ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

SAFEGUARDING CONDITION OF THE ROCK-HEWN CHURCHES OF LALIBELA

Case Study:- Wind Load And Structural Analysis On Free-Standing Canopy Structures Shading Five of Rock-Hewn Churches of Lalibela and Numerical Analysis On The Underground Tunnel

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A Thesis in Structural Engineering

By Naol Gebeyehu Duguma

December,2017

Addis Ababa, Ethiopia

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CASE STUDY:- WIND LOAD AND STRUCTURAL ANALYSIS ON FREE-STANDING CANOPY STRUCTURES SHADING FIVE OF ROCK-HEWN CHURCHES OF LALIBELA AND NUMERICAL ANALYSIS ON THE UNDERGROUND TUNNEL

A Thesis
Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

By
Naol Gebeyehu Duguma

Thesis Advisor
Dr. Esayas Gebreyouhannes

December,2017
Addis Ababa, Ethiopia

The undersigned have examined the thesis entitled '**Safeguarding Condition of the Rock-Hewn Churches of Lalibela**' presented by **Naol Gebeyehu Duguma**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I certify that research work titled "Safe Guarding Condition of the Rock-Hewn Churches of Lalibela" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources, it has been properly acknowledged / referred.

Naol Gebeyehu

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ABSTRACT

This research focuses on the wind and structural analysis of the steel canopy structures shading five of the eleven monumental churches, Bete-Mariam, Bete-Mesquel, Bete-Medhaniealem, Bete-Abalibanos and Bete-Amanueal, found in Lalibela. This world-renowned site with the rock-hewn monuments is one of the important heritage sites in Ethiopia, which is registered and protected by the United Nations Educational, Scientific and Cultural Organization (UNESCO) since 1978. Currently, the shades sheltering the five rock-hewn churches are showing visible signs of failure at the base of the columns, which is one of the main reason for doing this research. A numerical analysis is performed on an underground tunnel, on which one of the columns of the shade sheltering Bete-Amanueal is directly placed.

The current shelters are implemented with a design change for a reason that UNESCO found the previous design too permanent. However following the design change there is no investigation carried out to properly identify the characteristic behavior of the subsurface condition. These, coupled with the conjecture of the uplift wind loading creating instability, has created the fear that the failure of the canopy shelters may result in extensive damage or even total failure of the rock-hewn churches.

In this study, the structural safety of the steel canopy structures is investigated based on the raw wind data collected from the National Metrology of Ethiopia for the Lalibela area. The design basic fundamental wind speed is decided based on the probability distribution, which best fits with the available data. The wind load analysis performed on the structure clearly shows that the uplift wind is critical and is likely to cause further structural damage to the canopy structures. Based on this finding, long term and alternative short-term mitigation measures are discussed considering constraints, such as simplicity, non-labor intensiveness and compliance with the UNESCO regulations specified for the shelter project. The rock mass has also been studied and classified using one of the multi-parametric rock mass classification methods, the Rock Mass Rating (RMR), to quantify the in-situ condition of the tunnel.

Key words: - Rock-hewn church, Lalibela Monuments, Wind Loading, Canopy Roof, Steel shelters, Underground Tunnel/Cavity, Numerical Analysis.

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LIST OF SYMBOLS

R = Correlation factor

v_b = Basic wind velocity

$v_{b,0}$ = Fundamental value of basic wind velocity

C_{dir}= Directional factor

C_{season} = Seasonal factor

q_b= Basic wind pressure

ρ = Density of air

$v_m(z)$ = Mean wind velocity at reference height Z

Z = Reference height

$C_r\ (z)$ = Roughness factor

$C_o(z)$ = Orographic factor

Kr = Terrain factor

Z_0 = Roughness length

Z_{min} = Minimum height for the corresponding terrain category

$Z_{0,II}$= Roughness length for terrain category II

Φ = Upstream slope in the wind direction

S = Orographic location factor

$v_p(z)$ = Guest velocity (or peak velocity)

G = Guest actor

$I_v\ (z)$ = Wind turbulence intensity at height z

k_1 = Turbulence factor

$\sigma_v(z)$ = rms or variance

q_p = Peak velocity pressure

w_p = Wind pressure on canopy roofs

FW = Wind force on canopy roofs

$C_{p,net}$ = Net pressure coefficient for canopy roofs

$C_s\ C_d$ = Structural factors

C_f = Force coefficient

A_{ref} = Reference area

φ = Degree of blockage under a canopy roof

μ = Permeability of a skin

Comma (,) means multiplication

Dot (.) means decimal division

CHAPTER 1:- INTRODUCTION

1.1. Background of the study

Lalibela is located in the Semien Wollo Zone of the Amhara regional government in the northern-central part of Ethiopia, at roughly 2,500 meters above sea level, approximately 600 km north of Addis Ababa in Northern Wollo. It is the capital town of Lasta woreda, which was formerly part of Bugna woreda [2]. The region is well known for its monolithic churches which still is the place for religious activities.

A monolithic rock hewn church is a church made from a single block of stone [3]. The medieval monolithic churches of the 12th century 'new Jerusalem' are situated in mountainous region in the heart of Ethiopia near a traditional village. Lalibela is an important center of Ethiopian Christianity and even today is a place of pilgrimage and devotion.

The name of the town and the church as well named after the king Lalibela replacing the former name "Roha'', whom believed to construct the church in the late 12th century.

There are eleven magnificent churches carved out of a rock having a title of the "eighth wonder of the ancient world". These churches are grouped in to three categories according to their locations, the Northern groups; (Bete-Medhaniealem (Church of the Holy Savior), Bete Mariam (Church of St. Mary), Bete-Mesqel (Church of the Cross), Bete Denagel (Church of the Virgins), Bete- Debre Sina (Church of Mount Sinai) and Bete Golgotha (Church of Golgotha). Eastern groups; Bete Gabriel-Rufael (Church of St. Gabriel–Rafael), Bete-Amannueal (Church of Emmanuel), Bete-Merqourios (Church of St. Mark) and Bete Abba Libanos (Church of Father Libanos) the third group found in the Western direction is represented by the isolated church of Bete Giorgis (Church of St. George). These churches are connected together by underground tunnels or channels.

In 1978 the General Assembly of the United Nations, at the urging of UNESCO, charged with the protection of world heritage sites. Among the first sites on the World Heritage List, fulfilling's the first three criterion, were the rock-hewn churches of Lalibela in Ethiopia [4].

This ancient, magnificent rock cut monument which chiseled out from a single rock is built when the science of civil engineering profession doesn't even exist. The amazing thing about these structures is not only their construction but also their resistance to different types of engineering loads; supporting thick roof and lateral loads such as wind and seismic loads without acquiring reinforcement bars. With the homogeneity of the rock, it is expected to resist both the compression and tension forces from different exerting load conditions. But through time these churches started to deteriorate which needs a serious attention and preservation activities.

These world's treasure are believed to be in danger by a lot of trustworthy reasons mentioned by different scholars, the community and the UNESCO. To mention some of the threats; "the drainage ditches were filled by earth for several centuries, before being cleared in the 20th century, and have been disrupted by seismic activity. This has resulted in a severe degradation of the monuments from damage, and most of them are now considered to be in a critical condition. Other threats include encroachment on the environment of the churches by new public and private construction, housing associated with the traditional village adjacent to the property and infrastructure of tourism [2]". The other threat was mentioned as "increasingly falling under the lustful eye of the tourism, Tigrayan and especially Lalibelas churches are entering a period where heavy pilgrimage and visitor numbers might start to pose a threat to these monuments, the danger of erosion is starting to lurk around the corner [5]". Rain, wind and sun also takes the major role on the degradation of the monuments, and also lichens aggravated the deterioration too.

The developing infrastructure like road ways, making things easy for access and construction for tourists to stay on near to the sculptures would obviously help to endure the tourism industry problems but at the same time it is dangerous for the sculptures, the vibration from the repetitive movements of people as well as vehicles damage the peace stayed rock-hewn churches of Lalibela. As a result the capacity of the structure has been and will be decreased from time to time and will cause loosing this monuments in the earliest time. Since ceasing such tourist attraction spot will be a great loss for the economy of the country, assessing the potential failure causes of these structures and finding a preventive mechanism would be the only way to protect and allow them to stand more long age.
The report paper of Prof. Claudio Margottini classifies the damage level of the monuments referring the research done by Burland et.al (1975), the classification starts from grade 1 to grade 5 depending on the damage level. The grade 1 classification is for the most preserved one and grade 5 is for the monument found in the worst condition. The damage classification is summarized in below;

Bete-Medhaniealem (Grade 4)

The roof is completely altered, as consequence of rainfall erosion, degradation and bad restoration with epoxidic resins, removed by the restorer Angelini in 1965-1968.

Bete-Mariam (Grade 2)

Largely restored to protect from the weathering processes. Presently the small entrance on the west and the two north and south facades are suffering for degradation.

Bete-Mesquel (Grade 2)

It is hewn in very good tuff. Only one important discontinuities (about 2mm wide) has been detected and minor discontinuities are also present.

Bete-Denagel (Grade 3)

It exhibit an important discontinuity cutting the entire structure, and dipping toward the façade, kinematic analysis confirm that the façade is almost at the condition of sliding.

Bete-Debra Sina and Bete-Golgotha (Grade 3)

A major crack, cutting the statue of S. Peter and a similar meter lower, cutting a back statue produce a major attention. The North wall of Bete-Mikael-Golgotha is completely weathered and statues are irreparably damaged. Water flowing from the upper level should be avoided to prevent surface tuff corrosion.

Bete-Amanueal (Grade 5)

It is probably the monument in the worst condition. The static weight of the roof, the carrying capacity of walls and weathering process are all together acting negatively for future conservation. A long fracture about 10 cm wide is crossing the whole church. Almost all arches in both longitudinal and transversal direction have cracks in the center.

Bete-Merqourios (Grade 4)

The edifice is composed by chambers supported by pillars opening onto a longitudinal courtyard of irregular shape. The roof of the church is partially collapsed and also recently new small portion went down. The remaining roof is completely weathered and water is flowing inside, especially in the western wall. Some of the outside columns present weathering phenomena in the lower part, while an internal pillar is affected by a wide sub horizontal crack.

Bete-Abalibanos (Grade 4)

This was a monolithic church anchored to the rock out of which it was carved. This church is affected by two phenomena: alveolization in the lower part of the edifice and the presence of a geological discontinuity.

Bete-Gabriel-Rufael (Grade 3)

This church is a very impressive structure, with different architectural style with respect to others. In this area there are a major weathering process especially in the two courtyard that surround the edifice South and North. One large horizontal crack is crossing the church. Other fissures are inside, one of those 10mm wide. Also the roof exhibits a vertical crack, as well as one of the internal pillar.

Bete-Giorgis (Grade 1)

General good conditions. Small minor cracks on external walls. The major identifies problem is the weathering of the south wall as consequence of water falling on the roof, channeled, concentrated through a short gutter and from this going inside the church.

The above damage levels are sorted out for the purpose of prioritization of a restoration activity that can be done for the churches.

Different kinds of restoration activities has been done at different times, the first restoration was in the mid 1950's by the Italian Sebastiano Console handled by the Ministry of public works. Console proceeded to cover many of the churches with tar, painted a reddish color in an attempt to match the color of the natural rock. Along with tar hundreds of corrugated iron sheets were bolted to the roofs of the churches to protect them from further damage due to rain and sun. But the tar has made the rock not to breath and the bolts made holes which leads to further damage [4].

The second restoration was done on 1966 following the request of crown prince Merid Assefa to prof. John Brew, the chairman of UNESCO's committee on monuments, the restoration work has been done under the supervision of Dr. Sandro Angelini, the director of the Archeological Museum of Bergamo in Italy. The primary goal of the restoration were to safeguard the churches from further deterioration and to restore "where aesthetically permitted, the monolithic form and character of churches" something that had been damaged by the previous restoration activities. The tar was removed and replaced by neutral cement mixed with crushed local stones for color was used to fill in the cracks. Rebar were also installed to stabilize the shifting rocks. The metal roof was removed and replaced by heavy layer of cement along with a water repellent solution [4]. This restoration activity has protected the deterioration a little but not good enough to minimize the further degradations because the interaction of the cement with the rocks was not studied [6].

In 1989, five of the churches were enclosed in temporary shelters, built by the ministry of culture, to protect the churches from the rain which continued to threaten the paintings inside and the rock itself. In 1995, at the request of the Ethiopian government, UNESCO began work on a new project to protect and conserve the churches. A project identification study was carried out, and preparation for an international design competition was begun [4].

In 2002, UNESCO, has implemented the construction of four new metal structures that will cover and protect five of the churches from the corrosive effects of rain and sun.

UNESCO along the international union of Architects (UIA) held a design competition for new shelters intended to replace the existing ones protecting the churches at Lalibela. The idea behind the design was to create shelters which were simple, unobtrusive and easy to assemble and dissemble. According to UNESCO, the shelters had to be easily removed once conservation of the churches had concluded (international design competition). Of the eight European architectural firms to be recognized by the jury, the Italian company TEPRIN ASSOCIATION was unanimously chosen for their design which the jury described as an “elegant, unobtrusive and easily comprehensive project.” [4].

The original design was simple; large metal columns would use cables to hold up a translucent covering the churches. This covering would allow light to come through, but protect from sun and rain [4].

These structures which are due to be completed by late February, 2007, have created controversy amongst the local community and architectural experts, for design changes and problems with construction that have occurred [4].

Issues has raised on the shelters construction, one of the biggest issues facing the construction of the shelters is the change in design that has occurred. Originally the support columns for the shelter were to be anchored outside the complex of the churches; this was done to limit the visibility of the shelter from inside the site. UNESCO, however, found this original plan too permanent. The foundation of the columns would have to be anchored into the rock, requiring excavation. This excavation would leave permanent holes in the ground which would conflict with the temporary nature of the project. Ironically these 'temporary' shelters are expected to be in place for the next twenty years, leading to question about the reality of UNESCO's original plan [4].

The new design places the support columns in the actual church complex. This change in design has sparked immense controversy among local residents, who see the columns an obtrusive to the churches.

Concerns have also arisen about the stability of the columns themselves, which now are simply resting on to bedrock. Design changes have already had to occur at Beta-Amannueal, because of the risk of high winds, and in an area that experiences unpredictable weather, these winds could threaten the stability of the shelters [4].

Besides the environmental loads that are acting on the Rock-hewn churches the so provided temporary shelters has also become a great threats. One is that the structural columns stability is doubtful since they are not anchored in the ground which they counter balance the load only by their weights that the uplift wind pressure could lift up the steel structure causing a structural failure. So that it brings the total loose of the monuments and the other is the heavy steel structure is laid on the ground around the monumental complex which the tunnel is directly under it, so it might also late disruption of the tunnels access by destroying them and could decrease the effective bearing capacity of the around soil stratum too.

1.2. Statement of the problem

According to the researchers' observation, this temporary shelters become a threat to the monuments despite their intended purpose, and the researcher believes that this are the critical safe guarding condition for the monuments.

And as we know Ethiopia has become one of the world best tourist destination of the year by 2015. Therefore [7], in order to remain in this industry, even to grow, protecting the major tourist attraction places such as Lalibela is undoubtedly necessary measure.

Currently the stability of the shelters are become doubtful against uplift wind pressure and perhaps their load affects the stability of the underground tunnels beneath the shelters. Which the researcher is interested about and initiated to check and give an appropriate solution.

1.3. Objective

1.3.1. General objective

The core objective of this research is to check if the monuments are safe from the provided temporary steel shelters.

1.3.2. Specific objectives

1. To check if the shelters are safe against uplift wind pressure with the new experienced wind speed using engineering assessments and if so figuring out the reason why it doesn't fail already.
2. To check if the unfavorable vertical loads from the shelter are the threats for the under tunnels/cavities.
3. Providing short term and long term solutions if the above items are proved to be the threats.

1.3.3. Research questions

1. Does the shelters are contributing their intended purpose?
2. Will the shelters really be the threats on the monuments?
3. How does a civil engineering research could help the tourism industry by preventing ancient structural monuments?

1.4. Significance of the Study

The research will have significances particularly for the structures of Lalibela rock-hewn churches by engaging civil engineering profession case studies, analysis and then giving solutions (short term and long term). Which indirectly also have significance on the tourism industry by assuring the long term income of cashes from the tourism and also on preserving the long lived history.

This research paper would also help other researchers as an input, source, guideline or reference for further and upcoming researches.

1.5. Methodology and Approach

This research is done on basis of the temporary shelters provided on the rock-hewn churches of Lalibela. Structural analysis on the shelters and numerical analysis on underground tunnels is assessed and the research will incite the method of improvements and failure mitigations. To achieve all this goals the researcher will use different kinds of methods, procedures and materials. And it is aimed to follow the following methodologies and procedures.

A. Essential data's are going to be collected

I. Primary data

- Interview
- Site visit

II. Secondary data

- Papers on Lalibela rock-hewn churches will be assessed
- Literatures and national codes on how to analyze canopy roof systems against wind pressure and underground tunnel analysis using solid mechanics system.
- Papers and books on how to analyze underground tunnels.

B. The collected data's are going to be analyzed

III. Analyzing the temporary steel shelters

- Modeling the shelters using SAP2000V16.
- Manual computation of the wind loading on the canopy shelters, on the roof (of canopy roof system) and side covers
- The analysis output obtained from step 2 will be loaded on the model already made on step 1
- Checking if the model and loadings are correct
- Analyzing the model
- Reading the analysis output especially of the magnitude and direction of the reaction force on base foundation, using both the design and the already experienced wind speeds on Lalibeal region.

- Making a discussion on the result and giving conclusion on the obtained results.

IV. Numerical Analysis on the underground tunnel

- The rock mass quality of the underground tunnels will be classified using one of the rock mass classification method, which helps us to know the initial condition of the rock mass.
- The ground surface will be simulated and modeled on a finite software (DuCOM, COM3D) having an assumption of solid mechanics.
- The reaction force obtained from step (I) with the unfavorable loading condition will be loaded in this model and analysis will be done.
- Checking out the displacement on the top of the roof and on the ground surface and also checking if crack is encountered.
- Making a conclusion having the results obtained above.

C. Providing solutions depending on the results obtained from I and II.

1.6. Location of the study area

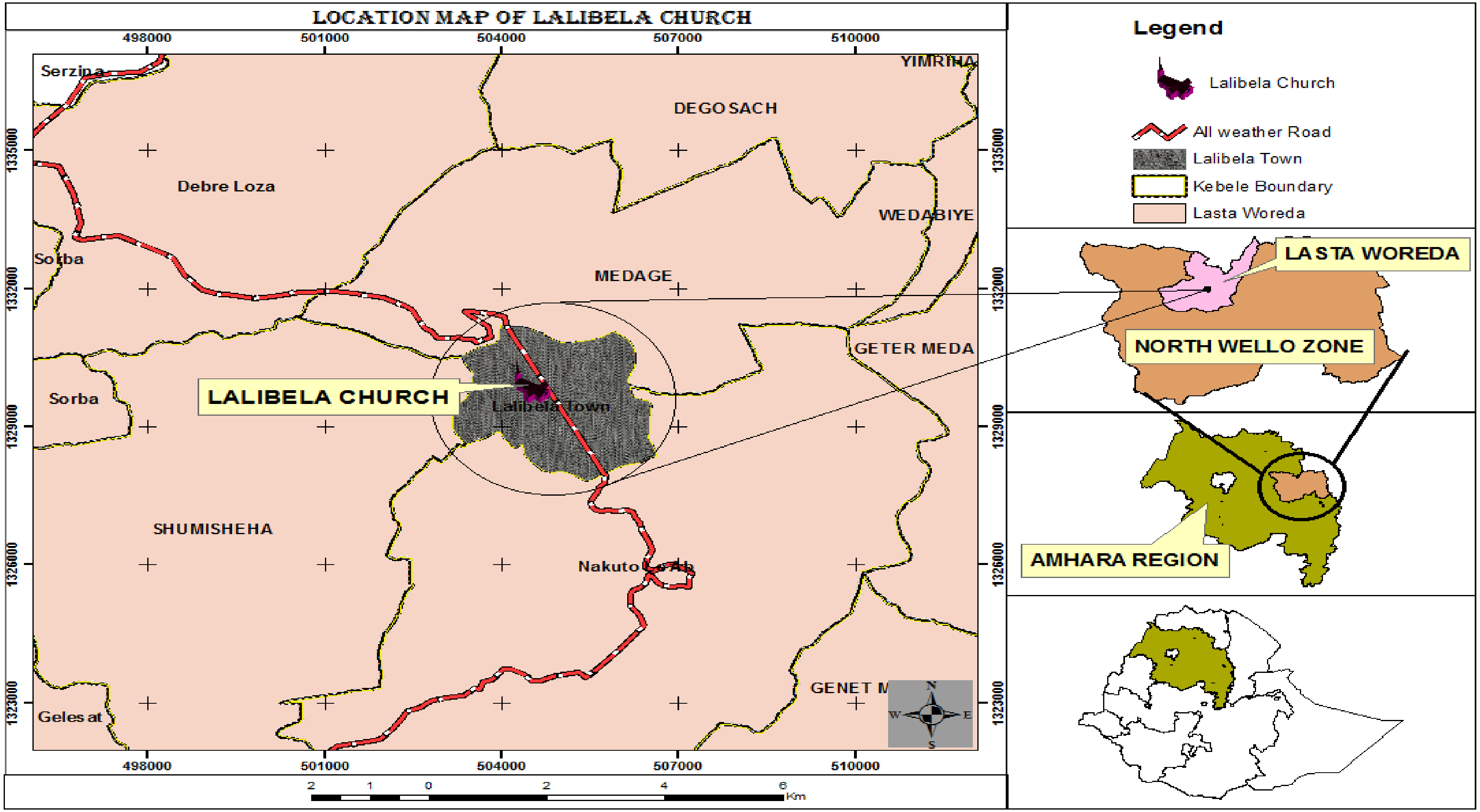

Figure 1.6.1 :- GIS map on location of the study area

CHAPTER 2:- LITRATURE REVIEW

2.1. History of the Project

In 2002, UNESCO along the international union of Architects (UIA) held a design competition for new shelters intended to replace the existing ones protecting the churches at Lalibela. The idea behind the design was to create shelters which were simple, unobtrusive and easy to assemble and dissemble. According to UNESCO, the shelters had to be easily removed once conservation of the churches had concluded [4].

The project plans to have $5{,}5\,M$ € for the construction of new shelters, $0{,}16\,M$ € for supervision of the construction $1{,}2\,M$ €, for the conservation study, $1{,}65\,M$ € for other services and $0{,}59\,M$ € for contingency, with total construction budget of $9{,}1\,M$ € [8].

Of the eight European architectural firms to be recognized by the jury, the Italian company Teprin Associate was unanimously chosen for their design which the jury described as an "elegant, unobtrusive and easily comprehensive project" [4].

Teprin Associate completed the detail designs of the shelters in February 2002. And the construction contract of the shelters was signed with the main contractor named ENDECO, in October 2006 [6].

The original design was simple; large metal columns would use cables to hold up a translucent covering the churches. This covering would allow light to come through, but protect from sun and rain. These structures which are due to be completed by late February 2002, have created controversy amongst the local community and architectural experts, for design changes and problems with construction that have occurred. After 5 years of logistical delays, construction finally began on these shelters, in February of 2007 [4].

Issues has raised on the shelters construction, one of the biggest issues facing the construction of the shelters is the change in design that has occurred. Originally the support columns for the shelter were to be anchored outside the complex of the churches; this was done to limit the visibility of the shelter from inside the site. UNESCO, however, found this original plan too permanent. The foundation of the columns would have to be anchored into the rock, requiring excavation. This excavation would leave permanent holes in the ground which would conflict with the temporary nature of the project. Ironically these 'temporary' shelters are expected to be in place for the next twenty years, leading to question about the reality of UNESCO's original plan [4].

Despite the controversies the construction had to proceed and by February 2009, the supervision contractor issued the certificate of final acceptance of the shelters [6]. Following the completion of the shelters, the state party (the Ethiopian Government) planned to carry out regular inspection and monitoring of the shelters, however there is been no inspection and monitoring of the shelters done yet except once in 2014.

2.2. According to Architectural Aspect

The new design places the support columns in the actual church complex. This change in design has sparked immense controversy among local residents and technical persons, who see the columns an obtrusive to the churches, and a hindrance to the church services that take place every day. [4]

In addition, the original columns, which were to be covered with "sheathing of thin layers of eucalyptus" to help them blend into the local scenery and "create a harmonious relationship between the old and the new" are now bare metal, modern and flashy than the previous one. This design also has a dominating appearance in the church compound, detracting from the beauty of the churches they were intended to protect. It appears, with this new design, that the shelters which were to remain in the background of the church complex have now become one of the main attraction [4].

While I was site investigating, I have tried to interview some tourists from local and foreign countries what they feel about the shelters, and they literally mention that the appearance of shelters do not go with the nature of the monuments, and they have raised a question that is providing those huge and intrusive shelters the only option they have got to protect the monuments, wouldn't be nice if another isolated alternatives are implemented for the preservation.

2.3. According to Structural Aspect

Concerns have also arisen about the stability of the steel shelters themselves, which now are simply resting on to bedrock. Design changes have already had to occur at Beta-Amannueal, because of the risk of high winds, and in an area that experiences unpredictable weather, these winds could threaten the stability of the shelters [4].

Besides the natural (biological, sun and rain) effects on the monuments the so provided temporary shelters has also become a great threats. It seems that the efforts to reduce the impact of the shelters on the area are bringing new threats to the churches.

One of the threat is that the structural columns stability, since they are not anchored in the ground which they counter balance the upwind load only by their weights which perhaps if the provided counter weight is not enough to resist the upwind load, seeing the signs of failure specially on the base plates is quite a serious thing to focus on, because it might lead to the structural failure so that it brings the total loose of the monuments.

The researcher is quite anxious on the effect of the upwind. It’s because as per the report paper of Dr. Esayas on Lalibela, the shelters are designed for the maximum annual fundamental basic wind speed of 14 m/s believed to be occurred by the year 1992, and according to the National Metrology Agency Lalibela region has experienced maximum annual fundamental basic wind speed of 28 m/s by the year 2013. And which the shelters has to be checked for the newly experienced wind speed occurred on the region.

2.4. According to Geotechnical Aspect

Teprin Associate has done an intensive study by the year 2002 [8], on the geotechnical formation of Lalibela where the previous design is thought to be implemented. But for some reasons described above, the design had to change and came up with the implemented shelters. However following the design change there were no study done regarding to the geotechnical property and characteristics and/or occurrence of cavity on the underground rocks. Which basically lead them to place the columns near to or directly on the tunnels/cavities and/or trenches.

In addition to the geotechnical studies, the underground tunnels/cavities should also be checked for the additional load coming from the newly provided shelters. However there were no studies or analysis done to check the stability of the tunnels/cavities.

For instance the tunnel connecting Bete-Mariam and Bete-Mesquel is 1.6 meter far away from the column (A1) shading Bete-Mariam. The tunnel so called the DARK tunnel (sign of hell) leading to the back courtyard of Bete-Abalibanos is 1.2 meter far away from the column (C3) shading Bete-Abalibanos and one of the column (D1) shading Bete-Amannueal is directly placed on the other DARK tunnel connecting Bete-Merqorious and Bete-Amanueal. And the trench at the side courtyard of Bete-Abalibanos leading to Bete-Amanueal is loaded at approximate distance of 92 cm.

Prof. Margottini in his report paper of the recommendation part mentions that “Also the location of proposed shelter should be better investigated to see if alternative solutions may provide similar benefit to monuments (e.g. Swiss project for impermeabilisation of soil above Churches where there is no roof of artistic value), which clearly shows that he had a doubt on the location of the shelters.

CHAPTER 3:- RESEARCH APPROACHING

3.1. Data Collection

3.1.1. Primary Data

This research basically needs a site visit for collecting important data as an input to reach into the realistic results. The data collection system is to be done using different approaches that generally be applied on the steel shelters and the underground tunnels.

And those important data's are collected from different firms and places using different techniques like;

i. Site visit

The main aim of the site visit was to inspect and investigate both the shelters and the underground tunnels, and the site visit is only of visual observation and no tests or samples are taken;

- If sign of failures are encountered; trying to know the causes of the failures, especially focusing on the base plates and counter mass plates, connections, corrosion occurrence, shading skins and downpipes, occurrence of cracks on the underground tunnels, visible displacements on top of the roof the tunnel/cavity.
- Trying to figure out the reference height of the shelters from the normal ground level.
- By using one of the method of rock mass classification (RMR), inspecting the tunnels for sake of classifying the condition of the rock mass.

Site investigation

The site investigation has been done on four of steel structure shelters shading five of the Lalibela rock-hewn churches and on the underground tunnels/cavities and trench, it is basically of supported by visual observation.

For simplicity and understanding the shelters are designated and described as similar to the already designation made by the consulting firm on the working drawing they had, and are illustrated in below:

a. SHELTER A :- Shading Bete Mariam and Bete Mesqel
b. SHELTER B :- Shading Bete Medhaniealem
c. SHELTER C :- Shading Bete Abalibanos
d. SHELTER D :- Shading Bete Amannueal

The site investigation has brought important data which invokes for detail researching as described below;

i. Investigating the shelters

a. There are no corrosions occurred on all of the steel structure shelters
b. Some of the cover of the shelters start to deteriorate so that the rain water is getting into the compound specially the one shading Bete-Amannueal, and also most of the down pipes become wrecked so that during rainy seasons the rain water is getting into the churches and the courtyard walls.
c. On almost all of the shelters the plates to be act like a foundation, creates opening and a tensile bending seems to be created by up wind pressure. The following pictures does clearly shows what really is happening on each and every base plates of each shelter;

ON SHELTER - A

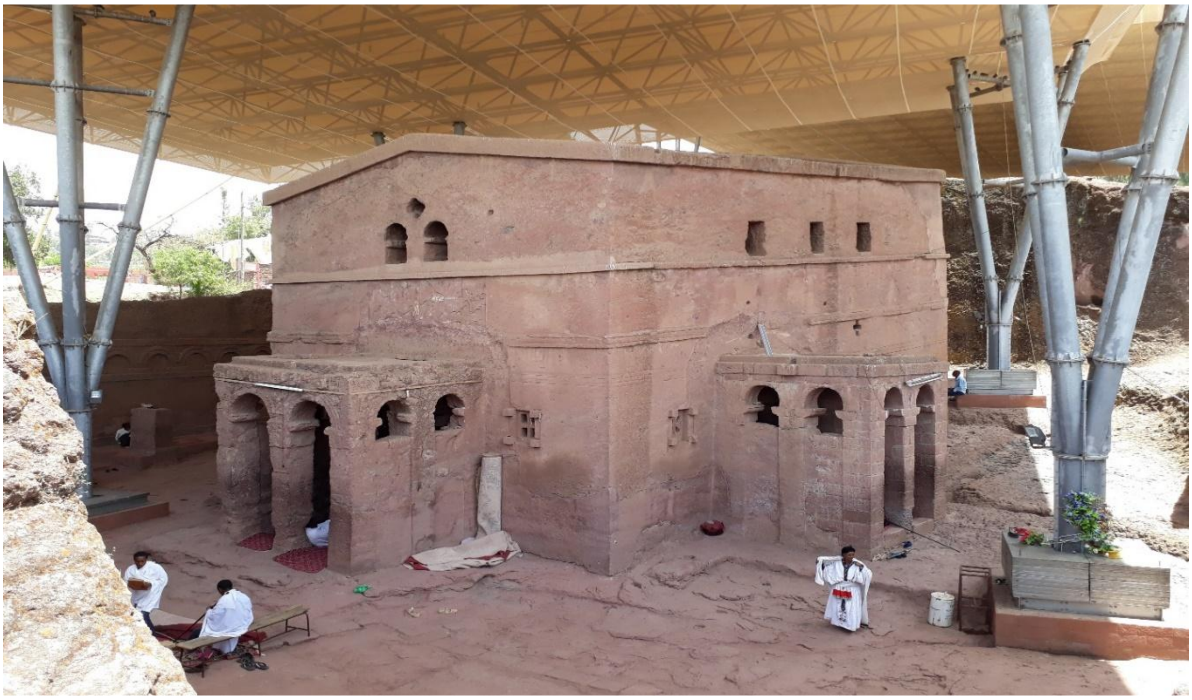

Figure 3.1.1-1:- Shelter-A (on Bete-Mariam and Bete-Mesquel,)

Figure 3.1.1-2 Column-A1and A3 (it's been witnessed that there are no openings encountered on the base plates)

Figure 3.1.1-3 Column-A2 and A4 (It's been witnessed that different width of openings occurred on the base plates in four direction)

ON SHELTER - B

Figure 3.1.1-4:-Shelter B, It's been witnessed that there are no openings encountered in any of the base plates at each column.

ON SHELTER - C

Figure 3.1.1-5:- Shelter C, on Bete-Abalibanos

Figure 3.1.1-6 Column-C3 and C4 (There are no openings occurred between the plates but slight over turning of the base plates from the place where they bolted are witnessed in this two columns).

Figure 3.1.1-7:-Column-C1 (It's been witnessed that there are no openings encountered on the base plates but Bending of the base plates is encountered and witnessed).

Figure 3.1.1-8:-Column-C2 (It's been witnessed that a slight openings and Bending of the base plates encountered on the base plates in four direction) (photo by Naol G.).

ON SHELTER - D

Figure 3.1.1-9:-Column-D1 (It's been witnessed that different width of openings occurred on the base plates in four direction) (photo by Naol G.).

Figure 3.1.1-10:- Column-D2 (It's been witnessed that different width of openings occurred on the base plates in four direction) (photo by Naol G.).

Figure 3.1.1-11:-Column-D3 (Bending of plates towards the column center and Slight uniform opening of plate is witnessed on this column)

Figure 3.1.1-12:-Column-D4 (Bending of plates towards the column center and opening of base plates are witnessed on this column)

d. Most of the columns of the shelters placed on/and near to the underground tunnels.

e. The reference height of the shelters from normal ground level, number of plates and there details has been collected and described as follows;

Table 3.1.1-1:-Number of plate and reference height of the shelters

shleter	No of plate				Reference height			
A on Bete-Mesquel and Bete - Mariam	A1	A2	A3	A4	A1	A2	A3	A4
	6	60	6	60	6	6	6	6
B on Bete-Medhanialem	B1	B2	B3	B4	B1	B2	B3	B4
	6	6	6	6	8	8	8	8
C on Bete Abalibanos	C1	C2	C3	C4	C1	C2	C3	C4
	72	2	2	72	15.6	4	4	15.76
D on Bete Amannueal	D1	D2	D3	D4	D1	D2	D3	D4
	27	27	27	27	7	7	4	4

The base plate is directly welded to the bottom of the column as a foot, and number of L-shaped counter mass plates are put over it and C- shaped plates are used to connect the base plates and the counter mass plates. Size and shape of the foundation detail is described in below.

i. The base plate having a thickness of 20 mm

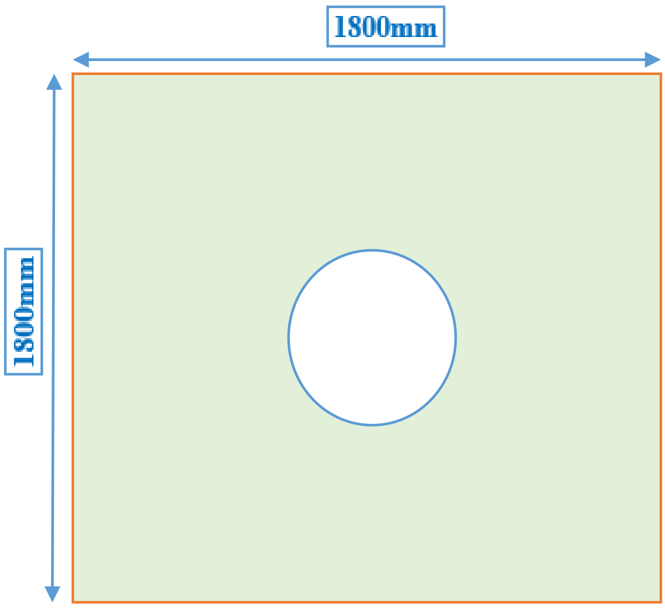

Figure 3.1.1-13:- The base

ii. The counter mass plate

This plate is just put up on the base plate as of only acting as a counter mass. It's shaped as L-shaped plates having a thickness of 10 mm with the dimension shown in the picture below.

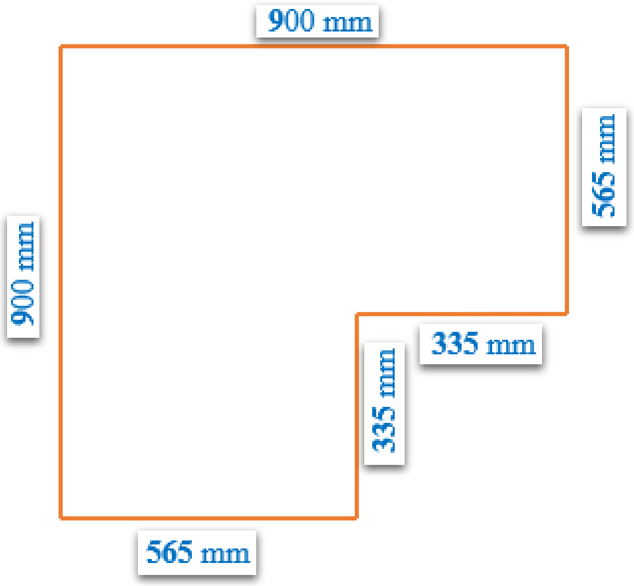

Figure 3.1.1-14:- The counter mass plate

iii. C-shaped /connecter plate

This plate can be described as a C- shaped plate with a thickness of 15 mm and it's used as a connecter of the base plate and the counter mass plates, its size and shape is described in the picture below.

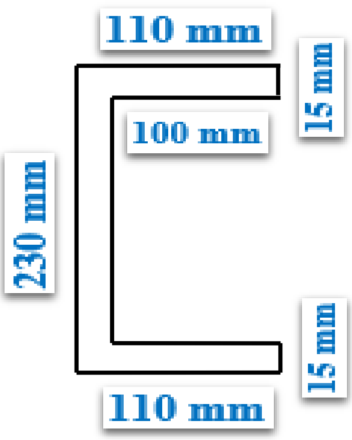

Figure 3.1.1-15:- C-Shaped/connecter plate

ii. Investigating the underground tunnels

The investigation mainly focuses on two of the underground tunnels, one is the dark tunnel which lead to the backyard of Bete-Abalibanos and the other dark tunnel which is currently closed for public use for security reason, found on the courtyard of Bete-Amanueal connecting Bete-Merqorious and Bete-Amannueal.

The tunnel investigation does use the method of RMR (Rock Mass Rating) which helps us to know and classify the condition of the rock mass, mainly of the underground tunnel roof and the tunnel walls (refer sub-chapter 3.3.1.).

And here below are the collected data's about the two distinguished tunnels/cavities.

a. The column designated as C_3 at Bete-Abalibanos found approximately at $1{,}2\ m$ far from the dark tunnel which leads to the back yard of Bete Abalibanos.

The tunnel dimension and shape is

- Its rectangular in shape
- Tunnel roof thickness is approximately $2{,}4\ m$
- Internal width is approximately $1\ m$
- Internal tunnel height is $2\ m$

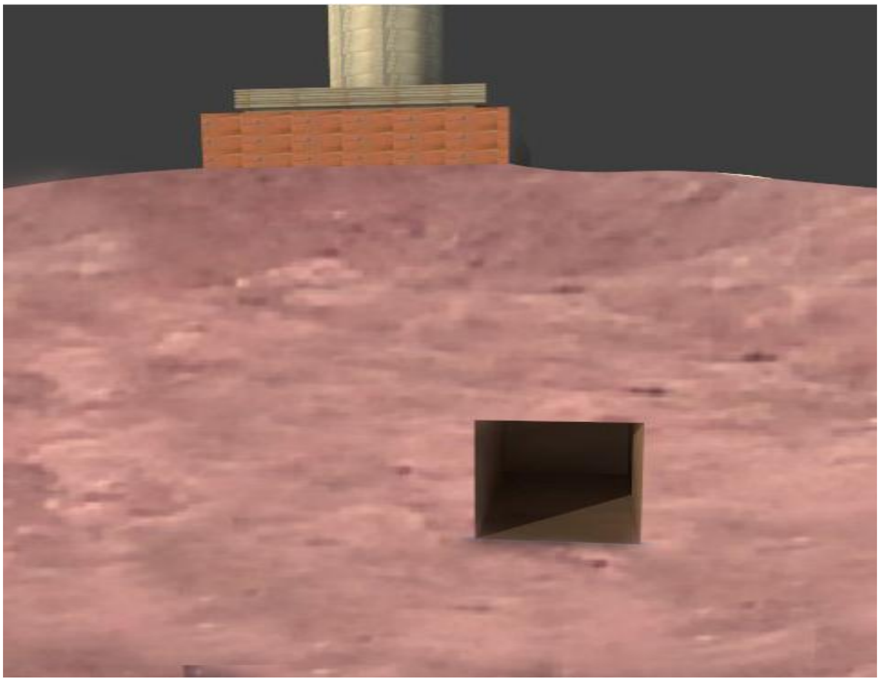

Figure 3.1.1-16:- Section view of the underground tunnel at Bete- Abalibanos (ARCHICAD drawing)

b. The column designated as D_1 at Bete-Amannueal placed directly on the dark tunnel connecting Bete-Merqorious and Bete-Amanueal

The tunnel dimension and shape is

- Its rectangular in shape
- Tunnel roof thickness is approximately 2 m
- Internal width is approximately 1m
- Internal tunnel height is 2 m

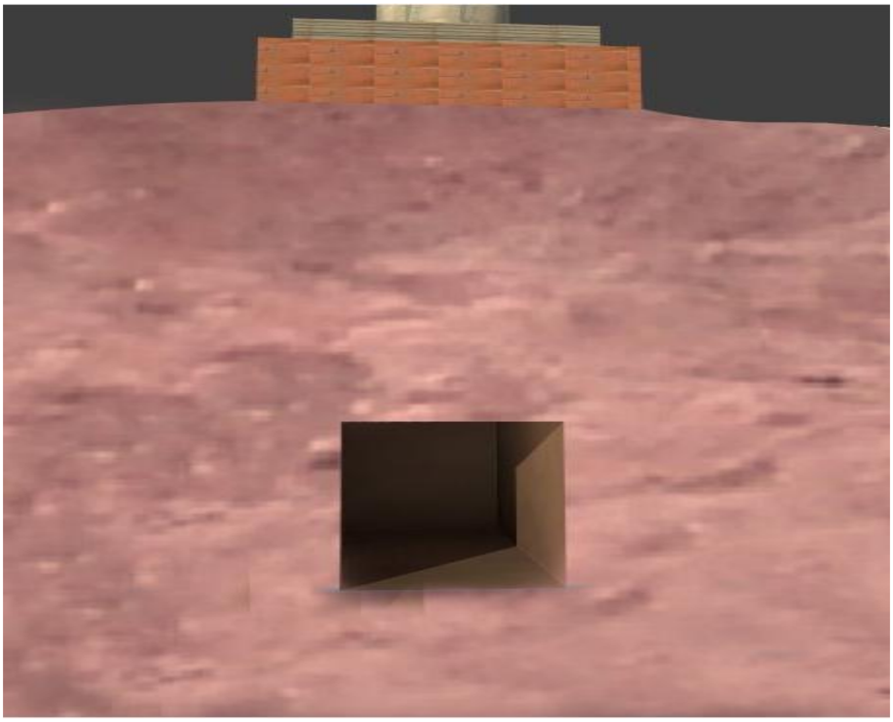

Figure 3.1.1-17:- Section view of the underground tunnel at Bete-Amanueal (ARCHICAD drawing)

iii. Interview

The interview has been collected from six local and high administrative peoples which are more close to this issue and important peoples around the churches. Basically the interview focuses on the following six points listed in below:-

a. Have you ever noticed any change on the steel shelters?
b. Were there any maintenance activities or inspections made right after the construction?
c. Does the wind has made any feasible movements on the shelters?

d. What opinions do you have about the steel shelters?
e. How many of the steel pillars placed on the underground tunnels or near to trenches and their locations?
f. Have you ever notice any change or cracks witnessed on the underground tunnels or trenches right after the construction?

All most all of the peoples that I interviewed has raised similar points on the question I have asked, they all agreed that the opening occurred on the base plates are gradual but what they didn't understood is the reason that brought this signs. And they have told me that there were no inspection, or periodic checking has taken over as per the promise and agreement made by ARCCH. However they have all agreed that besides making noisy sound of the Shelter cover the wind doesn't make any noticeable movements on the structures.

And as it is described on the underground investigation part Diacon Mekonnen G/Meskel has told me about the underground tunnels that are thought to be under risk by the community and the priests. However since there were no training gave to the community, they didn't even think about checking or inspecting the tunnels and the shelters as well.

3.1.2. Secondary Data

i. Metrological data of Lalibela Region

The wind speed for Lalibela region is collected from National Meteorology Department. The row wind speed is measured at the last 10 minutes of the 180 minute interval wind at 10 meter height, at an elevation of 2487 above sea level, 39.03980 of longitude and 12.0390 latitude.

Daily maximum wind speed with 10 minutes average data at 10 meter height is selected. Out of it the maximum annual wind speed are selected. Having these initially, different probability distributions are going to be tested for suitability of the wind speed and out of them the best fit is going to be selected as the maximum fundamental basic wind speed.

Table 3.1.2-1:-maximum annual wind speed of Lalibela region (source: - National Meteorology Department.)

Year	2006	2007	2008	2009	2010	2011	2012	**2013**	2014	2015	2016
maximum wind speed	12	9	22	12	21	10	9	**28**	27	10	10

And the wind rose diagram shown below illustrates the frequency and speed of wind occurrence in Lalibeal region. And most of the highly frequent wind speeds are generated from South, South-East and East directions with the magnitude of greater than $\mathbf{5}\ \boldsymbol{m/s}$.

Wind speed and direction changes throughout the day and year, and is not as universally predictable as the sun's movement [9]. Knowing your location's wind patterns influences important environmental and structural considerations.

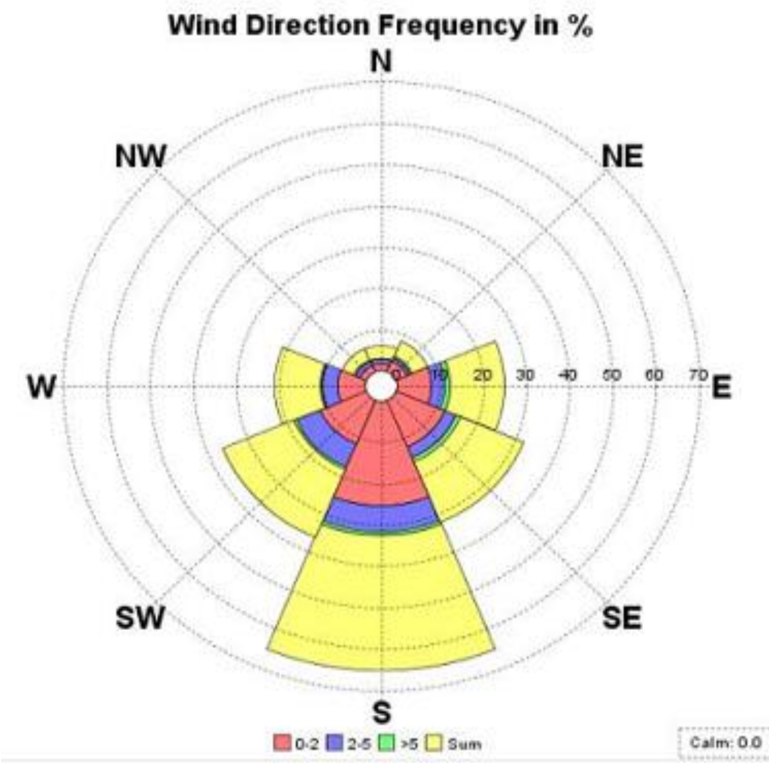

Figure 3.1.2-1:- Wind Rose Diagram for Lalibela Region from 2006-2015 at 10 meter height (source National Meteorology Department.)

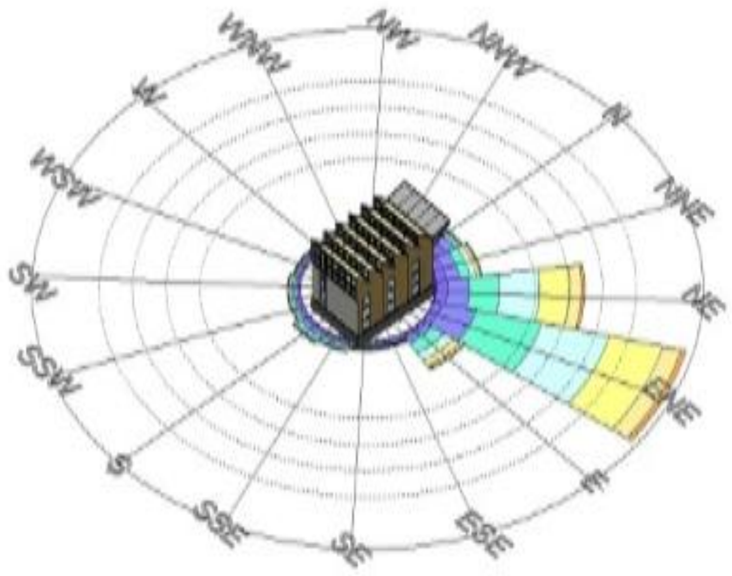

Figure 3.1.2-2:- Sample Wind Rose Diagram (source: - [6])

A “wind rose” diagram is the most common way of displaying wind data, and can be measured in a “speed distribution” or a “frequency distribution”. Wind roses can be a yearly average, or can be made for specific seasons; some even include air temperature information [9].

The above chart shows the frequency and speed of wind blowing from each direction.

As you move outward on the radial scale, the frequency associated with wind coming from that direction increases. Each spoke is divided by color into wind speed ranges. The radial length of each spoke around the circle is the percentage of time that the wind blows from that direction [9].

Annual maximum wind speed were selected from the daily maximum wind speeds recorded at station found in Lalibela at an altitude of 2487 above sea level, 39.039^0 longitude and 12.039^0 latitude.

Different probability distribution has been tested for suitability of the collected data. The probability distribution has been done by Fikade Alamirew (whom his MSc paper was on determining the basic wind speed for the specific regions) specifically for this research. We have used the following probability distribution listed below;

i. Gumbel Distribution
ii. Log Gumbel Distribution
iii. Normal Distribution
iv. Log Normal Distribution
v. Pearson Distribution
vi. Log Pearson Distribution

- **Gumbel distribution**

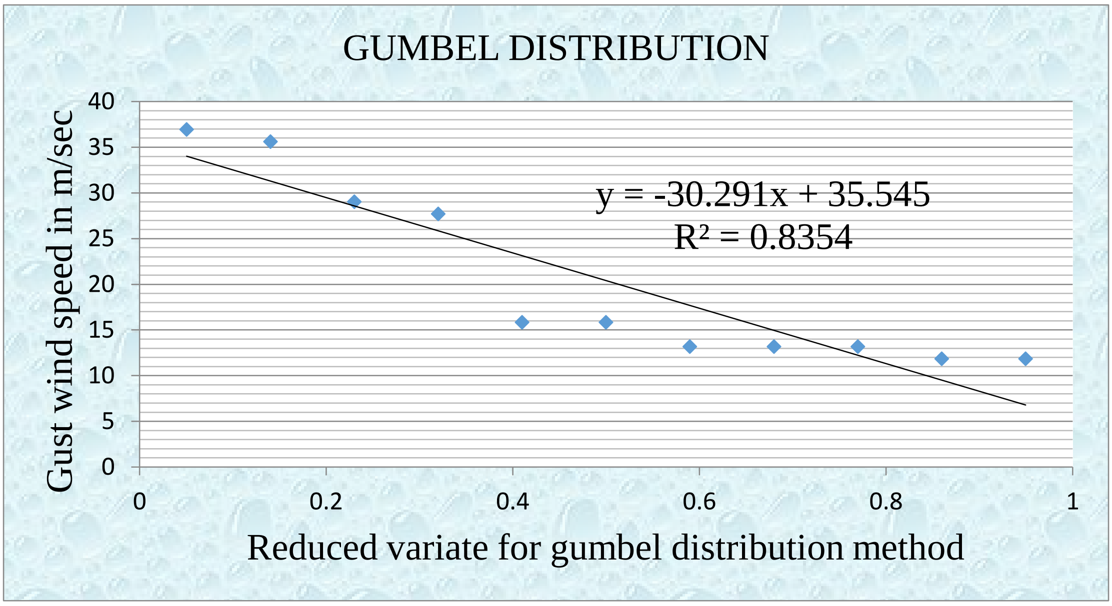

Figure 3.1.2-3:- Graph of Gumbel distribution method

Table 3.1.2-2:- Gust wind speed for different return periods using Gumbel distribution method.

Return Period	Gust wind speed m/sec (Gumbel)
5	28
10	33
15	37
20	**39**
25	41
30	42
50	**46**
100	51
200	57
500	64
1000	69

- **Log Gumbel distribution**

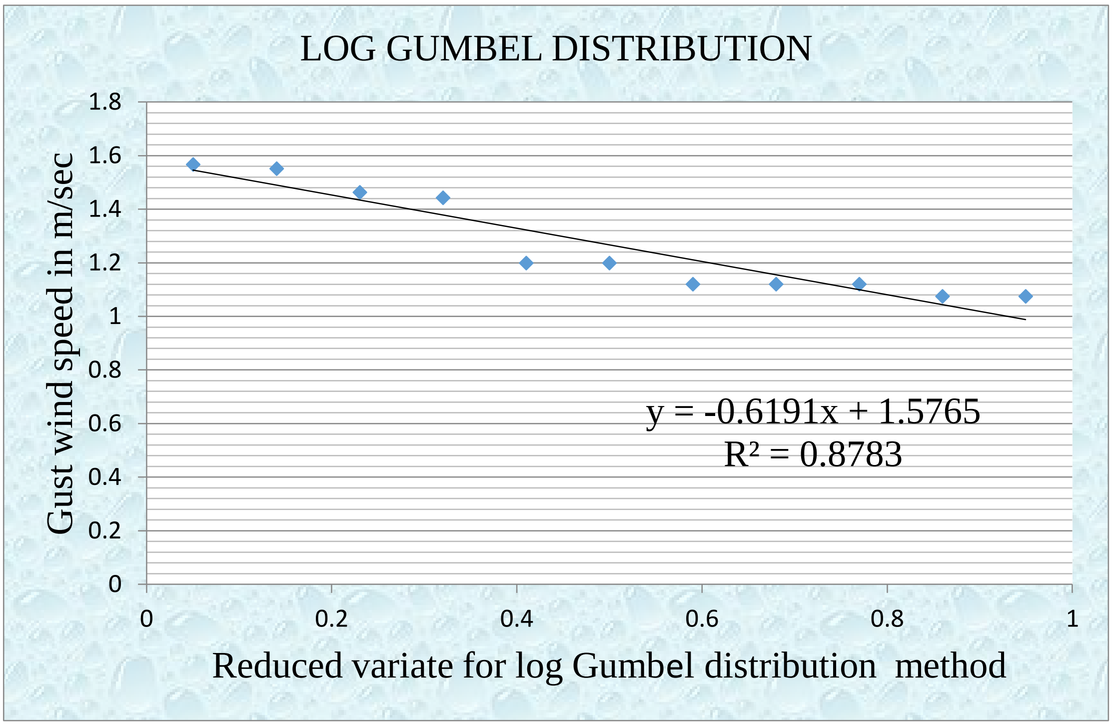

Figure 3.1.2-4:- Graph of Log Gumbel distribution method

Table 3.1.2-3:- Gust wind speed for different return periods using Log Gumbel distribution method

Return Period	Gust wind speed m/sec (Log Gumbel)
5	26
10	33
15	39
20	**43**
25	47
30	50
50	60
100	77
200	98
500	136
1000	174

- Normal distribution

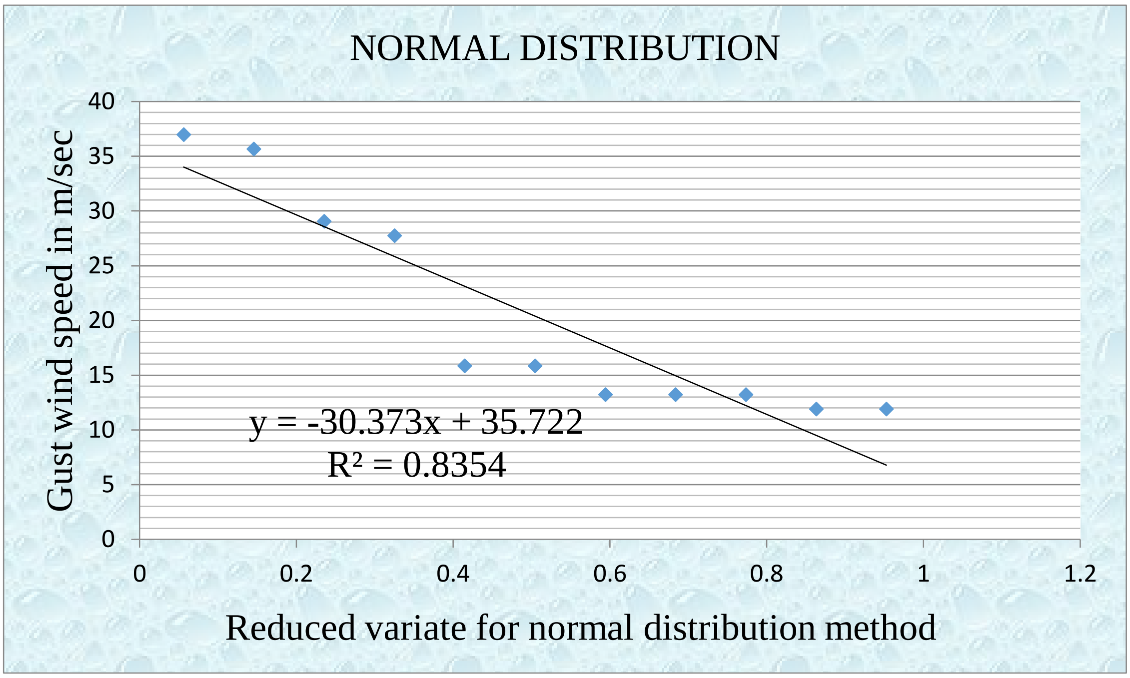

Figure 3.1.2-5:- Graph of Normal distribution method

Table 3.1.2-4:- Gust wind speed for different return periods using Normal distribution method

Return Period	Gust wind speed m/sec (Normal)
5	29
10	33
15	35
20	**37**
25	38
30	39
50	41
100	43
200	46
500	49
1000	51

- Log Normal distribution

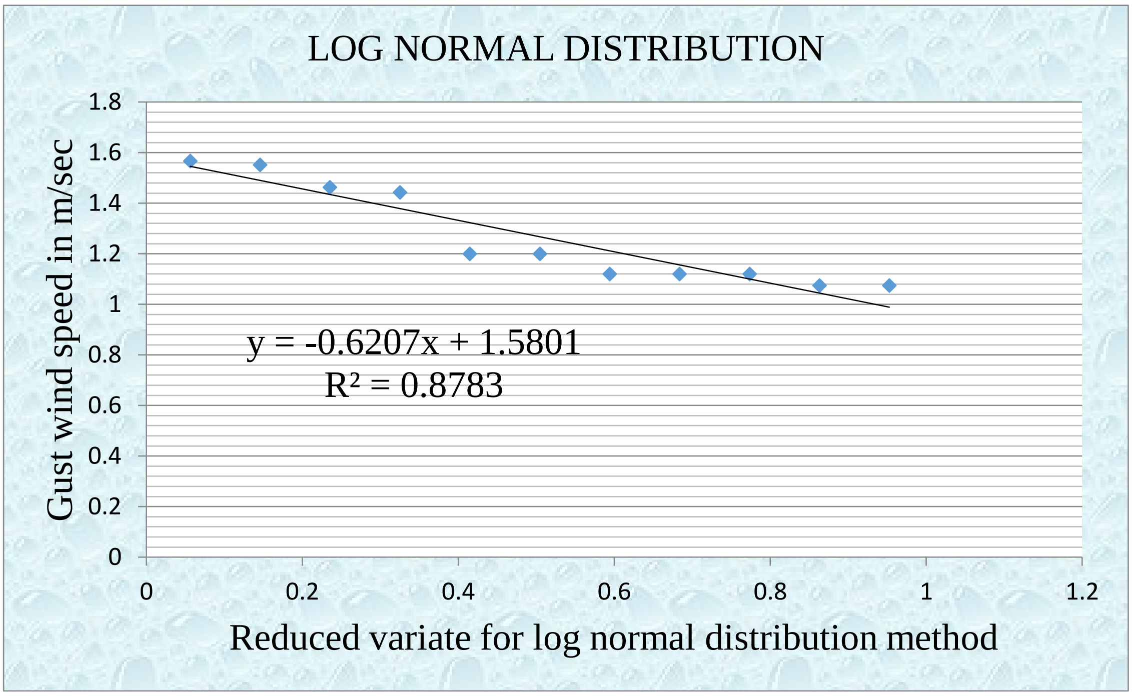

Figure 3.1.2-6:- Graph of Log Normal distribution method

Table 3.1.2-5:- Gust wind speed for different return periods using Log Normal distribution method

Return Period	Gust wind speed m/sec (Log Normal)
5	27
10	33
15	37
20	**39**
25	41
30	42
50	47
100	53
200	60
500	68
1000	75

- Pearson distribution

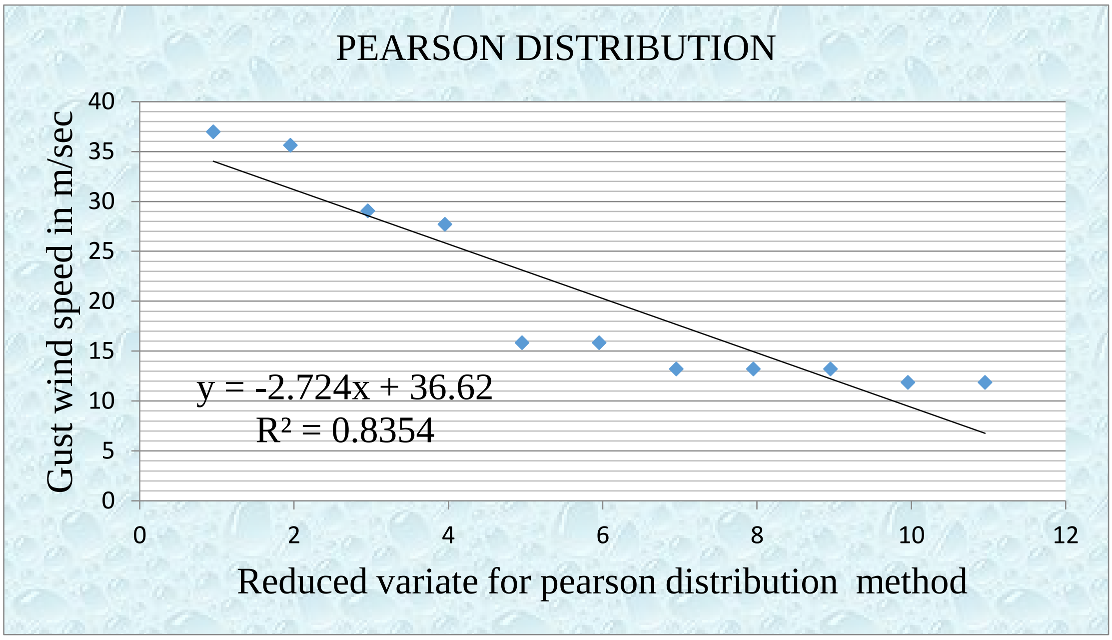

Figure 3.1.2-7:- Graph of Pearson distribution method

Table 3.1.2-6:- Gust wind speed for different return periods using Pearson distribution method

Return Period	Gust wind speed m/sec (Pearson)
5	28
10	34
15	37
20	**39**
25	40
30	41
50	45
100	49
200	54
500	59
1000	63

- **Log Pearson distribution**

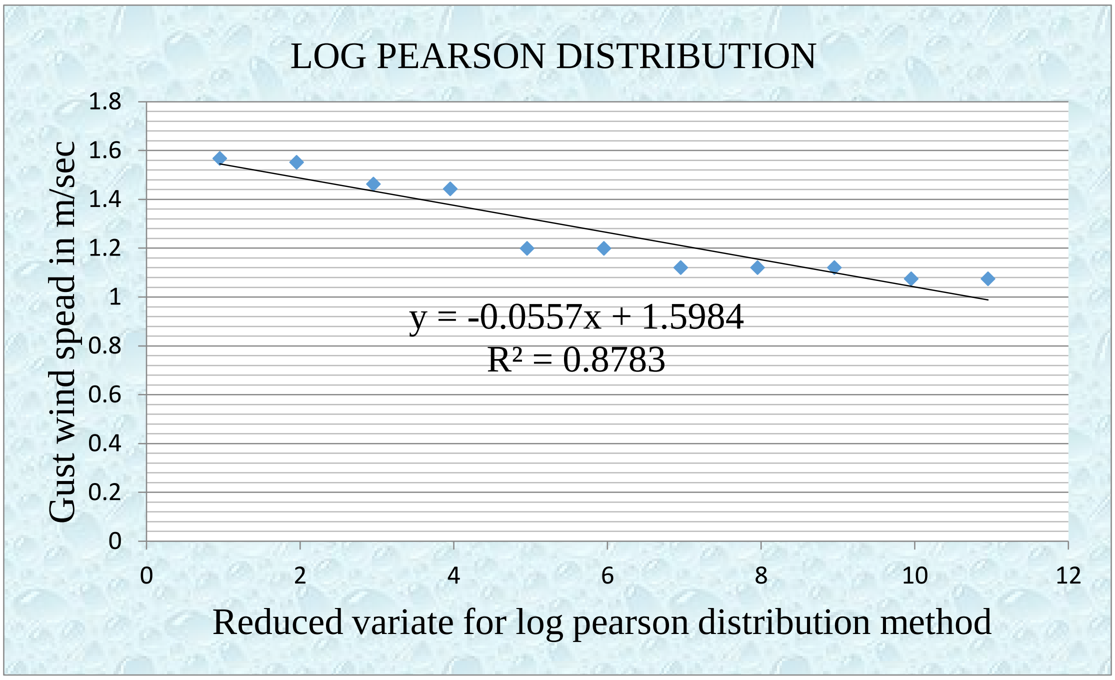

Figure 3.1.2-8:- Graph of Log Pearson distribution method

Table 3.1.2-7:- Gust wind speed for different return periods using Log Pearson distribution method

Return Period	Gust wind speed m/sec (log Pearson)
5	27
10	34
15	38
20	**42**
25	45
30	47
50	54
100	65
200	78
500	97
1000	114

As we can see from the distribution graph the log normal distribution does the best fit graph that we prefer to choose this distribution. So we get that the fundamental value of basic wind velocity is 39 m/s. However it is lately known that the wind data is measured at the last 10 minutes of the 180 minute interval reading in which there is no need of conversion by the factor of 1,32 that Fikade has obtained on his research paper [10]. Hence the value of 39 m/s has to be divide by the factor 1,32 and gives us the fundamental basic wind velocity of Lalibela region as 29,5 $\mathrm{m/s}$ for 20 years of return period which is the design period of the shelters.

ii. Geological data of the study area

The region of Lalibela is characterized by the out cropping of tertiary volcanic rocks, composed of basalt; trachyte and tuff [8].

The churches of Lalibela are carved out within the Amba-Aiba formations in two distinct unit as the lower basalt and the basic tuff [8].

A deep investigation has carried out on 2002 by Teprin Associate for the “International design competition for the shelter of five churches” and came up with the best they have got as: stratigraphy around the churches, starting from the top is characterized by a fill layer by-product of tuff alteration of tuff debris from trench excavation and, where the tuff comes in contact with atmosphere; by heavy weathered tuff (porosity and water absorption is the mean future). Immediately a tuff from moderately weathered to hard follow in depth lean on basaltic bed rock, from fractured to hard. Distribution of different units is described in the following pictures where some of the geotechnical section of Teprin (2002) have been re-assembled to give a better 3D spatial view [8].

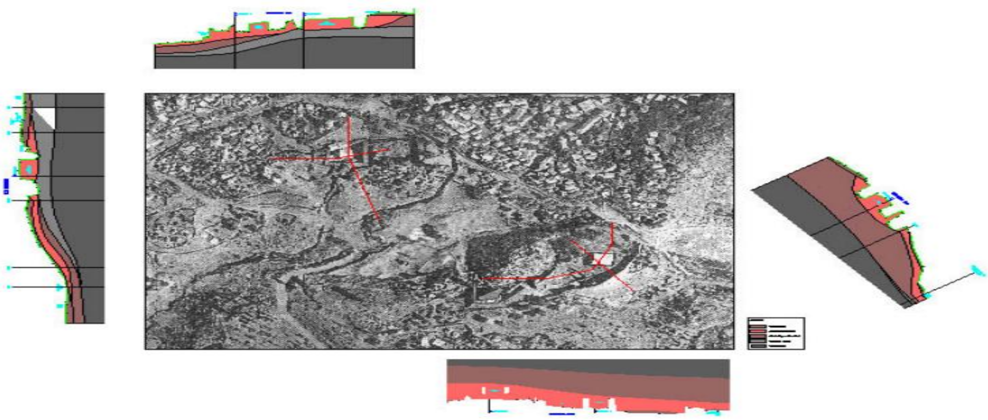

Figure 3.1.2-9:- Geotechnical section of the Lalibela church sites with traces displayed over an aerial photograph. (Source: [7])

Hear below is the table that shows reconstruction of the stratigraphic unit in Lalibela area by gathering different researches done at different time [8].

Table 3.1.2-8:- reconstruction of the stratigraphic unit in Lalibela area (source: [7])

Age	Merla et al. (1973)	ICCROM (1978)	ACEL-SAVA (1997)		TEPRIN (2002)
Holocene-recent			fill	fill	fill
			Weathering regolith	Weathering regolith	
Oligocene to miocene	Amba Aiba basalt	Scoria	Basic Tuff	highly weathered reddish tuff	highly weathered tuff
				moderately weathered tuff	moderately weathered tuff
		Basalt	Basalt	Basalt	fractured basalt
					fresh basalt

Mechanical properties of rocks

The basalt is a good material with bulk density ranging 2.25 – 2.43 g/cm3. And the tuff material it seems to belong only to the highly weathered unit of the basic tuff. The in-situ density was established is about 1.78 g/cm3 while the bulk density was estimated by TEPRIN (2002) ranging from 2.04 – 2.16 g/cm3 [8].

The tuff exhibits an average uniaxial compressive strength of about 36 Mpa in natural condition (γ = 16 KN/m3), decreasing to 28 Mpa when saturated. The basalt exhibits a uniaxial compressive strength of about 44 Mpa (γ = 20 KN/m3) [8].

The Pilot study of Bete-Gebrieal-Rufael described the mechanical properties for the Scoriacous basalt with two different samples. Sample A is the intact/ harder Scoriaceous basalt and sample B is for fine grained Scoria levels with variable sized fragments [11]. And the averaged data for both samples is summarized and indicated in the table shown below.

Table 3.1.2-9 Mechanical property of the rock in Lalibela (Bete-Gebrieal-Rufael, source [11])

Sample	Unit Wt. (γ) $({}^{Kg}/_{m^3})$	UCT(σ_c) (MPa)	Young's Modulus(E) (MPa)	Poisons ratio
A	1900.5	18.53	2850	0.12
B	1647.2	6.5	1204.7	0.27

And the uniaxial compression strength of the rocks found in Lalibela region done by Teprin Associate is listed in below;

Table 3.1.2-10:- Uniaxial compressive (UCS) in MPa for the investigated sites and topology of Material (source [7])

	Uniaxial Compressive Strength (Mpa)				
	Fresh tuff	Alveolizer tuff	Alveolizer wet tuff	wet tuff	Basalt
Bete-Medhaealem	44	45	38	38	55
Bete-Mariyam	37	38		18	
Bete-Mesquel	37				
Bete-Danagel	38				
Gologota	42		38		
Bete-Amanueal	32 42 37	32			
Bete-Merqorewos	28	27			
Bete-Abalibanos	38 27 42				
Bete-Gabriel-Rufael	34			30 31	
Bete-Giyorgis	37	30 25			33

As we can see from the above two tables, the tests are taken at different time by different researcher's and results quite different values while referring to the compressive strength of the rocks at Bete-Gebrieal-Rufael. And if we see table 3.1.2-10 the test result for the fresh tuff on Bete-Amanueal and Bete-Gebreal-Rufael are quite the same since the geological formation of the two churches are similar (Scoriacous basalt) [12].

Hence it is fair enough to take the mechanical property from table 3.1.2-9 of sample A and B as an input for the numerical analysis as far as safety is concerned. Both scenarios are needed to be checked on analysis so that we could tell about the realistic situations we are facing.

iii. Description of the characteristics of the structural parts

All the structural characteristics of the steel shelter are illustrated clearly on the working drawing of the project that Teprin Associate does prepare as a working structural drawing. And I have copied here below so that anyone could understand the characteristics of the structure and the inputs to be feed into the model that we are about to prepare on SAP2000V16.

Joints:- Balls, forged at high temperature by using a steel mould, hardened and tempered according to the regulation DIN 17200, quality of steel C45.

- Structural rods: obtained from pipes, welded according to DIN 2458(2/1981) quality of steel Fe 430C or higher.
- Cones: drop-forged pieces according to DIN 17200, shaped as truncuated cones with a diameter gauged for the connection to required pipes; quality of steel Fe 430C or higher.
- Welding of cones and pipes according DIN 4100 and DIN 4115
    - From 42 to 127mm diameter with laser welding
    - More than 127mm diameter with bare electrode are welding
- Connection parts for fixing trusses to joints
    - Bolds: Class 5, 6, 8.8, 10.9 according to DIN 267 and DIN 601
    - Nuts: 9 S Mn 28K according to DIN 555 e DIN 1651 and pins
- Maximum dimensional tolerance of parts construction
    - Trusses: lower value between $\pm$ 1mm e $\pm$ 0.5%
    - Joints: $\pm$ 0,2mm
- Steel surface protection against corrosion
    - Trusses, cones and nuts: hot galvanizing on outer and inner surfaces (coating thickness 50-80*10^{-6}m) according to DIN50976
    - Joints bolts and pipes: galvanizing in lightly bold both for a 20*10-6m thickness equivalent to a zinc layer.

3.2. Wind Load Analysis on the steel shelters

3.2.1. General

Wind is highly turbulent and random and variable in nature. Considering their variation in time and space, wind actions are classified as variable fixed actions, it means that the wind actions are not always present and the wind actions have for each considered wind directions fixed distributions along the structure [13].

The wind loads are ‘characteristic values’, i.e. values with a characteristic annual risk of being exceeded of 0.02 in each and every year that the structure remains in service. This level of risk is alternatively described by the mean recurrence interval or ‘return period’ given by the reciprocal of the annual risk, in this case 50 years [14].

Each wind load is determined by a probabilistic-statistical method based on the concept of “Equivalent static wind load”, on the assumption that structural frames and components/cladding behave elastically in strong wind. Wind loads on structural frames and on components/cladding are different, because there are large differences in their sizes, dynamic characteristics and dominant phenomena and behaviors and the correlation they have. Wind loads on structural frames are calculated on the basis of the elastic response of the whole building against fluctuating wind forces [15].

Classification of wind actions

a. Direct and Indirect wind actions
   - Direct for external structures and internal surfaces of open structures
   - Indirect for internal surfaces of enclosed structures.
b. According to the nature and/or structural response
   - Quasi-Static response
   - Dynamic and aero-elastic response.

A mean wind force acts on a building. This mean wind force is derived from the mean wind speed and the fluctuating wind force produced by the fluctuating flow field. The effect of the fluctuating wind force on the building or part thereof depends not only on the characteristics of the fluctuating wind force but also on the size and vibration characteristics of the building or part thereof. Therefore, in order to estimate the design wind load, it is necessary to evaluate the characteristics of fluctuating wind forces and the dynamic characteristics of the building. [16]

The following factors are generally considered in determining the fluctuating wind force.

1) Wind turbulence (temporal and spatial fluctuation of wind).

2) Vortex generation in wake of building.

3) Interaction between building vibration and surrounding air flow.

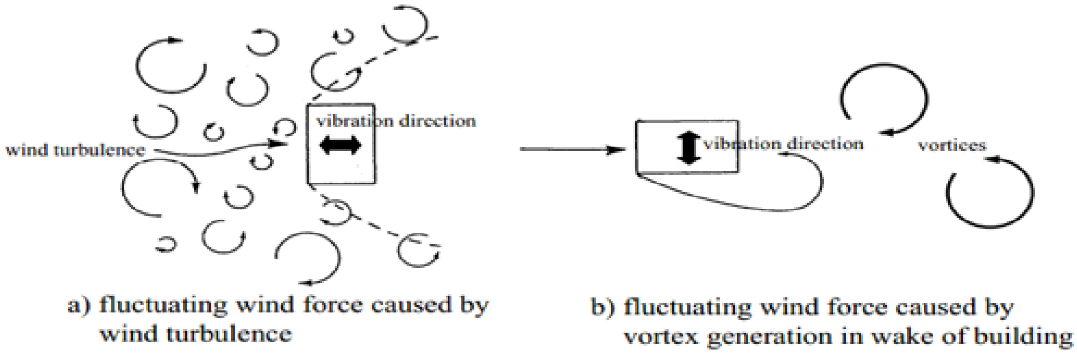

Figure 3.2.1-1- Fluctuating wind forces based on wind turbulence and vortex generation in wake of building (Source:-[10].

Fluctuating wind pressures act on building surfaces due to the above factors. Fluctuating wind pressures change temporally, and their dynamic characteristics are not uniform at all positions on the building surface. Therefore, it is better to evaluate wind load on structural frames based on overall building behavior and that on components/cladding based on the behavior of individual building parts [17].

3.2.2. Wind loading According to Pr EN 1991-1-4:2004

The characteristic peak velocity pressure q_p is influenced by the regional wind climate, local factors like terrain roughness and orography/ terrain topography and the height above terrain. This parameter is in fact the characteristic pressure due to the wind velocity of the undisturbed wind field.

The wind climate for different regions is described by values related to the characteristic 10 minutes mean wind velocity at 10 meter above ground of a terrain with low vegetation (terrain category II). These characteristic values correspond to annual probabilities of exceedance of 0.02 which corresponds to a mean return period of 50 years but for our case the correspondence annual probability of exceedance of 0.05 which corresponds to a mean return period of 20 years.

The characteristic peak velocity pressure depends on different basic parameters listed in below and will be discussed in detail;

Basic wind velocity (v_b)

$v_b = C_{dir} C_{season} v_{b,0}$ …………………..Eqn 3.1 (Source:-Eqn 4.1 from Pr En 1991-1-4:2004)

Where: $v_{b,0}$ = fundamental value of basic wind velocity

v_b = basic wind velocity

C_{dir} = directional factor

C_{season} = seasonal factor

The directional factor C_{dir} accounts for the fact that for particular wind directions the velocity v_b could be decreased, whereas the seasonal factor C_{season} takes account that in case of temporary structures for particular periods the probability of occurrence of high wind velocities is relatively low. For simplification the directional factor C_{dir} and the seasonal factor C_{season} are in general equal to 1.0 [18].

Mean wind speed (v_m)

The basic value of the velocity pressure has to be transformed into the value at the reference height of the considered structure. Velocity at a relevant height and gustiness of the wind depend on the terrain roughness. The roughness factor describing the variation of the speed with height has to be determined in order to obtain the mean wind speed at the relevant height:

$v_m(z) = C_r(z) \,.\, C_o(z) \,.\, v_b$ …………Eqn 3.3 (source: - Eqn 4.3 from Pr EN 1991-1-4:2004)

Where: $v_m(z)$ = mean wind velocity

$C_r\ (z)$ = roughness factor

$C_o(z)$ = orographic factor

The roughness factor Cr(Z) accounts for the variability of the mean wind velocity at the site of the structure due to:

- Height above ground level
- Ground roughness of the terrain upwind of the structure in the wind direction considered

The roughness factor has effects on wind speed and turbulence. The rougher the surface the lower the wind speed but the greater the turbulence.

Cr(Z) =Kr . ℓn. $\left({Z}/{Z_{0,II}}\right)$ For $Z \geq Zmin$ …………Eqn 3.4 (source:-Eqn 4.4 from Pr EN 1991-1-4:2004)

Where: Kr is terrain factor depending on the roughness length Z_o and can be calculated using

Kr = 0.19. $\left({Z_o}/{Z_{0,II}}\right)^{0.07}$ …………Eqn 3.5 (source: - (Eqn 4.5 from Pr EN 1991-1-4:2004)

Z = reference height

Z_0 = roughness length

Z_{min} = minimum height for the corresponding terrain category

= 0.05m for terrain category II --- Pr EN 1991-1-4:2004 table 4.4

$Z_{0,II}$ = 0.05 m (terrain category II) --- Pr EN 1991-1-4-:2004 table 4-1

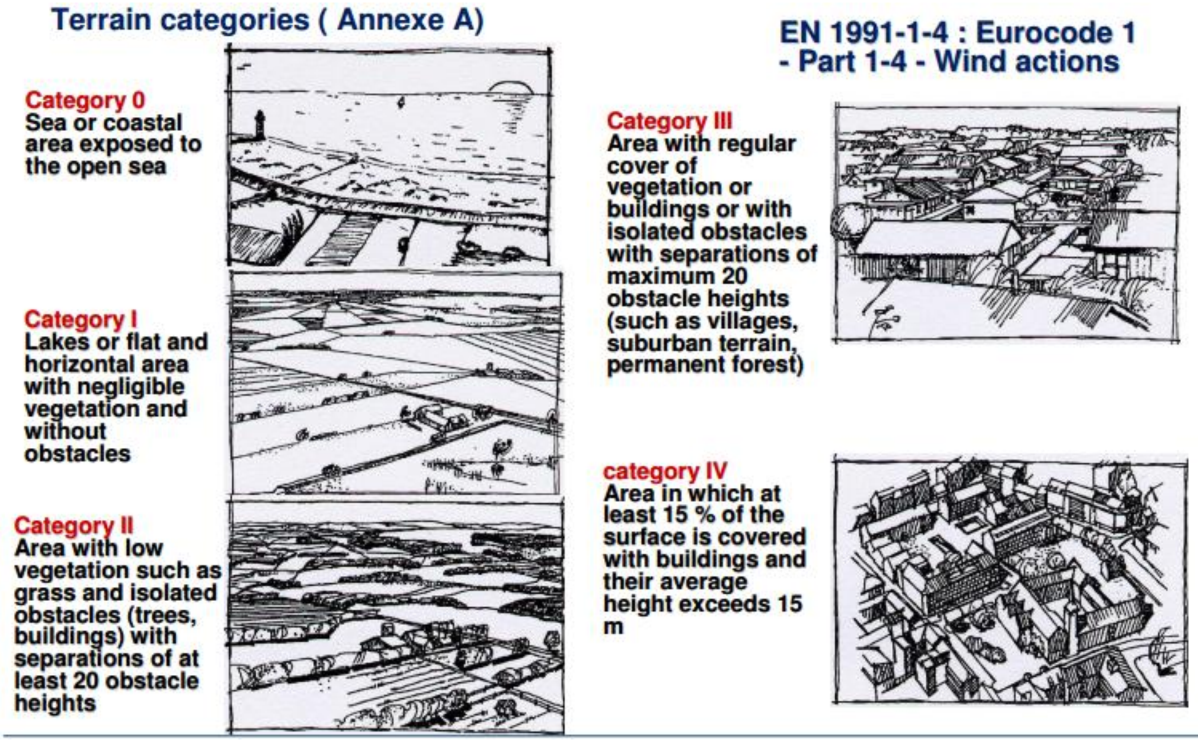

Figure 3.2.2-1:- terrain category, (source: Eurocodes and the Egyptian construction Industry).

Table 3.2.2-1:- Terrain category (source: - from Pr EN 1991-1-4:2004)

Terrain category	Z_0 (m)	Z_{min} (m)
Sea or coastal area exposed to the open sea	0.003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0.05	2
III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages , suburban terrain, permanent forest)	0.3	5
IV Area in which at least 15 % of the surface is covered with buildings	1.0	10

Co(Z) is orography or topology factor, Pr EN 1991 1-4 2004 section 4.3.3 (1) states that where orography (e.g. hills, cliffs etc.) increases wind velocities by more than 5% and the effects should be taken into account using the orography factor Co.

At isolated hills and ridges or cliffs and escarpments different wind velocities occur dependent on the upstream slope Φ=H/Lu in the wind direction, where the height H and the length Lu are defined in Figure 1.

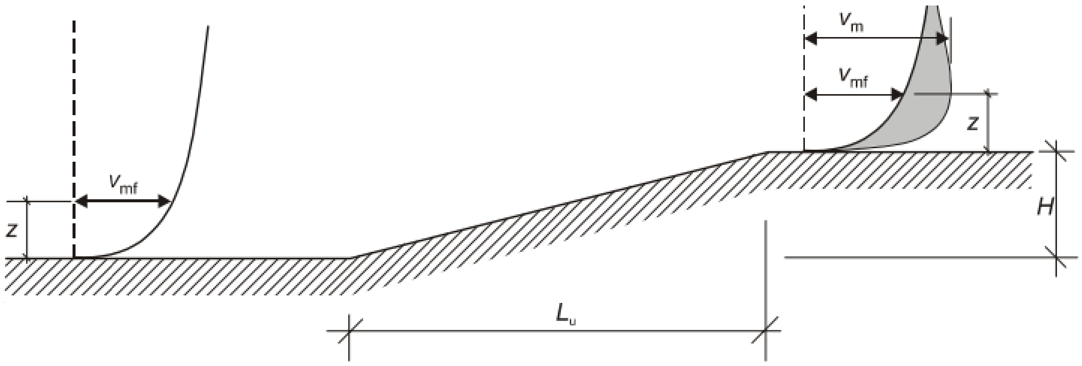

Figure 3.2.2-2:- Illustration of increase of wind velocities over orography (source: - from Pr EN 1991-1-4:2004)

For 0.05 < Φ < 0.3 i.e. shallow where $L_e = L_u$.

Co$(Z) = 1 + 2 \, . \, S \, . \, \Phi$ ……………………………Eqn 3.6 (source: - Pr EN 1991 1-4 2004, A.3)

Where S = orographic location factor

For the range of $-1{,}5 \leq \frac{X}{L_u} \leq 0$ and $0 \leq \frac{Z}{L_e} \leq 2{,}0$, Take:

S $= A \, . \, e^{(B \cdot \frac{X}{L_u})}$ …………………………………...Eqn 3.7 (source:-Pr EN 1991 1-4 2004, A.4)

Where;

$A = 0{,}1552 \, . \left(\frac{z}{L_e}\right)^4 - 0{,}8575 \, . \left(\frac{z}{L_e}\right)^3 + 1{,}8133 \, . \left(\frac{z}{L_e}\right)^2 - 1{,}9115 \, . \left(\frac{z}{L_e}\right)^1$ …………Eqn 3.8 (source:-Pr EN 1991 1-4 2004, A.5)

$B = 0{,}354 \,.\left(\frac{z}{L_e}\right)^2 - 1{,}0577 \,.\left(\frac{z}{L_e}\right)^1 + 2{,}6456$ Eqn 3.9 (source:-Pr EN 1991 1-4 2004, A.6)

Guest velocity/or peak velocity (v_p)

The peak wind velocity accounts for the mean wind velocity and a turbulence component [18]. The guest velocity (or peak velocity) $v_p(z)$ at the reference height of the considered terrain category is calculated with the mean velocity and gust factor G:-

$v_p(z) = v_m(z)$. G………………………………………………..…Eqn 3.10 (source: - [19])

$= C_r(z)\,.\; C_o(z)\,.\; v_b\,.\,G$

Where: G is gust factor and accounts the effect of turbulence in a structure

Turbulence is generated by the friction on the ground and drag on surface obstacles, and is influenced by the terrain roughness just as is the mean velocity.

G = $\sqrt{C_e(z)} = \sqrt{1 + 7\,.\; I_v\,(z)} = \sqrt{1 + 7\,.\frac{\sigma_v(z)}{v_m(z)}} = \sqrt{1 + 7\,.\frac{K_1}{C_0(z)\,.\ln(Z/Z_0)}}$ for $Z \geq Z_{min}$

…………Eqn 3.11 (source: - [13])

Where: k_1 = turbulence factor (usually taken as 1)

$I_v\,(z)$ = wind turbulence intensity at height z, which is the standard deviation of the turbulence divided by the mean wind velocity.

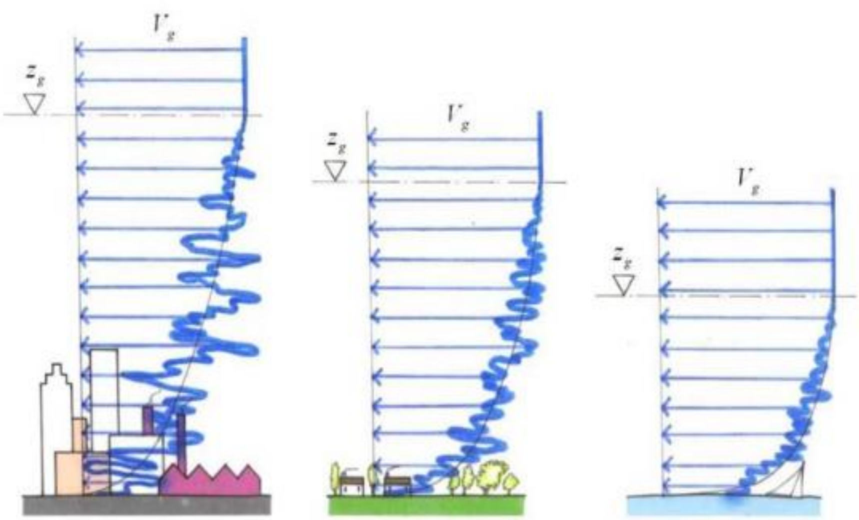

Figure 3.2.2-3:- Mean and Gust Wind velocity profile (source :- [19])

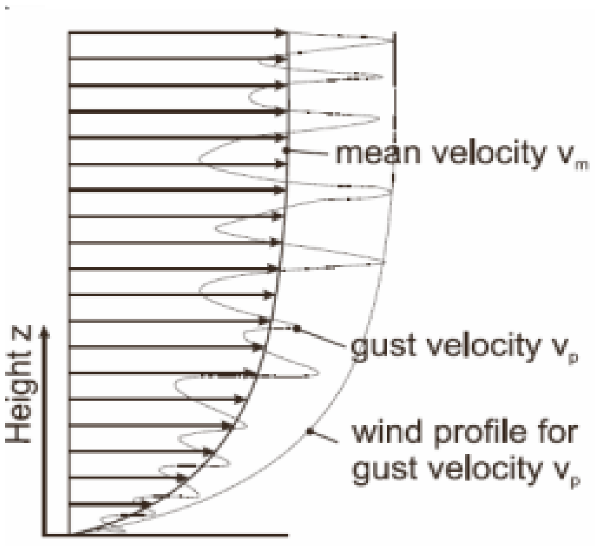

Figure 3.2.2-4:- Wind profile for Peak velocity (Source: - [27])

The peak wind pressure (q_p)

One of the main parameters in the determination of wind action on structures is the characteristic peak velocity pressure (q_p).

The peak velocity pressure at the reference height z, which includes mean and short-term velocity fluctuations, should be determined using an expression:

$q_p(z) = q_b \,.\, [C_r(z)]^2 \,.\, \left[1 + \frac{7}{\ln(Z/Z_0)}\right]$ ……………Eqn 3.10 (source: - Pr EN 1991-1-4:2004)

$q_p = \rho/2\,.\,{v_p}^2$

$q_p = \rho/2\,.\,{v_m}^2\, G^2$

Wind pressure for determination of quasi-static response

The wind pressure on canopy roof is obtained from the expression shown below which is of the combined expression of section 5.2 (5.2) and 5.2 (5.2) of Pr EN 1991-1-4:2004 as of stated in section 5.2 (3).

$w_p = q_p C_{p,net}$ ……………………………………Eqn 3.11 (source: - Pr EN 1991-1-4:2004)

Determination of the wind force

The resulting wind force can be determined by integration of the wind pressure over the whole surface or by applying appropriate force coefficients that are given in Pr EN 1991-1-4 for different kinds of structures. It is noted here, that for many structures force coefficients result into more accurate results than integration of pressure coefficients, it also lead to a time saving determination of wind effects [18].

The wind force F_w is obtained using the equation:

$F_w = C_s C_d \, . C_f \, . q_p(Z_e) \, . \, A_{ref}$ ……………………Eqn 3.12 (source: - Pr EN 1991-1-4:2004)

Where: $F_w =$ wind force

$C_s \, C_d$ = are together the *structural factors* which are the size factor and dynamic factor respectively, they account the effect on wind actions from the non-simultaneous occurrence of peak wind pressure on the surface (C_s) together with the effect of the vibrations of the structure due to turbulence (C_d). And for structures which are not susceptible to turbulence induced vibrations the recommended value of Cs Cd = 1 from Pr EN 1991 -1-4: 2004, 6.2 (1) (a)

C_f = force coefficient, force coefficients give the overall effect of the wind on a structure, structural element or component as a whole, including friction, if not specifically excluded [20].

A_{ref} = reference area of the structure or structural element.

Z_e = reference height (maximum height of structure above ground level)

NOTE: section 7 of Pr EN 1991 -1-4: 2004, gives C_f values for structures or structural elements such as prisms, cylinders, roofs, signboards, plates and lattice structures etc. these include friction effects.

3.2.3. Wind Loading on Canopy roof

3.2.3.1. General

Canopy roof is defined as the roof of a structure that does not have permanent walls, such as petrol stations, Dutch barns, etc. and their general configuration is shown on the code at table 7.6 and 7.7 of Pr EN 1991 -1-4: 2004 [20].

The American Society of Civil Engineers (ASCE), defines a canopy roof as a free roof, which is an open building with no enclosing walls underneath the roof surface [17].

Many free-standing canopy roofs of membrane structures are constructed to provide shade and weather protection in public spaces. Because they are lightweight and flexible, wind resistance is critical to their structural design [21].

Since the roofs are supported by columns and no walls, wind action is directly exerted both on the upper and lower surfaces. Therefore, these roofs seem more vulnerable to wind actions than those of enclosed buildings [22].

A canopy roof can be of monopitch, gable/duopitch or troughed/Multibay, and one of the studies has shown that the magnitude is larger for mono-sloped canopy roofs than for gable and troughed roofs [22].

One of the research mentions that the wind pressure distribution on the flat canopy roof is highly influenced by wind direction as well as blockage [23]. And as we can observe on the pictures of the shelters some of the plates within the same shelters exhibit different width of openings, even one side of the shelters plate has opened while the other is still intact, which somehow imply that the direction of wind load has an impact.

The wind forces on the roofs are influenced by many factors, such as roof shape, roof pitch, obstruction under the roof and wind direction [22]. And all this factors will be tried to be addressed on determining the wind loading.

3.2.3.2. General considerations on a Mono-Pitch canopy roof

i. Wind loading on the roof cover

Degree of blockage under a canopy roof

The degree of blockage under a canopy roof has a major effect on wind pressure/force, and it depends on the blockage $\boldsymbol{\varphi}$ which is;

$$\boldsymbol{\varphi} = \frac{\text{area of feasible actual obstruction under the canopy}}{\text{cross sectional area under the canopy}}$$

Both areas are being normal to the wind direction $\boldsymbol{\varphi}$ = 0 represents an empty canopy and $\boldsymbol{\varphi}$ =1 represents canopy with full blocked with contents to the downwind eaves only (this is not a closed building). As we can see on table7.6 of Pr EN 1991 -1-4: 2004 the net upward wind action increases in increase of blockage degree from 0^0 to 20^0 of roof angle and it immediately start to decrease starting from 25^0 to 30^0.

The overall force coefficients (C_f), and net pressure coefficients (C_p)

This values are given on tables 7.6 for φ = 0 and φ = 1 take account of the combined effect of wind acting on both the upper and lower surfaces of the canopies for all wind directions. Intermediate values may be found by linear interpolation.

The overall force coefficient's represents the resulting force. The net pressure coefficient's represents the maximum local pressure for all wind directions. It should be used in the design of roofing elements and fixings [20].

Each canopy must be able to support the load cases as defined below*:*

- For a mono pitch canopy (Table 7.6 of the code) center of pressure should be taken at d/4 from the wind ward edge (d= along wind dimension, Figure 7.16)
- For canopies with double skins, the rules in 7.2.10 (4) of PrEN1991-1-4:2004 should be used.
- The reference height Z_e should be taken as h as shown in Figures 3.2-5.

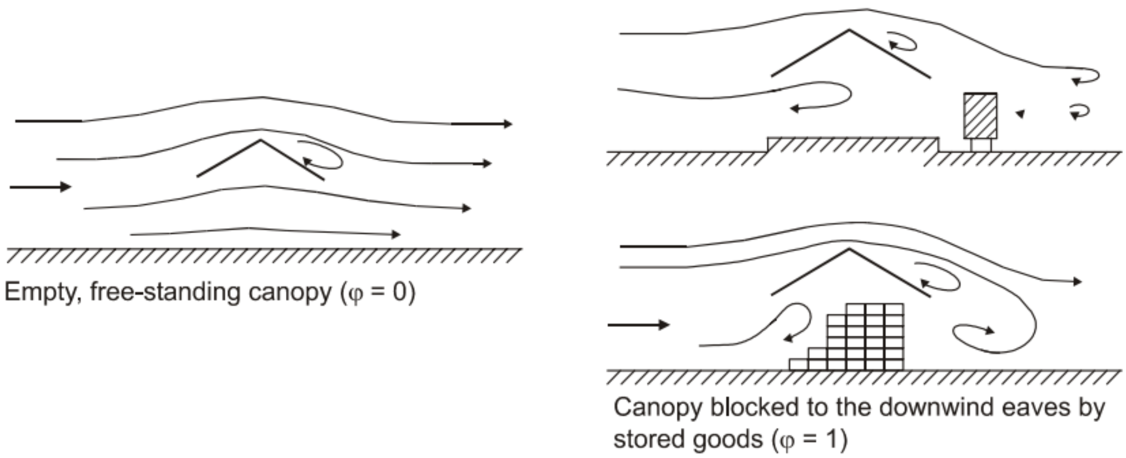

Figure 3.2-4:- Airflow over canopy roofs (source [13]).

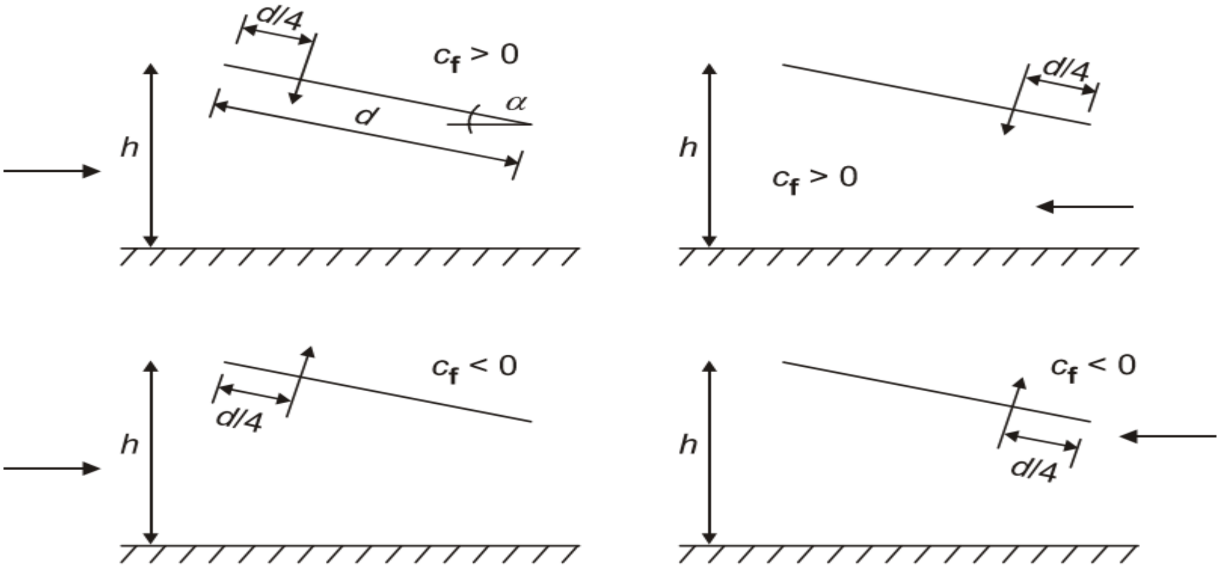

Figure 3.2-5:- Location of the center of force for monopitch canopies and reference height (source [13]).

And for canopies with double skins the rules of EN 1991:4, 7.2.10 (4) states that

i. The wind force is to be calculated separately on each skin.
ii. The permeability μ of a skin is defined as the ratio of the total area of openings to the total area of the skin. A skin is defined as impermeable if the value of μ is less than 0.1%.
iii. If only one skin is permeable, then the wind force on the impermeable skin should be determined from the difference between the internal and the external wind pressure as described in Section 5.1. As a first approximation it is recommended that the wind pressure on the most rigid skin may be taken as the difference between the internal and the external pressures.

So as an approximation the euro code gives some recommended rules which actually comply with our condition as; for walls and roofs with a permeable inside skin with approximately uniformly distributed openings and an impermeable outside skin, the wind force on the outside skin may be calculated from Cp, net = Cpe- Cpi, and the wind force on the inside skin from Cp, net = 1/3 ·Cpi.

But when we refer the code, unlike other roof types, on canopy roof it's only possible to obtain the net wind pressure coefficients' or net wind force coefficients' rather than obtaining the internal and external wind pressure coefficients apparently. Which the code provision has a limitation and contradicts each other which basically needs our engineering judgment.

The figure in below is taken during the site visiting which clearly shows the inside cover of the shelter is air permeable.

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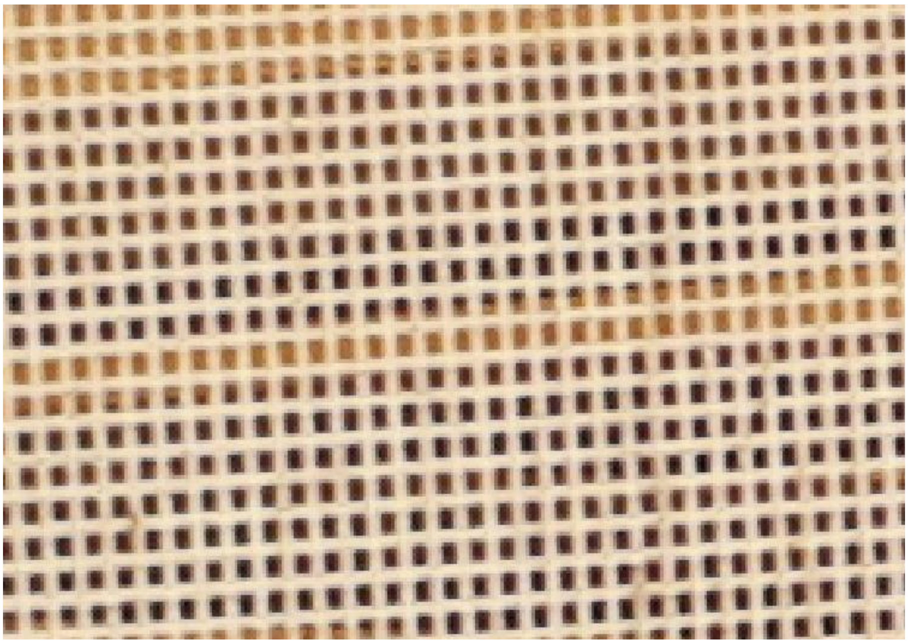


Figure 3.2.3-1:- The internal and side covers of the shelter

As we clearly see on the above picture the openings are uniformly distributed, and approximately 60% of the area of the roof covers are open. Which imply the permeability of the roof cover is 0,6% which is greater than 0,1%. That we can definitely say the skin is air permeable skin.

So we use super position method for determining the internal wind force coefficient for the loading on the inside skin, depending on the basic assumptions listed below;

- Considering that there is only an external wind load exerted on the roof covering while on empty canopy
- Considering that there is a combined action from the external and internal winds exerted on the roof covering while on fully blocked canopies

So having the above helpful assumptions we can determine the internal wind load coefficient by deducting the very first assumption from the second one for the specific pitch angle and degree of blockage.

ii. Wind loading on the sides of the roof cover

Most practitioners on structure believed that the sides of the roof cover should also be loaded by wind since the height found in between the top and the bottom roof cover is significant that the lateral wind load believed to affect the stability of the structure which in our case is 1.7 meter.

However, we do not have any provisions regarding to the condition we have. Hence the researcher has made some realistic assumptions;

- Considering the four sides of the roof cover as a vertical wall.
- Considering that the reference height taken for the wind load analysis kept to be the same.
- Considering the height of the side cover as of height of the wall.
- Neglecting the effect of the internal wind pressure and considering only of the external wind pressure.

3.2.3.3. Wind Load computation Chart

WIND LOAD

Wind load on the roof

$$F_w = C_s C_d \, . \, C_f \, . \, q_p(z_e) \, . \, A_{ref}$$

wind pressure on side cover of the roof

$$W_e = q_p(z_e) \, . \, Cp_e$$

$$C_s C_d = structural\ factor$$

$$C_f = force\ coefficient$$

$$A_{ref} = reference\ area$$

$$q_p(z_e) = peak\ velocity\ pressure$$

$$q_p(z_e) = \frac{\rho}{2} \, . \, V_p(z_e)^2$$

$$\rho = density\ of\ air$$

$$V_p(z_e) = peak\ wind\ velocity$$

$$V_p(z_e) = V_m(z_e) \, . \, G$$

Mean Wind Velocity

$$V_m(z_e) = C_r(z) . C_o(z) \, . \, V_b$$

$$G = Gust\ wind\ factor$$

$$G = \sqrt{1 + 7 \, . \, \frac{1}{C_o(z) \, . \ln(\frac{z}{z_0})}}$$

$$C_r(z) = roughness\ coefficient$$

$$C_0(z) = orographic/topographic\ coefficient$$

$$V_b = Basic\ wind\ velocity$$

$$V_b = C_{dir} \, . \, C_{season} \, . \, V_{b,0}$$

$$C_{dir} = directional\ factor$$

$$C_{season} = Seasonal\ factor$$

$$V_{b,0} = Fundamental\ value\ of\ basic\ wind\ velocity$$

3.3. Numerical Analysis on the underground tunnel/cavity

3.3.1. Rock mass classification

Rock mass classification schemes have been developing for over 100 years since Ritter (1879) attempted to formalize an Empirical approach to tunnel design in particular for determining support [24].

Rock mass classification is a process of placing a rock into groups or classes on defined relationships (Bieniawski, 1989) and assigning a unique description (or number) to it on the basis of similar properties/ characteristics such that the behavior of the rock mass can be predicted [24].

Rock mass is referred to an assemblage of rock material separated by rock discontinuities, mostly by joints, bedding planes, dyke intrusions and faults etc. [24].

Classification of rock mass improve the quality of site investigations by calling for a systematic identification and quantification of input data. A rational, quantified assessment is more valuable than a personal (non-agreed) assessment [24].

Most of the multi-parameter classification schemes Wickham et al (1972), Bieniawski (1973, 1989) and Barton et al (1974) were developed from civil engineering case histories in which all of the components of the engineering geological character of the rock mass [24].

Classifying the rock mass is done for different reasons, hear below is listed some of the uses of classifying the rock mass of a rock [1].

- Developed for estimation of tunnel support
- Used at project feasibility and preliminary design stages
- Simple checklist or detailed schemes
- *Used to develop a picture of the rock mass and its variability*
- Used to provide initial empirical estimation of tunnels support requirements
- *Are practical engineering tools which force the user to examine the properties of the rock mass*
- They do not replace the detailed design method
- *Used for classifying the condition of the in-situ rock.*

From the above statements that has made in italics form are the main objectives of classifying the rock mass for the underground tunnel.

There are different methods developed by different persons for classifying the rock mass, but for our case we choose one of the most used and multi parametric method which lead to the perfection [1].

Rock Mass Rating (RMR*)*

The RMR system or the Geomechanics classification was developed by Bieniawski during 1972-1973 in South Africa to assess the stability and support requirements of tunnels. The advantage of this system is that only a few basic parameters relating to the geometry and mechanical conditions of the rock mass are used [24].

To classify a rock mass, the RMR system incorporates the following six basic parameters (Bieniawski, 1989) [24].

- The uniaxial compressive strength of the intact rock (σc): for rocks of moderate to high strength, point load index is also acceptable (Bieniawski, 1989).
- Rock Quality Designation (RQD)
- Discontinuity spacing
- Condition of discontinuity surfaces
- Groundwater conditions
- Orientation of discontinuities relative to the engineered structure It does not include in-situ stress conditions.

Rock quality designation index (RQD) (Deere, 1967)

The aim is to provide a quantitative estimate of rock mass quality from drill core logs. It is defined as the percentage of intact core pieces longer than 100 mm in the total length of the core having core diameter of 54.7 mm.

Palmstrom (1982) suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the RQD may be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free rock masses is [24]

$RQD = 115 - 3.3\,J_v$ ……………….Eqn 3.13 (Source: - [24])

Where J_v is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count.

RQD is a measure of degree of fracturing of the rock mass and is aimed to represent the *in situ rock mass quality.*

Table 3.3.1-1: - classification of rock mass with RQD

RQD	**Rock Mass Quality**
< 25	**Very Poor**
25 – 50	**Poor**
50 – 75	**Fair**
75 – 90	**Good**
99 – 100	**Excellent**

RQD is used as an input parameter in RMR systems. It has a limitation to use it for rock mass classification in that it only considers the extent of fracturing of the rock mass and does not account for the strength of the rock or mechanical and other geometrical properties of the joints, and that's why RMR is used.

The following table in below is developed by Bieniawski, 1989, and are the detailed input parameters for classifying the rock mass using RMR method.

Table 3.3.1-2: - Inputs Parameters for RMR and RMR Classification

INPUT PARAMETERS TO RMR $_{1989}$

(from Bieniawski, 1989)

PARAMETER			Range of values // RATINGS						
1	Strength of intact rock material	Paint –load strength index	> 10 Mpa	4-10 Mpa	2-4 Mpa	1-2 Mpa	For this range unlaral comer.greength is prefemed		
		uniaxial com. Pressive strength	>250 Mpa	100-250 Mpa	50-100 Mpa	25-60 Mpa	5-25 Mpa	1-5 Mpa	<1 Mpa
	RATING		15	12	7	4	2	1	0
2	Drill care quality RQD		90-100 %	75-90 %	50-75 %	2-50%	< 25%		
	RATING		20	17	13	8	5		
3	Spacing of discontiruties		> 2m	0.6.2m	200-600 mm	60-200 mm	< 60 mm		
	RATING		20	15	10	8	5		
4	Condition of discon tinulties	Length, persistence	< 1m	1.3m	3-10 m	10-20m	>20m		
		Rating	6	4	2	1	0		
		Separation	none	<0.1 mm	0.1-1 mm	1-5 mm	>5 mm		
		Rating	6	5	4	1	0		
		Roughness	very rough	rough	Silghtly rough	Smooth	Sickersided		
		Rating	6	5	3	1	0		
		infilling (Gouge)	none	Hard filing		Soft filing			
			_	< 5mm	> 5 mm	< 5 mm	> 5 m		
		Rating	6	4	2	2			
		weathening	Unweathered	Slightly weathered	moderately weathered	highly weathered	decomposed		
		Rating	6	5	3	1	0		
5	Ground Water	inflow per 10m tunnel length	none	< 10 Liters /min	10-25 liters/min	25-125 liters /mm	> 125 Liters /Min		
		P_w / 01	0	0.0.1	0.1-0.2	0.2.05	> 0.5		
		General condtions	Completely dry	damp	wet	dripping	flawing		
		RATING	15	10	7	4	0		
pa = Joint water pressure at = major principal stress									

RATING ADJUSTMENT FOR DISCONTINUITY ORENTATION

		very favourable	Favourable	Fair	unfavourable	very unfavourable
RATINS	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	stopes	0	-5	-25	-50	-40

ROCK MASS CLASS DETERMINED FROM TOTAL RATINGS

Rating	100-81	80-61	60-41	40-21	< 20
Class No	I	II	III	IV	V
Description	VERY GOOD	GOOD	FAIR	POOR	VERY POOR

MEANING OF ROCK MASS CLASSES

Class No	I	II	III	Iv	V
Average stand up time	10 years for 15m span	6 months for 8mspan	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
cihesion of tge rock mass	> 400 kpa	300-400 kpa	200-300 kpa	100-200kpa	< 100kpa
Friction angle of the rock mass	$< 45^{0}$	$35\text{-}45^{0}$	$25\text{-}35^{0}$	$15\text{-}25^{0}$	$<15^{0}$

Hence classifying the Rock Mass of the underground tunnel;

Table 3.3.1-3:- Rock mass classification of the underground tunnel

Description	Value	Rate
a. Uniaxial compressive strength	18.5 Mpa	2
b. RQD	75 (Assumption)	17
c. Spacing of discontinuity	> 2 m	20
d. Condition of discontinuity		
- Length persistent	< 1 m	6
- Separation	None	6
- Roughness	Very rough	6
- Infilling (Gouge)	None	6
- Weathering	Slightly weathered	5
- Ground water	Completely dry	15
	TOTAL	**83**

Hence the rock founds with the total value of 83 that is classified to be class number 1 with a Rock Mass classification of a grade of very good. And also et al Federico Sani on their pilot study they found the Rock Mass Rating (RMR; Bieniawski 1989) of the massive basalt equals to 82 and scoriaceous basalt equals to 80 [11], which is quite near to our result and supports our result on rock mass classification too.

3.3.2. How to analyze the underground tunnel

Most of the under tunnels/cavities connecting the churches one another are loaded by the provided steel shelters. For instance the tunnel connecting Bete-Mariam and Bete-Mesquel is approximately 1.6 meter far away from one of the column shading Bete-Mariam, The tunnel so called the DARK tunnel (sign of hell) leading to the back courtyard of Bete-Abalibanos is approximately 1.2 meter far away from one of the column shading Bete-Abalibanos and one of the column shading Bete-Amannueal is directly placed on the other DARK tunnel connecting Bete-Merqorious and Bete-Amanueal.

Hence checking the cavities for the unfavorable vertical load exerted by the shelter is undoubtable. The tunnel found at a courtyard of Bete-Amanueal is the critical one out of the above described scenarios. So it is selected to be analyzed.

The analysis is basically focuses on determining the displacement and occurrence of cracks on any side interface of the tunnels especially at the roof and ground surface. The analysis is going to be done by modeling the tunnel on one of finite element software (DUCOM, COM3D) using a solid mechanics system. All the mechanical properties of the rock to be used in the finite element analysis is selected from table 3.3.2.1 and the analysis will also be done for the two scenarios regarding to their difference in mechanical properties and is summarized as follows;

Table 3.3.2-1:- Mechanical property of the rock for the numerical analysis

	Sample A	Sample B
Unit weight	$1900{,}5\ {}^{Kg}/_{m^3}$	$1647{,}2\ {}^{Kg}/_{m^3}$
Compressive strength	$18{,}53\ MP_a$	$6{,}5 MP_a$
Tensile strength	$1{,}9\ MP_a$	$0{,}7\ MP_a$
Young's Modulus	$2850\ MP_a$	$1204{,}7\ MP_a$
Poisons ratio	$0{,}12$	$0{,}27$

The mechanical property of the rock is taken from the study done on 2012,which is the most recent study I could ever get and it make things harder to find out the rate of degradation of the rock mass to evaluate the current condition of the rock. But we can assume that the rate of degradation from 2012 to 2017 is negligible as far as the degradation property of rock mass concerns. However for the sake of safety a 30 % reduction on the mechanical property of the rock is used for performing a numerical analysis on the underground tunnel, hence, $4{,}5\ Mpa$ of compressive strength and $0{,}45\ Mpa$ of tensile strength is used.

3.3.3. Loading on the tunnel

There are two types of loading on the tunnel, vertical loading on the roof and horizontal/lateral loading on the wall. Which are the critical loading conditions on tunneling. But for the time being we are only interested on the vertical loading since the cavity is newly loaded vertically and likely to find the response of the tunnel due to the additional loading coming from the shelters.

The most important potential loads acting on the underground structures are earth/rock pressure and water pressure. Whenever artificial cavities are excavated in rock the weight of the overlying rock layers will act as a uniformly distributed load on the deeper strata and consequently on the roof of the cavity [1].

By excavation of the cavity opportunity is given for deformation towards its interior in order to maintain the cavity the intrusion of the rock masses must be prevented by supporting structures and according to Terzagi, secondary rock pressure should be understood as the weight of a rock mass of a certain height above the tunnel, which were left unsupported would gradually dropout of the roof and the only consequence of installing no support props would be that this rock mass would fall into the cavity. Successive displacement would result in the gradual development of an irregular natural arch above the cavity without necessarily involving the complete collapse of the tunnel itself [1]. However the cavities found in Lalibela did not acquire this supporting systems for the past 900 years and we did load them with additional loading from the shelters. That critically need a structural assessment on the cavities.

The loading acting on the supports is referred to rock pressure. There is a rock pressure on the cavity even if we do not have those supporting systems, hence we consider that there is an imaginary support on the roof and we do follow all the procedures that the rock pressure theories takes account.

Rock pressure depend on not only on the quality of the rock and on the magnitude of stress and strains around the cavity, but also on the amount of time elapsing after the outbreak of the underground cavity [1].

In nature deep lying rocks are affected by the weight of the overlying strata and by their own weight. These factors develop stresses in the rock mass. In general every stresses produces a strain and displaces individual rock particles. But to be displaced a rock particle needs to have a space available for movement [1].

There are different rock pressure theories, with two distinguished parts differ by considering the height of the overburden above the cavity. And out of this two I have choose the theory that consider the height of the overburden above the cavity, since we are interested on evaluating the behavior of the roof after loading it vertically. And there are also different pressure theories which consider the overburden height, and out of them we choose the theory of *Bierbaumer's theory*. Which is the most used and easy theory for computation the vertical loading.

Bierbaumer's theory was developed during the construction of the great Alpine tunnels. The theory states that a tunnel is acted upon by the load of a rock masses bounded by a parabola of height $h = \propto . H$.

Where $\propto$ is the reduction factor, and it is equal to 1 for shallow overburden. Hence we do have to consider the overburden height of the rock while we model the tunnel on the finite software.

And the tributary area which is to be affected by the vertical loading is within the width B, where B is shown in the figure and is equal to $b + 2m\,.\tan(45 - \frac{\varphi}{2})$.

Where: - m is the height of the tunnel.

b is the width of the tunnel and.

φ is frictional angle of the rock.

B is effective width for loading

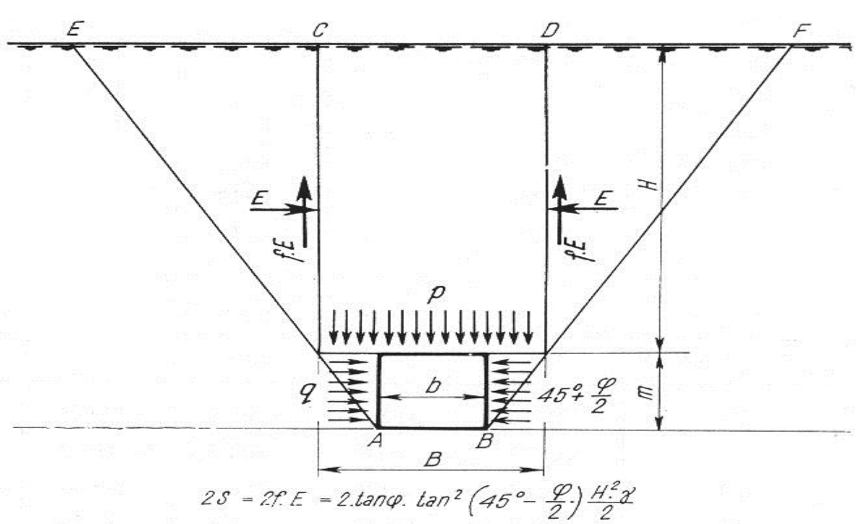


Figure 3.3.3-1:-Failure mechanism due to vertical loading (source: - [1])

3.4. Earthquake analysis

3.4.1. Seismicity of the area

According to the report paper of Margottin, there are no information on earthquakes affecting the town of Lalibela are available and this information encourages in the assumption that seismic risk is not a major problem for the area. However the pilot study done on Bete Gabriel-Rufael summarizes the seismic activities occurred near to Lalibela area.

The town of Lalibela is located approximately 70-80 Km west of the western escarpment of the Afar Rift system, an area where the seismicity is strongly concentrated. And with some uncertainties three recorded earthquakes are believed to be occurred near to the town of Lalibela. The first one occurred in 1942 (18 November) with its epicenter location 70-80 Km SSE of Lalibela with a magnitude of $5{,}3 - 5{,}9$ and it is related to the damage registered by the Lalibela churched to this earthquake. Although successively, in 1961, earthquake with a maximum stronger magnitude (6,1 near Wollo) stroke the area, by this time Lalibela were registered only minor effects and the strongest shock was felt with minor intensities. The third earthquake, felt in Lalibela with minor intensities corresponds to the 1971 seismic series (13-23 November) which occurred 10 Km south of Dese, about 130 Km SSE of Lalibela [11].

And this recorded earthquake activities could affect the structural stability of the shelters, and invokes another task to check the shelters for seismic activity.

CHAPTER 4:- ANALYSIS

4.1. Analysis on the Shelters

4.1.1. Wind Analysis for Shelters

Here below is the analysis done for Shelter-A, shading Bete-Mariam and Bete-Mesquel. It's a detail manual wind loading analysis focusses on determining the wind force on the canopy roof and wind pressure on the sides of the roof. Right after the wind loading analysis, modeling and analyzing the shelter is performed on SAP2000V16 and finally extracting and summarizing the vertical reaction at the base of each column is done, so that we could tell that if wind loading is critical on the base columns.

The wind load analysis for the remaining three shelters are attached on ANNEX-A, but the results are discussed in detail at the end of these chapter.

The wind loading analysis is done on two scenario scenes, one is using the deign wind speed for the characteristic values correspond to is annual probability of exceedance of 0.05 which corresponds to a mean return period of 20 years, and the other is using the already experienced wind speed by the year 2013, which is the 10 minute annual maximum basic wind speed measured at 10 meter height.

For both scenarios there will be analysis to be done on the two significant wind directions, 0^0 and 180^0 except to that of the shelter shading Bete-Amanueal. This wind directions are used because the wind is too significant while it exerts on parallel to the slope of the roof, and most of the practitioners and scholars including Eurocode doesn't consider the wind direction on the other remaining (90^0 and 270^0) wind directions and whenever it is needed to be checked the pitch angle of the roof has to be made 0^0.

Wind Analysis for Shelter - A, on Bete-Mariam and Bete-Mesquel

- Details of the shelter
  - Length = $33\ m\ = b$
  - Width = $30\ m\ = d$
  - Slope = 5.5 % and pitch angle = 3.15^0
  - Reference height = $6\ m$
  - Terrain category = II
  - Height between the two top and bottom cover = $1{,}5m$

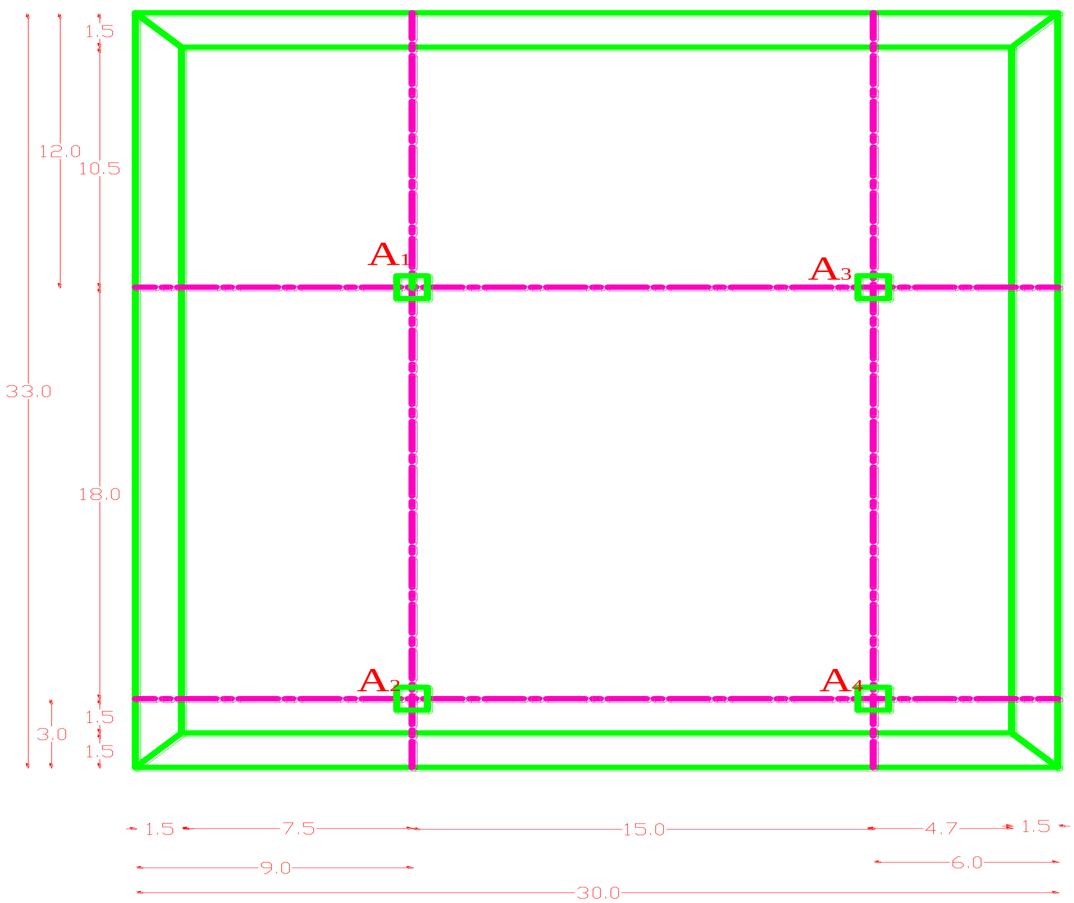


Figure 4.1.1-1:- AutoCAD drawing for Shelter-A roof plan

Solution

And the terrain of Lalibela can be categorized as terrain category II, Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights, having Z_0 = 0.05 m, Z_{min} = 2 m, Z_{max}= 200 m and reference height Z= 6 m.

1. Wind load Analysis using design wind speed

Determining the basic wind velocity ($\mathcal{V}_b$)

$$\mathcal{V}_b = C_{dir} \, . \mathrm{C}_{season} . \ \mathcal{V}_{b,o}$$

$$= 1 * 1 * 29{,}5 \ m/s$$

$$= 29{,}5 \, m/s$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z) \ . \mathrm{Co}(Z) \ . \ \mathcal{V}_b$$

$$\mathrm{Cr}(Z) = \mathrm{Kr} \, . \, \ell\mathrm{n} . \left({}^{Z}/_{Z0}\right) \quad \text{For } Z \geq Z_{min}$$

$$Kr = 0.19 . \left({}^{Z_o}/_{Z_{0,II}}\right)^{0.07}$$

$$Kr = 0.19 . \left({}^{0.05}/_{0.05}\right)^{0.07}$$

$$= \underline{\underline{\mathbf{0.}19}}$$

$$\mathrm{Cr}(Z) = 0.19 \, . \, \ell\mathrm{n} \, . \left({}^{6}/_{0.05}\right)$$

$$= \underline{\underline{0.9096}}$$

$$\mathrm{Co}(Z) = 1 + 2 \, . S \, . \Phi \quad , \quad \text{for } 0.05 < \Phi < 0.3 \text{ for shallow where} L_e = L_u .$$

The figure shown in below is the topographic section detail of Shelter-A taken from ARCCH which is prepared by TEPRIN ASSOCIATES as a working drawing. Which helps us to find the vertical elevation and the horizontal data for determining the topographic factor.

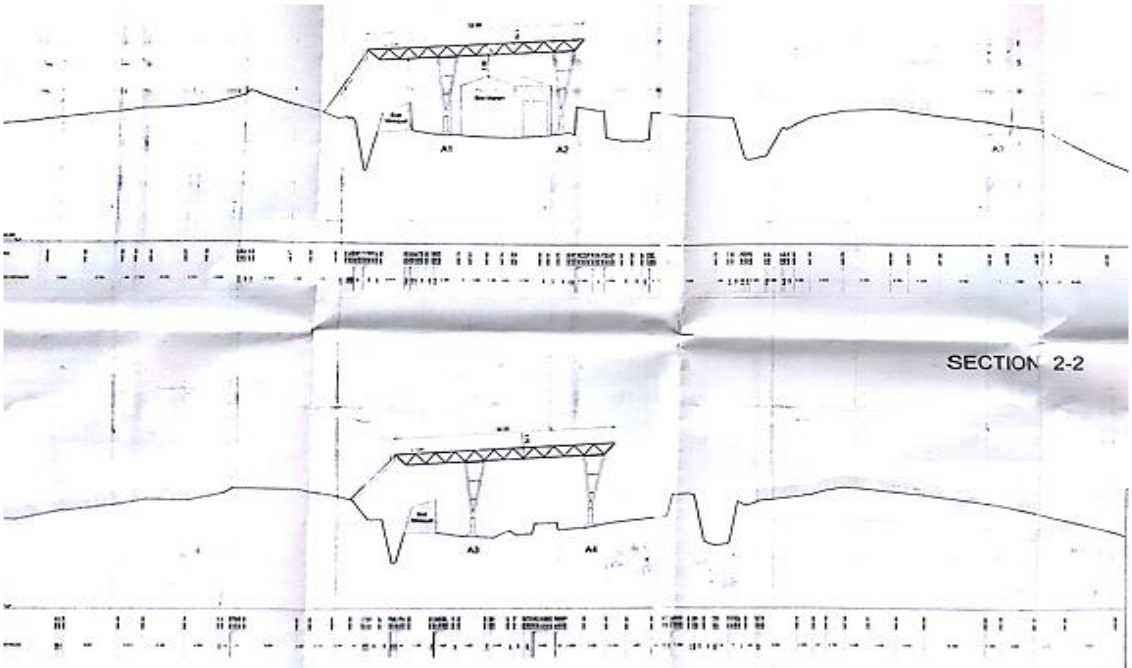


Figure 4.1.1-2:- section drawing for shelter-A (source from ARCCH, working drawing)

-Wind loading along 180^0

From the section drawing $H = 13{,}144\ m$ and $Lu = 102{,}47\ m$

$$\Phi = \frac{H}{L_u} = \frac{13.144}{102.47} = 0{,}128$$

$$S = A\,.\,e^{(B\,\cdot\,\frac{X}{L_u})}$$

Conditions to be fulfilled

For the range $-1{,}5 \leq \frac{X}{L_u} \leq 0$ and $0 \leq \frac{Z}{L_e} \leq 2{,}0$

$\frac{X}{L_u} = \frac{-120}{102.47} = -1.1$ (satisfied)

$\frac{Z}{L_e} = \frac{6}{102.47} = 0.06$ (satisfied)

$$A = 0{,}1552\,.\left(\frac{Z}{L_e}\right)^4 - 0{,}8575\,.\left(\frac{Z}{L_e}\right)^3 + 1{,}8133\,.\left(\frac{Z}{L_e}\right)^2 - 1{,}9115\,.\left(\frac{Z}{L_e}\right)^1$$

$$= 0{,}1552 \,.\, (0.06)^4 - \; 0{,}8575 \,.\, (0.06)^3 + 1{,}8133 \,.\, (0.06)^2 - 1{,}9115 \,.\, (0.06)^1$$

= 0.9

$$B = 0{,}354 \,.\left(\frac{Z}{L_e}\right)^2 - \; 1{,}0577 \,.\left(\frac{Z}{L_e}\right)^1 + 2{,}6456$$

$$= 0{,}354 \,.\, (0.06)^2 - \; 1{,}0577 \,.\, (0.06)^1 + 2{,}6456$$

= 2.58

S = 0.05

Hence

$$\text{Co}(Z) = 1 + 2 \,.\, 0{,}05 \,.\, 0{,}128 \quad = 1{,}01$$

-Wind loading along 0^0

From the section drawing $H = 4{,}49\, m$ and Lu $= 25{,}48\, m$

$$\Phi = \frac{H}{L_u} = \frac{4.49}{102.47} = 0.176$$

$$S = A \,.\, e^{\left(B \cdot \frac{X}{L_u}\right)}$$

Conditions to be fulfilled

$\frac{X}{L_u} = \frac{-35}{25.48} = -1{,}37$ (satisfied)

$\frac{Z}{L_e} = \frac{6}{25.48} = 0{,}235$ (satisfied)

$$A = 0{,}1552 \,.\left(\frac{Z}{L_e}\right)^4 - \; 0{,}8575 \,.\left(\frac{Z}{L_e}\right)^3 + 1{,}8133 \,.\left(\frac{Z}{L_e}\right)^2 - 1{,}9115 \,.\left(\frac{Z}{L_e}\right)^1$$

$$= 0{,}1552 \,.\, (0{,}235)^4 - \; 0{,}8575 \,.\, (0{,}235)^3 + 1{,}8133 \,.\, (0{,}235)^2 - 1{,}9115 \,.\, (0{,}235)^1$$

$$= 0.65$$

$$B = 0{,}354 \,.\left(\frac{z}{L_e}\right)^2 - 1{,}0577 \,.\left(\frac{z}{L_e}\right)^1 + 2{,}6456$$

$$= 0{,}354 \,.(0{,}235)^2 - 1{,}0577 \,.(0{,}235)^1 + 2{,}6456$$

$$= 2{,}41$$

$$S = 0{,}0239$$

Hence

$$\mathrm{Co}(Z) = 1 + 2 \,. 0{,}05 \,. 0{,}128 = 1{,}01$$

As we can see from the results obtained for both wind directions we can use $\mathrm{Co}(Z)$ equal to 1,01.

Therefore,

$$\mathcal{V}_m(Z_e) = 0{,}9096 \,.\ 1{,}01 \,.\ 29{,}5 = \mathbf{27{,}14\ m/s}$$

Determining the peak wind velocity ($\mathcal{V}_p(Z_e)$)

$$\mathcal{V}_p(Z_e) = \mathcal{V}_m(Z_e)\,.G$$

Where:-

$$G = \sqrt{1 + 7.\frac{1}{1{,}01 \,.\ln\left(6/_{0.05}\right)}} = 1{,}565$$

$$\boldsymbol{\mathcal{V}_p(Z_e) = 27{,}14\,\frac{m}{s}\,.\,1{,}565 = 42{,}48,\frac{m}{s}}$$

Determining the peak velocity pressure (q_p)

$$q_p = \frac{\rho}{2}.\ (\mathcal{V}_p(Z_e))^2$$ Pr EN 1991 1-4 2004

$$= \frac{0.94}{2}\,.(42{,}48)^2$$

$$\boldsymbol{q_p = 0{,}848\ \frac{KN}{m^2}}$$

Determining wind forces

i. **Wind load on external part of the roof covers**

The wind force for the whole structure or a structural component should be determined by using one of the recommended method in Pr EN 1991 -1-4: 2004 which is using force coefficient method.

$$F_w = C_s \,.\, C_d \,.\, C_f \,.\, q_p(z_e).\, A_{ref}$$

Determining structural factors ($\mathbf{C_s C_d}$)

C_s Cd = is the structural factors which are the size factor and dynamic factor respectively

= 1 for structure which are not susceptible to turbulence induced vibration. The recommended value of $C_s C_d$ = 1 from Pr EN 1991 -1-4: 2004, 6.2 (1) (a)

Determining the force coefficient (C_f) on the external most rigid roof cover

- Roof slope = 5.5%
- Roof (pitch) angle (α) = $\tan^{-1} \frac{5.5}{100} = 3.15^0$
- Reference area of the roof is = 33 * 30 = 990 m^2
- Determining degree of blockage:- Has negligible effect
- Using linear interpolation for calculating the value of C_f for pitch angle of α $=3.15^0$

⇒ At $\alpha = 3.15^0$

$$\left.\begin{array}{ll} \alpha = 0^0 & -0{,}5 \\ \alpha = 3.15 & C_{f\,\text{up}} \\ \alpha = 5^0 & -0{,}7 \end{array}\right\} \quad \frac{0-3.15}{0-5} = \frac{-0{,}5-C_{fu}}{-0{,}5+0{,}7}$$

$$\boldsymbol{C_{f,exernal} = -0,626}$$

Therefore the net upwind force coefficient $\boldsymbol{C_{f,external} = -0,626}$

Therefore,

The overall wind load on the external roof cover becomes

$$F_w = 1 . (-0.626) . 0{,}848 . \ 990$$

$$\boldsymbol{F_w = -525{,}606 \quad KN}$$

ii. Wind load on side roof covers

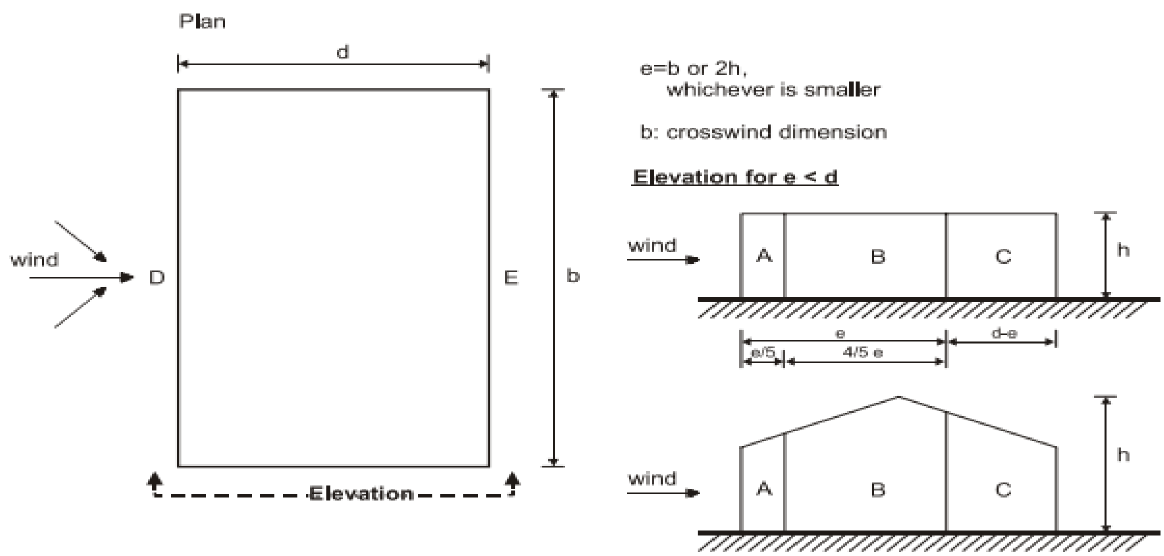


Figure 4.1.1-3:- wind loading keys on vertical wall (Source:-Pr EN 1991-1-4)

Based on the assumptions made on the methodology part, the wind loading analysis is be like;

Basic data's:-

- Height of roof $= \ 1{,}7 \ m$
- Reference height $= \ 6 \ m$
- $b \ = \ 30 \ m$
- $d \ = \ 33 \ m$
- e = b or 2h (whichever is the smaller)

$$= \ 30 \ m \ or \ 2 \ . \ 1{,}5 = 3 \ m$$

$$e \ = \ 3 \ m$$

Determining areas of the regions:-

- $A = 1{,}5\,.\frac{3}{5} = 0{,}9\ m^2$ $(A < 1\ m^2)$ then use 1 m^2
- $B = \frac{4}{5}\,.1{,}5\,.3 = 3{,}6m^2$ $(1\ m^2 < A < 10\ m^2)$
- $C = (33 - 3)\,.1{,}5 = 45m^2$ $(> 10\ m^2)$
- $D = 30\,.1{,}5 = 45\ m^2$ $(> 10\ m^2)$
- $E = 30\,.1{,}5 = 45\ m^2$ $(> 10\ m^2)$

Table 4.1.1-1:-Recommended values for external pressure coefficients, for vertical walls of rectangular in plan (source: - Pr EN 1991-1-4)

Zone	A		B		B		D		E	
h/d	$Cpe,_{10}$	$Cpe,_{1}$	$Cpe,_{10}$	$Cpe,_{1}$	$Cpe,_{10}$	$Cpe,_{1}$	$Cpe,_{10}$	$Cpe,_{1}$	$Cpe,_{10}$	$Cpe,_{1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
$\leq$0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

$$\frac{h}{d} = \frac{1.5}{30} = 0{,}05 \leq 0{,}25$$

And for the regions with the condition of $(1\ m^2 < A < 10\ m^2)$ we have a formula of;

$Cp_e = Cp_{e,1} - (Cp_{e,1} - Cp_{e,10})log10^A$……….. …….. (Figure 7.2 of Pr EN 1991-1-4)

For region A

$$Cp_{e,1} = -1{,}4\ and\ Cp_{e,10} = -1{,}2$$

$$Cp_{e,0,9} = -1{,}4$$

For region B

$$Cp_{e,1} = -1{,}1\ and\ Cp_{e,10} = -0{,}8$$

$$Cp_{e,3,6} = -1{,}1 - (-1{,}1 - (-0{,}8))log10^{3.6}$$

$$Cp_{e,3,6} = -0{,}933$$

For region C

$$Cp_{e,10} = -0{,}5$$

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore, the wind pressure in each distinguished regions become;

We have to note that the effective area to be loaded is only 40% of the total area of the side cover because of the uniformly distributed openings.

$$W_e = q_p \, . Cp_e$$

Wind pressure for region A

$$W_{e,A} = \left(0{,}848 \, . (-1{,}4)\right) . 0{,}4 = -0{,}474 \, \frac{KN}{m^2}$$

Wind pressure for region B

$$W_{e,B} = (0{,}848 \, . (-0{,}933)) \, . 0{,}4 = -0{,}32 \, \frac{KN}{m^2}$$

Wind pressure for region C

$$W_{e,C} = -0{,}17 \, \frac{KN}{m^2}$$

Wind pressure for region D

$$W_{e,D} = +0{,}23 \, \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}102\ \frac{KN}{m^2}$$

2. Wind load Analysis with the already experienced wind speed

By the year 2013 the Lalibela region experienced 28 m/s, 10 minute gust wind speed measured at 10 meter height, therefore we use the wind speed as it is for the sake of checking the structural stability of the structure.

Determining the basic wind velocity ($\mathcal{V}_{\mathbf{b}}$)

$$\mathcal{V}_b\ = C_{dir}\ .C_{season}.\ \mathcal{V}_{b,o}$$

$$= 1 *\ 1 * 28\ m/s$$

$$= 28 \text{ m/s}$$

Determining the mean wind velocity ($\mathcal{V}_{\mathbf{m}}$)

$$\mathcal{V}_m(Z_e)\ = \mathrm{Cr}(Z\ )\ .\mathrm{Co}(Z\ ).\ \mathcal{V}_b$$

$$\mathcal{V}_m(Z_e)\ =\ \ 0{,}9096\ .\ 1{,}01\ .\ 28 = 25{,}72 \text{ m/s}$$

Determining the peak wind velocity ($\mathcal{V}_{\mathbf{p}}(\mathbf{Z_e})$)

$$\mathcal{V}_p(Z_e)\ = \mathcal{V}_m(Z_e)\ .G$$

Where:-

$$\mathcal{V}_\mathrm{p}(\mathrm{Z_e}) = 25{,}72\frac{\mathrm{m}}{\mathrm{s}}.1{,}565 = 40{,}25/s$$

Determining the peak velocity pressure (q_p)

$$q_p\ = \frac{\rho}{2}.\ (\mathcal{V}_p(Z_e))^2$$

$$= \frac{0.94}{2}\ .(40{,}25)^2$$

$$q_p = 0{,}7615 \frac{KN}{m^2}$$

Determining wind forces

i. **Wind load on top of roof covers**

$$F_w = C_s \, . \, C_d \, . \, C_f \, . \, q_p(z_e) . \, A_{ref}$$

The overall wind load on the external roof cover becomes

$$F_w = 1 \, . \, (-0.626) \, . \, 0{,}7615 \, . \, 990$$

$$F_w = -471{,}9 \text{ KN}$$

ii. **Wind load on side roof covers**

For region A

$$Cp_{e,1} = -1{,}4 \text{ } and \text{ } Cp_{e,10} = -1{,}2$$

$$Cp_{e,0,9} = -1{,}4$$

For region B

$$Cp_{e,1} = -1{,}1 \text{ } and \text{ } Cp_{e,10} = -0{,}8$$

$$Cp_{e,3,6} = -0{,}933$$

For region C

$$Cp_{e,10} = -0{,}5$$

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore,

The wind pressure in each distinguished regions become;

$$W_e = q_p \,.\, Cp_e$$

Wind pressure for region A

$$W_{e,A} = \left(0{,}7615 \,.\, (-1{,}4)\right) .\, 0{,}4 = -0{,}43 \; \frac{KN}{m^2}$$

Wind pressure for region B

$$W_{e,B} = (0{,}7615 \,.\, (-0{,}933)) \,.\, 0{,}4 = -0{,}28 \; \frac{KN}{m^2}$$

Wind pressure for region C

$$W_{e,C} = -0{,}15 \; \frac{KN}{m^2}$$

Wind pressure for region D

$$W_{e,D} = +0{,}2 \; \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}0914 \; \frac{KN}{m^2}$$

Modeling and analyzing the structure on SAP2000V16

One of the structural integrated software of finite element which is SAP2000V16 is used for modeling and analyzing of the steel shelters. Each construction material, structural elements, and the structural system and integrity is taken from the structural detail of the shelters which was a working drawing that provided me by the ARCCH. All the input data to be feed in the software are described below;

i. Material property of the structural elements including the connections.
   - All the structural elements are steel tubes with grade of S-450

Yield strength (F_y) = $\frac{F_u}{\delta_s} = \frac{450}{1.15} = 391.304$ Mpa

Ultimate strength (F_y) = 450 Mpa

ii. Section property of the steel materials

- Bottom column :- tubular section 508 x 12.5 mm
- Columns :- tubular section 300 x 15 mm
- Column braces :- tubular section 164 x 6.7 mm
- Top and Bottom chord :- tubular section 102x4 mm
- Diagonal trusses :- tubular section 80 x 4 mm

iii. Imposed loads on the structures

- Dead load

The own self weight of the shelter to be included on the model by making the self-weight multiplier equals to 1.

From the skin membranes

The skin membrane is PVC-Coated Polyester Fabrics, which is believed to have high tensile strength probably five times greater than steel.
From the design report of TEPRIN ASSOCIATE the total weight of the roof cover is [25]

$$W = 0{,}15 \frac{KN}{m^2}$$

From the connection (balls and cones)

Weight of the ball;

Maximum diameter of the ball sphere = 160 mm and thickness = 4mm

Volume of hollow sphere =

$$\mathrm{V} = \frac{4}{3}\pi\ (R^3 - r^3)$$

Where R =external diameter = $160\ mm$

R = internal diameter = $160 - 4 = 156\ mm$

$$\mathrm{V} = \frac{4}{3}\pi\ (0{,}16^3 - 0{,}156^3)$$

$$\mathrm{V} = 0{,}00125\ m^3$$

$$\mathrm{W} = \gamma\ .\ \mathrm{V}$$

$$\mathrm{W} = 78\ \frac{KN}{m^3}\ .\ 0{,}00125\ m^3 = 0.098\ KN$$

Weight of the cone;

There are maximum of 8 cones per connection, and a single cone weights 1 kg;

Total mass of the cone = $8\ .\ 1\ kg = 8\ kg$

Weight of the cone $= \frac{8\ .9.81}{1000} = 0.08\ KN$

Therefore, the total weight of the connection is

$$W_T = \ 0{,}098 + 0{,}08 = 0{,}178\ KN$$

From the base plates and counter weights;

- weight of base plate with 20mm thickness

Weight = $76{,}9729\ \ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}02 = 4{,}99\ KN$

- weight of a counter mass plate

Weight = $76{,}9729\ \ .\ [(1{,}8\ .\ 1{,}8) - (0{,}67\ .\ 0{,}67)].\ 0{,}01 = 2{,}148\ KN$

And the number of counter weight varies from column to column, so the weight of the counter mass plate in each column is;

At base of the column A1 and A3 = $6\ \ .\ \ 2{,}148 = 12{,}89\ KN$

At base of the column A2 and A4 = $60\ \ .\ \ 2{,}148 = 128{,}9\ KN$

- weight of L-shaped plate

Weight = $76{,}9729\ \ .\ [4\ .\ (0{,}11 + 0{,}11 + 0{,}2]\ .\ 1{,}8\ .\ 0{,}01 = 3{,}49\ KN$

From the concrete foundation

A concrete foundation is used in every column with different thickness but with similar dimensions, it is used as a counter wait and also as surface leveling so that the counter mass plates could place simply. The concrete is grade C-25 Mpa that is found from those involved in the project. The thickness of the pad is taken to be the average of the four side thickness.

At base of the column $A1 = 25 \,.\, 1{,}8 \,.\, 1{,}8 \,.\, 0{,}25 = 20{,}25\ KN$

At base of the column $A2 = 25 \,.\, 1{,}8 \,.\, 1{,}8 \,.\, 0{,}4 = 32{,}4\ KN$

At base of the column $A3 = 25 \,.\, 1{,}8 \,.\, 1{,}8 \,.\, 0{,}35 = 28{,}35\ KN$

At base of the column $A4 = 25 \,.\, 1{,}8 \,.\, 1{,}8 \,.\, 0{,}3 = 24{,}3\ KN$

Therefore, the total weight in the base column is;

At base of the column $A1 = 20{,}25 + 12{,}89\ + 4{,}99 + 3{,}49 = 38{,}13\ KN$

At base of the column $A2 = 32{,}4 + 128{,}9\ + 4{,}99 + 3{,}49 = 169{,}78\ KN$

At base of the column $A3 = 28{,}35 + 12{,}89\ + 4{,}99 + 3{,}49 = 49{,}72\ KN$

At base of the column $A4 = 24{,},3 + 128{,}9\ + 4{,}99 + 3{,}49 = 161{,}68\ KN$

The tensile strength of the rock or the concrete doesn’t included in the analysis not because we didn’t know we should consider but it depends on the bond strength between the two Medias, and if it’s good so, we also depends on the tensile strength of the rock and the concrete.

However we assume that no matter how strong the tensile capacity of the rock or the concrete is, they both depends on the bond strength between them and is believed to be time and load dependent and could lose its strength due to the repeated/cyclic and variable load from the wind. Hence we didn’t take the tensile strength as a counter reaction for the shelter on the analysis.

- Live load

From Pr EN 1991-1-1-2009 table 6.9 if the roof is categorized as Category H, roofs not accessible except for normal Maintenance and repair the imposed loads are given in table 6-10, for category H, $\boldsymbol{q_k}$ may be selected within the rage of 0.0 KN/M^2 to 1 KN /M^2 and $\boldsymbol{Q_k}$ may be selected within the range of 0.9 KN to 1.5 KN. And the recommended values are $\boldsymbol{q_k}$= 0.4 KN/M^2 and $\boldsymbol{Q_k}$= 1 KN.

- Wind load

Is considered as a variable load and taken from the previous analysis and feed in the model.

The figure shown below is the 3D modeling of Shelter-A on SAP2000V16.

- Load combination used for Analysis

1.5 W.L + D.L in KN

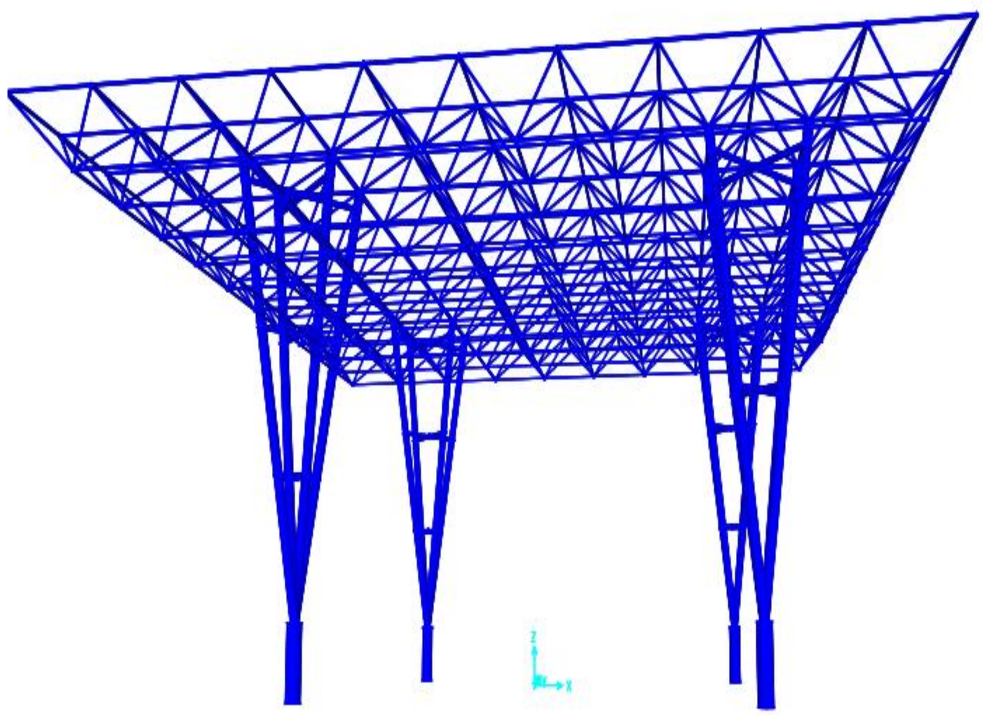


Figure 4.1.1-4:- SAP2000V16 model for Shelter-A

And the analysis output is tabulated as;

Table 4.1.1-2:-Analysis result of Shelter-A

FOR SHELTER-A

Wind Direction, 0^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
A1	202.9	-73.3	-69.993	92.95	132.907
A2	263.01	-242.295	-215.746	-100.4325	47.264
A3	180.5	-57.2	-55.456	94.7	125.044
A4	237.83	-173.194	-135	-21.961	102.83
Total	**884.24**	**-545.989**	**-476.195**	**65.2565**	**408.045**

Wind Direction, 180^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
A1	202.9	-398.2	-346.43	-394.4	-143.53
A2	263.01	74.3	63.23	374.46	326.24
A3	180.5	-279.55	-237.42	-238.825	-56.92
A4	237.83	58.17	48.6	325.085	286.43
Total	**884.24**	**-545.28**	**-472.02**	**66.32**	**412.22**

While performing the analysis using design wind speed, it's used 20 years of return period with 0.05 probability of exceedance, in which the shelter is designed for. And note that there are no directional factor used for the analysis since the analysis is performed in the direction where the maximum wind speed experienced.

And likewise the analysis result for the rest of the three shelters is summarized and shown in the tables here below.

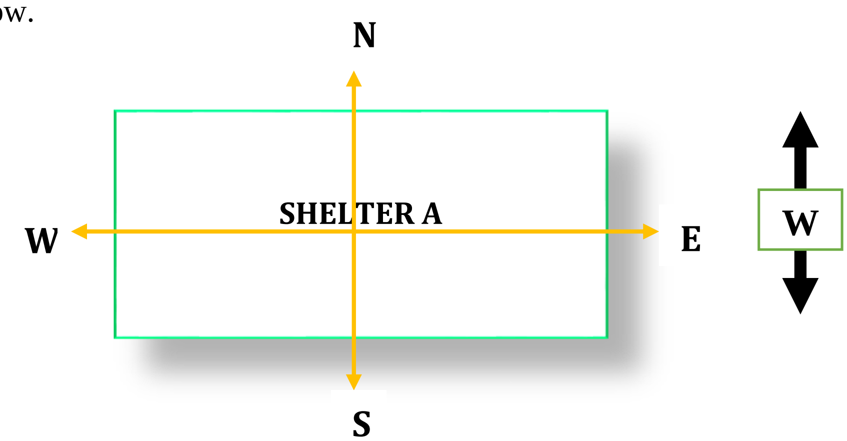


Table 4.1.1-3:-Analysis result of Shelter-B

FOR SHELTER - B

Wind Direction, 0^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
B1	258.411	-50.406	-56	182.802	202.411
B2	261.854	-270.3	-219	-143.596	42.854
B3	259.155	-60.5	-63.5	168.405	195.655
B4	267.236	-299	-243	-181.264	24.236
Total	**1046.656**	**-680.206**	**-581.5**	**26.347**	**465.156**

Wind Direction, 180^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
B1	258.411	-275.41	-242	-154.704	16.411
B2	261.854	-71	-48	155.354	213.854
B3	259.155	-278.11	-243.3	-158.01	15.855
B4	267.236	-78.33	-54.34	149.741	212.896
Total	**1046.656**	**-702.85**	**-587.64**	**-7.619**	**459.016**

While performing the analysis using design wind speed, it's used 20 years of return period with 0.05 probability of exceedance, in which the shelter is designed for. And note that directional factor is used for the analysis since the analysis is performed in the direction different from the direction where the maximum wind speed experienced.

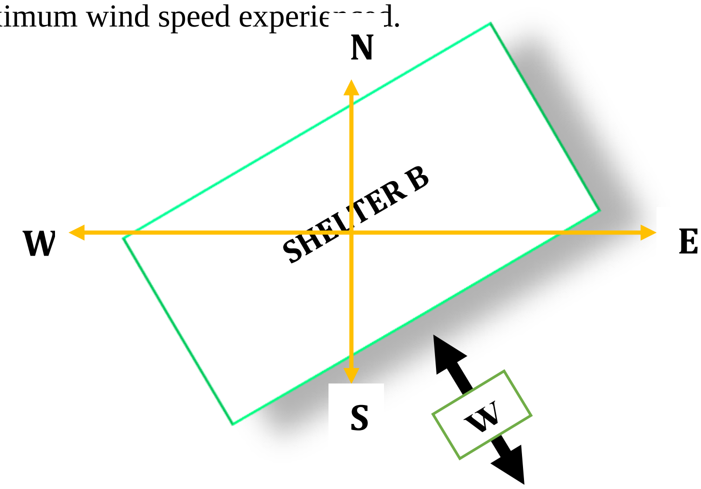


Table 4.1.1-4:-Analysis result of Shelter-C

FOR SHELTER - C

Wind Direction, 0^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
C1	300.611	-312.48	-215.2	-168.109	85.411
C2	294.93	-312.468	-215.2	-173.772	79.73
C3	188.413	-9	-6.83	174.913	181.583
C4	145.48	-9	-6.83	131.98	138.65
Total	**929.434**	**-642.948**	**-444.06**	**-34.988**	**485.374**

Wind Direction, 180^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
C1	300.611	-67	-52.3	200.111	248.311
C2	294.93	-67	-52.3	194.43	242.63
C3	188.413	-64.5	-48.4	91.663	140.013
C4	145.48	-64.5	-48.4	48.73	97.08
Total	**929.434**	**-263**	**-201.4**	**534.934**	**728.034**

While performing the analysis using design wind speed, it's used 20 years of return period with 0.05 probability of exceedance, in which the shelter is designed for. And note that directional factor is used for the analysis since the analysis is performed in the direction different from the direction where the maximum wind speed experienced.

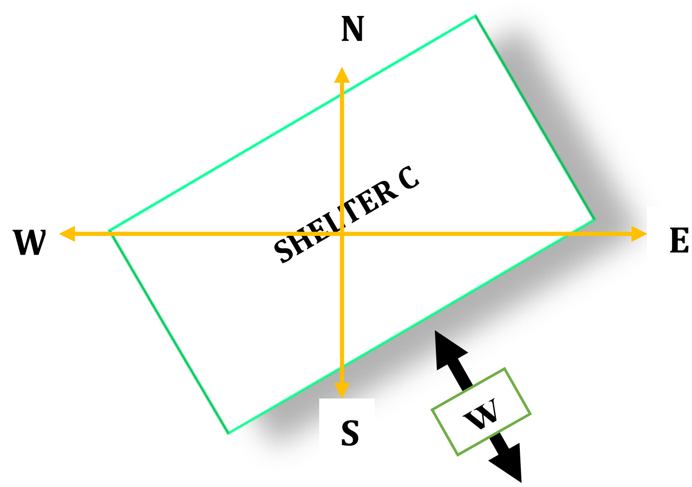


Table 4.1.1-5:-Analysis result of Shelter-D

FOR SHELTER - D

Wind Direction, 90^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
D1	205.786	-97.14	-91.4	60.076	114.386
D2	204.166	-10.4	-9.11	188.566	195.056
D3	199.306	-97.12	-91.4	53.626	107.906
D4	198.496	-10.4	-9.11	182.896	189.386
Total	**807.754**	**-215.06**	**-201.02**	**485.164**	**606.734**

Wind Direction, 270^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
D1	205.786	-14.68	-13.43	183.766	192.356
D2	204.166	-92.82	-87.1	64.936	117.066
D3	199.306	-14.68	-13.43	177.286	185.876
D4	198.496	-92.82	-87.1	59.266	111.396
Total	**807.754**	**-215**	**-201.06**	**485.254**	**606.694**

Wind Direction, 0^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
D1	205.786	-121	-110.35	24.286	95.436
D2	204.166	-102.42	-98	50.536	106.166
D3	199.306	-31.067	-27	152.7055	172.306
D4	198.496	-13.4	-15	178.396	183.496
Total	**807.754**	**-267.887**	**-250.35**	**405.9235**	**557.404**

Wind Direction, 180^0

Column Name	Dead Load (D.L)	Wind Load (W.L)(KN)		1.5 W.L + D.L(KN)	W.L + D.L(KN)
		with design wind Speed	with the experienced wind Speed	with design wind Speed	with the experienced wind Speed
D1	205.786	-31.067	-27	159.1855	178.786
D2	204.166	-13.4	-15	184.066	189.166
D3	199.306	-121	-110.35	17.806	88.956
D4	198.496	-102.42	-98	44.866	100.496
Total	**807.754**	**-267.887**	**-250.35**	**405.9235**	**557.404**

FOR TUNNEL LOADING

FOR SHELTER - D

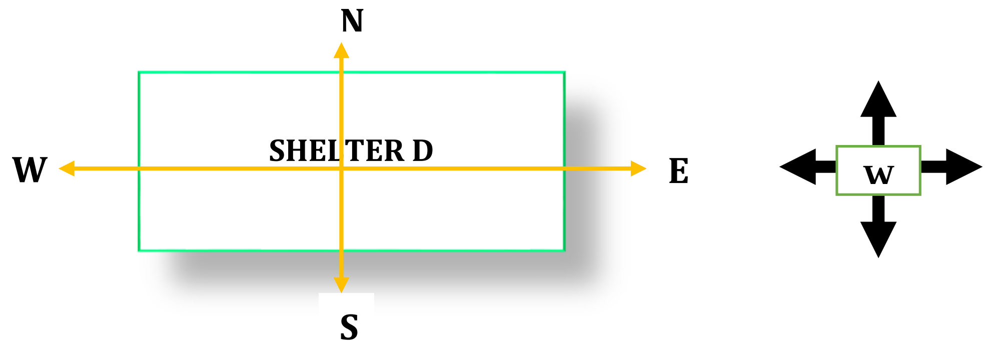
Wind Direction, 0^0

Column Name	Dead Load (D.L)	1.35 D.L + 1.5 W.L+1.05L.L(KN)
		with design wind Speed
D1	205.786	468.219

Wind Direction, 180^0

Column Name	Dead Load (D.L)	1.35 D.L + 1.5 W.L+1.05L.L (KN)
		with design wind Speed
D1	205.786	257.748

Analysis is performed in all possible directions where maximum effect could happen, the wind analysis is performed for 20 years of return period with the corresponding probability of exceedance of 0.05, in which the shelter is designed for. And while performing analysis along 0^0 (South) and 180^0 (North) direction the pitch angle is made to be 0^0. And it is noted that there is no directional factor used for the analysis since the analysis is performed in the direction where the maximum wind speed experienced. And also analysis is performed for the remaining directions 90^0 (Western) and 270^0 (Eastern) direction using the respective pitch angle, and note that directional factor is used for the analysis since the analysis is performed in the direction different from the direction where the maximum wind speed experienced.



4.1.2. Checking the Capacity of the structural members

The capacity of the shelters are checked for the design load using the unfavorable load combination of dead load and wind pressure which is $D.L + 1{,}5\, W.L$, the wind pressure is applied on the distinguished regions that are clearly shown on Table 7.6 of Pr EN 1991-1-4:2004. The demand capacity ratio is extracted from the design that is performed on SAP2000V16, and some of the results are shown on the table below and the 3D design output with color sparks has also shown below for further illustration, if red color has displayed on the analysis it means the demand capacity ratio does exceed the limitation i.e. $0{,}95$, and we can say the structural member fails to resist the design load.

Table 4.1.2-1:- Demand Capacity Ratio for some structural members of Shelter-A

For Top Chord	For Bottom Chord	For Diagonal Chord	For Column
0,009	0,011	0,026	0,088
0,11	0,056	0,058	0,061
0,045	0,034	0,014	0,029
0,138	0,073	0,069	0,051
0,84	0,336	0,014	0,02
0,77	0,228	0,074	0,008
0,203	0,227	0,09	0,015
0,269	0,044	0,01	0,036

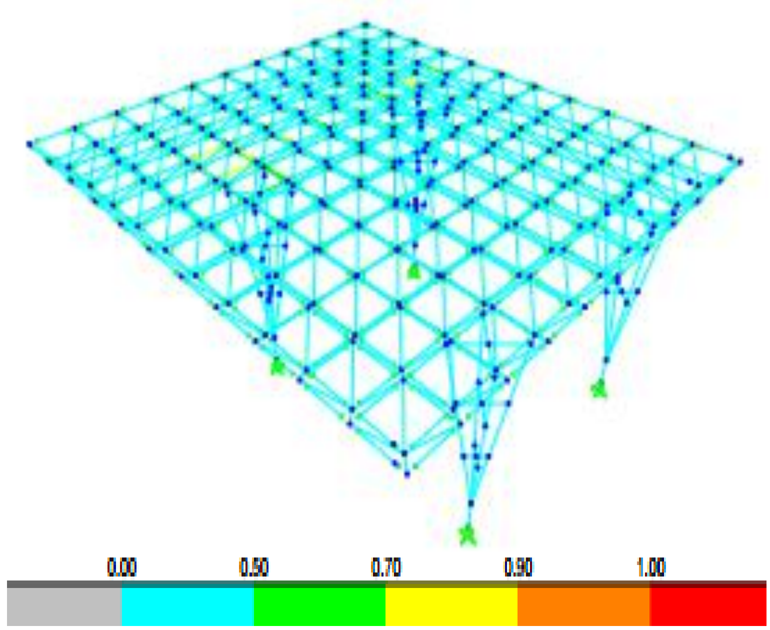


Figure 4.1.2-1:- 3D Design output of D/C ratio of Shelter - A

Table 4.1.2-2:- Demand Capacity Ratio for some structural members of Shelter-B

For Top Chord	For Bottom Chord	For Diagonal Chord	For Column
0,019	0,197	0,07	0,059
0,13	0.58	0,034	0,102
0,713	0.798	0,014	0.16
0,76	0.554	0,069	0.08
0.8	0.157	0,014	0,09
0,77	0.263	0,074	0,043
0.152	-0.159	0,09	0,244
0.469	0.815	0,01	0,25

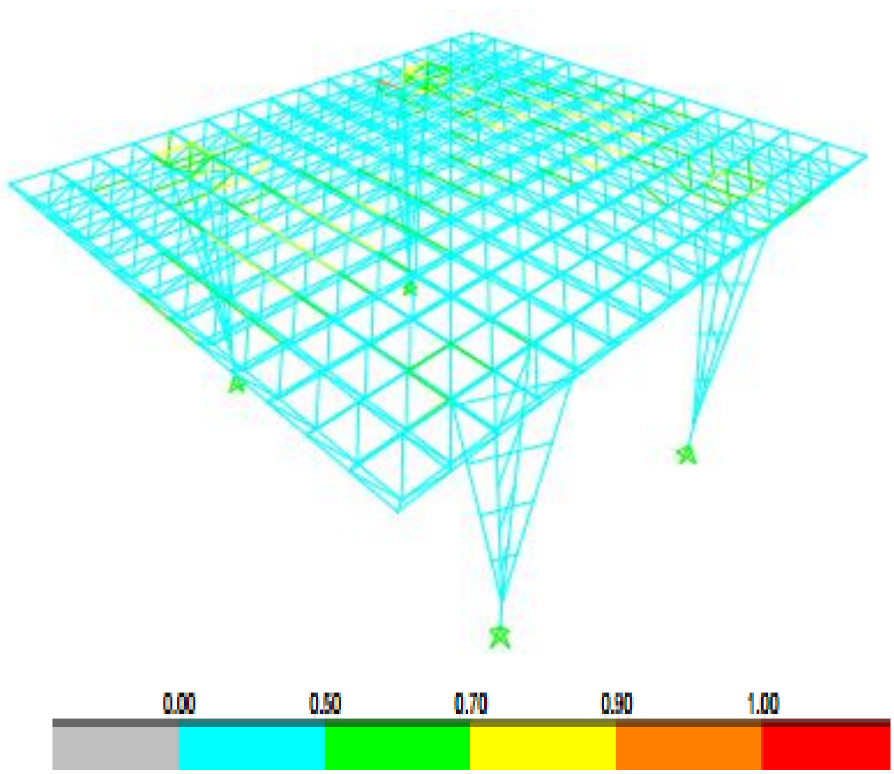


Figure 4.1.2-2:- 3D Design output of D/C ratio of Shelter - B

Table 4.1.2-3:- Demand Capacity Ratio for some structural members of Shelter-C

For Top Chord	For Bottom Chord	For Diagonal Chord	For Column
0,074	0,454	0,123	0,055
0,303	0,3	0,34	0,06
0,84	0,422	0,411	0.014
0,46	0,325	0,26	0.074
0,122	0,157	0,338	0,019
0,77	0,053	0,151	0,043
0,54	0,08	0,123	0,004
0,145	0,223	0,151	0,09

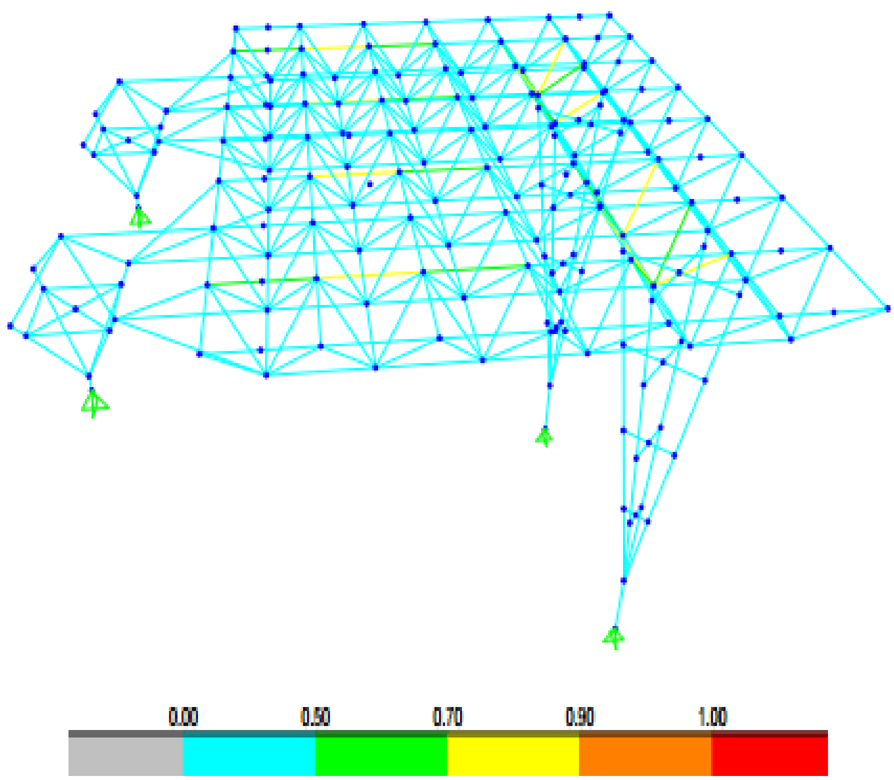


Figure 4.1.2-3:- 3D Design output of D/C ratio of Shelter - C

Table 4.1.2-4:- Demand Capacity Ratio for some structural members of Shelter-D

For Top Chord	For Bottom Chord	For Diagonal Chord	For Column
0,018	0,065	0,066	0,031
0,036	0,224	0,019	0,007
0,098	0,169	0,041	0.016
0,146	0,1	0,007	0.036
0,06	0,119	0,038	0,011
0,27	0,169	0,012	0,02
0,17	0,026	0,012	0,07
0,257	0,14	0,034	0,068

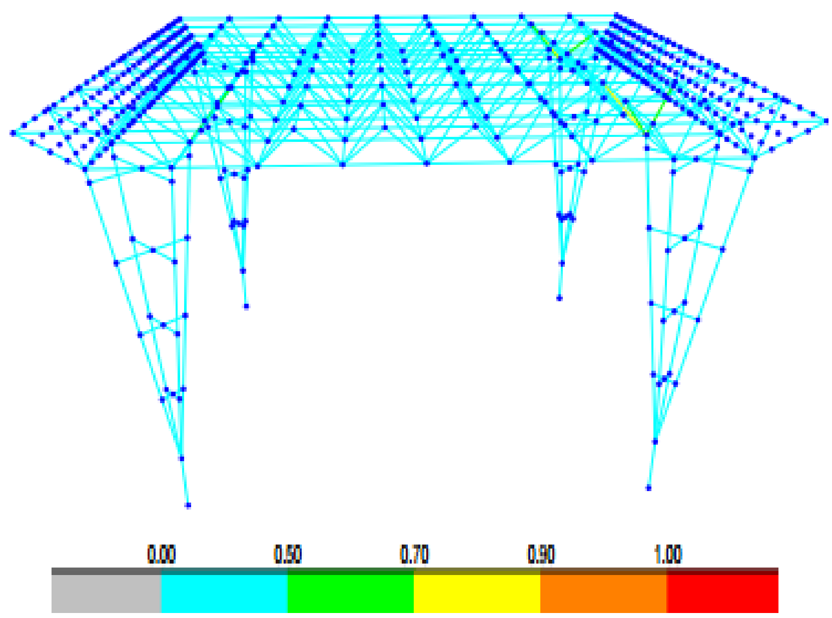


Figure 4.1.2-4:- 3D Design output of D/C ratio of Shelter - D

4.2. Analysis on the Underground tunnel/cavity

One of the underground tunnel/cavity found in under the courtyard of Bete-Amanueal church connecting Bete-Amanueal and Bete-Merqourios is selected for the analysis/ checking, since it is found to be loaded directly by one of the column D_1 shading Bete-Amanueal.

The finite element software DuCOM-COM3 is used for modeling and analyzing the underground tunnel/cavity. COM3 is a 3D finite-element analysis platform for structural elements with and without cracks.

This finite software is developed by the University of Bazel, Tokyo. The software is powerful in that it is possible to consider different types of non-linearity basically material and geometrical non-linearity. It is possible to simulate different solid elements in this finite software with their respective mechanical properties.

4.2.1. Modeling of the underground tunnel/cavity

Solid element with eight nodes are used for the simulations. A solid element having a dimension of $1{,}8\ m$ in the x-direction, $6\ m$ in the y- direction and $6\ m$ in the z- direction and a cavity opening having $1{,}8\ m$ in the X-direction, $1\ m$ in the Y- direction and $2\ m$ in the Z- direction placed at the center of the cube element is to be modeled in the finite software.

The system of symmetry is applied during modeling the tunnel/cavity, and the axis of symmetry passes at the center of the model perpendicular to axis Y.

The restraint condition at the axis of symmetry made to restrain x and Y direction translation/displacement and allows translation/displacement along Z-direction, since we are interested on finding the displacement at the top of the roof and ground surface, along the axis of symmetry. And the restrain condition at the far ends of the right and bottom sides (Refer Figure 4.2-2) are made to restrain the three directional translation.

The nodes that are in contact with the shelter foundation are loaded for the unfavorable vertical loading, which is the sum of the vertical load due to the already existed shelter and the load due to the counter plates added as a solution to counter balance the upwind loading. And the magnitude is $462{,}8 + 97 = 560\ KN$.

The loading has 50 steps, the 1st step is for the self-weight, and the vertical load from the shelter to be applied is divided for the remaining 49 steps. The loading increase step by step starting from step 2 and to the final loading step 49.

And the load combination used for the analysis is $\mathbf{1,35}\ \boldsymbol{D.L} + \mathbf{1,5}\ \boldsymbol{W.L} + \mathbf{1.05}\ \boldsymbol{L.L}$

Figure 4.2-1:- The underground tunnel/cavity (source [6])

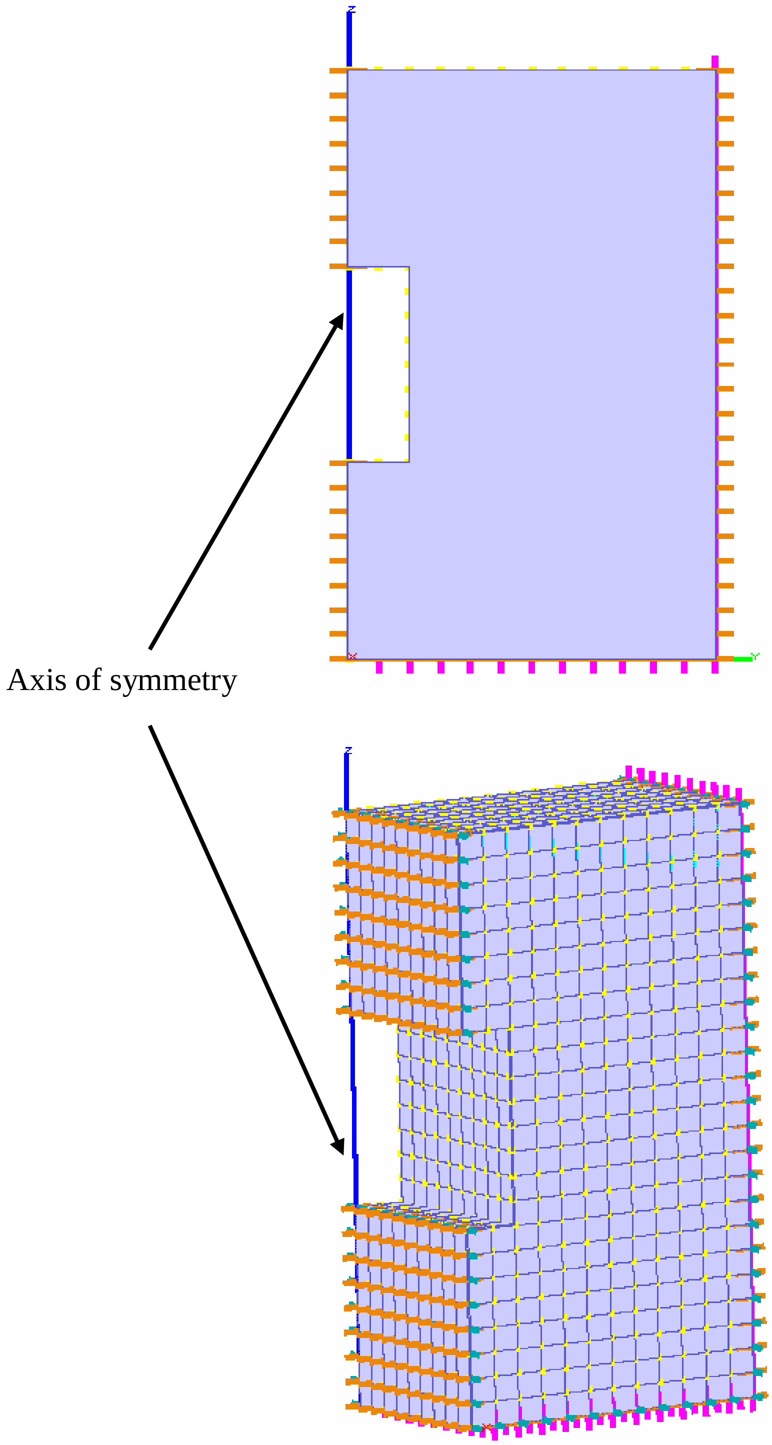


Figure 4.2-2:-3D modeling of the tunnel (COM3 modeling)

4.3. Earthquake analysis

Determining the base shear;

$F_b = S_d(T_1).W.\lambda$ .... Eqn 4.3.1 (Source:-Eqn 3.15 from EBCS EN 1998-1-1:2013)

Where: - S_d is design spectrum for elastic analysis for the fundamental period of vibration of the structure (T_1)

$W =$ Seismic dead load

λ= correction factor = 1

T_1 = Fundamental period of vibration of the structure in second

Where

$$T_1 \leq \begin{cases} 4\,.\,T_c \\ 2\text{ S} \end{cases}$$

$T_1 = C_t\,.\,H^{3/4}$

$C_t = 0{,}085$ For moment resistant space steel frame

$H =$ Height of building = $17{,}4\ m$, taking the maximum height out of the shelters

$$T_1 = 0{,}085\,.\,17{,}4^{3/4} = 0{,}724 \leq \begin{cases} = 4\,.\,0{,}4 = 0{,}8,\ T_c = 0{,}4 \text{ for type 2 elastic spectrum curve} \\ 2\text{ S} \end{cases}$$

Therefore it is possible to use elastic analysis for determining the base shear/

W is seismic dead load

λ is correction factor = 1

$$S_d(T_1) = \begin{cases} \alpha_g \cdot S \cdot \frac{2.5}{q} \left[\frac{T_c}{T}\right] \\ \geq \beta \cdot \alpha_g \end{cases}$$ Eqn 4.3.2 (Source:-Eqn 3.15 from EBCS EN 1998-1-1:2013)

α_g = Reference peak ground acceleration

= 0,07g, is the value given for town of Sekota which is the quite near place for Lalibela

S = Soil factor

= 1 for class A

q = Behavior factor

$q = 4{,}5 \cdot \frac{\alpha_u}{\alpha_1}$, For DCH structures

Where, $\frac{\alpha_u}{\alpha_1} = 1{,}1$ for one story building from table 5,1 of EBCS EN 1998-1-1:2013

$q = 4{,}5 \cdot 1{,}1 = 4{,}95$

Shelter	Weight	Height of structure	T_1	S_d	$F_B = S_d \cdot W \cdot \lambda$	$F_T = 0.07 \cdot T_1 \cdot F_B$	$F_i = \frac{(F_b - F_t)Wi\,hi}{\sum Wihi}$
A	884.24	6	0.3259	0.0434	38.38	0.88	10.44
B	1046.656	8	0.4043	0.035	36.61	1.04	16.11
C	929.434	15.6	0.6672	0.0212	19.7	0.93	1.84
D	807.754	6	0.3259	0.0434	35.06	0.8	7.43

The magnitude we have found from the earthquake analysis are quite small when we compare them to the values we have found on the wind analysis, hence we can say that earthquake has a negligible effect on the structures. So it is no more important for further checking of the capacity of the shelters for earthquake.

CHAPTER 5:- RESULT AND DISCUSSION

5.1. For the steel shelters

5.1.1. Analysis result versus collected data

In this part, we will discuss and compare in detail about the result obtained from the structural analysis and the data we already collected from site investigation and interview. In addition to that the Wind Rose diagram and wind direction of the design fundamental wind speed will also be used as a verification on the discussion.

For Shelter - A

Site Investigation and inspection

- No openings on the counter weight base plates encountered at column A_1 and A_3
- Openings on the counter weight base plates encountered at column A_2 and A_4 (from southern direction) seems to be a tension crack with wide opening at the top and getting narrower at the bottom (refer to the data collection part of shelter A).

Orientation of the shelter Vs wind rose diagram

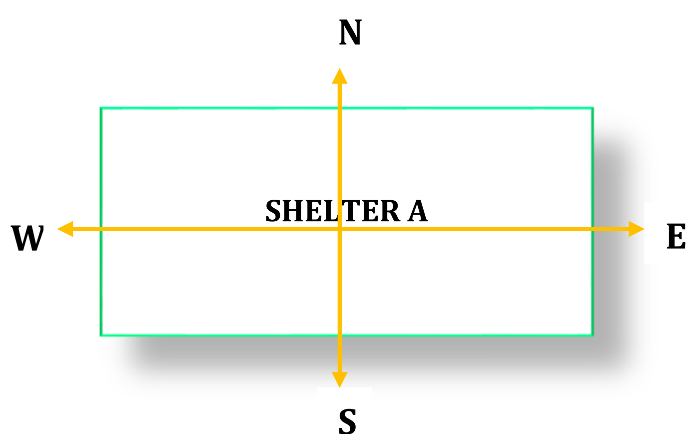


Figure 4.2.1-1:- Northing of Shelter A in plan view

The maximum annual wind speed were exerted from 180^0 (South direction), and the analysis is done on South-North direction and also the already recorded signs of failure on the base columns are also in this direction.

Analysis result

Along 0^0 (South) wind Direction

- Uplifting is encountered at the base reaction on A_2 and A_4 when using design wind speed.
- There is no uplift encountered at any of the base columns when using the already experienced wind speed.

When we use the wind speed that the Lalibela region experienced there are no columns that shows uplift is critical. And while we use the design wind speed the base reaction on A_2 and A_4 show that uplift is critical for the structure which comply with the site investigation and also with wind rose diagram.

Along 180^0 (North) wind Direction

- Uplift is encountered at the base reaction only on column A_1 and A_3 when using the design wind speed.
- There is no uplift encountered at any of the base columns when using the already experienced wind speed.

When we use the wind speed that the Lalibela region experienced there are no columns that shows uplift is critical and there are no signs witnessed at this two columns during the site investigation, since the maximum annual wind speed was exerted from south direction and there are no winds with large magnitude exerted from North direction while referring to the wind rose diagram. And while we use the design wind speed the base reaction on A_1 and A_3 show that uplift is critical for the structure.

For Shelter - B

Site Investigation and inspection

- No openings on the counter weight base plates encountered in any of the columns base.

Orientation of the shelter Vs wind rose diagram

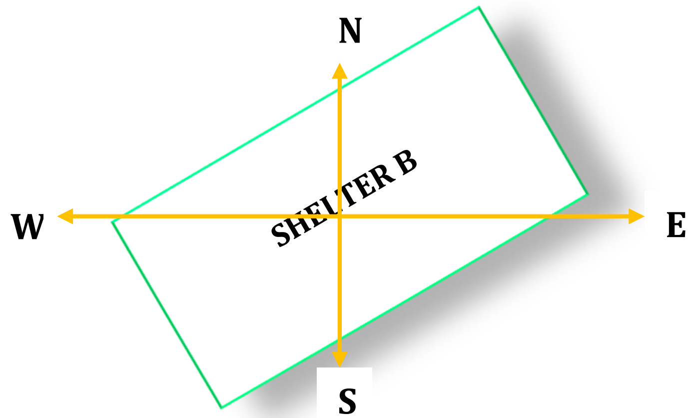


Figure 4.2.1-2:- Northing of Shelter B in plan view

The maximum annual wind speed were exerted from (South direction), but the analysis is performed on South-East and North-West direction by introducing a directional factor.

Analysis result

Along 0^0 (South-East) wind Direction

- There is no uplifting encountered in any of the column base when using already experienced wind.
- Uplift is encountered at the base reaction only on column B_2 and B_4 when using the design wind speed.

When we use the already experienced wind speed there is no uplifting encountered in any of the base columns and complies with the site investigation, and while we use the design wind speed the base reaction of column B_2 and B_4 does show that uplift is critical.

Along 180^0 (North-West) wind Direction

- There is no uplifting encountered in any of the column base when using already experienced wind.
- Uplift is encountered at the base reaction only on column B_1 and B_3 when using the design wind speed.

When we use the already experienced wind speed, there is no uplifting encountered in any of the base columns and complies with the site investigation, and while we use the design wind speed the base reaction of column B_1 and B_3 does show that uplift is critical.

For Shelter - C

Site Investigation and inspection

- Openings and bending of the counter weight base plates encountered at column C1 and C_2 seems to be a tension crack with wide opening at the top and getting narrower at the bottom (refer to the data collection part of shelter C).
- Overturning of base plates witnessed at Column C_3 and C_4 but there are no openings of the counter base plate witnessed (refer to the data collection part of shelter C).

Orientation of the shelter Vs wind rose diagram

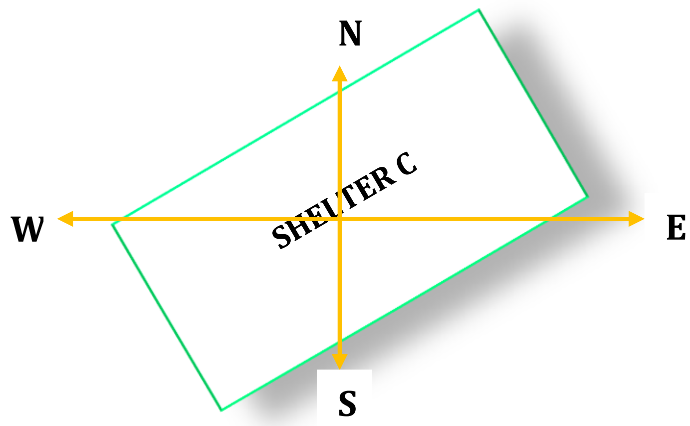


Figure 4.2.1-3:- Northing of Shelter C in plan view

The maximum annual wind speed were exerted from (South direction), but the analysis is performed on South-East and North-West direction by introducing a directional factor.

Analysis result

Along 0^0 (South-East) wind Direction

- There is no uplifting encountered in any of the column base when using already experienced wind.
- Uplift is encountered at the base reaction only on column C_1 and C_2 when using the design wind speed.

When we use the design wind speed the base reaction of columns C_1 and C_2 does show that uplift is critical and which comply with the sign witnessed at the columns during the site investigation.

And while using the already experienced wind speed there are no columns that are critical for uplift.

Along 180^0 (North-West) wind Direction

- There is no uplift encountered in every column while using both design wind speed and experienced speed.

For Shelter – D

Site Investigation and inspection

- Openings and bending of the counter weight base plates encountered in all of the columns seems to be a tension crack with wide opening at the top and getting narrower at the bottom (refer to the data collection part of shelter D).

Orientation of the shelter Vs wind rose diagram

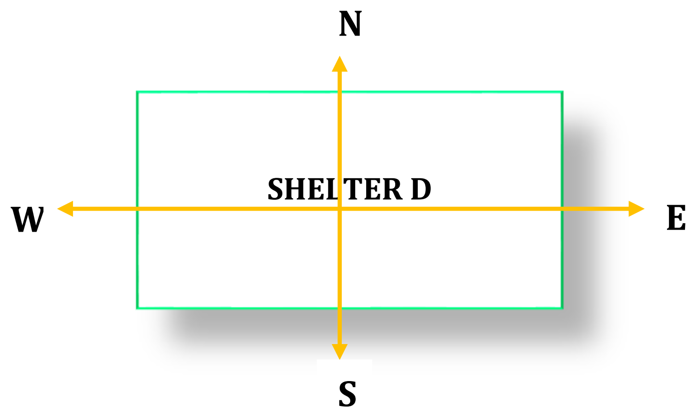


Figure 4.2.1-4:- Northing of Shelter D in plan view

And while performing analysis along 90^0 (North) and 270^0 (South) direction the pitch angle is made to be 0^0. And note that there is no directional factor is used for the analysis since the analysis is performed in the direction where the maximum wind speed experienced. And also analysis is performed for the remaining directions 0^0 (Western) and 180^0 (Eastern) direction using the respective pitch angle by introducing directional factor that ASCE (American Society of Civil Engineers) provides since the analysis is performed in different direction from the direction where the maximum wind speed experienced.

Analysis result

Along 0^0 wind Direction

- Uplift is encountered at the base reaction only on column D_1 and D_3 when using the design wind speed.

- There is no uplift encountered in every column while using the already experienced wind speed.

When we use the design wind speed the base reaction of columns D_1 and D_3 show that uplift is critical, which comply with the sign witnessed during the site investigation. And while using the already experienced wind speed there are no columns that are critical for uplift.

Along 90^0 wind Direction

- There is no uplift encountered in any of the columns while using both design and the already experienced wind speed.

When we use the design wind speed and the wind speed that the Lalibela region experienced there are no columns that shows uplift is critical.

Along 180^0 wind Direction

- Uplift is encountered at the base reaction only on column D_2 and D_4 when using the design wind speed.
- There is no uplift encountered in every column while using the already experienced wind speed.

When we use the design wind speed the base reaction of columns D_2 and D_4 show that uplift is critical, which comply with the sign witnessed during the site investigation. And while using the already experienced wind speed there are no columns that are critical for uplift.

Along 270^0 wind Direction

- There is no uplift encountered in any of the columns while using both design and the already experienced wind speed.

When we use the design wind speed and the wind speed that the Lalibela region experienced there are no columns that shows uplift is critical.

5.2. For the underground tunnel/cavity

Analysis has done for the two scenarios, regarding to the difference in their mechanical property and we obtain the results that we are interested on, finding the displacement on the top of the roof and ground surface and also to check the occurrence of crack.

The underground tunnel is checked for the unfavorable combined vertical load coming from the already existed shelter (dead load), live load and wind load and also for the vertical load to be added on as solution and for the overburden weight from the rock itself.

From the analysis output of sample A, the maximum displacement at the axis of symmetry on top of the roof is found to have a magnitude of $-1{,}05 * 10^{-2}$ cm and the maximum displacement at the ground surface have a magnitude of $-2{,}08 * 10^{-2}$ cm. And it is also affirmed that there are no crack witnessed during the analysis that can be caused by the exerted vertical load.

And also the analysis output for sample B is, displacement at the axis of symmetry on top of the roof is found to have a magnitude of $-1{,}82 * 10^{-2}$ cm and the maximum displacement at the ground surface have a magnitude of $-3{,}44 * 10^{-2}$ cm. And it is also affirmed that there are no crack witnessed during the analysis that can be caused by the exerted vertical load.

The figure in below is the sample displacement Vs reaction (applied load) and tries to show different graphs at different loading steps for the randomly chosen nodes, which they basically have a linear relationship in that the displacement increases when the reaction increases.

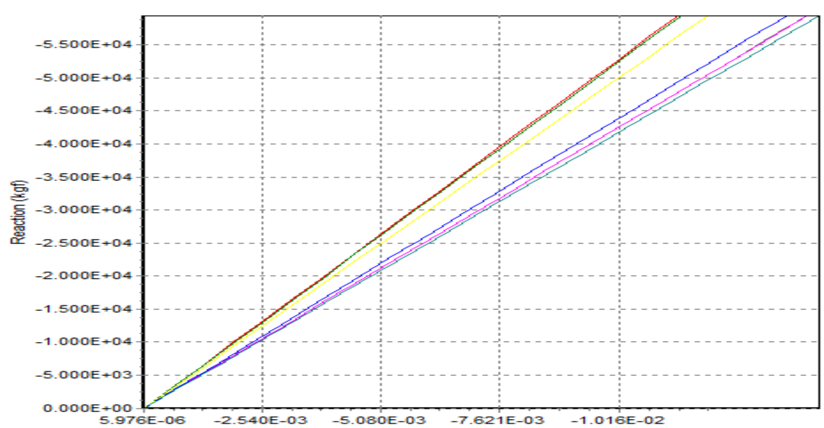


Figure 4.2.1-1:- Displacement Vs Applied Vertical Load Graph for sample nodes

And the figure shown below is the magnified displacement at the randomly chosen loading steps. In the designation "D*i*" under every figure, the prefix "D" is for the displacement and suffix "*i*" is for the step loading, for instance, D1 is a displacement caused by the first step of loading and likewise the rest follows the same designation.

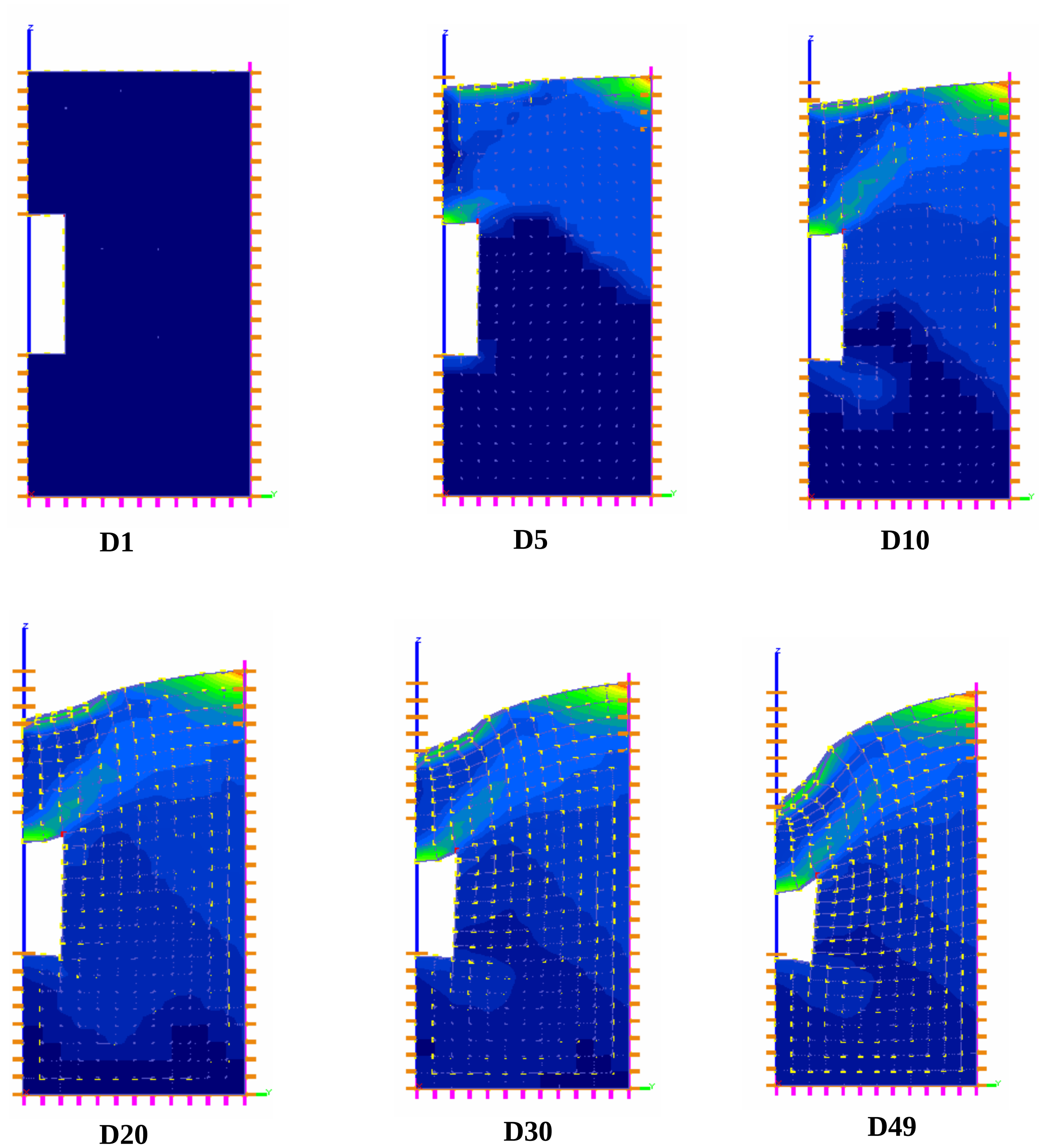


Figure 4.2.1-2:- displacement caused by Vertical loading (Analysis output)

And the underground tunnel using sample B (with low mechanical property) is loaded until it reaches to a yielding point/ shows crack so that we can extract the maximum displacement and the failure load. This helps us to find the limit of safety for verifying if the underground tunnel is safe or not. If we refer the graph on figure 5.2-1 the relationship we get from the reaction to displacement is linear when the tunnel is loaded by a force of 280 KN (symmetrical analysis) however when we see the graph in below which shows a linear relationship of reaction to displacement until it reach to a yielding point of a force of $7500\ KN$ which lurks a crack and a maximum displacement at the ground surface of $-3{,}9\ cm$ and $-4{,}9 * 10^{-1}\ cm$ at the top of the roof.

And determining the factor of safety Fs = $\frac{YIelding\ Load}{Applied\ Load} = \frac{7500}{280} = 26$

Which we have a way much capacity of a rock mass.

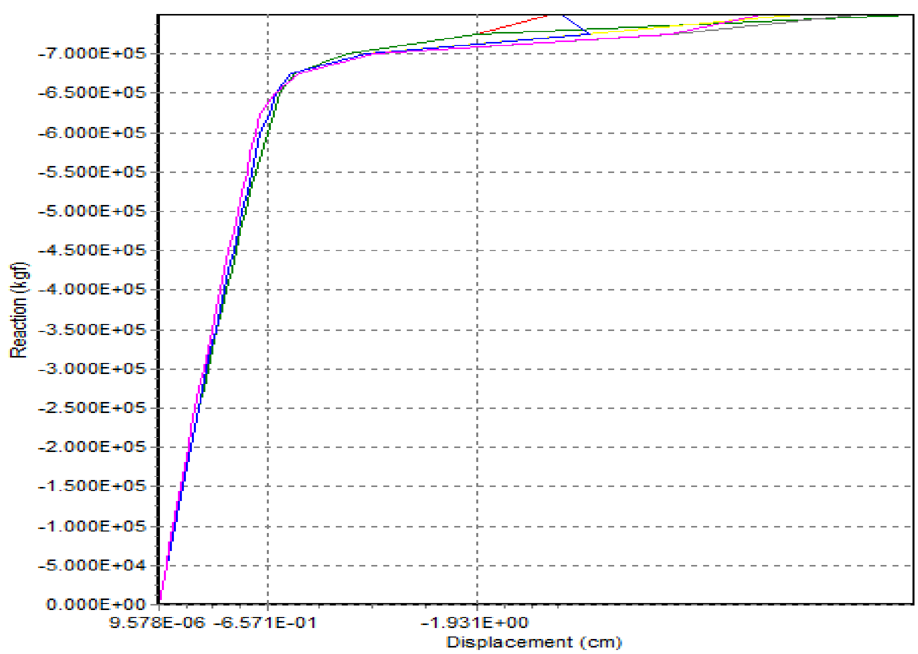


Figure 4.2.1-3:- Displacement Vs Applied Vertical Load Graph for sample nodes (to determine the failure load)

CHAPTER 6:- CONCLUSION AND RECOMMENDATION

6.1. Conclusion

6.1.1. For The Steel Shelters

From the analysis results its obtained that the shelters are proofed to be a threats for the monuments found under the shelters, which ones they fail it could lead to the total loss of the monuments which we don't even want to think about it. Hence it is recommendable if the shelters are to be removed and to look for another solutions as of a restoration activity done on Bete-Gabriel Rufael that goes together with the appearance of the monuments and meets with its purpose.

But this solution shall not be implemented before doing restoration and protection for the monuments, hence we should also look for short term solutions that simultaneously goes with the removal and restoration and protection activity.

As a short term solution three possible solutions are brought to conclude the research on the steel shelters, and are described in detail so that we could select the best that fulfills easy for workmanship, which is not labor intensive, simple for implementation, that do not contradict with the idea of the project and that we could rely in confident. And the three possible solutions are;

i. Adding the counter mass plates

It is the simplest and easy implementation since it is already practiced. Once the plates are manufactured they are easy for implementation without needing a skilled manpower and is not labor intensive as well. And also it agrees with the idea of the project and goes with the already experienced one.

ii. Using high tension cable

The manufactured high tension cables as per the design is to be connected with the roof of the shelter and anchored deep into the ground. So that it resist the upwind effect. This method is quite helpful but it needs a highly skilled manpower, labor intensive and will contradict with the idea of the project, since it needs an excavation, which will leave a permanent hole in the area and all together makes it difficult for implementation.

iii. Increasing the bond between the rock and the concrete

Increasing the bondage will increase the capacity of the base not to be lifted up by the wind. Hence high bondage chemical is aimed to be injected in between the concrete and the rock. However, the implementation needs a highly skilled manpower.

Hence out of the above three proposed solutions, the first proposal is found to be the best to use. Besides proposing the solution the underground tunnels/cavity has also been checked with the load of the existed one and newly provided one (as a short term solution).

For Shelter - A

Every structural member of the shelter are safe against the unfavorable loading and from corrosion. The structural stability of the steel shelter is safe against the unfavorable loads except the overall effect of uplift wind on the shelters.

The downpipe gutters are not functioning to the expected purpose and during the rainy season they are letting in the rain into the courtyard wall of the church complex and to the adjacent church of Bete-Golgotha.

Column A_2 and A_4 are unsafe against upwind loading with a design wind coming form 0^0 (South) direction having a magnitude of upwind load on the base columns; $-100{,}43\ KN$ and $-21{,}96\ KN$ respectively. And column A_1 and A_3 are unsafe against upwind loading with wind coming from 180^0 (North) direction having a magnitude of upwind load on the base columns; $-394{,}4\ KN$ and $-238{,}83\ KN$.

However, referring to the wind rose diagram there are no winds coming from 180^0 (North) direction that could be a threat to the shelters, hence we can only take the result found from 0^0 (South) and neglecting the effect coming from the North direction while providing a solution.

Hence the above justified magnitude of forces should be counter balanced. And since the columns are placed on a firm ground with no tunnel/cavity under them.

A single base plate weights $2{,}148\ KN$, hence the number of counter mass plates to be provided in every columns base is;

Table 6.1.1-1:- Number of counter mass plates to be added (for Shelter A)

Column Name	No of counter mass plates to be added
A_1	0
A_2	47
A_3	0
A_4	11

For Shelter - B

Every structural member of the shelter are safe against the unfavorable loading and from corrosion. The structural stability of the steel shelter is safe against the unfavorable loads except the overall effect of uplift wind on the shelters.

The downpipe gutters are not functioning to the expected purpose and during the rainy season they are letting in the rain into the courtyard wall of the church complex (on Mame-Gedel)

Column B_2 and B_4 are unsafe against upwind loading with a design wind coming form 0^0 (South-East) direction having a magnitude of upwind load on the base columns; $-143{,}6\ KN$ and $-81{,}26\ KN$ respectively. And column B_1 and B_3 are unsafe against upwind loading with wind coming from 180^0 (North-West) direction having a magnitude of upwind load on the base columns; $-154{,}7\ KN$ and$-158{,}01\ KN$.

However, referring to the wind rose diagram there are no winds coming from 180^0 (North-West) direction that could be a threat to the shelters, hence we can only take the result found from 0^0 (South-East) and neglecting the effect coming from the North-West direction while providing a solution.

Hence the above justified magnitude of forces should be counter balanced. And since the columns are placed on a firm ground with no tunnel/cavity under them we could definitely provide additional counter mass plates

Hence the number of counter mass plates to be provided in every columns base is;

Table 6.1.1-2:- Number of counter mass plates to be added (for Shelter B)

Column Name	No of counter mass plates to be added
B_1	0
B_2	67
B_3	0
B_4	38

For Shelter - C

Every structural member of the shelter are safe against the unfavorable loading and from corrosion. The structural stability of the steel shelter is safe against the unfavorable loads except the overall effect of uplift wind on the shelters.

We can say that this shelter is not achieving its purpose to some extent, especially it is not shading the church at the backyard where C_3 and C_4 found. The rain still can directly flow into the roof of the church. The community made some ditches to prevent the entering of the rain into the church infiltrating into the roof. They are only protecting the sun and rain coming directly to the roof. It doesn't consider the geometrical formation of the church.

Column C_1 and C_2 are unsafe against upwind loading with a design wind coming form 0^0 (South-East) direction having a magnitude of upwind load on the base columns; $-168{,}1\ KN$ and $-173\ KN$ respectively, and referring to the wind rose diagram the South-East direction could be a threat to the shelters, hence we can only take the result found from this direction while providing a solution.

Hence the above justified magnitude of forces should be counter balanced. And since the columns are placed on a firm ground with no tunnel/cavity under them we could definitely provide additional counter mass plates

Hence the number of counter mass plates to be provided in every columns base is;

Table 6.1.1-3:- Number of counter mass plates to be added (for Shelter C)

Column Name	No of counter mass plates to be added
C_1	79
C_2	81
C_3	0
C_4	0

For Shelter - D

Every structural member of the shelter are safe against the unfavorable loading and from corrosion. The structural stability of the steel shelter is safe against the unfavorable loads except the overall effect of uplift wind on the shelters.

The downpipe gutters are not functioning to the expected purpose and during the rainy season they are passing in the rain into the courtyard wall of the church complex.

Fort this church particularly analysis is performed in four direction looking for the maximum effect of uplift on the structure, and none of the columns shows that they are susceptible for uplift wind loading.

6.1.2. For The Underground Tunnel

From the analysis output for the two scenarios of the rock samples, the results are measured at the two critical interfaces that are the roof and ground surface. And the maximum displacement at the roof surface for the intact rock is $-1{,}05 * 10^{-2}$ cm and $-1{,}82 * 10^{-2}$ for the slightly fragmented rock, and the maximum displacement at the ground surface for the intact rock is $-2{,}08 * 10^{-2}$ cm and $-3{,}44 * 10^{-2}$ for the slightly fragmented rock. Which are the insignificant displacements to cause a failure while comparing it to the failure displacement at ground surface of $-3{,}9\ cm$ and $-4{,}9 * 10^{-1}\ cm$ at the top of the roof.

And it's found that the rock mass has a safety factor of 26, which is a way much capacity that a rock could have, hence the so provided shelter is no more a threat to the underground tunnel/cavity for the one found under one of the column of the shelter shading Bete-Amanueal.

From the results obtained the maximum deflection does occur on the ground surface, even if it is smaller in magnitude it is believed that it will increase through time if support is not provided (as per Terzagis' theory) and we know the ground surface is exposed for rain and sun, which are the main reasons for the degradation of the rock mass capacity, so the ground surface is more vulnerable for weathering effect than the internal/ embedded rock surface, and this effect believed to increase the displacement on the ground surface than the result we already found, which could not be a threat directly but could have an impact through time that being displaced around a footing could be a place that a water would flow to it and make the water to collect around and leading the water seep into the internal part and deteriorate the rock mass and beside that the displacement could create a differential settlement on the shelters and could cause a structural instability too.. And to tackle that it is advised to make a very small line ditch that take the water away from around the footing pad so that it cannot collect water.

And also, there should be a checking/analysis to be carried out for the rest of the underground tunnels/cavities and for the trench also, that are identified and listed on Chapter 3 data collection part, however having the rock mass quality of the area and the vertical loading we obtained from the shelters we can blindly say that the tunnels are going to be safe against the provided shelters.

6.2. Recommendations

i. A periodic inspection on the shelters has to be done for the sake of preserving the Rock Hewn Churches.

ii. The approximated thickness of the tunnel is taken at the only possible place which is the entrance roof depth, however the depth might vary place to place so it is recommended to check the thickness of the tunnel at the desired place using any laser technology.

iii. There should be a checking/analysis to be carried out for the rest of the underground tunnels/cavities and for the trench also, that are identified and listed on Chapter 3 data collection part.

iv. The wind data collecting system of the National Metrology Agency is not confidential, because while I was doing this research I have faced difficulties on the offered data's of wind on Lalibela, and I have discussed with Ato Zerihun (Head of raw data supply) and he admit that there are problems. So I recommend the Agency to properly collect and store the data's, knowing that they are the only and important sources that the country have.

v. For the sake of safety the wind analysis is done using the maximum annual wind speed which is coming from South direction, however some of the shelters orientation is along the South-East direction, which the wind analysis should be done using the maximum wind speed coming from this direction, and one can do the analysis using the specific direction for the sake of Economical aspect.

vi. Eurocode provision of pressure and/or force coefficient for canopy roof in two significant directions (0^0 and 180^0) are the same, which I doubt and not realistic, and also it do not give us the coefficient for the wind direction along 90^0, hence it's recommended if a further research is carried out on this areas.

vii. The wind analysis is done to consider the wind effect within the reference height, however the nature of the Lalibela if quite different in that there are trenches that the wind flows freely and in the guided way that the wind is believed to have a great impact on the shelter. Hence it is recommended to use computational or Wind tunnel test for further and accurate evaluation.

viii. Different assumptions has been made while doing the wind analysis, that the provisions of the code recommends do not clearly describe how to use or how to determine them. This basic assumptions believed to reach us to the reality but they has to be testified by computational or Wind tunnel test.

ix. The researcher has tried his best to consider the topology effect of the area on the shelters, but due to the erratic nature of the area it is recommended to use computational or Wind tunnel test so that we could simulate the nature.

x. The code of PrEn1991-1-4:2004 for canopy roof is found to have a limitation on providing force/pressure coefficients especially for double skinned membrane canopy roofs. Hence it is recommended for further research to be done in this area.

xi. I would like to appreciate the cooperation I have been offered by ARCCH, they have been kind and helpful at any time I was in need of things, but I still have a complain on their data storage system. It took so much time to find a data, perhaps there are also data's we couldn't get. So please do your best on the data collection and storing system.

References

[1] Asfaw (Phd), "rock mass classification," in *lecture note,AASTU*, addis ababa, 2016.

[2] Wikipedia, "Wikipedia, the free encyclopedia," [Online].

[3] A. Liakounaou, "Christianity and architecture in Ethiopia, Monuments Carved out of Rock," *Middle East and Islamic studies, University of Peloponnese,* 5 April 2012.

[4] K. McClure, "NO Shelter: UNESCO's efforts to save Lalibela's culture," University of Vermont, 2007.

[5] "Report on the UNESCO mission to Lalibela, Ethiopia," july, 2004.

[6] E. G.youhannes, "LALIBELA, A heritage under danger," oct 1,2014.

[7] (ECTT), "Google," QUARTZ Africa, 08 july 2015. [Online]. Available: https://qz.com//447128/ethiopia-has-been-voted-the-worlds-best-tourism-destination-for-2015/. [Accessed december 2017].

[8] Prof., Margottini "ENGINEERING GEOLOGY ASPECTS RELATED TO THE EU FEASIBILITY PROJECT FOR REHABILATION OF ROCK HEWN CHURCHES OF LALIBELA (NORTHERN ETHIOPIA)," JULY 2004.

[9] A. U. Workshop, "http://auworkshop.autodesk.com/library/building-science/wind-wind-rose-diagrams," 12 march 2015. [Online].

[10] F. Alamirew, "REFERENCE BASIC WIND SPEED FOR SELECTED TOWNS/CITIES IN ETHIOPIA," JULY,2016.

[11] FEDERICO SANI GIOVANNA MORATTI, "Integrated geological-architectural pilot study of the Biet Gabriel-Rufael," Società Geologica, Italiana, Roma, 2012.

[12] Nick Schiavon, "A multianalytical approach to investigate stone biodeterioration at a UNESCO world heritage site: the volcanic rock-hewnchurches of Lalibela, Northern Ethiopia," *Springer-Verlag Berlin Heidelberg,* 30 April 2013.

[13] L. D. V. P. project, Implemetation of Eurocode, Action Effects for buildings, Handbook 3, Leonardo Da Vinci Pilot project, 2005.

[14] N. COOK, DESIGNERS' GUIDE TO EN 1991-1-4, London: Thomas Telford, 2007.

[15] "www./wind load/Recommendations for Loads on Buildings.com," [Online]. Available: www.Recommendations for Loads on Buildings.com.

[16] Tony Gibbs, "Determination of Wind Loads for Use in Analysis," November 2000.

[17] ASCE. 7-05, "CHAPTER 6, WIND LOADS," in *Minimum design loads for buildings and other structures*, 2005.

[18] Prof. MilanHolicky, "HANDBOOK 3, ACTION EFFECTS FOR BUILDINGS, IMPLEMENTATION OF EUROCODES," Leonardo da vinci pilot project, Aachen, 2005.

[19] I. W. l. Standard, Instructions For the Evaluation of Actions and effect of wind on Buildings,CNR-DT 207/2008, Roma: National Research Counsil, Commission to study circuit connection and Analysis of Technical regulations relating to construction, 19 Februray 2009.

[20] P. 1991-1-4:2004, WIND ACTIONS.

[21] Fumiyoshi Takeda, "design wind Force coefficienTs For Free- standing canopy roofs of membrane structures," 2015.

[22] Y. Uematsu, "Wind loads on free-standing canopy roofs, Part 1 local wind pressures," *Journal of Wind Engineering and Industrial Aerodynamics* , p. 1016, june 2008.

[23] Neelam Rani, "Wind pressure distribution on Flat Canopy roofs," *Youth Education and Research Trust (YERT)*, may 2013.

[24] Dr.Habil, "ROCK MASS CLASSIFICATION SYSTEM," December 2016.

[25] TEPRIN, "Shelters for five churches in lalibela," Addis Ababa, 2002.

[26] R. Janardhanan, "https:/slideshare.net/rijurijesh/loads-on-tunnels," Eduacation,Technology<business, 8 April 2102. [Online].

[27] F. C. Paolo, "EUROCODE 1, ACTIONSON BUILDIIGN STRUCTURES," in *EU-Russia cooperation on satandardization for consturction,A tool for building safety and realibility enhancement*, Moscow, 10 october 2010.

APPENDIX-A

For Shelter - B on Bete – Medhaniealem

- Details of the shelter
  - Length = 45 m = b
  - Width = 39 m = d
  - Slope = 5 % and pitch angle = $2{,}86^0$
  - Reference height = 8 m
  - Average coverage area of the monument = 801,56 m^2
  - Terrain category = II

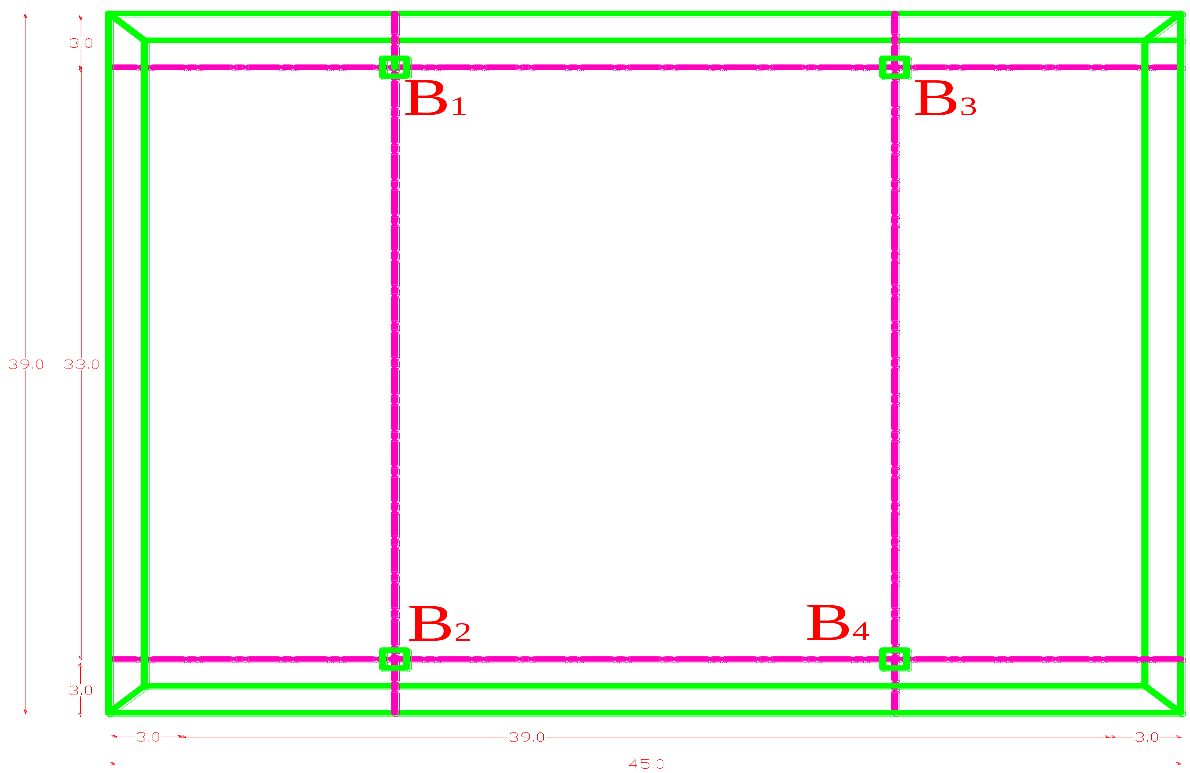


Figure A1:- AutoCAD drawing of the roof plan

Solution

And the terrain of Lalibela can be categorized as terrain category II, Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights, having Z_0 = 0.05 m, Z_{min} = 2 m, Z_{max}= 200 m and reference height Z= 8 m.

1. Wind load Analysis using design wind speed

Determining the basic wind velocity ($\mathcal{V}_b$)

$$\mathcal{V}_b = C_{dir} \ . \mathrm{C}_{season} . \ \mathcal{V}_{b,o}$$

$$= 0,85 * \ 1 * 29,5 \ m/s$$

$$= 25,07 \ m/s$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) \ = \mathrm{Cr}(Z) \ . \mathrm{Co}(Z) \ . \ \mathcal{V}_b$$

$$\mathrm{Cr}(Z) = \mathrm{Kr} \, . \, \ell \mathrm{n} . \left({Z}/{Z0} \right) \quad \text{For } Z \geq Z \text{ min}$$

$$\mathrm{Kr} = 0,19 . \left({Z_o}/{Z_{0,II}} \right)^{0.07}$$

$$\mathrm{Kr} = 0,19 . \left({0,05}/{0,05} \right)^{0.07}$$

$$= \underline{\underline{\mathbf{0,19}}}$$

$$\mathrm{Cr}(Z) = \ 0,19 \, . \, \ell \mathrm{n} \, . \left({8}/{0,05} \right)$$

$$= \underline{\underline{\mathbf{0,9642}}}$$

$\mathrm{Co}(Z) = 1 + 2 \, . \, S \, . \, \Phi$, for $0.05 < \Phi < 0.3$ i.e. shallow where $L_e = L_u$.

The figure in below shows the topographic section detail of Shelter-B taken from ARCCH which is prepared by TEPRIN Associates as a working drawing. Which helps us to find the vertical elevation and the horizontal data for determining the topographic factor.

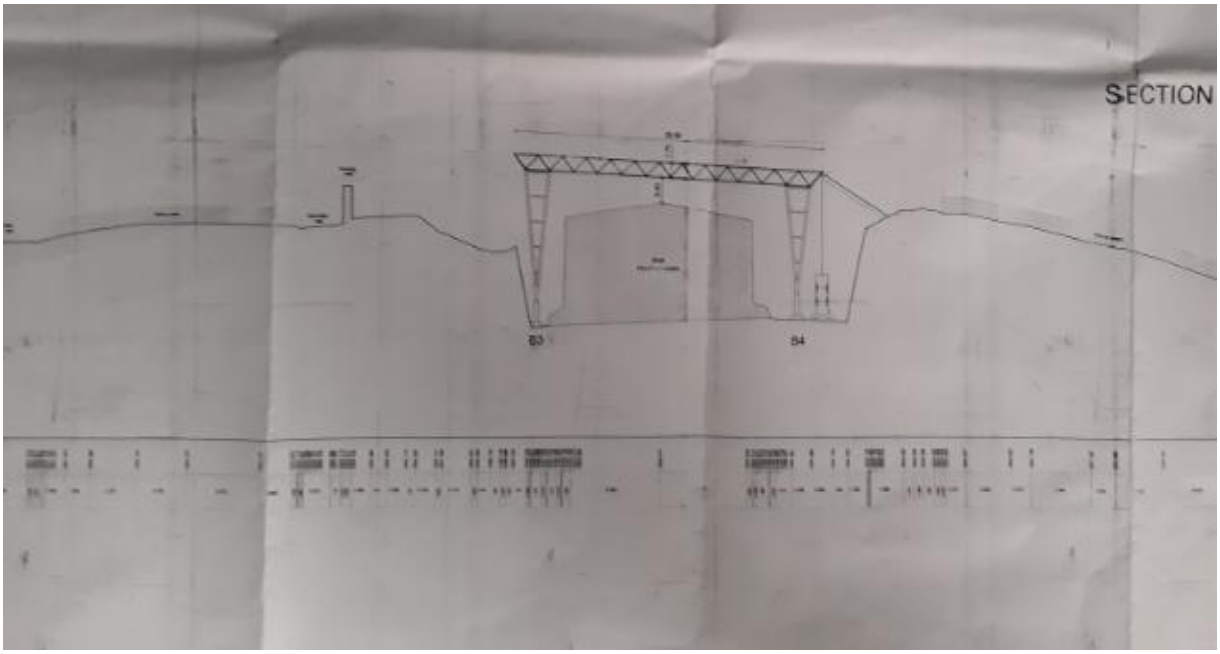


Figure A2:- section drawing for shelter-A (source from ARCCH, working drawing)

-Along 0^0 and 180^0

From the section drawing $H = 17{,}89\ m$ and $Lu = 64\ m$

$$\Phi = \frac{H}{L_u} = \frac{17{,}89}{64} = 0{,}279$$

$$\mathbf{S} = \boldsymbol{A} \,.\, \boldsymbol{e}^{(\boldsymbol{B} \cdot \frac{X}{L_u})}$$

Conditions to be fulfilled

For the range; $-1{,}5 \leq \frac{X}{L_u} \leq 0$ and $0 \leq \frac{Z}{L_e} \leq 2{,}0$

$\frac{-X}{L_u} = \frac{-80}{64} = -1{,}25$ (satisfied)

$\frac{Z}{L_e} = \frac{8}{64} = 0{,}125$ (satisfied)

$$A = 0{,}1552\,.\left(\frac{z}{L_e}\right)^4 - \;0{,}8575\,.\left(\frac{z}{L_e}\right)^3 + 1{,}8133\,.\left(\frac{z}{L_e}\right)^2 - 1{,}9115\,.\left(\frac{z}{L_e}\right)^1$$

$$= 0{,}1552\,.\,(0.125)^4 - \;0{,}8575\,.\,(0.125)^3 + 1{,}8133\,.\,(0.125)^2 - 1{,}9115\,.\,(0.125)^1$$

$$= -0{,}2122$$

$$B = 0{,}354\,.\left(\frac{z}{L_e}\right)^2 - \;1{,}0577\,.\left(\frac{z}{L_e}\right)^1 + 2{,}6456$$

$$= 0{,}354\,.\,(0.125)^2 - \;1{,}0577\,.\,(0.125)^1 + 2{,}6456$$

$$= \;2{,}52$$

$$\boldsymbol{S \;=\; -\,0.0091}$$

Hence

$$\mathrm{Co}(Z\,) = 1 + 2\,.\,(-0{,}0091)\,.\,0{,}137 \quad = \underline{\mathbf{0,975}}$$

Therefore,

$$\mathcal{V}_m(Z_e) \;=\; 0{,}9642\,.\;0{,}975\;\;.\;25{,}07 = 23{,}57\ \mathrm{m/s}$$

Determining the peak wind velocity ($\boldsymbol{\mathcal{V}_p(Z_e)}$)

$$\mathcal{V}_p(Z_e)\;=\mathcal{V}_m(Z_e)\,.\,G$$

Where:-

$$G = \sqrt{1 + 7.\frac{1}{0{,}975\,.\,\ln(8/_{0.05})}} = 1.553$$

$$\boldsymbol{\mathcal{V}_p(Z_e) = 23{,}57\,.\;1{,}553 = 36{,}6\ \mathrm{m/s}}$$

Determining the peak velocity pressure ($\mathbf{q_p}$)

$$q_p\;=\;\frac{\rho}{2}.\;(\mathcal{V}_p(Z_e))^2 \qquad\qquad \text{Pr EN 1991 1-4 2004}$$

$$= \frac{0.94}{2} \cdot (36{,}6)^2$$

$$\boldsymbol{q_p = 0,63 \frac{KN}{m^2}}$$

Determining wind forces

iii. Wind load on external part of the roof covers

The wind force for the whole structure or a structural component should be determined by using one of the recommended method in Pr EN 1991 -1-4: 2004 which is using force coefficient method.

$$F_w = C_s \cdot C_d \cdot C_f \cdot q_p(z_e) . A_{ref}$$

Determining structural factors ($\mathbf{C_s\ C_d}$)

C_s Cd = is the structural factors which are the size factor and dynamic factor respectively

= 1 for structure which are not susceptible to turbulence induced vibration. The recommended value of $C_s\ C_d$ = 1 from Pr EN 1991 -1-4: 2004, 6.2 (1) (a)

Determining the force coefficient ($\mathbf{C_f}$) on the external most rigid roof cover

- Roof slope = 5 %
- Roof (pitch) angle (α) = tan^{-1} $\frac{5}{100}$ = 2,86^0
- Reference area of the roof is $= 45 . 39 = 1755\ m2$

⇒ At α = 2,86^0

$$\left.\begin{array}{ll} \alpha = 0^0 & -0{,}5 \\ \alpha = 2{,}86 & C_{f,external} \\ \alpha = 5^0 & -0{,}7 \end{array}\right\} \quad \frac{0-2{,}86}{0-5} = \frac{-0{,}5-C_{f,external}}{-0{,}5+0.7} \qquad \boldsymbol{C_{f,exernal} = -0,6144}$$

Therefore the net upwind force coefficient $\boldsymbol{C_{f,external} = -0,6144}$

Therefore,

The overall wind load on the external roof cover becomes

$$F_w = 1\,.\,(-0{,}6144)\,.\,0{,}63\,.\;1755$$

$$\boldsymbol{F_w = -\,679{,}31}\ \ \textbf{KN}$$

iii. Wind load on side roof covers

Based on the assumptions made on the methodology part, the wind loading analysis is be like;

Basic data's:-

- Height of roof = 1.5 m
- Reference height = 8 m
- b = 45 m
- d = 39 m
- e = b or 2h (whichever is the smaller)

= 45 m or $2\,.\,1{,}7 = 3{,}4$ m

e = $3{,}4\ m$

Determining areas of the regions:-

- $A = 1{,}7\,.\frac{3}{5} = 1{,}53\ m^2$ $(1\ m^2 < A < 10\ m^2)$
- $B = \frac{4}{5}\,.\,1{,}7\,.\,3 = 4{,}08\ m^2$ $(1\ m^2 < A < 10\ m^2)$
- $C = (39 - 3)\,.\,1{,}5 = 108\ m^2$ $(> 10\ m^2)$
- $D = 45\,.\,1{,}5 = 67.5\ m^2$ $(> 10\ m^2)$
- $E = 45\,.\,1{,}5 = 67.5\ m^2$ $(> 10\ m^2)$

$$\frac{h}{d} = \frac{1.7}{39} = 0{,}0435 \leq 0{,}25$$

And for the regions with the condition of $(1\ m^2 < A < 10\ m^2)$ we have a formula of;

$Cp_e = Cp_{e,1} - (Cp_{e,1} - Cp_{e,10})log10^A$……….. (Figure 7.2 of Pr EN 1991-1-4)

For region A

$$Cp_{e,1} = -1,1 \ \ and \ Cp_{e,10} = -0,8$$

$$Cp_{e,1,53} = -1,1 - (-1,1 - (-0,8))log10^{1,53}$$

$$Cp_{e,1,53} = \ -1,044$$

For region B

$$Cp_{e,1} = -1,1 \ and \ Cp_{e,10} = -0,8$$

$$Cp_{e,4,08} = -1,1 - (-1,1 - (-0,8))log10^{4,08}$$

$$Cp_{e,3,6} = \ -0,917$$

For region C

$$Cp_{e,10} = \ -0,5$$

For region D

$$\mathrm{Cp}_{\mathrm{e},10} = \ +0,7$$

For region E

$$Cp_{e,10} = \ -0,3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p \, . Cp_e$$

Wind pressure for region A

$$W_{e,A} = \big(0{,}63\,.\,(-1{,}044)\big)\,.\,0{,}4 = -0{,}263\ \frac{KN}{m^2}$$

Wind pressure for region B

$$W_{e,B} = (0{,}63\,.\,(-0{,}917))\,.\,0{,}4 = -0{,}23\ \frac{KN}{m^2}$$

Wind pressure for region C

$$W_{e,C} = -0{,}126\ \frac{KN}{m^2}$$

Wind pressure for region D

$$W_{e,D} = +0{,}1764\ \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}076\ \frac{KN}{m^2}$$

2. Wind load Analysis using the already experienced wind speed

By the year 2013 the Lalibela region experienced 28 m/s 180 minute gust wind speed measured at 10 meter height, therefore we use the wind speed as it is for the sake of checking the structural stability of the structure.

Determining the basic wind velocity ($\mathcal{V}_{\mathrm{b}}$)

$$\mathcal{V}_b = C_{dir}\ .\mathrm{C}_{season}.\ \mathcal{V}_{\mathrm{b},o}$$

$$= 0{,}85 * 1 * 28\ m/s$$

$$= 23{,}8\ m/s$$

Determining the mean wind velocity ($\mathcal{V}_{m}$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z\ )\ .\mathrm{Co}(Z\ )\ .\ \mathcal{V}_{\mathrm{b}}$$

$$\mathcal{V}_m(Z_e) \ = 0,9642 \ . \ 0,975 \ \ . \ 23,8 = 22,37 \ m/s$$

Determining the peak wind velocity ($\boldsymbol{\mathcal{V}}_{\mathbf{p}}(\mathbf{Z_e})$)

$$\mathcal{V}_p(Z_e) \ = \mathcal{V}_m(Z_e) \ . G$$

Where:-

$$\boldsymbol{\mathcal{V}}_{\mathbf{p}}(\mathbf{Z_e}) = \mathbf{22,37 \ . \ 1,553 = 34,74 \ m/s}$$

Determining the peak velocity pressure ($\mathbf{q_p}$)

$$q_p \ = \ \frac{\rho}{2}. \ (\mathcal{V}_p(Z_e))^2$$ Pr EN 1991 1-4 2004

$$= \ \frac{0.94}{2} \ . (34,74)^2$$

$$\boldsymbol{q_p} \ = \ \mathbf{0,567} \ \frac{\boldsymbol{KN}}{\boldsymbol{m^2}}$$

Determining wind forces

iii. Wind load on top of roof covers

$$F_w = C_s \ . C_d \ . C_f \ . q_p(z_e). A_{ref}$$

The overall wind load on the external roof cover becomes

$$F_w = 1 \ . (-0,6144) \ . 0,567 \ . \ 1755$$

$$\boldsymbol{F_w} = \ -\mathbf{611.88 \ KN}$$

iv. Wind load on side roof covers

For region A

$$Cp_{e,1} = -1,1 \ \ and \ Cp_{e,10} = -0,8$$

$$Cp_{e,1,53} = -1,1 - (-1,1 - (-0,8))log10^{1,53}$$

$$Cp_{e,1,53} = \ -1,044$$

For region B

$$Cp_{e,1} = -1{,}1 \; and \; Cp_{e,10} = -0{,}8$$

$$Cp_{e,4,08} = -1{,}1 - (-1{,}1 - (-0{,}8))log10^{4,08}$$

$$Cp_{e,3,6} = -0{,}917$$

For region C

$$Cp_{e,10} = -0{,}5$$

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p \, . Cp_e$$

Wind pressure for region A

$$W_{e,A} = \big(0{,}567 \, . (-1{,}044)\big) \, . \, 0{,}4 = -0{,}237 \; \frac{KN}{m^2}$$

Wind pressure for region B

$$W_{e,B} = (0{,}7615 \, . (-0{,}917)) \, . \, 0{,}4 = -0{,}208 \; \frac{KN}{m^2}$$

Wind pressure for region C

$$W_{e,C} = -0{,}113 \; \frac{KN}{m^2}$$

Wind pressure for region D

$$W_{e,D} = +0{,}158\ \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}068\ \frac{KN}{m^2}$$

Imposed loads

From the base plates and counter weights

- weight of base plate with 20mm thickness

Weight = $76{,}9729\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}02 = \mathbf{4{,}99\ KN}$

- weight of a counter mass plate

Weight = $76{,}9729\ .\ [(1{,}8\ .\ 1{,}8) - (0{,}67\ .\ 0{,}67)].\ 0{,}01 = \mathbf{2{,}148\ KN}$

And the number of counter weight varies from column to column, so the weight of the counter mass plate in each column is;

For all column bases = $\mathbf{6\ .\ 2{,}148 = 12{,}888\ KN}$

- weight of L-shaped plate

Weight = $76{,}9729\ .\ [4\ .\ (0{,}11 + 0{,}11 + 0{,}2]\ .\ 1{,}8\ .\ 0{,}01 = \mathbf{3{,}49\ KN}$

From the concrete foundation

A concrete foundation is used in every column with different thickness and similar sizes, it is used as a counter wait and also as surface leveling so that the counter mass plates could place simply. The concrete is grade C-25 Mpa that is found from those involved in the project. The thickness of the pad is taken to be the average of the four side thickness.

At base of the column $B1 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}45 = \mathbf{36{,}45\ KN}$

At base of the column $B2 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}5 = \mathbf{40{,}5\ KN}$

At base of the column$B3 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}4 = \mathbf{32{,}4\ KN}$

At base of the column $B4 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}47 = \mathbf{38{,}07\ KN}$

Therefore, the total weight in the base column is;

At base of the column $B1 = 36{,}45 + 12{,}88\ + 4{,}99 + 3{,}49 = \mathbf{57,81}\ \boldsymbol{KN}$

At base of the column $B2 = 40{,}5 + 12{,}88\ + 4{,}99 + 3{,}49 = \mathbf{61,86}\ \boldsymbol{KN}$

At base of the column $B3 = 32{,}4 + 12{,}89\ + 4{,}99 + 3{,}49 = \mathbf{53,76}\ \boldsymbol{KN}$

At base of the column B4 $= 38{,}07 + 128{,}9\ + 4{,}99 + 3{,}49 = \mathbf{59,43}\ \boldsymbol{KN}$

For Shelter - C on Bete – Abalibanos

- Details of the shelter
  - Length = 24m = b
  - Width = 18m = d
  - Slope = 33 % and pitch angle = $18{,}26^0$
  - Reference height along 0^0 wind direction = 15,5 m
  - Reference height along 180^0 wind direction = 3,22 m
  - Average cover area of the monument = 78,75 m^2
  - Terrain category = II

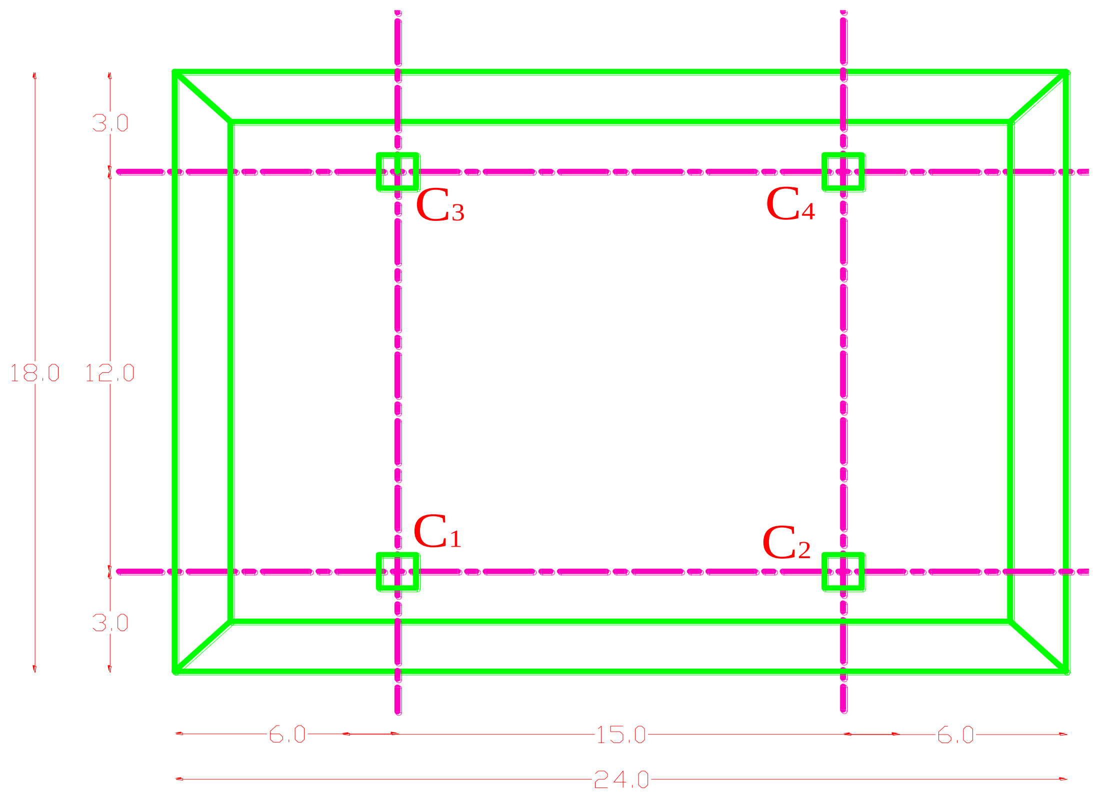


Figure A4:- Cad drawing of the roof plan

Solution

And the terrain of Lalibela can be categorized as terrain category II, Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights, having Z_0 = 0.05 m, Z_{min} = 2 m, Z_{max} = 200 m.

1. Wind load Analysis with design wind speed

i. Along 0^0 Wind Direction

Determining the basic wind velocity ($\mathcal{V}_b$)

$$\mathcal{V}_b = C_{dir} . C_{season} . \mathcal{V}_{b,o}$$

$$= 0{,}85 * 1 * 29.5\ m/s$$

$$= 25.075\ m/s$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z).\mathrm{Co}(Z).\ \mathcal{V}_b$$

$$\mathrm{Cr}(Z) = \mathrm{Kr}.\ell\mathrm{n}.\left(^{Z}/_{Z0}\right) \text{For } Z \geq Z \text{ min}$$

$$\mathrm{Kr} = 0{,}19.\left(^{Z_o}/_{Z_{0,II}}\right)^{0.07}$$

$$\mathrm{Kr} = 0{,}19.\left(^{0{,}05}/_{0{,}05}\right)^{0.07}$$

$$= \underline{\underline{\mathbf{0{,}1}9}}$$

$$\mathrm{Cr}(Z) = 0{,}19.\ell\mathrm{n}.\left(^{15{,}5}/_{0{,}05}\right)$$

$$= \underline{\underline{\mathbf{1{,}09}}}$$

$\mathrm{Co}(Z) = 1 + 2 . S . \Phi$, for $0.05 < \Phi < 0.3$ i.e. shallow where $L_e = L_u$.

The figure in below shows the topographic section detail of Shelter-C taken from ARCCH which is prepared by TEPRIN Associates as a working drawing. Which helps us to find the vertical elevation and the horizontal data for determining the topographic factor.

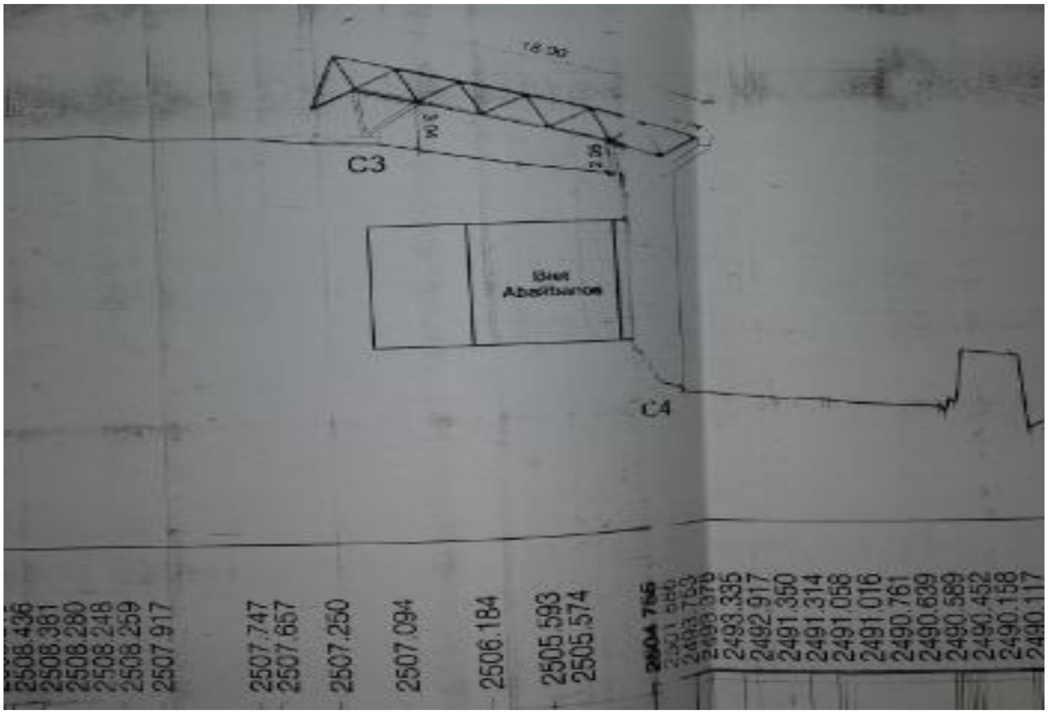


Figure A5:- section drawing for shelter-A (source from ARCCH, working drawing)

From the section drawing $H = 19\ m$ and $Lu = 40\ m$

$$\Phi = \frac{H}{L_u} = \frac{19}{40} = 0{,}48$$

$$\mathbf{S} = \boldsymbol{A} \cdot \boldsymbol{e}^{(\boldsymbol{B} \cdot \frac{\boldsymbol{X}}{\boldsymbol{L_u}})}$$

Conditions to be fulfilled

For the range $-1{,}5 \leq \frac{X}{L_u} \leq 0$ and $0 \leq \frac{Z}{L_e} \leq 2{,}0$

$\frac{-X}{L_u} = \frac{-50}{40} = -1{,}25$ (satisfied)

$\frac{Z}{L_e} = \frac{15,5}{40} = 0,3875$ (satisfied)

$$A = 0,1552 \,.\left(\frac{Z}{L_e}\right)^4 - 0,8575 \,.\left(\frac{Z}{L_e}\right)^3 + 1,8133 \,.\left(\frac{Z}{L_e}\right)^2 - 1,9115 \,.\left(\frac{Z}{L_e}\right)^1$$

$$= 0,1552 \,.\, (0,3875)^4 - 0,8575 \,.\, (0,3875)^3 + 1,8133 \,.\, (0,3875)^2 - 1,9115 \,.\, (0,3875)^1$$

$$= -0,5148$$

$$B = 0,354 \,.\left(\frac{Z}{L_e}\right)^2 - 1,0577 \,.\left(\frac{Z}{L_e}\right)^1 + 2,6456$$

$$= 0,354 \,.\, (0,3875)^2 - 1,0577 \,.\, (0,3875)^1 + 2,6456$$

$$= 2,29$$

$$\boldsymbol{S = -0,0294}$$

Hence

$$\mathrm{Co}(Z) = 1 + 2 \,.\, (-0,0294) .\, 0,48 = 0,97176$$

Therefore,

$$\mathcal{V}_m(Z_e) = 1,09 \,.\, 0,97176 \,.\, 25.075 = 26,56 \text{ m/s}$$

Determining the peak wind velocity ($\mathcal{V}_p(Z_e)$)

$$\mathcal{V}_p(Z_e) = \mathcal{V}_m(Z_e) \,.\, G$$

Where:-

$$G = \sqrt{1 + 7 . \frac{1}{0,97176 \,.\, \ln(15,5/0.05)}} = 1.5$$

$$\boldsymbol{\mathcal{V}_p(Z_e) = 26,56 \frac{m}{s} . 1,5 = 39,84 \text{ m/s}}$$

Determining the peak velocity pressure (q_p)

$$q_p = \frac{\rho}{2} . (\mathcal{V}_p(Z_e))^2$$ Pr EN 1991 1-4 2004

$$= \frac{0.94}{2} . (39{,}84)^2$$

$$\boldsymbol{q_p = 0{,}746 \frac{KN}{m^2}}$$

Determining wind forces

Wind load on external cover of the roof covers

The wind force for the whole structure or a structural component should be determined by using one of the recommended method in Pr EN 1991 -1-4: 2004 which is using force coefficient method.

$$F_w = C_s . C_d . C_f . q_p(z_e) . A_{ref}$$

Determining structural factors ($\mathbf{C_s C_d}$)

C_s Cd = is the structural factors which are the size factor and dynamic factor respectively

= 1 for structure which are not susceptible to turbulence induced vibration. The recommended value of $C_s C_d$= 1 from Pr EN 1991 -1-4: 2004, 6.2 (1) (a)

Determining the force coefficient ($\mathbf{C_f}$) on the external most rigid roof cover

- Roof slope = 33%
- Roof (pitch) angle (α) = $\tan^{-1}\frac{33}{100}$ = $18{,}26^0$
- Reference area of the roof is $= 24 . 18 = 432\, m2$
- Determining degree of blockage = $\frac{\text{actual obstruction}}{\text{crossectional area under the canopy}}$

  $\varphi = \frac{13 . 24}{15{,}5 . 24} = 0{,}8387$

Using linear interpolation for calculating the value of C_f for $\varphi = 0{,}8387$ and for pitch angle of $\alpha = 18{,}26^0$

⇒ At $\alpha = 15^0$

$$\left.\begin{array}{ll} \varphi = 0 & -1{,}1 \\ \varphi = 0.{,}8387 & C_{f\,0.8387} \\ \varphi = 1 & -1.4 \end{array}\right\} \quad \frac{0-0{,}8387}{0-1} = \frac{-1{,}1-C_f 0.1823}{-1{,}1+1{,}4} \qquad C_{f_{0.1823}} = -1{,}35$$

⇒ At $\alpha = 20^0$

$$\left.\begin{array}{ll} \varphi = 0 & -1{,}3 \\ \varphi = 0.8387 & C_{f\,0.8387} \\ \varphi = 1 & -1.4 \end{array}\right\} \quad \frac{0-0.8387}{0-1} = \frac{-1{,}3-C_f 0.{,}8387}{-1{,}3+1.4} \qquad C_{f_{0.331}} = -1{,}38387$$

⇒ At $\alpha = 18{,}26^0$

$$\left.\begin{array}{ll} \alpha = 15^0 & -1{,}355 \\ \alpha = 18{,}26^0 & C_{f,external} \\ \alpha = 20^0 & -1{,}38387 \end{array}\right\} \quad \frac{15-18{,}26}{15-20} = \frac{-1{,}355-C_{fu}}{-1{,}355+1{,}38387} \qquad \boldsymbol{C_{f,exernal} = -1{,}374}$$

The net upwind force coefficient $\boldsymbol{C_{f,external} = -1{,}374}$

Therefore,

The overall wind load on the external roof cover becomes

$$F_w = 1\,.\,(-1{,}374)\,.\,0{,}746\,.\;432$$

$$\boldsymbol{F_w = -593.35}\ \textbf{KN}$$

Wind load on the internal roof cover

Determining the force coefficient (C_f) on the inner air permeable roof cover

As described in the methodology part the force coefficient on the internal side is going to be solved using superposition method.

⇒ At α = 15^0

$$\left.\begin{array}{ll} \varphi_0 & -1{,}1 \\ \varphi_1 & -1{,}4 \end{array}\right\}$$

$$\varphi_1 - \varphi_0 = \varphi_{1,internal,15^0} = -(1{,}4 - 1{,}1) = -0{,}3$$

Then use linear interpolation for the specific degree of blockage

$$\left.\begin{array}{ll} \varphi = 1 & -0{,}8 \\ \varphi = 0{,}8387 & \mathrm{X} \end{array}\right\}$$

$$\varphi_{0,8387,internal,15^0} = -0{,}671$$

⇒ At α = 20^0

$$\left.\begin{array}{ll} \varphi = 0 & -1{,}3 \\ \varphi = 1 & -1{,}4 \end{array}\right\}$$

$$\varphi_1 - \varphi_0 = \varphi_{1,internal,20^0} = -(1{,}4 - 1{,}3) = -0{,}1$$

Then use linear interpolation for the specific degree of blockage

$$\left.\begin{array}{ll} \varphi = 1 & -0{,}7 \\ \varphi = 0{,}8387 & \mathrm{X} \end{array}\right\}$$

$$\varphi_{0,8387,internal,20^0} = -0{,}5871$$

⇒ At α = $18{,}26^0$

$$\left.\begin{array}{ll} \varphi_{0,8387,internal,15^0} & -0{,}671 \\ \varphi_{0,8387,internal,18,26^0} & X \\ \varphi_{0,8387,internal,20^0} & -0{,}5871 \end{array}\right\}$$

$$\boldsymbol{C_{f,internal}} = \varphi_{0,8387,internal,18,26^0} = -0{,}6162$$

Here we have to note that the effective area to be loaded is only 40% of the total area of the cover because of the uniformly distributed openings. Which the area of the internal cover is equal to 432 and the effective area equal 315.

Therefore,

The overall wind load on the internal roof cover becomes,

$$F_w = \frac{C_s \cdot C_d \cdot C_f \cdot q_p(z_e) \cdot A_{ref}}{3}$$

$$F_w = \frac{1 \cdot (-0{,}6162) \cdot 0{,}746 \cdot 315}{3}$$

$$\boldsymbol{F_w = -48{,}3} \textbf{ KN}$$

Wind load on side roof covers

Based on the assumptions made on the methodology part, the wind loading analysis is be like;

Basic data's:-

- Height of roof = 1.7 m
- Reference height = 15,5 m
- b = 24 m
- d = 18 m
- e = b or 2h (whichever is the smaller)

= 24 m or $2 \cdot 1{,}7 = 3{,}4$ m

e = 3,4 m

Determining areas of the regions:-

- $D = 24 \cdot 1{,}7 = 40{,}8\ m^2$ ($> 10\ m^2$)
- $E = 24 \cdot 1{,}7 = 40{,}8\ m^2$ ($> 10\ m^2$)

Determining the force coefficient ($\mathbf{C_f}$) on the inner air permeable roof cover

$$\frac{h}{d} = \frac{1.7}{18} = 0{,}094 \leq 0{,}25$$

And for the regions with the condition of $(1\ m^2 < A < 10\ m^2)$ we have a formula of;

$Cp_e = Cp_{e,1} - (Cp_{e,1} - Cp_{e,10}) log10^A$………. (Figure 7.2 of Pr EN 1991-1-4)

For region D

$$Cp_{e,10} = +0,7$$

For region E

$$Cp_{e,10} = -0,3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p\ .\ Cp_e$$

Wind pressure for region D

$$W_{e,D} = +0,21\ \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0,09\ \frac{KN}{m^2}$$

- **Wind load Analysis with the already experienced wind speed**

By the year 2013 the Lalibela region experienced 28 m/s, 180 minute gust wind speed measured at 10 meter height, therefore we use the wind speed as it is for the sake of checking the structural stability of the structure.

Determining the basic wind velocity ($\mathcal{V}_\mathbf{b}$)

$$\mathcal{V}_b = 23,8\ m/s$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) = 25{,}21\ m/s$$

Determining the peak wind velocity ($\mathcal{V}_p(Z_e)$)

$$\boldsymbol{\mathcal{V}_p(Z_e) = 37{,}82\ m/s}$$

Determining the peak velocity pressure (q_p)

$$\boldsymbol{q_p = 0{,}672\ \frac{KN}{m^2}}$$

Determining wind forces

Wind load on external roof cover

$$\boldsymbol{F_w = -398{,}87\ \mathrm{KN}}$$

The overall wind load on the internal roof cover,

$$\boldsymbol{F_w = -43{,}463\ \mathrm{KN}}$$

Wind load on side roof covers

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded on region D is only 40% of the total area of the side cover because of the uniformly distributed openings, however we have to take the wind pressure to be loaded on region E as it is, since it do not have a uniformly distributed openings.

$$W_e = q_p \,.\, Cp_e$$

Wind pressure for region D

$$W_{e,D} = +0{,}188 \ \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}08 \ \frac{KN}{m^2}$$

ii. Along 180^0 wind direction

1. Wind load Analysis with design wind speed

Unlike the previous shelters the wind analysis for this shelter is quite different in that the reference height in 0^0 and 180^0 wind directional are quite different and which significantly affects the value of the wind load. The reference height along 180^0 wind direction is 3,22 m.

Determining the basic wind velocity ($\mathcal{V}_{\mathbf{b}}$)

$$\mathcal{V}_b = 25.075 \text{ m/s}$$

Determining the mean wind velocity ($\mathcal{V}_{\mathbf{m}}$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z\,).\,\mathrm{Co}(Z\,).\ \mathcal{V}_\mathrm{b}$$

$$\mathrm{Cr}(Z\,) = 0{,}19.\ell\mathrm{n}.\left({}^{3{,}22}/_{0{,}05}\right)$$

$$= \mathbf{0{,}791}$$

Co(Z) =1; as we can see in the section drawing of this shelter there is no a change in the slope along this direction.

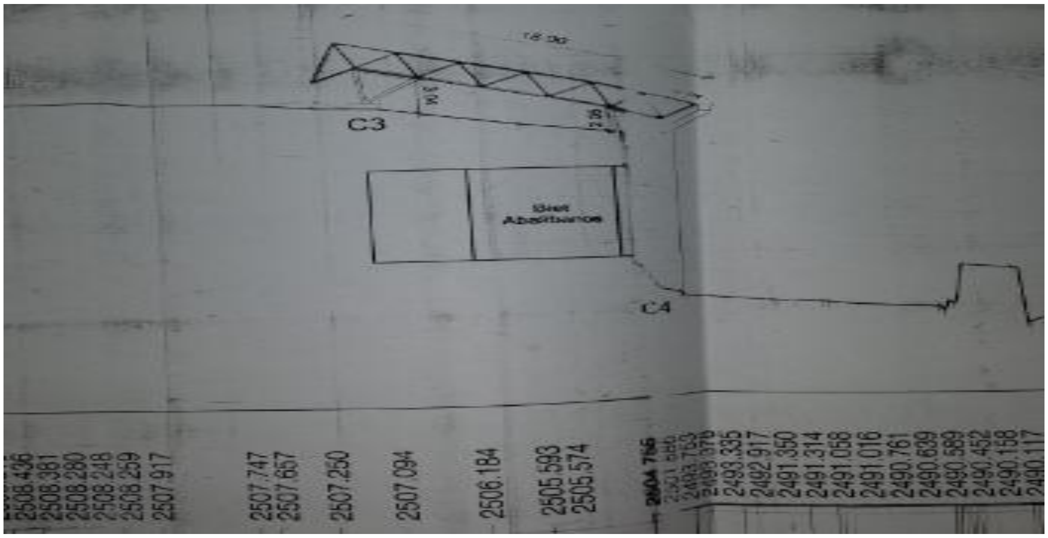


Figure A7:- section drawing for shelter-A (source from ARCCH, working drawing)

Therefore,

$$\mathcal{V}_m = 19{,}8 \text{ m/s}$$

Determining the peak wind velocity ($\mathcal{V}_{\mathbf{p}}(\mathbf{Z_e})$)

$$\mathcal{V}_p(Z_e) = \mathcal{V}_m(Z_e) \,.\, G$$

$$\boldsymbol{\mathcal{V}_{\mathrm{p}}(\mathrm{Z_e}) = 32{,}41\mathrm{m/s}}$$

Determining the peak velocity pressure ($\mathbf{q_p}$)

$$q_p = \frac{\rho}{2} \cdot (\mathcal{V}_p(Z_e))^2$$ Pr EN 1991 1-4 2004

$$q_p = 0{,}494 \frac{KN}{m^2}$$

Determining wind forces

Wind load on external cover of the roof

The wind force for the whole structure or a structural component should be determined by using one of the recommended method in Pr EN 1991 -1-4: 2004 which is using force coefficient method.

$$F_w = C_s \,.\, C_d \,.\, C_f \,.\, q_p(z_e).\, A_{ref}$$

Determining the force coefficient (C_f) on the external most rigid roof cover

- Roof slope = 33%
- Roof (pitch) angle (α) = $\tan^{-1}\frac{33}{100}$ = $18{,}26^0$
- Reference area of the roof is $= 24\,.\,18 = 432\; m2$
- Determining degree of blockage = $\frac{\text{actual obstruction}}{\text{crossectional area under the canopy}}$

  $\varphi = \frac{78.75}{432} = 0{,}1823$

Using linear interpolation for calculating the value of C_f for pitch angle of α $=18{,}26^0$

$\Rightarrow$ At $\alpha = 15^0$

$\varphi = 0$ $-1{,}1$

$\Rightarrow$ At $\alpha = 20^0$

$\varphi = 0$ $-1{,}3$

At α $= 18{,}26^0$

$\varphi = 0$ x

$$\frac{15-18{,}26}{15-20} = \frac{-1{,}1-X}{-1{,}1+1{,}3}$$

$$\boldsymbol{C_{f,external} = -1{,}23}$$

Therefore,

The overall wind load on the external roof cover becomes

$$F_w = 1\,.\,(-1{,}23)\,.\,0{,}494\,.\;432$$

$$\boldsymbol{F_w = -262{,}4}\ \textbf{KN}$$

Wind load on the internal roof cover

In the very beginning we have assumed that there is no an internal wind pressure when there is no blockage under the roof, hence we do not need to calculated the internal wind loading

Wind load on side roof covers

Based on the assumptions made on the methodology part, the wind loading analysis is be like;

Basic data's:-

- Height of roof = 1.7 m
- Reference height = 15,5 m
- b = 24 m
- d = 18 m
- e = b or 2h (whichever is the smaller)

= 24 m or $2 \,.\, 1{,}7 = 3{,}4$ m

e = 3,4 m

Determining areas of the regions:-

- $D = 24 \,.\, 1{,}7 = 40{,}8\ m^2$ ($> 10\ m^2$)
- $E = 24 \,.\, 1{,}7 = 40{,}8\ m^2$ ($> 10\ m^2$)

Pressure coefficients for the region

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p \,.\, Cp_e$$

Wind pressure for region D

$$W_{e,D} = +0{,}1976 \,\frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}085 \,\frac{KN}{m^2}$$

2) Wind load Analysis with the already experienced wind speed

Determining the basic wind velocity ($\mathcal{V}_b$)

$$\mathcal{V}_b = 23{,}8 \; m/s$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) = 18{,}82 \;\; m/s$$

Determining the peak wind velocity ($\mathcal{V}_p(Z_e)$)

$$\boldsymbol{\mathcal{V}_p(Z_e) = 30,81 \;\; m/s}$$

Determining the peak velocity pressure (q_p)

$$\boldsymbol{q_p = 0,446 \,\frac{KN}{m^2}}$$

Determining wind forces

Wind load on external roof cover

$$\boldsymbol{F_w = -196,96 \text{ KN}}$$

The overall wind load on the internal roof cover,

$$F_w = \frac{C_s \,.\, C_d \,.\, C_f \,.\, q_p(z_e) \,.\, A_{ref}}{3}$$

$$\boldsymbol{F_w = -3,2} \textbf{ KN}$$

Wind load on side roof covers

For region D

$$Cp_{e,10} = +0,7$$

For region E

$$Cp_{e,10} = -0,3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded on region D is only 40% of the total area of the side cover because of the uniformly distributed openings, however we have to take the wind pressure to be loaded on region E as it is, since it do not have a uniformly distributed openings.

$$W_e = q_p \,.\, Cp_e$$

Wind pressure for region D

$$W_{e,D} = +0,125 \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0,054 \frac{KN}{m^2}$$

From the base plates and counter weights

- weight of base plate with 20mm thickness

$$\text{Weight} = 76,9729 \,.\, 1,8 \,.\, 1,8 \,.\, 0,02 = \boldsymbol{4,99\ KN}$$

- weight of a counter mass plate

$$\text{Weight} = 76,9729 \,.\, [(1,8 \,.\, 1,8) - (0,67 \,.\, 0,67)].\, 0,01 = \boldsymbol{2,148\ KN}$$

And the number of counter weight varies from column to column, so the weight of the counter mass plate in each column is;

For C1 and C2 column bases = $\mathbf{72\ .\ 2,148 = 152,656}\ \boldsymbol{KN}$

For C3 and C4 column bases = $\mathbf{2\ .\ 2,148 = 4,269}\ \boldsymbol{KN}$

- weight of L-shaped plate

Weight = $76{,}9729\ .\ [4\ .\ (0{,}11 + 0{,}11 + 0{,}2]\ .\ 1{,}8\ .\ 0{,}01 = \mathbf{3,49}\ \boldsymbol{KN}$

From the concrete foundation

A concrete foundation is used in every column with different thickness and similar sizes, it is used as a counter wait and also as surface leveling so that the counter mass plates could place simply. The concrete is grade C-25 Mpa that is found from those involved in the project. The thickness of the pad is taken to be the average of the four side thickness.

At base of the column $C1 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}42 = \mathbf{34,02}\ \boldsymbol{KN}$

At base of the column $C2 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}35 = \mathbf{28,35}\ \boldsymbol{KN}$

At base of the column $C3 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 1{,}65 = \mathbf{133,65}\ \boldsymbol{KN}$

At base of the column $C4 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 1{,}12 = \mathbf{90,72}\ \boldsymbol{KN}$

Therefore, the total weight in the base column is;

At base of the column $C1 = 34{,}02 + 152{,}656 + 4{,}99 + 3{,}49 = \mathbf{195,156}\ \boldsymbol{KN}$

At base of the column $C2 = 28{,}35 + 152{,}656\ + 4{,}99 + 3{,}49 = \mathbf{189,486}\ \boldsymbol{KN}$

At base of the column $C3 = 133{,}65 + 4{,}269\ + 4{,}99 + 3{,}49 = \mathbf{146,399}\ \boldsymbol{KN}$

At base of the column C4 $= 90{,}72 + 4{,}269\ + 4{,}99 + 3{,}49 = \mathbf{103,469}\ \boldsymbol{KN}$

For Shelter - D on Bete – Amannueal

- Details of the shelter
    - Length = 24 m = b
    - Width = 27 m = d
    - Slope = 2.5 % and pitch angle = 1.43^0
    - Reference height = 5.5 m
    - Average cover area of the monument = 214.43 m^2
    - Terrain category = II

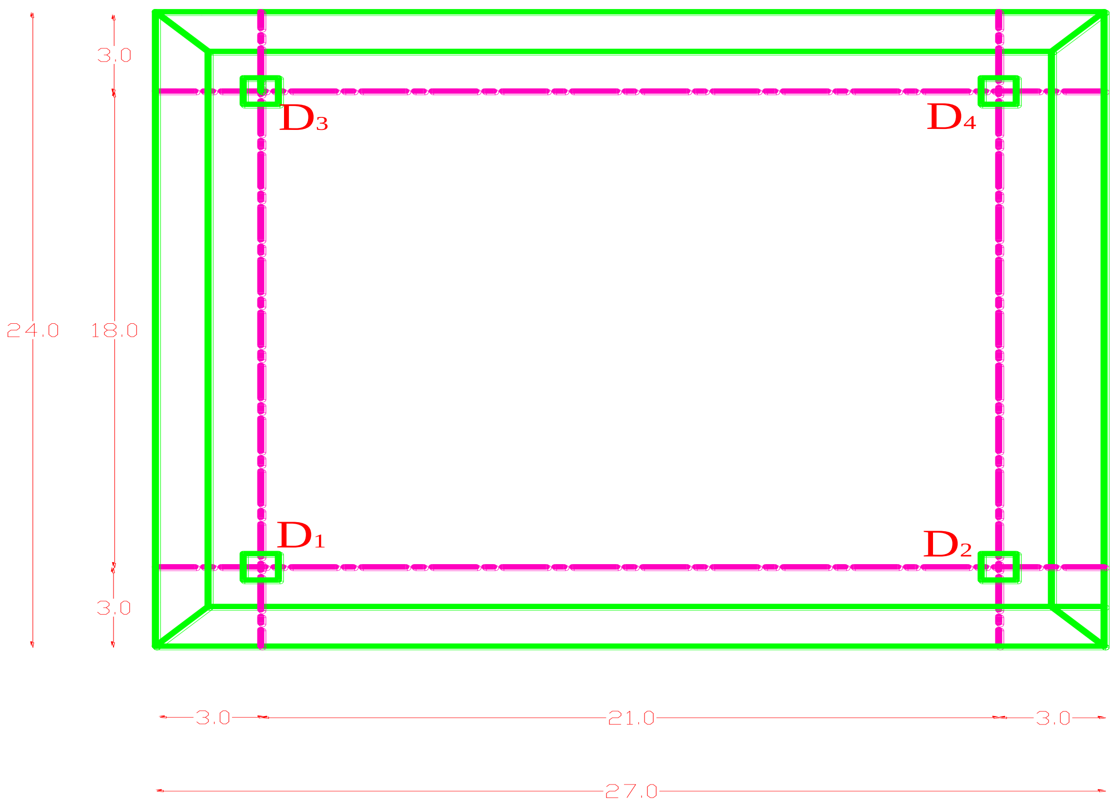


Figure A9:- Cad drawing of the roof plan

Solution

And the terrain of Lalibela can be categorized as terrain category II, Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights, having Z_0 = 0.05 m, Z_{min} = 2 m, Z_{max}= 200 m and reference height Z= 6 m.

Wind load Analysis using design wind speed

Determining the basic wind velocity ($\mathcal{V}_{\mathbf{b}}$)

$$\mathcal{V}_b = C_{dir} \ . C_{season} . \ \mathcal{V}_{b,o}$$

$$= 1 * \ 1 * 29{,}5 \ m/s$$

$$= 29{,}5 \ m/s$$

Determining the mean wind velocity ($\mathcal{V}_{\mathbf{m}}$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z) \ . \mathrm{Co}(Z) \ . \ \mathcal{V}_b$$

$$\mathrm{Cr}(Z) = \mathrm{Kr} \ . \ \ell \mathrm{n} . \left({}^{Z}/_{Z0} \right) \quad \text{For } Z \geq Z \text{ min}$$

$$\mathrm{Kr} = 0{,}19 . \left({}^{Z_o}/_{Z_{0,II}} \right)^{0.07}$$

$$\mathrm{Kr} = 0{,}19 . \left({}^{0{,}05}/_{0{,}05} \right)^{0.07}$$

$$= \underline{\underline{\mathbf{0,1}9}}$$

$$\mathrm{Cr}(Z) = 0{,}19 \ . \ \ell \mathrm{n} \ . \left({}^{5{,}5}/_{0{,}05} \right)$$

$$= \underline{\underline{\mathbf{0,893}}}$$

$\mathrm{Co}(Z) = 1 + 2 \ . S \ . \Phi$, for 0.05 < Φ < 0.3 i.e. shallow where $L_e = L_u$.

The figure in below shows the topographic section detail of Shelter-D taken from ARCCH which is prepared by TEPRIN Associates as a working drawing. Which helps us to find the vertical elevation and the horizontal data for determining the topographic factor.

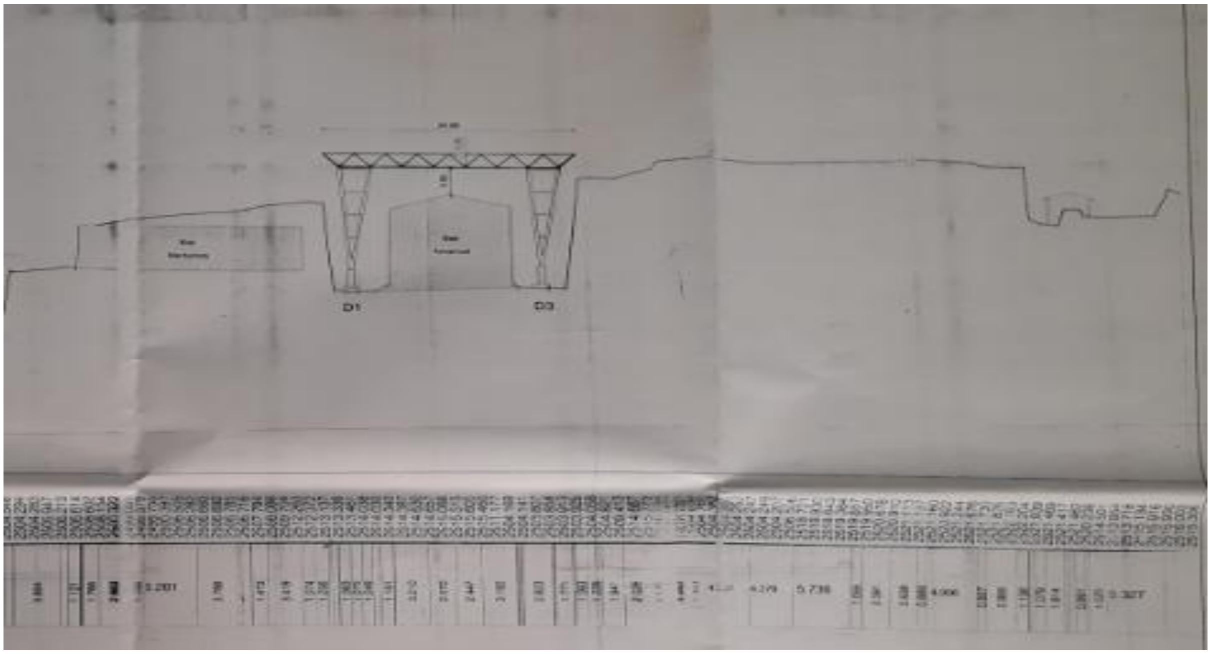


Figure A10:- section drawing for shelter-A (source from ARCCH, working drawing)

-Along 0^0 (South) and 180^0 (North)

From the section drawing $H = 8{,}64\ m$ and $Lu = 63\ m$

$$\Phi = \frac{H}{L_u} = \frac{8{,}64}{63} = 0{,}137$$

$$\mathbf{S} = \boldsymbol{A} \,.\, \boldsymbol{e}^{(\boldsymbol{B} \cdot \frac{X}{L_u})}$$

Conditions to be fulfilled

For the range $-1{,}5 \leq \frac{X}{L_u} \leq 0$ and $0 \leq \frac{Z}{L_e} \leq 2{,}0$

$\frac{-X}{L_u} = \frac{-80}{63} = -1{,}269$ (satisfied)

$\frac{Z}{L_e} = \frac{5{,}5}{63} = 0{,}08$ (satisfied)

$$A = 0{,}1552\,.\left(\frac{z}{L_e}\right)^4 - \ 0{,}8575\,.\left(\frac{z}{L_e}\right)^3 + 1{,}8133\,.\left(\frac{z}{L_e}\right)^2 - 1{,}9115\,.\left(\frac{z}{L_e}\right)^1$$

$$= 0{,}1552\,.(0.08)^4 - \ 0{,}8575\,.(0.08)^3 + 1{,}8133\,.(0.08)^2 - 1{,}9115\,.(0.08)^1$$

$$= -0{,}1417$$

$$B = 0{,}354\,.\left(\frac{z}{L_e}\right)^2 - \ 1{,}0577\,.\left(\frac{z}{L_e}\right)^1 + 2{,}6456$$

$$= 0{,}354\,.(0{,}08)^2 - \ 1{,}0577\,.(0.08)^1 + 2{,}6456$$

$$= \ 2{,}56$$

S = 0.044

Hence

$$\text{Co}(Z\,) = 1 + 2\,.\,0{,}044\,.\,0{,}137 \quad = \underline{1{,}012}$$

Therefore,

$$\mathcal{V}_m(Z_e) \ = \ \ 0{,}893\,. \ 1{,}012 \ . \ 29{,}5 = 26{,}66 \text{ m/s}$$

Determining the peak wind velocity ($\mathcal{V}_\mathbf{p}(\mathbf{Z_e})$)

$$\mathcal{V}_p(Z_e) \ = \mathcal{V}_m(Z_e)\,.\,G$$

Where:-

$$G = \sqrt{1 + 7.\frac{1}{1{,}012\,.\ln({}^{5{,}5}/_{0.05})}} = 1.57$$

$$\boldsymbol{\mathcal{V}_\mathbf{p}(\mathbf{Z_e}) = 26{,}66\,\frac{\mathbf{m}}{\mathbf{s}}.\,1{,}57 = 41{,}85\mathbf{m/s}}$$

Determining the peak velocity pressure ($\mathbf{q_p}$)

$$q_p \ = \ \frac{\rho}{2}.\ (\mathcal{V}_p(Z_e))^2 \qquad\qquad \text{Pr EN 1991 1-4 2004}$$

$$= \frac{0.94}{2} \,.\,(41{,}85)^2$$

$$\boldsymbol{q_p = 0{,}823\ \frac{KN}{m^2}}$$

Determining wind forces

iv. Wind load on external part of the roof covers

The wind force for the whole structure or a structural component should be determined by using one of the recommended method in Pr EN 1991 -1-4: 2004 which is using force coefficient method.

$$F_w = C_s \,.\, C_d \,.\, C_f \,.\, q_p(z_e).\, A_{ref}$$

Determining structural factors ($\mathbf{C_s}$ $\mathbf{C_d}$)

C_s Cd = is the structural factors which are the size factor and dynamic factor respectively

= 1 for structure which are not susceptible to turbulence induced vibration. Pr EN 1991 -1-4: 2004, 6.2 (1) (a)

Determining the force coefficient ($\mathbf{C_f}$) on the external most rigid roof cover

- Roof slope = 2,5%
- Roof (pitch) angle (α) = 0^0
- Reference area of the roof is $= 27 \,.\, 24 = 648\, m2$

⇒ At α = 0^0

$$\boldsymbol{C_{f,exernal}} = -0{,}5$$

Therefore,

The overall wind load on the external roof cover becomes

$$F_w = 1 \,.\, (-0{,}5) \,.\, 0{,}823.\ 648$$

$$\boldsymbol{F_w = -266{,}74}\ \ \textbf{KN}$$

Wind load on side roof covers

Based on the assumptions made on the methodology part, the wind loading analysis is be like;

Basic data's:-

- Height of roof = 1.5 m
- Reference height = 5,5 m
- b = 24 m
- d = 27 m
- e = b or 2h (whichever is the smaller)

= 24 m or $2\,.\,1{,}5 = 3$ m

e = 3 m

Determining areas of the regions:-

- $A = 1{,}5\,.\frac{3}{5} = 0{,}9\ m^2$ $(A < 1\ m^2)$ then use 1 m^2
- $B = \frac{4}{5}\,.\,1{,}5\,.\,3 = 3{,}6\ m^2$ $(1\ m^2 < A < 10\ m^2)$
- $C = (27-3)\,.\,1{,}5 = 36\ m^2$ $(> 10\ m^2)$
- $D = 24\,.\,1{,}5 = 36\ m^2$ $(> 10\ m^2)$
- $E = 24\,.\,1{,}5 = 36\ m^2$ $(> 10\ m^2)$

$$\frac{h}{d} = \frac{1.5}{24} = 0{,}0625 \leq 0{,}25$$

And for the regions with the condition of $(1\ m^2 < A < 10\ m^2)$ we have a formula of;

$Cp_e = Cp_{e,1} - (Cp_{e,1} - Cp_{e,10})log10^A$……….. (Figure 7.2 of Pr EN 1991-1-4)

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p \, . Cp_e$$

Wind pressure for region D

$$W_{e,D} = +0{,}23 \, \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}1 \, \frac{KN}{m^2}$$

Wind load Analysis with the already experienced wind speed

By the year 2013 the Lalibela region experienced 28 m/s 180 minute gust wind speed measured at 10 meter height, therefore we use the wind speed as it is for the sake of checking the structural stability of the structure.

Determining the basic wind velocity ($\mathcal{V}_b$)

$$\mathcal{V}_b = C_{dir} \, . \mathrm{C}_{season} . \, \mathcal{V}_{b,o}$$

$$= 1 * 1 * 28 \, m/s$$

$$= 28 \text{ m/s}$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z) \, . \mathrm{Co}(Z) \, . \, \mathcal{V}_b$$

$$\mathcal{V}_m(Z_e) = 0{,}9096 \, . \, 1{,}012 \, . \, 28 = 25{,}77 \, m/s$$

Determining the peak wind velocity ($\mathcal{V}_p(Z_e)$)

$$\mathcal{V}_p(Z_e) = \mathcal{V}_m(Z_e) . G$$

Where:-

$$\mathcal{V}_p(Z_e) = 25,77 \frac{m}{s} . 1,57 = 40,46 \frac{m}{s}$$

Determining the peak velocity pressure (q_p)

$$q_p = \frac{\rho}{2} . (\mathcal{V}_p(Z_e))^2$$ Pr EN 1991 1-4 2004

$$= \frac{0.94}{2} . (40,46)^2$$

$$q_p = 0,7694 \frac{KN}{m^2}$$

Determining wind forces

Wind load on top of roof covers

$$F_w = C_s . C_d . C_f . q_p(z_e) . A_{ref}$$

The overall wind load on the external roof cover becomes

$$F_w = 1 . (-0,5) . 0,7694 . 648$$

$$F_w = -249,3 \text{ KN}$$

Wind load on side roof covers

For region A

$$Cp_{e,4,5} = Cp_{e,1} = -1.4$$

For region B

$$Cp_{e,1} = -1,1 \text{ and } Cp_{e,10} = -0,8$$

$$Cp_{e,3,6} = -1,1 - (-1,1 - (-0,8)) log10^{3.6}$$

$$Cp_{e,3,6} = -0,933$$

For region C

$$Cp_{e,10} = -0{,}5$$

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p \,.\, Cp_e$$

Wind pressure for region A

$$W_{e,A} = \left(0{,}7694 \,.\, (-1{,}4)\right) .\, 0{,}4 = -0{,}43 \ \frac{KN}{m^2}$$

Wind pressure for region B

$$W_{e,B} = (0{,}7615 \,.\, (-0{,}933)) \,.\, 0{,}4 = -0{,}287 \ \frac{KN}{m^2}$$

Wind pressure for region C

$$W_{e,C} = -0{,}154 \ \frac{KN}{m^2}$$

Wind pressure for region D

$$W_{e,D} = +0{,}215 \ \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}0923 \ \frac{KN}{m^2}$$

-Along 90^0 (West) and 270^0 (East)

Wind load Analysis using design wind speed

Determining the basic wind velocity ($\mathcal{V}_b$)

$$\mathcal{V}_b = C_{dir} \ . C_{season} . \ \mathcal{V}_{b,o}$$

$$= 0{,}85 * 1 * 29{,}5 \ m/s$$

$$= 25{,}075 \ m/s$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z) \ . \mathrm{Co}(Z) \ . \ \mathcal{V}_b$$

$$\mathrm{Cr}(Z) = \mathrm{Kr} . \ \ell n . \left(Z/_{Z0}\right) \quad \text{For } Z \geq Z \text{ min}$$

$$\mathrm{Kr} = 0{,}19 . \left(Z_o/_{Z_{0,II}}\right)^{0.07}$$

$$\mathrm{Kr} = 0{,}19 . \left(0{,}05/_{0{,}05}\right)^{0.07}$$

$$= \underline{\underline{\mathbf{0,19}}}$$

$$\mathrm{Cr}(Z) = 0{,}19 . \ \ell n . \left(5{,}5/_{0{,}05}\right)$$

$$= \underline{\underline{\mathbf{0,893}}}$$

$\mathrm{Co}(Z) = 1 + 2 . S . \Phi$, for $0.05 < \Phi < 0.3$ i.e. shallow where $L_e = L_u$.

The figure in below shows the topographic section detail of Shelter-D taken from ARCCH which is prepared by TEPRIN Associates as a working drawing. Which helps us to find the vertical elevation and the horizontal data for determining the topographic factor.

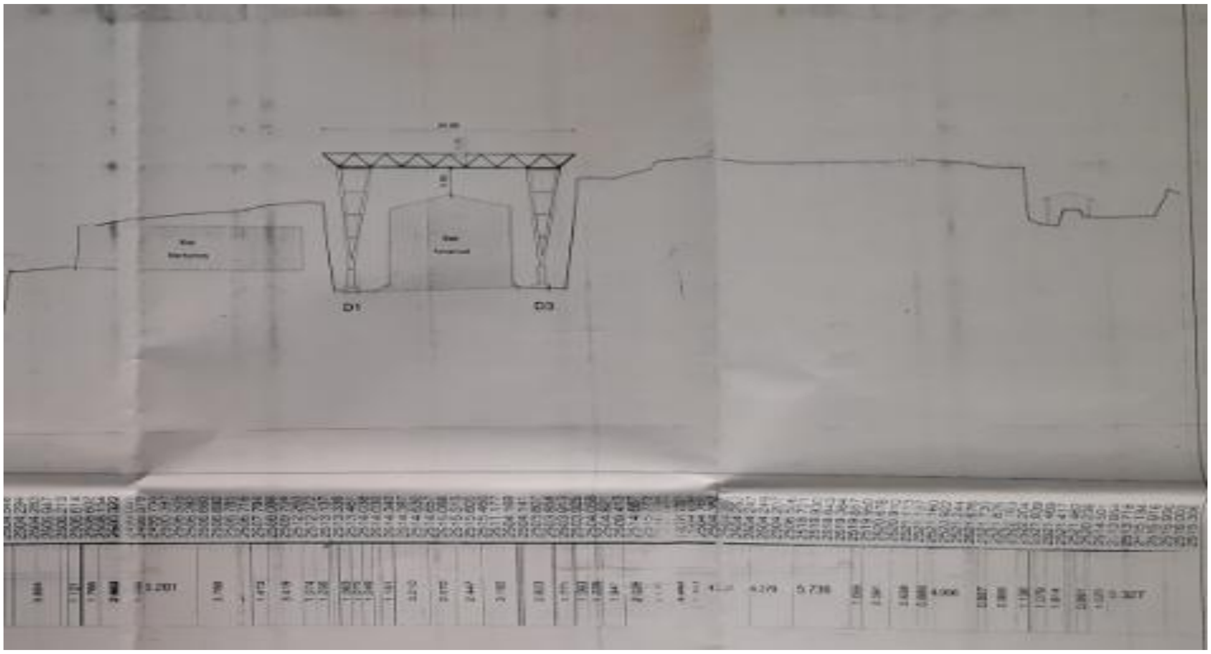


Figure A10:- section drawing for shelter-A (source from ARCCH, working drawing)

From the section drawing $H = 8{,}64\ m$ and $Lu = 63\ m$

$$\Phi = \frac{H}{L_u} = \frac{8{,}64}{63} = 0{,}137$$

$$\mathbf{S} = \boldsymbol{A}\,.\,\boldsymbol{e}^{(\boldsymbol{B}\,\cdot\,\frac{\boldsymbol{X}}{\boldsymbol{L_u}})}$$

Conditions to be fulfilled

For the range $-1{,}5 \le \frac{X}{L_u} \le 0$ and $0 \le \frac{Z}{L_e} \le 2{,}0$

$\frac{-X}{L_u} = \frac{-80}{63} = -1{,}269$ (satisfied)

$\frac{Z}{L_e} = \frac{5{,}5}{63} = 0{,}08$ (satisfied)

$$A = 0{,}1552\,.\left(\frac{Z}{L_e}\right)^4 - 0{,}8575\,.\left(\frac{Z}{L_e}\right)^3 + 1{,}8133\,.\left(\frac{Z}{L_e}\right)^2 - 1{,}9115\,.\left(\frac{Z}{L_e}\right)^1$$

$$= 0{,}1552\,.\,(0.08)^4 - 0{,}8575\,.\,(0.08)^3 + 1{,}8133\,.\,(0.08)^2 - 1{,}9115\,.\,(0.08)^1$$

$$= -0{,}1417$$

$$B = 0{,}354\,.\left(\frac{z}{L_e}\right)^2 - 1{,}0577\,.\left(\frac{z}{L_e}\right)^1 + 2{,}6456$$

$$= 0{,}354\,.\,(0{,}08)^2 - 1{,}0577\,.\,(0.08)^1 + 2{,}6456$$

$$= 2{,}56$$

S = 0.044

Hence

$$\mathrm{Co}(Z) = 1 + 2\,.\,0{,}044\,.\,0{,}137 = \underline{1{,}012}$$

Therefore,

$$\mathcal{V}_m(Z_e) = 0{,}893\,.\,1{,}012\,.\,25{,}075 = 22{,}66\ \mathrm{m/s}$$

Determining the peak wind velocity ($\mathcal{V}_p(Z_e)$)

$$\mathcal{V}_p(Z_e) = \mathcal{V}_m(Z_e)\,.\,G$$

Where:-

$$G = \sqrt{1 + 7.\frac{1}{1{,}012\,.\ln(5{,}5/0.05)}} = 1.57$$

$$\boldsymbol{\mathcal{V}_p(Z_e) = 22{,}66\,\frac{m}{s}.\,1{,}57 = 35{,}6\ m/s}$$

Determining the peak velocity pressure (q_p)

$$q_p = \frac{\rho}{2}.\,(\mathcal{V}_p(Z_e))^2$$ Pr EN 1991 1-4 2004

$$= \frac{0.94}{2}\,.\,(35{,}6)^2$$

$$\boldsymbol{q_p = 0{,}594\ \frac{KN}{m^2}}$$

Determining wind forces

v. Wind load on external part of the roof covers

The wind force for the whole structure or a structural component should be determined by using one of the recommended method in Pr EN 1991 -1-4: 2004 which is using force coefficient method.

$$F_w = C_s \,.\, C_d \,.\, C_f \,.\, q_p(z_e).\, A_{ref}$$

Determining structural factors ($\mathbf{C_s\ C_d}$)

C_s Cd = is the structural factors which are the size factor and dynamic factor respectively

= 1 for structure which are not susceptible to turbulence induced vibration. Pr EN 1991 -1-4: 2004, 6.2 (1) (a)

Determining the force coefficient ($\mathbf{C_f}$) on the external most rigid roof cover

- Roof slope = 2,5%
- Roof (pitch) angle (α) = $\tan^{-1}\frac{2.5}{100}$ = $1{,}43^0$
- Reference area of the roof is $= 27\,.\,24 = 648\ m2$

⇒ At α = $1{,}43^0$

$$\left.\begin{array}{ll} \alpha = 0^0 & -0.5 \\ \alpha = 1{,}43 & C_{f_{1,43}} \\ \alpha = 5 & -0.7 \end{array}\right\} \quad \frac{0-1{,}43}{0-5} = \frac{-0{,}5-\boldsymbol{C_{f,exernal}}}{-0.5+0{,}7} \quad \boldsymbol{C_{f,exernal}} = -0{,}5572$$

Therefore the net upwind force coefficient $\boldsymbol{C_{f,external} = -0.5572}$

Therefore,

The overall wind load on the external roof cover becomes

$$F_w = 1\,.\,(-0{,}5572)\,.\,0{,}594\,.\,\ 648$$

$$\boldsymbol{F_w = -214,8}\ \ \textbf{KN}$$

Wind load on side roof covers

Based on the assumptions made on the methodology part, the wind loading analysis is be like;

Basic data's:-

- Height of roof = 1.5 m
- Reference height = 5,5 m
- b = 24 m
- d = 27 m
- e = b or 2h (whichever is the smaller)

  = 24 m or $2\,.\,1{,}5 = 3$ m

  e = 3 m

Determining areas of the regions:-

- $A = 1{,}5\,.\,\frac{3}{5} = 0{,}9\ m^2$ ($A < 1\ m^2$) then use 1 m^2
- $B = \frac{4}{5}\,.\,1{,}5\,.\,3 = 3{,}6\ m^2$ ($1\ m^2 < A < 10\ m^2$)
- $C = (27 - 3)\,.\,1{,}5 = 36\ m^2$ ($> 10\ m^2$)
- $D = 24\,.\,1{,}5 = 36\ m^2$ ($> 10\ m^2$)
- $E = 24\,.\,1{,}5 = 36\ m^2$ ($> 10\ m^2$)

Determining the force coefficient ($\mathbf{C_f}$) on the inner air permeable roof cover

$$\frac{h}{d} = \frac{1.5}{24} = 0{,}0625 \leq 0{,}25$$

And for the regions with the condition of ($1\ m^2 < A < 10\ m^2$) we have a formula of;

$Cp_e = Cp_{e,1} - (Cp_{e,1} - Cp_{e,10})log10^A$……….. (Figure 7.2 of Pr EN 1991-1-4)

For region A

$$Cp_{e,4,5} = Cp_{e,1} = -1.4$$

For region B

$$Cp_{e,1} = -1{,}1 \; and \; Cp_{e,10} = -0{,}8$$

$$Cp_{e,3,6} = -1{,}1 - (-1{,}1 - (-0{,}8))log10^{3.6}$$

$$Cp_{e,3,6} = -0{,}933$$

For region C

$$Cp_{e,10} = -0{,}5$$

For region D

$$Cp_{e,10} = +0{,}7$$

For region E

$$Cp_{e,10} = -0{,}3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p \,.\, Cp_e$$

Wind pressure for region A

$$W_{e,A} = \left(1{,}239 \,.\, (-1{,}4)\right) .\, 0{,}4 = -0{,}694 \; \frac{KN}{m^2}$$

Wind pressure for region B

$$W_{e,B} = (1{,}239 \,.\, (-0{,}933)) \,.\, 0{,}4 = -0{,}46 \; \frac{KN}{m^2}$$

Wind pressure for region C

$$W_{e,C} = -0{,}2478\ \frac{KN}{m^2}$$

Wind pressure for region D

$$W_{e,D} = +0{,}3469\ \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0{,}148\ \frac{KN}{m^2}$$

Wind load Analysis with the already experienced wind speed

By the year 2013 the Lalibela region experienced 28 m/s 180 minute gust wind speed measured at 10 meter height, therefore we use the wind speed as it is for the sake of checking the structural stability of the structure.

Determining the basic wind velocity ($\mathcal{V}_b$)

$$\mathcal{V}_b = C_{dir}\ .\mathrm{C}_{season}.\ \mathcal{V}_{b,o}$$

$$= 0{,}85 * 1 * 28\ m/s$$

$$= 23{,}8\ m/s$$

Determining the mean wind velocity ($\mathcal{V}_m$)

$$\mathcal{V}_m(Z_e) = \mathrm{Cr}(Z)\ .\mathrm{Co}(Z)\ .\ \mathcal{V}_b$$

$$\mathcal{V}_m(Z_e) = 0{,}9096\ .\ 1{,}012\ .\ 23{,}8 = 21{,}9\ m/s$$

Determining the peak wind velocity ($\mathcal{V}_p(Z_e)$)

$$\mathcal{V}_p(Z_e) = \mathcal{V}_m(Z_e)\ .\ G$$

Where:-

$$\boldsymbol{\mathcal{V}_p(Z_e) = 21,9\frac{m}{s}.1,57 = 34,4\ \frac{m}{s}}$$

Determining the peak velocity pressure (q_p)

$$q_p = \frac{\rho}{2}.\ (\mathcal{V}_p(Z_e))^2$$ Pr EN 1991 1-4 2004

$$= \frac{0.94}{2}\ .(44,4)^2$$

$$\boldsymbol{q_p = 0,556\ \frac{KN}{m^2}}$$

Determining wind forces

Wind load on top of roof covers

$$F_w = C_s\ .C_d\ .C_f\ .q_p(z_e).A_{ref}$$

The overall wind load on the external roof cover becomes

$$F_w = 1\ .(-0,5572)\ .0,556\ .\ 648$$

$$\boldsymbol{F_w = -200,82\ \text{KN}}$$

Wind load on side roof covers

For region A

$$Cp_{e,4,5} = Cp_{e,1} = -1.4$$

For region B

$$Cp_{e,1} = -1,1\ and\ Cp_{e,10} = -0,8$$

$$Cp_{e,3,6} = -1,1 - (-1,1 - (-0,8))log10^{3.6}$$

$$Cp_{e,3,6} = -0,933$$

For region C

$$Cp_{e,10} = -0,5$$

For region D

$$Cp_{e,10} = +0,7$$

For region E

$$Cp_{e,10} = -0,3$$

Therefore,

The wind pressure in each distinguished regions become;

We also have to note that the effective area to be loaded is only 40% of the total area of the side cover for their specific regions because of the uniformly distributed openings.

$$W_e = q_p \, . Cp_e$$

Wind pressure for region A

$$W_{e,A} = \left(0,7694 \, . (-1,4)\right) . 0,4 = -0,43 \; \frac{KN}{m^2}$$

Wind pressure for region B

$$W_{e,B} = (0,7615 \, . (-0,933)) \, . 0,4 = -0,287 \; \frac{KN}{m^2}$$

Wind pressure for region C

$$W_{e,C} = -0,154 \; \frac{KN}{m^2}$$

Wind pressure for region D

$$W_{e,D} = +0,215 \; \frac{KN}{m^2}$$

Wind pressure for region E

$$W_{e,D} = -0,0923 \; \frac{KN}{m^2}$$

Imposed loads on the structures

From the base plates and counter weights

- weight of base plate with 20mm thickness

$$\text{Weight} = 76{,}9729\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}02 = \mathbf{4{,}99}\ \boldsymbol{KN}$$

- weight of a counter mass plate

$$\text{Weight} = 76{,}9729\ .\ [(1{,}8\ .\ 1{,}8) - (0{,}67\ .\ 0{,}67)].\ 0{,}01 = \mathbf{2{,}148}\ \boldsymbol{KN}$$

And the number of counter weight varies from column to column, so the weight of the counter mass plate in each column is;

$$\text{For all column bases} = \mathbf{27\ .\ 2{,}148 = 58}\ \boldsymbol{KN}$$

- weight of L-shaped plate

$$\text{Weight} = 76{,}9729\ .\ [4\ .\ (0{,}11 + 0{,}11 + 0{,}2]\ .\ 1{,}8\ .\ 0{,}01 = \mathbf{3{,}49}\ \boldsymbol{KN}$$

From the concrete foundation

A concrete foundation is used in every column with different thickness and similar sizes, it is used as a counter wait and also as surface leveling so that the counter mass plates could place simply. The concrete is grade C-25 Mpa that is found from those involved in the project. The thickness of the pad is taken to be the average of the four side thickness.

At base of the column D1 $= 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}38 = \mathbf{30{,}78}\ \boldsymbol{KN}$

At base of the column$D2 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}36 = \mathbf{29{,}16}\ \boldsymbol{KN}$

At base of the column $D3 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}3 = \mathbf{24{,}3}\ \boldsymbol{KN}$

At base of the column $D4 = 25\ .\ 1{,}8\ .\ 1{,}8\ .\ 0{,}29 = \mathbf{23{,}49}\ \boldsymbol{KN}$

Therefore, the total weight in the base column is;

At base of the column $D1 = 30{,}78 + 58 + 4{,}99 + 3{,}49 = \mathbf{97{,}26}\ \boldsymbol{KN}$

At base of the column $D2 = 29{,}16 + 58\ + 4{,}99 + 3{,}49 = \mathbf{95{,}64}\ \boldsymbol{KN}$

At base of the column $D3 = 24{,}3 + 58\ + 4{,}99 + 3{,}49 = \mathbf{90{,}78}\ \boldsymbol{KN}$

At base of the column D4 $= 23{,}49 + 58\ + 4{,}99 + 3{,}49 = \mathbf{89{,}97}\ \boldsymbol{KN}$