 to 6 Kg/cm^2
Water absorption at 28 days (after 3 days immersion)	8 to 12% (by weight)
Apparent bulk density	1700 to 2000 Kg/m^3
Energy consumption (Ref. Development Alternatives 1998) (To be compared with kiln fired bricks (wire cut) = 539 MJ and country fired bricks = 1657 MJ	110 MJ

Table 6.2: Basic data on CSEB [Auroville, 2006]

These low percentages are part of the cost effectiveness of CSEB. The strength of a block is related to the level of compression and to the quantity of stabiliser.In table 6.2 some basic data about CSEBs can be found.

The Auroville Earth Institute in India recommends the use of heavy manual presses as equipment. Cheap light manual presses have a low durability, a low output and do not produce very well compressed blocks. A motorized press will present the advantage of a high productivity, with a better and more regular quality. But it will require energy and a more complicated maintenance, and its cost are much higher. Therefore, heavy manual presses are most of the time the best choice in terms of optimisation for the investment, output and quality ratio. This does also apply for the situation in Sri Lanka, where only additonal construction is done with CSEBs.The institute developed the heavy press shown in figure 6.7.

Auram press 3000 - Multimould press (16 moulds)

Figure 6.7: Press developed by the Auroville Earth Institute, India [Auroville, 2006]

Apart from the fact that CSEB is consumes less energy and pollutes less than fired bricks, they are most of the time cheaper than fired bricks. This will vary from place to place and specially according to the cement cost. The costs would be within these figures when using the AURAM press 3000 [Auroville, 2006]:
Labor: 20-25%
Soil and sand: 20-25%
Cement: 40 - 60 %
Equipment: 3 - 5 %

Advantages of CSEB:

- It uses local materials, saving on transport, fuel, time and costs.
- Requiring only little stabilizer the energy consumption is very low.
- A cost effective alternative, through the use of natural resources and semi skilled labor. The final price will in most cases be cheaper than fired bricks.
- It is a simple technology requiring semi skills that are easy to aquire in only a few weeks.
- CSEB allows unskilled and unemployed people to aquire a skill, creating a job opportunity.

6.5 Conclusion

Reinforced concrete and ferrocement are considered the most suitable materials to construct a monolithic dome with in Sri Lanka. Compressed Stabilized Earth Blocks could be introduced for additional straight walls.

Chapter 7

Climate Responsive Design

'Climate responsive design is the first and oldest craft of sedentary civilization. It is the knowledge of how to go about using houses and mansions for cover and shelter. This is because man has the natural disposition to reflect on the outcome of things. Thus it is unavoidable that he must reflect on how to avert harm arising from heat and cold'

Ibn Khalddum [1]

Comfort can be defined as the complete physical and mental well-being. Thermal comfort is a subset of the broad definition of comfort and relates to human and environmental factors. It is a complex area of study in fundamental terms, but for the designer the key issues relate to the building and environmental factors that affect comfort since these are amenable to manipulation in the design of the building . The main environmental factors affecting thermal comfort are air temperature, radiation, air velocity or air movement and humidity.

Research has pointed out that basic physiological response to thermal comfort is moderated by acclimatization to respective climate conditions. Thus people living in temperate climates may have a different sensation of climate to that of people living in tropical climates. The person-specific nature of comfort means that defining precise levels of comfort for buildings is impossible. However, research has generated some methods whereby designer can measure performance of buildings in terms of thermal comfort. The global effects are defined as the level of comfort provided by the environment in which the building is located. Thus for a given air temperature and humidity a zone of comfort can be found. The bioclimatic chart in figure 7.1 describes a set of temperatures and humidity at which it is thought humans feel comfortable [Hyde, 2000].

[1] [Hyde, 2000]

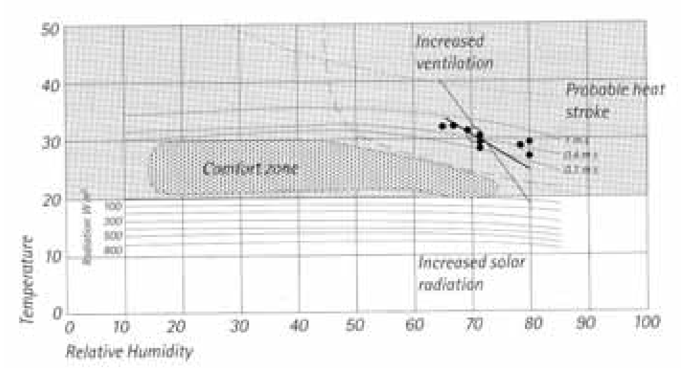


Figure 7.1: Bioclimatic chart showing climate modification strategies for extending the comfort zone by use of air-flow for over-heating and radiation for under-heating periods [Hyde, 2000]

7.1 Climate investigation

With dots the maximum average monthly temperatures and the relative monthly humidity in Trincomalee (see page 14) are added to the bioclimatic chart in figure 7.1. A circle indicates the location of the average yearly maximum temperature and the average humidity. The thick line shows the trend between the more humid winter and the hotter and slightly dryer summer, based on minimum and maximum average monthly temperatures and relative monthly humidity. The thin line uses monthly record maximum and minimum temperatures instead. The Sri Lankan hot humid climate clearly does not fit inside the comfort zone. Over-heating is prevalent during the whole of the year. Climate modification strategies are necessary to moderate the conditions of temperature and humidity in order to provide for a comfortable living. The use of air-flow can extend the comfort zone considerably, as is indicated by the lines in the bioclimatic chart.

General strategies for extending the comfort zone in case of over-heating are:

Airflow, which improves the efficiency of cooling the body and removes heat from inside the building

Building mass, mass can be used to store heat during periods of high temperatures and thus cool air, discharging the heat in cooler periods of the day

Evaporative cooling, the latent change of water absorbs heat and can thus cool the air

Dehumification, using a plant to reduce the moisture in the air

However the high humidity of the Sri Lankan climate precludes the effectiveness of evaporative cooling and the small range in diurnal temperatures restricts the use of mass. Considering costs and electricity supply plants for dehumification are no option for housing in Inspector Eatham either. Therefore strategies that maximize airflow are desirable, with wind speeds up to 1.5 ms^{-1}. Higher wind speeds are not practicable for internal conditions as they cause flying papers and other disturbances.

Levels of airflow from mean wind speeds are modified by the site and through the building skin. The amount of mass [2], the volume of airflow through the building as well as temperature difference are crucial parameters for the effectiveness of ventilation. Thus in designing for ventilation the main issues are the exposure provided by the site, the amount of opacity in the skin and the amount of mass that needs to be cooled.

Natural ventilation is generated by pressure differences in and around the building. These pressure differences come from air movement generated by air temperature and by wind. Temperature driven ventilation uses the natural buoyancy of the hotter air to rise and displace cooler air (also called 'stack effect'). Advantage is staken in buildings where the external temperature is lower than the internal temperature. The internal air will rise up and exhaust from the building, bringing in fresh air. In warm climates the effectiveness of stack is questionable, the temperature differences between inside and outside being small. Since stack is driven by temperature differences, the pressures are small. Wind-driven ventilation therefore is commonly used in warm climates. Wind driven ventilation can be maximized by:

- Reduction of the plan depth and increased opennes of the section to facilitate cross-flow and vertical flow of air
- Optimum orientation of the rooms to the prevailing breeze and the linkage btween leeward and windward side to utilize pressure differences
- Maximize the skin opacity through the number, positioning and size of openings
- Reduction of internal obstruction
- Site selection and building situation to increase exposure to airflow

[2] structural cooling of the building and personal cooling of the occupants

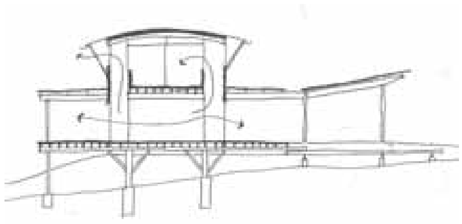


Figure 7.2: Buderim house by Lynsey Clare, section showing air movement in the open section. [Hyde, 2000]

7.2 Climate matching strategies

In the following overview the Sri Lankan climatic elements, climate modification strategies and matching building responses are summed up:

Adverse climatic elements	*Climate modification strategies*	*Building response*
Rain	Minimize heat gain	Thin plan with axis east-west
Heat	Maximize ventilation	Cross-ventilation, high ceilings
Humidity	Maximize shading	Ventilated roof
Small diurnal range		Window shading all year
		Shaded veranda

Ideally, the building response should have an integrated landscape and building response. These response tactics can be developed for different levels. The first one relates to general building and environmental control characteristics such as materials, plan shape and section. The second relates to specific aspects of building form such as the plan orientation, landscaping, veranda and courtyards.

materials :
Due to little variation in temperature the material is non-determinant, but lightweight material is preferred for it's quick response.

plan shape :
A thin plan to maximize cross ventilation and to provide high levels of natural light, avoiding dark areas as this encourage mould growth.

section :
An open section to maximize ventilation (figure 7.2).

plan orientation :
Windows facing prevailing breezes in summer for ventilation and the smallest building aspect to east and west to reduce solar gain.

landscape :
Use of tree canopy in summer to shade the building but still allow breeze. Provide for heat and glare [3] dissipating planting.

[3]Glare can be found from contrast between the exterior light levels and the wall surfaces around the window.

Figure 7.3: In these colonial bungalows in Singapore the most important function of the roof is to provide shade and protect the walls, allowing skin cooling through the use of lightweight materials. The large overhangs allow the windows to be left open in case of rain, allowing ventilations at all times except in storms. [Hyde, 2000]

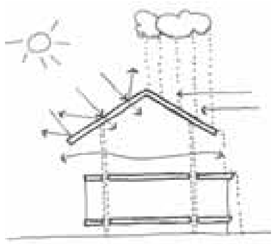


Figure 7.4: The roof acting as a parasol, protecting the inner spaces from sun and rain as well as maximizing ventilation below the roof [Hyde, 2000]

verandas :
: Verandas can provide rain and sun protection to walls and external space for extreme heat as well as diffused light.

courtyards :
: Courtyards provide light and ventilation to deep plan buildings, they also facilitate diffused light and glare reduction.

In addition the main building elements are also related to climate types. Response tactics for the building elements 'roof', 'walls' and 'floor' are discussed, concluding with a summary of each.

In hot humid climates the desired response is to use the roof for cooling. In this kind of configureation the roof acts as a parasol to simply protect the inner spaces from sun and rain as well as maximizing ventilation below the parasol (figure 7.4). The strategy is to select a geometry that fits airflow and functional requirements of the building design. Lightweight construction that is reflective and has little thermal mass is preferred as it can cool rapidly.

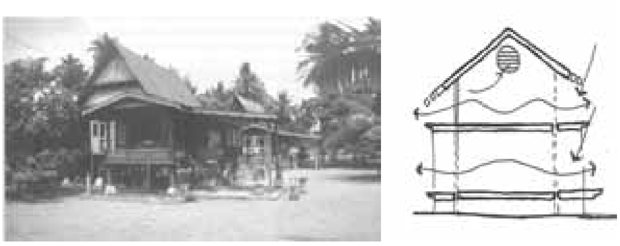


Figure 7.5: The steeply pitched roof of a Malay house, the roof utilizes gable vents to promote cooling. This is illustrated by a sketch of the design approach. [Hyde, 2000]

For single-storey buildings in hot humid climates the common strategy is to use a steeply pitched roof, as hot air will rise. However ventilation is crucial to removing this heat in the upper part of the roof (see figure 7.5 and 7.6). Overhanging eaves are introduced to reduce wind-blown rain penetration and solar access. As the building increases in height the effect of the roof has to be extended. This can be achieved by setting walls back or by adding jack roofs or verandas.

The earth temperature has an effect on the temperature of the floors, particularly those adjacent or connected to it. Earth has thermal mass; connecting floors to the earh means that floors behave thermally like the earth. If the floor is shaded its surface temperature will be similar to the ground temperature, which is at or below the shade temperature. Raising the floor allows acces to the breeze and makes the floor behave in correspondance with air temperature. A thick high desity floor reduces temperature swings in the building, while a thin lightweight floor does little to affect the fluctuations in temperature.

Roof :

- Light colored to reflect solar radiation
- Parasol type to maximize ventilation

Figure 7.6: Ventilation is crucial to removing heat in an attic roof, otherwise back radiation from the roof surface heats the attic air which is radiated via the ceiling to the internal air below [Hyde, 2000]

- Reflecting foil laminate under roof to reflect radiation
- Minimize area of roof lights

Walls :

- Avoid windows to the east and west
- Reflecting foil laminate in walls to reflect radiation

Floors :

- Light coloured to reflect solar radiation
- Lightweight elevated or shaded

7.3 Applied on domes

In this section matching building responses to climate modification strategies are applied on dome design. The most important climate modification strategie is the promotion of airflow. Matching buildig responses and their application on dome design are:

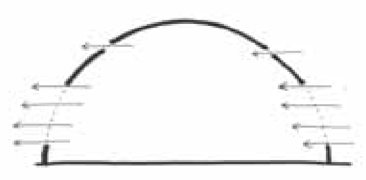


Figure 7.7: Provide for openings for ventilation.

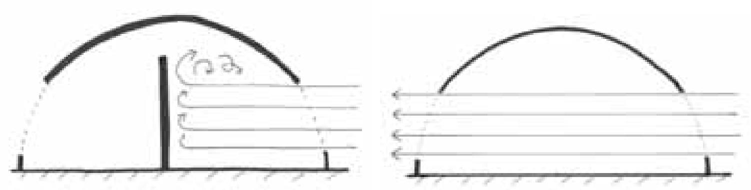


Figure 7.8: Reduce internal obstructions of the airflow.

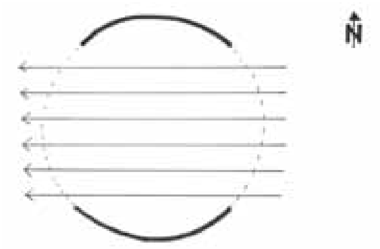


Figure 7.9: Position the openings towards the prevailing breeze. In case of Inspector Eatham east wind is prevailing.

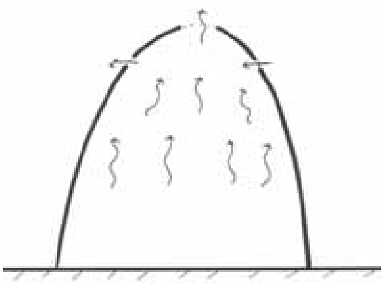


Figure 7.10: The application of a more steeply pitched dome, hot air can rise to the upper part of the dome. Important is ventilation of this part. Cross ventilation (wind driven) is probably more effective than ventilation through an opening at the top (temperature driven), as there is no large difference between inside and outside temperatures. Only at night the opening in the top might provide for some cooling, during the day however the entrance of sunlight should be prevented.

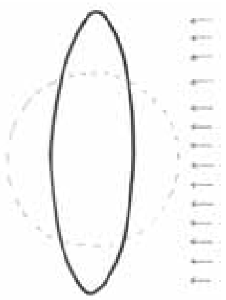


Figure 7.11: For large domes, it might be sensible to use an ellipse shaped floor plan to facilitate cross ventilation.

Concerning the minimization of heat gain and the maximization of shading the following responses can be mentioned:

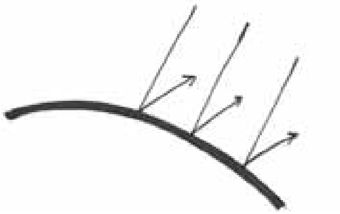


Figure 7.12: A light colored roof reflects solar radiation.

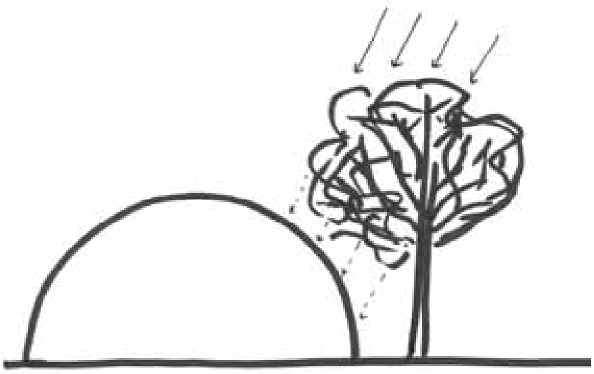


Figure 7.13: A tree canopy provides shading of the roof.

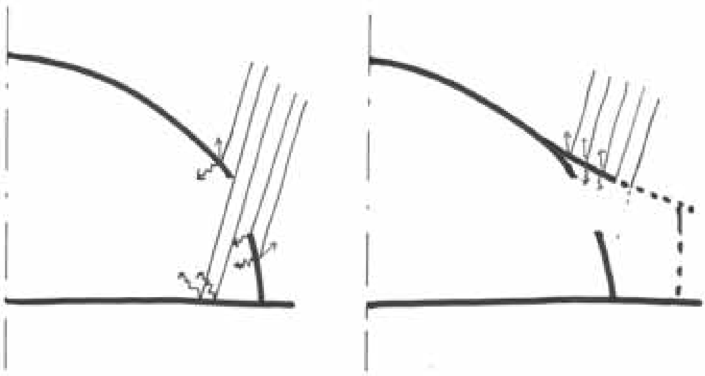


Figure 7.14: An overhanging roof or a veranda provides shading. Concequently less sunlight is entering through windows, reducing heat gain. The shaded floor will take on the ground temperature, which is at or below shade temperature. Part of the wall is shaded as well, reducing heat gain through radiation.

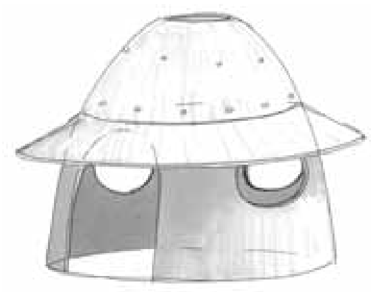


Figure 7.15: An example of a dome taking several of the matching building responses into account. However the hole in the roof should preferably be shaded by an elevated cover

ANALYSES

Analyses of stresses in both the shell and the foundation of the Solid Houses as a result of different load cases.

Chapter 8

Stresses in SolidHouse Shells

In this chapter the results of the first analyses in ANSYS are presented. Domeshells have been modelled in ANSYS in order to have an insight in what situations are normative and the effects of changes in the design. The chapter starts with an introduction about stresses in shells in general before embarking on the analyses in ANSYS.

8.1 Stresses in Shells in General

Stresses in a shell can be divided in stresses that originate from shell forces and stresses that originate from edge disturbances. Edge disturbances (moments) occur as a result of concentrated loads or where deformations are limited by the geometry of the structure. In case of the SolidHouse Shells this is the rigid connection between the foundation and the shell, see figure 8.1. The resulting moments cause stresses on the inside and outside of the shell. The moment dims with the distance from the connection. Stresses can be absorbed by reinforcement on the outside of the cross-section. However as soon as the concrete cracks, stresses are reduced to zero.

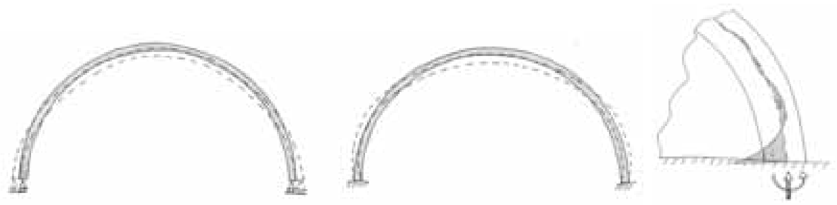


Figure 8.1: The connection to the foundation is rigid and prevents the shell from deforming as in most left sketch. Consequently the shell deforms as in the middle sketch. The moments that stem from this edge disturbance are shown in the sketch on the right.

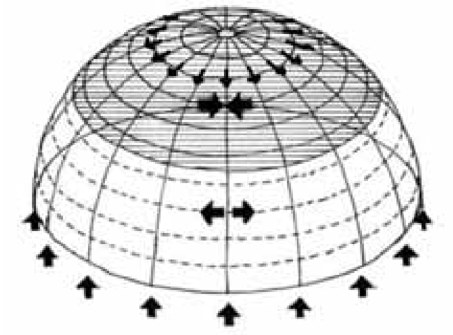


Figure 8.2: Shell forces act in meridional and circumferential direction.

Stresses caused by shell forces need to be absorbed by reinforcement. In ANSYS these stresses can be made visible by showing stresses in the middle layer of the shell, as in the middle layer the stresses caused by edge disturbances are zero. Figure 8.4 shows the stresses in the outer layer of a dome shell, while figure 8.5 shows the stresses in the middle layer of the domeshell. The difference is very small, concludingly edge disturbances are very small.

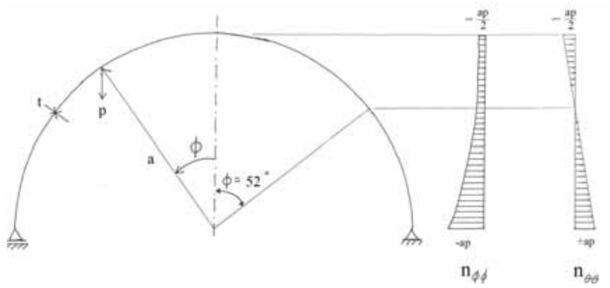


Figure 8.3: Distribution of stress resultants over the height of the hemisphere. The distribution of the circumferential stress $n_{\theta\theta}$ is almost linear. Theorethical background in Appendix C.1

Shell forces can be divided in two directions; shell forces acting in meridional direction and shell forces acting in circumferential direction. In figure 8.2 both have been illustrated. Forces in meridional direction are compression forces. Forces in circumferential direction are compression forces or '-rings' in the upper part and tension forces or '-rings' in the lower part of the shell. The transition of compression to tension rings occurs at 38 degrees from the horizontal. Theorethical background can be found in Appendix C.1.

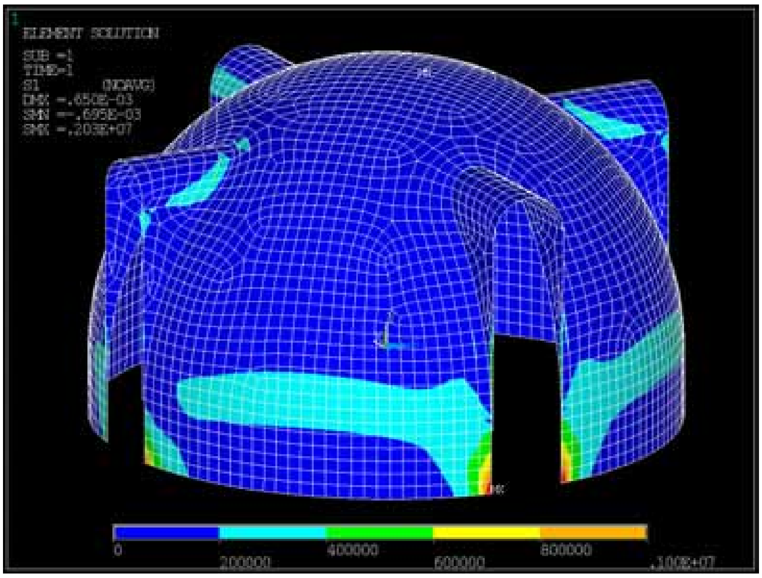


Figure 8.4: Shell loaded by wind and gravity, stresses in outer layer

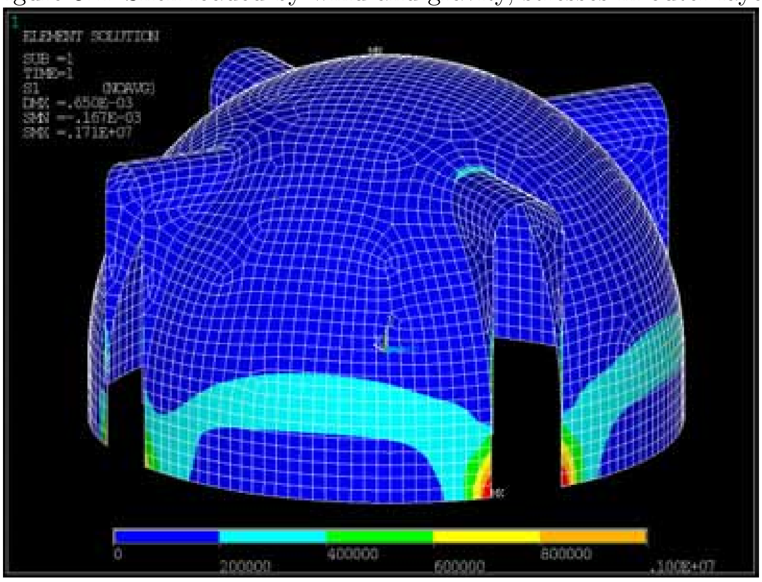


Figure 8.5: Shell loaded by wind and gravity, stresses in the middle layer

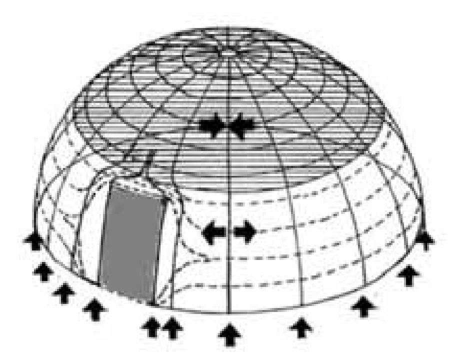


Figure 8.6: Meridional compression forces around a rectangular and a round opening

Openings in the shell interrupt the tension rings and the meridional forces. The shell redistributes the forces around the opening, resulting in more concentrated forces, see figure 8.6. Figure 8.7 shows how the forces in each of the two directions redistribute and cause additinal forces. If the opening is a door, the only way for tension rings to redistribute around the openings is to use the foundation. Consequently a good connection to the foundation is necessary next to the openings, otherwise the shell will tear from the foundation.

Additional reinforcement is required to avoid tears where tension stresses have increased. Traditionally the rebar that are 'cut away' by an opening are compensated by the same amount of rebar around an opening, see figure 8.8.

Originally the Solid House Foundation is using rectangular openings for windows and doors. A circular form would create less tension above and under the opening. In figure 8.9 this is illustrated with a simplified representation of the flow of the meridional compression lines around a rectangular respectively circular opening.

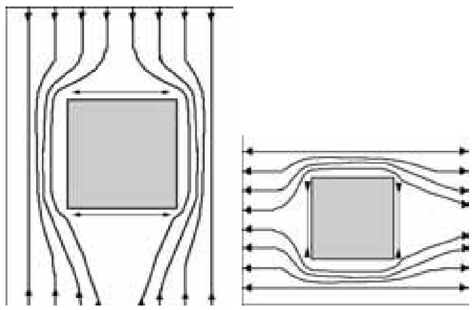


Figure 8.7: The left picture shows how meridional compression lines are concentrated at the side of the opening and cause tension above and under the opening. The picture on the right shows the redistribution of forces in circumferential direction, in case that the opening is situated in the lower area of the shell (where ringtension is present). The tension stresses are concentrated below and above the opening, at the sides compression forces originate. In the upper part of the hemispere the redistribution of the forces in circumferential direction will be the same as the picture on the left turned 90 degrees.

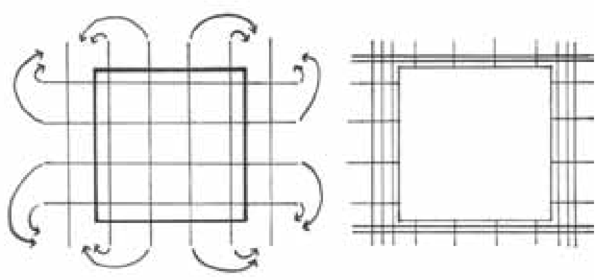


Figure 8.8: Compensate for 'missing' rebar around the opening

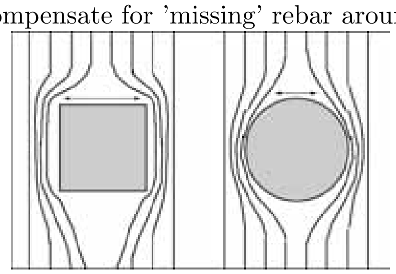


Figure 8.9: Meridional compression lines around a rectangular and a round opening

8.2 Modelling Hemispheres

The model of a thin concrete hemispherical shell has been analysed in ANSYS. The shell has a thickness of 0,1 meter and has been free meshed into 'shell-93' elements (8 node elements). The material is taken as linear-elastic-isotropic, using and Youngs' module [1] of 17 300 N/mm^2 and a contraction coefficient of zero. The model has been analysed for different loads and different dimensions. The most detailed analyses was made of a dome of 6 meter diameter, as in this case a mesh with an element edge length of 0,1 meter could be applied without exceeding the maximum number of nodes in ANSYS. For 12 meter diameter domes a grid of 0,4 meter has been applied. In the plots the contours have been adapted to show the low stresses more clearly. Therefore different colors are used than in other parts of this thesis.

Hemispherical domeshell, gravity load
The maximum principal tension stress in the shell resulting from gravity varied from $0,035\ N/mm^2$ for an edge with all degrees of freedom constraint to $0,072\ N/mm^2$ for a vertically constraint edge (see figure 8.10). Both of these values have been checked with a handcalculation, see Appendix C. For a 12 meter dome these values vary from $0,089\ N/mm^2$ to $0,142\ N/mm^2$. These stresses are very low. The tension strenght of B15 is even higher ($0,9\ N/mm^2$) [inf, 2002]. However this value is only to give an idea of the smallness of the stresses, as concrete should not be designed on tension stress without reinforcement. For the next three hemispherical models the base edge has been constraint in all degrees of freedom (simulating a ringbeam), the reactions on the constraints are summarized at the end of this section.

Hemispherical domeshell, windload
A windload has been simulated by applying a gravity load [2] in horizontal direction, creating a total reaction force of 3,4 kN. The latter is the total windforce on a 6 m dome according to the Indian Standards (see page 15). The load generates a maximum tension stress of $0,004\ N/mm^2$ and a maximum pressure of $-0,004\ N/mm^2$. These occur very locally on the inside of the shell's edge (base). Displacements are maximum 0.001 mm, which is in horizontal direction.

[1] The Youngs' Module is taken two third of the Youngs' Module of B15 to take the low quality of the concrete and creep into account.
[2] The horizontal gravity has a value of $0,255\ m/s^2$, creating a horizontal force of 3461 N.

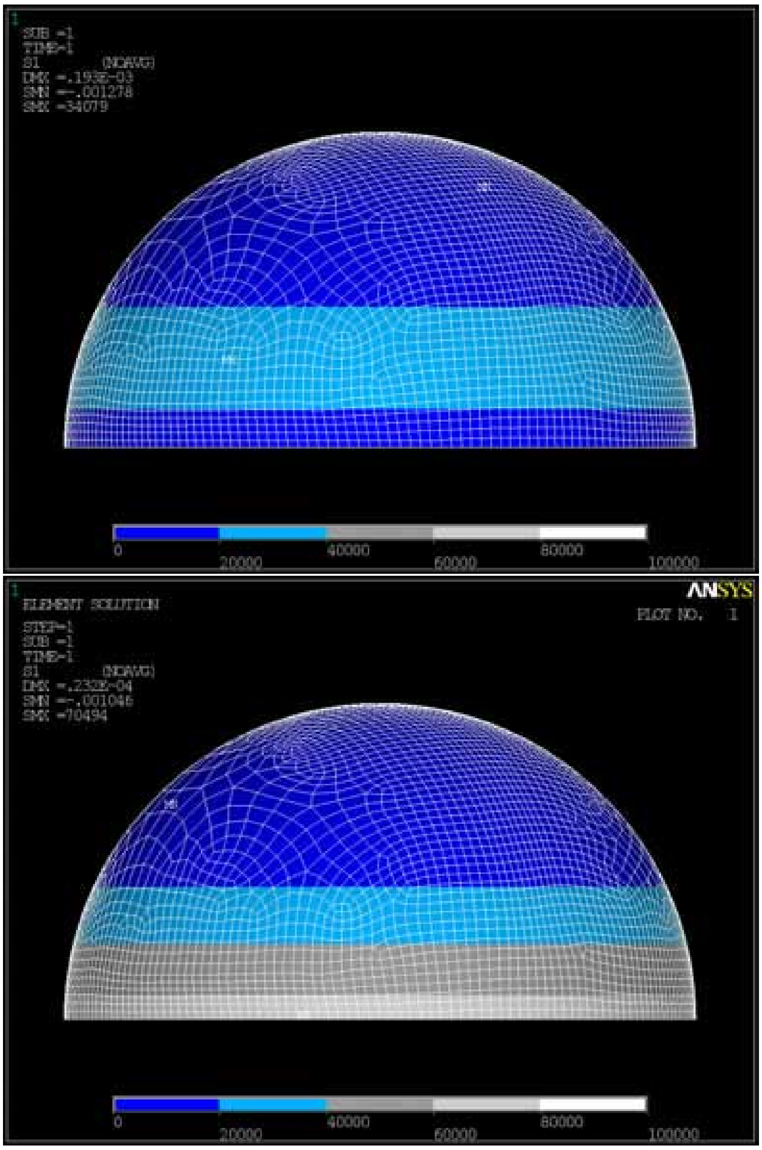


Figure 8.10: Principal tension stress in middle layer as a result from gravity in the shell model with an edge constraint in all degrees of freedom and tension stress as a result from gravity for a model that is only vertically constraint at the edge. The latter will not occur in reality, but is used to check the results in Ansys with handcalculations (See Appendix C). The pictures also illustrate the impact of a constraint edge on the stresses. Stresses are clearly lower, but the tension rings with the maximum tension have moved upward as the deformation of the shell is constraint in the edge.

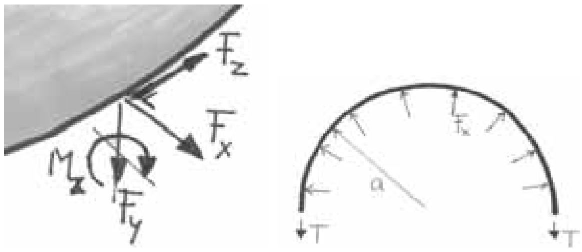


Figure 8.11: Reactions of shell on ringbeam and Tensionforce T in ringbeam caused by F_x

Hemispherical domeshell, gravity and windload
Gravity load and wind load are combined in a model, calculating according to the ultimate limit state with $\gamma_g = 1,2$ and $\gamma_q = 1,5$ [3]. The results are only slightly different from a load case with only (vertical) gravity. Maximum values of the principal stresses are at the inside and the outside of the shell edge, amounting to $0,057\ N/mm^2$ and $-0,132\ N/mm^2$, see figure 8.12.

Hemisperical domeshell, gravity and concentrated load
A pressure load is modelled on an area of $0,48 \times 0,48\ m^2$ halfway the top of the 6 meter dome, creating a load of $\gamma_q \cdot 1,5\ kN$. Combined with gravity load this results in a maximum tension stress of $0,053\ N/mm^2$.

Load:	gravity	gravity and concentrated load	gravity and windload	
F_x	981	1 000	1 058	N/m
F_z	11	168	461	N/m
M_z	171	175	188	Nm/m

The combination of gravity and windload turns out to be the normative situation, both for stress in the ringbeam as for stress in the shell. The maximum stress in the shell in this load case is $0,057\ N/mm^2$. This is very low. Note that the theoretical maximum tensile strength of concrete [inf, 2002] is $0,9\ N/mm^2$. However this is a theoretical value, concrete should not be designed on tensile strength. For this load combination F_z causes a shear stress on the connection between ringbeam and shell of $0,0046\ N/mm^2$. The maximum shear stress of unreinforced B15 concrete is $0,36 N/mm^2$. The reaction F_x on the ringbeam causes a tension force in the ringbeam of $a \cdot F_x = 3 \cdot 1\ 058 = 3,2\ kN$ (figure 8.11). To withstand this stress a concrete beam would need to be at least $3600\ mm^2$, which expresses the superfluity of a ringbeam in this case (if not taking soil conditions into account).

[3] Thus applying a vertical gravity of $11,8 m/s^2$ and a horizontal gravity of $0,38 m/s^2$.

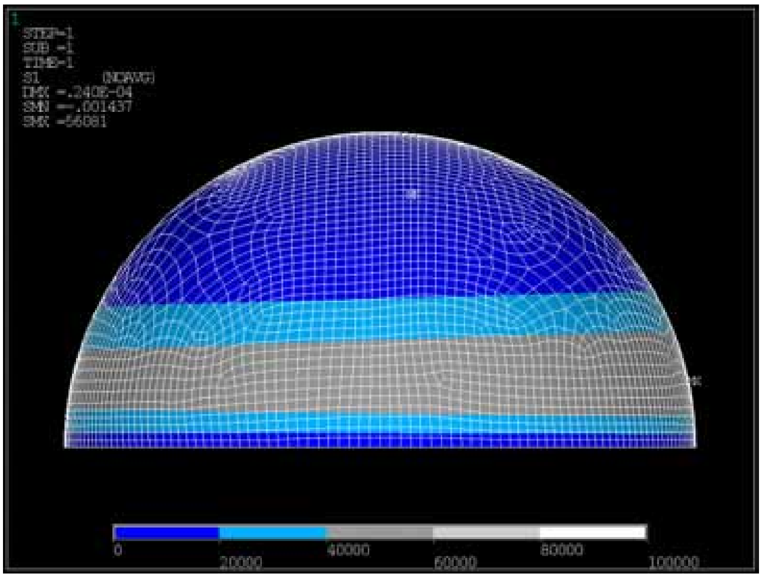


Figure 8.12: Principal tension stress in the middle layer as a result of wind and gravity load.

8.3 Modelling SHF denktank design

Domeshell with openings according to SHF design, gravity load
To have an idea of the influence of openings on the tension stress in the domes, the openings of the SHF design (see Appendix D) have been modelled in ANSYS. This is a hemispherical dome, 9 meter in diameter on a 1 meter high cilinder, with four 1,2 meter wide openings up to 4,6 meter high. The grid consists of elements with a wallthicknes of 0,1 meter and an element edge length of 0,2 meter. The ring beam is connected to the shell by vertical reinforcement. It is supposed that the ring beam constraints the lower edge of the shell in all degrees of freedom. This is modelled into ANSYS with a gravity load of $\gamma_g \cdot 9,8\ m/s^2$.

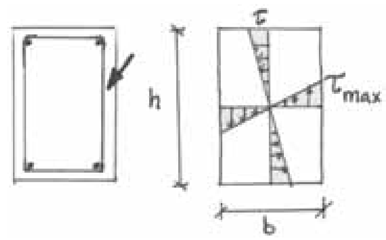


Figure 8.13: Stirrups for shear stress caused by the torsional moment

Resulting stress levels in the shell's tension rings do not exceed $0,2\ N/mm^2$. Increased tension stress can be found above the dooropenings, resulting from diverting meridional pressure lines (as explained in paragraph 8.1. In figure 8.14 can clearly be seen how the ring tension forces divert around the opening through the foundation. Peak stresses with a maximum of $0,7\ N/mm^2$ occur next to dooropenings at the lower corners, as here the shell's ringtension forces are concentrated and transferred on the ringbeam. This can be seen in figure 8.14, showing the principal stresses the elements that are connected to the ringbeam, white arrows representing the principal tension stress and blue arrows representing the principal pressure. Maximum forces on the ringbeam can be found next to the dooropenings, the maximum nodeforce found being and 9,8 kN/m perpendicular to the shell in the horizontal plane, creating a tension force of $a \cdot 9,8 = 44\ kN$ (figure 8.11). Based on the latter a minimum ringreinforcement of the beam of one rebar of 13 mm diameter would be needed. To have an idea of the stress level: the ringbeam should have at least a cross-section of 49 000 mm^2 ($230 \times 230\ mm^2$) to reduce the tension stress below the maximum tension stress of concrete [inf, 2002], however this is only a theoretical value and concrete may not be designed on tension. The maximum torsional moment M_z on the ringbeam can be found in the middle between the openings, being about 323 Nm/m. The torsional moment [4] creates a shear stress. If the cross-section of the ringbeam is smaller than approximately $160 \times 160\ mm^2$ the maximum shear stress exceeds the maximum shear capacity of B15, being $0,36\ N/mm^2$. In this case stirrups should be provided to increase the shear capacity (figure 8.13).

[4] $\tau_{max} = \left(3 + \frac{2,6}{0,45+\frac{h}{b}}\right) \cdot \frac{M_z}{b^2 h}$ with a ringbeam of $200 \times 200\ mm^2$ this results in a maximum shear stress of $0,2\ N/mm^2$

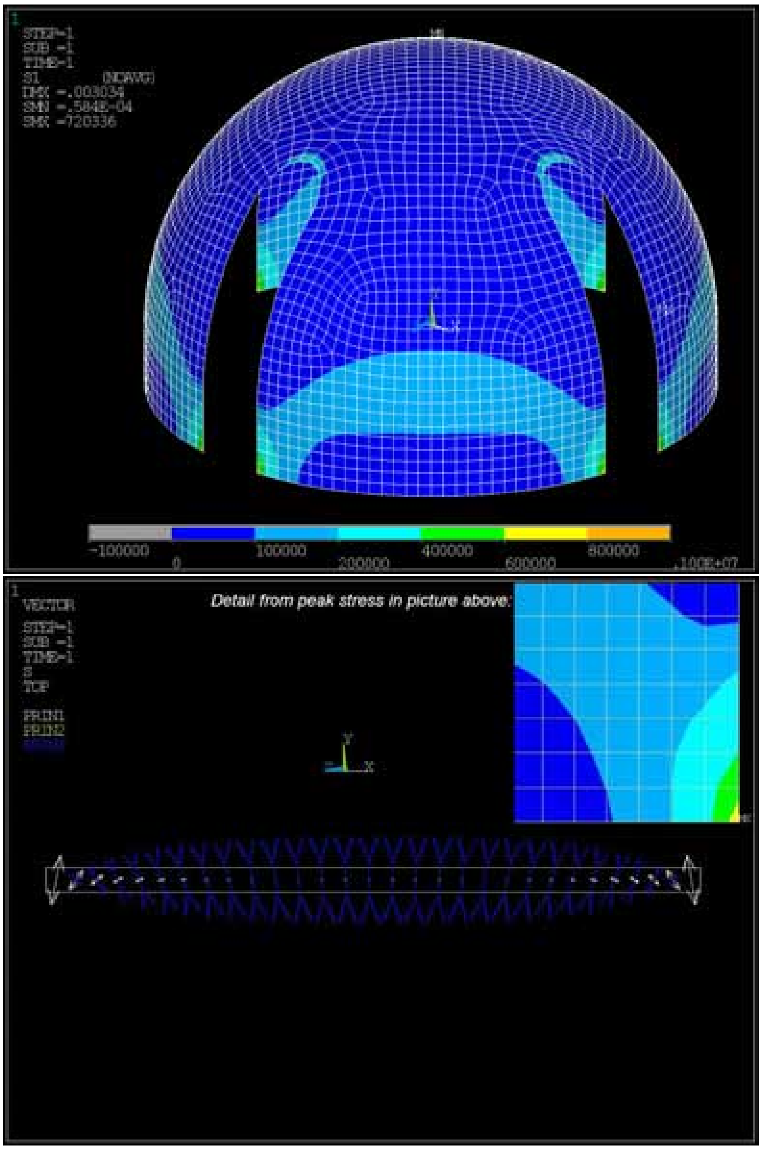


Figure 8.14: Principal tension stress in the middle layer of the shell as a result of gravity load in a perforated hemisphere with a base constraint for all degrees of freedom and a vector plot of the principal stresses in the elements connected to the ringbeam.

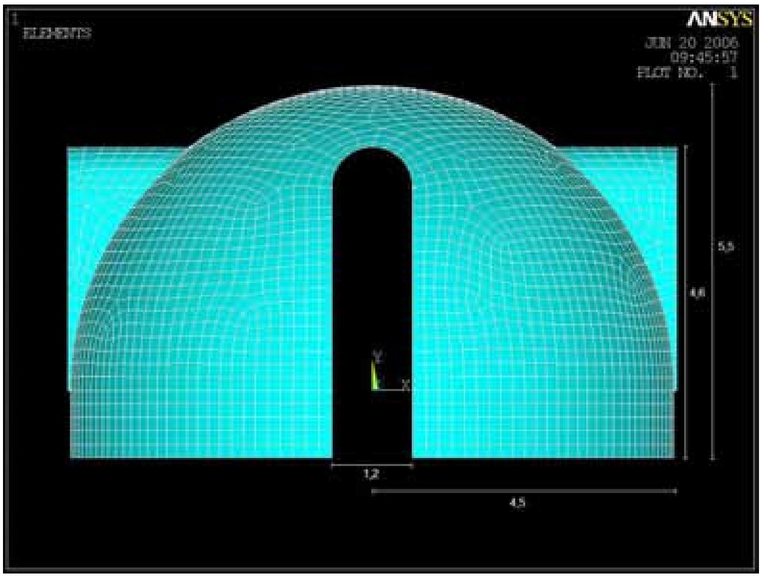


Figure 8.15: The design of the SHF denktank modelled in ANSYS

Domeshell according to SHF design, gravity load
The covers above the openings (according to SHF design) are now added to the model, resulting in the model shown in figure 8.15. The lower edges are constraint in all degrees of freedom and a gravity load of $\gamma_g \cdot 9,8\ m/s^2$ is applied. A vector plot of the resulting principal stresses can be seen in figure 8.16. The resulting principal tension stresses resulting from shell forces are shown in figure 8.17, the middle layer of the shell has been plotted. The plot shows how the strongest tension rings are diverted to the foundation by the openings. Maximum tension stresses in the tension rings are below $0,2\ N/mm^2$. Peak stresses up to$0,79\ N/mm^2$ occur next to the openings, where the tension forces are concentrated and diverted to the foundation. Above the dooropenings stress levels are slightly higher than in the rest of the shell, caused by diverting meridional pressure lines.

If the lower edge is constraint in all directions, but not for rotation, the results look similar (see figure 8.18). The maximum stress caused by the tension ring diverting to the foundation (next to the door openings) is with $1,0\ N/mm^2$ higher than in case of a totally restraint edge.

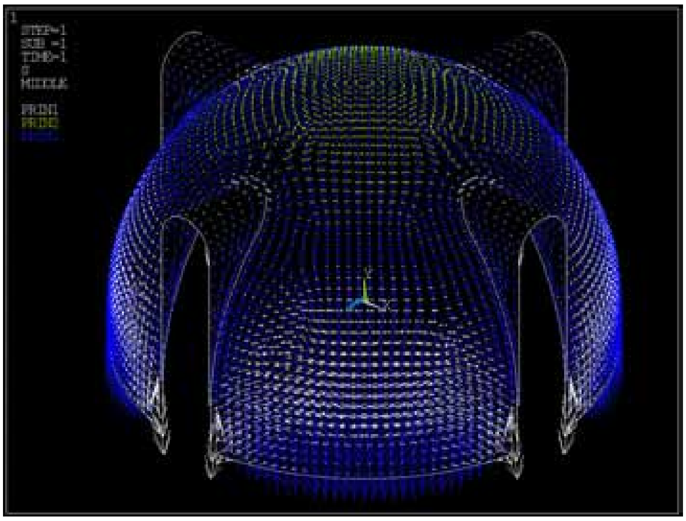


Figure 8.16: Vector plot of the principal stresses as a result of gravity load.

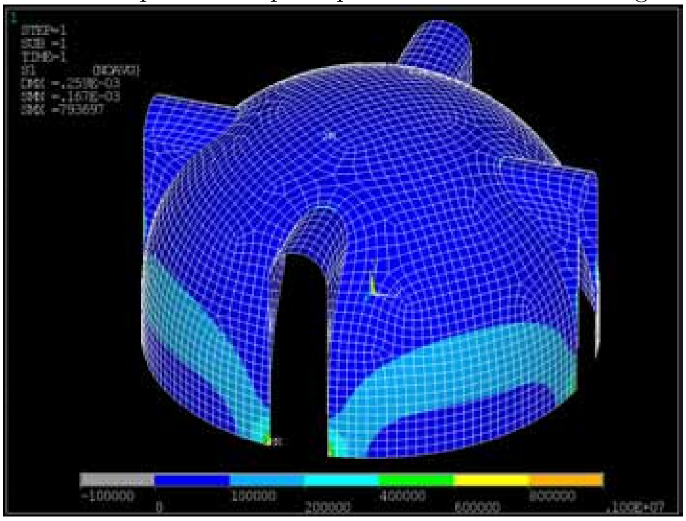


Figure 8.17: Principal tension stresses in middle layer of the shell as a result of gravity load (the edge is constraint in all degrees of freedom)

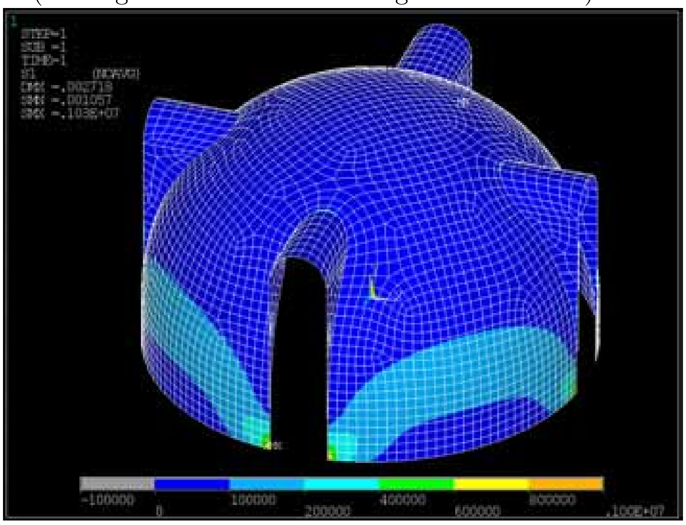


Figure 8.18: Tension stresses as a result of gravity load in the model if the edge is only constraint in all directions.

Domeshell according to SHF design, gravity and windload
A windload [5] of $\gamma_q \cdot 12,1\ kN$ is added to the gravity load $\gamma_g \cdot 9,8\ m/s^2$ on the model with edges constraint in all degrees of freedom. Stress levels in the shell increase only slightly, but the maximum ringtension stress in the shell stays below $0,2\ N/mm^2$. Peak stresses in the lower corner of dooropenings increase from $0,79$ to $0,93\ N/mm^2$, see figure 8.19.

Resulting maximum reactions on the ringbeam					
F_x	11,1	kN/m	M_x	6	Nm/m
F_y	55,9	kN/m	M_y	607	Nm/m
F_z	37,2	kN/m	M_z	747	Nm/m

This results in a maximum ringtension force of $T = a \cdot 11,1 = 50\ kN$. Based on the latter a minimum ringreinforcement of the beam of one rebar of 14 mm diameter would be needed. To have an idea of the size of this stress, the ringbeam should have at least a cross-section of 56 000 mm^2 ($240 \times 240\ mm^2$) to reduce the tension stress below $0,9\ N/mm^2$ which is the theorethical unreinforced tension stress of B15 [inf, 2002] (however concrete may not be designed on tension). If the cross-section of the ringbeam is smaller than approximately $215 \times 215\ mm^2$ the maximum shear stress caused by the maximum torsional moment M_z exceeds the maximum shear capacity of unreinforced B15, being $0,36\ N/mm^2$. In this case stirrups should be provided to increase the shear capacity (figure 8.13). Shear stress between the ringbeam and the shell, as a result of F_z, amounts to $0,37\ N/mm^2$. This needs to be absorbed by shear reinforcement (vertical rebars, socalled 'uprights', connecting shell and ringbeam).

To have an idea of stresses caused by moments in the shell the stresses in the outside and the inside of the shell have been plotted in figure 8.20. The difference in stresses with figure 8.19 are caused by moments in the shell. Stresses due to these edge disturbances will result in cracks on the inside and outside of the shell. Reinforcement would be most efficient at the outer edges of the cross-section instead of in the middle, however then the cover is very small. Yet a crack 'takes away' the stress and will not develop further. Consequently the shell does not necessarily need extra reinforcement for stresses caused by moments.

[5] According to the Indian Standards the cylindrical part of the dome receives a horizontal load of 4,5 kN and the spherical part receives a horizontal load of 7,5 kN, which results in a total horizontal force of 12,1 kN caused by windload. The horizontal gravity is set on $0,5\ m/s^2$, which results in a total horizontal load of 18,4 kN. This is a rough approximation of the windload.

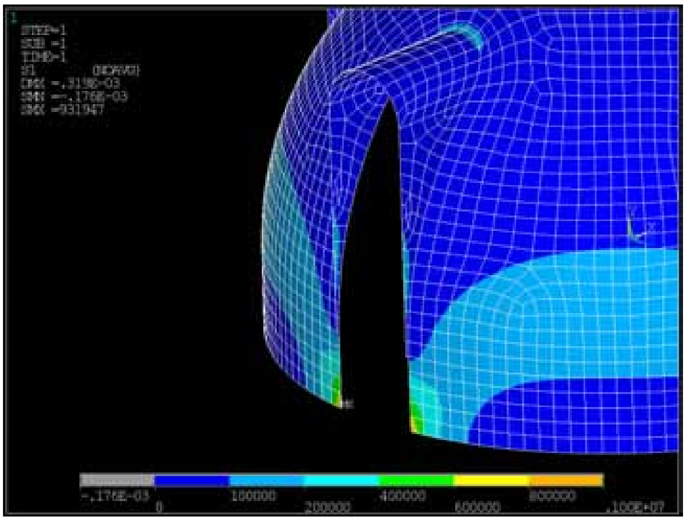


Figure 8.19: Principal tension stresses around the door opening caused by shell forces (in the middle layer) in a shell loaded by a combination of gravity and windload (with a totally constraint edge).

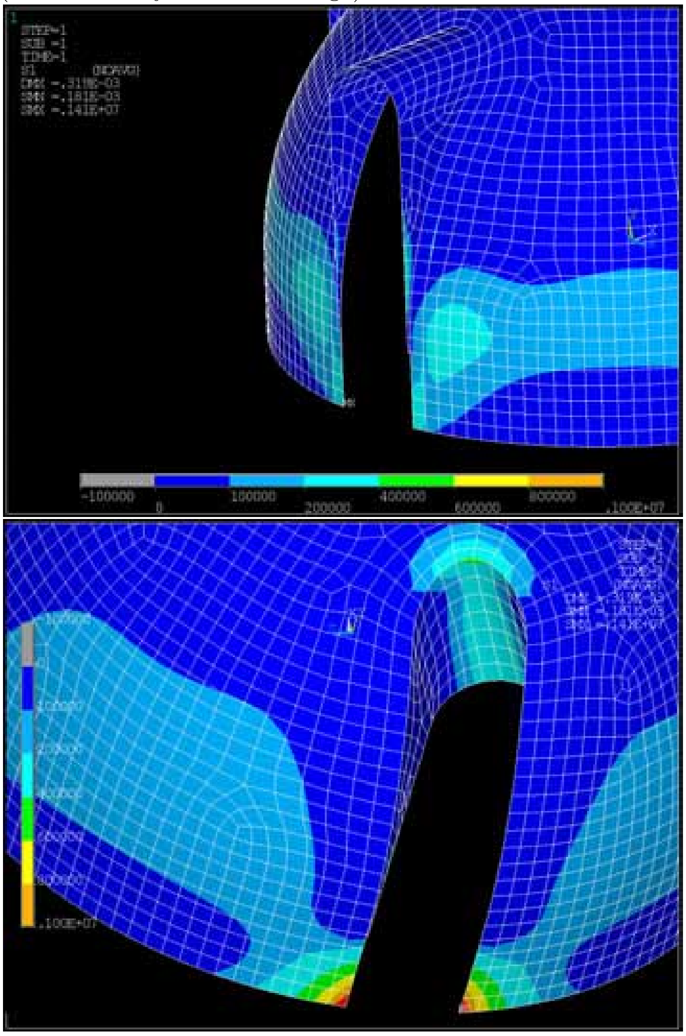


Figure 8.20: Principal tension stresses around the door opening caused by shell forces (in the middle layer) in a shell loaded by a combination of gravity and windload (with a totally constraint edge).

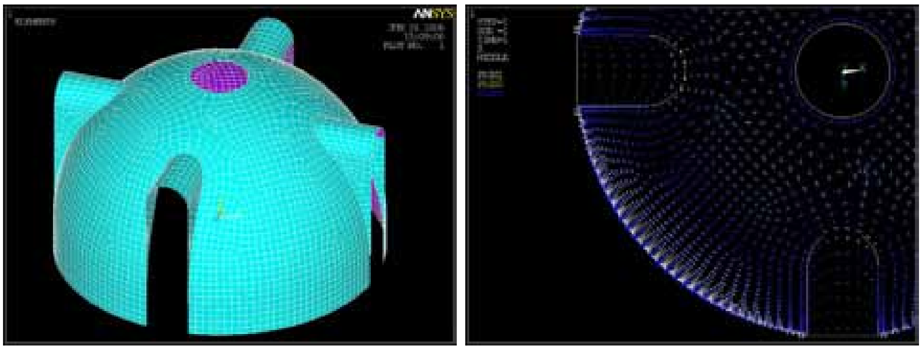


Figure 8.21: Model in ANSYS completed with opening in the top and the principal stresses resulting from gravity load.

Added opening at the top, gravity and windload
The model is completed by modelling a 1,6 m diameter opening in the top, which results in a pressure ring around the opening, see figure 8.21. Due to a reduction of weight the tension stresses in the shell decrease slightly. In the vector plot of figure 8.21 can clearly be seen that the stresses are redistibuted around the opening in the top to form a pressure ring.

Domeshell according to SHF design, conclusions
The stresses caused by shell forces in the SHF denktank shell (Appendix D) are so low that in theory the shell could do without reinforcement except for a reinforced connection between foundation and shell. However concrete may not be designed on stress. For safety reasons, i.e. to avoid brittle failure, the concrete needs to be reinforced. Besides, the mortar would not stick to the pneumatic formwork itself, rebar or something else is needed to prevent the mortar from sliding down.

Therefore it is advised to build with a cheaper and easier method of reinforcement, which also decreases the amount of mortar needed. A design has been made in chapter 10 on the application of ferrocement on the SHF denktank design.

Stresses on the in- and outside of the shell that are caused by edge disturbances are small. Besides these do not need additional reinforcement. Cracks will occur, but these are not endanger the stability of the shell. The cracks could be plastered. Most important is that moisture is prevented from entering the cracks and cause corrosion of the reinforcement.

8.4 Comparing catenaric with spherical dome

In this section the stresses in a spherical dome are compared to a dome with a catenary's cross section. The spherical dome consists of an eight meter diameter hemisphere on top of a 2 meter high cylinder, a design which the SHF is planning to use in Africa. The catenaric dome has a base with the same diameter and is plotted [6] to reach the same height as the spherical dome; being 6 meter. The spherical dome has a larger volume than the catenaric dome, but also needs 25% more material. The shells have been free meshed into 'shell-93' elements (8 node elements) with a thickness of 0,1 meter and an element edge length of 0,2 meter). The material is taken as linear-elastic-isotropic, using and Youngs' module [7] of 17 300 N/mm^2 and a contraction coefficient of zero. Both domes have been modelled in ANSYS for gravity load and for the combination of gravity and windload [8]. After an analyses without openings, openings are added to the model. In the plots the contours have been adapted to show the low stresses more clearly. Therefore different colors are used than in other parts of this thesis.

Resulting maximum principal tension (peak)stress (S1), maximum reactions on the ringbeam (see figure 8.11) and the tension T in the ringbeam caused by F_x are shown in the tables below. A_{rb} is the minimal cross section of a beam based on T if no reinforcement is used. This value is just to give an idea of the stress level as in practise reinforcement will always be applied (the used maximum tensile stress of the concrete of $0,9\ N/mm^2$ [inf, 2002] is only a theoretical value).

Gravity load

'Spherical dome'			'Catenaric dome'		
$S1$	0,08	N/mm^2	$S1$	0,0003	N/mm^2
F_x	7,4	kN/m	F_x	3,2	kN/m
F_y	14,9	kN/m	F_y	13,4	kN/m
F_z	14,9	kN/m	F_z	7,5	kN/m
M_x	0	Nm/m	M_x	0	Nm/m
M_y	2,6	Nm/m	M_y	15,0	Nm/m
M_z	2,6	Nm/m	M_z	38,6	Nm/m
T	30	kN	T	12,7	kN
A_{rb}	183×183	mm^2	A_{rb}	118×118	mm^2

[6]The formula for the catenary is $y = a \cdot cosh(\frac{x}{a})$. The parameter a is varied till the desired height of 6 meter is reached.

[7]The Youngs' Module is taken two third of the Youngs' Module of B15 to take the low quality of the concrete and creep into account.

[8]The windload on the spherical dome is defined according to the Indian Standards, see page 15. The total horizontal force on the spherical dome amounts $\gamma_q \cdot 22\ kN$. This force is simulated by applying a horizontal gravity load of $0,92\ m/s^2$, which is a rough approximation. The windload on the catenaric dome is assumed to be similar, which is also a rough estimation. As the catenaric dome uses less material the horizontal gravity load needs to have a value of $1,14\ m/s^2$ in order to result in a horizontal force of 33 kN on the dome.

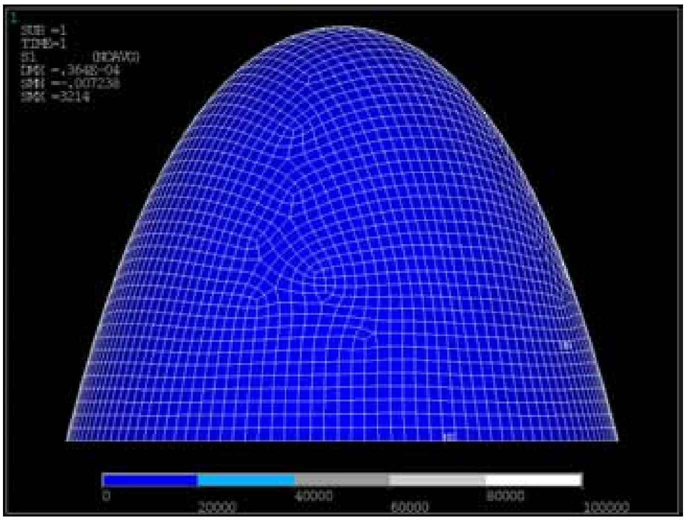


Figure 8.22: Resulting principal tension stresses from shell forces (middle layer) caused by a combination of wind and gravity load in the catenaric model. Stresses stay below $0,02 N/mm^2$.

Wind- and gravity load

'Spherical dome'			*'Catenaric dome'*		
$S1$	0,094	N/mm^2	$S1$	0,003	N/mm^2
F_x	8,4	kN/m	F_x	3,8	kN/m
F_y	16,6	kN/m	F_y	15,1	kN/m
F_z	15,2	kN/m	F_z	2,0	kN/m
M_x	0,3	Nm/m	M_x	0,1	Nm/m
M_y	14,8	Nm/m	M_y	0,4	Nm/m
M_z	44,1	Nm/m	M_z	86,3	Nm/m
T	34	kN	T	15	kN
A_{rb}	194×194	mm^2	A_{rb}	130×130	mm^2

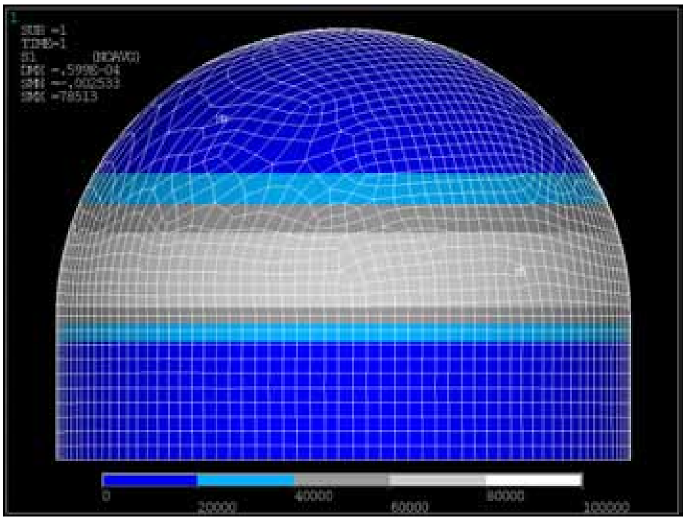


Figure 8.23: Resulting principal tension stresses from shell forces (middle layer) caused by a gravity load in the spherical model

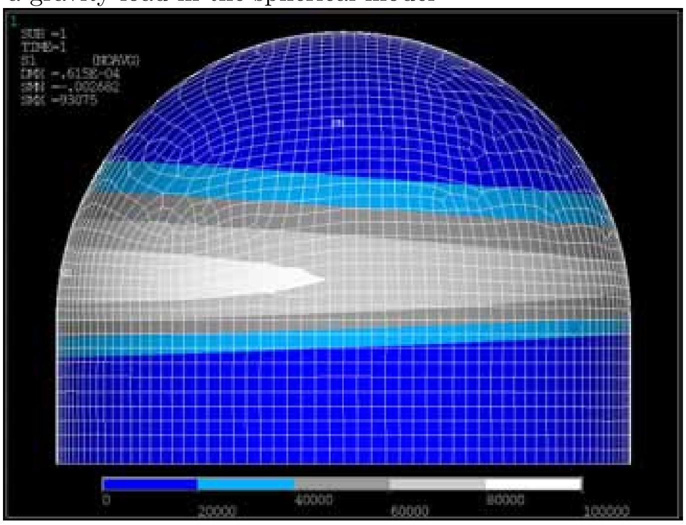


Figure 8.24: Resulting principal tension stresses from shell forces (middle layer) caused by a combination of wind and gravity load in the spherical model. The assymetric loading is clearly visible.

Openings are added to both models,

Resulting maximum principal tension (peak)stress (S1), maximum reactions on the ringbeam (see figure 8.11) and the tension T in the ringbeam caused by F_x are shown in the tables below. A_{rb} is the minimal cross section of a beam based on T if no reinforcement is used. This value is just to give an idea of the stress level as in practise reinforcement will always be applied (the used maximum tensile stress of the concrete of $0,9\ N/mm^2$ [inf, 2002] is only a theoretical value).

Gravity load

'Spherical dome'			'Catenaric dome'		
$S1$	0,28	N/mm^2	$S1$	0,03	N/mm^2
F_x	8,7	kN/m	F_x	3,5	kN/m
F_y	17,3	kN/m	F_y	14,5	kN/m
F_z	15,3	kN/m	F_z	0,25	kN/m
M_x	0,7	Nm/m	M_x	0,2	Nm/m
M_y	42,8	Nm/m	M_y	0,6	Nm/m
M_z	51	Nm/m	M_z	47,3	Nm/m
T	35	kN	T	14	kN
A_{rb}	197×197	mm^2	A_{rb}	124×124	mm^2

Wind- and gravity load

'Spherical dome'			'Catenaric dome'		
$S1$	0,34	N/mm^2	$S1$	0,10	N/mm^2
F_x	10,8	kN/m	F_x	5,3	kN/m
F_y	19,8	kN/m	F_y	21,9	kN/m
F_z	20,5	kN/m	F_z	2,5	kN/m
M_x	1,4	Nm/m	M_x	0,5	Nm/m
M_y	86,6	Nm/m	M_y	1,9	Nm/m
M_z	125,3	Nm/m	M_z	153,8	Nm/m
T	43	kN	T	21	kN
A_{rb}	219×219	mm^2	A_{rb}	154×154	mm^2

Conslusions

Stresses in the shell and loads on the ringbeam are generally largest in the spherical model, except for the torsional moment on the ringbeam which is generally much larger in the catenaric dome. As the windload is only an approximation the results for both domes in case of a combination of wind- and gravity load should be seen as a very rough indication. Clear is that the stresses in a catenaric domeshell are considerably smaller than in the spherical domeshell.

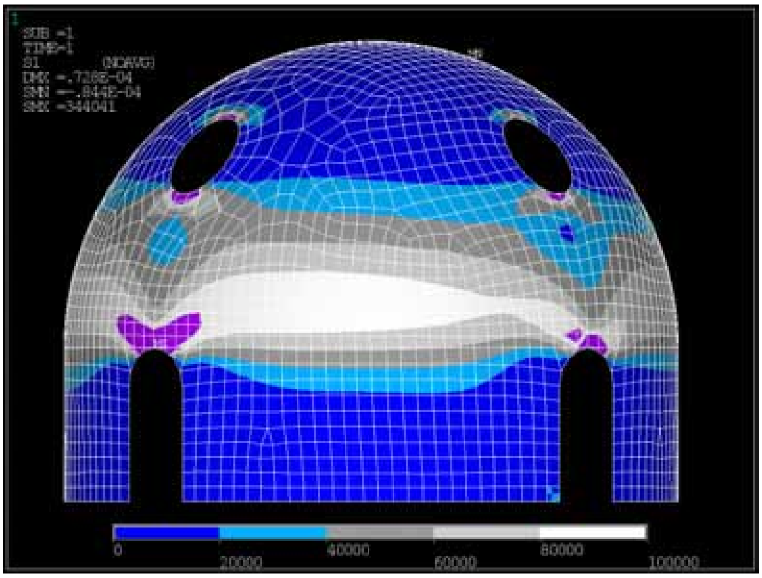


Figure 8.25: Resulting principal tension stresses from shell forces (middle layer) caused by a combination of wind and gravity load if openings are added to the spherical model

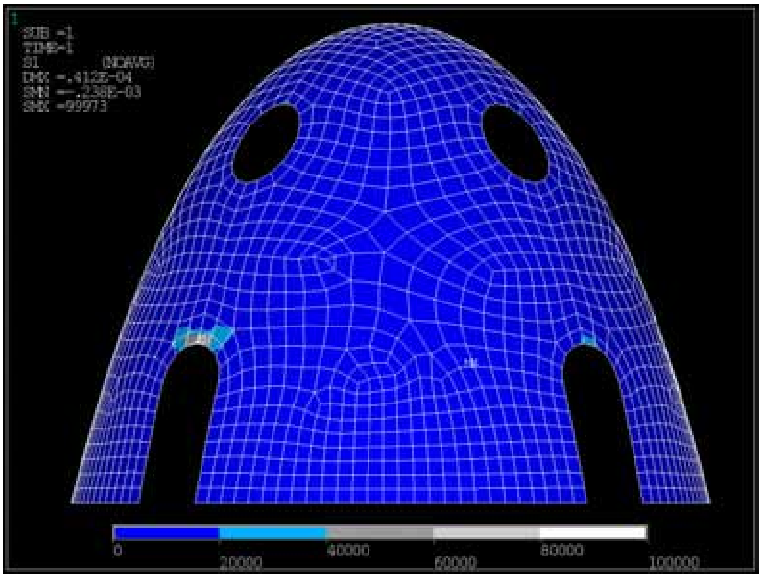


Figure 8.26: Resulting principal tension stresses from shell forces (middle layer) caused by a combination of wind and gravity load if openings are added to the catenaric model

Chapter 9

Foundation

In figure 9.2 can be seen that the foundation counts for approximately half of the construction materials. Therefore an analysis of the currently used foundation and the loads acting on it was done. The results can be found in this chapter.

9.1 Load Cases

The foundation has to be analysed for two loadcases (figure 9.1):

- Loads acting on the foundation during construction
- Loads acting on the foundation during usage

In both cases ground pressure has to be taken into account. During construction the foundation has to counterbalance the uplift of the formwork. After the Solid House has been completed the foundation has to absorb some shear force from windload and the foundation floor is loaded by variable floor load. According to NEN6702 the floor should be able to absorb a load of $p_{rep} = 1,75\ kN/m^2$.

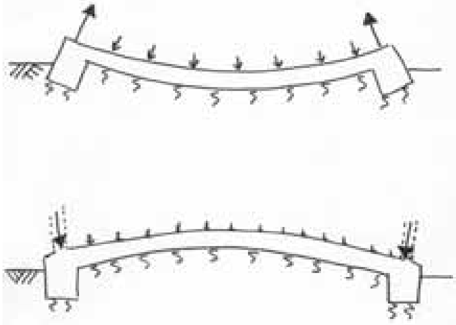


Figure 9.1: Load cases

	SHF Bolivia 6 meter dome
foundation	
cement	2250 kg
sand	6,00 m3
rock	4,50 m3
rebar	480,77 kg
dome	
cement	2450 kg
sand	6,00 m3
rock	5,10 m3
rebar	519,23 kg
total 6 meter dome	
cement	4700 kg
sand	12 m3
rock	10 m3
rebar	1000 kg

Figure 9.2: Material used by SHF in Bolivia for the construction of a 6 m dome

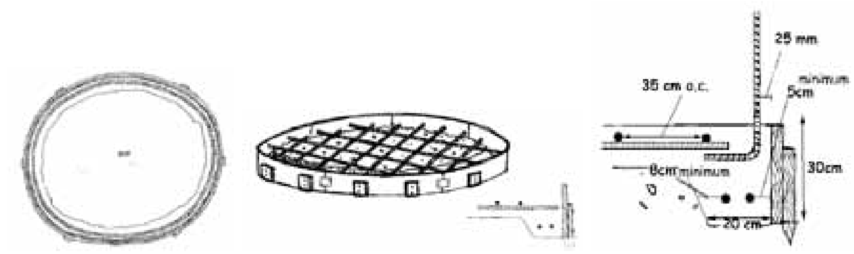


Figure 9.3: Construction of the foundation [MDI, 2005]

9.2 Currently used foundation

The foundation is constructed as show in the figure 9.3. The ring beam has to counterbalance the uplift of the hemishperical formwork. In order to calculate the minimum crossection of the ring beam, this load case is shown in figure 9.4.

The anchoring force n loads the circumference of the foundation, while the air pressure p loads the circular foundation area under the inflated formwork.
Therefore: $n \cdot 2 \cdot \pi \cdot r = p \cdot \pi \cdot r^2$ and $n = \frac{1}{2}pr$

A formwork measuring 6 meter in diameter that is inflated to a pressure of $1,5\ kN/m^2$ needs anchoring:
$n = \frac{1}{2} \cdot 1,5 \cdot 3 = 2,25\ kN/m$
The total uplift then is $2.25 \cdot 2 \cdot \pi \cdot 3 = 42,4\ kN$.

If the density of the concrete ring beam is assumed to be 24 kN/m^3, and a load factor $y_q = 1,5$ is taken into account, a ring beam with a cross section of $141 \cdot 10^-3\ m^3$ is needed (about 375 by 375 mm). However if floor and ring beam are well connected the weight of the floor may be taken into account, which

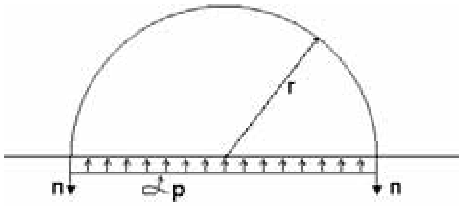


Figure 9.4: The inflated hemispherical formwork creates an uplift that has to be counterbalanced by the reinforcement ring. The anchoring force n loads the circumference of the foundation, while the air pressure p loads the circular foundation area under the inflated formwork.

results in a smaller ringbeam. The MDI recommends a ring beam of with a cross section of 200 mm and a height of 300 mm and a floor of 100 mm.

The concrete foundation needs to be reinforced to absorb the radial and circumferential bending stresses. Perimeter rebar (ring beam tendons) are applied to absorb circumferential bending stresses in the ring beam. The MDI recommends two 12 mm perimeter rebar to be placed at least 80 mm off the ground and 50 mm in from the formed edge in the ring beam. A cross pattern of floor rebar is introduced to absorb stresses in the floor. The MDI recommends a cross pattern of 10 mm rebar 350 mm on center.

The concrete floor has been analysed for different loadcases in Appendix B. The floor needs a minimum of reinforcement (0,16% in each orthogonal direction). Even if higher airpressures are used, the load case during usage is normative. The reinforcement will not be heavily loaded, especially when considering lifestyle in Sri Lanka. Reinforcement of the concrete will be mainly necessary for stresses caused by temperature as the soil is said to be quite firm [1] and the loads are not that high. It might even be possible to replace the rebar with a few layers of chicken wire in the upper part of the floor. Besides, as the floor is not of structural importance, the foundation could do without a reinforced concrete floor. The question is whether a rammed earth or tile covered floor is accepted.

Another variable load on the foundation after completion of the construction is windload. The total wind force on a hemisphere is smaller than on a rectangular building and it lacks peaks of wind pressure around sharp corners. To have a rough idea of the force on the foundation in case of wind load the assumption is made that the resulting shear force will act over $\frac{1}{6}$th of the circumference of the dome on both sides. According to the Indian Standards (IS: 875) the design wind pressure of $1,2\ kN/m^2$ will result in a total windload of $3,4\ kN$ on a dome of 6 meter diameter. Taking into account a load factor $y_q = 1,5$, this causes a shear force of 2,55 kN over a length of $\frac{1}{6} \cdot \pi \cdot 6m$ with a shell thickness of 0,1 m. The resulting shear stress is $8,1 \cdot 10^{-3}\ N/mm^2$. The maximum shear stress of unreinforced concrete B15 is $0,36\ N/mm^2$, the windload on the foundation is neglegible.

[1] According to Rik Lurinks who supervised the laying of the first foundation in Inspector Eatham

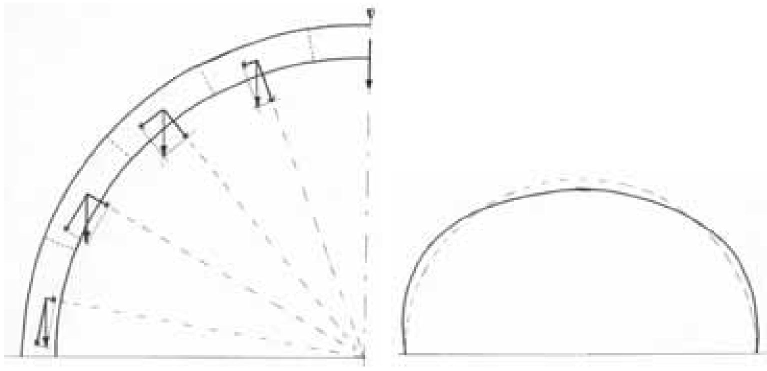


Figure 9.5: Pressure of fresh concrete against the membrane

9.3 Currently applied airpressure sufficient?

According to Haim Heifetz working with airpressures below 3 kN/m^2 'has proved to be very difficult if not impossible to control the shape, location and dimensions of the former, these being very susceptible to distortion by the weight of the applied materials and the force of application as well as to the external influences such as variations in temperature, winds or the like. In consequence, domes produced under such conditions are subject to disintegration in view of the development of planes of rupture and it has in fact been proposed to provide such forms with expensive reinforcing rings designed to support the shell during application and setting.' [Heifetz, 1971]

He proposes 'pressures of inflation of the order of 3 to 6 kN/m^2, which are not so high as to require form materials of exceptional strength or so low as to render difficult if not impossible maintenance of the former shape against the distorting effect of the weight of the applied material and against extraneous disturbing influences, such as winds, temperature variations or the like.' [Heifetz, 1971]

Petra van Hennik also emphasizes that 'at all times, sagging of the fresh concrete caused by a pressure that is too low has to be prevented, because the load-carrying behavior of the resulting shape will be inferior, which can result in failure.'

The weight of the applied fresh concrete is pressing against the form. In the higher part of the hemisphere this resulting pressure is higher than in the lower part (see figure 9.5). As a result the formwork has the tendency to bulge outwards in the lower portions and inwards at the top. In the top the pressure from the fresh concrete equals the weight of the concrete. If the concrete would be applied at once in a 100 mm layer, the air pressure would have to withstand a pressure of 2,4 kN/m^2 in the top. In other words after 60% of the concrete layer is applied at the topportion the balance becomes critical (the currently applied air pressure being 1,5 kN/m^2). The same goes for the concentrated loads of

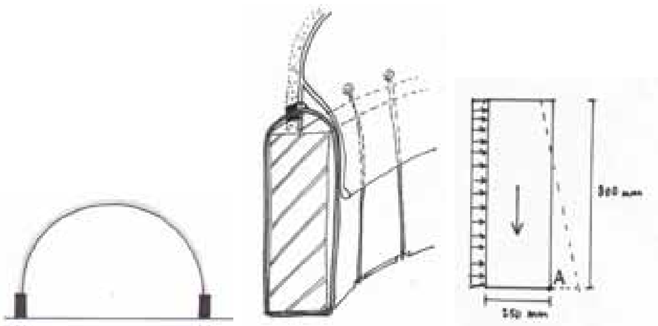


Figure 9.6: Circular wall counteracting uplifting forces

construction workers, who, standing on the formwork, achieve better access to the surface. The NEN dictates to take 1,5 kN concentrated load (to be acting on 1 square meter) into account for loads on roofs.

However the membrane is minimally extendable and will therefore lessen this deformation, although it will wrinkle if the pressure on the top exceeds the air pressure. Also the risk of bulging is reduced by applying the concrete in layers, as the MDI prescribes, not adding a layer before the previous layer is hard to the touch. In case of using a pressure of 1,5 kN/m^2 the first layer should be thinner than 60 mm. Another possibility would be to apply a foam or other material on the balloon that provides a first form-defining layer (like the MDI uses on the inside of their shells see section 4.3.3).

9.4 Alternatives

If higher air pressures are used, the ring beam shall have to be considerably heavier. For an air pressure of 3 kN/m^2 a ring beam of at least 500 by 500 mm is needed for a 6 meter dome, this would be 650 by 650 mm for a 9 meter dome (with load factor $y_q = 1,5$). In this section an overview can be found of alternatives for anchorage, in case higher air pressures are applied. Some of these have been used in practice already, others do not seem to be of practical value. They are grouped into 4 subsections. After presenting an alternative, disadvantages and (potential) problems are listed.

9.4.1 Alternative anchorage

Instead of constructing a permanent foundation for the anchorage of a load (uplift) that is non permanent (only present during construction), the uplift could either be anchored to a permanent structure that does have a function after construction or to a temporary anchor.

Permanent structure: The permanent structure could be a vertical circular wall (figure 9.6). The mass of the wall counteracts the uplifting forces.

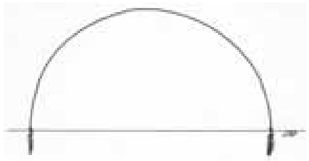


Figure 9.7: Anchoring uplifting forces with groundanchors

Holes at the base of the wall are used to anchor the formwork with cables, these holes can later serve ventilation purposes. The cables need to be protected for the concrete as they enter the exterior of the formwork. The wall could be made of bricks, for example the Compressed Stabilised Earthblocks (see section 6.4), which have a density of approximately 1700 to 2200 kg/m^3. In the previous section was calculated that an uplifting force of 2,25 kN/m has to be absorbed. Taking into account a load factor $y_q = 1,5$, a weight of 337,5 kilogram per meter ring (circumference) would be neccessary to counteract the uplift. This would mean a wall of 250 mm thick and at least 800 mm high.

- Important to note is that if another shape than a hemisphere is used, the uplifting force resultant might not be vertical which results in horizontal forces that need to be absorbed. Concequently a heavier wall will be needed.

- Also, ring tension forces present in the lower section of the hemisphere, created by dead an live load, can not be absorbed by the wall unless it is quite heavy. Therefore a reinforced ring at the base of the hemisphere (on top of the wall) might still be needed.

- When the formwork is pressurized, the walls (openings at the bottom being sealed) are loaded. This is illustrated in figure 9.6 for a ring wall 250 mm thick and 800 mm high. The wall needs to be made wider at the base (dotted line figure 9.6) or heavier.

- Another problem is the fact that the formwork needs to fit well on the wall, to prevent uneven loading of the wall. This requires accurate building.

- Tension concentrates around the holes in the formwork. If the holes are spaced to far apart the formwork might rip.

- Openings in the 'foundationwall' for doors would preferably be made after deflation of the formwork. Another possibility would be to use a i.e. sandbags to fill an opening, but the opening will have to be sealed for airflow.

Figure 9.8: Reusable groundanchors; The Spira-Lock Anchor and the American Earth Anchor, source:www.terra-lock.co.uk

Ground anchor: another alternative is to achor the formwork with reusable ground anchors. An example of a ground anchor can be seen in figure 9.8. The Spira-Lock Anchor [2] is reusable and a 600 mm can resist a vertical pull force of 6 kN, although inital rise starts at 1,4 kN. To anchor a force of 3,4 kN/m, a 600 mm Spira-Lock anchor every 400 mm would do according to Terra-Lock Systems Limited. This results in 47 anchors needed for a 6 meter diameter dome.

- Releasing the anchors is problematic as the wall of the concrete shell curves inwards, there might not be enough space to screw the anchors out completely.
- The amount of anchors that has to be brought in the ground is quite high, so there is not much saving in time needed for anchorage. The fact that some rocks can be found in the soil, complicates the process. Besides, a footing will still be needed for the dome walls to even out pressure on the ground and absorb ring tension.
- The formwork is not very evenly anchored. If the positioning of the anchors is not precise enough this could lead to deformation of the formwork, as well as concentration of tension in the membrane.
- These anchors are not suitable for sandy soils that have low cohesive properties. The soil in Sri Lanka does have cohesive properties, but is sandy and small rocks can be found as well. ABS Alaskan Inc. [3] also provides 460 mm anchors suitable for sandy soil (see 'American Earth Anchor' in figure 9.8). Around the same amount of anchors is probably needed. However, too little is known about soil properties in Sri Lanka to be able to tell whether anchorage with these anchors is possible.
- An initial investment is needed. The American Earth Anchor's are priced $ 26,50 a piece on the web.

[2]http://www.terra-lock.co.uk/SLIntro.htm
[3]http://www.absak.com/catalog/product_info.php/products_id/819

Figure 9.9: Temporary elevated construction

9.4.2 Complete sphere

Uplifting forces could be prevented by using a complete sphere for the formwork. The sphere only needs to be kept in place.

Temporary elevated construction: If the sphere is inflated on ground level, the part of the sphere that is actually used as formwork for a dome is elevated. A possibility is to support the applied concrete on jacks. After setting of the concrete the formwork can be deflated and the jacks can lower the construction to the ground.

- An investment in jacks needs to be made. In case of a 6 meter diameter dome the jacks will have to carry a weight of about 1400 kg.
- It is not a very stable construction, especially if no concrete has been applied yet and considering the wind conditions in Sri Lanka.
- The support stress on the concrete dome needs to be distributed as evenly as possible, otherwise the dome might crack. Either a lot of jacks are required or the supports stress needs to be distributed by a stiff beam, which can not be re-used (as the dome will be on top of it).
- All jacks need to lower the construction at the same speed to prevent uneven loading of the concrete shell.
- High scaffolding will be needed to give workers access to the formwork's surface.
- In section 4.3.1 was mentioned how the formwork deforms under the weight of the fresh concrete. Swelling of a complete sphere will even be larger. The swelling can be reduced by using a higher airpressure, reinforcing the outer horizontal circumference of the sphere (which is under tension) and by using a material for the membrane which is as non-elastic as possible.

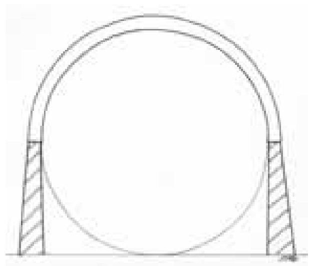


Figure 9.10: Permanently elevated construction

Permanently elevated construction: Another possibility is to support the concrete that is applied on the sphere (inflated on ground level) on a permanent construction. A circular wal should be erected around the circumference of the sphere. The wall will have the same height as the radius of the sphere. If a lower wall is preferred the sphere will have to be partly excavated.

- Swelling, see previous option.
- Due to the weight of the fresh concrete the balloon can deform sideways and eventually press on the wall. As the wall is not reinforced this should be prevented. The outer horizontal circumference of the sphere can be reinforced to deform less under ring tension. Another option might be to introduce a horizontal net or open membrane in the middle of the shere. The horizontal membrane in between can prevent swelling sideways by absorbing tension.
- Floors can only be introduced after the sphere is deflated.
- Assuming the wall is not reinforced, i.e. made of Compressed Stabilized Earth Blocks, it can not absorb ring tension. Therefore tension forces present in the lower section of the hemisphere need to be absorbed by reinforcement in the hemisphere.
- Important to note is that if another shape than a hemisphere is used, the bearing stress resultant of the concrete dome is not vertical and the wall will also experience horizontal loading. A heavier wall would be needed.

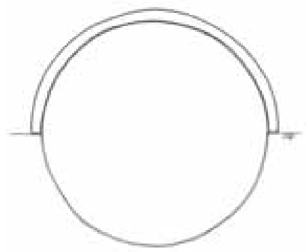
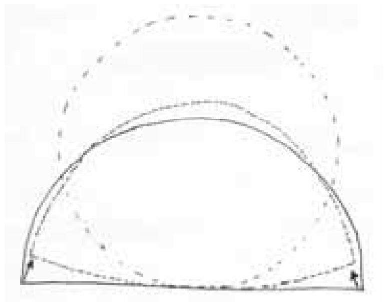


Figure 9.11: Excavated construction

Excavated construction: Another option is to dig a cavity in which the lower half of the sphere fits and apply the concrete on ground level (figure 9.11).

- The volume that needs to be excavated equals the volume of the future dome. For a dome of 6 meter diameter $57m^2$ would need to be excavated, and later put back in place. This requires a lot of time and energy. The cavity might be used as a place for storage, but during monsoon water will enter.
- The concrete dome is constructed on the edge of the cavity. If the soil is not stable enough, it will move after the deflation of the balloon due to the weight of the concrete dome. Consequently the concrete structure would move as well and serious damage could occur.



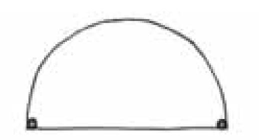


Figure 9.12: Deformation of hollow elastic membrane

Figure 9.13: Stiffened ring

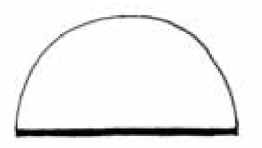
9.4.3 Stiffened closed formwork

The original hemispherical formwork could be made hollow, like a sphere, by adding a horizontal membrane to the formwork. However due to the airpressure the formwork has a tendency to deform into a sphere if the membrane would be elastic, see figure 9.12. However, the membrane is made of a material that does not stretch in any direction [4], which limits the deformation. Nevertheless high pressure could cause the membrane to wrinkle at the base circumference. To further limit deformations one or a combination of the following options could be applied.

Stiffened ring: As seen in figure 9.12 the circumference of the base is loaded under compression. If this circumference is stiffened by a base ring the deformation will be limited(see figure 9.13). However, to be able to remove the form after the concrete has set, the base ring needs to be flexible as well. The ring could be stiffened by water pressure or it could be made out of several parts.

- The stiffened ring will make the formwork more expensive. In case of a water pressurized ring, extra machines are necessary as well and the construction is even more dependent on elektricity supply. Therefore loose parts will probably be more practical. The investment should be compared to the costs of a heavier foundation for the number of domes to be built.

[4] www.bingfo.nl



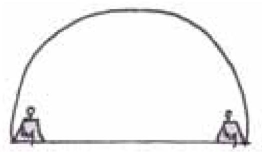


Figure 9.14: Stiffened lower membrane

Figure 9.15: Loaded lower membrane

Stiffened lower membrane: The bottom of the form could be stiffened so that it can resist bending stress, resulting in less deformation of the formwork(see figure 9.14). However the bottom needs to stay flexible as well, to be able to remove the formwork out of the shell after the concrete has set. A floor of linked elements that is placed on top (and connected with) the horizontal membrane might be a solution.

- The stiffened bottom will make the formwork more expensive. This investment should be compared to the costs of a heavier foundation for the number of domes to be built. What should be taken into account is the simplicity of this system compared to a foundation. It simplicity reduces the probability of mistakes in the building process.

Loaded lower membrane: The outer ring of the horizontal membrane could be loaded to prevent uplift of this part of the formwork. This could be done by stones, sandbags or water filled pipes(see figure 9.15). The initial uplifting force will be equivalent to the uplift calculated in section 9.2; 3,4 kN/m circumference(including load factor $y_q = 1,5$). If using soil [5], having a density of approximately 15 kN/m^3, about 7.3 m^3 would be needed to load the outer 0,5 meter circumference.

- A lot of extra work is involved loading and unloading the form.
- If the pressure varies, or a slightly higher pressure is used, the load could start to move and the formwork will deform.

[5]Sweet water being scarce in Inspector Eatham, the sea is about 3 km away

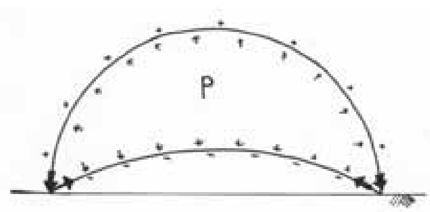


Figure 9.16: Reduce deformation by curved bottom

9.4.4 Exterior support of closed formwork

The hollow formwork as described in the previous subsection could also be supported by an outer framework to reduce deformation. The architect Haim Heifetz (see section 4.3) invented three different systems to remove the need of a heavy foundation for his high pressure formwork (4,0 - 10,0 kN/m^2). These three will be presented in this section. In all three systems the bottom of the formwork is not horizontal, but spherical. The uplift of the outer circumference is counterbalanced by a force resulting from a supporting system of this curved bottom(figure 9.16).

This can be explained by calculating the resulting forces in the supporting framework. The case of an upwardly ('standing') curved spherical bottom and the resulting forces in it's system is shown in figure 9.16. The system can be split into two parts; a tensioned spherical membrane and a spherical membrane under pressure(see figure 9.17). In both cases the load of the air pressure, perpendicular to the membrane, can be replaced by a vertical load as it is a universal pressure situation .

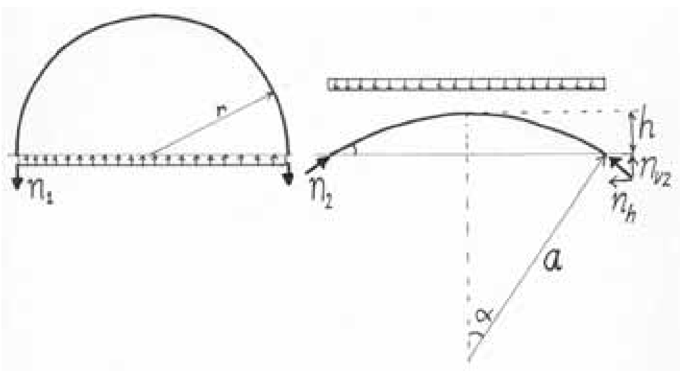


Figure 9.17: The system can be split into a tensioned spherical membrane and a spherical membrane under pressure

Tensioned spherical membrane	*Pressurized spherical membrane*
$n_{v1} \cdot 2 \cdot \pi \cdot r = p \cdot \pi \cdot r^2$	$n_{v2} \cdot 2 \cdot \pi \cdot r = p \cdot \pi \cdot r^2$
$n_1 = n_{v1} = \frac{1}{2}pr$	$n_{v2} = \frac{1}{2}pr$
$n_{v.tot} = 0$	
	$sin\alpha = \frac{r}{a} = \frac{n_{v2}}{n_2} = \frac{h}{r}$
	$n_2 = n_{v2}\frac{a}{r} = \frac{1}{2} \cdot p \cdot a$
	$cos\alpha = \frac{n_h}{n_2} = \frac{a-h}{a}$
	$n_h = n_2 \cdot \frac{a-h}{a} = \frac{1}{2} \cdot p \cdot (a-h)$
$n_h = \frac{1}{2} \cdot p \cdot (a - \frac{r^2}{a})$	

The same rules can be applied if the bottom is curved downwardly ('hanging'). In that case the horizontal force will point in the opposite direction.

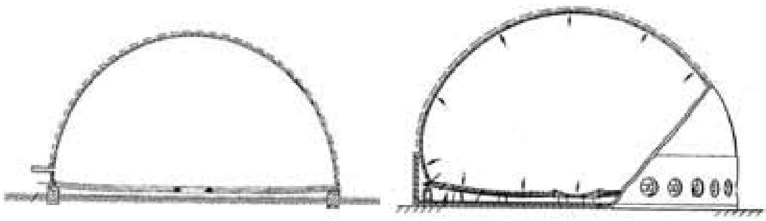


Figure 9.18: Rods system [Heifetz, 1971]

The systems require a higher initial investment in the formwork. This investment should be compared to the costs of a heavier foundation for the number of domes to be built.

Rods system: The formwork is slightly elevated on a circumferential ring. The ring is connected to radially arranged rods, on which the bottom of the formwork rests. The air pressure pushes down the lower membrane on the downwardly curved rods system, tensioning the radially arranged rods or cables (figure 9.18 and 9.19). The rods transfer the fores to the circumferential ring, which is loaded on pressure. The rods prevent the bottom to deform and therewith counterbalance the uplift, creating pressure in the circumferential ring. The ring can be a permanent concrete base ring or the ring can be a part of the temporary construction, see figure 9.19.

- The fastening of the rods and the membrane on a permanent ring, makes the design of the permanent base ring rather complicated, see figure 9.19. As a part of the temporary construction the ring can be re-used and requires less actions.
- The formwork can not be placed directly on the ground (to allow for the downwoard displacement of the tensioning cables).

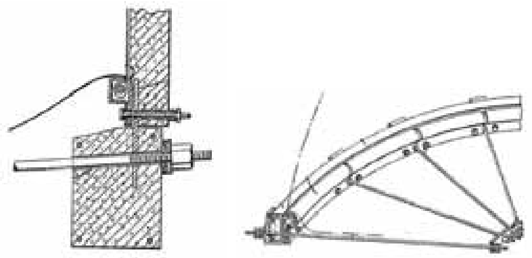


Figure 9.19: Rods system [Heifetz, 1971]

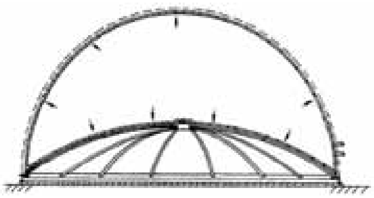
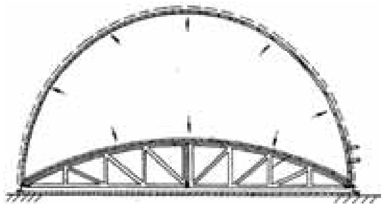



Figure 9.20: Arches system [Heifetz, 1971]

Figure 9.21: Trusses system [Heifetz, 1971]

Arches: In this system the bottom of the formwork is spherical (figure 9.20). It is supported by a system of radially arranged struts, whose inner ends are coupled to a central hub and whose outer ends are secured to a base ring. When the formwork is filled with air, the lower membrane presses down on the compression struts. As a result these struts press against the ring which is tensioned. The vertical component of the resulting force in the rods counterbalances the uplifting force in the membrane, there is no necessity to anchor the base ring to the ground. The base ring and the compression struts are re-usable.

Trusses: This system is basically the same as the previously described system. The only difference is that the bottom of the formwork is supported by trusses that can absorb tension created by the airpressure on the spherical bottom, see figure 9.21. Concequently no heavy base ring is needed. The trusses are demountable, so that they can be removed after the concrete dome has set.

ALTERNATIVES

As a result of research on domes in general (page 19) and analyses of the Solid House domes (page 71) alternatives were developed. Chapter 10 focusses on an alternative material, while making no changes to the application of the inflatable formwork nor to the formwork itself. However to improve issues such as the heavy foundation and the dependency on electricity, a different design of the formwork is required. Therefore possibilities for an alternative design of the formwork are discussed in chapter 11. All alternatives are adaptations of the currently applied building concept; incorporating an inflatable formwork and resulting in a domeshell as this is within the scope of the thesis. The alternative suggested in chapter 10 is designed to provide the Solid House Foundation with proposal(s) for improved use of the 'traditional balloons', while the alternatives in chapter 11 have an impact on the appearance of the domes and require a (sometimes high) initial investment.

Chapter 10

Traditional Balloon, New Shell

From analyses of the shell in ANSYS (section 8.3) followed that stresses in the Solid House shells are very low. Consequently a much thinner shell is possible and there is no need for such heavy reinforcement as is currently applied. Therefore a design is made by using ferrocement as material, as recommended in chapter 6. However this does not solve the issue of the heavy foundation nor the problem of keeping the balloon up to pressure. To solve these issues adaptation of the inflatable form is necessary.

10.1 Ferrocement Design

In section 6.3 a first introduction to ferrocement can be found. Ferrocement is a suitable material for roofing because of its relatively low cost, durability and weather resistance. Ferrocement can be easily shaped into domes, vaults, extruded type shapes, flat surfaces or free-form areas. Often it is used for on-site-manufacure of tiles or other roofing elements. For example in Auroville City, Pondicherry (India) the Arobindo Ashram built houses with ferrocement roofings in 1977. The unsupported span of these houses varies from 5 to 15 meter with ferrocement roofs 50 mm thick at the most . In Sri Lanka the technique of ferrocement is often applied for the construction of water tanks. Either two layers of hexagonal mesh or one layer of square woven mesh is used. Generally practical reinforcement is applied for ferrocement structures.

Several scientists studied the behaviour of ferrocement. Their findings resulted in advice on the design of ferrocement structures [Naaman, 1985] of which an extract can be found in this section.

The design strength for the mesh reinforcement shall be based on the yield strength f_y of the reinforcement but shall not exceed 690 Mpa. Recommended design yield strengths of various meshes are given in table 10.1 and may be used instead of test data.

The area of reinforcement per layer of mesh considered effective to resist tensile

		Woven Square Mesh	Welded Square Mesh	Hexagonal Mesh	Expanded Metal Lath	Longitudinal Bars
Yield Strength	f_y, ksi (MPa)	65 (450)	65 (450)	45 (310)	45 (310)	60 (414)
Effective Modulus	$(E_r)_{long.}$, 10^3ksi (10^3 MPa)	20 (138)	29 (200)	15 (104)	20 (138)	29 (200)
	$(E_r)_{tran.}$, 10^3ksi (10^3 MPa)	24 (165)	29 (200)	10 (69)	10 (69)	- -

Table 10.1: Minimum values of yield strength and effective modulus for steel meshes and bars recommended for design [Naaman, 1985]

stresses in a cracked ferrocement section can be determined as follows.
$A_{si} = \eta V_{fi} A_c$

A_{si} effective area of reinforcement for mesh layer i
η global efficiency factor of mesh reinforcement in loading direction considered, see table 10.2
V_{fi} volume fraction of reinforcement for mesh layer i
A_c gross cross sectional area of mortar section

$V_{fi} = \frac{Volume\ of\ mesh}{Volume\ of\ ferrocement\ section}$
$= \frac{theoretical\ thickness\ steel}{thickness\ mortar\ section}$
$= \frac{total\ kg/m^2\ mesh}{density\ steel\ \cdot thickness\ mortar\ section}$
$= \frac{NW_m}{\rho_m h}$

N number of mesh layers
h thickness of ferrocement
W_m unit weight of mesh
ρ_m density of steel

10.1.1 Tension

The nominal resistance of cracked ferrocement elements subjected to pure tensile loading can be approximated by the load-carrying capacity of the mesh reinforcement alone in the direction of loading.

$N_n = A_s f_y$

N_n nominal tensile load resistance in direction considered
A_s effective cross sectional area of reinforcement in direction considered
f_y yield strength of mesh reinforcement, see table 10.1

		Woven Square Mesh	Welded Square Mesh	Hexagonal Mesh	Expanded Metal Lath	Longitudinal Bars
Global Efficiency Factor	Longitudinal: η_l	0.50	0.50	0.45	0.65	1
	Transverse: η_t	0.50	0.50	0.30	0.20	0
	At 45°: $\eta_{\theta=45}$	0.35	0.35	0.30	0.30	0.70

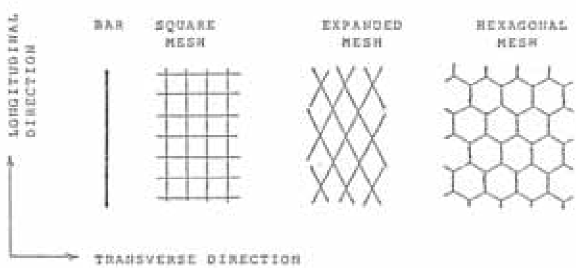


Table 10.2: Recommended design values of the global efficiency factor of reinforcement η for uniaxial tension or bending [Naaman, 1985]

The value of A_s is given by
$A_s = \sum_{i=1}^{n} A_{si}$

n number of mesh layers
A_{si} effective area of reinforcement for mesh layer i

10.1.2 Compression

The nominal resistance of ferrocement sections subjected to pure compression can be estimated as a first approximatin from the load-carrying capacity of the unreinforced mortar matrix assuming a uniform stress distribution of 0.85 f_c', where f_c' is the design compressive strength of the mortar mix.

10.1.3 Flexure

The cracking moment M_r in Nm/m:
$M_r = f_{br,0}\frac{1}{6}h^2$
h is the height in mm
$f_{br,0} = (1,6 - h \cdot 10^{-3}) f_{bm,0}$
$f_{bm,0}$ is the average tension strength of the concrete

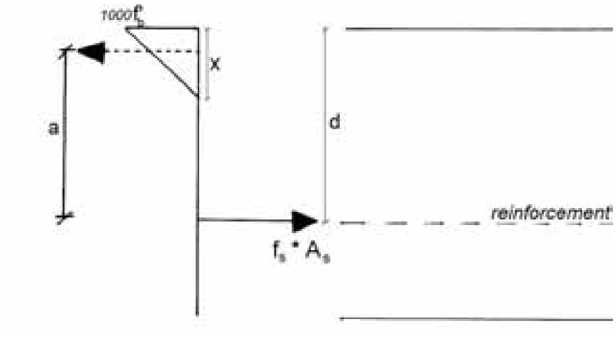


Figure 10.1: A cross section's ultimate moment capacity

The ultimate moment capacity M_u in Nm/m: (see figure 10.1)
$M_u = N_s a$
$N_s = A_s f_s = N'_b$ because $\sum H = 0$
N_s is the tension force in the mesh
A_s is the surface of mesh per meter cross-section
$f_s = f_{srep} \frac{1}{1,15}$ is the maximum tension stress in the mesh
N'_b is the pressure force in the concrete
$a = d - \beta x$ is the distance between the resultant pressure and the resultant tension force acting on the cross-section
d is the distance from the reinforcement to the edge where pressure occurs
β is a contant and is 0,39
$x = \frac{N'_b}{1000\alpha f'_b}$ is the height of the pressure zone
α is a constant and is 0,75
$f'_b = 0,6 f'_{ck}$, pressure strength concrete

To avoid brittle failure $M_r \ll M_u$

If there is a normal force present this should be taken into account. If a tensile force N is present:

$M_r = (f_{br,0} - \frac{N}{A})\frac{1}{6}h^2$
N being the tension force and A the surface of the cross-section.
Concludingly M_r is reduced with $\frac{1}{6}hN$

$\sum H = 0$ so $N'_b = N_s + N$, see figure 10.2
$x = \frac{N'_b}{1000\alpha f'_b}$
$a = d - \beta x$
$M_u = (N_s - N)a + N \cdot e$
Concludingly M_u is reduced with $(a - e)N$

If M_u is reduced more than M_r, so if $a - e > \frac{1}{6}h$, brittle failure should be checked.
To avoid brittle failure $M_r \ll M_u$

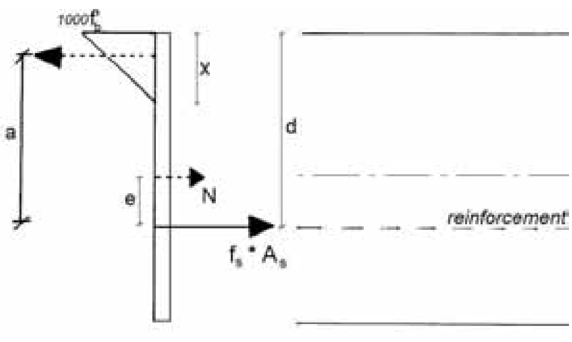


Figure 10.2: A cross section's ultimate moment capacity when a tension force is also present

10.1.4 Shell Thickness, Mesh Cover, Number of Layers

Maximum thickness of the shell is advised 5 cm. The desired volume fraction of steel will require so many layers of mesh in this case, that reinforcement with rebar is more effective and cheaper.

Concerning meshes; chicken wire is best available throughout the world, and relatively cheap. Square mesh performs best on impact loading. Also, the minimum value for yield strength and the efficiency factor for square meshes are higher. Therefore if available, mostly square mesh is applied. For a given ferrocement material of thickness t, the recommended mesh opening S should not be larger than t. Standard chicken wire dimensions are 0,7 mm wire and 13 mm openings.

Advice on the number of layers is rather vague. It is generally desirable that the number of mesh layers be not less than two, unless a heavier mesh is used. Some advice a maximum of 5 layers per cm and a volume fraction of the mesh of maximum 8%. Naaman [Betonvereniging, 1985] suggests a number of 1,6 times the shell thickness in cm. A minimum number of two layers is advised in most sources. Det Norske Veritas [Betonvereniging, 1985] advises a minimum reinforcement percentage of 1,75%. For a given volume fraction of reinforcement higher performance (not necessarily strength) can be achieved by uniformly distributing the reinforcement throughout the thickness and by increasing its specific surface. Important to note is that local increases of thickness of the mortar section are allowed (for example thicker rims), given that the present reinforcement is increased as well. This is necessary to keep the percentage of reinforcement up to level.

Steel cover is mostly not more than 2 to 3 mm and ranges from 1,5 to 5 mm. While for reinforced concrete the average cover ranges between 20 to 30 mm. IASS [1] proposes a minimum of 4 mm on the mesh and 8 mm on steel rods. The net cover of the reinforcement should be of the same order as twice the equivalent diameter of the mesh wire or other reinforcement used. However,

[1] International Association of Shell Structures, Ferrocement Workgroup [Betonvereniging, 1985]

Figure 10.3: Ferrocement roofs for houses by Arobindo Ashram in India [Paul and Pama, 1978]

a smaller cover is acceptable provided the reinforcement is not susceptible to rapid corrosion, the surface protected by an appropriate coating and the crack width limited [Naaman, 1985]. For ferrocement elements of thickness less than 25 mm a cover of the order of 2 mm has given satisfactory results. Some sources even propose a maximum cover of 5 mm. If skeletal reinforcement is used, it is recommended that the skeletal reinforcement does not occupy more than 50 percent of the thickness of the ferrocement material.

Although the mortar of the ferrocement is much denser than normal concrete, the cover is relatively small. It is advised to add ca. 300 ppm chroomtrioxide to the mortar mixture. The chroomtrioxide provides extra protection to the steel against corrosion. Also, it passivates electrochemical reactions [2] in the fresh concrete. Which prevents the forming of hydrogen gas along the surface of the steel rods in the fresh concrete [3].

[2]Galvanic actions occurs when two metals of different electrical potential are in contact with each other. It results in an accelerated rate of corrosion for the least noble of the two metals. In ferrocement, the two dissimilar metals are zinc coating of the galvanized mesh and iron in the ungalvanized steel reinforcement bars. The zinc coating of the mesh is the least noble of the two and will thus corrode.

[3]Up until the time the mortar sets, a large electron current flows from the zinc anode to iron cathode, where hydrogen ions acquire electrons and form hydrogen atoms, which is liberated as hydrogen gas, along the surface of the black steel anode.

10.2 Ferrocement Shell Performance in ANSYS

To have an idea of stress levels in a thinner shell a ferrocement shell is designed for analyses in ANSYS. A hexagonal mesh (chicken wire) is chosen, being cheap and widely available. The shell is designed 30 mm thick with three layers of chicken wire, as it is adviced to use a minimum of two layers. The 13 mm mesh has a unit weight of $0,58\ kg/m^2$ and consists of $0,7\ mm$ diameter wire. Parameters are defined according to section 10.1.

10.2.1 Parameters

$V_{fi} = \frac{3 \cdot 0,58\ kg/m^2}{7850\ kg/m^3 \cdot 30\ mm} = 0,74\ \%$ *Explanation on this parameter on page 114*

From table 10.2 and 10.1 follow $\eta = 0,3$ and $f_y = 310\ N/mm^2$. The reduction factor for steel is $\gamma_{m,steel} = 1,15$.

This results in a tension capacity of $\sigma = \frac{V_f \eta f_y}{\gamma_{m,steel}} = 0,60\ N/mm^2$

The compression strength [4] of the matrix is

$\frac{0.85\ f'_c}{\gamma_{m,concrete}} = 6,4\ N/mm^2$

The Youngs' module of the matrix is calculated in Appendix C.3 and amounts to 18 000 N/mm^2.

10.2.2 Wind and Gravity load in ANSYS

The ferrocement shell is applied on the dome designed by the SHF denktank (Appendix D). The model is loaded by a combination of wind and gravity load [5].

The resulting maximum principal tension stress in the shell is $0,3\ N/mm^2$, with peak stresses at the base of the doorjambs up to $2,02\ N/mm^2$. From the lower corner of the doorjambs this peak stress is dropping to $0,6\ N/mm^2$ within 0,2 m in horizontal and 0,5 m in vertical direction. Slightly higher stress levels (maximum $0,4\ N/mm^2$) were also indicated on the upper side of the opening, where the shell is connected to the cover. These result from the 'pressure meridians' spreading around the opening, see figure 8.9. The maximum moment can be found at the rim of the cover over the opening that is leeside from the wind and amounts 10,4 Nm/m. The maximum moment in the double curved shell is 2 Nm/m in all directions.

[4] $\gamma_{m,concrete} = 1,2$ and for B15 $f'_c = 9\ N/mm^2$

[5] Gravity load amounting $\gamma_g \cdot g = 1,2 \cdot 9,8\ m/s^2$. The wind load is modelled as a horizontal gravity of $1,63\ m/s^2$, creating a total horizontal load of $\gamma_q \cdot Q_w = 1,5 \cdot 12,1 kN$. This total horizontal load is based on the Indian Standards for wind loads on cylindrical and spherical structures.

Resulting maximum reactions on the ringbeam					
F_x	7,2	kN/m	M_x	0,4	Nm/m
F_y	32,4	kN/m	M_y	56	Nm/m
F_z	14,2	kN/m	M_z	42	Nm/m

F_x creates a maximum ringtension force of $T = a \cdot 7,2 = 32\ kN$. Based on the latter a minimum ringreinforcement of the beam of one rebar of 11 mm diameter is needed. If the ringbeam has at least a cross-section of 36 000 mm^2 ($190 \times 190\ mm^2$), tension stress in the cross section is reduced below $0,9\ N/mm^2$ (theoretical tension capacity of B15 concrete).

Shear stress between the ringbeam and the shell, as a result of F_z, amounts to $0,47\ N/mm^2$. The maximum unreinforced shear stress of B15 concrete is $0,36 N/mm^2$. A reinforced connection between ringbeam and shell is thus necessary to absorb shear force, apart from the fact that reinforcement is necessary to create bonding between the foundation and the shell. Per meter perimeter at least 40 mm^2 steel is necessary.

F_y causes a maximum compression strength of $1,08\ N/mm^2$ which is below the compression strength of the matrix. The shear stress caused by the torsional moment M_z in a $200 \times 200\ mm^2$ ringbeam is with $0,025\ N/mm^2$ below the shear stress of concrete.

The maximum displacement amounts 0,6 mm.

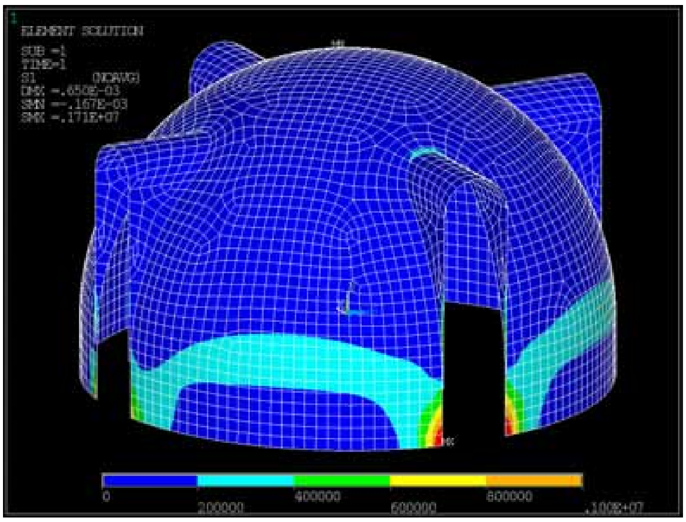


Figure 10.4: Design in ferrocement modelled in ANSYS and subjected to wind and gravity load. The contours show the principal tension stress in the middle of the shell. These are stresses as a result of 'shell forces'. Maximum (peak) stress next to door jambs, caused by openings that interrupt ringtension forces, amounting $2,03\ N/mm^2$. Stresses resulting from edge disturbances are neglegible. See section 8.1.

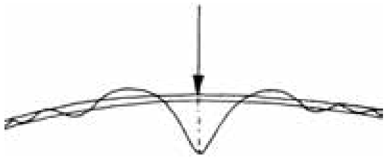


Figure 10.5: Moment distribution caused by concentrated load

10.2.3 Concentrated and Gravity Load

A concentrated load on a shell will result in a moment distribution as can be seen in figure 10.5. The dome needs to be accessible for maintenance. Imagine a person standing on one foot. The person weighs 70 kilograms and his foot has a surface of 130 cm^2. The pressure he imposes on the surface will amount $0,055N/mm^2$ (or MPa). A concentrated load of 709 N on a surface of 129 cm^2 in combination with a gravity load of $9,8$ m/s^2 is modelled on top of the dome in ANSYS, which results in a maximum moment of $1,52Nm/m$. The same load on a flat plate would result in a moment of about $\frac{1}{8}700 = 88$ Nm/m.

The 'one foot' concentrated load (709 N on a surface of 129 cm^2) is successively modelled on different locations on the dome, combined with a gravityload of $9,8$ m/s^2. In the double curved part of the shell the resulting maximum moment is $1,52$ Nm/m. Yet the covers above the openings are only singly curved. As a result the same concentrated load on the rim of one of these covers causes a higher maximum moment in the shell, amounting $6,76$ Nm/m. The tension in the middle layer at that location have a maximum value of $0,16N/mm^2$. In figure 10.7 the stresses that are caused at the underside of the shell (outer fibers) by the concentrated loads on the diffent locations is shown. It is clear that the concentrated load on the cover results in higher stresses.

Figure 10.6: Concentrated load [Viguurs]

According to the dutch regulations [inf, 2002] a roof should be able to bear a load of 1,5 kN distributed over an area of $0,5 \times 0,5$ m^2. Therefore this load, multiplied by $\gamma_q = 1,5$, is modelled as a pressure load on the the rim of one of the covers. The resulting maximum moment and principal stresses are plotted in figure 10.8 and 10.9 respectively. The maximum moment of 8,1 Nm/m is measured at the rim, but does not coincide with a high tension force (tension stress in the shell's middle layer is below 0,1 Nm/m at this spot).

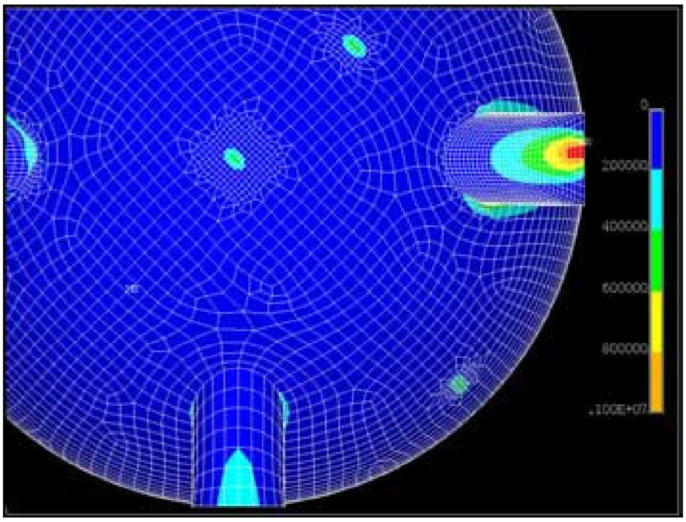


Figure 10.7: On different locations on the dome surface a concentrated load of 709 N is distributed on an area of 129 cm^2, representing the load of a person of 70 kg standing on one foot. The maximum resulting tension stress of $1,36 N/mm^2$ occurs at the rim of the covers, which are only single curved. The picture shows the underside of the shell. These stresses are thus a combination of stresses caused by shell forces and by moments in the shell.

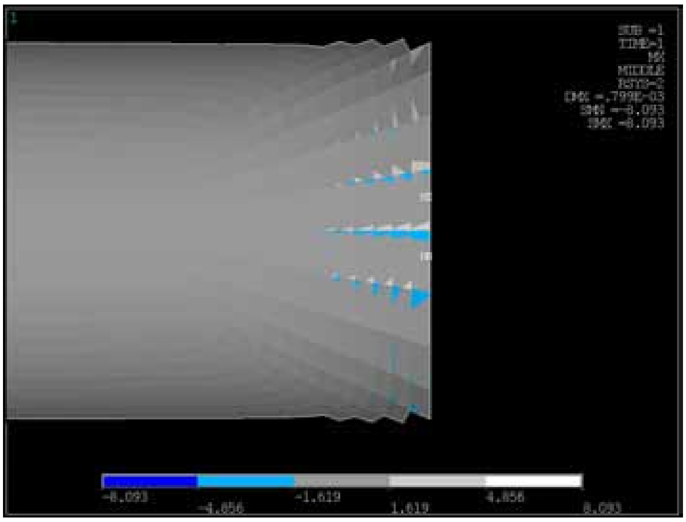


Figure 10.8: A load of $\gamma_q \cdot 1,5$ is loaded on a surface of $0,49 \times 0,49$ m^2 the rim of one of the covers and combined with a gravity load of $\gamma_g \cdot g = 1,2 \cdot 9,8$ m/s^2. As a result a maximum moment of 8,1 Nm/m can be found at the rim of the cover.

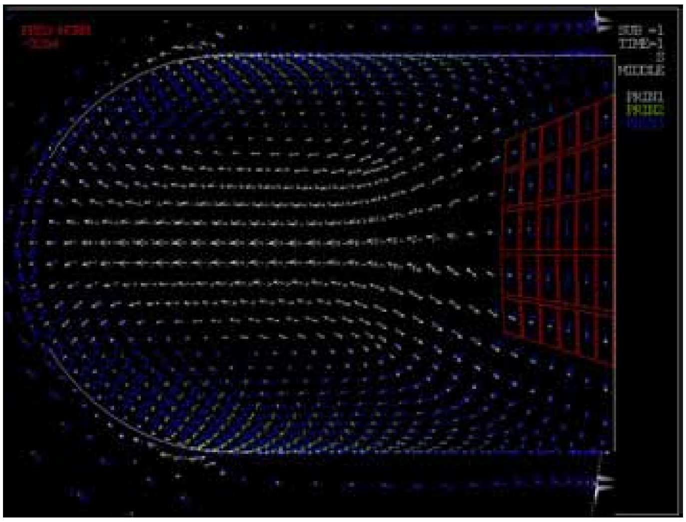


Figure 10.9: The principal stresses in the middle layer of the shell are shown in this vector plot. These result from the same load combination as for figure 10.8. The location of the load is shown with red marks. Directly under the concentrated load pressure prevails, pressure occurs in the curved direction, maximum tension stress amounts $0,1N/mm^2$. Tension prevails in the direction of the shell in the uncurved direction of the cover. Maximum tension stress amounts $0,15N/mm^2$.

10.2.4 Buckling Analyses

By calculating the theoretical buckling load in ANSYS, the sensitivity of the shell to small variations in the shell thickness and geometry can be checked. If the theoretical buckling load is more than 6 times higher than the normative load case, deviations of thickness and deviations of geometry of around half the thickness of the shell are acceptable.

ANSYS approaches this as an eigenvalue problem with the form:
$[K](\phi_i) = \lambda_i[S](\phi_i)$

Where [K] is the structure stiffness matrix, (ϕ) is the eigenvector, λ_i is the eigenvalue and [S] is the stress stiffness matrix. The eigenvalue is the multiplication factor of the load case and is increased until the theoretical buckling load is reached. A block shifted Lanczos algorithm is the theoretical basis of the eigensolver. The method employs an automated shift strategy, combined with Sturm sequence checks, to extract the number of eigenvalues requested.

The shell from the previous section is subjected to several load cases, resulting in the following eigenvalues:

$\gamma_g \cdot g + \gamma_q \cdot Q_w$	$\lambda = 192,3$
$\gamma_g \cdot g + concentratedloadof \gamma_q \cdot 1500N$	$\lambda = 217,1$
$\gamma_g \cdot g$	$\lambda = 257,1$
$concentratedloadof \gamma_q \cdot 1500N$	$\lambda = 260,5$

The minimal eigenvalue turns out to be much larger than 6. Small deviations in the shells dimensions should therefore not be a problem.

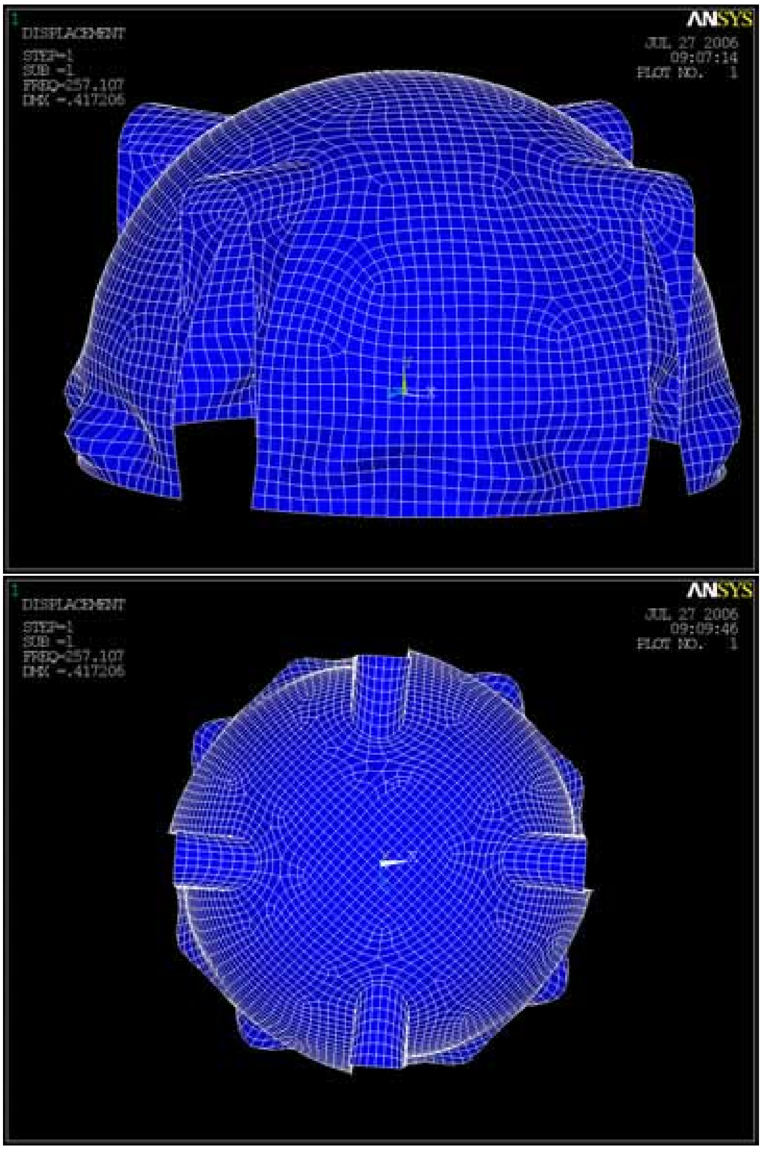


Figure 10.10: Buckled shape of a dome under theoretical buckling load of loading conditions $\gamma_g \cdot g$

Theorethical background:
The graph in figure 10.11 shows the theorethical failure of a hemisphere under pressure, the upper line indicating the theoretical buckling stress which can be calculated with a finite element package, i.e. ANSYS, slightly lower the critical stress that can be determined analytically. The diagram in figure 10.12 shows the resuls of experiments in which hemispherical shells (each having small imperfections) were loaded perpendicularly to their surface until failure [Hoefakker and Blaauwendraad, 2005]. The values along the vertical axis indicate the proportion of the critical stress. Remaining stress after failure (durchlschlagen) has a minimum stress level of about $0,1\sigma_{cr}$. In the graph examples of experimental values have been indicated by dotted lines. Concludingly the knockdown factor of a shell is 6, in other words: σ_{cr} should be at least 6 times larger than the maximum applied load on the shell to allow for imperfections. However this experiment was not carried out for ferrocement domes but probably for aluminium shells, also the pressure is perpendicular to the surface which is not the case for the ferrocement dome and the theoretical buckling load is slightly larger than the critical stress. Yet the knock down factor found for the ferrocement dome shell turns out to be around 200, this is so large that these differences are not significant. The structure can be regarded safe for small imperfections (about 0,5 times the thickness of the shell).

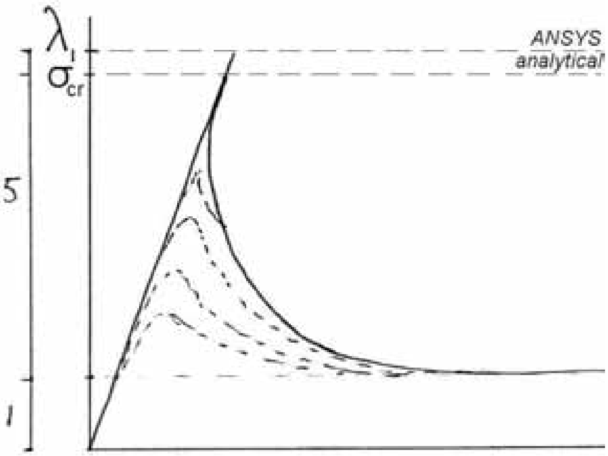


Figure 10.11: Theorethical buckling load λ_1

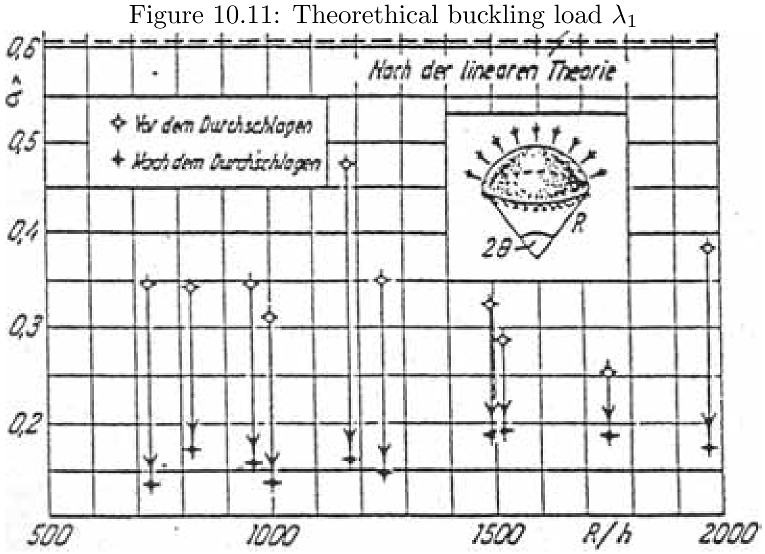


Figure 10.12: Failure of hemispheres under loading perpendicular to the surface [Hoefakker and Blaauwendraad, 2005]

10.3 Design for solidhouses

In this section a definitive design of the shell is made, based on guidelines of section 10.1 and the outcomes of the analyses in section 10.2. The first paragraph consists of a discussion on the shell design if a hexagonal mesh is applied, the second paragraph applies a square woven mesh. An experiment has been carried out by the author to check brittleness of the designed matrix and potential practical problems. This is described in paragraph 10.3.3.

10.3.1 Defining Shell Dimensions

Hexagonal mesh is chosen for the design as it is most widely available. The mesh has 13 mm openings, a unit weight of $0,58\ kg/m^2$ and consists of $0,7\ mm$ diameter wire. The tension capacity for different combinations of shell thickness and number of mesh layers is calculated with the method described in section 10.1.1 (applying safety factors).

Number of mesh layers and shellthickness

Thickness ferrocement shell (mm)	50	50	50	30	30	30	30
Number of layers of hexagonal mesh	5	8	15	2	3	4	5
Tension capacity (N/mm^2)	0,6	1,0	1,8	0,4	0,6	0,8	1,0

The maximum tension stress occurs as a result of a load combination of wind and gravity load, see section 10.2.2, amounting maximum $0,3N/mm^2$ up to $0,4N/mm^2$ above the openings. In applications of ferrocement generally 'practical reinforcement' is applied. A number of two layers of mesh is most common. To keep the level of reinforcement rather high (ductility) and at the same time use a practical number of layers of mesh a shell thickness of three centimeters is proposed, applying 3 layers of mesh. The tension capacity of this matrix will be $0,6N/mm^2$, which is 1,5 as high as the maximum tension stress in the shell in ultimate limit state of a wind and gravity load combination.

The same load combination leads to a maximum moment of 10,4 Nm/m in the outer rim of one of the covers above the openings and 2 Nm/m in the shell (see section 10.2.2).

Location of the mesh in the cross-section
After the mesh has been applied on the inflated membrane [6] 2 cm of mortar is applied on the surface of the dome. When the formwork is deflated one centimeter of mortar is plastered on the inside. This is most practical as it would be difficult to plaster more than 1 centimeter on the inside. Besides, protection of the mesh is needed more on the outside and edge disturbances are largest on the inside of the shell.

This design has a cracking moment of:
$Mr = f_{br,0}\frac{1}{6}h^2 = \ 424\ Nm/m$
$h\ = 30\ m$

[6] Concrete spacers are not used (to keep distance between the mesh and the inflated formwork) as the mesh would deform around the spacers.

$f_{br,0} = (1,6 - h \cdot 10^{-3}) f_{bm,0} = 2,83\ N/mm^2$
$f_{bm,0} =\ 1,8\ N/mm^2$ for B15 concrete

And an ultimate moment capacity of: $M_u = N_s a =\ 990\ Nm/m$
$N_s = A_s f_s$
$A_s =\ 221,6\ mm^2/m$
$f_s =\ 270\ N/mm^2$
$N'_b = N_s$
$a = d - \beta x =\ 11,1\ mm$
$d =\ 20\ mm$
$\beta =\ 0,39$
$x = \dfrac{N'_b}{1000 \alpha f'_b} =\ 8,9\ mm$
$\alpha =\ 0,75$
$f'_b = 0,6 f'_{ck}$
$f'_{ck} =\ 15$ for B15 concrete

$M_u > M_r$ so the design is ductile and the moment capacity is much larger than the maximum moment for a (ultimate limit state) load combination of wind and gravity.

Moments resulting from a concentrated load are highest if the load is positioned on one of the cover's rim, see section 10.2.3. But with a value of 8,1 Nm/m this is far below the cracking moment of the designed shell. Tension stresses that occur in combination with a moment are maximum $0,3\ N/mm^2$. This tension stress reduces the moment capacity only slightly; M_r decreases to 379 Nm/m and M_r to 935 Nm/m (see page 116).

For the calculations the properties of B15 concrete have been used. However this is an assumption. If the applied mortar turns out to be stronger, moment capacities increase and the matrix is still ductile.

Yet the number of layers advised in [Betonvereniging, 1985], [Paul and Pama, 1978] and [Naaman, 1985] is generally higher (see section 10.1.4). Naaman suggests a number of 1,6 times the shell thickness in cm, which would mean 5 layers in case of a 3 cm thick shell. Det Norske Veritas advises a minimum volumepercentage of the mesh of 1,75 %, while this design only applies a volumepercentage of 0,74%. This is probably caused by the small steel cover that is normally applied to ferrocement (mostly up to 5 mm). To be sure of the failure behaviour and have a better idea of the mortar strength, but especially to see more of the practical side of producing ferrocement, an experiment is carried out.

Shear
Shear stress between the ringbeam and the shell, as a result of F_z, amounts to $0,47\ N/mm^2$ (see section 10.2). Consequently a shear force of $14,1\ kN/m$ has to be absorbed. Assuming the mortar matrix does not contribute to the shear capacity, $\frac{14,1}{350} = 40,3\ mm^2$ of steel is needed per meter perimeter. A 6 mm steel upright every 0,5 meter around the perimeter will be sufficient. However next to the openings an extra upright within 0,1 meter should be planned to absorb stresses from tension forces, which are diverted to the foundation by the opening.

10.3.2 Adaptation: Square Woven Mesh Instead Of Hexagonal Mesh

During the completion of the design and calculations with hexagonal mesh, information was received from Tomas Viguurs [7] about available meshes. Square mesh turned to be well available as it is often applied for the construction of ferrocement water tanks. Most common in use is 18 gauge square woven mesh. This mesh has a wire diameter of 1,2 mm, openings of 10 mm and a unit weight of $1,56\ kg/m^2$. One layer of this mesh combined with a shellthickness of 30 mm, results in a tension capacity of $1,3\ N/mm^2$ of the matrix (see methods in section 10.1.1). The higher capacity origins in the orientation of the wire [8] and the higher density of steel per square meter mesh.

Concerning the moment capacity of a 3 cm shell with one layer of square woven mesh; $M_r = 424\ Nm/m$ and $M_u = 1206\ Nm/m$. Concludingly this design will be safe to resist the maximum moments occuring in the shell as a result of concentrated loads and windloads as described in chapter 10.2. Also, the design shows ductile behaviour in spite of it's low volume percentage of steel (0,55 %). One layer may be less than advised in literature, but in 'Sri Lankan' practise it is standard for this type of mesh and the design is safe against brittle failure. Besides, one should realise that in practise most spots will have a cover of 2 layers, caused by the way the mesh is applied, see Appendix G step 25 to 31.

The 18 gauge square woven mesh has a price of 1,76 $\$/m^2$ compared to a price of 1,27 $\$/m^2$ for hexagonal mesh. Three layers of chicken wire are thus more expensive than one layer of 18 gauge square woven mesh. Concludingly for 18 gauge square woven mesh less mesh layers needs to be applied, performance of the shell is better on tensile stress and impact load and the total price is lower.

Therefore the design is changed to one layer of square woven 18 gauge mesh. However the experiment of section 10.3.3 had already been set up by the time of this decision.

[7] based in Sri Lanka for the Solid House Foundation

[8] The global efficiency factor increases from 0,3 to 0,5 and the minimum value of the yield strength increases from 310 to 450 N/mm^2. See tabel 10.2 and tabel 10.1

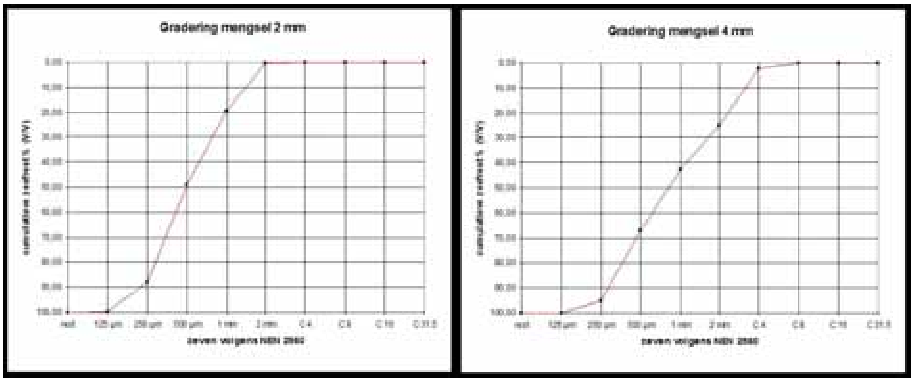


Figure 10.13: Graphs of the two sandfractions used for the first experiment

10.3.3 Experiment

Goal of this experiment[9] is to test mix proportions before prescribing them in the manual, as well as to observe what problems could arise in the application process. As the volumefraction of steel in the matrix [10] is below the normally required volumefraction, it is also desired to investigate the failure behaviour. If the volume fraction is too low, the tension strength of the chicken wire might not be lower than the tensile capacity of the mortar, which would result in brittle failure, which is undesirable. The experiment will check the behaviour of a flat plate with a cross-section similar to the cross-section of the shell designed in section 10.3.1: a thickness of 3 cm with 3 layers of chicken wire located at 1 cm from the bottom of the plate will be loaded by a concentrated load.

First a mix is designed. All standard cements can be used, though Portland cement is recommend. The percentage of cement in the mortar mix is much higher than for general purposes. The prescribed sand-cement proportions vary among the different articles. A sand-cement ratio of 2 is suitable according to most articles. The water-cement factor should be kept low to achieve a strong shell. Experiments with watertanks showed that for ratio's above 0,5 leakage can increase rather rapidly. Therefore it is advised to use a water-cement factor between 0,4 and 0,5. [Betonvereniging, 1985]

Concerning sandfractions, advise ranges from a maximum graindiameter of 2 to 5 mm. Consequently it is decided to test two mixtures before making a thin plate. One with a relatively fine sandfraction and one with a rougher sandfraction that corresponds to 'standard sand' used in the Netherlands. The graphs in figure 10.13 show their grain distributions. The mixes are each divided over three moulds of 40 mm deep, 40 mm wide and 160 mm long.

[9]Carried out by the author
[10]0,74 % or 19,3 kg steel/m^3 concrete

Figure 10.14: Mixtures with two different sandfractions are tested on pressure (left picture) and bending (right picture) capacity ($MPa = N/mm^2$)

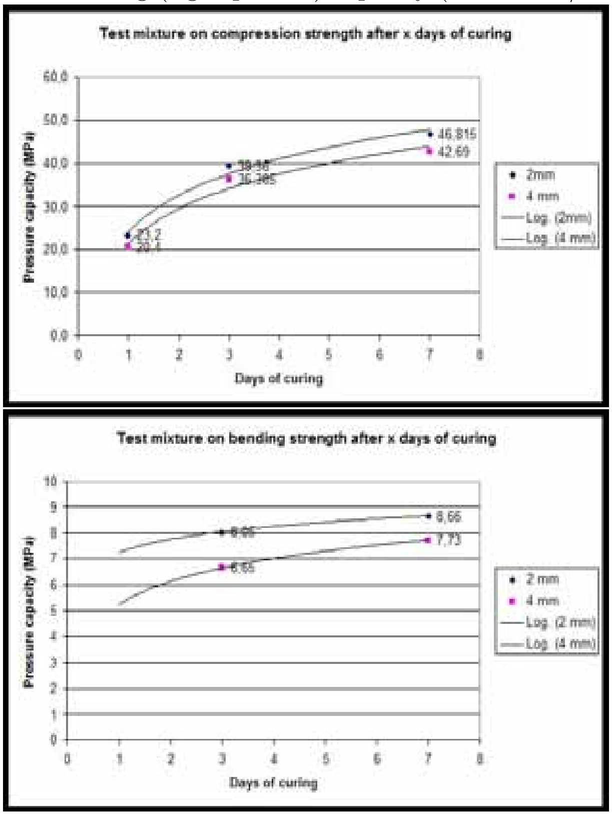


Figure 10.15: Increasing pressure and bending capacity of the two sandfractions over a number of days.

Ingredients		Mass kg/m3 concrete	Volume liter/m3 concrete
CEM I 52.5 R		625	199
Sand	0.125-0.250	250	95
Sand	0.250-0.500	375	142
Sand	0.500-1	375	142
Sand	1-2	250	95
Total sand		1251	474
Water		267	267
Aircontent (%)		6	60
Total		2144	1000
		by weight	*by volume*
sand-cement ratio		*2,00*	*2,39*
water-cement ratio		*0,43*	*1,35*

Table 10.3: Mortar mix used for the experiment.

Subsequently the two mixtures have been tested on compression and yield strength after 1, 3 and 7 days. Results can be found in figure 10.15. The tests show that both the compression strength as the bending capacity of the mortar is high. The maximum applied bending moment amounting 92 Nm [11].

Both grain distributions resulted in suitable mixes. As a fine grain distribution is easiest in application and as grain sizes in Sri Lanka are expected to be small as well, the 2 mm mixture is chosen for the production of the plates. The mix can be seen in tabel 10.3.

[11] $M = \frac{\sigma \cdot I_x}{y}$

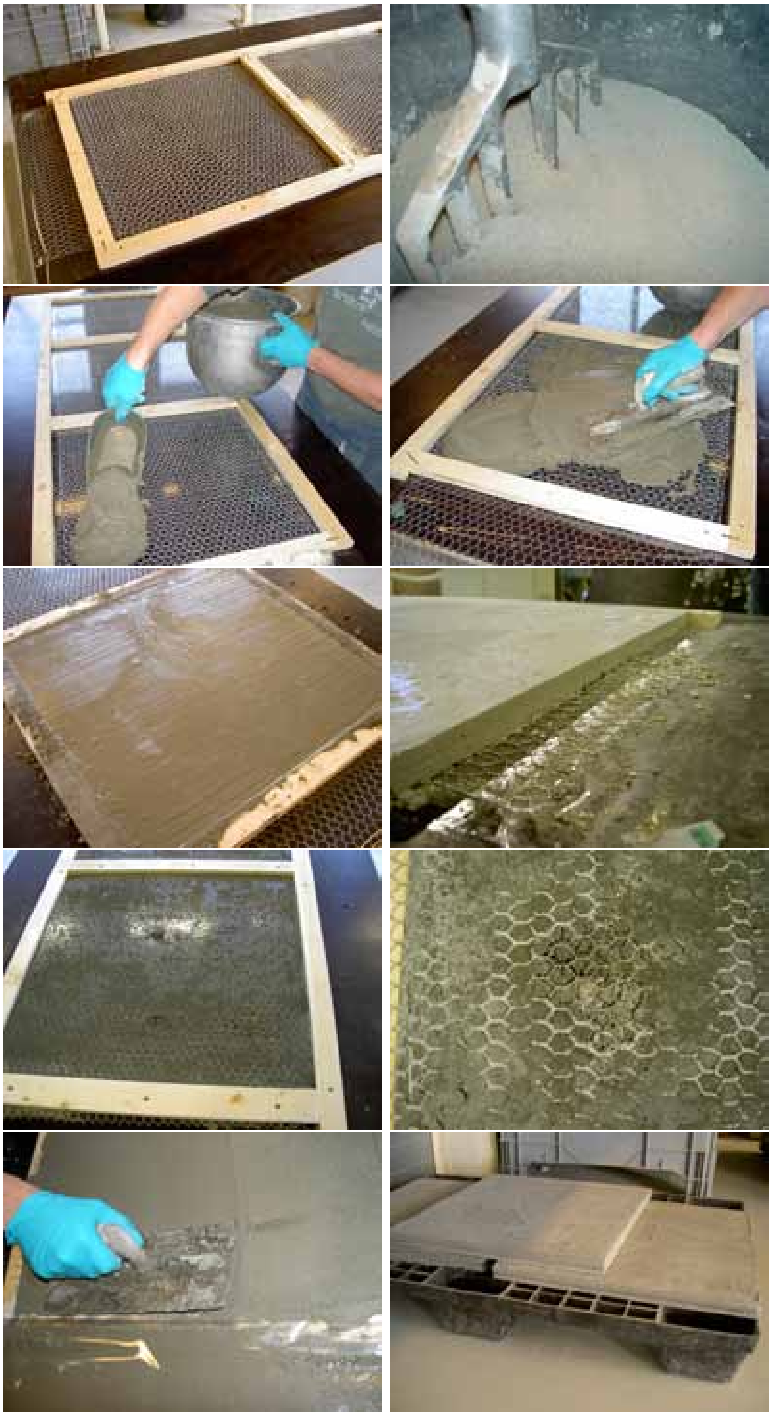


Figure 10.16: Process of making three plates for the experiment

In figure 10.16 the process of making the plates for the experiment can be seen. A mould with sides of 2 cm high is constructed. Three layers of chicken wire are laid on the bottom, which 'represents' the outside of the inflatable membrane. The chicken wire needs to be kept under tension to prevent it from twisting outside the form. This was achieved by clamping the third layer of chicken wire under the wooden laths of the mould. However the weight of the fresh mortar would probably keep the chicken wire in place as well. Yet it will be practical to span the chicken wire over the dome's surface. This has later been taken account when making the construction manual (see Appendix G). Subsequently the mortar is mixed according to the proportions in tabel 10.3. The mortar is applied on the chicken wire using a trowel. The surface is smoothened and covered by plastic to prevent dehydration. When constructing a dome the thickness of the applied mortar layer should either be checked by concrete spacers (attached on the outside of the wire mesh) or by randomly checking the thickness by protuding the layer with a marked length of i.e. a straw.

The next day the formwork is taken apart and the hardened plate of 2 cm thick is turned upside down. The mortar adheres well to the chicken wire. The smooth bottom of the mould has resulted in a very smooth surface. This might prevent the next layer of mortar from adhering well. The wooden laths of the formwork are attached to the bottom of the mould up to a height of 3 cm. The plate is wetted and a layer of 1 cm of mortar is applied. The layer is smoothed and covered by plastic to prevent dehydration. The day after the wooden laths and the plastic are removed and the three plates are stored until they are tested. No visible cracks developed in the plate. In spite of the high proportion of cement temperature stresses are apparently low because of the small thickness of the plate.

Figure 10.17: Testing a plate on concentrated load after two weeks of curing

For the test the plate, measuring 550 × 550 mm, is supported on 4 sides by a steel frame with a width of 40 mm. Thus the span of the plate is 470 mm in both directions. The middle of the plate is determined. Subsequently the plate is loaded by a pneumatic jack. The loading surface is circular and 110 cm^2. Two plates are tested exactly 2 weeks after the second layer has been applied. The pictures in figure 10.17 to 10.21 show the ductile failure of the ferrocement.

Figure 10.18: First cracks develop

Figure 10.19: Growth of cracks

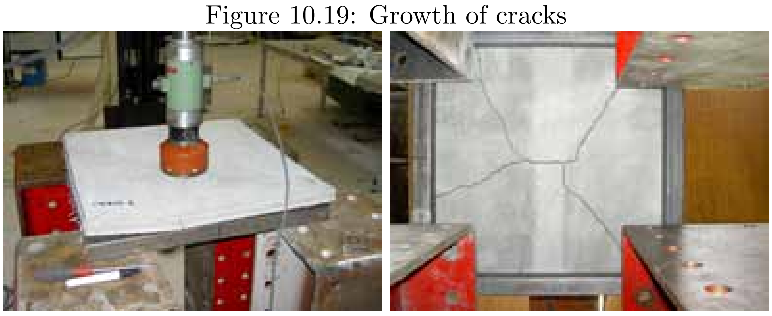


Figure 10.20: Crack develop on upper side of plate

Figure 10.21: Further loading (plate 2)

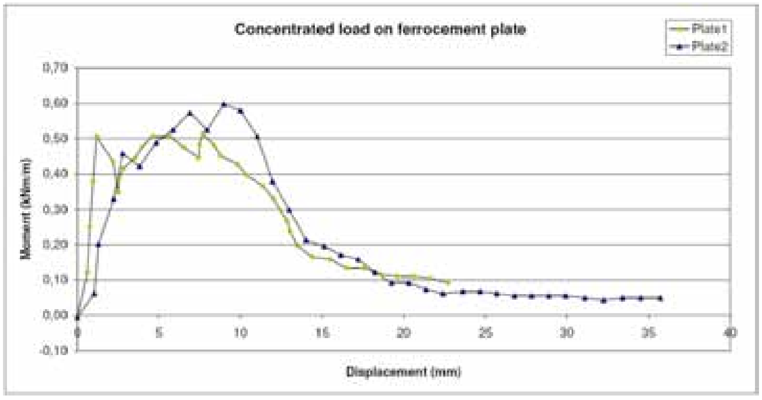


Figure 10.22: The graphs show the results of the two plates that were tested. Cracks develop at the underside of the plate when the load amounts 0,51 kNm/m for the first plate and 0.46 kNm/m for the second. The first plate is loaded until cracks have developed at the upper side of the plate as well. The second plate is loaded until the mesh is torn in the cracks close to the load surface. The displacement is the displacement of the jack, which is not exactly the same of the displacement of the plate itself.

The graphs in figure 10.22 show the results of the test. The force that was imposed on the plate has been translated to a moment with the proportion $M = \frac{1}{8}F$ for a concentrated load on a square plate supported on four sides [12]. The ferrocement displaces 2 to 4 mm until the first crack develops. The crack leads to a sudden increase of deformations and thus a sudden decrease in force, which can be seen in both graphs as a 'first dip' in both graphs. Further increase of the load increases deformations. The second dip is assumed to be caused by a first crack in the (underside of the) upper part of the plate, the part that has been concreted first. The final dip occurs when cracks appear on the upper side of the plate. Still deformations increase. Even after the concrete is cracked on both sides, it is able to bear a load of at least 50 kilograms.

[12] This proportion is based on calculations of plastic moments by Pierre Hoogenboom, taking the pattern of the cracks into account

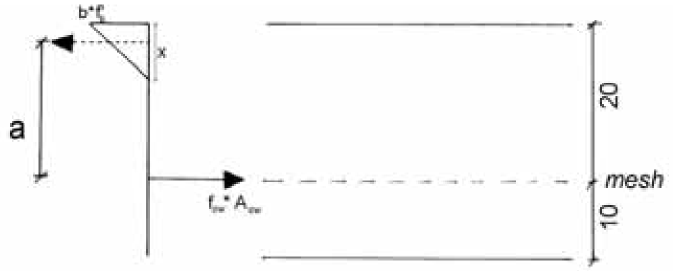


Figure 10.23: Distance 'a' between the resultant pressure and the resultant tension force acting on the cross-section of the plate. $a = 20 - \beta x$ and $x = \frac{f_{cw} A_{cw}}{b \alpha f'_b} = 2,8\ mm$, the height of the pressure zone. Values parameters: $\beta = 0,39$, $f_{cw} = 270\ N/mm^2$, $A_{cw} = 110,8\ mm^2$, $b = 500\ mm$, $\alpha = 0,75$ and $f'_b = 0,6 \cdot 47\ N/mm^2$ (pressure strength of 2 mm mixture according to experiment is applied, see figure 10.15). Resulting in a value of 18,9 mm for a.

The tension stress in the chicken wire (cw) at the point of the first crack is determined:

$$\sigma_{cw} = \frac{\frac{M}{a}}{A_{cw}}$$

M is the moment at which the first crack occurs, being 510 Nm/m thus 255 Nm for the first plate [13]
a is the distance between the resultant pressure and tension force in the cross-section, see[14] figure 10.23
A_{cw} is the area of the steel of the chicken wire, amounting 110,8 mm^2

As a result σ_{cw}, the tension stress in the chicken wire, amounts 122 N/mm^2 when the first crack in the concrete occurs. While 270 N/mm^2 is the maximum tension stress that is allowed in the mesh (the minimum value of the yield strength is 310 N/mm^2). *Thus there is enough tension capacity left in the chicken wire at the moment of the first crack in the concrete.* A moment of 649 Nm/plate or 1,3 kNm/m is needed to reach the yield strenght of the chicken wire. In case of this plate that moment corresponds with a concentrated load of 1039 kg on the plate.

Concludingly, there need not be fear of brittle failure of the matrix.

[13] The dimensions of the plate are simplified to 0,5 m by 0,5 m.

[14] Checking calculation of a: If a is approximated with $a = 0,9d$, d being 20 mm, a = 18 mm. If the pressuredistribution is regarded as a triangle, so α would be valued $\frac{1}{2}$ and β would be valued $\frac{1}{3}$, a = 18,6. These are both close to the calculated 'a' in figure 10.23.

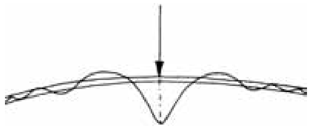


Figure 10.24: A concentrated load will cause a moment distribution (in each direction) as shown in this figure. The moment will dim with increasing distace from the concentrated load.

It is not possible to translate the quantitative capacity of the plate loaded by a concentrated load to the capacity of the domeshell loaded by a concentrated load. First of all the plate is not curved, secondly normal forces are hardly present in the plate and thirdly the plate is only vertically supported by the frame while each 'element' of the domeshell is rigidly supported by the surrounding 'elements'. As a result the moment caused by the concentrated load is much higher in a flat plate than in a double curved shell and thus the capacity of the double curved shell will be higher. In paragraph 10.3.1 a load of 70 kilogram distributed over a surface of 130 cm^2 was analysed on several surfaces on the dome. This resulted in a maximum moment of 2,7 Nm/m. A load of 70 kg on a flat plate will result in a moment of 88 Nm/m. The tested plates are both able to bear more than five times this load, the maximum moment amounting 460 Nm/m . As the double curved shell will even have a higher capacity, it can assumed to be safe against concentrated loads. Someone can stand on top of the dome.

What has not been taken into account is the stresses that occur on the upper side of the shell, caused by the fact that the shell is rigidly supported by it's surrounding 'shellelements' and not freely supported such as the plate. However these stresses will be smaller than the ones at the underside of the plate as the moment dims with the distance from the concentrated load, see figure 10.24.

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10.4 Comparing Costs to 'Traditional' Solid Houses

In this section the costs for raw materials of the traditional reinforced concrete design is compared to the costs for raw materials of a ferrocement design. The calculations in the first paragraph are based on Sri Lankan prices, the calculations in the second paragraph are based on prices in Kenia.

10.4.1 Based on Sri Lankan Prices and Experience

In tabel 10.4 the costs for a concrete dome are displayed. These are the costs of the raw materials cement, sand, stones, water and rebar for the 9 meter concrete dome that has been built in Sri Lanka (see Appendix E). Also, the costs of a 'minimum' version of this concrete shell are shown. This 'minimum' version applies smaller rebar (6 mm) and the shell thickness is reduced from 120 to 70 mm. The latter is based on the fact that the stresses in the shell are low and consequently 6 mm rebar should be able to replace the 10 mm rebar (see section 10.3.1). The price of 6 mm rebar is 0,78 dollar a piece (length 5,5 m) compared to 2,18 dollar a piece for 10 mm rebar. The length of the 10 mm rebar is assumed to be the same as the lenght of 6 mm rebar. The minimum shellthickness of 70 mm is based on a minimum cover of 30 to 40 mm on the outside and 20 mm on the inside [inf, 2002].

A distinction has been made between costs including plaster and costs excluding plaster in order to compare both with the costs for a ferrocement dome. The surface of cured ferrocement is already smooth, as no coarse aggregates are used for the mortar. The mortar could be regarded as a sort of plaster. Consequently additional plastering will hardly be necessary.

CONCRETE

Sri Lanka

Mortar	*costs/unit* $/m3	*concreting* m3	$	*plastering* m3	*total* m3	*costs* $
cement	382	1,7	667	0,8	2,5	982
sand	55	2,1	116	1,2	3,3	184
rock	275	1,8	485		1,8	485
water	7	4	29	4,5	8,5	62
Costs concrete		*excl plaster*	**1297**		*incl plaster*	**1713**
Rebar				*#/shell*	*$/piece*	*costs ($)*
Rebar 10 mm		*max rebar spacing 200 mm*		320	2,18	**698**
Total Costs Reinforced Concrete Dome Sri Lanka					*including plaster*	***2411***
					excluding plaster	***1995***

Minimum

Mortar	*shellthickness* m	*costs excl plaster* $	*costs incl plaster* $
Sri Lanka 9 m dome	0,12	1297	1713
Minimum thickness	0,07	**757**	**999**

For rebar minimum cover on outside 3-4 cm and inside 2 cm (based on [InfoMap, 2002])

Rebar		*#/shell*	*$/piece*	*costs ($)*
Rebar 6 mm	*max rebar spacing 200 mm*	320	0,78	**248**
Total Costs Reinforced Concrete 'Minimum' Dome			*including plaster*	***1248***
			excluding plaster	***1005***

Table 10.4: The first tabel shows the costs of raw materials for a 9 meter diameter reinforced concrete dome constructed in Sri Lanka, based on information from Tomas Viguurs, Inspector Eatham, Sri Lanka (see Appendix E). The second tabel shows the costs of raw materials for the same dome in Sri Lanka if the shell thickness is reduced to a minimum and smaller rebar are used.

FERROCEMENT

Costs per unit of mortar	$/m3	liter	$
cement	381,9	199	76
sand	54,9	474	26
rock	275,0	0	0
water	7,3	267	2
air (about 6%)		60	0
Total costs per m3 mortar			***104***

Mortar	*shellthickn* m	*D dome* m	*surface* m2	*volume* m3	*price/m3* $/m3	*costs* $
Mortar	0,03	9	127	3,8	104	**397**
Mesh		*layers mesh*	*extra mesh*	*m2 mesh*	*$/m2*	*costs ($)*
Hexagonal mesh		3	25%	477	1,28	**610**
Square woven mesh		1	25%	159	1,76	**280**
Total Costs Ferrocement Dome					*hexagonal mesh*	***1007***
					square woven mesh	***677***

Table 10.5: Costs of a 9 meter diameter ferrocement dome, based on information from Tomas Viguurs, Inspector Eatham, Sri Lanka (see Appendix E).

In tabel 10.5 the costs for a ferrocement dome are calculated. These are the costs of the raw materials cement, sand, water and mesh. Both for a design with hexagonal woven mesh (section 10.3.1) as for a design with square woven mesh (section 10.3.2) costs have been calculated based on prices in Inspector Eatham, Sri Lanka. 25% Extra mesh has been calculated to allow for overlapping pieces of mesh. If the extra mesh percentage is increased to 50% the total costs of (raw materials for a 9 meter diameter) ferrocement dome applying square woven mesh will amount 760 dollar.

In the diagram of figure 10.25 the total costs of the different options are compared. The raw material costs for a ferrocement dome with square woven mesh is clearly lowest in price. However a considerable improvement in costs is already possible by reducing material use to a minimum, the only question is whether this reduction in shell thickness is feasible using coarse aggregates.

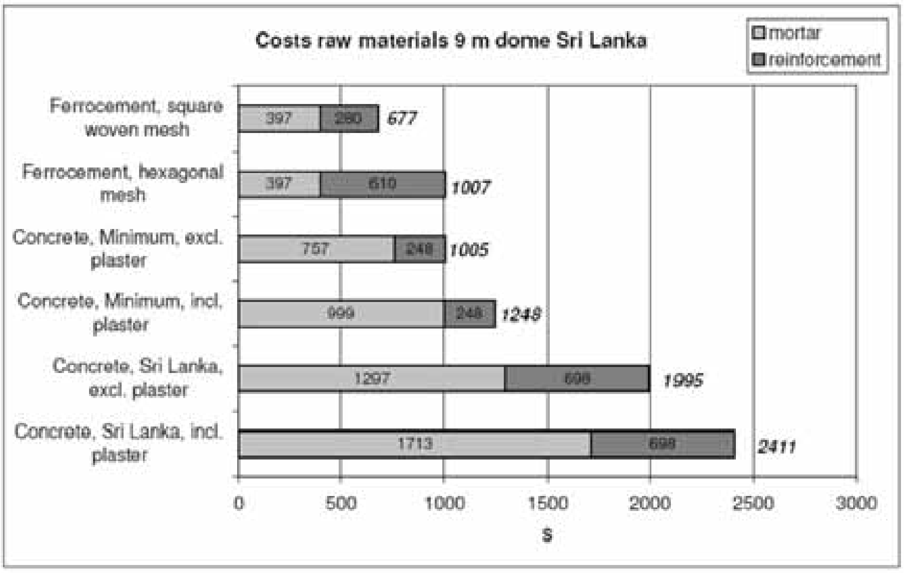

Figure 10.25: Graphs displaying the total costs of raw materials for a 9 meter dome in Sri Lanka for the different options of tabel 10.4 and 10.5. The bars are split into costs for reinforcement and costs for mortar.

The diagram also displays the share of reinforcement and the mortar to the total costs for each option. As the ferrocement shells are much thinner, costs of reinforcement are relatively higher than for the traditional building method applied in Sri Lanka. If hexagonal mesh is applied the costs for reinforcement are similar, while if square mesh is applied the costs for reinforcement decrease compared to the dome built in Sri Lanka [15].

Concludingly, a switch from 'traditional' (as built in Sri Lanka) to ferrocement SolidHouses will lead to considerable savings in costs of raw materials, more than could be achieved by reducing the shell thickness and the rebar size of the Shell. On top of this labour costs will also be reduced as plaster will not or hardly be necessary. Also, the mortar will harden much faster as the share of cement in the mixture is higher and the shell is thinner. Consequently the time that the formwork needs to stay inflated can be reduced.

[15] Even if 50% extra mesh is calculated, the costs for the mesh are below 400 $ while the costs for the 10 mm reabar for the dome in Sri Lanka amounted to 700$

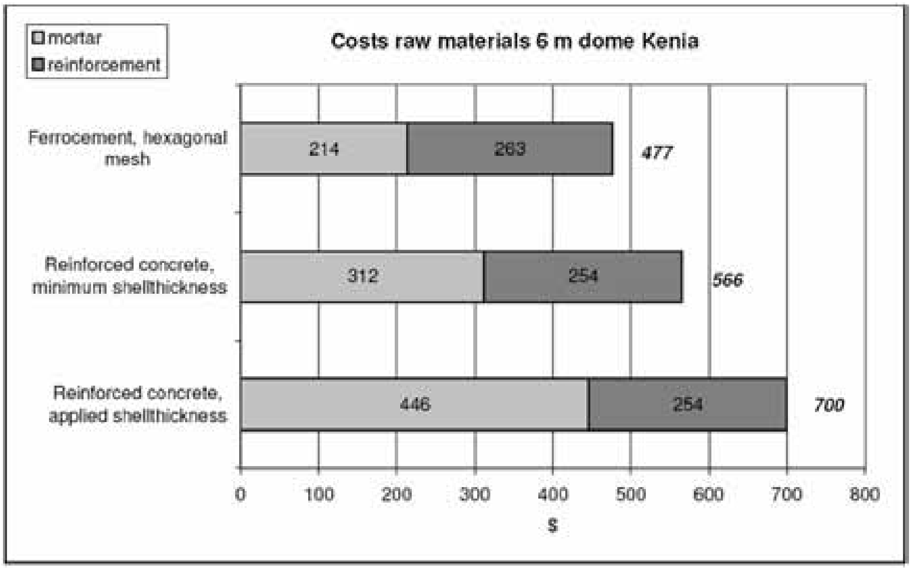

Figure 10.26: Graphs displaying the costs of rebar/mesh and mortar of a 6 meter dome for the different options of tabel 10.6

10.4.2 Based on African prices and Monolithic Dome institute

As data on prices in Kenia was best available at first, the first comparison was made with data from Kenia. Again this overview is restricted to raw material costs of cement, sand, stones, water, rebar or mesh. Mix proportions are based on recommendations of the Monolithic Dome Insitute [MDI, 2005]. Mix proportions of the mortar for the ferrocement matrix are based on the mix used for the experiment of section 10.3.3. Results for a 6 meter diameter dome can be found in tabel 10.6 and an overview is shown in the diagram of figure 10.26. Price of square woven mesh was not available. In Kenia the ferrocement version of the dome turns out to be most low-cost in raw materials as well.

Mix proportions and costs		Concrete Mix MDI		Mortar Mix Ferrocement	
Price per unit from Arjan Spit, SHF Kenia		*For 1 m3 concrete*		*For 1 m3 mortar*	
	$/m3	*liter*	*$*	*liter*	*$*
cement	485	80	39	199	96
sand	44	590	26	474	21
stones	53	118	6		
water	32	236	7	267	8
air (about 6%)		57	0	60	0
Total costs/m3			***79***		***126***

CONCRETE

Concrete MDI	*shellthickn.*	*D dome*	*surface*	*volume*	*price/m3*	*costs*
	m	*m*	*m2*	*m3*	*$/m3*	*$*
Mortar, minimum	0,07	6	57	4,0	79	**312**
Mortar, applied	0,1	"	"	5,7	"	446
For rebar minimum cover on outside 3-4 cm and inside 2 cm (based on [InfoMap, 2002])						
Rebar		*#/dome*	*share shell*	*#/shell*	*$/piece*	*costs ($)*
Ruai		95	50%	47,5	5,31	252
Kisumu		96	"	48	"	255
			estimation 'on safe side'		*D 10 mm*12 m, A.Spit, Kenia*	
Average						**254**
Total Costs Reinforced Concrete Dome					*minimum shellthickness*	***566***
					applied shellthickness	*700*

FERROCEMENT

Mortar	*shellthickn*	*D dome*	*surface*	*volume*	*price/m3*	*costs*
	m	*m*	*m2*	*m3*	*$/m3*	*$*
Mortar	0,03	6	57	1,7	126	**214**
Chicken wire		*layers mesh*	*extra mesh*	*m2 mesh*	*$/m2*	*costs ($)*
Hexagonal mesh		3,00	25%	212	1,24	**263**
					Mbara Town, Uganda	
Total Costs Ferrocement Dome						***477***

Table 10.6: Comparing costs of a 6 meter diameter reinforced concrete dome with a 6 meter diameter ferrocement dome, based on information from Kenia and from the Monolithic Dome Institute

10.5 Formwork

10.5.1 Deformations Formwork

The 9 meter dome of the Solid House Foundation consists of a hemisphere of 9 meter diameter and a cylindrical part with a diameter of 9 meter and a height of 1 meter. As can be seen in the resulting shell in figure 10.27, this part of the formwork deforms.

In the first place a pneumatic form that is not spherical will 'try' to deform under airpressure, making the vertical walls bulge outward. The load of fresh concrete on top of the dome will add to this 'swelling', as is shown in figure 10.28. Most important however is the contribution of the load of the fresh concrete on the vertical parts of the formwork. The formwork already bulges outward a bit because of the other airpressure inside the formwork. Consequently the resulting stress R from the airpressure and the load of the fresh concrete is directed outward and downward. Making the form bulge, especially if airpressures are below 3 kN/m^2. Deformations can be reduced by increasing the airpressure but also by applying less material. Deflections are expected to be much smaller if ferrocement is applied. However it is advised to use as little vertical parts for the form as possible.

If acceptable from an aesthetic point of view, a catenaric cross-section of the inflatable formwork could be applied (see figure 10.29). This removes the vertical part of the inflatable formwork, and thus reduces deformations. Also, the mortar can be more easily applied on a slope than on a vertical surface. Thirdly the resulting shell will be subject to considerably reduced stresses compared to a spherical dome, which was analysed in section 8.4. Concludingly a catenaric cross-section would be a very suitable alternative.

Figure 10.27: While lower meter of the inflatable formwork was vertical, the resulting shell bulges outward [Viguurs].

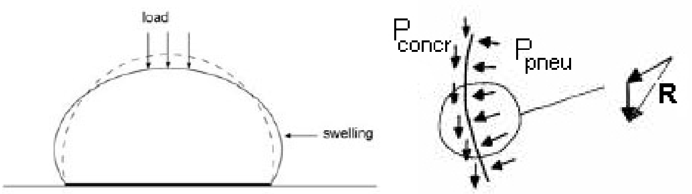

Figure 10.28: Due to the weight of the concrete the inflated formwork deforms. The vertical parts of the formwork deform most as the resulting stress R is directed outward and downward [Hennik, 2005].

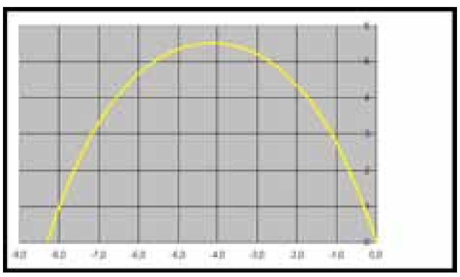

Figure 10.29: Catenary plotted for a height of 5,5 meter and a base diameter of 8,2 meter, which would be suitable for a house with a first floor.

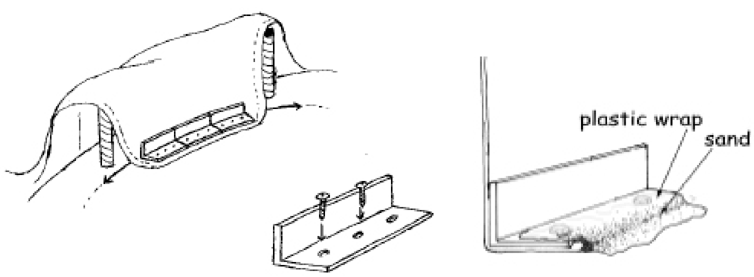

Figure 10.30: Connecting the formwork airtight to the foundation [MDI, 2005]

10.5.2 Anchorage Formwork

The formwork in use by the Solid House Foundation has no bottom, in other words, it is not a closed formwork. Consequently the formwork needs to be connected airtight to the foundation. In figure 10.30 is shown how this is achieved in Sri Lanka. The connection will never be completely airtight, thus continuous inflow of air is necessary. Another disadvantage is that the required anchors are quite expensive and not so well available in Sri Lanka. In practise they turn out not to be reusable.

The anchors need to anchor a considerable force. If the formwork measures 9 m in diameter and the applied pressure is 1,5 N/mm, the total uplift per meter perimeter amounts to $\frac{1}{2} \cdot 4,5 \cdot 1,5 = 3,4\ kN/m = 334\ kg/m$. Resulting in a total uplifting force of 9740 kg, which is the reason for the very heavy foundation of the dome. From section 10.2 follows that a reinforced ringbeam with a cros-section of 200×200 should be more than sufficient as a foundation for the resulting domeshell. However a ringbeam of 200×200 only weighs 96 kg/m. The only option apart from changing the formwork (see chapter 11 is to temporarily load the ringbeam. Per meter a weight of 238 kg needs to be added. This would mean 5 sacks of cement per meter or 30 Dutch pavement tiles per meter. The 30 tiles could be divided over three piles of 10 tiles, reaching a height of about 40 centimeters. In figure 10.31 this is illustrated by a sketch. The workman can hardly reach the first meter of formwork. A possibility would be to cast against wooden formboards that are supported by the tiles. Another option is to lay the tiles on the inside of the ringbeam.

All in all temporary loading is a possible, but rather unpractical solution to the problem of the heavy foundation.

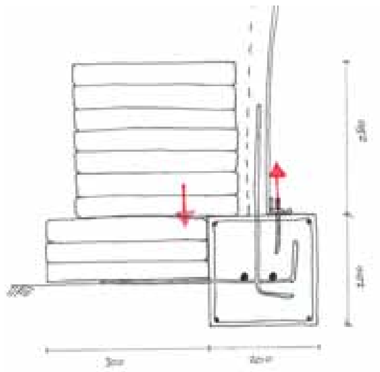

Figure 10.31: A sketch of a temporary loaded ringbeam. The tiles need to act as one mass with the ringbeam. Extra reinforcement in the ringbeam will be needed. The center of mass does not lie in the prolongation of the anchoring force, resulting in a moment. Consequently the ringbeam is loaded on torsion.

10.6 Conclusions and Recommendations

A ferrocement dome is a good alternative compared to a reinforced concrete dome. Costs of raw materials for the domeshell are only 30% of the costs of the domes that have so far been built by the Solid House Foundation. Even if the rebar of the reinforced concrete dome would be reduced to a smaller size and the wallthickness would be reduced to a minimum, costs of a ferrocement dome applying square woven mesh would amount 60% of this 'minimum' reinforced dome. Besides, this 'minimum' reinforced dome might even not be feasible as it is very difficult to apply a thin layer with mortar which contains coarse aggregates. The mortar mix of the ferrocement results in a smooth plaster, as it contains only sand, cement and water. The plaster is easy to apply in thin layers, sticks to the mesh and dries quickly to form a smooth surface which needs no additional plastering. This not only saves materials, but also a lot of labour costs. Another advantage is the decrease of deformations of the inflatable formwork, as the weight of the ferrocement shell is much lower than the reinforced concrete shell.

The design of the ferrocement domeshell consists of a three centimeter thick shell. The reinforcement is situated two centimeter from the outside of the shell and either consists of three layers of chicken wire or one layer of square woven mesh. The square woven mesh is preferred as it results in a lower price (in Sri Lanka) and it is less work to apply. In spite of the low level of reinforcement, the failure behaviour of ferrocement is ductile. This followed from the experiment that was carried out by the author at the Stevinlab in Delft.

A dome with a catenaric cross section would be preferred above a dome with vertical sections because of deformation of the inflatable as well as stress levels in the resulting shell. A recommendation is an analyses of the possibilities to create supports for a floor in the domeshell.

However the building concept incorparates still a number of important *disadvantages*:

- The inflatable form needs to be anchored by the foundation, which results in a very heavy and reinforced foundation (around 10 000 kg is needed for a 9 meter diameter formwork). For the 9 meter dome in Sri Lanka 35% of the total amount of used concrete and 28% of the used rebar were applied in the foundation. These raw materials could be reduced with 85% if only a ringbeam of 200×200 is applied (excluding material for a floor). As the inflatable formwork is not closed, but open at the bottom the inflatable form itself cannot be temporarily loaded. A temporary load could only be applied to the foundation ring. This is a possible but rather unpractical solution, see section 10.5.2.
- The inflatable form needs to be connected airtight to the foundation. This is difficult to achieve, leaks reduce the air-pressure capacity. The removable bolts needed are not well available and turn out not to be as reusable as they should be.
- The inflatable formwork can not be closed and air leaks through the seams. Consequently the formwork is sensible to powercuts. A partly closed formwork would therefore already be an improvement. Some airflow will always be needed to absorb pressure difference due to changing climatic circumstances. But it's dependency on power makes the method expensive. Electrical power is often not available at the site and the costs to keep a generator running for several consequent days is considerable.

These issues can only be solved by radically changing the formwork. In the next chapter, chapter 11, a study is made on alternative formworks that are focussed on solving the issues mentioned above.

Chapter 11

A New Inflatable Formwork

Currently, the loads to anchor the inflatable formwork are normative for the dimensions of the foundation (see chapter 9). In some cases up to 50% of the material was used for the foundation (Bolivia), in case of application of ferrocement this percentage will be even around 70%. An attempt was made to reduce these anchoring loads by changing the design of the formwork, so that less material will 'end up in the ground'. Besides the possibility to make the inflatable form airtight were investigated. An overview of analysed alternatives, as well as a resulting proposal for a design can be found in this chapter. All mentioned alternatives are closed or partly closed formworks, contrary to the formwork that is currently applied. Before starting new designs for the formwork a short study was made on membranes in general. In Appendix F a number of aspects are discussed.

Relevance of airtightness

The relevance of the latter was emphasized when at a final stage of this research a log of a dome built in Kenia became available:
There had been a lot of trouble anchoring the baloon in the first place. Anchors are not widely available and unfortunately they can often not be re-used either (which is also the case in Sri Lanka). After the anchoring was solved the formwork turned out to be leaking. Wet sand was used inside the airform to make it airtight. As electricity is relatively expensive in Kenia, the builders decided to keep the formwork as often deflated as possible. A cage of rebar was constructed, while the formwork was only inflated now and then to check the dimensions. During casting of the concrete, the form started to leak again, but this time it was not possible to go insight and improve the air-tightness by wetting the sand. Another problem was that the airform did not reach the top of the rebar cage, and it was impossible to cast (see figure 11.1). The cage was not made tight around the form, the leaking and the sagging of the form due to weight of the fresh concrete only increased the problem. A lot of effort had to be put in additional wooden formwok to fill the gap.

An airtight membrane could be inflated by hand pump and would not need a continuous flow of air to keep it up to pressure. Thereby the building concept

Figure 11.1: The airform did not reach the rebar cage in Ruia, Kenia [Spit]

would not be dependent on electricity, although you will need a considerable amount of time to inflate a large volume. Also the connection to the foundation would not need to be airtight itself, saving a lot of trouble. However atmospheric pressures vary during the day 0.05 - 0.1 kN/m^2 in mild climates, and due to extreme weather changes it can change 1-2 kN/m^2 within a few hours. An air pressure device might be needed to control the changes due to temperature and atmospheric changes to maintain one volume during curing.

Alternatives

In section 9.4 a number of options has already been generated. To a selection of these more ideas have been added. The alternatives are grouped into four sections:

- *Stiffened Closed Formwork*
- *Inflated Spheres Supporting Formwork*
- *Pneus*
- *Pneus for Roof Only*

In each section the alternative(s) is (are) presented and an evaluation is made.

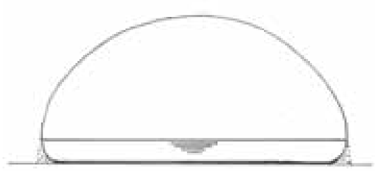

Figure 11.2: Closed formwork loaded with water, to reduce deformations at least 22 000 liter water is needed. Deformations at the sides are compensated with i.e. adobe before the ferrocement or concrete is applied.

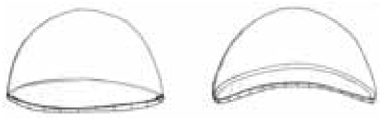

Figure 11.3: A pressure ring will buckle if it is not stiff enough.

11.1 Stiffened closed formwork

As loads are very high, interior support as mentioned in section 9.4.3 is not feasible; to load the lower membrane an enormous amount of material would be needed. The material is especially needed near to the perimeter, where loads reach 3,4 kN/m if a pressure of 1,5 kN/m^2 is applied on the formwork of 9 meter diameter. A standard dutch pavement tile of 30 by 30 cm weighs 80 N. That means a pile of at least 43 tiles every meter would be necessary...which is very unpractical. Neither is a stiffened (pressure-)ring an option, it would buckle (see figure 11.3) unless it is very heavy which is unpractical for system that needs to be demountable.

Loading the lower membrane with water would ask for at least 10 000 liter of water to compensate the 9740 kg uplift for a 9 meter diameter formwork under 1,5 kN/m^2 air pressure. This corresponds with 16 centimeters of water in the formwork. However at the sides the loads on the membrane are concentrated, the lower membrane will be lifted as water is not a stiff mass such as a reinforced concrete floor. Consequently a waterheight of at least 35 cm would be needed to reduce deformations to an acceptable level. These deformations could be compensated for with i.e. sand or adobe formwork around the perimeter (see figure 11.2). In total at least 22 000 liter would be needed. The water could be reused for several domes. However the water will evaporate, causing the dome to deform in the process, additional water will be needed. The dependency on a water pump and the scarcity of water (in Sri Lanka) are disadvantages.

This leaves the exterior (temporary) supports as invented by Heifetz (section 9.4.4) to stiffen the closed formwork. Of his designs, the systems 'arches' and 'trusses' in figure 11.4 are most practical in use, for explanation on these designs see section 9.4.4. The system should either be made in the Netherlands and be as lightweight as possible, or it should be produced locally. In that case the design should be simplified as much as possible. However even if simplified the materials and techniques needed for the system are quite complicated, especially

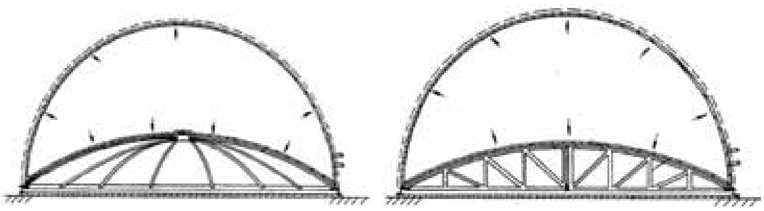

Figure 11.4: Left the 'arches' design and right the 'trusses' design of Heifetz [Heifetz, 1971]

as it needs to be demountable.

To have an idea of the amount of materials needed and the costs involved, the design 'arches' is further analysed for a dome with a 9 meter diameter base and for the case that the lower membrane is supported by 6 elements and reaches a maximum height of x = 1 meter.

In figure 11.5 the loads on the structure are shown and simplified into separate elements. As x =1 meter, a= 10 meter (see 9.17). With $p = 1,5\ kN/m^2$,

$R = \frac{1}{2}pa \cdot \frac{1}{6}\pi 2r = 35\ kN$
$q_R = \frac{1}{2}pr = 3,4\ kN/m$
$V_R = \frac{1}{2}pr \cdot \frac{1}{6}\pi 2r = 16\ kN$
$H_R = \frac{1}{2}p(a - \frac{r^2}{a}) \cdot \frac{1}{6}\pi 2r = 28\ kN$

$T = q_r * 2r * \frac{1}{2} = 27kN$
To absorb this load in the tension ring there is not much steel needed, about 115 N/mm^2. The load is not normative.

The vertical load case on the tension ring (A), see middle figure in 11.5, is simplified as a stiff supported. A beam on several supports would be more appropriate, assuming the ring's parts are well connected, but the stiff support produces a slightly larger maximum moment. As it is more practical and on the safe side, the moment is calculated based on a stiff support:
$M = 1/12 q_R(\frac{1}{6}\pi 2r)^2 = 6,3\ kNm$

A rectangular section with a height of 100 mm, a width of 50 mm and a wall thickness of 4 mm and $I_{zz} = 141,79 \cdot 10^4\ mm^4$ could be used for the ring.
Concludingly the deflection has a value between:
(Stiff support) $w = \frac{1}{384}\frac{q_R l^4}{EI} = 14\ mm$
(Hinged support) $w = \frac{5}{384}\frac{q_R l^4}{EI} = 73\ mm$

The maximum moment in one of the arches (B) will be around 24 kNm. An I-section with height 100 mm, width 40 mm and wall thickness of 5 and 10 mm will have a deflection of 0,6 meter if modeled as a straight beam on hinged supports, loaded by a varying distributed load. In reality the deflection will be smaller because it is not a straight beam, but an arch.

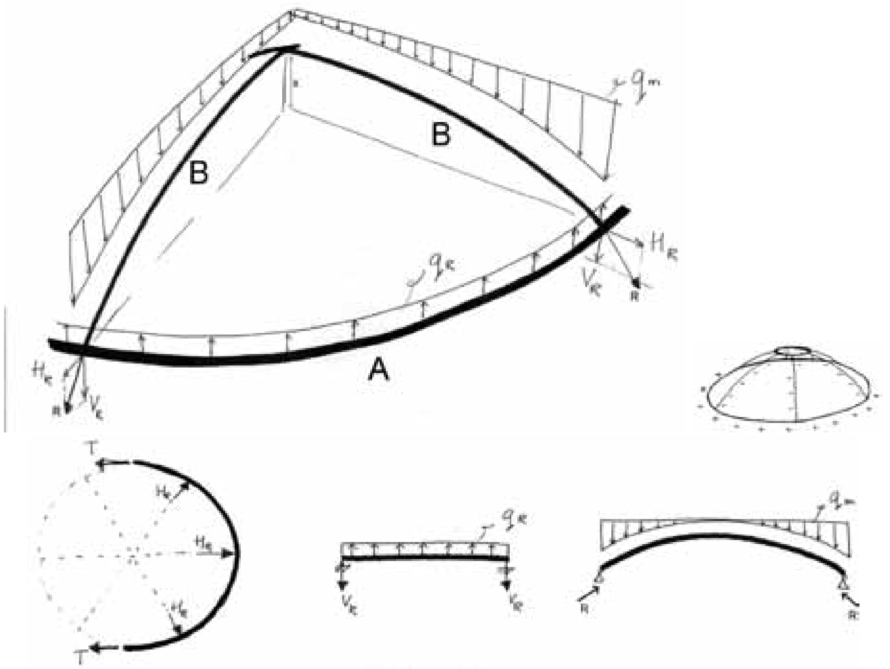

Figure 11.5: Loads on the 'arches' system of Heifetz in case the lower membrane is supported by 6 elements, the model is simplified into separate elements and their loads.

This rough estimation of needed dimensions for the different parts of the structure is calculated to kilograms of steel. This results in a weight of 518 kilograms of steel, excluding connections. The currently used membranes for a 6 meter, a 9 meter or a 12 meter diameter dome weigh respectively 60-90, 120-150 and 210-240 kilograms. Concludingly, initial costs would be considerately increased by the construction and the transport of the supporting system. [1]

To decrease dependency on electricity a rubber over sized inflatable could be installed between the upper and the lower membrane. The system could then be inflated by handpump and air control is only necessary for changes in weather conditions.

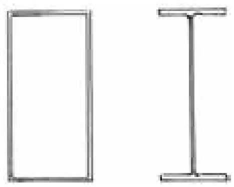

Figure 11.6: Steel section

11.1.1 Exterior support of closed formwork, Conclusions

The exterior supporting systems of Heifetz are re-usable and effective in their purpose to reduce the weight of the foundation. However the system is probably too complicated to be produced locally. Apart from the production costs the system increases weight of the 'construction package' of a 9 meter balloon by more than 4,5 times. So not only initial investment is higher, but transportation costs increase considerably as well. The system could reduce the current foundation to a ringbeam of 200×200 mm^2, which means saving 85% in materials in the foundation. For concrete material only (excluding rebar costs) this would mean a cost reduction of \$ 700 per dome [2]. These domes lack a concrete floor. The floor could be made of tiles or rammed earth, or a thin layer of unreinforced concrete if cracks are not a problem. In case of a large project the costs of transport and production of the system should be compared with the number of domes to be build times the cost reduction in materials per dome.

[1] Aluminium has a 35% lower unit mass, but it's stiffness is only a third of the stiffness of steel and the tensile strength only 20% of the tensile strenght of steel. As the stiffness of the elements is important in this design, the replacement of steel by aluminium is probably not an improvement.

[2] Calculated with price-data from Kenia, see section 10.4.2

Figure 11.7: Large reusable polyvinyl balloon (www.southernballoonworks.com)

11.2 Inflated Spheres Supporting Formwork

Spheres do not deform under air pressure. A spherical membrane can be equipped with an airtight 'inner sphere', for example a reusable polyvinylballoon (see figure 11.7). This allows the spherical membrane to be subject to higher pressures and the seams of the membrane need not be airtight. In this section membranes are discussed that are supported by an inflated spherical form. In section 9.4.2 several ideas put forward that directly use inflated spheres as formwork in combination with a brick wall. As the Solid House Foundation prefers to work with one construction method, the possibility of supporting a membrane with a spherical formwork is analysed first in this section. In section 11.4 the use of complete spheres or similar forms for construction of part of the dome are further discussed.

11.2.1 Inflated Sphere with Tensioned Skirt

If a 'skirt' is attached to the membrane, like in figure 11.8, the skirt can be put under tensile stress in order to support the fresh concrete. The resulting shape approximates the catenary, as shown in figure 11.9. However the lower part of the form, the skirt that is kept in shape by tension, is only curved in one direction. This decreases the stiffness of the resulting shell in this part, but it is very practical for the production of moulds for windows and doors.

To know more about the feasibility of this design, the tensile stress that is needed to keep the 'skirt' in shape is calculated. The calculation is based on the equation $N = qR$, which is explained in figure 11.10. This is actually a formula for cables; for elements stretched in only one direction. The 'skirt' will also be tensioned in circumferential direction and therefore deflections will be slightly smaller than is calculated here. Not taken into account is the deformation of the sphere as a result of the weight of the fresh concrete. The sphere will flatten a bit at the top and bulge at the sides. This effect could decrease the tension of the 'skirt' and therefore increase deformations (see figure 11.12).

$n = q_p R$, see figures 11.10 and 11.11
n is the tension force needed per meter circumference at the base to prevent deflection w, thus n is the anchoring force needed

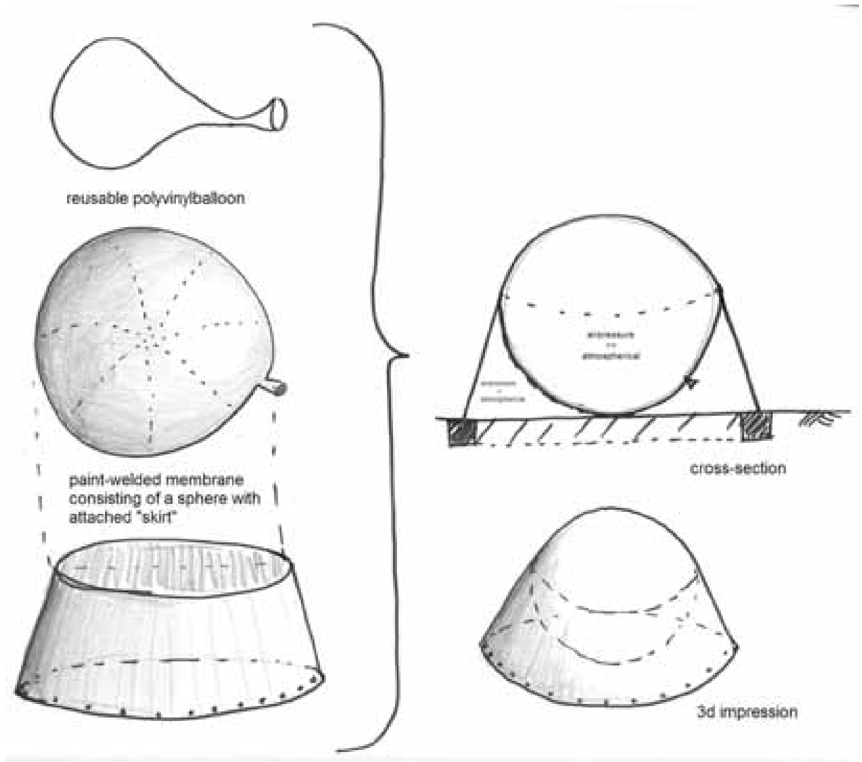

Figure 11.8: Inflated sphere with tensioned skirt

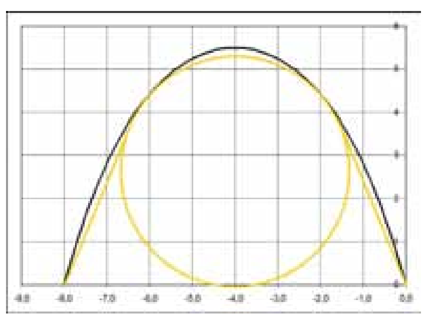

Figure 11.9: Compare the cross-section of a sphere with tensioned 'skirt' with a catenary

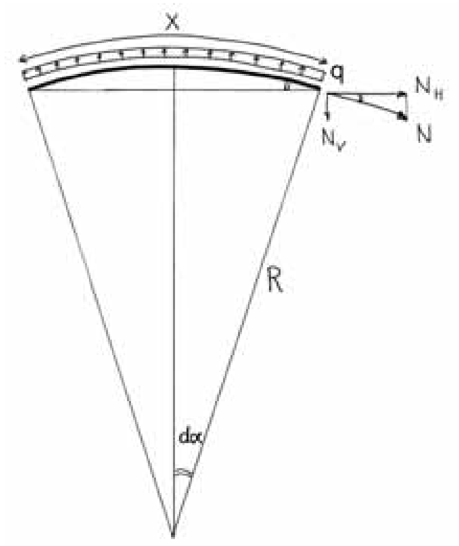

Figure 11.10: Based on vertical equilibrium: $2 \cdot N_V = q \cdot x$, $N_V = Nsind\alpha = N \cdot d\alpha$, $x = 2 \cdot d\alpha R$, $2 \cdot N \cdot d\alpha = 2 \cdot d\alpha \cdot R$, $N = q \cdot R$

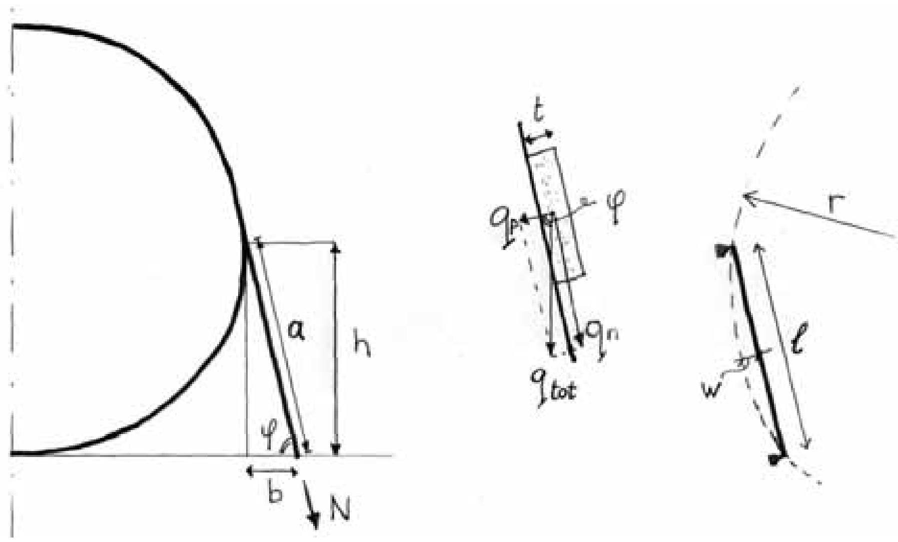

Figure 11.11: $q_p = \frac{b}{a} \cdot t \cdot 2426\ kg/m^3 \cdot 9,8\ m/s^2\ R = \frac{1}{8}\frac{l^2}{w} + \frac{1}{2}w$

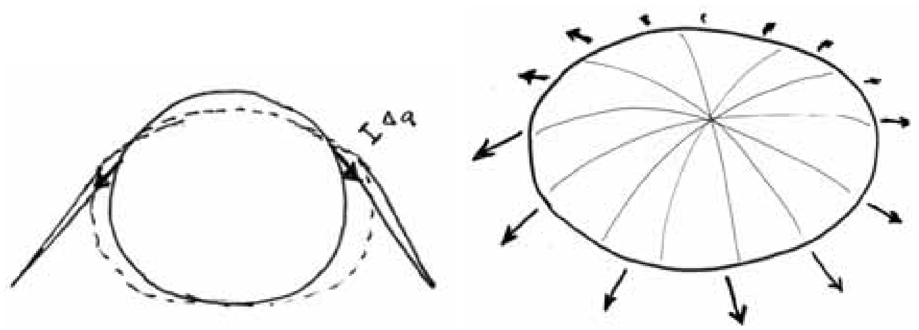

Figure 11.12: The load component q_n (kN/m^2) of figure 11.11 is causing a uniform distributed load q_{ntot} (kN/m circumference)on the spherical membrane. The load results in additional tension in the upper part of the spherical membrane. Depending on the pressure level of the sphere, the sphere will deform. The deformation may decrease tension (and thus increase deflections) in the 'skirt'.

q_p depends on the thickness of the ferrocement layer and the steepness of the 'skirt' as is shown in figure 11.11, $q_p = \frac{b}{a} \cdot t \cdot 2426\ kg/m^3 \cdot 9,8\ m/s^2$, q_p is actually a pressure as it is a uniform distributed load on a strip of 1 meter wide
R depends on the deflection w as is shown in the same figure, $R = \frac{1}{8}\frac{l^2}{w} + \frac{1}{2}w$

For a sphere of 6 meter diameter, and the following dimensions of the skirt:
a = l = 3,5 meter
b = 0,5 meter
t = 0,03 meter
The maximum w allowed is 30 mm:

$q_p = 0,1\ kN/m^2$
$R = 51\ m$
$n = 5,1\ kN/m$

In section 9.2 was calculated what force needs to be absorbed by the foundation to anchor the inflated formwork. For a dome with a diameter D = 6 meter this amounted:
$n = \frac{1}{2} \cdot p \cdot \frac{1}{2} \cdot D = 2,25\ kN/m$

Concludingly, an even heavier foundation is needed to keep a 'skirt' with these dimensions in shape than for the current formwork. To arrive at a similar force n of 2,25 kN/m with t=0,03 m, l=3,5 m and w_{max}=30 mm, b should be reduced to 0,21 meter.

On top of that the load q_n has not been taken into account yet. As long as the mortar is fresh it will not support itself but stick to the chicken wire and the membrane. In a standard single membrane, deformations will occur as described in section 10.5. In this case the force q_n pulls down and causes a load q_{ntot} on the spherical membrane (see figure 11.12). The upper part of the spherical membrane receives an additional tension load which results in deformations.

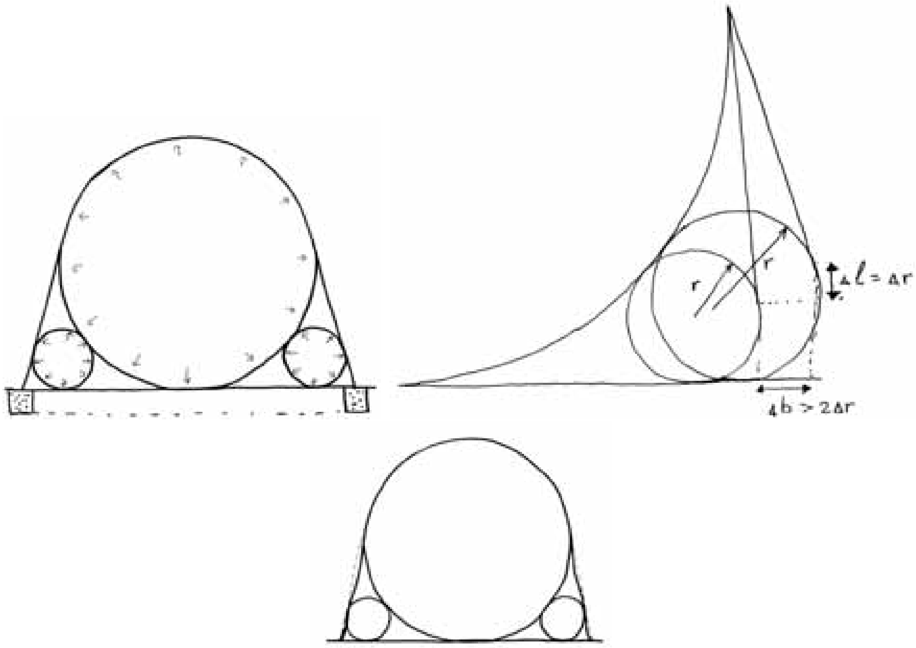

Figure 11.13: A 'donut' inflatable to reduce the length l (=a). The second sketch shows how an increasing diameter of the donut has more influence on b than l.

The deformations may even result in a relaxation of the tension in the 'skirt' (see Δa, figure 11.12), which would cause a larger deflection of the skirt.

Unless some adaptations are made to this alternative it is useless, as the amount of foundation required is similar and the deflections are higher than the current building concept.

A first adaptation that was analysed, is the addition of a donut-shaped inflatable membrane, see figure 11.13. This 'donut' can support the 'skirt', reducing the length a and l. With increasing diameter of the 'donut' the length l is further reduced. However b increases too and faster than l, so it does not improve the alternative.

If b = 0, deformations are theoretically 0, but hanging the reinforcement and applying the mortar on a vertical wall is very unpractical. The mortar will have the tendency to slide down and also the weight of the fresh mortar against the walls will cause a considerable load on the spherical membrane.

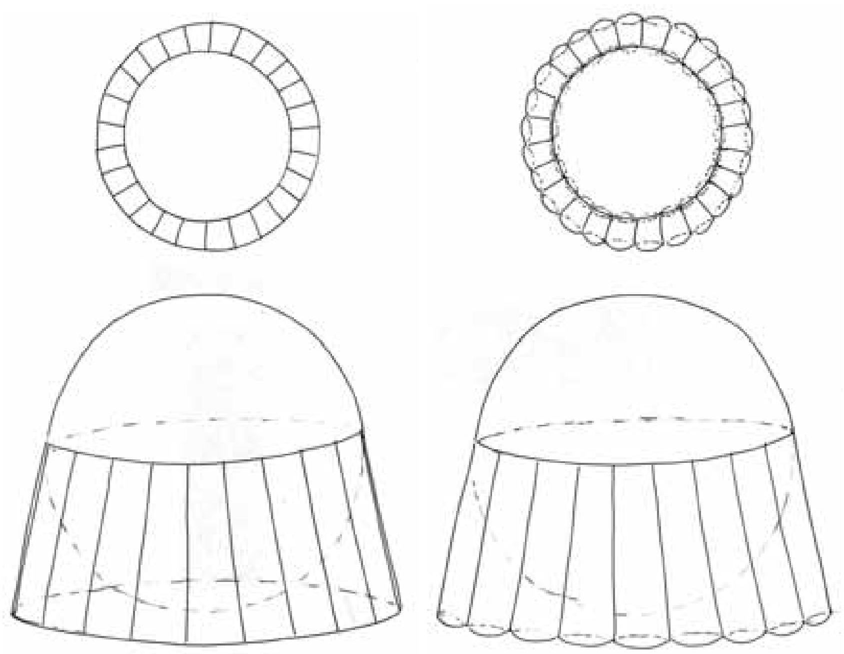

Figure 11.14: The 'skirt' stiffened by laths or by inflated tunnels

11.2.2 Inflated Sphere with Stiffened Skirt

An attempt is made to stiffen the 'skirt' with demountable wooden laths (figure 11.14). Laths of 6 × 6 cm would have a maximum deflection of 2,1 cm if the dimensions of the skirt are as in section 11.2.1 and the laths are maximum 1 meter apart. The membrane would still have to span a meter and would need a tension of at least 0,4 kN/m in circumferential direction if deflections should be smaller than 3 cm.

The laths complicate the design of the inflatable, there need to be holes in the 'skirt' or strings attached to be able to temporary fasten the laths. Also laths of the right size have to be found on location. The membrane of the skirt is no longer curved in one direction, but flat. This considerably decreases the stiffness of the shell. Deflections even cause curvature that is disadvantageous to the shell's stiffness.

Another option is to stiffen the skirt with air inflated tunnels, see figure 11.14. The skirt now has a ribbed outer membrane. The ribs contribute to the stiffness of the resulting shell. If desired the outer surface of the shell could be made smooth, while the inside keeps a ribbed surface.

Frank Braam [Braam, 2000] tested inflated tubes on deflections when loaded by a uniform distributed load, see figure 11.28. The inflated tube had a span of 3 meter, a diameter of 0,3 meter and was inflated to an air pressure of 250 mbar. The dimensions of the skirt in section 11.2.1 lead to a load perpendicular to the membrane's surface of $q_p = 0,1\ kN/m^2$, the span is 3,5 meter (see

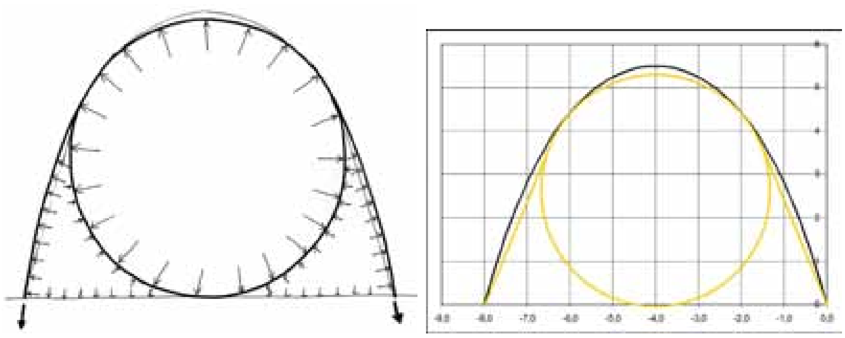

Figure 11.15: The 'skirt' stiffened by airpressure and the similarity in shape with a catenary

figure 11.11). Frank's inflated tube has a deflection of 12 mm when loaded by $Q = 3meter \times 0,3meter \times 0,1kN/m^2 = 110\ N$. As the length of the span of the skirt is about 1,2 times larger than this tube, the deflection will be $1,2^4 = 1,9$ times larger. Because of second order effects deflections will probably be even larger. Also, deflections caused by q_n have not yet been taken into account, see page 166.

Deflections are acceptable of this alternative, however the design of the membrane is very complicated. If an air pressure of 250 mbar is to be allowed in the tubes, a more airtight and stronger membrane and seams are necessary or an inner tube has to be used to guarantee airtightness. The skirt does not need to be heavily anchored as the one in the latter section, which is a big advantage. A disadvantage is the fact that the membrane is less stiff than a single pressurized membrane, so that it is more difficult to apply the concrete.

11.2.3 Inflated Sphere with Skirt under Pressure

Another way to stiffen the skirt is to use air pressure. A disadvantage is that the connection of the membrane to the foundation still needs to be airtight, like the currently applied membrane. However thanks to the sphere only a part of the membrane needs to be supported by air pressure. If a lower level of air pressure is needed, less foundation to stabilize the anchoring forces is needed as well.
Consider the graph in figure 11.15. For Kenia the SHF would prefer a dome with a height around 5,5 meter and a base diameter of about 8 meters. In the graph of figure 11.15 a catenary is plotted in these dimensions (in black). If a skirt is attached to an inflated sphere for these dimensions it will look like the yellow line. An advantage of a singly curved base ('skirt') is that it makes the moulds for windows and doors a lot easier. However a double curved surface is stiffer. Air pressure enables the production of double curved surfaces. So if the 'skirt' is supported by air pressure it could curve according to the catenary in figure 11.15. As proven in section 5.1 the flattened top of the cross-section compared to a catenary is actually advantageous.

The slope of a double curved skirt is least steep at the top. The angle of the skirt with the horizontal will then be about 60 degrees. If the dimensions of the skirt in the graph of figure 11.15 are taken and the thickness of the ferrocement layer (with a density of 2426 kg/m^3) is assumed 0,03 m, this results in a $q_p = 0,46\ kN/m^2$. Concludingly the airpressure needs to be at least $q_p = 0,42\ kN/m^2$, causing a anchoring force of $\frac{1}{2}pr = \ 0,84\ kN/m$ perimeter. This would have to be anchored by a weight of at least 84 kilograms per meter, which corresponds with a minimum concrete cross-section of 190 × 190 mm.

The slope of the single curved skirt of figure 11.15 has an angle with the horizontal of 70 degrees. With the same dimensions of the ferrocement layer, this results in a $q_p = 0,26\ kN/m^2$. Concludingly the airpressure needs to be at least $q_p = 0,26\ kN/m^2$, causing a anchoring force of $\frac{1}{2}pr = \ 0,52\ kN/m$ perimeter. This would have to be anchored by a weight of at least 52 kilograms per meter, which corresponds with a minimum concrete cross-section of 150 × 150 mm.

The current anchoring force on the foundation amounts 3 kN/m for a dome with a base diameter of 8 meter (see section 9.2). Which results in a heavy foundation, in concrete this corresponds with a minimum concrete cross-section of 360 × 360 mm. According to analyses of loads during usage in section 10.2, a ringbeam of 200 × 200 would be sufficient.

It can be concluded that this design of the membrane does not require a heavier foundation than needed for loads during usage of the dome. This represents a considerable saving of material for the foundation (about 85%). However the connection of the membrane'skirt' to the foundation still needs to be airtight and also the design is dependent on electricity to keep the pressure under the 'skirt' on a required level. Yet in case of a power cut, the sphere keeps it's shape so there is no risk of collapse of the dome. The pressure needed for the skirt is quite low and the mortar that is plastered against the skirt will soon be self-bearing. Care must be taken to keep airpressure between certain levels, so that there is enough pressure to support the fresh mortar but not so much that the foundation is pulled out. It might be difficult to keep the transition from the skirt tot the sphere smooth. The load q_n (see page 166) will cause additional deformations.

Other shapes than an inflatable sphere are desirable for houses with only a ground floor. Imagine a house with a maximum height of 3 meters, the cross-section of a sphere-supported membrane would then look like the yellow drawing in figure 11.16. The diameter of the base is around 4 meter, of which only 3 have a height of more than 1,5 meter. These are not practical dimensions for a house. In the same graph a catenary is plotted in black with a height of 3 meter and a base diameter of 6 meter. The orange figure shows the inflatable that would be able to support this catenary; an inflatable with an elliptical cross-section. Yet it is obvious that this is not a practical shape eiter as only the blue area is really useful to live in, which almost equals the 'leftover' area.

Concludingly the membrane should be changed to a cross-section similar to the orange one in the upper left corner of figure 11.16. In case of the use of an elleptical cross-section of the inflatable, care should be taken to keep the roof

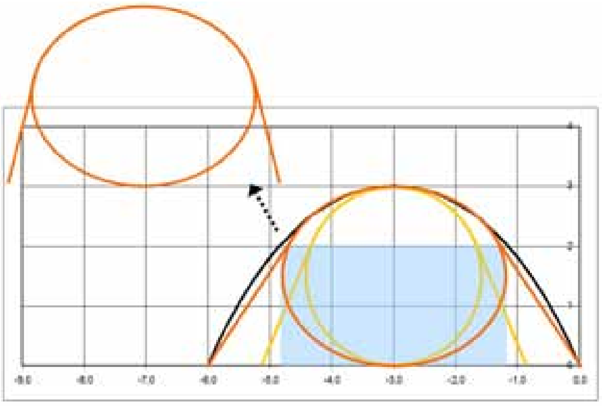

Figure 11.16: A house with only a ground level would require a different shape of the supporting airform, a catenary-based cross-section would not be practical either.

sufficiently steep to garantee the drain of rainwater and leaves. Especially as the top of the ellips will flatten as a result of load q_n, which needs to be taken into account when the membrane is designed.

11.2.4 Inflated Spheres Supporting Formwork, Conclusions

From calculations can be concluded that the part of the membrane that is not directly supported by the sphere, the 'skirt', needs considerable adaptations to prevent large deformations. Unfortunately these deformations are disadvantagous from a structural point of view. Tensioning of the 'skirt' nor stiffening this part by laths gives satisfying results if issues such as simplicity, deflections and the required foundation mass are considered.

Feasible options are 'support of the skirt by airpressure' and 'stiffening of the 'skirt' by inflated tubes'. The latter requires very expensive formwork, but no significant tension forces to be absorbed by the foundation nor airtight connection of the membrane to the foundation. It has quite an impact on the appearance of the shell's shape, as the inflated skirt results in a ribbed shell.

If the skirt is supported by airpressure, airtight anchorage of the airform to the foundation is necessary as well as a constant airflow. The anchoring force however, can be reduced to such a level that the foundation does not need to be heavier than is required for loads during usage of the ferrocement shell. Also, the dependency on electricity is less high as the form is mainly supported by the spherical form, which can be made airtight. As a consequence effects of a powercut will not be as severe as they are using a single membrane supported by airpressure (the current situation).

Figure 11.17: Fuji Group Pavillion,Expo'70, Osaka [Huybers, 1999]

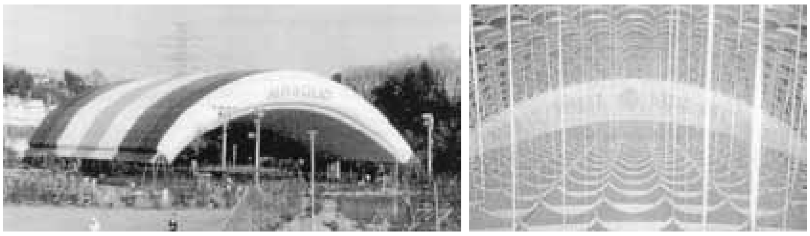

Figure 11.18: Airsolid, Japan [Huybers, 1999]

11.3 Pneus

Air inflated double membranes or 'pneus' are more similar to conventional structures in behaviour than pressurized single membranes. Textile airfilled tubes are used as columns and beams. Contrary to single membranes, like a SolidHouse's formwork or a cover for tennis courts, the structural parts are under pressure and not the structure itself. As a consequence much higher pressures are needed.

An example of a large inflatable structure is the Fuji Group Pavillion, that was build on the occasion of the Expo'70 in Osaka (figure 11.17). This very large inflatable structure consists of 16 arched tubes, each 72 meters in length and a 4 meter cross-section. The tubes are connected by strips and placed on the perimeter of a circular floor which has a cross-section of 50 meters. The pressure needed for the building to stay upright is 80 mbar, in case of bad weather conditions the pressure could be increased up to 250 mbar. To allow for changes in pressure due to climatic circumstances and small punctures, a continuous inflow of air was combined with a continuous but limited outflow of air. [Huybers, 1999]

Another example is the roofs of the Techno-Cosmos Pavillion in the Tsukuba Scientific Expo '85. These roofs or solcalled 'airsoilids' were made of inflatable matresses. The upper and lower membrane are connected by strips. Stiffness and stabiity is achieved by airpressure. The design was further developed and applied as a cover for tennis courts (figure 11.18). [Braam, 2000]

Pneus are especially used for events, temporary expositions. They can be found in various sizes and shapes. Often not the whole wall or roof is an inflated double membrane, but single membranes are supported by inflated tubes. A good example of this technique are the tents of Goldfinch (fig 11.19.

Figure 11.19: Examples of pneus (www.balloonworks.com)

Figure 11.20: The permanent roof of a parking garage in Montreux (Switzerland) is supported by inflatable girders [Lombardi, 2006].

In Switzerland inflatable girders have been developed for permanent use. These 'Tensairity®' combine an airbeam, a compression element and a cable for tension. First application is a roof over a parking garage in Montreux railway station, Switzerland. The roof covers 1700 m^2 and is composed 11 sections of tensioned single saddle shaped membranes, supported by 12 Tensairity®beams of about 27 m span and a vertical supporting steel structure (figure 11.20) [Luchsinger, 2006].

As a result of this orientation on pneus two designs were made for an inflatable dome. These will be discussed in section 11.3.3 and 11.3.2.

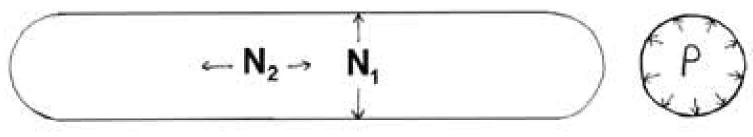

Figure 11.21: Stresses in the membrane of a pneumatic tube

11.3.1 Mechanics of Pneumatic Tubes

In the membrane of a pneumatic tube is (pre)tensioned by airpressure. In the circumferential direction of the membrane (n_1 in figure 11.21) this tensile stress amounts to:
$n_1 = p \cdot r$

In direction of the span, the tensile stress of the membrane n_2 depends on conditions at the ends of the tube, as normally here the loads are transferred to the membrane. The load on the end of a tube as a result of airpressure is $p \cdot \pi \cdot r^2$. As a concequence the membrane is prestressed in direction of the span by:
$n_p = \frac{p \cdot \pi \cdot r^2}{2 \cdot \pi \cdot r} = \frac{p \cdot r}{2}$

If the pneumatic tube is loaded by a uniform distributed load, the upper side of the tube will shorten, while the lower side will increase in length. Consequently the tensile stress caused by the airpressure in the upper part of the cross-section decreases and on the lower side it increases. Maximum resulting stresses can be found in the cross-section midspan as the moment resulting from the distributed load reaches a maximum $M_{max} = \frac{1}{8}ql^2$. See figure 11.22.

Based on Hooke's Law, stresses in the cross-section midspan are
$\sigma(z) = \frac{n_p}{t} + \frac{M_{max}z}{I_{zz}}$
n_p =prestress of the membrane as a result of airpressure
z =height of cross-section = r
$I_{zz} = \pi t r^3$

$n_2(z) = \sigma(z)t = \frac{pr}{2} - \frac{ql^2}{8\pi r^2}$

To avoid wrinkling of the membrane on the upper side, the upper side of the membrane must stay under tension. $n_2(r) = \frac{pr}{2} - \frac{ql^2}{8\pi r^2} \geq 0$
so $p \geq \frac{ql^2}{4\pi r^3}$
or more general $p \geq \frac{2Mt}{I_{zz}}$

If the pressure equals $\frac{2Mt}{I_{zz}}$ than the membrane stress midspan on the upper side of the beam is zero. With increasing load q this very local 'tension-free' area will grow until the pneumatic tube collapses.

Frank Braam tested a pneumatic tube with a 3 meter span, a diameter of 30 cm and an airpressure of 250 mbar (see figure 11.24). The tube was equipped with an (oversized) inner rubber membrane to make the beam very airtight. The membrane was made of PVC coated polyester with a membrane stiffness

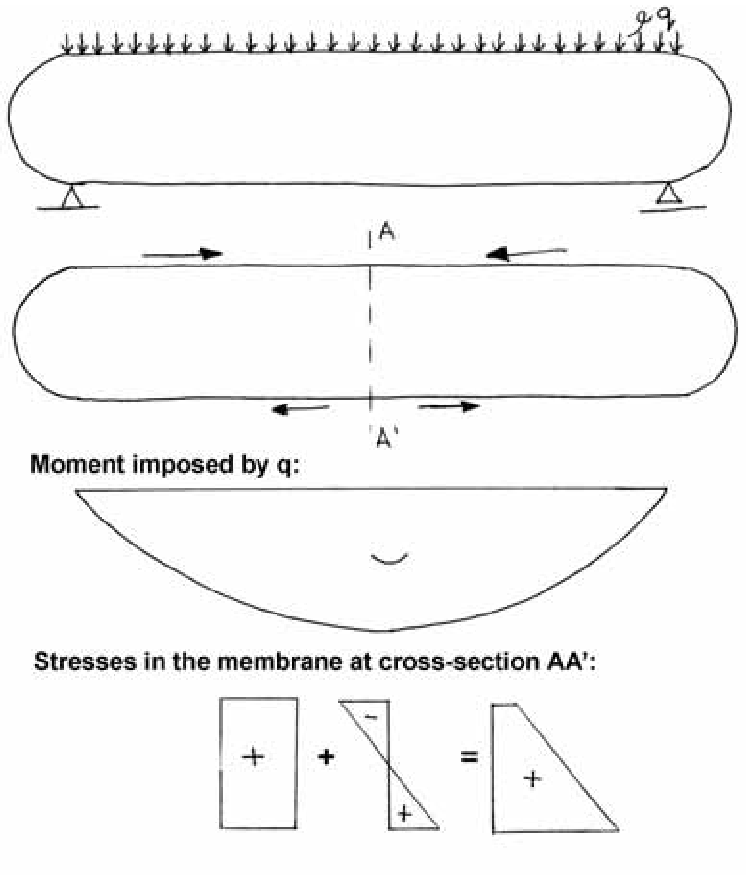

Figure 11.22: Stresses in the cross-section of a pneumatic tube loaded by a uniform distributed load.

Figure 11.23: Experiment of Frank Braam; a pneumatic tube loaded by a (roughly) uniform distributed load pulling on the upper side of the beam [Braam, 2000]

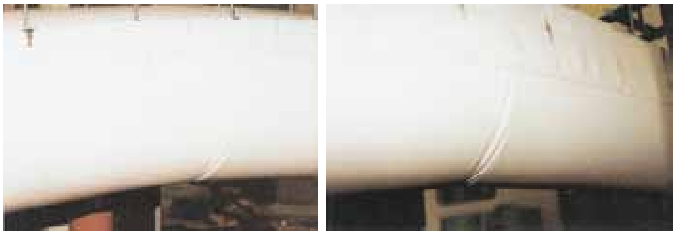

Figure 11.24: Experiment of Frank Braam; a pneumatic tube loaded by a (roughly) uniform distributed load pulling on the upper side of the beam, two picture of a wrinkle that develops at the lower side as a result of the uniform distributed load [Braam, 2000]

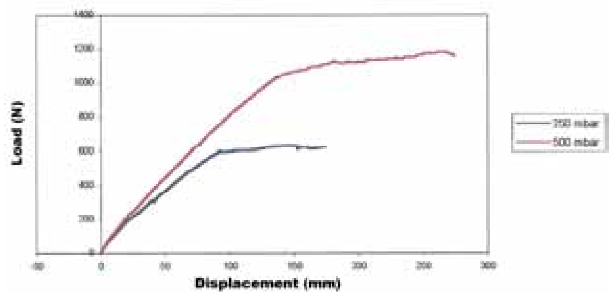

Figure 11.25: A higher pressure increases the capacity of the pneumatic beam, but the deflections as well [Braam, 2000].

(Et) of 575 kN/m. In figure 11.28 the results of his experiment are compared to theoretical values (i.e. the wrinkling moment according to the described formula's). This data can be used to have an idea of deflections of a pneumatic tube.

From other tests, see figure 11.25, Frank concluded that a higher pressure does increase the maximum load the beam can absorb, but also considerably increases the deflections. A larger cross-section of the tube or a smaller span is therefore more effective to decrease deflections than a higher pressure level [Braam, 2000].

Concerning normal loads (N) on the beam: $N < \pi p r^2$ Frank Braam carried out a test on buckling load as well. The results can bee seen in figure 11.31.

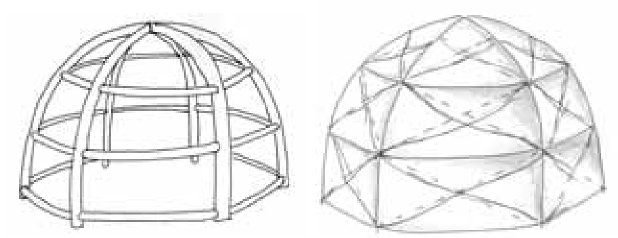

Figure 11.26: A membrane supported by an system of inflatable tubes will undergo large deflections under loading conditions.

11.3.2 Air Inflated Supporting Structure

A dome could be made by creating an inflatable supporting structure for a hemispherical membrane. The tubes could be orientated in circumferential and meridional directions or as a frame for a geodesic dome. Tunnels in the membrane could be filled by rubber (cycle) tires, in order to make the system airtight and increase the admissable level of airpressure.

However, as shown by calculations in section 11.2.1 a loaded single membrane results in relatively large deflections. Even if the pressure in the tubes would result in a very stiff supporting structure, the membrane would deflect under the load of the fresh concete. Apart from the fact that this is not desirable from an aesthetic point of view, it is not advantagous from a structural point of view. Moreover, the surface of the dome would consist of single curved facets instead of a double curved shell, which increases stresses in the shell considerably.

Therefore this design is considered as unsuitable for use by the Solid House Foundation.

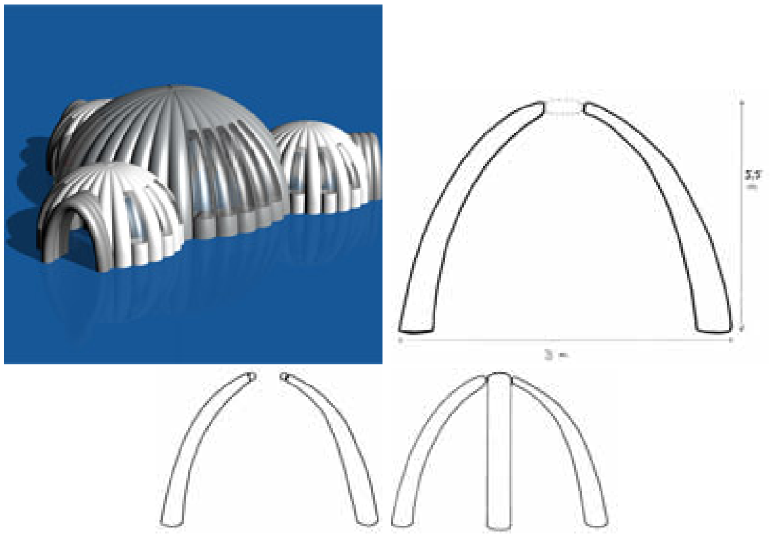

Figure 11.27: Design applying a double membrane with possible adaptations that might decrease deflections (by author).

11.3.3 Dome-Mattress

Contrary to the design in the last section, this dome consists entirely of air-filled tubes. Actually it is a sort of 3D mattress. As all tubes are coupled ringtension and ringpressure stresses can occur to a certain extent, which stiffens the structure. The ribbed surface results in a ribbed domeshell, which is stiffer than an unribbed shell (see section 4.2). In figure 11.27 several sketches, impressions and possible cross-sections can be seen.

As can be seen in figure 11.28 relatively small loads cause considerable deflections already. The design could easily handle the load according to the theory of 11.3.1: Moments will be small in the structure under the equally distributed load of the fresh concrete because of the arched shape of the tube. The maximum normal load on the arches can be calculated with the maximum loads on the ringbeam of the catenaric dome of section 8.4 with a shell thickness of 0,03 meter, assuming a catenaric cross section of the inflatable dome. $N_{max} = \sqrt{F_{y,max}^2 + F_{x,max}^2} = 4.6\ kN/m$, with p = 1 bar this results in a minimum cross section of 43 mm. However this is not realistic, thus deflections are normative.

Unfortunately the only data available on deflections of pneumatic tubes is on straight tubes with constant cross-section. To give a precise expectation of deflections the design should be modelled in Easy. Unfortunately this was not possible within the time span of this thesis. Therefore it is tried to give a rough estimation of the deflections to be expected.

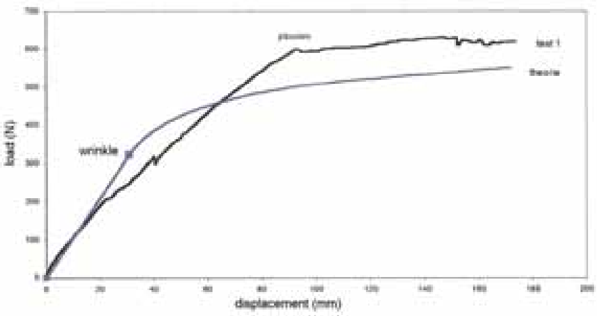

Figure 11.28: Results of the experiment of Frank Braam; a pneumatic tube loaded by a uniform distributed load. The pneumatic tube has a span of 3 meter, a diameter of 30 cm and an airpressure of 250 mbar [Braam, 2000].

Unfavourable Approach
A dome with a base of 8 meter across loaded by a uniform distributed load (fresh concrete) is compared with the beam in figure 11.28. The beam in the figure had a length of 3 meter, a cross-section of 0,3 m and was pretensioned by 250 mbar airpressure. The concrete, assuming a layer of maximum 3 cm is applied, causes a load of
$Q = 24\ kN/m^3 \cdot 0,03\ m * 0,3\ m * 3\ m =\ 650\ N$
According to figure 11.28 the beam will have a deflection of more than 10 cm, it will collapse. Allthough the span of the dome is more than 2 times larger, which in case of a beam causes a 16 times larger deflection, the deflection is limited by the arched shape and ringstresses. The load is not perpendicular to the cross-section and moments are small. Allthough ringstresses will be less strong as the dome is more flexibel in circumferential direction than a normal double curved shell, they do decrease deflections.

Now the dome is simplified as a two beams with three hinges (figure 11.29), the dome's base still being 8 meter across while the heigt is 5,5 meter. Each (straight) beam has a span of 6,4 meters and the angle with the floor of 54 degrees. If a load of $Q = 650 \cdot \frac{4}{6,4} = 400\ N$ is applied on the beam of figure 11.28 the deflection is 5 cm. Considering the span being more than twice as large, deflections are around 80 cm. This assumption is on the safe side as the ringstresses and the shape of the span (decreasing moments to a minimum) are not taken into account.

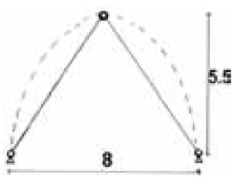

Figure 11.29: Three hinges

However, unfavourable is the fact that the diameter of the tubes decreases in the direction of the top. The loaded surface decreases also, but q has less influence on the deflection than the diameter of the tubes [3]. Also, the load component perpendicular to the surface of the dome increases in the direction of the top (figure 11.30).

[3]The formula for the deflection of a free supported beam loaded by a uniform distributed load is $y = \frac{5}{384}\frac{ql^4}{\pi E t r^3}$.

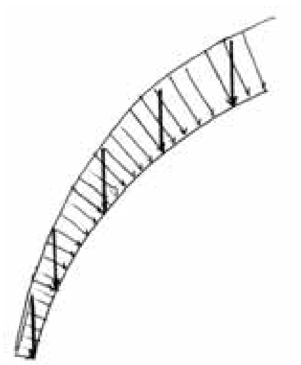

Figure 11.30: The load component perpendicular to the membrane's surface increases in direction of the top.

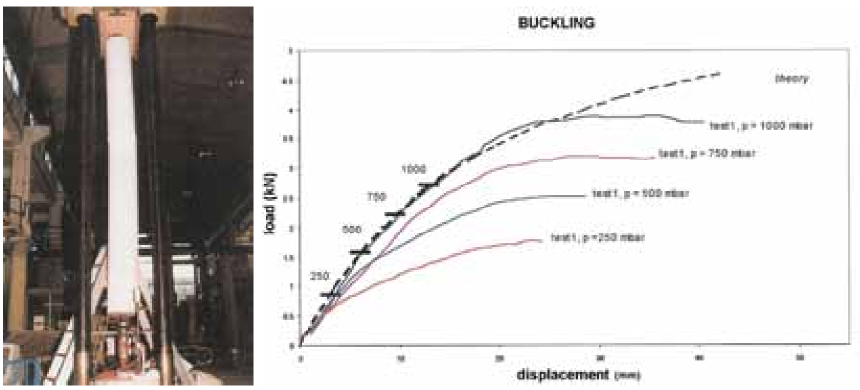

Figure 11.31: Results of the experiment of Frank Braam; a pneumatic tube loaded on pressure. The pneumatic tube has a span of 3 meter and a diameter of 30 cm [Braam, 2000].

Favourable Approach
If the design is approached as a dome, moments caused by a uniform distibuted load will be very small. Normal forces will be causing the deflections. The maximum load on the ringbeam of a 8 meter diameter catenaric dome with a shell thickness of 0,03 meter amounts 4.6 kN/m. For a beam with a minimum diameter of 0,3 meter, which has an initial angle of approximately zero this results in a normal force of 1400 N. In figure 11.31 this normal load causes a deflection of 13 mm for a beam with a diameter of 0,3 meter, a span of 3 meter and an internal pressure of 250 mbar. Considering Euler's formula for buckling [4] this deflection will be more than 2^2 larger for a beam spanning 6,4 meter, amounting to 6 cm.

This results in a very rough assumption:
The deflection of a 8 meter diameter catenaric dome loaded by a 3 cm layer of fresh concrete, has a value between 6 cm and 1 meter, if a minimum tube diameter of 0,3 meter and an airpressure of 250 mbar is applied.

The relatively large deflections, combined with a high price for the membrane in order to make pressures of 250 mbar (25 kN/m^2) do not make the design a very favourable option. However if Easy is used to model the membrane in such a way that it approaches the desired shape when deflected as a result of the load of the fresh concrete, it is a good option from a structural point of view. Also, it would mean a large simplification of the building process; no anchorage is needed except for windload and the membrane is completely airtight.

11.3.4 Pneus, Conclusions

Only a completely double membrane could be considered as an option. The design 'Dome-Mattress' of section 11.3.3 needs further research in Easy to make a more precise estimation of the deflections of this model. If deflections turn out not to be very large the membrane could be designed in such a way that it attains it's desired shape under loading conditions. However, the costs for the membrane will be very high as apart from the fact that it is double instead of single it should be able to resist airpressures up to 250 mbar (25 kN/m^2) and has to be made airtight, if necessary with a rubber inner tube.

[4] $F_E = \frac{\pi EI}{l^2}$

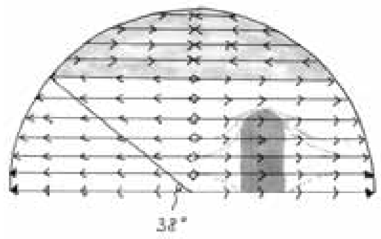

Figure 11.32: Holes in a shell interrupt tension rings and weaken the shell

11.4 Pneus for Roof Only

A full sphere can be made airtight, does not need anchorage and does not deform by airpressure. In section 9.4.2 several ideas were put forward to use the upper half of a full sphere as formwork. Most practical is to build a wall first and put the sphere inside, if necessary it can be partly excavated. The Solid House Foundation does not directly approve of this method, afraid that a combination of building methods will complicate the building process. Also the straight walls change the appearance of the SolidHouses. However this combination can render a solution to problems such as the dependence on electricity and the heavy foundation. Also the combination of a straight wall with openings and a double curved roof with only an opening at the top is more logical from a structural point of view. Openings weaken a shell (see figure 11.32) and are much easier to make in a straight wall. Besides, a straight wall is more practical in use than a doube curved wall. All in all the use of a part of an inflated sphere or similar form needs to be considered more thoroughly, which is done in this section.

The wall could be made with Concrete Stabilised Earth Blocks, see section 6.4. In Inspector Eatham, an earthblock press is already present. It is now used to make bricks for the inner walls. Why not use these blocks for structural walls as well? The blocks consist for the plupart of free material. Thanks to the added cement they are stabilised and not sensible to rain. The skills of laying blocks need not require a lot of training. The recently constructed inner walls of the first dome in Inspector Eatham prove this, see figure 11.33.

Besides, preconstructed walls make the construction of floors easier; supports for the first floor can be provided for in the walls. Currently (in Sr Lanka) the construction of the floor is not thought about until the construction of the shell has been completed. As no supports are created on the inside of the shell, the floor needs to be supported by inner walls. Consequently the placement of the inner walls is restricted to allow for a minimum span of beams and girders for the floor. This completely undoes one of the main strenght of a dome: a dome facilitates free lay-out of the floor plan as the roof is completely supported by the outer walls. Actually creating bearing walls inside a dome equals the buiding of 'a house inside a house'.

On top of the wall that bears the domeshell a ringbeam should be allowed for. The outer rim of the wall can be made several centimeters higher than the inner

Figure 11.33: Inner walls built with 'home-made' CSEB's (see section 6.4) in the first dome in Inspector Eatham [Viguurs].

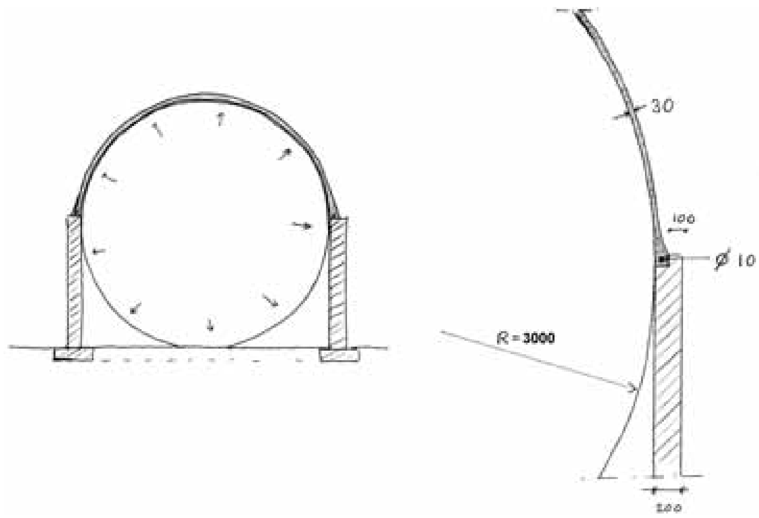

Figure 11.34: A sketch of the idea of using a full sphere in combination with a wall. A small ringbeam on top of the wall, reinforced with rebar, prevents ring tension forces to develop in the wall. The detail of the ringbeam is scaled. Measurements are in millimeters.

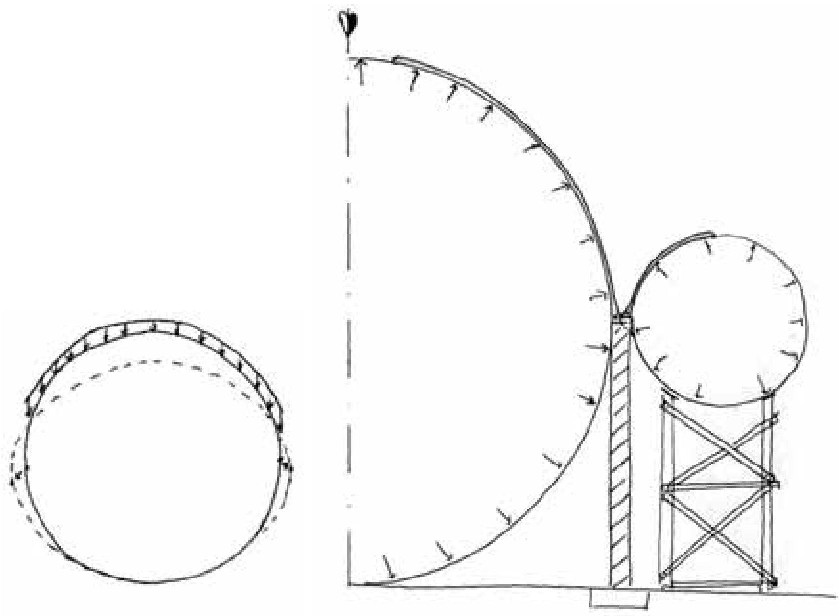

Figure 11.35: Swelling of the sphere, depending on the airpressure. Idea of a donut shaped inflatable form to counteract swelling forces and to provide formwork for a cover and raingutter.

rim, creating room for a ringbeam. A round 10 mm rebar is lain on spacers and the void can be filled with concrete. This rebar can absorb the ringtension forces that have developed in the shell. Consequently no ringtension forces have to be taken by the brick walls and openings do not cause peak stresses. The design of a raingutter to collect rainwater should be considered, as well as the connection of a cover to provide shade next to the house.

A potential problem is space between the wall and the sphere due to inaccuracies when building the wall. However the sphere will deflect a bit under the load of the fresh concrete, swelling at the side and filling the void. Care should be taken that the pressure of the balloon does not create high loads in the wall. A 'donut'-shaped volume might be a possibility to counteract these forces and at the same time provide a formwork for a cover combined with a raingutter (figure 11.35).

In figure 11.36 cross-sections of several sphere diameters and their corresponding domes are shown and compared with human height. For a dome with a first floor, a sphere of 2,5 to 3 meter would be most practical. However if a larger ground surface is desired, heights tend to get rather unpractical. A dome higher than 6 meter is unpractical in construction and a lot of space is useless unless another floor is added. The only good thing about extra height is that it enables hot air to lift. But it is a waste to use the materials to reach a certain height of which only a part is used. Besides, a hole in the top facilitates the lift of hot air as well.

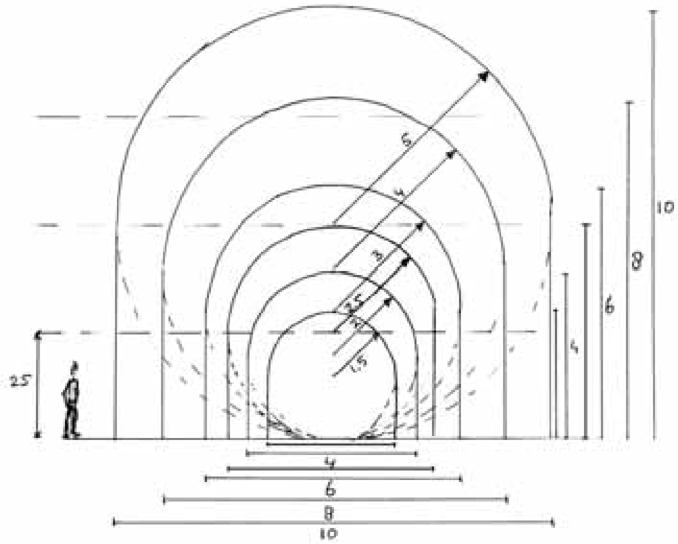

Figure 11.36: Using full spheres does not always result in practical dimensions. A dome with only a ground floor would use a sphere with a maximum diameter of 3 meters, which corresponds with a very small base surface. A larger base surface however also results in larger heights. For a dome with a first floor, a sphere of 5 or 6 meter diameter would be most suitable. Larger base surfaces are unpractical as they result in even higher domes. Measurements in meters.

A dome with only a ground floor is unpractical to construct with a complete sphere, as can be seen in 11.36. If the maximum height of the dome is 3 meter, the diameter of the base is 3 meter as well, which is far too small. An option would be to use a larger diameter and partly excavate the sphere. Consider that a reasonable base diameter for a house with only a ground level would be 6 meter. In that case a volume of 57 m^3 would need to be excavated to reach a maximum height of 3 meter. This volume corresponds with 870 wheelbarrow loads! (65 liters per barrow)

Another disadvantage of a complete sphere is that the construction of the first floor has to wait until the domeshell has hardened and the formwork is deflated. Subsequently the supply of materials and the space available for the construction workers is restricted by the shell.

Concludingly, another shape of the inflatable would be more practical. In the past eleptical shapes have been used for inflatable roofs. An example is the cover for the Boston Arths Centre Theatre by Koch, Ross and Weidlinger [Huybers, 1999](see figure 11.38). Cables were used to reduce the necessary height of the construction. Two nylon membranes are span in a steel pressure ring with a diameter of 35 meter. The maximum distance between the two membranes is 6 meter in the middle.

In figure 11.37 is shown how the inflatable membrane is pretensioned with n by

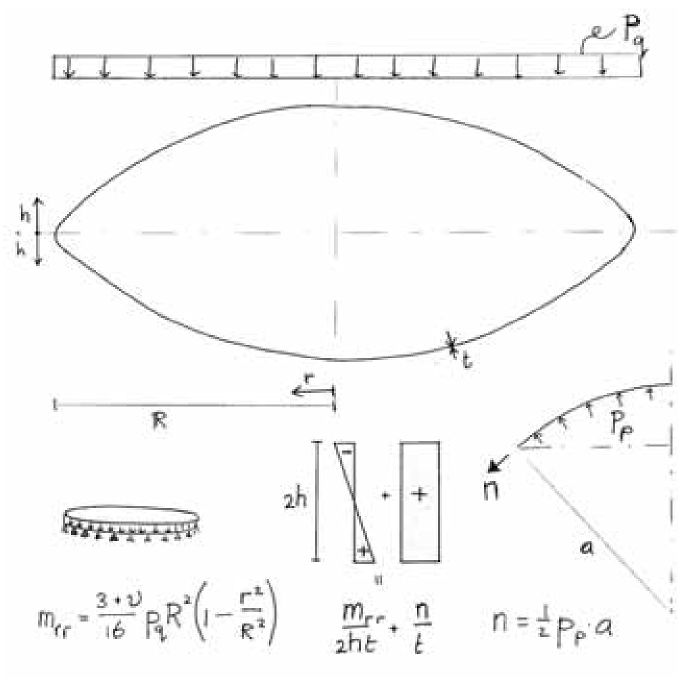

Figure 11.37: The airpressure prespans the membrane with tension n. The uniform distributed load causes a moment m_{rr} in the air inflated form. The moment reduces the tension in the upper part of the membrane and increases the tension in the lower part.

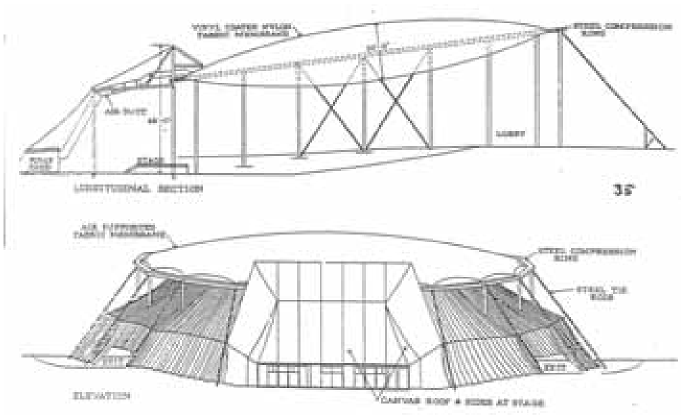

Figure 11.38: Boston Arts Centre Theatre by Koch, Ross and Weidlinger. Cables were used to reduce the necessary height of the construction. [Huybers, 1999]

airpressure p_p and loaded by the pressure p_q of the fresh concrete. p_q imposes a moment $m_r r$ on the air inflated ellips. This reduces the tension in the upper part of the membrane and increases the tension in the lower part. If no tension is present in the upper membrane, it will wrinkle and ultimately the cushion will collapse. Therefore the tension n caused by the airpressure should be larger than the 'pressure' imposed by the moment $m_r r$:
$\frac{n}{t} > \frac{m_{rr}}{2ht}$
$n > \frac{m_{rr}}{2h}$
(see figure 11.37)

For a simply supported plate $m_{rr} = \frac{3+\nu}{16} p_q R^2 (1 - \frac{r^2}{R^2})$ [Blaauwendraad, 2002]
ν is the contraction coefficient, which is 0,5 for an inflatable form as air is incompressible (and it is an assumption on the safe side). In the middle of the span the moment is maximum:
$m_{rr} = \frac{3+\nu}{16} p_q R^2$

$p_q = 2400\ kg/m^3 \cdot 9,8\ m/s^2 \cdot 0,03\ m + 2 \cdot 0,9\ kg/m^2 \cdot 9,8\ m/s^2 \cdot \pi R^2 = 1,2\ kN/m^2$
The span is 6 meter and the elliptical shape consists of two membranes, with each a radius a of 5 meter (so h_{max} is 1 meter):

$m_{rr} = \frac{3,5}{16} \cdot 0,72 \cdot 3^2 = 2,36\ kN$
$n = \frac{1}{2} p_p a = \frac{1}{2} \cdot 1,5 kN/m^2 \cdot 5\ m = 3,75\ kN/m$
$a = \frac{h^2+R^2}{2h}$

$p_p > \frac{2 \cdot \frac{3+\nu}{16} p_q R^2}{h^2+R^2}$
$p_p > \frac{\frac{7}{16} 1,2 \cdot 3^2}{1^2+3^2}$
$p_p > 0,47\ kN/m^2$

The pressure needed to support the weight of the fresh conrete and the weight of the membrane is reasonable. However the membrane should be accessible as well in order to apply the ferrocement. A concentrated load of 740 kg over an area of 140 cm^2 causes a maximum moment $m_{max,rr,F} = 22,8\ Nm/m$.
$n > \frac{m_{rr,q}+m_{rr,F}}{2h}$
$p_p > \frac{m_{rr,q}+m_{rr,F}}{ah}$
$p_p > \frac{2,36+0,023}{5 \cdot 1}$
$p_p > 0,48\ kN/m^2$

However deformations are probably normative. This should be further analysed, for example with EASY.

Instead of cables and a pressure ring, an inner tube could be used to span the sides to increase tension in the membranes. This also prevents wrinkling of the sides of the membranes as a result of airpressure and it makes the sides stiffer so that supporting stresses can more easily be absorbed (see figure 11.39). More important, it can be inflated first, so that this ring can be put on top of the walls before the elliptical membrane is inflated.

Considering the weight of the current formwork for a 6 meter dome being 60

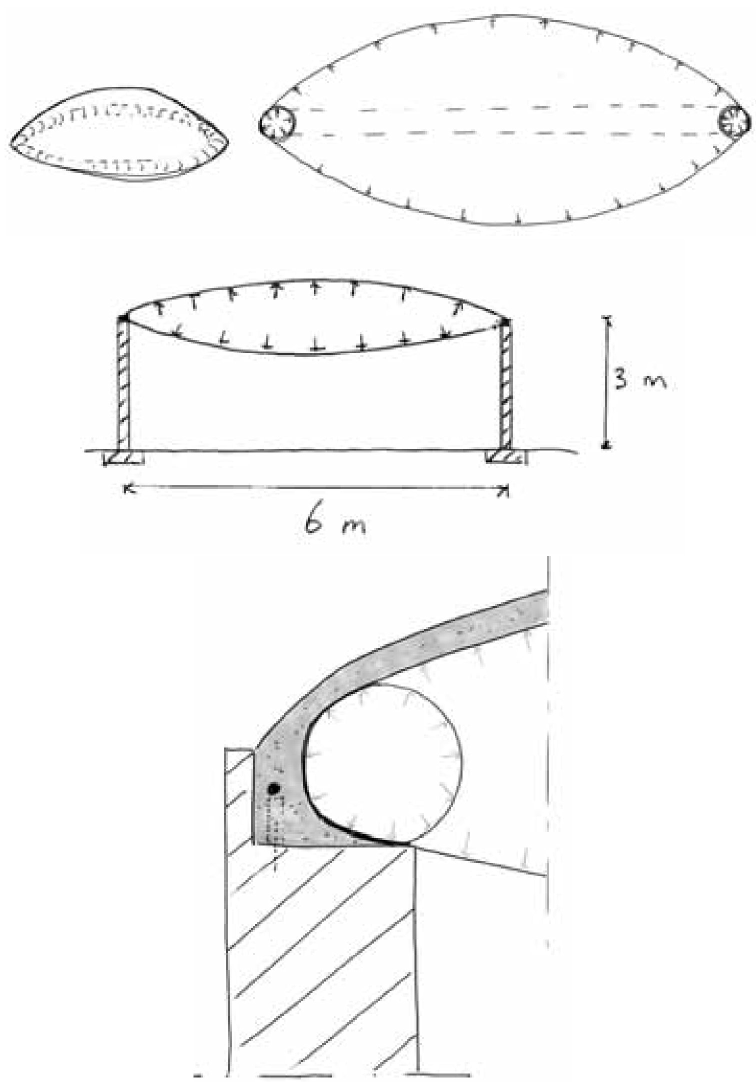

Figure 11.39: The airpressure prespans the membrane with tension n. The uniform distributed load causes a moment m_{rr} in the air inflated form. The moment reduces the tension in the upper part of the membrane and increases the tension in the lower part.

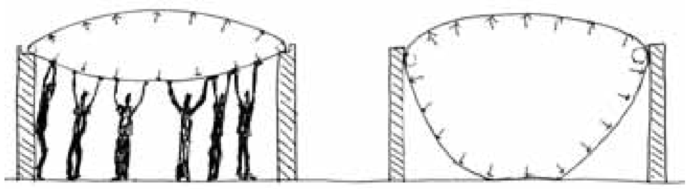

Figure 11.40: People will have to lift the formwork above their heads until it is completely inflated. Therefore, and to decrease deflections, the lower part of the formwork could be increased so that it reaches the ground. However in case of a first floor this increased demands on the strenght of the first floor.

to 90 kilograms, this formwork will probably be even heavier. If the membrane weighs 1 kg/m^2 and the span is 9 m, the weight will be at least 130 kg. Imagine the angle between the wall and the lower membrane to be 45 degrees. Then the load of the inflated membrane will cause a pressure in the inflated ring of 280 N [5]. The pressure in the inner tube must be very high to prevent it from buckling. Apart from that, it will be difficult to get the inflatable on top of the wall.People will have to lift the formwork, put the inflated ring of the membrane on top and keep the formwork above their heads untill the elliptical membrane is inflated. Therefore it will be practical to increase the height of the lower part of the elliptical membrane, so that it can rest on the floor (or first floor) as is shown in figure 11.40. In that case the inflatable ring will not be put on top of the wall, but it will push the formwork against the wall. This will look similar as the cross-section in figure 11.34. A disadvantage is that this design will impose extra requirements on the strength of a potential first floor. The floor will need to be able to support both the weight of the membrane as the weight of fresh concrete on top.

The resulting shell of 6 meter span and with a height of 1 meter is modelled in ANSYS. Gravity load combined with two concentrated loads (pressure 55 000 N/m^2 on two areas of 125 cm^2 each). Maximum tension stress in the shell amounts $0,6\ N/mm^2$ and a maximum reaction solution of $F_x = 1,6\ kN$ results in a ringtension force of 2,4 kN. A 10 mm diameter rebar as reinforcement for the ringbeam will be sufficient.

[5] $\pi * 4,5^2 * 2 * 1\ kg/m^2\ /\ 9 * \pi = 4,5\ kg/m\ perimeter$, with an angle of 45 degrees the horizontal load on the inner ring will amount to $4,5\ kg/m * 9,8\ m/s^2\ /\ cos45 = 62\ N/m$ which results in a pressure of $62\ N/m * 4,5\ m = 280\ N$

11.4.1 Pneus for Roof Only, Conclusions

This alternative of using a full sphere is worth considering as the method is very simple. The formwork is very straightforward and problems such as the heavy foundation are solved. However this alternative is not practical to use for houses with only a ground floor, or where a floor diameter of more than 6 meter is required. Therefore research was done on elliptical air inflated membranes. If deformations are not too large, this will be a very suitable method. The formwork can be inflated on ground level or on the first floor and needs not be anchored. In this way the first floor can be constructed in the open air. A disadvantage is that the formwork needs to be lifted in place until it has been inflated completely. Also, the required airpressures will be considerably higher than for the spherical membrane, puts higher demands on the strenght and the airtightness of the membrane. From this practical point of view as well as far as deflections of the loaded inflatable are concerned, it would be even better if the lower part of the inflatable reaches the floor. However in case of a dome with a first floor, this requires the first floor to be able to bear the load of both the inflatable as the fresh concrete on top, unless the floor is propped (stutten). An oversized rubber inner membrane can garantuee airtightness. As the lower part of the inflatable is supported by the ground, pressures need not be as high as for an inflatable as described in section 11.3.3.

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11.5 Summary

The design of the inflatable formwork will need to change in order to facilitate airtightness of the formwork and to take away or reduce dependency on electricity and the need of a heavy foundation for anchorage of the formwork. A change in design will require a (sometimes high) initial investement in new formwork. This investment should be compared to the number of domes that is going to be build and the potential cost savings in electricity and foundation material.

Of each section a short summary is given of the most feasible alternatives and their advantages and disadvantages. Concerning airtightness, all closed formworks can be made airtight. In case of air pressures above 2 kN/m^2 an oversized rubber inner membrane is most practical. In case of pressures around 2 kN/m^2 and lower a coating of the inside of the membrane will be sufficient. The amount of compartments has a significant influence on the price of the formwork, especially if these will be pretensioned by a high airpressure.

Stiffened Closed Formwork
An *exterior supporting system* can take away the need for anchorage of the inflated formwork to the foundation. The system is assumed to be too complicated to build on site and should therefore be designed and constructed as lightweight as possible in the Netherlands. A first design of the system, using steel sections, increases weight and thus the transportation costs of the whole package at least 4,5 times.

Inflated Spheres Supporting Formwork
A general disadvantage of using full spheres is the restrictions in height and span ratio. An *inflated full sphere with a skirt stiffened by airfilled tubes* is a feasible but very expensive inflatable formwork. Deflections of the skirt caused by the weight of fresh concrete are disadvantagous from a structural point of view. High pressure will be needed to reduce these deflections of the skirt, which consists of a lot of different compartments. Heavy anchorage is not needed though.

A *inflated full sphere with a skirt that is kept under pressure* does not take a way the need of airtight anchorage to the foundation nor the dependency of electricity, though it does reduce both.

Pneus
The only feasible option is the *dome-mattress*, a dome which solely consists of connected air-inflated tubes. Deflections need to be analysed in a program such as EASY, as for this design deflections will be normative. The design will need to take deflections into account in such a way that the formwork will deflect to the desired dimensions when loaded by the fresh concrete. Because of the many different compartment and the high level of air-pressure needed, the formwork will be very expensive.

Pneus for Roof Only
In this section inflatable membranes are combined with brick walls. The brick walls can be constructed of the Concrete Stabilized Earthblocks that are now only used to construct the inner walls. The shell is not weakened by openings,

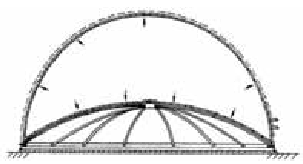

Figure 11.41: Stiffened Closed Formwork

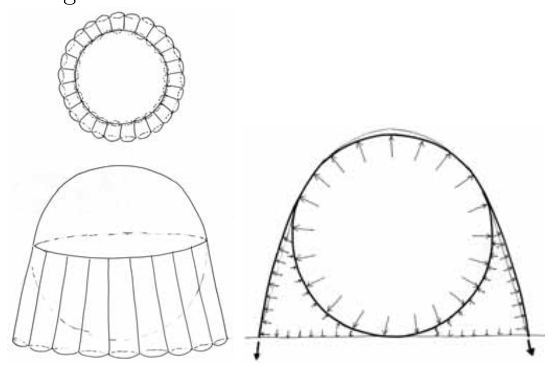

Figure 11.42: Inflated Spheres Supporting Formwork

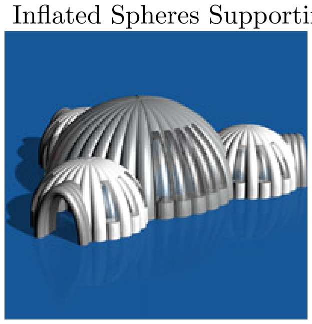

Figure 11.43: Pneus

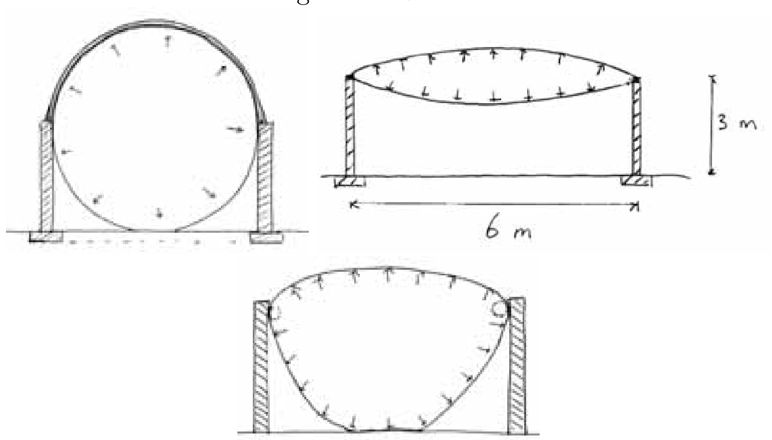

Figure 11.44: Pneus for Roof Only

Figure 11.45: An overview of the feasible alternatives

openings can be made more easily and a straight wall is more practical in use. Also, it facilitates the construction of a first floor.

A *full sphere* requires a very uncomplicated formwork which does not need heavy anchorage to the foundation and is easy to make airtight. However the sphere restricts the span-height ratio. This is extremely unpractical when constructing a dome with only a ground floor as a small height will also result in a very small span.

An *elliptical inflatable* would facilitate a wider range of combinations. Also it facilitates the construction of a first floor. However it will require quite an effort to get the inflatable in place and to keep it in place. Besides the inflatable, especially the inner tube, will require a much higher pressure than the spherical inflatable to reduce deflections.

A possibility is to *increase the height of the lower part of the inflatable to the floor*. Thereby reducing deformations and removing the need to support the membrane until it has been completely inflated. Yet this is less suitable for domes with a first floor as it sifnificantly increases demands on the bearing capacity of the first floor.

11.6 Conclusions and Recommendations

The most straightforward alternative formwork is the full sphere used only for roof construction. It needs not be bolted to the foundation, nor heavily anchored in another way. Also it will not be an expensive formwork, nor will it be difficult to make airtight. The shell needs not be weakened by openings, and the placement of openings in the vertical wall is easier and more flexible. Also, the vertical wall will be more practical to use for the inhabitants, creating less 'useless' space. The outer walls can be constructed similarly as the inner walls, supports for a floor can be provided for. An inflatable, such as one with an the adapted elliptical cross-section (which reaches the floor), would even facilitate the construction of a first floor before the construction of the roof. More important, the height and span ratio of the construction are no longer be limited.

However, such a different formwork and construction method does change the appearance of the solid-houses. The question is whether this is acceptable for the Solid House Foundation.

A compromise would be the alternative 'inflated full sphere with a skirt that is kept under pressure', from section 11.2.3. This design of the membrane does not require a heavier foundation than needed for loads during usage of the dome. However the 'skirt' still needs to be bolted to the foundation and this connection needs to be made airtight as well. Also, this inflatable formwork is dependent on electricity to keep the pressure under the 'skirt' on a required level. Yet in case of a power cut, the sphere keeps it's shape so there is no risk of collapse of the dome. The pressure needed for the skirt is quite low and the mortar that is plastered against the skirt will soon be self-bearing. Care must be taken to keep airpressure between certain levels, so that there is enough pressure to support the fresh mortar but not so much that the foundation is pulled out. It might be difficult to keep the transition from the skirt tot the sphere smooth.

There are other options, such as the Dome-mattress of section 11.3.3. Further research on deflections of this inflatable are needed before a conclusion concerning the suitability of this option can be drawn. What can be stated though, is that this alternative will result in a very expensive formwork and a ribbed surface of the dome.

Figure 11.46: Building brick walls in the first dome in Inspector Eatham, Sri Lanka. [Viguurs]

11.7 Reflection

In the scope of this thesis I have restricted my research on the use of inflatable forms to make domes, but in practice there is no such restriction. Therefore I would like to reflect upon the application of this building concept in general.

The strength of a dome is that it can enclose a large space with little material, facilitating a flexible lay-out of the plan as no bearing inner walls are necessary. In the current Solid Houses in Sri Lanka however, inner walls are made of heavy bricks and their placement is not even flexible as they need to support the first floor. One of the main problems after the construction of the dome is made, is the partition of the inner space in a practical way, while at the same time creating supports for the first floor. Walls are sometimes built up to the roof, which requires a lot of building material while these walls are not necessary from a structural point of view. In other words, if brick walls are desired to divide the inner space up to the roof, why not use them to carry a roof?

The ringtension and -pressure forces make a domeshell very strong. Openings however weaken the shell as the ringforces need to be diverted. In Sri Lanka inhabitants wish to have as many openings as possible in their homes.

Concludingly some of the main strengths of a domeshell are undone by the way it is now used. Why then construct domes when they are adapted to resemble a standard home as much as possible?

The Solid House Foundation is aware of this contrast and would rather use light materials indoors to allow for future changes in the dome's function, creating sustainable shelters. However, community building is at least as important for the SHF as 'dome building'. Consequently if the inhabitants propose brick walls for the first dome this is not overruled. The change of the domes into a more sustainable practical solution should therefore be approached very delicately and be stretched over a number of 'test-domes'.

In section 11.4 a proposal is made to use the shell only for the roof. This not only solves the problem of the anchorage of the inflatable formwork but also facilitates openings in the walls without weakening the shell. Yet the plan is still circular and partition walls will be made thought they do not carry the roof. What is then the advantage compared to a house with a rectangular floorplan and a roof made of elements such as corrugated plates? And do these advantages outweigh the advantage of a rectangular floorplan?

A dome can resist earthquakes and survive other extreme climatic circumstances such as tornadoes. However as the shell is very exposed to the weather circumstances throughout the year, material use is very restricted. More important, the walls are not protected against direct sunlight, which is a disadvantage in a tropical climate such as Sri Lanka's. Besides, for the construction of a house with brick walls and an overhanging roof of corrugated plates no formwork, nor electricity is needed.

On the other hand, the Solid House Foundation gets a lot of attention with it's unusual way of building, which enables the Foundation to raise funds and build many SolidHouses. Although it might not be the most practical or affordable concept, it can be built with little training and so far their future inhabitants are very happy to have a house at all.

Yet in this reflection I would like to emphasize that one should not blindly press upon the application of domes for housing purposes. Care should be taken not to push people to live in an adapted dome while they would actually prefer a 'normal' house, which might even be cheaper and easier in construction as well.

The Solid House Foundation is conscious of this issue, and therefore discusses the concept extensively with future inhabitants to be certain to provide them with a house that is according to their needs and wishes. At the same time this explains the apparent discrepancy mentioned above.

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List of Figures

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APPENDICES

Appendix A

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Appendix B

Foundation Floor

To determine the necessary level of reinforcement for the floor a number of calculations, assumptions and simplifications has been made. An overview is presented in this section.

The foundation floor is schematized as a circular concrete slab supported by springs. Depending on the ratio of stiffnes between floor and ground, the spring stiffnes is either constant or variable, see figure B.1 [van Tol, 2000].

The stiffnessratio is defined as:
$c = \frac{1}{12}\frac{Eh^3}{E_g l^3}$
$E,\ Young\ Modulus\ floor = 26\ 000\ N/mm^2$
$h,\ height\ floor = 100\ mm$
$l,\ span\ floor = 6\ 000\ mm$
$E_g,\ Bq_c$ [van Tol, 2001]
$B = 2$
$q_c,\ conoïd\ \ resistance\ = 2\ N/mm^2$ [1]

This results in a value of 0,0025 for c, which is smaller than 0,01. Consequently the ground can be seen as relatively stiff and the spring stiffness may assumed to be constant. The spring stiffnes is assumed $k = 0,04\ N/mm^3$(see figure B.2).

[1] Little is known about the soil properties in Inspector Eatham. According to Rik Lurinks, who has been working on the first foundation of the first SolidHouse in the area, the red soil is a mixture of sand and a bit of clay. Sometimes little rocks can be found. The soil is hard to dig. To be on the safe side the conoïd resistance of clay has been used [CUR/PBV, 2000].

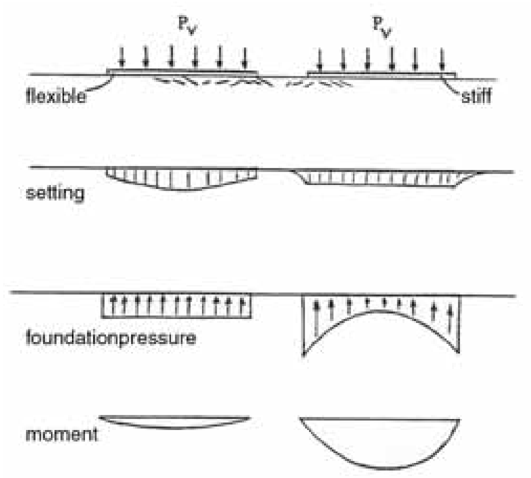

Figure B.1: The spring stiffnes depends on the ratio of stiffness [van Tol, 2000]

Enkele richtwaarden voor verschillende grondeigenschappen

Soort ondergrond	Conuswst. q_c [N/mm²]	Beddinggetal[1] k [N/mm³]	Elast.mod. E_{dyn} [N/mm²]	CBR-waarde[2] [%]	$\frac{E_{stat}}{q_c \cdot a}$	$\frac{E_{dyn}}{q_c \cdot b \cdot a}$
Veen	0,1-0,3	0,01–0,02	10–35	1–2	0,5	10,0
Klei	0,2-2,5	0,02–0,04	15–60	3–8	1-2	7,5
Leem	1,0-3,0	0,03–0,06	50–100	5–10	–	5,0
Zand	3,0-25,0	0,04–0,10	70–200	8–18	4-7	2,5
grind/zand	10,0-30,0	0,08–0,13	120–300	15–40	–	1,5

1 In geval van gelijkmatig verdeelde belastingen is de k-waarde ten minste een factor 3 kleiner (k/3); zie ook de verhouding E_{stat}/E_{dyn}
2 CBR = California Bearing Ratio

Figure B.2: Approximation of soil properties [CUR/PBV, 2000]

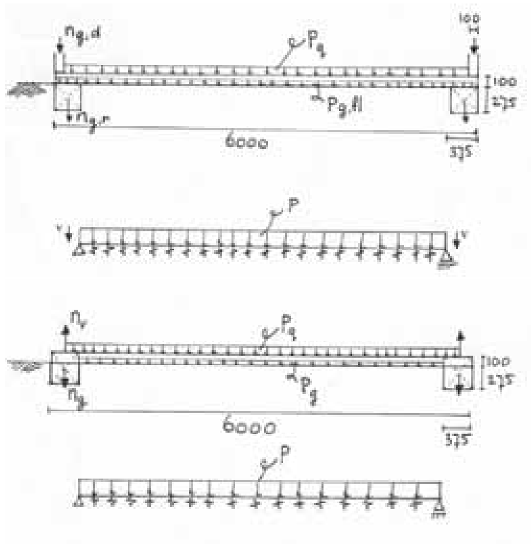

Figure B.3: Load cases A 'after completion of dome' and B 'during dome construction'

The loadcases are grouped into A 'after completion of dome' and B 'during dome construction' and can be found in figure B.3. They have been schematized into circular plates on hinges and supported by springs.

For A:
Load p is a combination of the variable load ($p_{q,A} = 1,75\ kN/m^2$) [2] and the floor's deadweight ($p_{g,fl} = 2,4$).
Load v represents the resulting weight of the ring beam (being approximately 3.4 kN/m) and the weight of the dome shell (being approximately 7 kN/m). An approximation of 10 kN/m is used.

For B:
Load p is a combination of the variable load ($p_{q,B} = 1,5\ kN/m^2$) resulting from airpressure and the floor's deadweight $p_{g,fl} = 2,4$.
Load v is zero as the dead weight of the ring beam and the uplift of the formwork are assumed to be equal.

The following differential equation for plates without shear deformation has been solved using Maple:
$D(\frac{d^4w}{dr^4} + \frac{2}{r}\frac{d^3w}{dr^3} - \frac{1}{r^2}\frac{d^2w}{dr^2} + \frac{1}{r^3}\frac{dw}{dr}) = \frac{p-k\cdot w(r)}{K}$ [Blaauwendraad, 2002]

In case of load case A by introducing initial conditions:
1 the differential of setting (w(r)) is zero in the middle
2 the shear load[3] at the circumference $v = -K(\frac{d^3}{da^3}w(a))$

In case of load case B by introducing initial conditions:
1 the differential of setting (w(r)) is zero in the middle
2 setting at circumference is zero (w(a) = 0)

The following parameters are defined:
- Diameter of the floor is 6 meter
- Floor thickness is 0,1 meter
- Young Modulus of floor is 26000 N/mm^2
- Contraction coefficient $v_{concrete}$ is 0,15
- $k = 0,04\ N/mm^3$
- Stiffness (K=EI) is defined as $\frac{1}{12}\frac{Et^3}{1-v^2}$
- Distributed load = p

With $k_{rr} = -\frac{d^2w}{dr^2}$ and $k_{\theta\theta} = -\frac{1}{r}\frac{dw}{dr}$, the moments can be defined:
$m_{rr} = K \cdot (k_{rr} + v \cdot k_{\theta\theta})$ and $k_{\theta\theta} = K \cdot (k_{\theta\theta} + v \cdot k_{rr})$.

Using Mohr's circle these moments are transferred into orthogonal moments m_{xx}, m_{yy} and m_{xy}. In case of m_{xx} and m_{yy}, reinforcement is needed in two directions. The moment m_{xy} causes the need for some additional reinforcement in both directions. The reinforcement may be calculated on the basis of two moments m_{xx} and m_{yy}, for which the following formulas are valid:
$m_{xx} = m_{xx} + \mid m_{xy} \mid$

[2] Variable floor load for dwellings according to NEN
[3] To be verified, as a differential equation for plates *without shear deformation* is used.

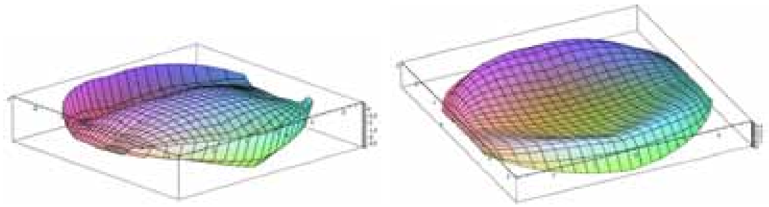

Figure B.4: Results from maple for m_{xx} in case of load case AI and load case BI

$m_{yy} = m_{yy} + \mid m_{xy} \mid$

The following loadcases have been calculated:

Load case	p	*p(kN/m²)*	v	*v(kN/m)*
AI	$0.9 \cdot p_{g,fl}$	2,2	$0.9 \cdot v$	12
AII	$1.2 \cdot p_{g,fl} + 1.5 \cdot p_{q,A}$	5,5	$1.2 \cdot v$	9
BI	$1.2 \cdot p_{g,fl} + 1.5 \cdot p_{q,B}$	5,1	0	0
BII	$1.2 \cdot p_{g,fl} + 1.5 \cdot p_{q,B2}$	8,9	0	0

Load case A has been split in 2 load cases, as p and v are acting in opposite direction.
Load case BII shows results in case an air pressure of $p_{q,B2} = 4\ kN/m^2$.

Resulting in:

Load case	$m_{xx,max}(kN)$	$minA_{re}(mm^2)$	$min\ \%_{re}$
AI	$-2,85$	163	0.16
AII	$-2,15$	123	0.12
BI	$+0,42$	24	0.02
BII	$+0,75$	43	0.04

The square meters of reinforcement needed are calculated assuming the reinforcement is in the middle of the floor and assuming $f_s = 350\ N/mm^2$.

Load case A1 turns out to be normative, even if the airpressure during construction is increased to 4 kN/m^2. The reinforcement should preferably be above the middle of the floorslab.

Appendix C

Handcalculations

C.1 For a dome which base is only constraint in vertical direction

The membrane solution [1] is applicable since the supports are compatible with the membrane stress resultants and they allow deformation in circumferential direction. This results in a stress resultant (in circumferential direction) of $n_{\theta\theta} = p \cdot a$ at the lower edge of the hemisphere (see figure C.1). In case of a 6 meter diameter dome the tension stress in the lower edge amounts to a stress level of $n_\theta = 2400kg/m^3 \cdot 9,8m/s^2 \cdot 3m = 0,071\ N/mm^2$, which corresponds with the results in Ansys.

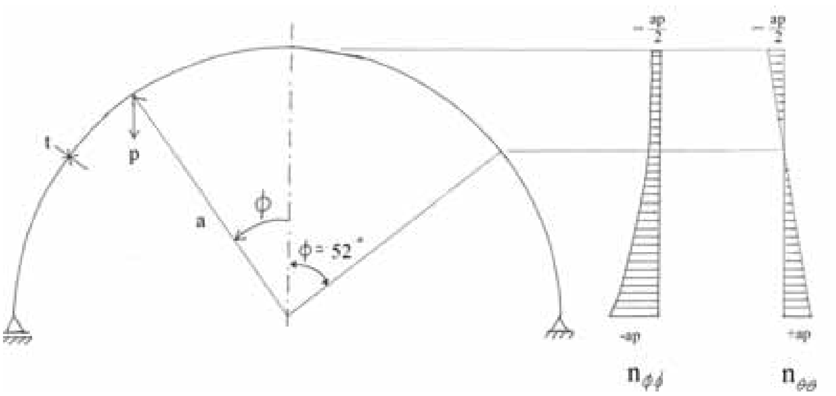

Figure C.1: Distribution of stress resultants over the hemisphere. The distribution of $n_{\theta\theta}$ is almost linear.

[1] Calculated with formula $n_{\phi\phi} = \frac{1}{r \cdot sin\phi} F(\phi)$ and $n_{\theta\theta} = p_z r_2 - \frac{1}{sin^2\phi} \frac{F(\phi)}{r_1}$, see page 146 of [Hoefakker and Blaauwendraad, 2005]

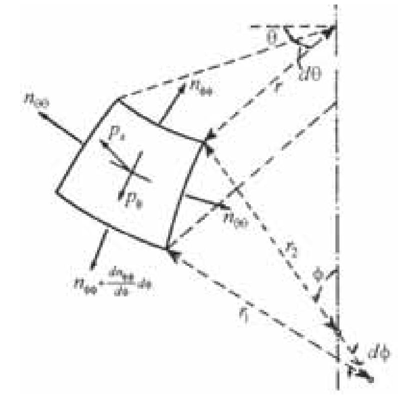

Figure C.2: Load components and stress resultants on an infinitesimal element [Hoefakker and Blaauwendraad, 2005]

C.2 For a dome which base is constraint in all degrees of freedom

Ansys results for a 6 meter diameter dome:
Maximum principal tension stress is $0,035\ N/mm^2$
Maximum reaction force in horizontal plane perpendicular to shell is $-981\ N/m$.
Maximum reaction torsion moment is $171\ Nm/m$

The shell has a diameter of 6 meter (2a) and a wallthickness of 0,1 meter (t).
The ringbeam has a cross-section of 300×300 mm.
$p = 2400kg/m^3 \cdot 10m/s^2 \cdot 0,1m = \ 0,0024N/mm^2$

Membrane state:
Equilibrum equation shell element[2]: $-n_{\phi\phi}r - n_{\theta\theta}rsin\phi + rr_1 = 0$
sphere: $r_1 = r_2 = a$; $r = asin\phi$
$\frac{n_{\phi\phi}}{r_1} + \frac{n_{\theta\theta}}{r_2} = p_z$

$$\begin{cases} p_\phi = psin\phi \\ p_z = -pcos\phi \end{cases}$$

$$\begin{cases} n_{\phi\phi} = -pa(\frac{1}{1+cos\phi}) \\ n_{\theta\theta} = pa(\frac{1}{1+cos\phi} - cos\phi) \end{cases}$$

$$\phi = \frac{\pi}{2} \begin{cases} n_{\phi\phi} = -pa \\ n_{\theta\theta} = pa \end{cases}$$

$$\nu = 0 \begin{cases} \epsilon_{\phi\phi} = \frac{1}{Et}n_{\phi\phi} \\ \epsilon_{\theta\theta} = \frac{1}{Et}n_{\theta\theta} \end{cases}$$

$$\phi = \frac{\pi}{2} \begin{cases} u_r = \epsilon_{\phi\phi}a = \frac{pa^2}{Et} \\ \varphi_\phi = \varphi_x = 0 \end{cases}$$

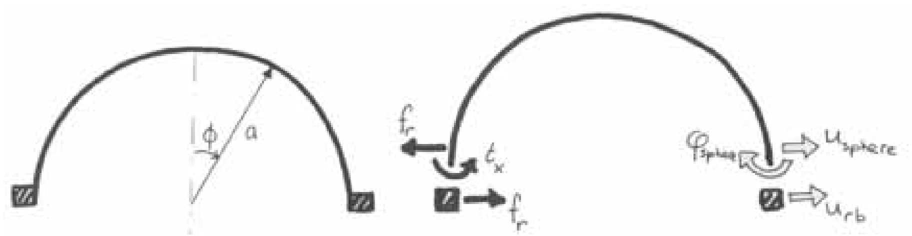

Figure C.3: The ring beam prevents the hemisphere to expand

Edge disturbance:
The ring beam prevents the hemisphere to expand C.3.
Membrane action hemisphere:

$$\overline{\mathbf{u}}_{\mathbf{m}} = \begin{bmatrix} u_r \\ \varphi_\phi \end{bmatrix} = \begin{bmatrix} \frac{pa^2}{Et} \\ 0 \end{bmatrix}$$

The ringbeam receives a force f_r
$N_{\theta\theta} = -f_r a$
$\epsilon_{\theta\theta} = -\frac{a}{EA} f_r$

Displacement ring beam:

$$\overline{\mathbf{u}}_{\mathbf{rb}} = \begin{bmatrix} u_r \\ \varphi_\phi \end{bmatrix} = \begin{bmatrix} -\frac{a^2}{EA} f_r \\ 0 \end{bmatrix}$$

Bending effect spherical shell with $\phi_0 = \frac{\pi}{2}$:

$$\overline{\mathbf{u}}_{\mathbf{b}} = \begin{bmatrix} u_r \\ \varphi_x \end{bmatrix} = \frac{1}{D_b} \begin{bmatrix} \frac{1}{2\mu^3} & -\frac{1}{2\mu^2} \\ -\frac{1}{2\mu^2} & \frac{1}{\mu} \end{bmatrix} \begin{bmatrix} f_r \\ t_x \end{bmatrix}$$

The stiffnesses are ($\nu = 0$): $D_b = \frac{Et^3}{12}$ and $D_m = Et$

Parameter μ with $r_y = a$: $\mu^4 = \frac{D_m}{4D_b a^2} = \frac{3}{(at)^2}$

The displacement of the spherical shell (by membrane and bending action) has to be equal to the displacement of the ring beam.
$\overline{u}_m + \overline{u}_b = \overline{u}_{rb}$

$$\begin{bmatrix} \frac{pa^2}{Et} \\ 0 \end{bmatrix} + \frac{1}{D_b} \begin{bmatrix} \frac{1}{2\mu^3} & -\frac{1}{2\mu^2} \\ -\frac{1}{2\mu^2} & \frac{1}{\mu} \end{bmatrix} \begin{bmatrix} f_r \\ t_x \end{bmatrix} = \begin{bmatrix} -\frac{a^2}{EA} f_r \\ 0 \end{bmatrix}$$

$\varphi_{phi} : -\frac{1}{2\mu^2} f_r + \frac{1}{\mu} t_x = 0,\ t_x = \frac{1}{2\mu} f_r$

[2]See 9.5 [Hoefakker and Blaauwendraad, 2005] and fig C.2

$u_r : \frac{pa^2}{Et} + \frac{1}{2D_b\mu^3} f_r - \frac{1}{2D_b\mu^2}\frac{1}{2\mu} f_r = -\frac{a^2}{EA} f_r$

$-\frac{pa^2}{Et} = f_r(\frac{1}{4D_b\mu^3} + \frac{a^2}{EA})$
$f_r = -(\frac{1}{4D_b\mu^3} + \frac{a^2}{EA})^{-1}\frac{pa^2}{Et}$

$4D_b = \frac{Et}{\mu^4 a^2}$

$f_r = -(\frac{\mu a^2}{Et} + \frac{a^2}{EA})^{-1}\frac{pa^2}{Et} = -(\frac{mu}{t} + \frac{1}{A})^{-1}\frac{p}{t}$

using $\mu = \frac{\sqrt[4]{3}}{\sqrt{at}} = \frac{\sqrt[4]{3}}{\sqrt{3 \cdot 0,1}} = 2,4\ m^{-1}$

$f_r = -(\frac{0,0024}{100} + \frac{1}{90.000})^{-1}\frac{0,0024}{100} = -0,68\ N/mm = -680\ N/m$
Which corresponds roughly with the maximum reaction force of $-981\ N/m$ found in Ansys.

$t_x = \frac{1}{2\mu} f_r = \frac{1}{2 \cdot 0,0024} \cdot -0,68 = -142 Nmm/mm = -142 Nm/m$
Which corresponds roughly with the maximum reaction (torsion) moment of 171 Nm/m found in Ansys.

Stresses due to edge disturbances in the shell wall where the ring beam is present:

Because the hoop strains in the shell edge and the supporting ring are generally different, a boundary disturbance (flexure) has to be expected in the vicinity of the edge. These flexural effects are damped out very fast. For example, the boundary disturbance in a cylindrical shell with radius a and wall thickness t is reduced to below 5% in a distance of about $d = 2,4\sqrt{at}$. Taking into account that in a spherical shell with positive Gaussian curvature the damping of a disturbance is even larger than for a cylinder, it is evident that the additional flexural reinforcement can be limited to a very narrow strip or it may even be disregarded. In the latter case, however, cracks will develop which may not be acceptable under service loads.

$\sigma_{xx}(z) = \frac{12m_{xx}z}{t^3}$
$\tau_{xz}(z) = \frac{3\nu_{xz}}{2t}(1 - \frac{4z^2}{t^2})$

Bending stresses are maximum for $z = \pm\frac{1}{2}t$ $\sigma_{xx}(z) = \pm\frac{6m_{xx}}{t^2} = \pm\frac{6t_x}{t^2} = \pm\frac{6 \cdot 142}{100^2} = \pm 0,085\ N/mm^2$
$\tau_{xz}(z) = \frac{3\nu_{xz}}{2t} = \frac{3f_r}{2t}\frac{3 \cdot -0,68}{2 \cdot 100} = -0,01\ N/mm^2$

Stress due to edge disturbance in the ring beam:

$n_{\theta\theta} = -f_r a$

$\sigma_{\theta\theta} = -\frac{f_r a}{A} = -\frac{0,68 \cdot 3000}{90.000} = 0,023\ N/mm^2$

The maximum tension stress of $0,035\ N/mm^2$ which was found in circumferential direction in the shell in Ansys is of equal magnitude. The tension stress in the ringbeam calculated with the maximum horizontal reaction force results of

Ansys amounts $\frac{3*981}{300^2} = 0,033\ N/mm^2$.

As the results in Ansys are of the same order of magnitude as the results of the handcalculation, the model in Ansys is considered as suitable for further analyses.

C.3 Youngs' module for a ferrocement matrix

With the volume fraction of the mesh V_f and the ratio E_{mortar}/E_f the Youngs' module of the matrix can be defined [Nimityongskul, 1985] [Raisinghani, 1985]. However, as chicken wire consists of wires orientated in different directions, adjustments have to be made. To simplify the calculation it is assumed that the volume fraction of mesh is divided over two instead of three layers, which are orientated perpendicular to each other.

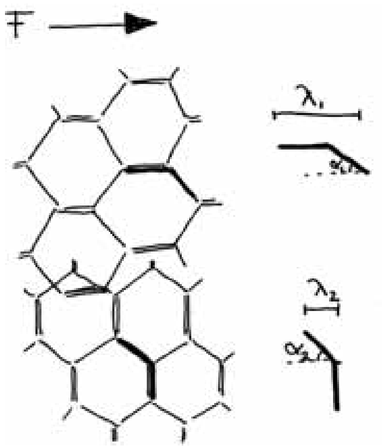

Figure C.4: Orientation of the wire meshes

For each layer the moduli of elasticity E_{c1} and E_{c2} are calculated. The Youngs' module E_c of the matrix can then be determined:

$\frac{1}{E_c} = \frac{\lambda_1}{E_{c1}} + \frac{\lambda_2}{E_{c2}}$

The parameters λ_1 and λ_2 denote the length fractions of the parts of a typical segment, see figure C.4.

The load acting on a composite section per unit area carried by the matrix and N types of fibers oriented at an angle α with the loading direction can be expressed as:

$\sigma_c = \sigma_m A_m + \sum_{i=1}^{N} F_i \sigma_{fi} A_{fi}$

σ_c	average stress in composite section	A_m	area fraction matrix
σ_m	stress in the matrix	A_{fi}	area fraction fiber i
σ_{fi}	stress in the fiber i	F_i	cosine of α

Multiplying this equivalation by the unit length in the direction of the load and noting that $A_{fi}/F_i = V_{fi}$ gives:

$\sigma_c = \sigma_m V_m + \sum_{i=1}^{N} F_i^2 \sigma_{fi} V_{fi}$

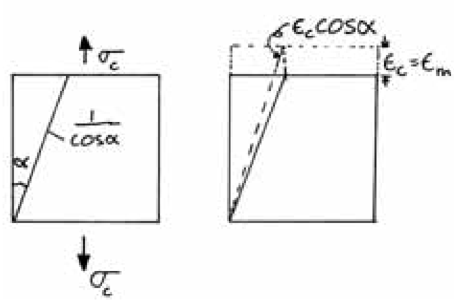

Figure C.5: Strain of the inclined fibers and the matrix

V_m	volume fraction matrix	V_{fi}	volume fraction fiber

The strain in the matrix is equal to the average strain $\epsilon_i$in the composite. In figure C.5 it is illustrated that $\epsilon_i = \frac{\epsilon_c F_i}{1/F_i} = F_i^2 \epsilon_c$

By Hooke's law this becomes:
$E_c = E_m V_m + \sum_{i=1}^{N} F_i^4 E_{fi} V_{fi}$

With the parameters from section 10.2:

E_m	17 300 N/mm^2	E_f	210 000 N/mm^2
V_m	$99,26\%$	V_f	$0,74\%$
α_1	$60°$	α_2	$30°$
F_1	$0,5(1+cos\alpha_1)$	F_2	$0,5cos\alpha_2$
E_{c1}	18 336 N/mm^2	E_{c2}	17 844 N/mm^2
λ_1	$0,63$	λ_2	$0,37$

this results in $E_c = 18\ 023\ N/mm^2$

Appendix D

Design by Denktank Solid House Foundation

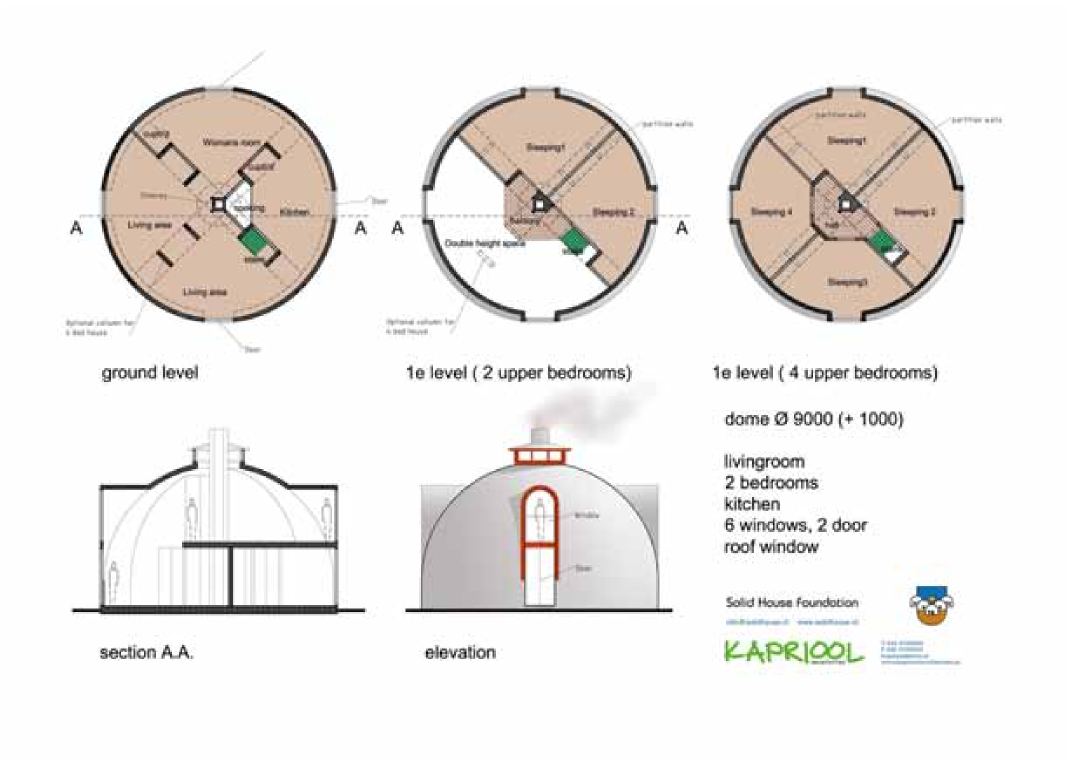

Figure D.1: Design for a dome in Sri Lanka by the SHF's architect 'Denktank'

Appendix E

Building Costs Dome Sri Lanka

INSPECTOR EATHTHAME REHABILITATION PROJECT

9 meter Dome (Model House) Actual Cost (in LKR)

#	DESCRIPTION	Item	Unit	Quantity	Rate	Amount	Total Amount
1	**Site clearing**	Skilled Labour	days	1	1000,00	1000,00	4025,00
		Unskilled labour	days	4	600,00	2400,00	
		Food expenses	nos	5	125,00	625,00	
2	**Excavation Foundation**	Skilled Labour	days	1	1000,00	1000,00	6925,00
		Unskilled labour	days	8	600,00	4800,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	9	125,00	1125,00	
3	**Rebar Binding (foundation)**	10mm steel	bars	172	225,00	38700,00	54950,00
		Binding	Kg	6	125,00	750,00	
		Polyttheen Sheet	meter	30	100,00	3000,00	
		Cookies	Nos	200	5,00	1000,00	
		Transport	trips	1	4500,00	4500,00	
		Skilled Labour	days	3	1000,00	3000,00	
		Unskilled labour	days	5	600,00	3000,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	8	125,00	1000,00	
4	**Concreting of the foundation**	Cement	bags	56	625,00	35000,00	144340,00
		Metal	cubs	6	10500,00	63000,00	
		Sand	cubs	6	2000,00	12000,00	
		Water	liters	4000	0,75	3000,00	
		Transport	trips	1	4500,00	4500,00	
		Deasel	liters	20	62,00	1240,00	
		Skilled Labour	days	4	1000,00	4000,00	
		Unskilled labour	days	28	600,00	16800,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	32	150,00	4800,00	
5	**Attach in balloon for the foundation**	Iron plates	Nos	130	125,00	16250,00	40945,00
		Nut & bolt	Nos	410	35,00	14350,00	
		Installing Generator				1000,00	
		Installing Ventilator				1000,00	
		Deasel	liters	10	62,00	620,00	
		Skilled Labour	days	3	1000,00	3000,00	
		Unskilled labour	days	6	600,00	3600,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	9	125,00	1125,00	
#	**DESCRIPTION**	**Item**	**Unit**	**Quantity**	**Rate**	**Amount**	**Total Amount**
6	**Scaffolding**	GI Pipes (1 1/2)	length	36	2150,00	77400,00	145675,00
		Timber 4"x2"	l.ft	510	40,00	20400,00	
		Timber 2"x1"	l.ft	250	10,00	2500,00	
		G.I.binding	Nos	75	300,00	22500,00	
		Planks	Sq.ft	100	40,00	4000,00	
		Transport	trips	2	4500,00	9000,00	
		Nails	Kg	5	125,00	625,00	
		Skilled Labour	days	5	1000,00	5000,00	
		Unskilled labour	days	5	600,00	3000,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	10	125,00	1250,00	
7	**Dome Reinforcement**	10mm steel	bars	320	225,00	72000,00	164325,00
		Binding	Kg	15	125,00	1875,00	
		Cookies	Nos	500	5,00	2500,00	
		Transport	trips	1	4500,00	4500,00	
		Deasel	liters	100	62,00	6200,00	
		Skilled Labour	days	30	1000,00	30000,00	
		Unskilled labour	days	60	600,00	36000,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	90	125,00	11250,00	
8	**Door & Window form works** **(4 Nos Doors & 2 Nos Windows)** **and lower shuttering**	Plywood 1/4	sheets	3	700,00	2100,00	105675,00
		Plywood 1/2	sheets	18	1300,00	23400,00	
		Plywood 3/4	sheets	12	1900,00	22800,00	
		Timber 4"x2"	l.ft	300	40,00	12000,00	
		Reapers	l.ft	100	10,00	1000,00	
		Transport	trips	1	4500,00	4500,00	
		Nails	Kg	5	125,00	625,00	
		Skilled (carpenter)	days	20	1000,00	20000,00	
		Unskilled labour	days	26	600,00	15600,00	
		Food expenses	days	46	125,00	5750,00	

#	DESCRIPTION	Item	Unit	Quantity	Rate	Amount	Total Amount
9	**Concreting of the dome**	Cement	bags	110	625,00	68750,00	267890,00
		Metal	cubs	5	10000,00	50000,00	
		Sand	cubs	6	2000,00	12000,00	
		Water	liters	4000	0,75	3000,00	
		Transport	trips	1	4500,00	4500,00	
		Deasel	liters	220	62,00	13640,00	
		Skilled Labour	days	20	1000,00	20000,00	
		Unskilled labour	days	120	600,00	72000,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	160	150,00	24000,00	
#	**DESCRIPTION**	**Item**	**Unit**	**Quantity**	**Rate**	**Amount**	**Total Amount**
10	**Curring**	Gunny bags	bags	200	125,00	25000,00	43250,00
		Water	liters	15000	0,75	11250,00	
		Transport	trips	0,5	4500,00	2250,00	
		Skilled Labour	days	1	1000,00	1000,00	
		Unskilled labour	days	5	600,00	3000,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	6	125,00	750,00	
11	**Inside plaster**	Cement	bags	18	625,00	11250,00	65125,00
		Sand	cubs	2	2000,00	4000,00	
		Lime	bags	25	200,00	5000,00	
		Water	liters	1500	0,75	1125,00	
		Transport	trips	0,5	4500,00	2250,00	
		Skilled Labour	days	24	1000,00	24000,00	
		Unskilled labour	days	20	600,00	12000,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	44	125,00	5500,00	
12	**Out side plaster**	Cement	bags	34	625,00	21250,00	58350,00
		Sand	cubs	1,5	2000,00	3000,00	
		Water	liters	3000	0,75	2250,00	
		Transport	trips	0,5	4500,00	2250,00	
		Skilled Labour	days	16	1000,00	16000,00	
		Unskilled labour	days	16	600,00	9600,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	32	125,00	4000,00	
13	**Out side painting**	water profing paint	L	24	175,00	4200,00	27400,00
		Wethercoat paint	L	20	450,00	9000,00	
		Piller paint	L	20	200,00	4000,00	
		Transport	trips	0,25	4500,00	1125,00	
		Skilled Labour	days	0	1000,00	0,00	
		Unskilled labour	days	12	600,00	7200,00	
		COM. Contribution	days	0	0,00	0,00	
		Food expenses	item	15	125,00	1875,00	
#	**DESCRIPTION**	**Item**	**Unit**	**Quantity**	**Rate**	**Amount**	**Total Amount**
14	**Divisioning wall**	Cement	bags	20	625,00	12500,00	69625,00
		Sand	Cubs	2	2000,00	4000,00	
		Water	L	1000	0,75	750,00	
		Bricks	Nos	3000	7,00	21000,00	
		Skilled	days	15	1000,00	15000,00	
		Un Skilled	days	20	600,00	12000,00	
		Food expenses	Items	35	125,00	4375,00	
15	**Wall plaster**	Cement	bags	8	625,00	5000,00	39275,00
		Sand	Cubs	1	2.000,00	2000,00	
		Lime	bags	12	200,00	2400,00	
		Transport	trips	1	4.500,00	4500,00	
		Skilled	days	12	1.000,00	12000,00	
		Un Skilled	days	15	600,00	9000,00	
		Food expenses	Items	35	125,00	4375,00	
16	**Gutter with pavement**	Cement	bags	8	625,00	5.000,00	24.100,00
		Sand	cubs	1	2.000,00	2.000,00	
		Water	L	2000	0,75	1.500,00	
		Transport	trips	1	4.500,00	4.500,00	
		Skilled	days	6	1.000,00	6.000,00	
		Un Skilled	days	6	600,00	3.600,00	
		Food expenses	Item	12	125,00	1.500,00	

17	**Pantry Cupbord with Slab**	Cement	bags	3	625,00	1.875,00	21.875,00
		Sand	cubs	1	2.000,00	2.000,00	
		Metal 3/4	cubs	0,5	10.500,00	5.250,00	
		10mm steel	Nos	19	225,00	4.275,00	
		water	L	1000	0,75	750,00	
		Skilled	days	3	1.000,00	3.000,00	
		Un Skilled	days	6	600,00	3.600,00	
		Food expenses	Item	9	125,00	1.125,00	
#	**DESCRIPTION**	**Item**	**Unit**	**Quantity**	**Rate**	**Amount**	**Total Amount**
18	**Doors and Windows**	Door Frame (Concrete)	No	3	3.250,00	9.750,00	128.525,00
		Door Slashes	No	5	10.000,00	50.000,00	
		Window Frame (Concrete)	No	4	2.950,00	11.800,00	
		Window Slashes	No	8	3.500,00	28.000,00	
		Transport	trips	2	4.500,00	9.000,00	
		Door Frame (Timber)	No	2	5.000,00	10.000,00	
		Carpenter	days	2	1.000,00	2.000,00	
		Skill	days	3	1.000,00	3.000,00	
		Un Skill	days	6	600,00	3.600,00	
		Food expenses	Item	11	125,00	1.375,00	
19	**Door and window plaster**	Cement	bags	5	625,00	3.125,00	25.725,00
		Sand	cubs	1	2.000,00	2.000,00	
		Skill	days	8	1.000,00	8.000,00	
		Un Skill	days	16	600,00	9.600,00	
		Food expenses	Item	24	125,00	3.000,00	
20	**Inside Painting**	Emultion Paint	liters	20	450,00	9.000,00	25.800,00
		Enamal Paint	liters	4	700,00	2.800,00	
		Skill	days	6	1.000,00	6.000,00	
		Un Skill	days	10	600,00	6.000,00	
		Food expenses	Item	16	125,00	2.000,00	
21	**Floor Rendering**	Cement	bags	5	625,00	3.125,00	23.675,00
		Sand	cubs	1	2.000,00	2.000,00	
		Red Cement	Kg	10	525,00	5.250,00	
		Water	L	1000	0,75	750,00	
		Skill	days	6	1.000,00	6.000,00	
		Un Skill	days	8	600,00	4.800,00	
		Food expenses	Item	14	125,00	1.750,00	

Total in LKR	1.487.475
Total in EURO	11.900

Appendix F

About Membranes

Before starting new designs for the formwork a study was made on membranes. In this section a number of aspects will be discussed.

F.1 Mechanics of single membranes under pressure

The stresses in a double curved single membrane depend on the pressure and the curvature of the membrane.

These stresses can be determined by the equilibrium of a small square element with angles of curvature δ_θ and δ_φ, see page 242.

Vertical equilibrium:
$2n_\theta r_2 \delta_\varphi sin(\frac{\delta_\theta}{2}) + 2n_\varphi r_1 \delta_\theta sin(\frac{\delta_\varphi}{2}) = pr_1 \delta_\theta r_2 \delta_\varphi$

For small angles $sin(\frac{\delta_\theta}{2}) = \frac{\delta_\theta}{2}$ and $sin(\frac{\delta_\varphi}{2}) = \frac{\delta_\varphi}{2}$.

Which renders:
$n_\theta r_2 \delta_\varphi \delta_\theta + n_\varphi r_1 \delta_\theta \delta_\varphi = pr_1 r_2 \delta_\theta \delta_\varphi$

Simplified: $\frac{n_\theta}{r_1} + \frac{n_\varphi}{r_2} = p$

r_1 and r_2 are the radiuses of the two positive curvatures
n_θ and n_φ are the membrane forces in these directions
The membrane force n = membrane stress $\times$ thickness of the membrane
The difference in stresses between the upper and lower level of the membrane is neglected because of the small thickness.

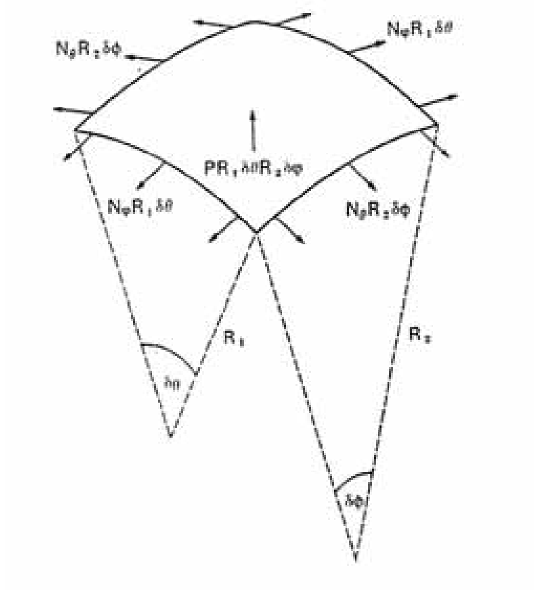

Figure F.1: Stresses in a small double curved square element under pressure [Huybers, 1999]

F.2 Materials

Many different materials can be used to make a membrane. A rough distinction can be made between isotropic and anisotropic materials [Braam, 2000]:

Isotropic Materials

- Plastic foils made of i.e. PVC, polyethyleen, polyester, polyvinylchlorid or synthetic rubbers. These foils are very airtight, flexible and easy to weld or glue.
- Rubber membranes are very flexible because of the large elongations possible. Rubber is airtight, but the strength of rubber is very low.
- Metal foils such as aluminium and steel foil are very airtight and have a high tensile strength. A big disadvantage is the stiffness which requires extremely precise cutting patterns.

Anisotropic Materials

- Tissues consist of threads which are woven in a specific pattern. The directions in which the threads are woven are called warp and weft. In the direction of the warp the tissue is strongest as the threads of the weft are woven through stretched threads of the warp. There are a lot of differ

Isotropic foils and synthetic rubbers are not suitable for a membrane because of their low Young's Module and sensit

Therefore membranes for structural applications mostly consist of coated tissue. The tissue absorbs stresses on the membrane while the coating makes the membrane airtight and increases it's durability. Polyester tissues with PVC coatings are most common because of their good properties and relatively low price. The current inflatable formwork of the Solid House Foundation consists of PVC coated polyester and has a weight of 900 gr/m^2

F.3 Seams

The seams have to be stronger than the membrane itself, as airtight as possible, durable and flexible. In general most seams are either stitched, welded or glued [1].

All tissues can be stitched. It is more expensive, but stronger than welded seams. However stitched seams are not airtight and dirt accumulates along the seams.

In case of welded seams, the connection is made by joining the coating. There are several techniques [2], depending on the kind of coating used. The level of airtightness depends on the technique. Most common is point-welding. The created seams are not airtight, so a continuous inflow of air is necessary to keep pressures on the required level. Instead of increasing the airtightness of the seams itself the membrane could also be filled with an over sized rubber membrane (a sort of inner tire). The rubber membrane will take care of the airtightness, while the polyester membrane provides strength.

F.4 Airpressure

Air pressure can be supplied by a ventilator, a blower or a compressor.

Ventilators have a low pressure capacity but a high volume capacity. Centrifugal ventilators are most suitable as they require most energy for high volumes with low pressure levels. The maximum air pressure a centrifugal ventilator can provide amounts approximately 300 mbar.

Blowers can handle higher pressures than ventilators. Some can create up to 2 bar overpressure. However volume capacity is lower than for ventilators.

Compressors are especially suitable for very high pressures as they can provide up to 15 bar overpressure. Yet their volume capacity is very small, which makes them unsuitable for large constructions.

As a comparison: a bicycle pump has a very small volume capacity, but can provide pressures up to 9 bar. A tire for a racing bike needs about 8 bar over-

[1] Buitink Zeilmakerij

[2] High frequency welding, point-welding and hot-air-welding

pressure.

Appendix G

Construction Manual Traditional Balloon with Ferrocement Shell