FLOR DA MANHÃ

an inclusive preschool shaped by nature in Xai-Xai, Mozambique

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The current thesis follows the APA quotation style.

SUMMARY

ABSTRACT

Education is the foundation of our existence, an inalienable right that every individual should possess, defend and be guaranteed. Nevertheless, illiteracy is one of the most severe problems in the world, especially in Africa where the right to education is all too often an authentic mirage. In our proposal, children and nature have been co-stars of the entire design process: the former as intended users; the second as the main resource to benefit from. Thus earth, air, water and fire, the elements from which everything is formed, have enabled us to develop a project for the community, arguing against the common idea that preschools are luxurious and affordable only for the few.

'Flor da manhã' is a place of tolerance, respect, cooperation and happiness. It is a place where children's well-being is put first. It is a place where they can feel safe, learn, play and discover.
'Flor da manhã' is a gathering place design to be suitable for every child, based on five pillars: nature, interaction, protection, inclusion and joy.
The natural elements dictate how buildings integrates within the context, thus defining the functional layout of the project. The result is a dynamic place, where kid-friendly spaces and nature constantly interact, also encouraging children's inclusion and integration. Simplicity and economy have been common threads of the project that aims at self-sufficiency and sustainability. Regular and simple shapes have been combined with the use of local materials and resources, respecting and enhancing the precious value of nature in all its shades.

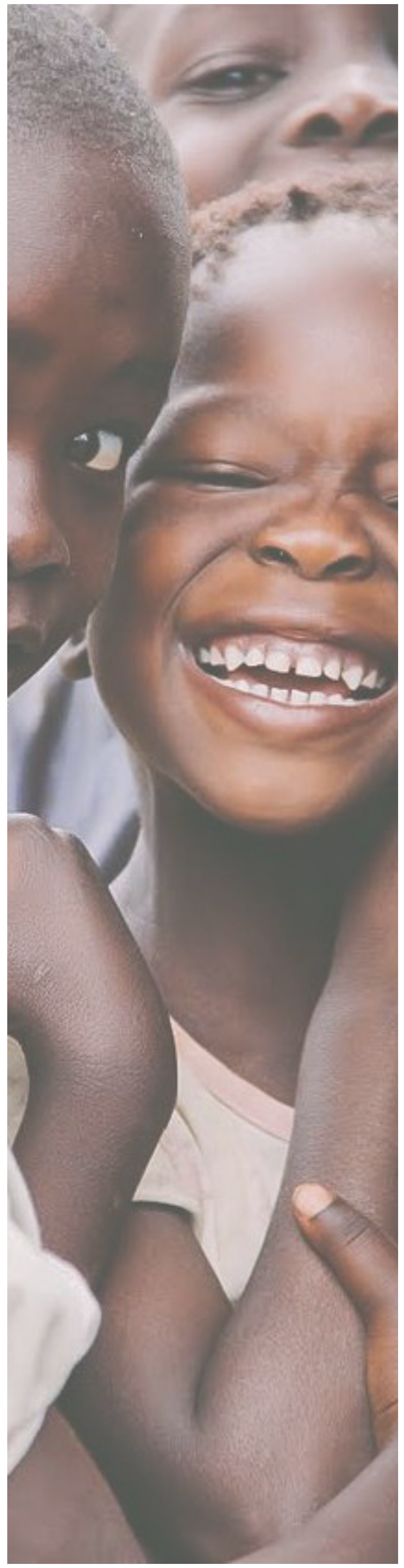
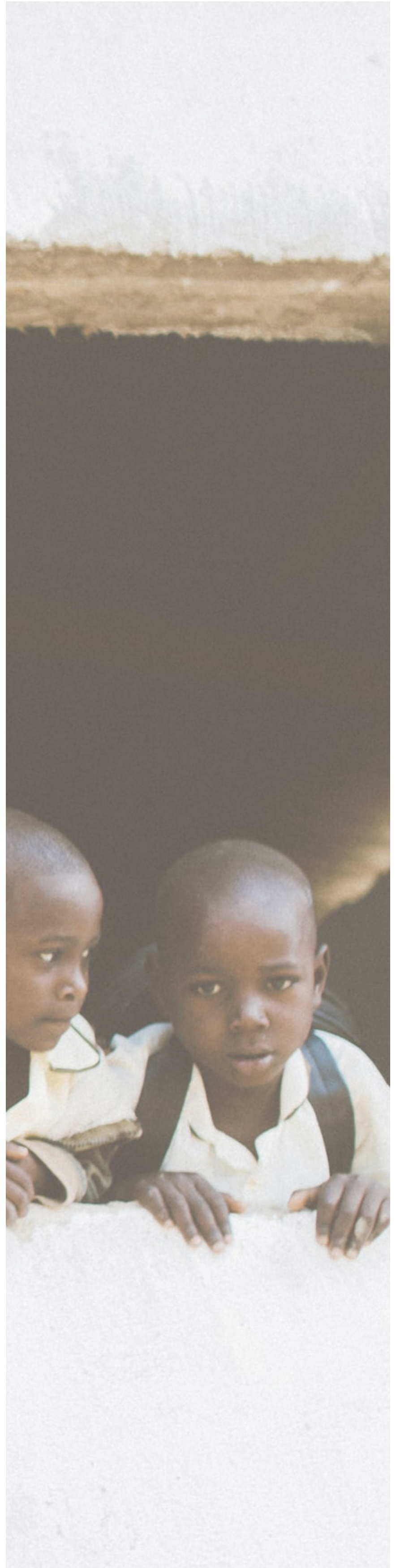


Fig.1.1 - Smiling kids. Source: https://www.web.500px.com

Fig.1.2 - Kids at school. Source: https://www.unsplash.com

“Everyone has the right to education. Education shall be free, at least in the elementary and fundamental stages. Elementary education shall be compulsory. Technical and professional education shall be made generally available and higher education shall be equally accessible to all on the basis of merit.
Education shall be directed to the full development of the human personality and to the strengthening of respect for human rights and fundamental freedoms. It shall promote understanding, tolerance and friendship among all nations, racial or religious groups, and shall further the activities of the United Nations for the maintenance of peace. Parents have a prior right to choose the kind of education that shall be given to their children.”

Art.26, The Universal Declaration of Human Right



0.1. INTRODUCTION

Education is the foundation of our existence, an inalienable right that every individual should possess, guarantee and defend. Nevertheless, illiteracy is one of the most serious problems in the world, especially in Africa where the right to education is all too often an authentic mirage. Indeed, one in three children is denied the right to study: in many cases, families are too poor to pay school fees and books to their children; in other cases, schools are too far away or even non-existent; sometimes parents themselves do not understand how important education is for the future of their own children, dooming children to be trapped in a vicious circle of poverty, discrimination and ignorance.

In the last decades, progress have been made in primary school enrolment, but there is still too little public understanding of the importance of an early childhood education programme. A great body of evidence has shown that mental stimulation through education and an enriched environment are fundamental for a healthy brain development. But the extreme poverty conditions in which many families live in Africa make these basic needs difficult to be met, thus hindering childrens' development.

It is clear that investing in childhood learning is an urgency worldwide and, above all, in African countries. Prioritizing this issue would benefit communities in terms of prosperity and economic growth, contributing to end extreme poverty.

It is precisely for this real need for schools that the thesis project presented in the following pages is inspired by the international competition "Mozambique Preschool: Flor da manhã" arranged by Archstorming. To conclude our university careers, we wanted to challenge ourselves facing a real and concrete problem. Operating in a completely different context than usual, there have been difficulties and obstacles. But the knowledge acquired over these years has enabled us to find a solution which is coherent with the context. Simplicity and economy have been the common threads of the project that aim at self-sufficiency and sustainability. Hence, children and nature have been co-stars of the entire design process: the former as intended users; the second as the main resource to benefit from. Thus earth, air, water and fire, the elements from which everything is born, have enabled us to develop a project for the community, arguing against the common idea of preschools being luxurious and affordable only for the few.

0.2. THE COMPETITION

As architecture competition organiser, Archstorming seeks to resolve humanitarian issues through architecture.
Given the harsh realities and environments lived by many people in need, they create humanitarian architecture competitions to prove the world how design can go beyond the buildings' wall and actually make a difference improving the wellness of people.

This competition focuses on children in Mozambique, a country where 70% of the population lives below the poverty line. Many families are struggling to cover even their most basic needs, such as running water, proper sanitation and regular access to food. In this context, education takes a back seat: an estimated 1.2 million children do not attend school at all. Among them, most of the children with disabilities are included since families prefer to hide them to keep them safe from violence and discrimination.

From this difficult situation arises the will of Assa, a Mozambican teacher, to build a preschool where everyone has a space, something that does not exist yet in the Gaza Province. Therefore, the challenge is not only to create a school in an underdeveloped country, but also to design it for disabled and socially excluded children.

The school will be located in the Xai-Xai District (Gaza Province), in a plot between the cities of Xai-Xai and Chongoene, about 1km

away from the road that connects them. The site characteristics and the current topography are very important, since the school will be built by volunteers and construction workers with no heavy equipment help. Therefore, the constructive system should be easy to build and adapted to the topography.
The goal of the competition is to design a sustainable, integrating and child-friendly preschool. As it will accommodate disabled children and kids in social exclusion, it is fundamental to create a dynamic school where they can interact with each other and the surrounding. Therefore, adapted spaces should be designed so children feel involved rather than rejected.
The preschool should be a place where kids feel comfortable, where they can safely play, run, learn and discover. Thus, it becomes fundamental to design child-friendly spaces to help them enjoy the beginning of their educational journey.
Moreover, the project should be as sustainable as possible: locally source materials should be used and energy self-sufficiency should be guaranteed.

To achieve the objectives set, an indicative program is proposed. The competition brief is open to additions and improved development strategies within the limited resources and financial capacities of the project.

Children-addressed spaces

The school has to be designed to accommodate many children from 0 to 5 years old. At least 25 students for each class are expected. Flexibility of spaces should be a leading parameter in the design process; each class should fit to different activities, from learning to playtime, as well as naptime. A multipurpose space should be included in the project intended for different educational activities, celebrations or parent meetings.
In preschool, children learn and develop cognitive and social skills through play. Aiming at fostering the interaction of children with the surrounding environment, it becomes crucial to create an open-air space where kids can play and have contact with nature protected from any weather conditions.

Adult-addressed spaces

A personal space for teachers during class breaks or meetings, and offices for the school management should be provided.

Services

Chronic malnutrition in Mozambique is a serious public health concern affecting one every two children under the age of 5. In this context, education to nutrition becomes fundamental. The project should enable the children to learn the importance of a varied diet not only eating different types of food, but also taking care and cultivating their own. Particular attention has to be paid to food conservation: creating an effective pantry is a huge challenge in Mozambique because of the climatic conditions and the abundant and resistant variety of insect and other pests.

Due to the lack of access to improved water supply and sanitation facilities all over the country, providing separated bathrooms for boys and girls (for both children and adults), as well as guaranteeing wastewater treatment and water supply become fundamental. Aiming at a sustainable self-sufficiency, solar panels and thermal collectors should be included in the project. Moreover, it would be interesting to consider a rainwater collection system.

Fig.3.1 - Project site. Source: ArchStorming competition material.

03
ARCHITECTURAL DESIGN

3.1. INTRODUCTION

‘Flor da manhã’ is a place of tolerance, respect, cooperation and happiness. It is a place where children’s well-being is put in the first place. It is a place where they can feel safe, learn, play, run and discover.

Based on five pillars: nature, interaction, protection, inclusion and joy, ‘Flor da manhã’ is a gathering place design to be suitable for every child.
The natural elements dictate how buildings integrates within the context, thus defining the functional layout of the project.
The preschool complex results in a dynamic place, where kid-friendly spaces and nature constantly interact, encouraging also children’s inclusion and integration.
This proposal seeks to respect and enhance the precious value of nature in all its shade to teach children the proper way to interact with the natural environment. Every design choice has been taken with the aim of minimizing the overall environmental impact, thus maximizing the sustainability of the project.

EDUCATION	dynamic spaces school bus educational open spaces
SUSTAINABILITY	local materials existing natural environment natural resources
INTEGRATION	teamwork by volunteers no architectural barriers aggregation points

3.2. PRELIMINARY DESIGN

3.2.2 PROJECT SITE EVOLUTION

The design phase began with the identification of the basic requirements the project should fulfil. Together with the points outlined in the competition brief, the urban analysis and the collected data led us to define more specific and fundamental characteristics for a preschool project in an underdeveloped country.
The project will be built by an NGO with the help of local volunteers and constructors, with limited resources and financial capacity. Thus, the ease of execution has been one of the key points guiding us in the design phase. This concept has been translated into regular and simple shapes that make the project as intuitive as possible from the construction point of view. All this has been combined with the use of local materials and resources to meet the low costs requirement.
Since heavy equipment like excavators are not affordable nor achievable for an NGO working in a third world country, it has been fundamental to take the site topography into account. The topography study reported in the competition brief reveals that there is a slight slope in the plot: the highest point has an altitude of 63 m while the lowest point has an altitude of 60 m. Considering that the ground will be prepared with small equipment and that the school is intended for vulnerable and disabled children, we decided to preserve the natural slope of the land and integrate the buildings in order not to create architectural barriers. This will help children to feel comfortable and not rejected by the society, which is a widespread feeling among disabled children as, in Mozambique, they are usually vulnerable to discrimination and violence. Thus, even the security of children assumes high importance to fight against the limited opportunities they have to pursue an education.

teamwork by volunteers

local materials

no architectural barriers

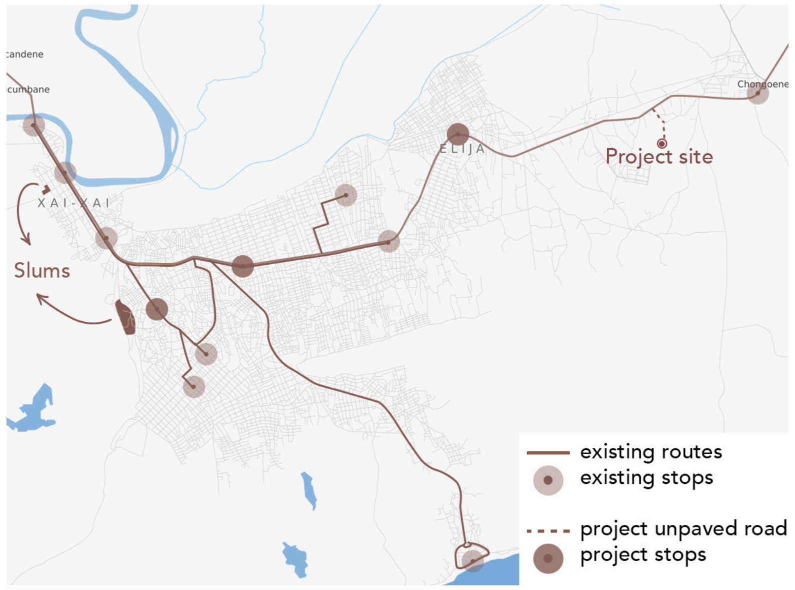


Fig.3.2 - Project site connection

school bus

The project site is located in a particularly isolated area and children will need to travel long distances to reach it. To allow all children to get to school safely, it is essential to organize a minibus transport system with stops located along the primary road. A new unpaved road will be built to provide for a direct connection to the preschool so as it will be easily reached.

dynamic spaces

To protect children from external dangers, it has been necessary to delimit the project site boundaries. Therefore, we decided that the building itself could serve as a barrier, thus creating a safe environment with a conscious use of resources.

existing natural environment

natural resources

Together with the safeguard and respect of the existing natural environment, this latter concept has been one of the main pivotal points of the entire project. Indeed, sustainability in all its shades is a prerogative. The careful use of the available natural resources played a role of fundamental importance in the design phase. On one hand, the study of the sun path and the analysis of the main wind flow direction has led us to define the most suitable orientation for reaching good indoor comfort conditions with the lowest possible environmental impact. On the other hand, the frequency with which cyclones hit the country has led us to take some precautions to reduce the vulnerability of the project. The building, initially idealized as a single construction, has then been separated,

giving rise to a series of blocks of regular shape and appropriate height, placed at a safe distance both from each other and from the existing vegetation. Indeed, since the connection with nature is fundamental, trees have been intended not as obstacles to the design, but as advantages of the site. Their positions helped define the layout of the school complex, partly determining the arrangement of the green areas and open spaces.

aggregation points

educational open spaces

It is well known that spending time outdoors and learning to interact with nature are two very important aspects that should not be overlooked in the education of every child. The open spaces have been therefore arranged to guarantee children interaction with nature from every area of the school complex. To emphasize the strong role of nature within the project, the courtyard has been intended as the green core of the preschools; the covered playground has been surrounded by educational gardens where children can learn about care and land cultivation.

3.2.3 BUILDINGS EVOLUTION

no architectural barriers

To preserve the natural topography, each building has been conceived as an element that is directly connected with the ground, rather than excavated into it. The grafting point of each building coincides with its entrance: to avoid the creation of architectural barriers and therefore guarantee accessibility to each child, buildings are placed at different heights of the topography, minimizing additions and removals of soil. The height is reached thanks to retaining walls that act as a basement intrinsically modeled by the topography. Thus, minimizing the impact of construction activities on the existing natural environment.

local materials

Above these elements, rammed earth walls and wooden ceilings define the interior spaces. The use of these materials combined with simple and regular shapes makes the proposal advantageous not only in terms of construction, but also in economic and environmental terms: wood is abundantly available in the region and traditionally used in construction; raw materials for rammed earth are available locally, reducing transport costs and therefore the overall emissions footprint.

natural resources

Designing in a third world country entails limited financial capacity thus it is fundamental to carefully and consciously use the available natural resources. In these terms, we decided to provide indoor comfortable conditions taking advantage of the natural wind flow. Openings are located according to the main wind direction on southeastern and northwestern facades allowing for air flowing within the indoor environment.
To further improve indoor comfort, buildings are covered with protruding roofs lying on timber trusses, providing protection against solar gains and collecting rainwater. A ventilated space therefore results between the roof and the ceiling which, in addition, foster the removal of the exhausted air rising from the indoor environment.

As previously mentioned, Xai-Xai region is particularly affected by cyclones which is why each opening is equipped with shutter with

orientable blades. The shutters placed on the northwestern facade are fixed to additionally act as barriers for children when windows are open.

aggregation points

Finally, to guarantee constant connection with nature in any weather condition, verandas have been designed, integrating built and natural environment and thus acting as filters. These sheltered spaces allow children's interaction with each other and nature, providing for a further protection against solar gains of indoor environment.

3.3. FLOR DA MANHÃ PRE-SCHOOL PROJECT

'Flor da manhã' kindergarten is located in such a way as to serve both Xai-Xai and the neighbouring city of Chongoene, in a plot approximately 1 km away of the principal road. From there, the fastest way to reach the school is to travel along a dig and bumpy road and to cross large unused areas. At present, the project site is not connected to the main road by a precise and defined road, which is why our project involves the construction of a dirt road and a pedestrian sidewalk along three sides of the school.

The fourth side is instead occupied by a portion of land which, in accordance to what is reported in the architectural call, is part of the lot but is not buildable. Therefore, the realisation of cultivated fields and orchards was hypothesised: they could bring benefits not only to the school, but also to the entire community.

The concept of food self-production, as previously seen in this chapter, assumes a relevant role for the school situation. This is why children learn to take care of crops from a very early age, with educational fields and gardens dedicated to them. For this purpose, all the necessary tools such as garden equipment and clothes are placed in the areas between the classrooms, to be easily accessible by kids.

Lastly, the relationship with nature also translates into physical benefits for people. In a climatic context such as the Mozambican one, the search for shade assumes a very important role not only to ensure indoor comfortable condition, but also to allow a full enjoyment of outdoor spaces. Where it is possible, the role has been entrusted to vegetation: on one side, the Marula with its thick foliage gives life to a small oasis of peace and tranquillity in which to find freshness in the torrid African days; on the other the orchard, whose plants together with the veranda of the multipurpose space create a large shaded space that can be enjoyed all day.

Fig.3.3 - Project site. Source: ArchStorming competition material.

MASTERPLAN

1:500

PLANS WITH DIMENSION: LEVEL I

1:250

PANTRY: 31 m²
to suit shelves and at least 2 fridges

near service entrance to facilitate loading and unloading goods operations.

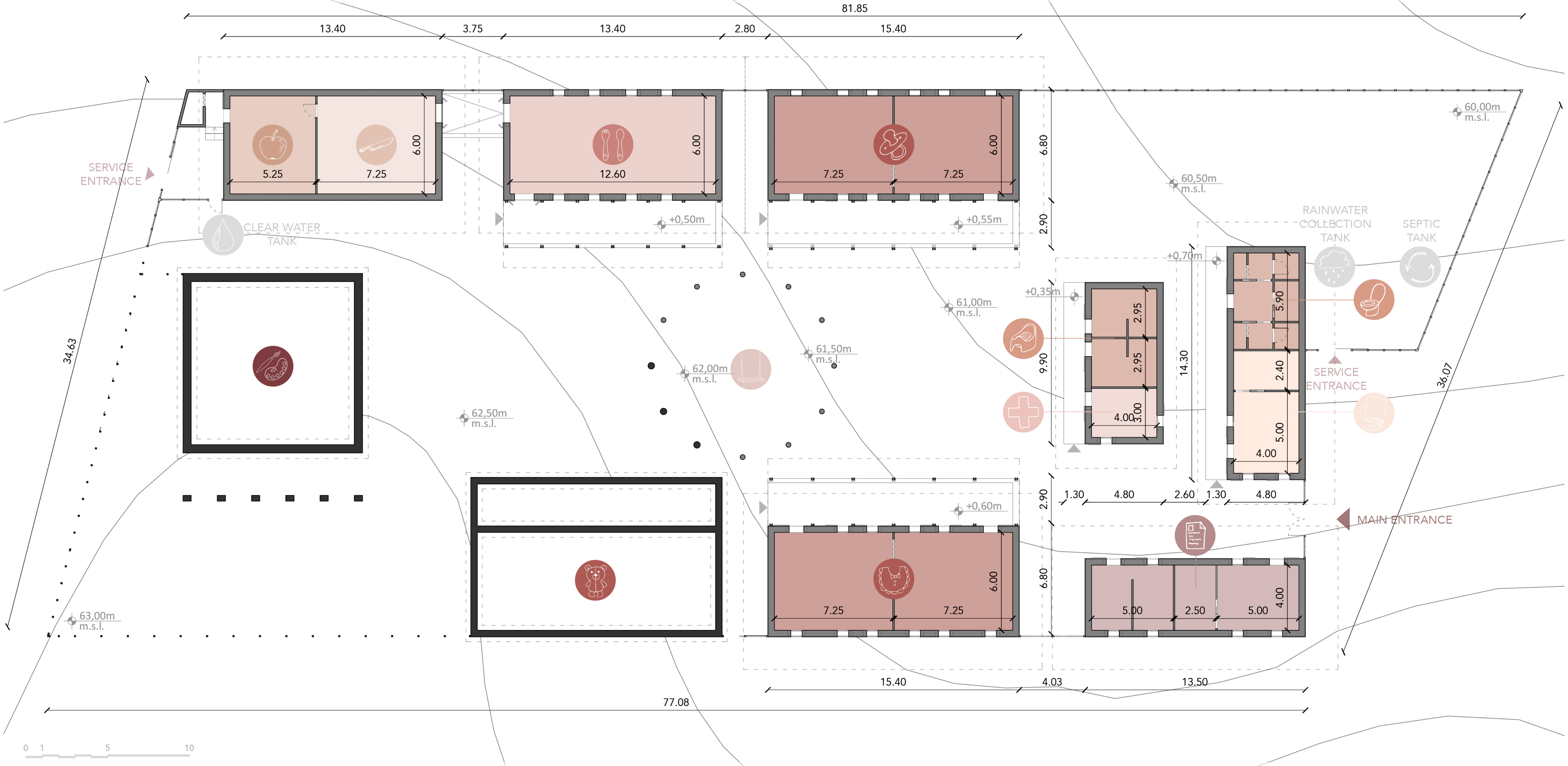
KITCHEN: 43 m²
to suit industrial cook stove, kitchen appliances and at least 2 freezers

DINING ROOM: 75 m²
designed as an open space to promote socialization and integration among children and facilitate teachers supervision

0 - 1 y.o. CHILDREN CLASSROOMS:
42 m² each
classroom are separeted via a partition wall with a big opening to allow for socialization and foster interaction among children, at any time and in any weather condition

COVERED SCHOOLYARD: 100 m²
designed as a permeable and light structure in the core of the site

BATHROOMS includes:
- adults' bathroom 23 m²
- children's bathroom 24 m²



2 - 3 y.o. CHILDREN CLASSROOMS:
42 m² each
classroom are separeted via a partition wall with a big opening to allow for socialization and foster interaction among children, at any time and in any weather condition

INFIRMARY: 12 m²
with a sink and a shelving, high enough to be inaccessible to kids.

TEACHERS' SPACE includes:
- teachers' room 20 m²
- storage (school material) 9 m²

ADMINISTRATION DEPARTMENT includes:
- principal and secretary office 20 m²
- storage (cleaning material) 10 m²
- management and janitory office 20 m²

PLANS WITH DIMENSION - IInd LEVEL

1:250

PANTRY: 31 m²
to suit shelves and at least 2 fridges

near service entrance to facilitate loading and unloading goods operations.

KITCHEN: 43 m²
to suit industrial cook stove, kitchen appliances and at least 2 freezers

OPENAIR KITCHEN: 12 m²
low walls prevent kids from getting in

DINING ROOM: 75 m²
designed as an open space to promote socialization and integration among children and facilitate teachers supervision

0 - 1 y.o. CHILDREN CLASSROOMS:
42 m² each
classroom are separeted via a partition wall with a big opening to allow for socialization and foster interaction among children, at any time and in any weather condition

COVERED SCHOOLYARD: 100 m²
designed as a permeable and light structure in the core of the site

BATHROOMS includes:
- adults' bathroom 23 m²
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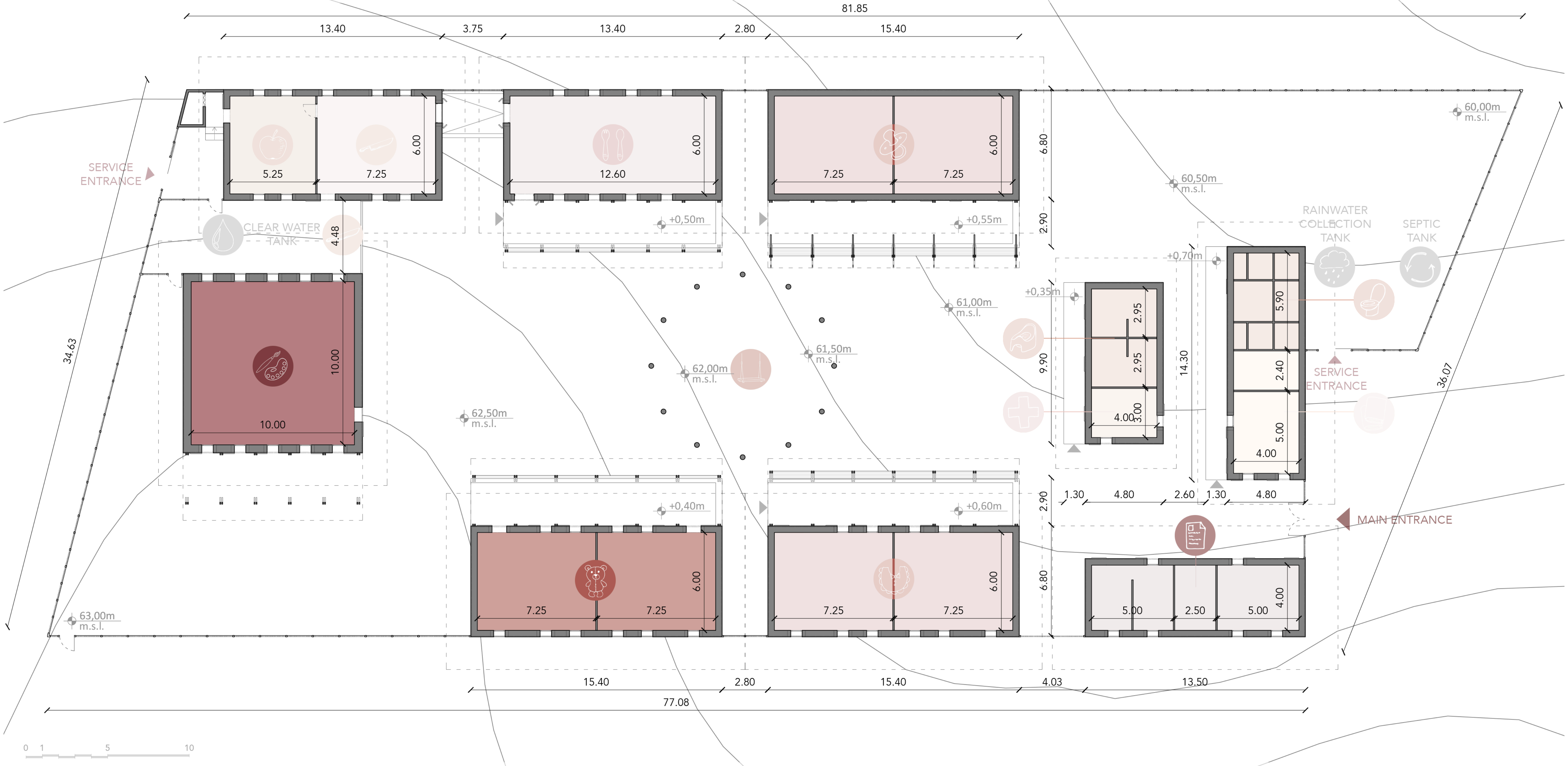

MULTIPURPOSE SPACE: 100 m²
designed as a big open space:
- to suit workshop activities
- to host children of different classroom at a time

4 - 5 y.o. CHILDREN CLASSROOMS:
42 m² each
classroom are separeted via a partition wall with a big opening to allow for socialization and foster interaction among children, at any time and in any weather condition

2 - 3 y.o. CHILDREN CLASSROOMS:
42 m² each
classroom are separeted via a partition wall with a big opening to allow for socialization and foster interaction among children, at any time and in any weather condition

INFIRMARY: 12 m²
with a sink and a shelving, high enough to be inaccessible to kids.

TEACHERS' SPACE includes:
- teachers' room 20 m²
- storage (school material) 9 m²

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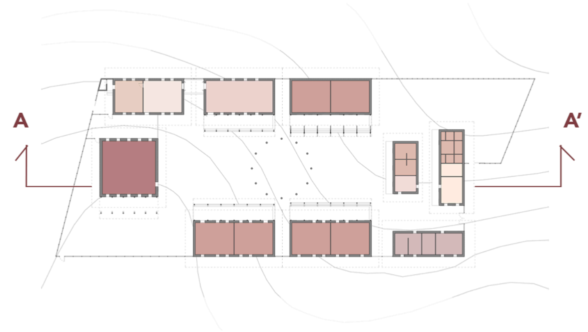
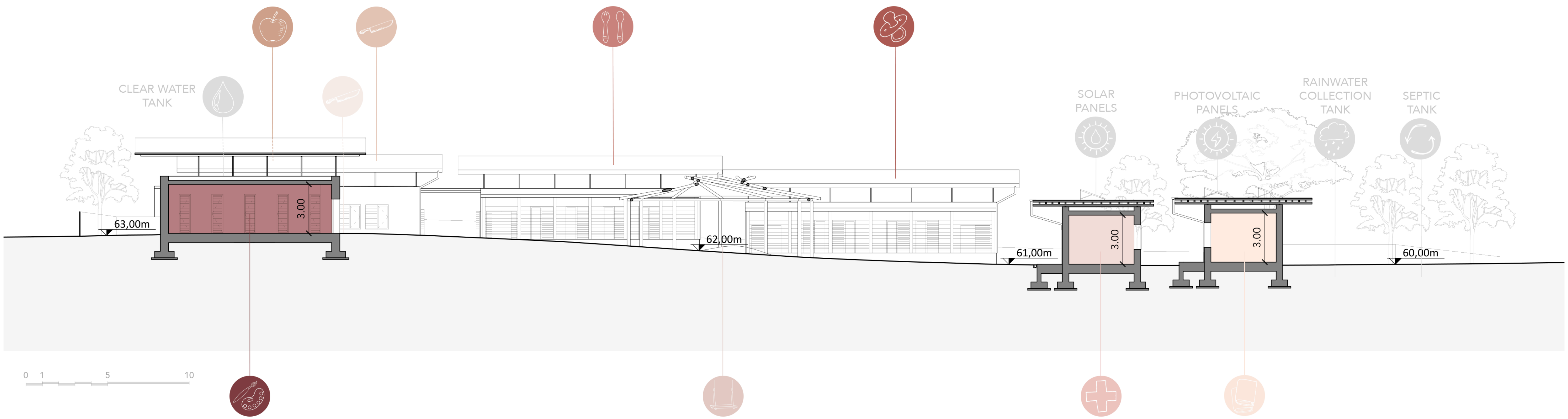
- management and janitory office 20 m²

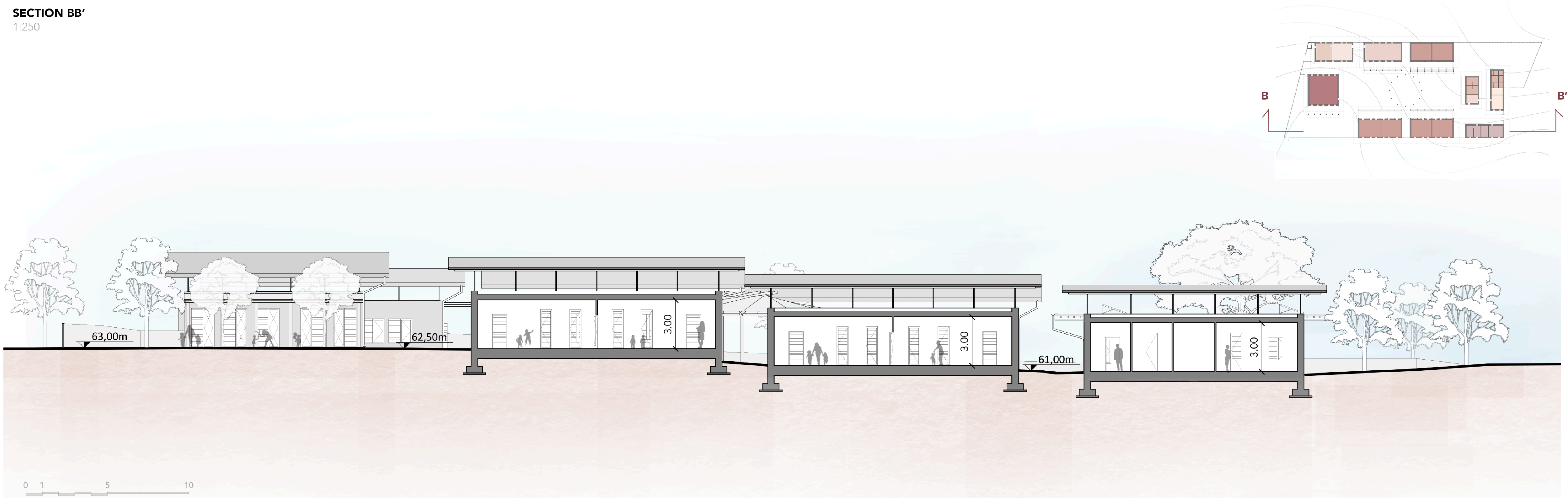
SECTION AA'
1:250





SCHOOLYARD - CHILD'S VIEW

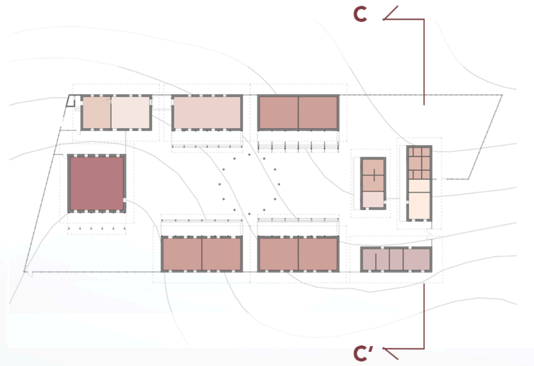
SECTION BB'
1:250

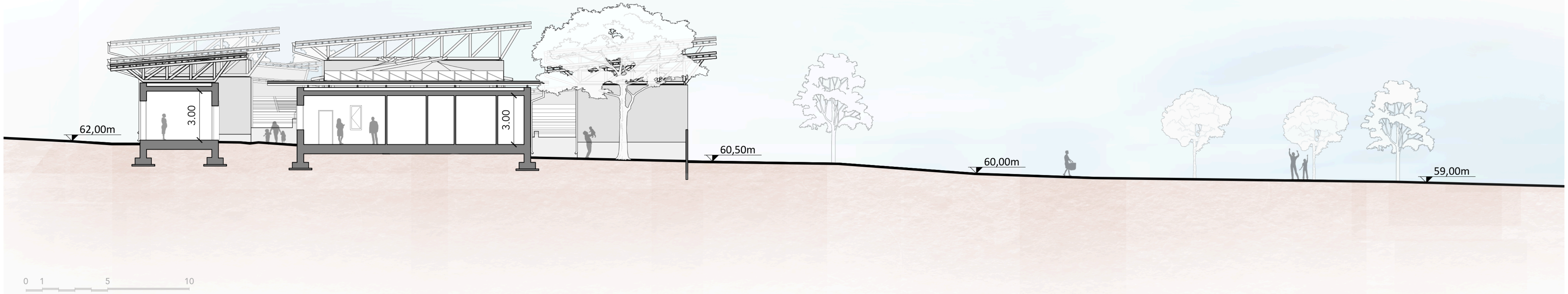


SOUTH-EAST ELEVATION
1:250

SECTION CC'

1:250





NORTH-EAST ELEVATION

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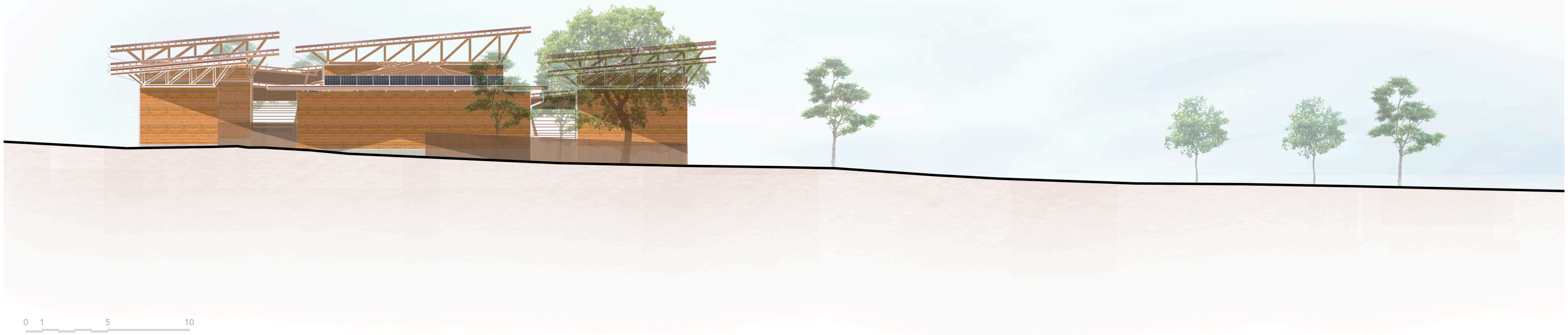

SECTION DD'
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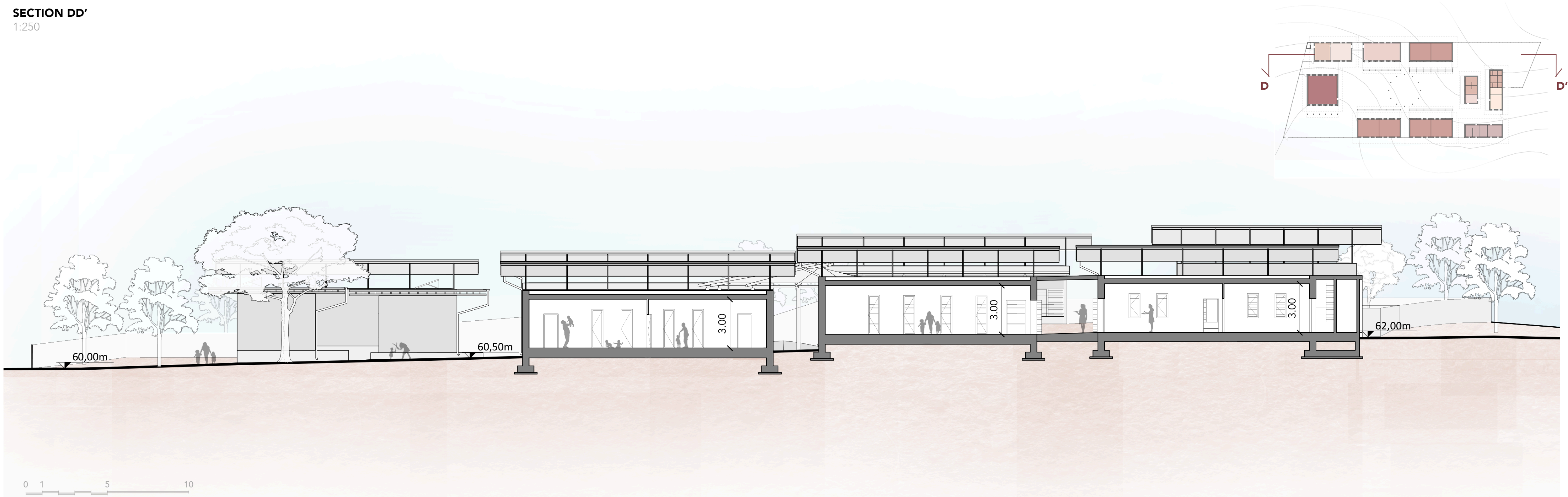

NORTH-WEST ELEVATION
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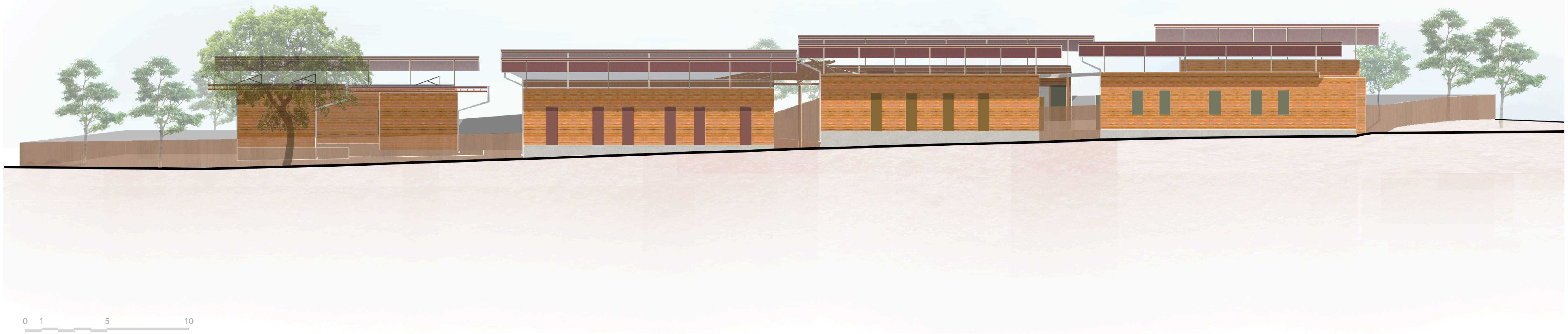

SECTION EE'
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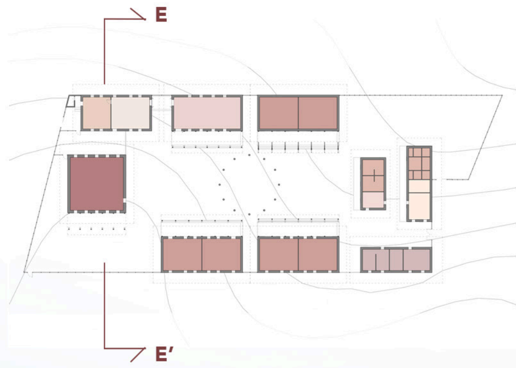
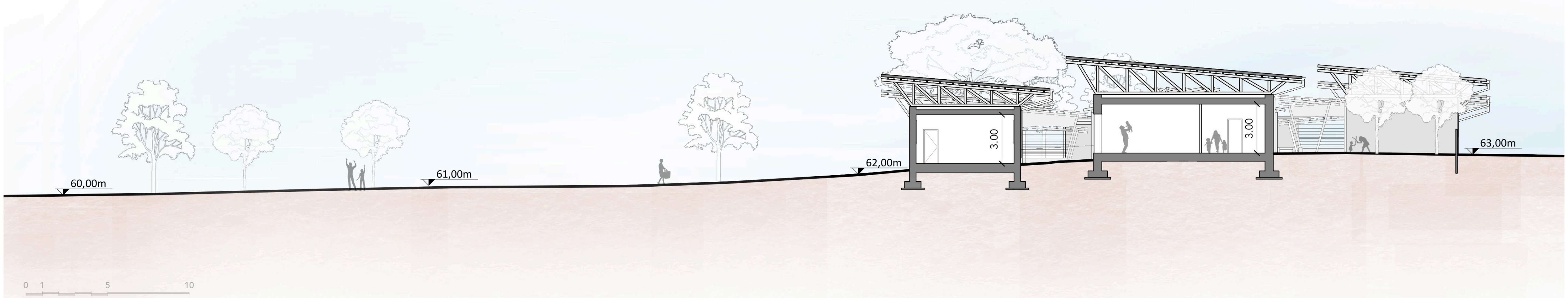
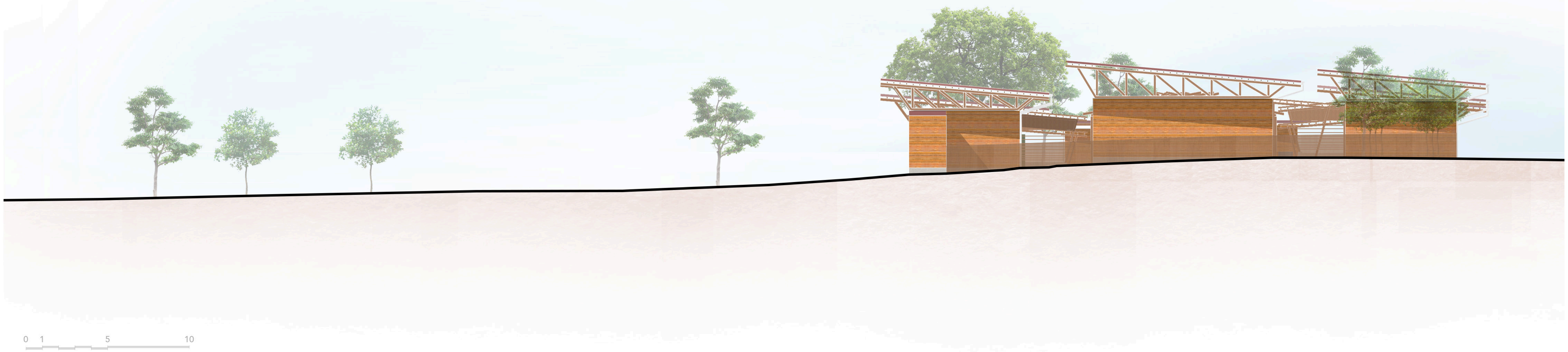




SOUTH-WEST ELEVATION
1:250



INTERACTION BETWEEN 0-1 YEARS OLD CLASSROOMS - ADULT'S VIEW

MULTIFUNCTIONAL SPACE - ADULT'S VIEW

Fig.5.1 - Cyclone Idai. Source: https://www.rainews.it

05
STRUCTURAL DESIGN

5.1. INTRODUCTION

The following chapter is focused on the structural analysis necessary to design a structure capable of safely withstanding the applied actions and loads. The main purpose is to understand the effective resistance of rammed earth structure particularly against cyclones, which represent the most critical load case.

The project is composed by single-story buildings with similar dimensions, except for the multifunctional space. Therefore, analyses have been carried out on a typical classroom and on the multifunctional space.

The load-bearing structure of each building consists of rammed earth walls and wooden beams. Bond beams in reinforced concrete guarantee the adequately distribution of the loads coming from the roof and the horizontal enclosure in the walls. Moreover, they function as connecting element, ensuring the correct behaviour of the structure against horizontal actions. To anchor the construction to the ground and ensure the transfer of loads, a shallow reinforced concrete foundation made up of foundation walls on continuous strip footings is used.

Internal forces and actions have been studied with different software: Ftool, RFEM and RWIND Simulation. The first was used to design each single element composing the structure, while the latter were used to study the overall structural behaviour of the buildings. A static model of the building under analysis was created on RFEM, defining material properties, elements cross-sections and structural constraints. The different loads involved were applied and the structural response was obtained. RWIND simulations were carried out to understand the influence the layout of the project has on the flow of the cyclones. This allowed us to better define the cyclone-induced loads affecting the structure.

5.2. RAMMED EARTH

The rammed earth technique, also known as pisè, is an ancient construction method involving a mixture of gravel, sand, silt and clay, tamped in place in layers inside temporary movable formwork. Together with other forms of unfired earthen construction, it has a long and continued history throughout many regions including North Africa, Australia, New Zealand, regions of North and South America, China and countries of Europe (France, Germany Spain and Italy). New and modern materials, embodying progress, have pushed earth and its relative historical tradition into the background leading to the nearly total disappearance of unfired earthen material in the construction field. However, in the last decades there has been a growing interest in unfired earthen material particularly for its very high sustainability.
Nevertheless, the lack of national standards is hindering the use of earthen building techniques. Only few countries have already published standards for earthen construction, whilst in many countries codes and regulations are still under development. Therefore, in these countries earthen constructions are essentially based on documents providing state-of-the-art advice and guidance.

For this project, the structural design verifications of the bearing walls have been assessed according to SADC Harmonized Standard for Rammed Earth Structures – Code of practice (SADC ZW HS 983:2014) and New Zealand Standard Codes (NZS 4297:1998). This decision has been mostly based on the availability and update of the documents.
SADC ZW HS 983:2014 gives guidance on the design, construction and test methods for rammed earth structures while NZS 4297:1998 seeks to provide for the structural and durability design of earth buildings, setting minimum criteria to comply with. More specifically, it sets out the structural design methods for earth walls and defines the requirements structural members shall meet to ensure adequate performance at limits state.

5.2.1 MATERIAL REQUIREMENTS

Mechanical properties of rammed earth structures strongly depend on the composition of the soil. Ideally, it should have a high sand/gravel content, with some silt and just enough clay to act as a binder and assist soil compaction.[1]

Since the use of appropriate soil is key to the success of rammed earth structures, the mixture as well shall be tested to prove it meets at least the minimum standards of strength and durability. Test procedures and required results the materials shall comply with are defined in NZS 4298:1998.

Tab.5.1 - *Tests for standard grade rammed earth construction*

Property	Frequency		Required result
	Prior to work start	During construction	
Compression or flexural tensile strength	1 sample of 5 individuals every 50 m^3 of wall sample of 5 individuals every 50 m^3 of wall		f_e>1,3 MPa f_{et}>0,25
Wet/dry appraisal	3 sample	not required	pass
Durability	1 spray or 2 drip	not required	as required by NZS 4297 or NZS 4298
Shrinkage	1 sample for each mixes tried 2 samples if one mix only being tried	not required	< 0,05% for rammed earth
On-site moisture handful drop test	1 sample for each test or batch		appendix G

Source: NZS 4297:1998

Since soil specimens are necessary, it has not been possible to perform these verifications. However, according to the information and data collected, the soil of the project site well suited to

1 Maniatidis,V., Walker, P. A review of rammed earth. *University of Bath*. Available at: https://people.bath.ac.uk

rammed earth building technique. Indeed, the topsoil (40-80cm depth) consists of 87% sand, 9% clay and 4% silt.[2]

5.2.2 STRUCTURAL DESIGN OF MEMBERS

For the structural design of members, NZS 4297:1998, table 4.1, gives the design strengths to be used in calculations. The standard specifies that their values have been reduced with respect to the laboratory tests results to account for aspect ratio, characteristic strength, and mortar effects.

Tab.5.2 - *Design strength to be used for standard grade earth wall*

Strength	Value MPa
Compressive strength (flexural, direct compression or bearing)	f_e = 0,5
Maximum total nominal shear stress	f_n = 0,09
Shear strength of earth for wind loading and for seismic load with elastic response	f_{es} = 0,08
Shear strength of earth for limited ductile (μ =2,0) seismic loading	f_{es} = 0,00
Shear strength of steel reinforced earth	f_{es} = 0,35
Tensile/flexural bond strength	f_{eb} = 0,02
Flexural tensile strength	f_{et} = 0,1

Source: NZS 4297:1998

Ultimate limit state

Section 5.3 of NZS 4297:1998 defines the general requirements for ultimate limit state analysis. The design strength of a member or cross section in terms of load, moment, shear, or stress shall be taken as the nominal strength, S_n, multiplied by a capacity reduction factor Φ. The design strength of a member or cross section shall be equal to or greater than the applied action, S*, resulting from the design loads:

$$S^* \leq \Phi S_n$$

where S is replaced by the actions of moment, axial force, shear or torsion as appropriate.

2 Brandt,M., Baumler, R., Samini, C. Agricultural suitability of dune system and Limpopo Basin soils near Xai-Xai, Mozambique. *South African Journal of Plant and Soil*. Available at: https://www.tandfonline.com

Tab.5.3 - *Capacity reduction factors*

Strength	Φ
Axial compression and bearing	0,6
Flexure	0,8
Shear	0,7
Metal connections embedded in earth	0,7
Flexure determined using Appendix B	0,6

Source: NZS 4297:1998

Serviceability limit state

Section 5.4 of NZS 4297:1998 states that the minimum thickness in the horizontal direction, where the walls are not supporting or attached to partitions or other construction likely to be damaged by large deflection, shall not be less than the following:

Tab.5.4 - *Minimum thickness*

Supports	th_{min}
Simply supported	h/18
One end continuous	h/21
Both end continuous	h/22
Cantilever	h/8

Source: NZS 4297:1998

5.3. WOOD

As in many African countries, timber structural system for roofs are very common in Mozambique. Wood is widespread throughout the Country, unlike steel which is an imported material involving a considerable economic expense. Mozambique has indeed vast forestry resources which include native tropical wood species whose high commercial values are internationally recognised. Although Mozambique has about 120 native usable wood species, only about 12–18 of these are exploited and used[3]. This is due to their high commercial value in selective logging system and to the lack of knowledge on the properties of the remaining species.
Investigating Mozambican laws and regulations concerning forest management, wood harvesting, processing and trade is out of the scope of this work but it is clear that research on lesser-known species should be promoted not only to increase the resources in the country but also to reduce the currently high pressure on the better-known ones.

To verify the resistance of the structure with respect to the acting loads, it is essential to know the mechanical properties of the material to be used. This has led us to exclude most of the available resources. Furthermore, due to the unavailability of Mozambican standards on timber construction, accurate data to compared with those of European standards allowed us to assess the structural verifications, with the necessary precautions, according to EN 1995.1.1 (2004) Eurocode 5: design of timber structures.

5.3.1 MATERIAL PROPERTIES

From the information collected it emerged that three main types of wood species are used for heavy construction in Mozambique: Afzelia Quanzensis commercially known as Chanfuta, Androstachys Johnsonii commercially known as Mecrusse, and Erythrophloeum

3 Ali, A.C., Uetimate Jr, E., Lhate, I.A., Terziev, N. Anatomical characteristics, properties and use of traditionally used and lesser-known wood species from Mozambique: a literature review. *Swedish University of Agricultural Sciences, department of Forest Products*. Available at: https://www.scopus.com

Suaveolens commercially known as Missanda. In terms of physical and mechanical properties, they are all classified as very hard, dense and with high strength[4].

Tab.5.5 - *Physical and mechanical properties of Mozambican wood species*

Wood species	Density	Bending strength	Impact bending strength	Modulus of elasticity
	kg/m^3	N/mm^2	N/mm^2	N/mm^2
Chanfuta	817	125	79,2	13 100
Mecrusse	720	129	66,0	-
Missanda	720	162	97,2	15 444

Source: Anatomical characteristics, properties and use of traditionally used and lesser-known wood species from Mozambique: a literature review

Unfortunately, most of the information available about characteristics and properties of wood species used in Mozambique is rather old and do not take account of the possible climate changing effects which may have affect vegetation characteristics.
However, it is evident that Mozambican above-mentioned wood species fall within the hardwood species strength classes outlined in UNI EN 338:2009.

Tab.5.6 - *Strength classes: characteristic values*

Properties		Hardwood species							
		D18	D24	D30	D35	D40	D50	D60	D70
Strength properties (in N/mm^2)									
Bending	$f_{m,k}$	18	24	30	35	40	50	60	70
Tension parallel	$f_{t,0,k}$	11	14	18	21	24	30	36	42
Tension perpendicular	$f_{t,90,k}$	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6
Compression parallel	$f_{c,0,k}$	18	21	23	25	26	29	32	34
Compression Perpendicular	$f_{c,90,k}$	7,5	7,8	8,0	8,1	8,3	9,3	10,5	13,5
Shear	$f_{v,k}$	3,4	4,0	4,0	4,0	4,0	4,0	4,5	5,0

Source: EN 338:2009

4 Ali, A.C., Uetimate Jr, E., Lhate, I.A., Terziev, N. Anatomical characteristics, properties and use of traditionally used and lesser-known wood species from Mozambique: a literature review. *Swedish University of Agricultural Sciences, department of Forest Products.* Available at: https://www.scopus.com

Tab.5.6 - *Strength classes: characteristic values*

Stiffness properties (in kN/mm^2)									
Mean modulus of elasticity parallel	$E_{0,mean}$	9,5	10	11	12	13	14	17	20
5% modulus of elasticity parallel	$E_{0,05}$	8	8,5	9,2	10,1	10,9	11,8	14,3	16,8
Mean modulus of elasticity perpendicular	$E_{90,mean}$	0,63	0,67	0,73	0,80	0,86	0,93	1,13	1,33
Mean shear modulus	G_{mean}	0,59	0,62	0,69	0,75	0,81	0,88	1,06	1,25
Density (in kg/m^3)									
Density	ρ	475	485	530	540	550	620	700	900
Mean density	ρ_{mean}	570	580	640	650	660	750	840	1080

Source: EN 338:2009

Comparing the values and considering the uncertainties related to the Mozambican wood species data, the structural verifications have been assessed assuming a material belonging to D50 strength class and therefore referring to the strength and stiffness properties outlined in UNI EN 338:2009.

5.3.2 BASIC VARIABLES

To determine the resistances of structural elements, it is necessary to take into account some peculiarities of the wooden material such as the dependency of its strength on loads duration and humidity. Hence, actions shall be assigned to one of the load duration classes given in table 2.1 of EN 1995.1.1 (2005).

The load duration classes are characterized by the effect of a constant load acting for a certain period of time in the life of the structure. For a variable action the appropriate class shall be determined on the basis of an estimate of the typical variation of the load with time.

Tab.5.7 - *Load duration classes*

Class	Load duration	Example
Permanent	more than 10 years	self-weight
Long-term	6 months - 10 years	storage
Medium-term	1 week - 6 months	imposed floor load
Short-term	less than 1 week	snow, wind
Instantaneous	-	wind, accidental load

Source: EN 1995.1.1 (2005)

Generally, for calculations purposes the following conditions can be assumed:

- self-weight and non-removable loads during the service life of the structure belong to the permanent load duration class
- permanent loads susceptible to change during the structure's service life belong to the long-term load duration class
- variable loads belong to the medium-term load duration class
- climatic loads are usually assigned to the short-term load duration class, but this may vary according to National annex
- accidental loads belong to the instantaneous load duration class.

Dealing with cyclones, we refer to the short-term load duration class for wind loads.

Finally, to accurately define material strength values, structures shall be assigned to one of the service classes outlined in EN 1995.1.1 (2005). This enables also to correctly calculate deformations under defined environmental conditions.

As already mentioned, Xai-Xai is characterised by a sub-humid tropical climate which entails a muggy season from October to May. The very high humidity rate reached throughout most of the year lead us to assign the structure to service class 3. According to EN 1995.1.1 (2005), service class 3 is characterised by climatic conditions leading to higher moisture contents than in service class 2 which is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85% for a few weeks per year.

5.3.3 STRUCTURAL DESIGN OF MEMBERS

In order to satisfy structural safety requirements, the verifications at ultimate limit states and serviceability limit states shall be carried out. In the following paragraphs the main coefficients and parameters involved in both the verifications are defined.

Ultimate limit states

The design value X_d of a strength property to be used in ultimate limit states verifications shall be calculated as:

$$X_d = k_{mod} \cdot \frac{X_k}{\gamma_M}$$

where:

- X_k is the characteristic value of a strength property
- γ_M is the partial factor for a material property. For solid timber, the recommended value is 1,3
- k_{mod} is the modification factor taking into account the effect of the duration of load and moisture content for ultimate limit states analysis. For solid timber assigned to service class 3, the recommended values are the following:
-

Tab.5.8 - *Values of k_{mod} for solid timber, service class 3*

Load duration class				
Permanent action	Long-term action	Medium-term action	Short-term action	Instantaneous action
0,5	0,55	0,65	0,7	0,90

Source: EN 1995.1.1 (2005)

The design member stiffness property E_d and G_d shall be calculated as follow:

$$E_d = \frac{E_{mean}}{\gamma_M}$$

$$G_d = \frac{G_{mean}}{\gamma_M}$$

where:

- E_{mean} is the mean value of the modulus of elasticity
- G_{mean} is the mean value of shear modulus

Serviceability limit states

The deformation of a structure which results from the effects of actions and from moisture shall remain within appropriate limits, having regard to the possibility of damage to surfacing materials, ceilings, floors, partitions and finishes, and to the functional needs as well as any appearance requirements.

Wood's rheological characteristics play an important role in the overall deformation behaviour of an element: it shows an initial elastic behaviour that soon, under constant loads, evolves into a viscos-elastic one. This leads the deformation to increase over time. Hence, both the instantaneous and delayed deformations have to be considered to evaluate the final deformation of an element. It is important to specify that the delayed deformation can be calculated as a function of the instantaneous one.

According to EN 1995.1.1 (2004), the final deformation u_{fin} of a structure may be taken as:

$$u_{fin} = u_{fin,G} + u_{fin,Q_1} + \sum u_{fin,Q_i}$$

where:

- $u_{fin,G}$, $u_{fin,Q1}$, $u_{fin,Qi}$ are the final deformations for permanent actions, leading variable action and accompanying variable action respectively, evaluated through the following expressions:

$$u_{fin,G} = u_{inst,G}(1 + k_{\text{def}})$$

$$u_{fin,Q_1} = u_{inst,Q_1}(1 + \psi_{2,1} k_{\text{def}})$$

$$u_{fin,Q_{i1}} = u_{inst,Q_i}(\psi_{0,i} + \psi_{2,i} k_{\text{def}})$$

- $u_{inst,G}$, $u_{inst,Q1}$, $u_{inst,Qi}$ are the instantaneous deformations for permanent actions, leading variable action and accompanying variable action respectively
- $\Psi_{2,1}$ and $\Psi_{2,i}$ are the factor for the quasi-permanent value of variable actions;
- $\Psi_{0,i}$ is the factor for the combination value of variable actions
- k_{def} is the modification factor taking into account the effect of the duration of load and moisture content. For solid timber assigned to service class 3, the recommended value is 2,00.

5.4. ACTIONS ON STRUCTURE

Dealing with structural analysis procedures from standards provided by different countries, each referring to its own methods of determining actions and loads, and their combinations, we considered it appropriate to evaluate the latter in a uniform manner for all structural elements and, therefore, in compliance with one of the standards. Comparing the regulations, it emerged that the load increase and structural safety coefficients of NZS 4203:1992 Code of practice for general structural design and design loadings for buildings are albeit slightly less restrictive and strict than those of EN 1991.1.1 (2002).

Considering that there are still some uncertainties about the structural behaviour of the earthen material, especially in terms of resistance to cyclones, we believe it was important to allow for a quiet fair margin of structural safety. Hence the decision to evaluate actions and loads, and their combinations according to European standards.

Actions and loads therefore have been evaluated according to EN 1991.1 (2002), Eurocode 1: actions on structures.

For each element analysed, the total self-weight of structural and non-structural members, the imposed loads arising from occupancy and the variable actions induced by wind have been taken into account. The verifications at ultimate limit state and serviceability limit state reported in this chapter have been performed considering the most critical load combination possible.

5.4.1. PERMANENT LOADS

The pre-school is organized as a series of independent single-story buildings, therefore permanent loads are due to their roofs and horizontal enclosures, hanged equipment (fans) and self-weight of structural elements. The latter are specified in the following paragraphs since they have been evaluated and included in the calculations as linear loads. The others are reported in the tables below.

Tab.5.9 - *Roof (canopy)*

Layer	thickness	specific weight	weight
	m	kN/m³	kN/m²
sandwich panel	0,10		0,13
joist 3*(0,06m*0,12m)/1m	0,022	6,08	0,13
			0,26

Tab.5.10 - *Horizontal enclosure*

Layer	thickness	specific weight	weight
	m	kN/m³	kN/m²
osb panel	0,02	5,88	0,12
straw insulation	0,10	3,34	0,33
osb panel	0,02	5,88	0,12
			0,57

5.4.2. VARIABLE LOADS

Variable (or live) loads include actions varying in time such as imposed loads on building floors, beams and roofs, and wind actions. EN 1991.1.1., section 6, provides characteristic values for imposed loads according to the category of intended use, while wind loads have to be calculated following the procedure explained in EN 1991.1.4.

Imposed loads

Concerning the imposed loads on floors, the whole project belongs to category C of table 6.2 of EN 1991.1.1., section 6.3. More specifically, except the multifunctional space which falls within category C4, all the buildings can be associated to category C1.
Referring to table 6.2, the values of imposed loads on floors are the following:

Tab.5.11 - *Imposed loads on floors*

Categories of loaded area	q_k	Q_k
	kN/m²	kN
C1	2,0 to 3,0	3,0 to 4,0
C4	4,5 to 5,0	3,5 to 7,0

Source: EN 1991.1.1 (2002)

According to the Standard, the imposed loads on roofs depend on their accessibility. Since roofs are not accessible except for normal maintenance and repair, the characteristic values to be used are the ones of category H of table 6.10:

Tab.5.12 - *Imposed loads on roofs*

Categories of roof	q_k	Q_k
	kN/m²	kN
H	0,4	1,0

Source: EN 1991.1.1 (2002)

Wind loads

EN 1991.1.4 gives guidance on the determination of the characteristic values of wind actions determined from the basic values of wind velocity and pressure, both composed of a mean and fluctuating component.

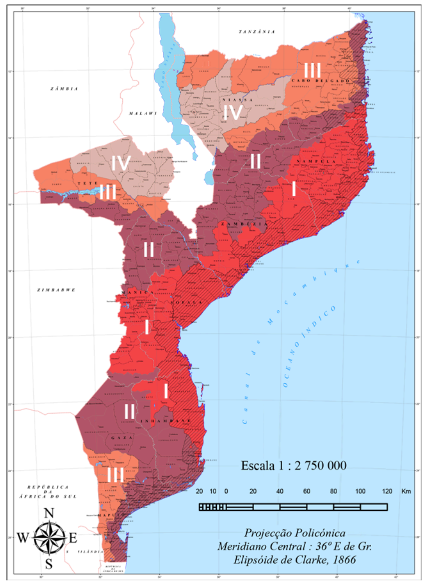
The mean wind velocity v_m depends on the terrain roughness and orography and on the basic wind velocity v_b which depends on the wind climate. According to the standard, the basic wind velocity shall be calculated from expression 4.1:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}$$

where:

- v_b is the basic wind velocity, defined as a function of wind direction and time of year at 10m above ground of terrain category II
- $v_{b,0}$ is the fundamental value of the basic wind velocity
- c_{dir} is the directional factor
- c_{season} is the season factor

As already seen, Mozambique is vulnerable to natural disasters especially to cyclones which hit the country at least one a year. Therefore, the basic wind velocity used for the calculations is the one recorded in the Gaza Province during those events.



	Valor Normal	Pressão Dinâmica Equivalente	Valor Extremo	Pressão Dinâmica Equivalente
Zona I	55 m/s (198 km/h)	185 dan/m²	57 m/s (207 km/h)	199 dan/m²
Zona II	36 m/s (129 km/h)	79 dan/m²	43 m/s (155 km/h)	113 dan/m²
Zona III	26 m/s (95 km/h)	41 dan/m²	31 m/s (112 km/h)	59 dan/m²
Zona IV	22 m/s (77 km/h)	30 dan/m²	24 m/s (86 km/h)	35 dan/m²
	Região Altamente Propensa aos Ciclones			

Fig.5.2 - Mozambique's map of wind.
Suorce: Repùblica de Mocambique. Ministèrio das obras pùblicas, habitacao e recursos hidricos.

According to the map of wind of Mozambique, the fundamental value of the basic wind velocity in Zone II is

$$v_{b,0} = 36\ {}^{m}/_{s}$$

which with c_{dir}=1 and c_{season}=1 (values recommended by the Standard) leads to:

$$v_b = v_{b,0} = 36\ {}^{m}/_{s}$$

The mean wind velocity $v_m(z)$ at a height z above the terrain should be determined using expression 4.3:

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b$$

where:
- $c_r(z)$ is the roughness factor
- $c_o(z)$ is the orography factor taken as 1,0

To determine the roughness factor $c_r(z)$ that accounts for the variability of the mean wind velocity at the site of the structure, the recommended procedure given by expression 4.4 has been applied as follow.
According to table 4.1, the project site belongs to terrain category II that is area with low vegetation such as grass and isolated obstacles. This leads to:
- z_0 = 0,05 m (roughness length)
- z_{min}= 2,00 m (minimum height)

Moreover, the Standard specifies that
- z_{max}= 200 m

Since the height of the buildings is not constant, due to the inclination of the roof of 5° to North-West and to the variable ground course:
- a height of 4,40 m has been taken for 0°-wind calculations
- a height of 5,75 m for 180°-wind calculations

In both cases, it can be assumed

$$z_{min} \leq z \leq z_{max} \text{ for which } c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right)$$

where:
- k_r is the terrain factor depending on the roughness length z_0 calculated using

$$k_r = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07}$$

Defined the mean wind velocity $v_m(z)$, the turbulence intensity $I_v(z)$ necessary to evaluate the peak velocity pressure $q_p(z)$ has been determined using expression 4.7 according to which:

$$I_v(z) = \frac{\sigma_v}{v_m(z)}$$

where:
- $\sigma_v = k_r \cdot v_b \cdot k_I$ is the standard deviation of the turbulence

Finally, the peak velocity pressure $q_p(z)$ at height z, which includes mean and short-term velocity fluctuations, has been determined according to expression 4.8:

$$q_p(z) = [1 + (7 \cdot I_v(z))] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z)$$

where:
- ρ is the air density $(1{,}25\ {}^{kg}/_{m^3})$

Tab.5.13 - Summary of the parameters calculated

wind direction	$v_{b,0}$ m/s	v_b m/s	v_m m/s	$I_v(z)$	$q_p(z)$ N/m²
θ=0°	36,00	36,00	30,62	0,22	1502,63
θ=180°	36,00	36,00	32,46	0,21	1629,57

According to EN 1991.1.4, wind actions on structures shall be determined considering both external and internal wind pressures which should be obtained respectively from expression 5.1 and 5.2:

$$w_e = q_p(z_e) \cdot c_{pe}$$

$$w_i = q_p(z_i) \cdot c_{pi}$$

where:

- $q_p(z_e)$ and $q_p(z_i)$ are the peak velocity pressures
- z_e and z_i are the reference height respectively for the external and internal pressure
- c_{pe} and c_{pi} are the pressure coefficients respectively for the external and internal pressure

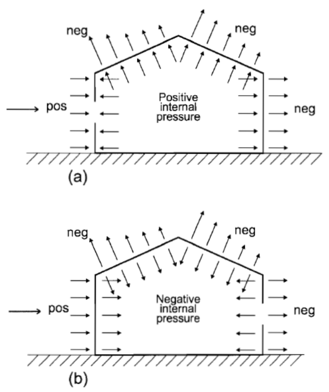


Fig.5.3 - Wind pressure representation. Source: EN 1991.1.4

The net pressure on a wall, roof or element is the difference between the pressure on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative. The external pressure coefficient c_{pe} for buildings and parts of buildings depend on the size of the loaded area A which is the area of the structure that produces the wind action in the section to be calculated. The analyses have been carried out referring to the overall coefficients $c_{pe,10}$ which is the recommended coefficient to be used when designing the overall load bearing structure of a building.

The pressures on the different buildings' surfaces have been determined according to the procedure explained in the Eurocode. To determine the external pressure coefficients for walls, interpolation has been necessary for walls D and E.

Internal and external pressure shall be considered to act at the same time. The worst combination of pressures shall be considered for every combination of possible openings and other leakage paths. The internal pressure c_{pi} depends on the size and distribution of the openings in the building envelope.

According to EN 1991.1.4, section 7.2.9, if an external opening is considered to be closed in the ultimate limit state during severe windstorms, the condition with door or window open should be considered as an accidental design situation in accordance with EN 1990.

Since none of the buildings have a dominant face (i.e. a face where the area of openings is at least twice the area of openings and leakages in the remaining faces), the internal pressure coefficient c_{pi} should be determined as a function of the ratio of the height and depth of the building, h/d, and the opening ratio μ for each wind direction θ.

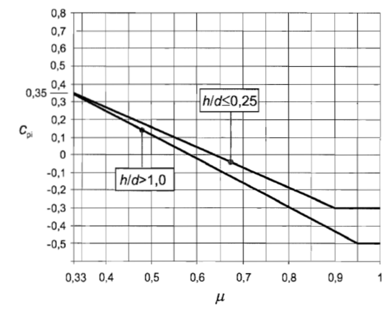
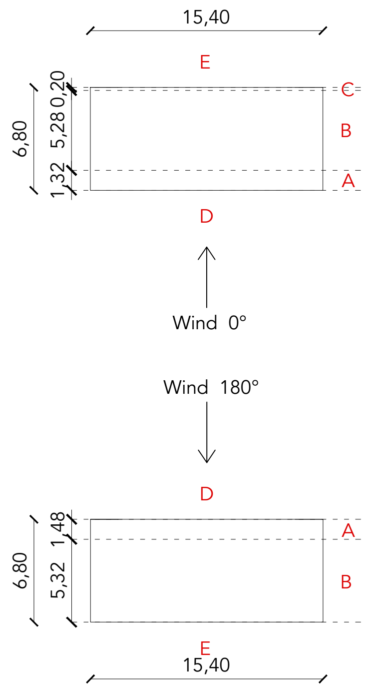


Fig.5.4 -internal pressure coefficients for uniformly distributed openings. Source: EN 1991.1.4.

In the following tables, the values of external and internal pressures, and the corresponding wind actions are presented for a typical classroom and the multifunctional spaces.



Tab.5.14 - Typical classroom

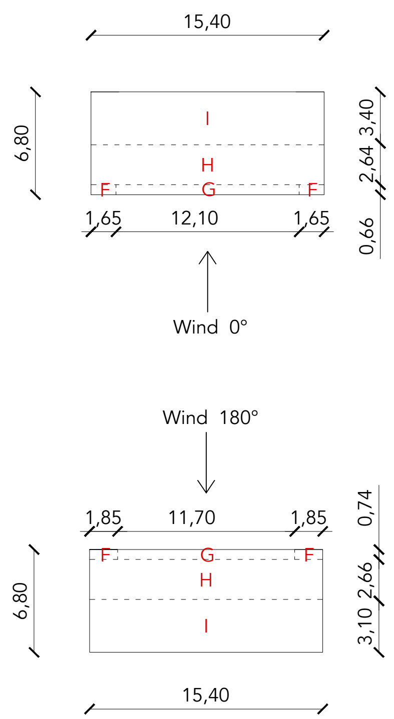
External surfaces: WALL

wind: θ=0°

general data			$C_{pe,10}$	w_e
b	15,40 m			N/m²
z_e=h	3,30 m	**A**	-1,2	-1803,16
d	6,80 m	**B**	-0,8	-1202,11
e=2h	6,60 m	**C**	-0,5	-751,32
h/d	0,49	**D**	0,75	1126,98
		E	-0,4	-601,05

wind: θ=180°

general data			$C_{pe,10}$	w_e
b	15,40m			N/m²
z_e=h	3,70m	**A**	-1,2	-1995,48
d	6,80 m	**B**	-0,8	-1303,66
e=2h	7,40 m	**D**	0,75	1222,18
h/d	0,54	**E**	-0,4	-651,83



External surfaces: FLAT ROOF

wind: θ=0°

	$C_{pe,10}$	w_e
		N/m²
F	-1,8	-2704,74
G	-1,2	-1803,16
H	-0,7	-1051,84
I	0,2	300,53
	-0,2	-300,53

wind: θ=180°

	$C_{pe,10}$	w_e
		N/m²
F	-1,8	-2933,22
G	-1,2	-1955,48
H	-0,7	-1140,70
I	0,2	325,91
	-0,2	-325,91

Tab.5.14 - Typical classroom

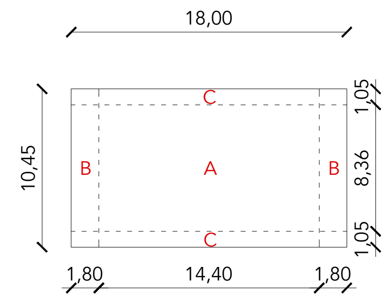
External surfaces: CANOPY

wind: θ=0°

general data			$C_{pe,NET}$	w_e
b	18,20 m			N/m²
d	10,45 m	**A**	-1,1	-1652,90
		B	-1,7	-2554,48
		C	-1,8	-2704,74

wind: θ=180°

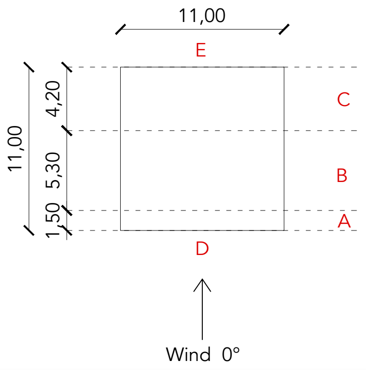
general data			$C_{pe,NET}$	w_e
b	18,20 m			N/m²
d	10,45 m	**A**	-1,1	-1792,53
		B	-1,7	-2770,27
		C	-1,8	-2933,22

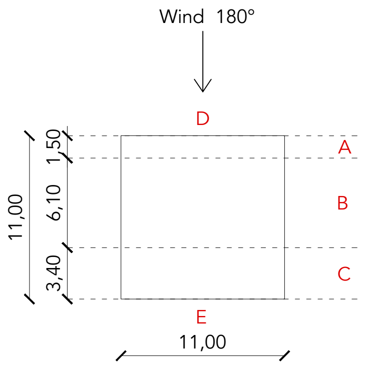


*valid for both wind directions (0° and 180°)

Internal surfaces

general data			μ	$C_{pi,10}$	w_i
A_{tot}	46,20 m²				N/m²
$A_{wall,I}$	38,22 m²	**θ=0°**	0,49	0,16	240,42
$A_{wall,II}$	39,90 m²	**θ=180°**	0,60	0,05	81,48
$A_{openings,I}$	7,98 m²				
$A_{openings,II}$	6,30 m²				
$A_{vent.gaps}$	1,50 m²				
h/d	0,44				





Tab.5.15 - Multifunctional space

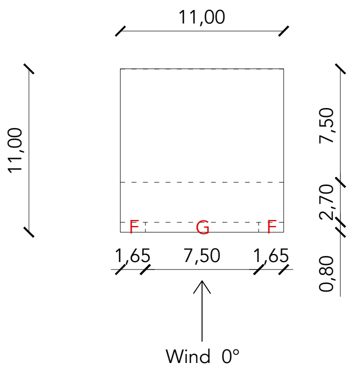
External surfaces: WALL

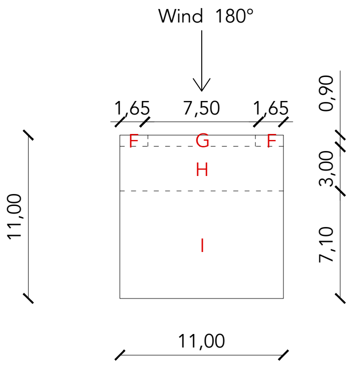
wind: θ=0°

general data			$C_{pe,10}$	w_e
b	11,00 m			N/m²
z_e=h	3,30 m	**A**	-1,2	-1803,16
d	11,00 m	**B**	-0,8	-1202,11
e=2h	6,60 m	**C**	-0,5	-751,32
h/d	0,31	**D**	0,75	1126,98
		E	-0,4	-601,05

wind: θ=180°

general data			$C_{pe,10}$	w_e
b	11,00 m			N/m²
z_e=h	3,70 m	**A**	-1,2	-1995,48
d	11,00 m	**B**	-0,8	-1303,66
e=2h	7,40 m	**C**	-0,5	-711,34
h/d	0,34	**D**	0,75	1222,18
		E	-0,4	-651,83





External surfaces: FLAT ROOF

wind: θ=0°

	$C_{pe,10}$	w_e
		N/m²
F	-1,8	-2704,74
G	-1,2	-1803,16
H	-0,7	-1051,84
I	0,2	300,53
	-0,2	-300,53

wind: θ=180°

	$C_{pe,10}$	w_e
		N/m²
F	-1,8	-2933,22
G	-1,2	-1955,48
H	-0,7	-1140,70
I	0,2	325,91
	-0,2	-325,91

Tab.5.15 - Multifunctional space

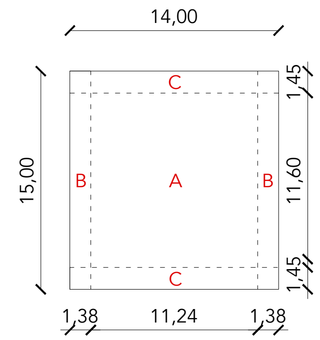
External surfaces: CANOPY

wind: θ=0°

general data			$C_{pe,NET}$	w_e
b	14,00 m			N/m²
d	14,50 m	**A**	-1,1	-1652,90
		B	-1,7	-2554,48
		C	-1,8	-2704,74

wind: θ=180°

general data			$C_{pe,NET}$	w_e
b	14,00 m			N/m²
d	14,50 m	**A**	-1,1	-1792,53
		B	-1,7	-2770,27
		C	-1,8	-2933,22



*valid for both wind directions (0° and 180°)

Internal surfaces

general data			μ	$C_{pi,10}$	w_i
A_{tot}	33,00 m²				N/m²
$A_{wall,I}$	24,60 m²	**θ=0°**	0,50	0,05	75,13
$A_{wall,II}$	24,60 m²	**θ=180°**	0,50	0,05	81,48
$A_{openings,I}$	8,40 m²				
$A_{openings,II}$	8,40 m²				
$A_{vent.gaps}$	2,00 m²				
h/d	0,27				

5.4.3. LOAD COMBINATION METHOD

The calculations have been performed using the so-called partial factor method which implies the use of partial coefficients for loads and actions, and their combinations.

Ultimate limit state

The following expression is the general format applied for load combination at ultimate limit state:

$$F_d = \gamma_g G_k + \gamma_q \left[Q_{k,1} + \sum_{i>1} (\psi_{0,i} Q_{k,i}) \right]$$

where:

- G_k is the characteristic value of permanent action
- $Q_{k,1}$ is the characteristic value of the leading variable action in each combination
- $Q_{k,i}$ is the characteristic value of accompanying variable action in each combination
- γ_g is the safety coefficient for permanent action. The recommended value is 1,35
- γ_q is the safety coefficient for variable action. The recommended value is 1,50.
- $\Psi_{0,i}$ is the combination factor at ultimate limit state to be determined through static considerations. The recommended values applied in the calculations are listed in the following table.

Tab.5.16 - Ψ factor values for building

Action	Ψ_0	Ψ_1	Ψ_2
Imposed loads:			
- Category C: congregation areas	0,7	0,7	0,6
-Category D: roofs	0	0	0
Wind loads on building	0,6	0,2	0

Source: EN 1990 (2002)

Serviceability limit state

Verifications shall take into account the deformations arising during the use of the structure. For this limit state condition, the combinations of loads and actions are usually expressed as follow:

- characteristic combination (used for irreversible limit states)

$$F_d = G_k + Q_{k,1} + \sum_{i>1} (\psi_{0,i} Q_{k,i})$$

- frequent combination (used for reversible limit states)

$$F_d = G_k + \psi_{1,1} Q_{k,1} + \sum_{i>1} (\psi_{2,i} Q_{k,i})$$

- quasi-permanent combination (used for long-term effects and the appearance of the structure

$$F_d = G_k + \sum_{i>1}^{n} (\psi_{2,i} Q_{k,i})$$

5.5. RWIND SIMULATIONS

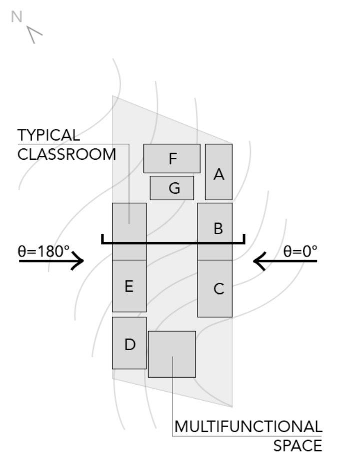
The verifications presented in the following paragraphs have been supported by a more accurate study on the interaction between buildings and cyclones, which were carried out with RFEM and RWIND Simulation. For both the buildings analysed, a static model was created on the former and then exported on the latter.

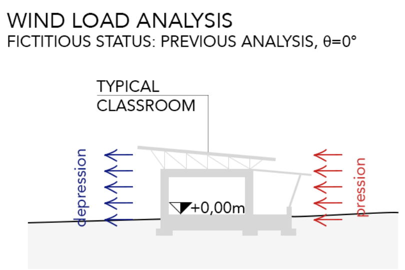
5.5.1. INTERFERENCE PHENOMENA

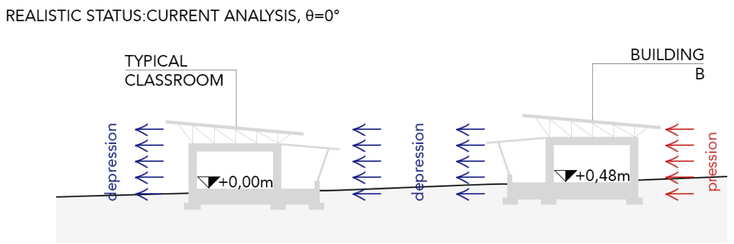
As already seen, the project is composed by single-story buildings whose arrangement has been based on the guidelines developed by MINEDH for the "Projecto Escolas Seguras" (outlined in chapter 1, paragraph 1.1.6). Avoiding U and L-shaped structural layout due to their weakness to torsional forces and designing the buildings with a length-to-width ratio not greater than 1/3, the resulting layout is characterised by a succession of volumes and voids affecting the wind flow and thus its velocity. Indeed, the presence of contiguous buildings determines interference phenomena which can reduce or amplify the effects of wind. RWIND simulations enable us to better understand how these phenomena act within the site of the project.

In previous calculation, the building under study was considered as an isolated construction: in both the wind directions analysed, the air flow hit the building causing pressure on the windward side and consequently depression on the leeward side.

However, considering the context, RWIND simulations show that in case of wind at 0° on both the facades of the "typical classroom" depression arises. This situation occurs because actually the air flow directly hit building B, the building facing the typical classroom, which is placed at a slightly higher altitude.







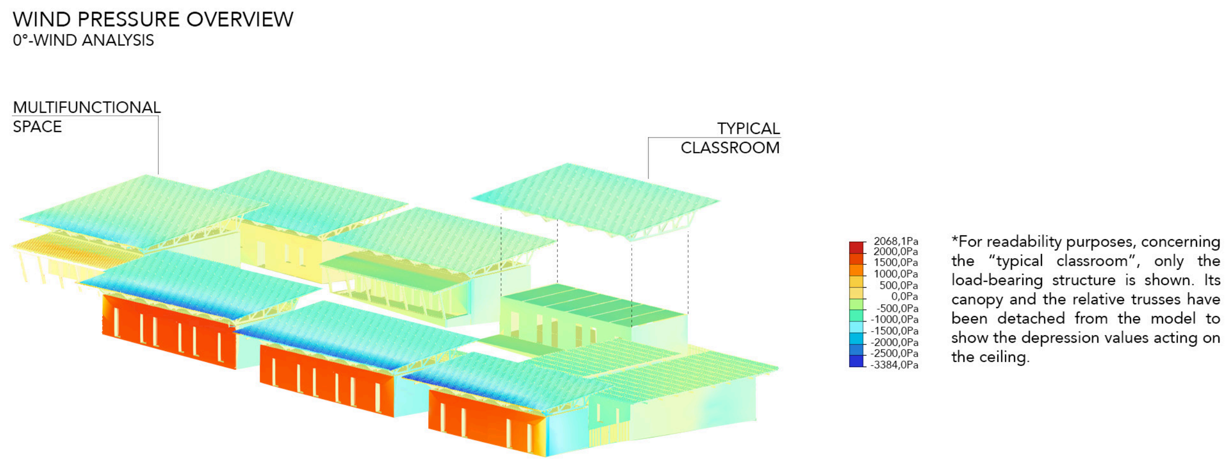


Fig.5.5 - 0° wind pressure analysis carried out with RWIND Simulation

The wind generates depression vortexes on the leeward side of building B which do not run out aided by the land slope; rather they hit the typical classroom façade, causing depression as shown in the image above. The wind pressure acting on this latter façade is equal to -315,4 Pa, rather than 1126,98Pa as previously evaluated, resulting in a horizontal bending moment definitely lower than the one related to the second value.

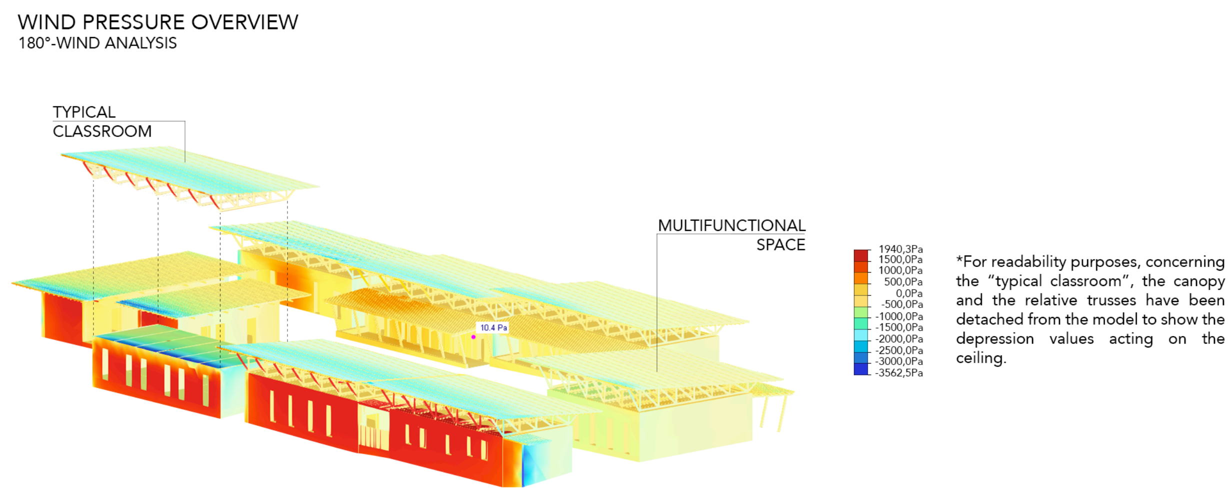


Fig.5.6 - 180° wind pressure analysis carried out with RWIND Simulation

On the other end, the case of wind at 180° is different: the air flow directly hit the typical classroom causing pressure on the windward side and consequently depression on the leeward side. The

depression vortexes run out and do not affect the facing building, aided by the upward land.
The above graphical representation might be misleading due to the fact that yellow shades correspond to a range of values going from positive (pressure) to negative (depression). Therefore, more accurate analyses have been carried out on the facade of building B. The lowest value of pressure acting on this facade correspond to 10,4 Pa, as shown in the image.
Moreover, hindering the air flow, the typical classroom slows down the wind velocity. Therefore, the pressure acting on the windward side of building B is actually less severe than the one acting on the typical classroom as shown in the image above.

Furthermore, it is worth pointing out that an argument similar to the one previously done for the typical classroom applies to the multifunctional space. Indeed, the 180°-wind do not directly hit this space, but rather it impacts on building D. The consequent depression vortexes arising on the leeward side of this latter construction do not have enough space to run out and therefore depression occurs on both the façades of the multifunctional space. The pressure acting on the multifunctional space is about -750 Pa, rather than 1222,18 Pa as previously evaluated. It follows that the acting horizontal bending moment will be lower than the one resulting from previous calculations.

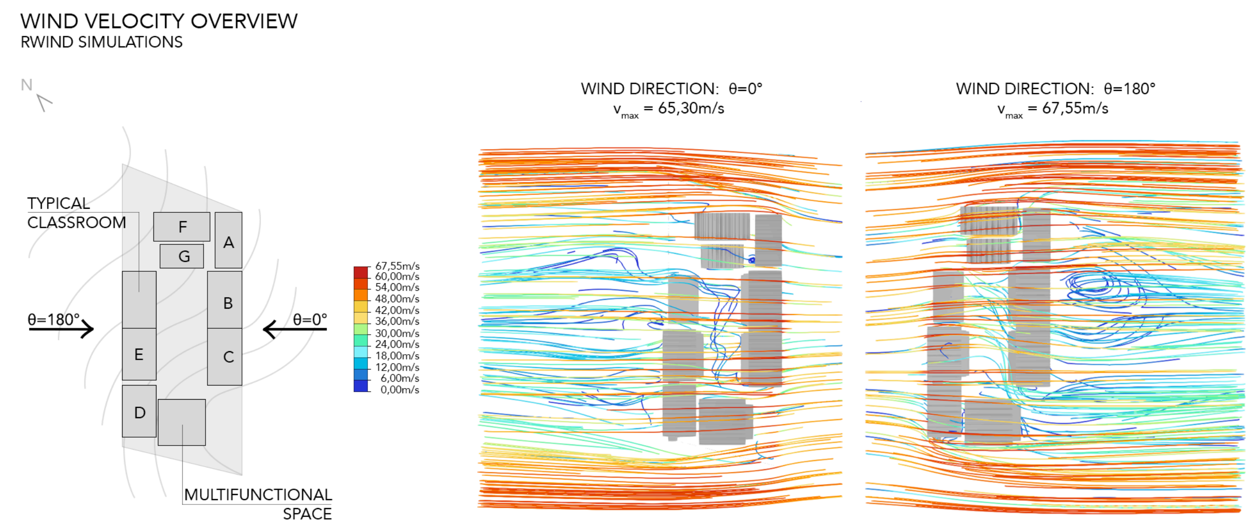


Fig.5.7 - Interference phenomena analyses carried out with RWIND Simulation

In conclusion, RWIND simulations clearly show the interference phenomena occurring within the site: the layout of the project significantly influences wind action in both its directions as shown in the images above. But it is worth pointing out that, despite the boundary buildings hinder and divert the air flow, the distances among the buildings and the dimensions of the open spaces, thanks also to the lay of the land, do not foster the air flow nor allow wind to regain its most severe flow speed. Indeed, the wind hit at its maximum velocity only the first buildings perpendicular to its direction. The resulting deviation do not involve the formation of particularly strong air flow that might affect the surrounding constructions. As a matter of fact, these buildings act as barriers, protecting the cover courtyard and the facing buildings.

5.5.2. VENTILATION GAPS

Then the attention has been paid to the ceiling: ventilation gaps have been integrated in the horizontal enclosure to allow hot air to escape upwards, improving the air quality inside the room. Furthermore, it is fundamental to verify that the presence of the ventilation gaps do not generate an inappropriate air flow which would lead to a general thermal discomfort inside the room because they cannot be closed.
According to previous analyses, RWIND simulations have been carried out considering the worst ever condition possible which corresponds to 180°-wind for the typical classroom and 0°-wind for the multifunctional space.

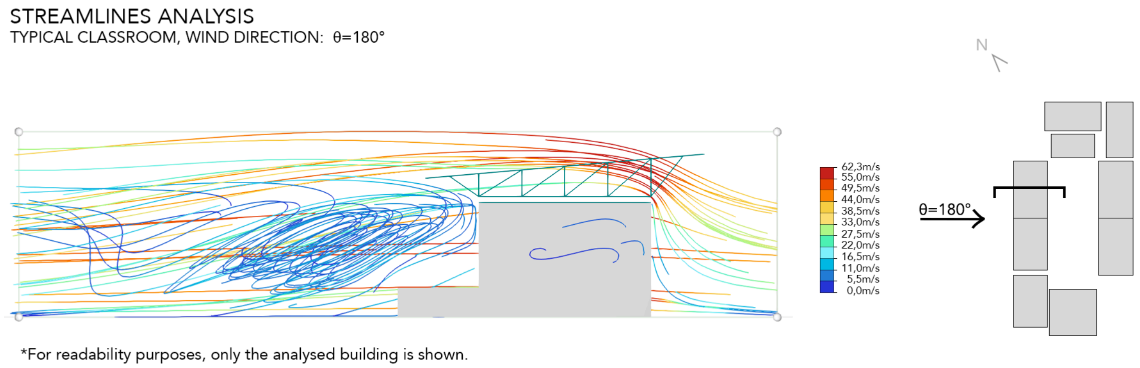


Fig.5.8 - Streamlines analysis carried out with RWIND Simulation

STREAMLINES ANALYSIS
MULTIFUNCTIONAL SPACE, WIND DIRECTION: θ=0°

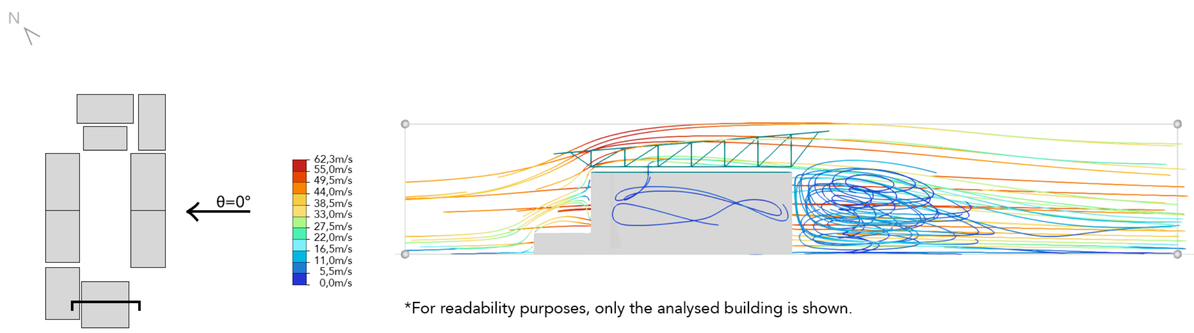


Fig.5.9 - Streamlines analysis carried out with RWIND Simulation

The sections above show the flows problem as a whole, clarifying how the wind interacts with the buildings not only outside, but also inside. As already said, internal pressure is strictly related to size and distribution of the openings in the building envelope. Windows and doors are supposed to be closed during cyclones, therefore only the ventilation gaps are left as available external openings. Their dimensions are small enough they do not significantly affect the net pressure acting on the ceiling, which has anyway negative direction as suction.
This is why in both cases, simulations do not show descending air flows: indeed, the difference in pressure between inside and outside surely causes an acceleration of the inner air flow, but the external strong currents do not enter despite the ventilation gaps.

The blue streamlines inside the buildings represent the movement of the internal air which, due to the difference in pressure, tends to leave the room throughout the ceiling integrated ventilation gaps. As shown by RWIND streamline animation, these air flows are neither constant in time, nor in velocity.
Analysing the results of these latter simulations in terms of velocity fields, the maximum velocity these gusts reached is equal to 2,7 m/s (i.e., 9,7 km/h) in the typical classroom and 3,3 m/s (i.e., 11,88 km/h) in the multifunctional space. According to Beaufort

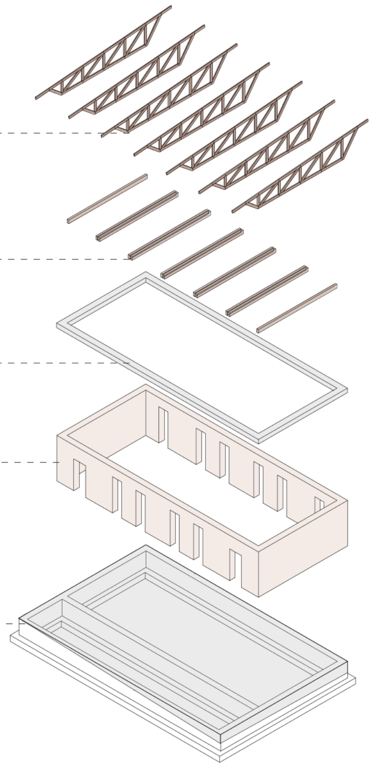
wind force scale, both the values fall within the limits of force 2 which is identify as a light breeze: wind is felt on faces and causes leaves rustled.
In both cases, the maximum speed of the internal air flow is lower than the average annual wind speed. This correlation is important since natural ventilation was one of the main strategies for providing acceptable indoor environmental quality.
As seen in chapter 4, in optimal climatic conditions (i.e., absence of cyclones), wind at an average speed of 14 km/h guarantees air exchanges. As a matter of fact, the drafts occurring in the event of cyclones due to the presence of the ventilation gaps are not different from the conditions sought to guarantee healthy and comfortable indoor environments throughout the year. Indeed, considering the lack of a mechanical ventilation system, the integrated ventilation gaps ensure air exchanges and therefore the healthiness of the closed environments allowing for exhausted air to be taken out of the room.

5.5.3. FINAL CONSIDERATION

RWIND simulations certainly helped us in better understating how interference phenomena act within the project site and how they affect the indoor environment of our buildings through the ventilation gaps integrated in the ceilings.
Unfortunately, however, cyclones do not have a regular evolution as they are real atmospheric vortexes. In particular, tropical cyclones are characterised by winds whose intensity normally increases inward towards the centre, where, however, they do not converge. They become tangent to a circle of variable diameter (between 5 and 30 km) known as the eye of the cyclone, characterized by weak winds or nothing but quiet.
In a context of particular uncertainty as this one, we considered it more appropriate to verify the resistance of our structure according to the more conservative values calculated according to EN 1991.1.4.

SCHOOLYARD - CHILD'S VIEW

5.6. TYPICAL CLASSROOM

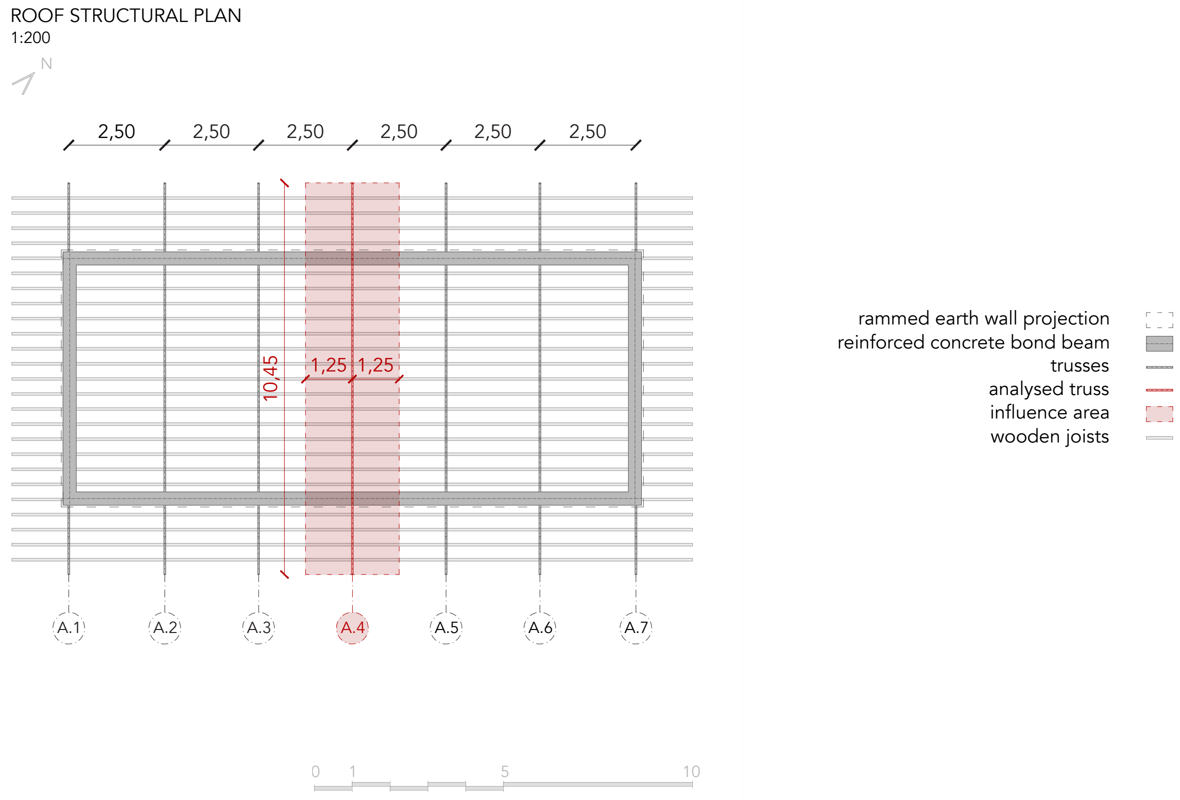


In the following paragraph, structural verifications concerning a typical classroom are presented.

Particular attention has been payed to the effects of wind action on the structure due to the vulnerability of the city to cyclones. Since buildings orientation allows for favourable wind conditions, taking advantage of the natural air flow to provide a good level of indoor comfort, the effects of cyclones on the load-bearing walls have been investigated according to two main direction: we assumed wind blowing firstly at an angle of 0° and secondly at an angle of 180°.

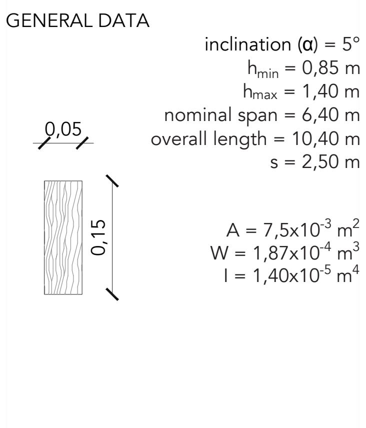
Moreover, concerning roof and ceiling structures, since in case of cyclone winds can reach the 155km/h, each load-bearing element has been analysed considering wind as the leading variable action. Thus, the imposed load on roof has been excluded from these calculations, considering also that during a tropical storm maintenance and repair are not supposed to occur. Furthermore, this vertical load would have reduced wind effects on the structure due to its opposite direction of action.

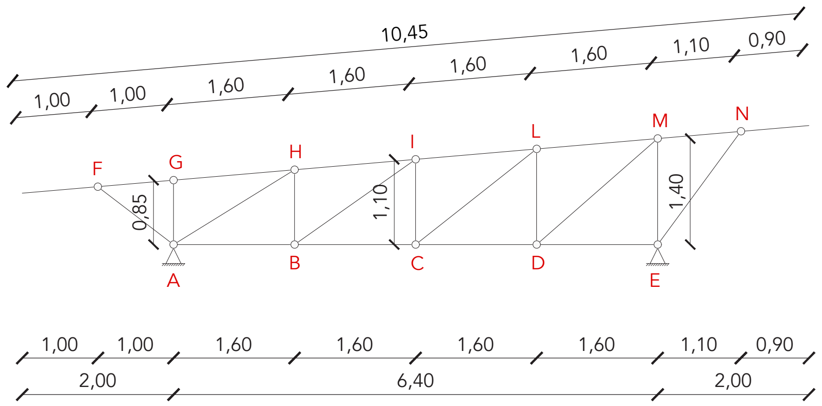
5.6.1. TIMBER ROOF TRUSS DESIGN



As shown in the drawing above, the outermost trusses run for the entire lower chord length on the reinforced concrete bond beams. Even though higher wind loads and slightly higher vertical loads coming from the roof weight on their upper chord, their constraint conditions involve almost no internal pressure acting on the lower chords, as well as a distribution of loads and internal forces, and a final deformation not representative for the structural element under analysis. Therefore, we decided to focus our attention to beam A.4, verifying the resistance of the outermost trusses at a later time in the global analysis of the structural behaviour of the building carried out with the software RFEM.

According to the guidelines and constructive principles developed by the Ministry of education and human development (MINEDH) of the Republic of Mozambique within the "Escolas Seguras" project, to fulfil structural safety requirements in areas vulnerable to cyclones, roof truss members' cross section should be at least equal to 150x50 mm^2 and steel plates 4mm thick should be used to joined them.[5]





Loads combinations at ultimate limit state

In order to determine wind loads, the roof have been idealised as a canopy. Not having permanent walls, it is considered as an open structure and thus there is no internal pressure to consider.

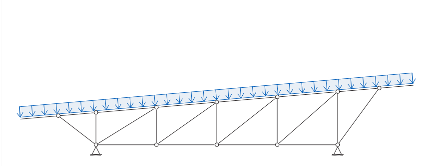
Therefore, three load combinations have been studied: first of all, we focus on verifying that the minimum recommended dimensions were appropriate with respect to our structural layout. For this purpose, only the permanent loads and the imposed load on roof have been considered and maximized. Then, we analysed the resistance of the structure introducing also the wind loads acting, respectively, at 0° in the second loads combination, at 180° in the third loads combination.

5 MINEDH, INGC, The World Bank Group, GFDRR, FAPF, UN-HABITAT. (2019). Projecto Escolas Seguras. Catálogo de Medidas Técnicas. *Ministerio da Educacao e Desenvolvimento Humano.*

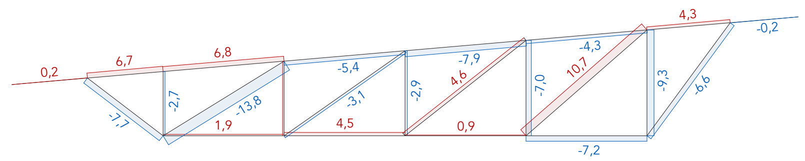
I. combination

Tab.5.17 - Load combination

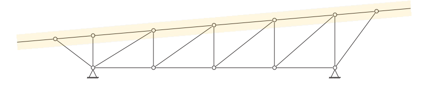
Permanent loads	kN/m	γ_G	Variable loads	kN/m		γ_Q	ψ_0	ψ_1	ψ_2
G_1	0,17	1,35	$Q_{CAT.H}$	1,00		1,50	0	0	0
G_2	0,77			0°	180°				
			$Q_{WIND,A}$	-4,15	-4,50		0,6	0,2	0
			$Q_{WIND,C}$	-6,69	-7,36				
	F_d =		$\gamma_G(G_1 + G_2) + \gamma_Q Q_{CAT.H}$			= 2,77 kN/m			



Axial forces diagram (kN)



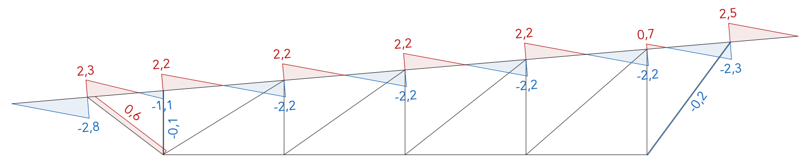
*The upper chord shows a linear distribution of axial forces. For readability purposes, only the maximum values have been reported.



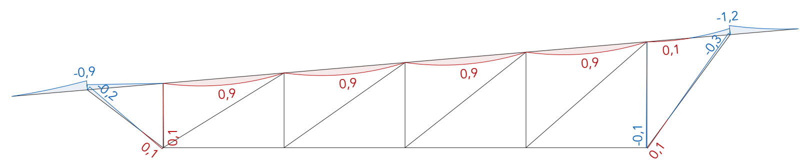
Compression (-)

Tension (+)

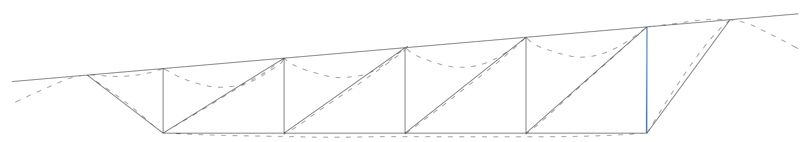
Shear forces diagram (kN)



Bending moments diagram (kNm)

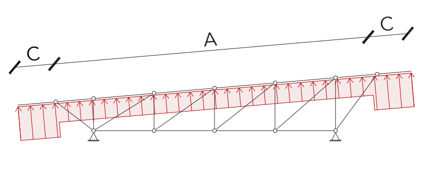


Deformation diagram



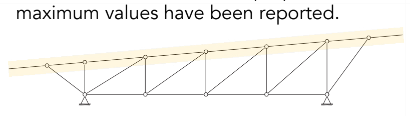
II. combination (wind 0°)

Tab.5.18 - Load combination



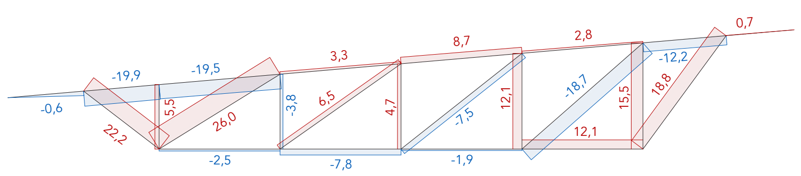
Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	Ψ_0	Ψ_1	Ψ_2
G_1	0,17	1,35	$Q_{CAT.H}$	1,00		1,50	0	0	0
G_2	0,77			0°	180°				
			$Q_{WIND,A}$	-4,15	-4,50		0,6	0,2	0
			$Q_{WIND,C}$	-6,69	-7,36				
		$F_{d,WIND\ C}$ =	$\gamma_G(G_1+G_2)+\gamma_Q[Q_{WIND,C}+(\Psi_0 Q_{CAT.H})]$			= -8,92 kN/m			
		$F_{d,WIND\ A}$ =	$\gamma_G(G_1+G_2)+\gamma_Q[Q_{WIND,A}+(\Psi_0 Q_{CAT.H})]$			= -4,96 kN/m			

*The upper chord shows a linear distribution of axial forces. For readability purposes, only the maximum values have been reported.

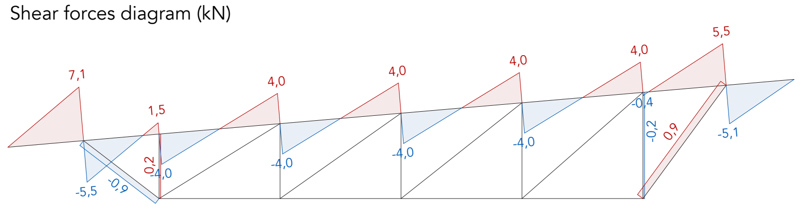


- Compression (-)
- Tension (+)

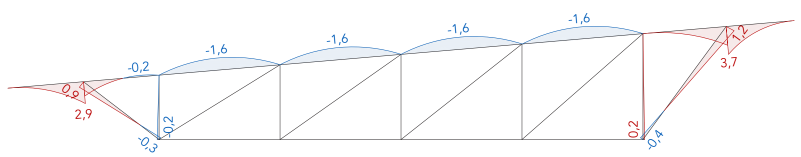
Axial forces diagram (kN)



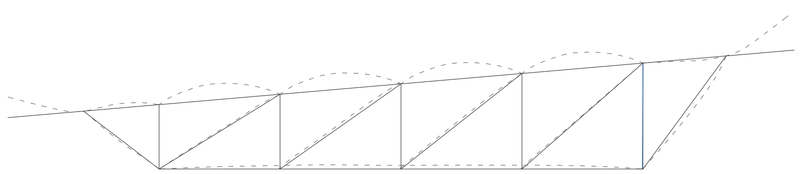
Shear forces diagram (kN)



Bending moments diagram (kNm)



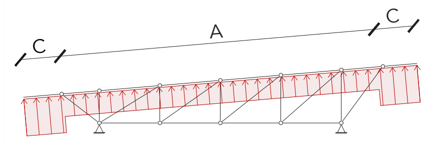
Deformation diagram



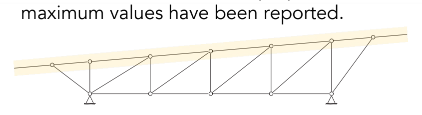
III. combination (wind 180°)

Tab.5.19 - Load combination

Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	Ψ_0	Ψ_1	Ψ_2
G_1	0,17	1,35	$Q_{CAT.H}$	1,00		1,50	0	0	0
G_2	0,77			0°	180°				
			$Q_{WIND,A}$	-4,15	-4,50		0,6	0,2	0
			$Q_{WIND,C}$	-6,69	-7,36				
		$F_{d,WIND\ C}$ =	$\gamma_G(G_1+G_2)+\gamma_Q[Q_{WIND,C}+(\Psi_0 Q_{CAT.H})]$			= -9,78 kN/m			
		$F_{d,WIND\ A}$ =	$\gamma_G(G_1+G_2)+\gamma_Q[Q_{WIND,A}+(\Psi_0 Q_{CAT.H})]$			= -5,49 kN/m			

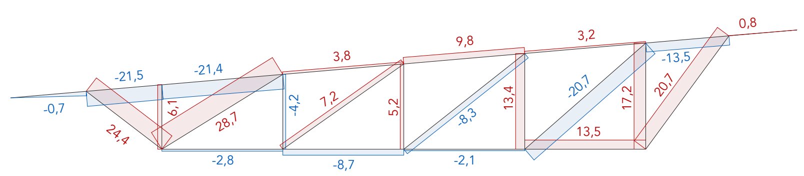


*The upper chord shows a linear distribution of axial forces. For readability purposes, only the maximum values have been reported.

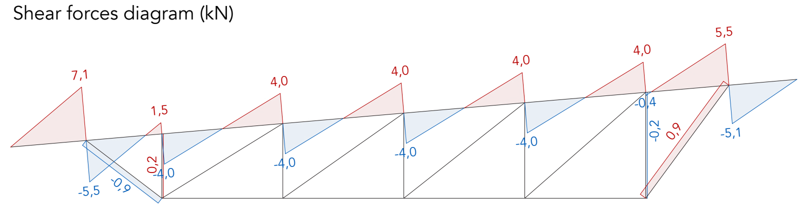


- Compression (-)
- Tension (+)

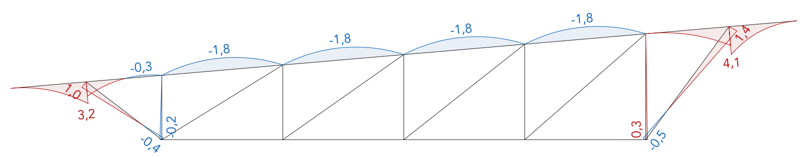
Axial forces diagram (kN)



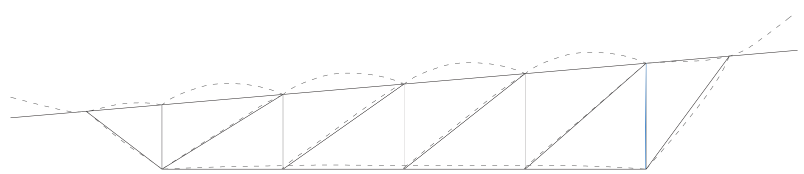
Shear forces diagram (kN)



Bending moments diagram (kNm)



Deformation diagram



Tab.5.20 - Design values (γ_m = 1,3)

Properties	$k_{mod,I}$	$k_{mod,II}$
(in N/mm²)	**0,5**	**0,7**
$f_{m,d}$	19,23	26,92
$f_{t,0,d}$	11,54	16,15
$f_{t,90,d}$	0,23	0,32
$f_{c,0,d}$	11,15	15,62
$f_{c,90,d}$	3,58	5,01
$f_{v,d}$	1,54	2,15

* $k_{mod,I}$ = 0,5 for permanent actions
$k_{mod,II}$ = 0,7 for short-term actions

Ultimate limit states verifications

The dimensioning of web members is generally governed by resistance verification to tension or compression.

Through the reticular arrangement and the hinged schematisation of the internal nodes:

- the bending moment translates into a compressive or tensile force acting in the members
- the shear force introduces a state of axial stress in the elements

Thus, even verifications to combined compression or tension and bending are fundamental to define members' dimensions.

Shear verification

It has to be verified that

$$M_{dh}^{*} \leq M_{ch}$$

Tab.5.21 - Shear verification at ULS

	V_{max}	A	τ_d	$f_{v,d}$	
	kN	mm²	N/mm²	N/mm²	
I.	-2,8	7500	0,56	1,54	**0,36<1**
II.	7,1	7500	1,42	2,15	**0,66<1**

Combined tension and bending moment verification

The condition to be satisfied is the following

$$M_{ch} = \Phi f_{et} Z_u$$

Tab.5.22 - Combined tension and bending moment verification at ULS

	N_{max}	M_{max}	k_m	$\sigma_{t,0,d}$	$\sigma_{m,d}$	$f_{t,0,d}$	$f_{m,d}$	
	kN	kNm		N/mm²	N/mm²	N/mm²	N/mm²	
I.	6,8	0,9	0,7	0,91	4,80	11,54	19,23	**0,20<1**
III.	9,8	1,8	0,7	1,31	9,60	16,15	26,92	**0,33<1**

Combined compression and bending moment verification

The resistance of elements is guaranteed if the following condition is satisfied

$$V^{*} \leq \Phi[\, f_{es} A_b + k_v f_d A_b]$$

However, it is not sufficient to guarantee structural safety. Timber structural elements cross-sections are usually characterized by height greater than width to ensure adequate bearing capacity as well as flexural rigidity in the vertical plane. And this may lead to instability phenomena. Indeed, the compression force arising in a part of the element may cause a roto-translational deflection of the cross-section. Thus, a lateral inflection and a rotation around the longitudinal x axis of the element would occur.

To avoid instability phenomena (buckling), EN 1995 (2004) provides an additional condition to satisfy

$$\left(\frac{\sigma_{m,d}}{k_{crit} f_{m,d}}\right)^2 + k_m \left(\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}}\right) \leq 1$$

where

- $k_{c,z}$=1 $\lambda_{rel} < \bar{\lambda}_{rel}$

Tab.5.23 - Combined compression and bending moment verifications at ULS

	N_{max}	M_{max}	k_m	$\sigma_{c,0,d}$	$\sigma_{m,d}$	$f_{c,0,d}$	$f_{m,d}$	
	kN	kNm		N/mm²	N/mm²	N/mm²	N/mm²	
III.	-0,7	3,2	0,7					**0,44<1**
	λ_{rel}	λ_{rel}	$k_{c,z}$	0,0001	17,07	15,62	26,92	
	0,40	0,5	1					**0,003<1**
III.	-21,4	1,8	0,7					**0,25<1**
	λ_{rel}	λ_{rel}	$k_{c,z}$	0,003	9,60	15,62	26,92	
	0,40	0,5	1					**0,01<1**

Tension verification

It has to be verified that

$$\sigma_{t,0,d} \leq f_{t,0,d}$$

Tab.5.24 - Parallel tension verification at ULS

	N_{max}	A	$\sigma_{t,0,d}$	$f_{t,0,d}$	
	kN	mm²	N/mm²	N/mm²	
III.	lower chord 13,5	7500	1,80	16,15	**0,11<1**
III.	vertical member 17,2	7500	2,87	16,15	**0,26<1**
III.	inclined member 28,7	7500	4,78	16,15	**0,44<1**

Compression verification

It has to be verified that

$$\sigma_{c,0,d} \leq f_{c,0,d}$$

Tab.5.25 - *Parallel compression verification at ULS*

	N_{max}	A	$\sigma_{t,0,d}$	$f_{t,0,d}$	
	kN	mm^2	N/mm^2	N/mm^2	
III.	lower chord -8,7	7500	1,16	15,62	**0,07<1**
I.	vertical member -9,3	7500	1,55	11,15	**0,15<1**
III.	inclined member -20,7	7500	3,45	15,62	**0,33<1**

Serviceability limit states verifications

As previously explained, the final displacement of a timber structural element depends on both the instantaneous and delayed deformation. The former have been evaluated via RFEM structural simulation analysis. In the following table, the maximum displacement are reported:

Tab.5.26 - *Deformation verification at SLE*

Upper chord

$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$	< l/300
mm	mm	mm	mm	mm
0,90	-6,1	0,4	-4,80	21,33

$u_{fin,G}$	$u_{fin,Q1}$	$u_{fin,Qi}$	$u_{fin,TOT}$	< l/200
mm	mm	mm	mm	mm
2,7	-6,1	0	-3,40	32

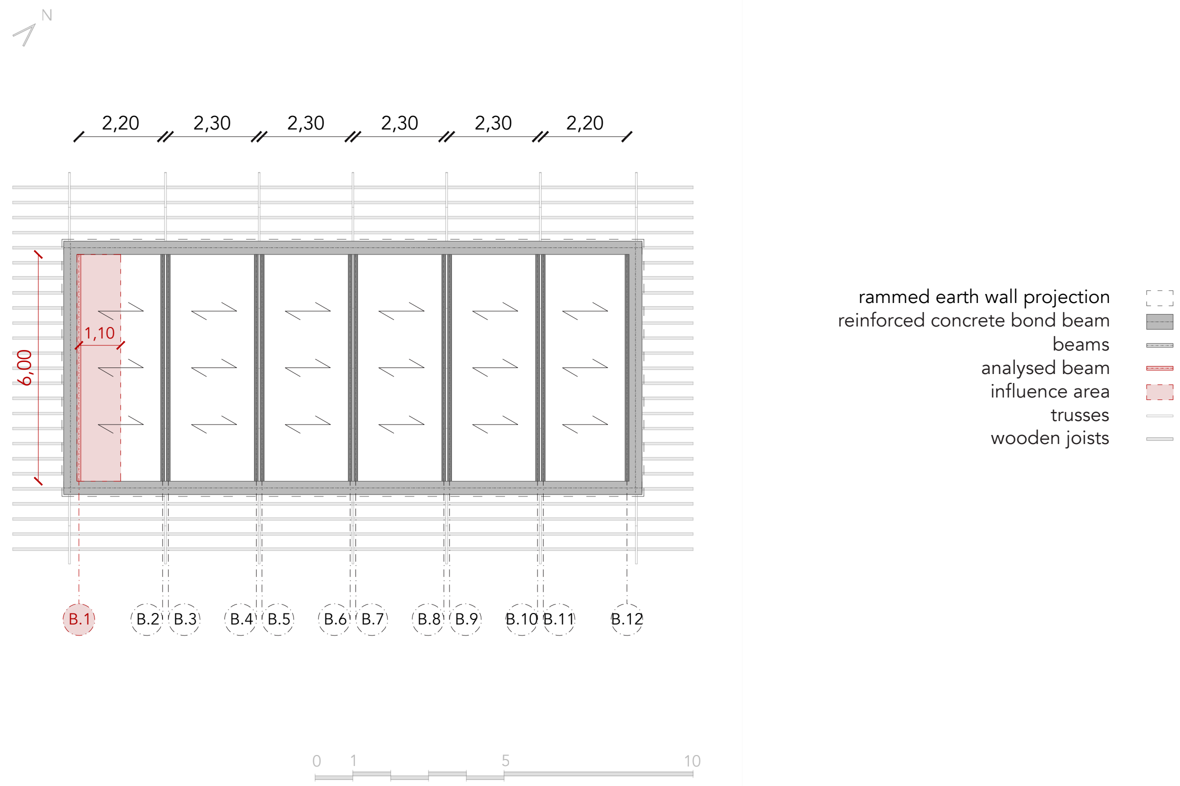
Lower chord

$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$	< l/300
mm	mm	mm	mm	mm
0,60	-4,2	0,3	-3,30	21,33

$u_{fin,G}$	$u_{fin,Q1}$	$u_{fin,Qi}$	$u_{fin,TOT}$	< l/200
mm	mm	mm	mm	mm
1,8	-4,2	0	-2,40	32

5.6.2. TIMBER CEILING BEAM DESIGN

CEILING STRUCTURAL PLAN
1:200



The calculations performed shows that wind has a stronger impact on the ceiling extremities, leading to higher stresses and, consequently, to greater deflections on the outermost beams rather than on the central ones. Verifying the formers, the resistance of the latter would be guaranteed as well.

The highlighted beam B.1 is the one selected for the design purpose of this section. A cross-section of 120x240 mm^2 and metal brackets connecting the beam to the concrete bond beams have been assumed.

Loads combinations at ultimate limit state

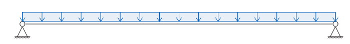
As for the previous element, three main load combinations have been studied: first of all, we focus on verifying that the assumed cross-section was appropriate for our structural layout. For this purpose, only the permanent loads and the imposed load on roof have been considered and maximized. Then, we analysed the resistance of the structure introducing also the wind actions, respectively, at 0° in the second loads combination, at 180° in the third loads combination. These latter combinations were splitted in two subcases to consider both the positive and negative values of external pressure coefficients in zone I, as recommended by EN 1991.1.4 (2005).

Since external openings are supposed to be closed during severe windstorms, the condition with doors and windows open has been considered as an accidental design situation as recommended by EN 1991.1.4 (2005).

I. combination

Tab.5.27 - *Load combination*

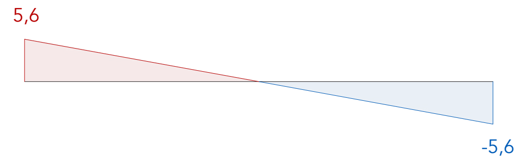
Permanent loads			**Variable loads**						
	kN/m	γ_G		kN/m		γ_Q	ψ_0	ψ_1	ψ_2
G_1	0,18	1,35	$Q_{CAT.H}$	0,46		1,50	0	0	0
G_2	0,65			0°	180°				
			$Q_{WIND,F}$	-3,08	-3,34		0,6	0,2	0
			$Q_{WIND,H}$	-1,20	-1,30				
			$Q_{WIND,I-}$	-0,34	-0,37				
			$Q_{WIND,I+}$	0,34	0,37				
			Q_{pi}	0,27	0,09		1	1	1
		F_d =	$\gamma_G(G_1 + G_2) + \gamma_Q Q_{CAT.H}$				**= 1,80 kN/m**		



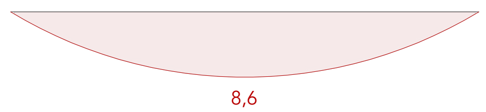
Shear forces diagram (kN)

Compression (-)

Tension (+)



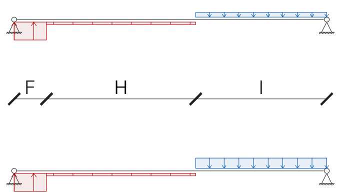
Bending moments diagram (kNm)



Deformation diagram

II. combination

Tab.5.28 - Load combination



Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	Ψ_0	Ψ_1	Ψ_2
G_1	0,18	1,35	$Q_{CAT.H}$	0,46		1,50	0	0	0
G_2	0,65			0°	180°				
			$Q_{WIND,F}$	-3,08	-3,34		0,6	0,2	0
			$Q_{WIND,H}$	-1,20	-1,30				
			$Q_{WIND,I-}$	-0,34	-0,37				
			$Q_{WIND,I+}$	0,34	0,37				
			Q_{pi}	0,27	0,09		1	1	1

$F_{d,WIND\ F}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,F}+(\Psi_0 Q_{CAT,H})]$	**= -3,24 kN/m**
$F_{d,WIND\ H}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,H}+(\Psi_0 Q_{CAT,H})]$	**= -0,41 kN/m**
$F_{d,WIND\ I-}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,I-}+(\Psi_0 Q_{CAT,H})]$	**= 0,87 kN/m**
$F_{d,WIND\ I+}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,I+}+(\Psi_0 Q_{CAT,H})]$	**= 1,90 kN/m**

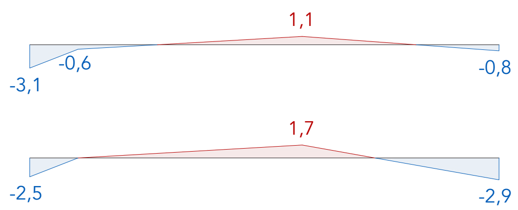
Compression (-)

Tension (+)

Shear forces diagram (kN)

Zone I:
NEGATIVE

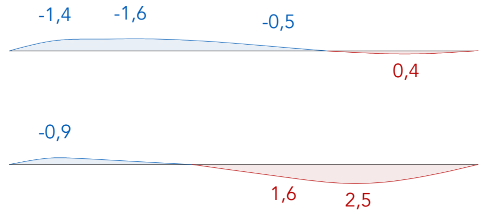
Zone I:
POSITIVE



Bending moments diagram (kNm)

Zone I:
NEGATIVE

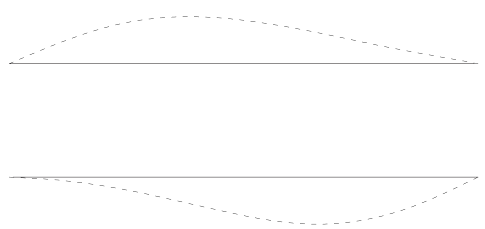
Zone I:
POSITIVE



Deformation diagram

Zone I:
NEGATIVE

Zone I:
POSITIVE

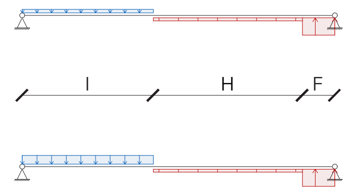


III. combination

Tab.5.29 - Load combination

Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	Ψ_0	Ψ_1	Ψ_2
G_1	0,18	1,35	$Q_{CAT.H}$	0,46		1,50	0	0	0
G_2	0,65			0°	180°				
			$Q_{WIND,F}$	-3,08	-3,34		0,6	0,2	0
			$Q_{WIND,H}$	-1,20	-1,30				
			$Q_{WIND,I-}$	-0,34	-0,37				
			$Q_{WIND,I+}$	0,34	0,37				
			Q_{pi}	0,27	0,09		1	1	1

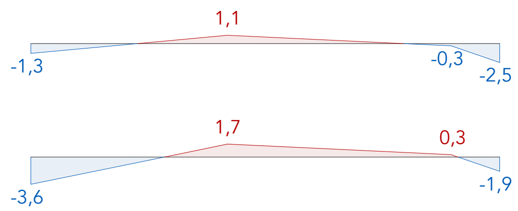
$F_{d,WIND\ F}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,F}+(\Psi_0 Q_{CAT,H})]$	**= -3,81 kN/m**
$F_{d,WIND\ H}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,H}+(\Psi_0 Q_{CAT,H})]$	**= -0,75 kN/m**
$F_{d,WIND\ I-}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,I-}+(\Psi_0 Q_{CAT,H})]$	**= 0,65 kN/m**
$F_{d,WIND\ I+}$ =	$\gamma_G(G_1+G_2)+\Psi_1 Q_{pi}+\gamma_Q[Q_{WIND,I+}+(\Psi_0 Q_{CAT,H})]$	**= 1,76 kN/m**



Shear forces diagram (kN)

Zone I:
NEGATIVE

Zone I:
POSITIVE



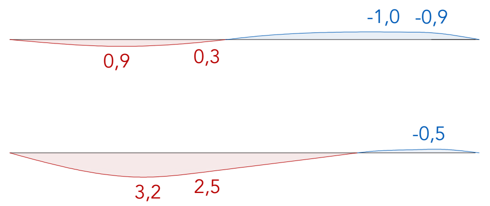
Compression (-)

Tension (+)

Bending moments diagram (kNm)

Zone I:
NEGATIVE

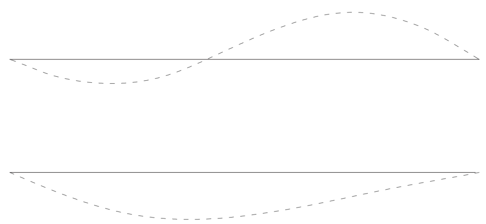
Zone I:
POSITIVE



Deformation diagram

Zone I:
NEGATIVE

Zone I:
POSITIVE



Tab.5.30 - *Design values (γ_m = 1,3)*

Properties	$k_{mod,I}$	$k_{mod,II}$
(in N/mm²)	**0,5**	**0,7**
$f_{m,d}$	19,23	26,92
$f_{t,0,d}$	11,54	16,15
$f_{t,90,d}$	0,23	0,32
$f_{c,0,d}$	11,15	15,62
$f_{c,90,d}$	3,58	5,01
$f_{v,d}$	1,54	2,15

** $k_{mod,I}$ = 0,5 for permanent actions*
$k_{mod,II}$ = 0,7 for short-term actions

Ultimate limit states verifications

The most critical criteria for medium-span beams is bending which causes shear stresses to arise.

Both bending moment and shear reach their maximum values under the first load combination. Indeed, due to the lack of wind loads acting in contrast to the compressive loads, the first load combination is more severe than the others analysed.

Bending verification

It has to be verified that

$$M_{dh}^{*} \leq M_{ch}$$

Tab.5.31 - *Bending verification at ULS*

	M_{max}	W	$\sigma_{m,d}$	$f_{m,d}$	
	kN	mm³	N/mm²	N/mm²	
I.	8,6	1152000	7,46	19,23	**0,38<1**
III.	3,2	1152000	2,77	26,92	**0,10<1**

Shear verification

It has to be verified that

$$M_{dh}^{*} \leq M_{ch}$$

Tab.5.32 - *Shear verification at ULS*

	V_{max}	A	τ_d	$f_{v,d}$	
	kN	mm²	N/mm²	N/mm²	
I.	5,6	7500	0,29	1,54	**0,19<1**
III.	-3,6	7500	0,18	2,15	**0,08<1**

Serviceability limit states verifications

As previously explained, the final displacement of a timber structural element depends on both the instantaneous and delayed deformation. The former have been evaluated via RFEM structural simulation analysis. In the following table, the maximum displacement are reported:

Tab.5.33 - *Deformation verification at SLE*

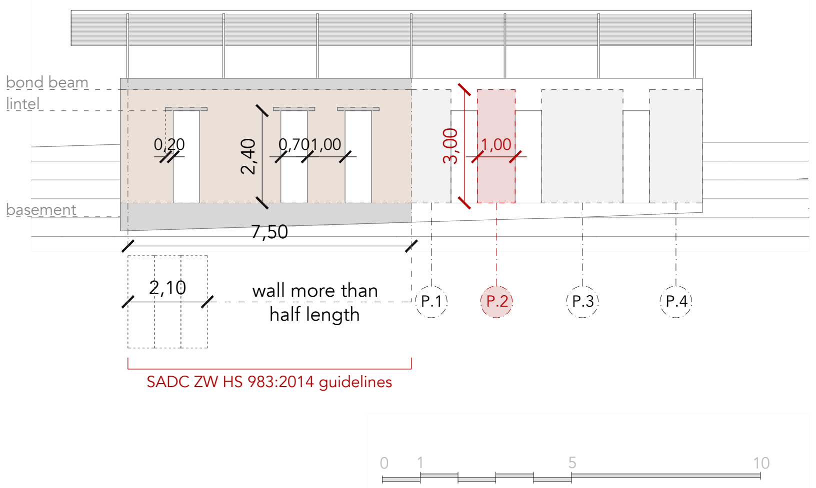
$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$ < l/300	
mm	mm	mm	mm	mm
4,20	-3,0	3,1	4,30	20,67

$u_{fin,G}$	$u_{fin,Q1}$	$u_{fin,Qi}$	$u_{fin,TOT}$ < l/200	
mm	mm	mm	mm	mm
12,6	-3,0	0	9,6	31

5.6.3. RAMMED EARTH WALL DESIGN

NORTH WEST ELEVATION
1:200

rammed earth wall
reinforced concrete
wall sub-panel
analysed wall sub-panel



Walls with openings are considered to form sub-panels either side of the opening. The edges of the sub-panels adjacent to the opening are regarded as being unsupported (ie no lateral support or rotational restraint) with the remaining edges being supported. Generally, to simplify calculations, the openings are assumed to extend for the full height of the wall. The pressure on the opening is considered to be fully transferred to the edge of the two adjoining masonry sub-panels.

The north-west elevation above reported shows the most stressed sub-panel on which compressive and flexural capacity verifications have been performed assuming the sub-panel as supported at top and bottom.

Moreover, according to the guidelines of SADC ZW HS 983:2014, the wall has been assumed 40 cm thick to respect the provided slenderness ratio (1:8).

Verifications have been carried out with the hypothesis of unreinforced masonry since the project site is located in an area of very low earthquake risk.

Ultimate limit states verifications

Generally, walls shall be designed to resist compressive forces, with or without simultaneously acting bending moments, and to withstand short-term transient nature forces such as wind and earthquake loads.

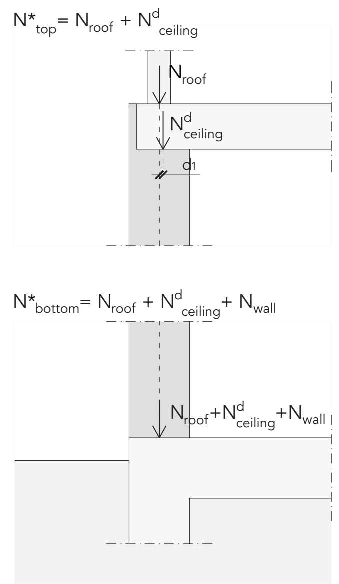
Uniaxial bending and compression

According to NZS 4297:1998, a rammed earth member shall be design such that the following relationship is satisfied:

$$N^* \leq k\Phi N_o$$

where:

- $N_0=f_e A_b$
- k is the reduction factor depending on slenderness and eccentricity to thickness ratio



Tab.5.34 - *Uniaxial bending and compression verifications at ULS*

S_r	e/t_w	k	N_0 (N/mm²)		N^* (kN)	$k\Phi N_0$ (kN)	
7,50	0,11	0,7	0,5	**top**	29,48	84	**0,35<1**
				bottom	65,04		**0,77<1**

Horizontal bending from transient out-of-plane forces

According to NZS 4297:1998, a wall shall be proportioned so that the following relationship is satisfied under each combination of simultaneously acting design horizontal bending moment (M^*_{dh}):

$$M_{dh}^* \leq M_{ch}$$

where

- $M_{ch} = \Phi f_{et} Z_u$
- $Z_u = \frac{bt^2}{6}$

Tab.5.35 - *Horizontal bending verifications at ULS*

f_{et} (N/mm²)	Φ	Z_u (mm³)	M_{dh}^* (kNm*m)	M_{ch} (kNm*m)	
0,1	0,8	26666667	2,06	2,13	**0,97<1**

Shear verification

Walls aligned in parallel to wind direction contribute to the overall stability of the structure against horizontal lateral forces transmitting these latter forces to the ground by in-plane shear.

According to NZS 4297:1998, the design of an unreinforced earth wall subject to shear forces, with or without simultaneous compressive forces acting across the shear plane, shall be such that the following relationship is satisfied under each combination of simultaneously acting design shear force, V *, and (minimum) compressive stress (f_d) acting at the cross section under consideration:

$$V^* \leq \Phi[\, f_{es}A_b + k_v f_d A_b]$$

Tab.5.36 - *Shear verifications at ULS*

f_{es}	A_b	k_v	Φ	V*	$\Phi\,(f_{es}A_b+k_vf_dA_b)$
N/mm²	m²	m²		kN	kNm*m
0,08	1,36	0	0,7	1,66	81,26

Serviceability limit states verification

According to NZS 4297:1998, to satisfy the verification at serviceability limit states it is sufficient to verify that the assumed thickness of the walls, where they are not supported or attached to partitions or other construction, is at least equal to the minimum thickness provided. For our project, the minimum value corresponds to

Tab.5.37 - *Minimum thickness*

Supports	th_{min}	th_{wall}
Simply supported	h/18	
One end continuous	h/21 = 14,28 cm	40 cm
Both end continuous	h/22	
Cantilever	h/8	

Source: NZS 4297:1998

Hence, the provided thickness satisfies the limit established by NZS 4297:1998 and furthermore it guarantees a contained deflection even when cyclones occur.

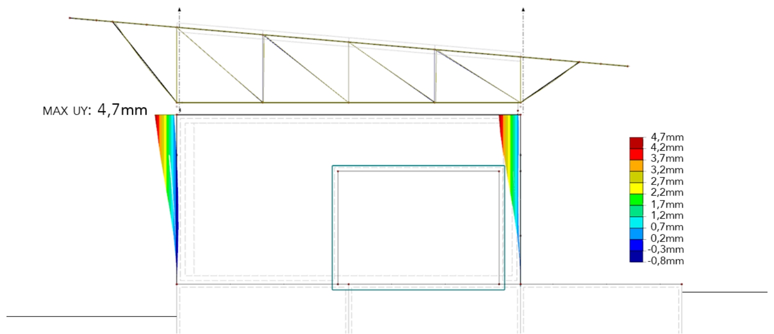
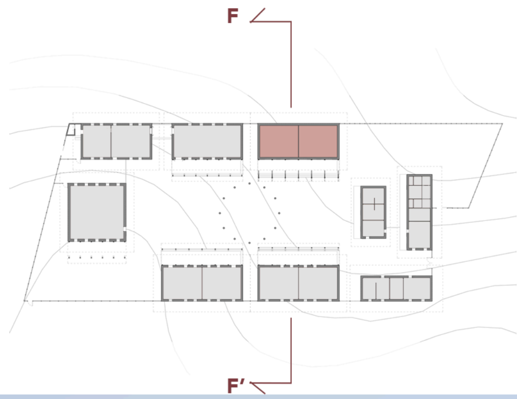
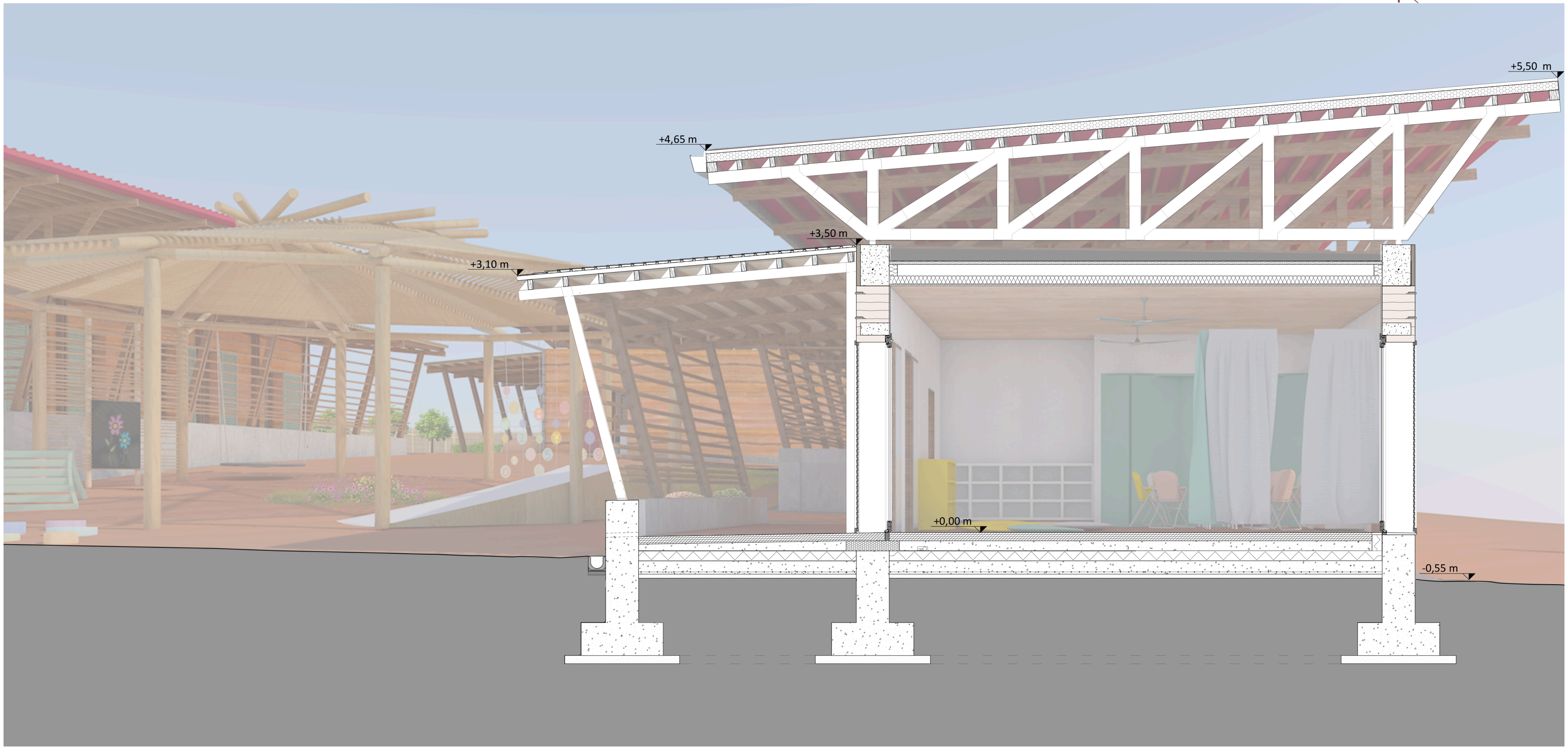


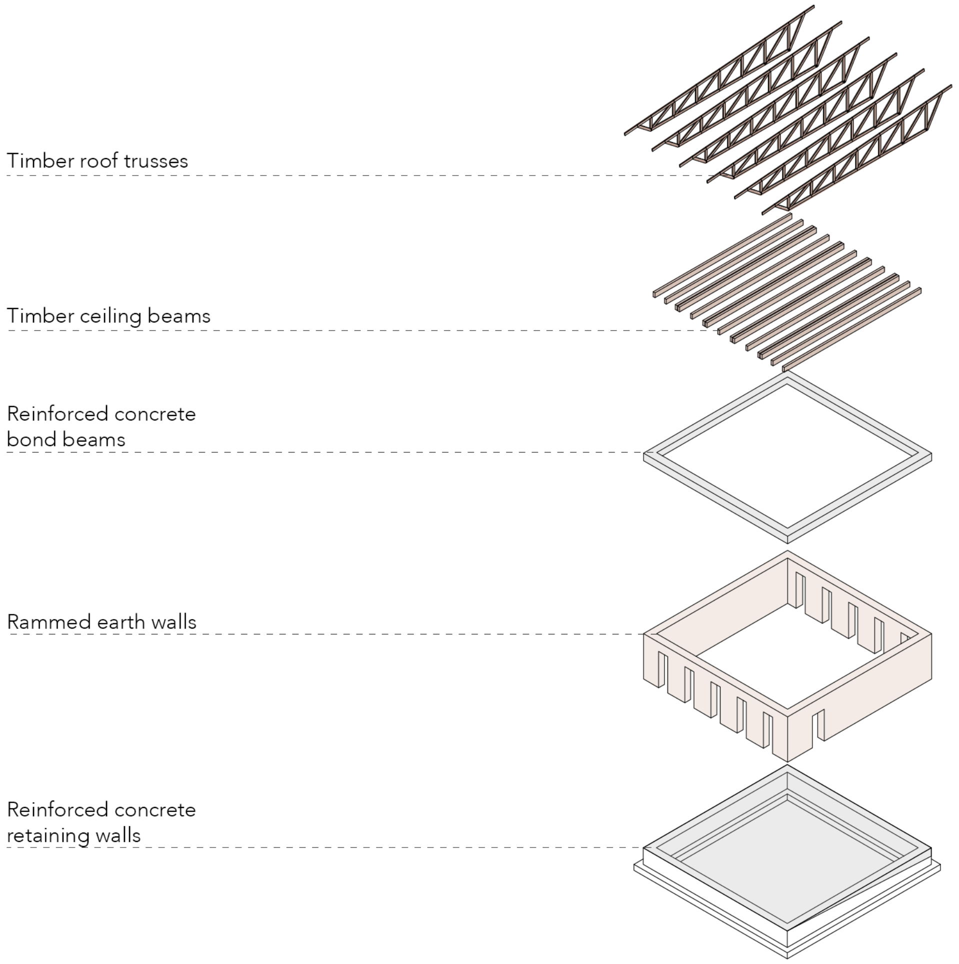
Fig.5.10 - Maximum wall displacement: analysis performed with RFEM.

TYPICAL CLASSROOM: SECTION FF'
1:50





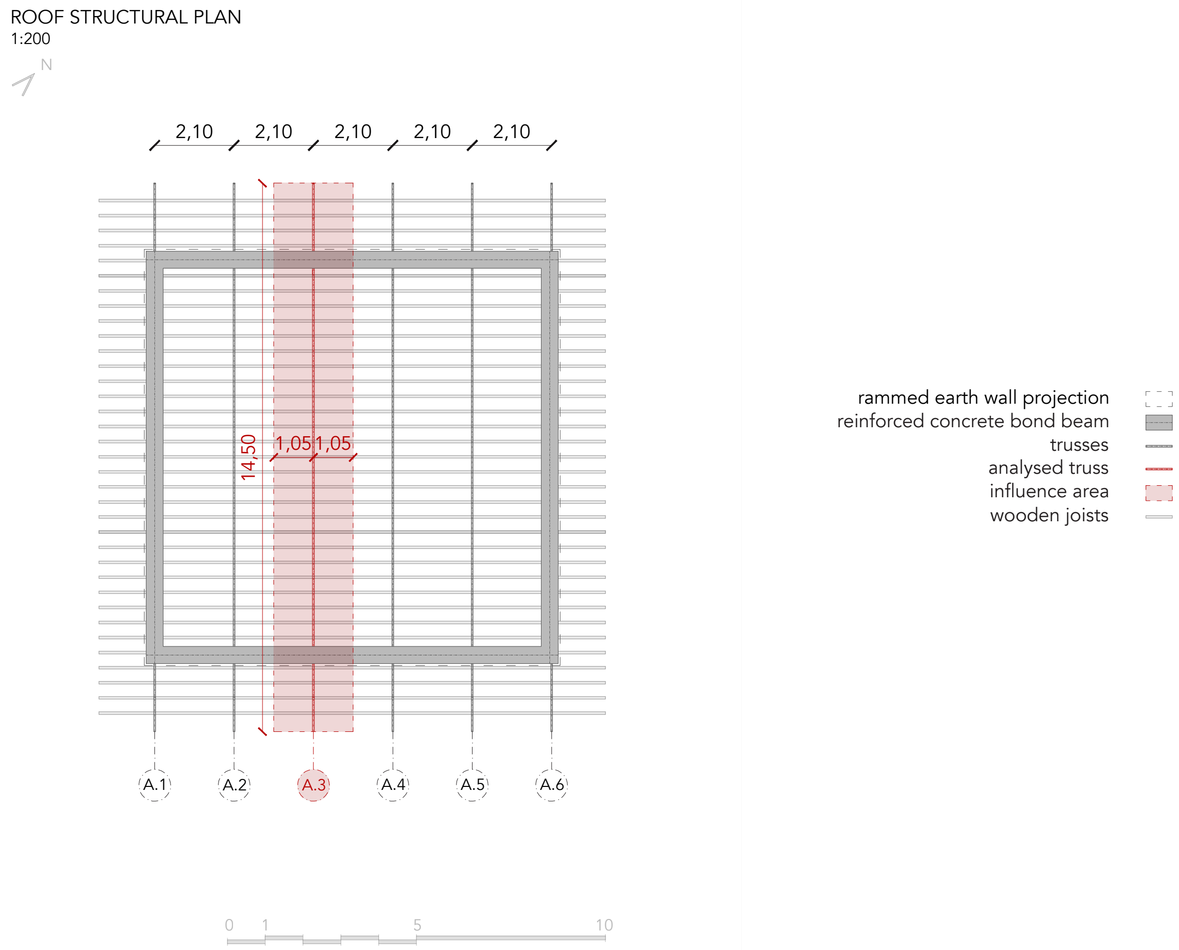
5.7. MULTIFUNCTIONAL SPACE



In the following paragraph, structural verifications concerning the multifunctional space are presented. As in previous verifications, particular attention has been payed to the effects of wind action on the structure. Therefore, the analyses have been performed considering two main direction of action for wind loads: we assumed wind blowing firstly at an angle of 0° and secondly at an angle of 180°.

Concerning roof and ceiling structures, calculations have been performed according to previous assumptions: wind has been considered as the leading variable action, since in case of cyclone it can reach the 155km/h. Therefore, the imposed load on roof has been excluded from these load combinations, considering also that during a tropical storm maintenance and repair are not supposed to occur. Furthermore, this vertical load would have reduced wind effects on the structure due to its opposite direction of action.

5.7.1. TIMBER ROOF TRUSS DESIGN



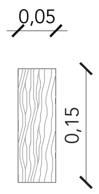
As already outlined, the multifunctional space has different dimensions compared to the other buildings the project is made up of. However, it has to be mentioned that the type of load-bearing elements employed is the same, as well as their structural arrangement. Therefore, the outermost trusses are not representative enough to be analysed and verified here too.

Even though higher wind loads and slightly higher vertical loads coming from the roof weight on their upper chord, they run for the entire lower chord length on the reinforced concrete bond beams. This condition involves almost no internal pressure acting on the lower chords, as well as a distribution of loads and internal forces, and a final deformation that cannot be considered as typical for the structural element under analysis.

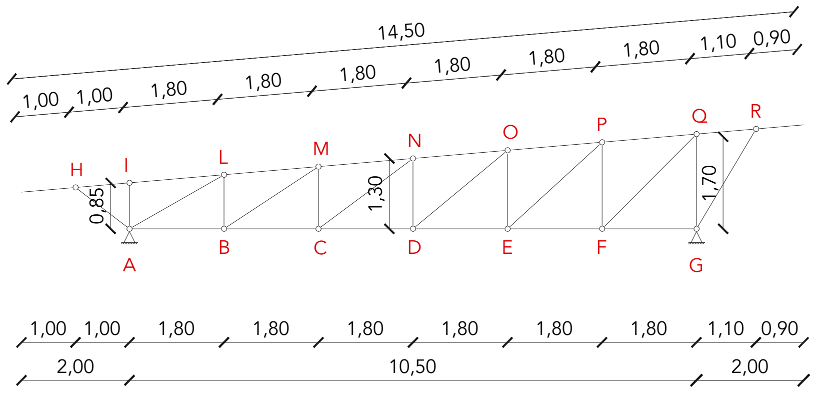
As already mentioned, according to the guidelines and constructive principles developed by the Ministry of education and human development (MINEDH) of the Republic of Mozambique within the “Escolas Seguras” project, roof truss members’ cross section should be at least equal to 150x50 mm² and joining steel plates 4mm thick to fulfil structural safety requirements in areas vulnerable to cyclones.[6]

GENERAL DATA

inclination (α) = 5°
h_{min} = 0,85 m
h_{max} = 1,70 m
nominal span = 10,50 m
overall length = 14,50 m
s = 2,10 m



$A = 7{,}5\times10^{-3}\ m^2$
$W = 1{,}87\times10^{-4}\ m^3$
$I = 1{,}40\times10^{-5}\ m^4$



Loads combinations at ultimate limit state

In order to determine wind loads, the roof have been idealised as a canopy. Not having permanent walls, it is considered as an open structure and thus there is no internal pressure to consider.

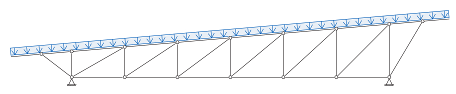
Therefore, three load combinations have been studied: first of all, we focus on verifying that the minimum recommended dimensions were appropriate with respect to our structural layout. For this purpose, only the permanent loads and the imposed load on roof have been considered and maximized. Then, we analysed the resistance of the structure introducing also the wind loads acting, respectively, at 0° in the second loads combination, at 180° in the third loads combination.

6 MINEDH, INGC, The World Bank Group, GFDRR, FAPF, UN-HABITAT. (2019). Projecto Escolas Seguras. Catálogo de Medidas Técnicas. *Ministerio da Educacao e Desenvolvimento Humano*.

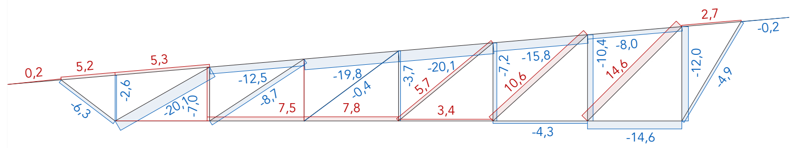
I. combination

Tab.5.38 - Load combination

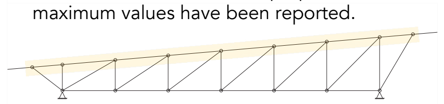
Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	ψ_0	ψ_1	ψ_2
G_1	0,17	1,35	$Q_{CAT.H}$	0,83		1,50	0	0	0
G_2	0,63			0°	180°				
			$Q_{WIND,A}$	-3,44	-3,73		0,6	0,2	0
			$Q_{WIND,C}$	-5,63	-6,10				
	F_d =		$\gamma_G(G_1 + G_2) + \gamma_Q Q_{CAT.H}$			**= 2,34 kN/m**			



Axial forces diagram (kN)



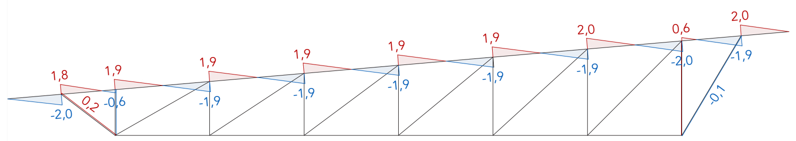
*The upper chord shows a linear distribution of axial forces. For readability purposes, only the maximum values have been reported.



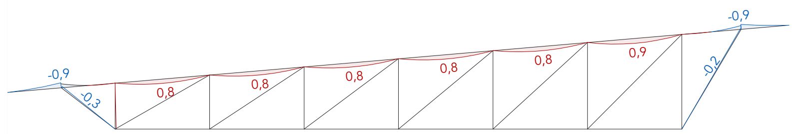
Compression (-)

Tension (+)

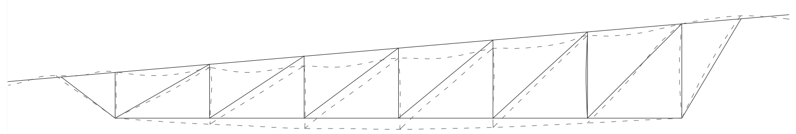
Shear forces diagram (kN)

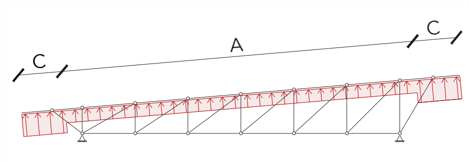


Bending moments diagram (kNm)



Deformation diagram





II. combination (wind 0°)

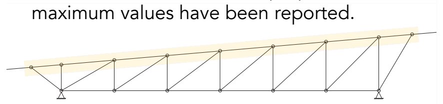
Tab.5.39 - Load combination

Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	ψ_0	ψ_1	ψ_2
G_1	0,17	1,35	$Q_{CAT.H}$	0,83		1,50	0	0	0
G_2	0,77			0°	180°				
			$Q_{WIND,A}$	-3,44	-3,73		0,6	0,2	0
			$Q_{WIND,C}$	-5,63	-6,10				

$F_{d,WIND\ C} = \gamma_G(G_1 + G_2) + \gamma_Q [Q_{WIND,C} + (\Psi_0 Q_{CAT.H})]$ **= -7,35 kN/m**

$F_{d,WIND\ A} = \gamma_G(G_1 + G_2) + \gamma_Q [Q_{WIND,A} + (\Psi_0 Q_{CAT.H})]$ **= -4,07 kN/m**

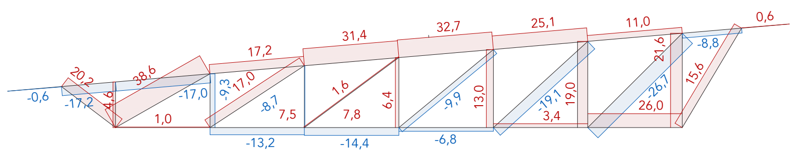
*The upper chord shows a linear distribution of axial forces. For readability purposes, only the maximum values have been reported.



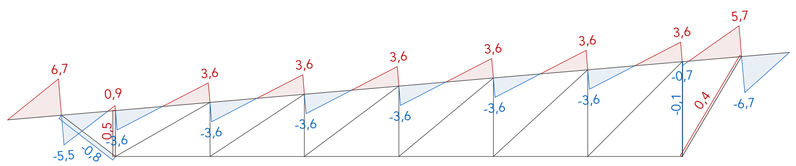
Compression (-)

Tension (+)

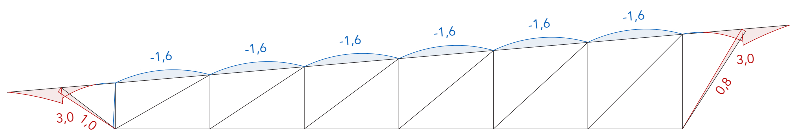
Axial forces diagram (kN)



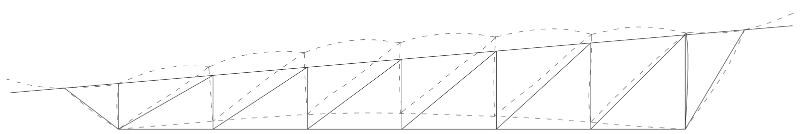
Shear forces diagram (kN)



Bending moments diagram (kNm)



Deformation diagram



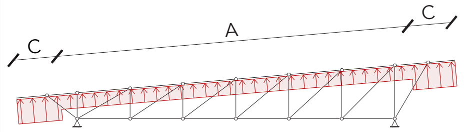
III. combination (wind 180°)

Tab.5.40 - Load combination

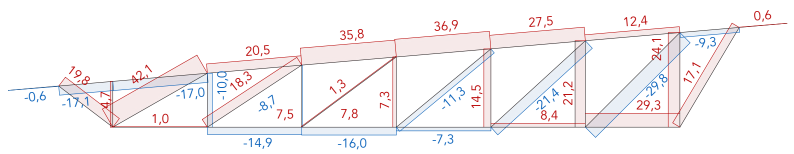
Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	ψ_0	ψ_1	ψ_2
G_1	0,17	1,35	$Q_{CAT.H}$	0,83		1,50	0	0	0
G_2	0,77			0°	180°				
			$Q_{WIND,A}$	-3,44	-3,73		0,6	0,2	0
			$Q_{WIND,C}$	-5,63	-6,10				

$F_{d,WIND\ C} = \gamma_G(G_1 + G_2) + \gamma_Q [Q_{WIND,C} + (\Psi_0 Q_{CAT.H})]$ **= -8,06 kN/m**

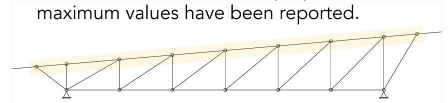
$F_{d,WIND\ A} = \gamma_G(G_1 + G_2) + \gamma_Q [Q_{WIND,A} + (\Psi_0 Q_{CAT.H})]$ **= -4,50 kN/m**



Axial forces diagram (kN)



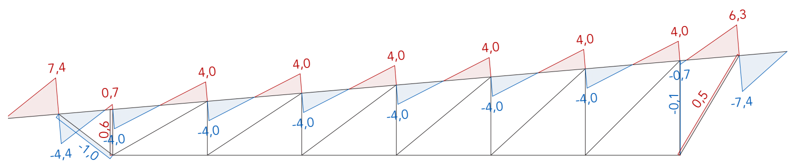
*The upper chord shows a linear distribution of axial forces. For readability purposes, only the maximum values have been reported.



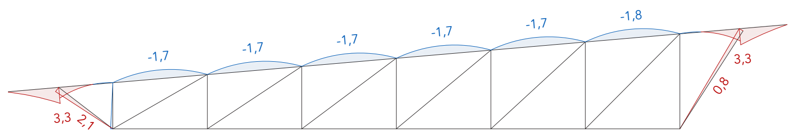
Compression (-)

Tension (+)

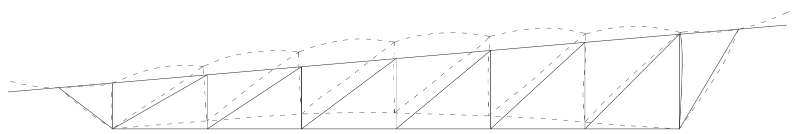
Shear forces diagram (kN)



Bending moments diagram (kNm)



Deformation diagram



Tab.5.41 - Design values (γ_m = 1,3)

Properties	$k_{mod,I}$	$k_{mod,II}$
(in N/mm²)	**0,5**	**0,7**
$f_{m,d}$	19,23	26,92
$f_{t,0,d}$	11,54	16,15
$f_{t,90,d}$	0,23	0,32
$f_{c,0,d}$	11,15	15,62
$f_{c,90,d}$	3,58	5,01
$f_{v,d}$	1,54	2,15

* $k_{mod,I}$ = 0,5 for permanent actions
$k_{mod,II}$ = 0,7 for short-term actions

Ultimate limit states verifications

The dimensioning of web members is generally governed by resistance verification to tension or compression.
Through the reticular arrangement and the hinged schematisation of the internal nodes:
- the bending moment translates into a compressive or tensile force acting in the members
- the shear force introduces a state of axial stress in the elements

Thus, even verifications to combined compression or tension and bending are fundamental to define members' dimensions.

Shear verification

It has to be verified that

$$M_{dh}^{*} \leq M_{ch}$$

Tab.5.42 - Shear verification at ULS

	V_{max}	A	τ_d	$f_{v,d}$	
	kN	mm²	N/mm²	N/mm²	
I.	2,0	7500	0,40	1,54	**0,26<1**
III.	7,4	7500	1,48	2,15	**0,69<1**

Combined tension and bending moment verification

The condition to be satisfied is the following

$$M_{ch} = \Phi f_{et} Z_u$$

Tab.5.43 - Combined tension and bending moment verification at ULS

	N_{max}	M_{max}	k_m	$\sigma_{t,0,d}$	$\sigma_{m,d}$	$f_{t,0,d}$	$f_{m,d}$	
	kN	kNm		N/mm²	N/mm²	N/mm²	N/mm²	
I.	5,3	0,8	0,7	0,71	4,27	11,54	19,23	**0,22<1**
III.	36,9	1,7	0,7	4,92	9,07	16,15	26,92	**0,54<1**

Combined compression and bending moment verification

The resistance of elements is guaranteed if the following condition is satisfied

$$V^{*} \leq \Phi[\, f_{es}A_b + k_v f_d A_b]$$

However, it is not sufficient to guarantee structural safety. Timber structural elements cross-sections are usually characterized by height greater than width to ensure adequate bearing capacity as well as flexural rigidity in the vertical plane. And this may lead to instability phenomena. Indeed, the compression force arising in a part of the element may cause a roto-translational deflection of the cross-section. Thus, a lateral inflection and a rotation around the longitudinal x axis of the element would occur.
To avoid instability phenomena (buckling), EN 1995 (2004) provides an additional condition to satisfy

$$\left(\frac{\sigma_{m,d}}{k_{crit} f_{m,d}}\right)^2 + k_m \left(\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}}\right) \leq 1$$

where
- $k_{c,z}$=1 $\lambda_{rel} < \bar{\lambda}_{rel}$

Tab.5.44 - Combined compression and bending moment verifications at ULS

	N_{max}	M_{max}	k_m	$\sigma_{c,0,d}$	$\sigma_{m,d}$	$f_{c,0,d}$	$f_{m,d}$	
	kN	kNm		N/mm²	N/mm²	N/mm²	N/mm²	
I.	-20,1	0,8	0,7					**0,11<1**
	λ_{rel}	λ_{rel}	$k_{c,z}$	0,003	4,27	11,15	19,23	
	0,33	0,5	1					**0,05<1**
III.	-16,5	2,1	0,7					**0,29<1**
	λ_{rel}	λ_{rel}	$k_{c,z}$	0,002	11,20	15,62	26,92	
	0,40	0,5	1					**0,18<1**

Tension verification

It has to be verified that

$$\sigma_{t,0,d} \leq f_{t,0,d}$$

Tab.5.45 - Parallel tension verification at ULS

	N_{max}	A	$\sigma_{t,0,d}$	$f_{t,0,d}$	
	kN	mm²	N/mm²	N/mm²	
III.	lower chord 29,3	7500	3,91	16,15	**0,24<1**
III.	vertical member 24,1	7500	4,02	16,15	**0,37<1**
III.	inclined member 42,1	7500	7,02	16,15	**0,65<1**

Compression verification

It has to be verified that

$$\sigma_{c,0,d} \leq f_{c,0,d}$$

Tab.5.46 - Parallel compression verification at ULS

	N_{max}	A	$\sigma_{t,0,d}$	$f_{t,0,d}$	
	kN	mm^2	N/mm^2	N/mm^2	
III.	lower chord -16	7500	2,13	15,62	**0,14<1**
I.	vertical member -12,0	7500	2,00	11,15	**0,27<1**
III.	inclined member -29,87	7500	4,97	15,62	**0,47<1**

Serviceability limit states verifications

As previously explained, the final displacement of a timber structural element depends on both the instantaneous and delayed deformation. The former have been evaluated via RFEM structural simulation analysis. In the following table, the maximum displacement are reported:

Tab.5.47 - Deformation verification at SLE

Upper chord

$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$ < l/300	
mm	mm	mm	mm	mm
0,60	-10,1	1,2	-8,40	35,00

$u_{fin,G}$	$u_{fin,Q1}$	$u_{fin,Qi}$	$u_{fin,TOT}$ < l/200	
mm	mm	mm	mm	mm
1,8	-10,2	0,72	-7,68	52,5

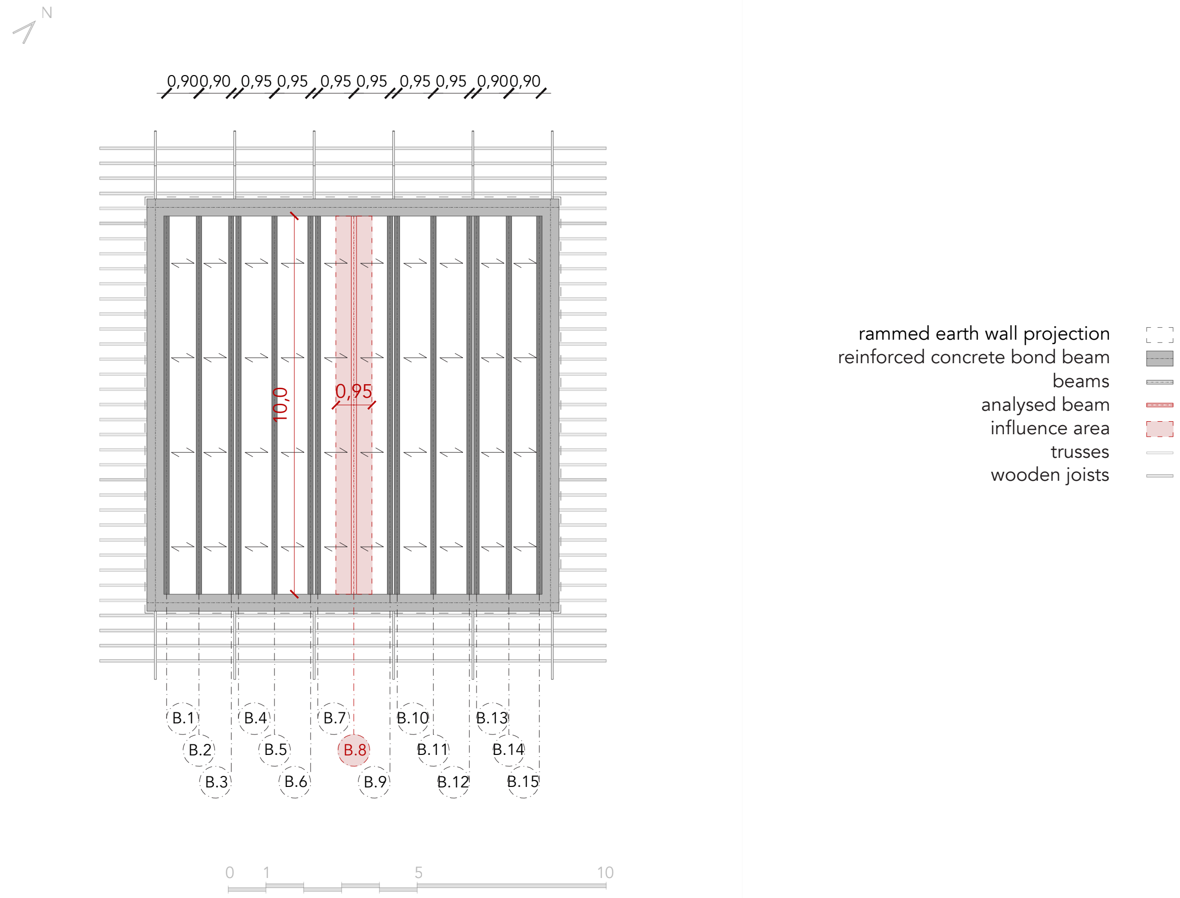
Lower chord

$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$ < l/300	
mm	mm	mm	mm	mm
0,20	-1,3	0,4	-0,70	35,00

$u_{fin,G}$	$u_{fin,Q1}$	$u_{fin,Qi}$	$u_{fin,TOT}$ < l/200	
mm	mm	mm	mm	mm
0,6	-1,3	0,24	-0,46	52,5

5.7.2. TIMBER CEILING BEAM DESIGN

CEILING STRUCTURAL PLAN
1:200



Previous calculations show that wind action is higher on the ceiling extremities, rather than on its central area. However, verifications have been performed on the highlighted beam B.8 due to its bigger area of influence which entails higher stresses and, consequently, greater deflections.

A cross-section of 140x320 mm^2 and metal brackets connecting the beam to the concrete bond beams have been assumed.

GENERAL DATA

span = 10,00 m
s = 0,94m

$A = 4{,}48 \times 10^{-2}\ m^2$
$W = 2{,}38 \times 10^{-3}\ m^3$
$I = 3{,}82 \times 10^{-4}\ m^4$

Loads combinations at ultimate limit state

As for the previous element, three main load combinations have been studied: first of all, we focus on verifying that the assumed cross-section was appropriate for our structural layout. For this purpose, only the permanent loads and the imposed load on roof have been considered and maximized. Then, we analysed the resistance of the structure introducing also the wind actions, respectively, at 0° in the second loads combination, at 180° in the third loads combination. These latter combinations were splitted in two subcases to consider both the positive and negative values of external pressure coefficients in zone I, as recommended by EN 1991.1.4 (2005).

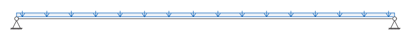
Since external openings are supposed to be closed during severe windstorms, the condition with doors and windows open has been considered as an accidental design situation as recommended by EN 1991.1.4 (2005).

I. combination

Tab.5.48 - Load combination

Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	ψ_0	ψ_1	ψ_2
G_1	0,27	1,35	$Q_{CAT.H}$	0,38		1,50	0	0	0
G_2	0,53			0°	180°				
			$Q_{WIND,G}$	-1,69	-1,84		0,6	0,2	0
			$Q_{WIND,H}$	-0,99	-1,07				
			$Q_{WIND,I-}$	-0,28	-0,31				
			$Q_{WIND,I+}$	0,28	0,31				
			Q_{pi}	0,07	0,08		1	1	1

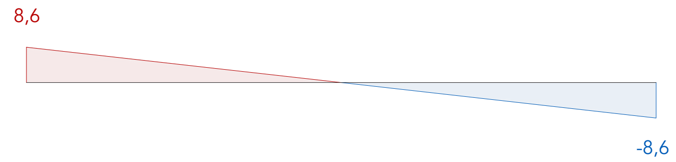
$F_d = \gamma_G(G_1 + G_2) + \gamma_Q Q_{CAT.H}$ $= 1,65$ kN/m



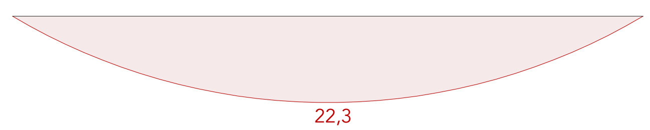
Compression (-)

Tension (+)

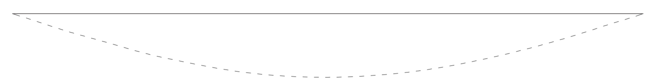
Shear forces diagram (kN)

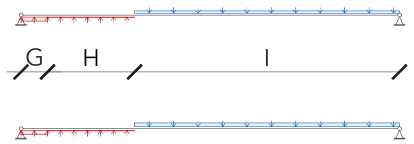


Bending moments diagram (kNm)



Deformation diagram





II. combination

Tab.5.49 - Load combination

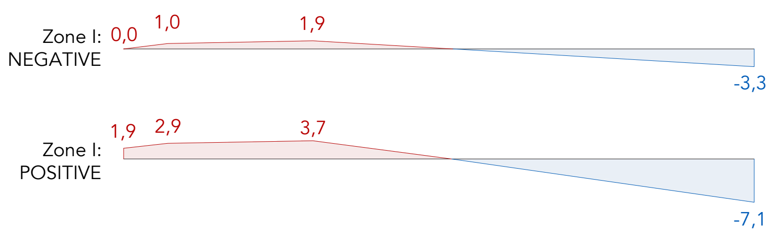
Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	Ψ_0	Ψ_1	Ψ_2
G_1	0,18	1,35	$Q_{CAT.H}$	0,38		1,50	0	0	0
G_2	0,65			0°	180°				
			$Q_{WIND,G}$	-1,69	-1,84		0,6	0,2	0
			$Q_{WIND,H}$	-0,99	-1,07				
			$Q_{WIND,I-}$	-0,28	-0,31				
			$Q_{WIND,I+}$	0,28	0,31				
			Q_{pi}	0,07	0,08		1	1	1

$F_{d,WIND\ G}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,G} +(\Psi_0 Q_{CAT.H})]$	= -1,38 kN/m
$F_{d,WIND\ H}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,H} +(\Psi_0 Q_{CAT.H})]$	= -0,32 kN/m
$F_{d,WIND\ I-}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,I-} +(\Psi_0 Q_{CAT.H})]$	= 0,74 kN/m
$F_{d,WIND\ I+}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,I+} +(\Psi_0 Q_{CAT.H})]$	= 1,58 kN/m

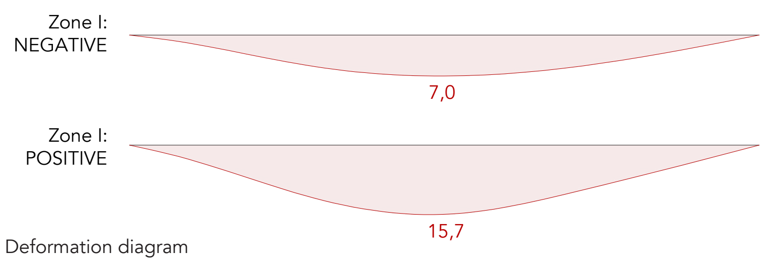
Compression (-)

Tension (+)

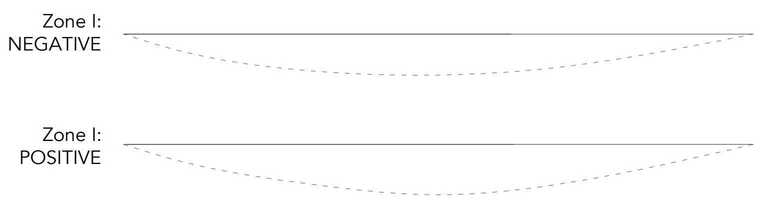
Shear forces diagram (kN)



Bending moments diagram (kNm)



Deformation diagram

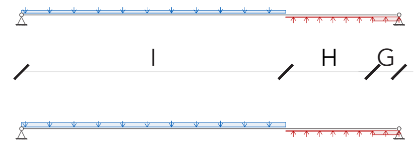


III. combination

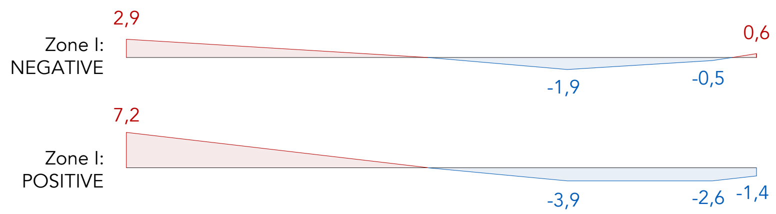
Tab.5.50 - Load combination

Permanent loads			Variable loads						
	kN/m	γ_G		kN/m		γ_Q	Ψ_0	Ψ_1	Ψ_2
G_1	0,18	1,35	$Q_{CAT.H}$	0,38		1,50	0	0	0
G_2	0,65			0°	180°				
			$Q_{WIND,G}$	-1,69	-1,84		0,6	0,2	0
			$Q_{WIND,H}$	-0,99	-1,07				
			$Q_{WIND,I-}$	-0,28	-0,31				
			$Q_{WIND,I+}$	0,28	0,31				
			Q_{pi}	0,07	0,08		1	1	1

$F_{d,WIND\ G}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,G} +(\Psi_0 Q_{CAT.H})]$	= -1,59 kN/m
$F_{d,WIND\ H}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,H} +(\Psi_0 Q_{CAT.H})]$	= -0,44 kN/m
$F_{d,WIND\ I-}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,I-} +(\Psi_0 Q_{CAT.H})]$	= 0,71 kN/m
$F_{d,WIND\ I+}$ =	$\gamma_G(G_1 + G_2)+ \Psi_1 Q_{pi}+ \gamma_Q [Q_{WIND,I+} +(\Psi_0 Q_{CAT.H})]$	= 1,65 kN/m



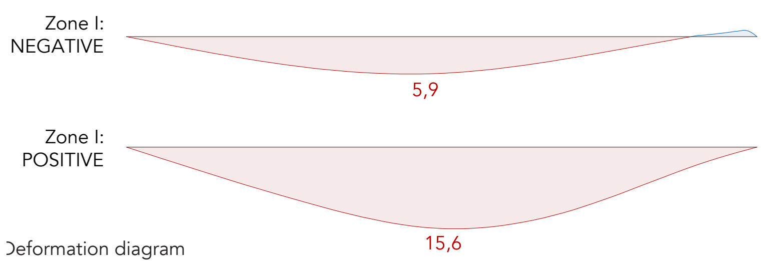
Shear forces diagram (kN)



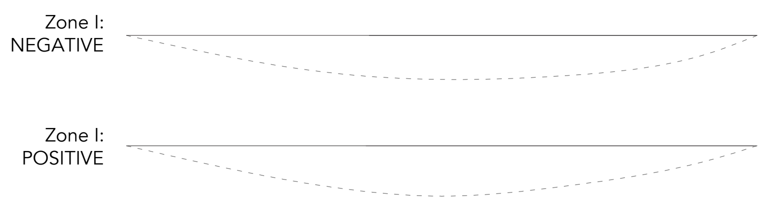
Compression (-)

Tension (+)

Bending moments diagram (kNm)



Deformation diagram



Tab.5.51 - Design values (γ_m = 1,3)

Properties	$k_{mod,I}$	$k_{mod,II}$
(in N/mm²)	**0,5**	**0,7**
$f_{m,d}$	19,23	26,92
$f_{t,0,d}$	11,54	16,15
$f_{t,90,d}$	0,23	0,32
$f_{c,0,d}$	11,15	15,62
$f_{c,90,d}$	3,58	5,01
$f_{v,d}$	1,54	2,15

* $k_{mod,I}$ = 0,5 for permanent actions
$k_{mod,II}$ = 0,7 for short-term actions

Ultimate limit states verifications

As for the typical classroom analysis, the first load combination is the most severe: not involving wind loads, both bending moment and shear reach their maximum values since there are no loads hindering the compressive loads acting on the beam.

Bending verification

It has to be verified that

$$M_{dh}{}^{*} \leq M_{ch}$$

Tab.5.52 - Bending verification at ULS

	M_{max}	W	$\sigma_{m,d}$	$f_{m,d}$	
	kN	mm³	N/mm²	N/mm²	
I.	22,3	2389333	9,33	19,23	**0,48<1**
III.	15,6	2389333	6,52	26,92	**0,24<1**

Shear verification

It has to be verified that

$$M_{dh}{}^{*} \leq M_{ch}$$

Tab.5.53 - Shear verification at ULS

	V_{max}	A	τ_d	$f_{v,d}$	
	kN	mm²	N/mm²	N/mm²	
I.	8,6	7500	0,28	1,54	**0,18<1**
III.	7,2	7500	0,24	2,15	**0,11<1**

Serviceability limit states verifications

As previously explained, the final displacement of a timber structural element depends on both the instantaneous and delayed deformation. The former have been evaluated via RFEM structural simulation analysis. In the following table, the maximum displacement are reported:

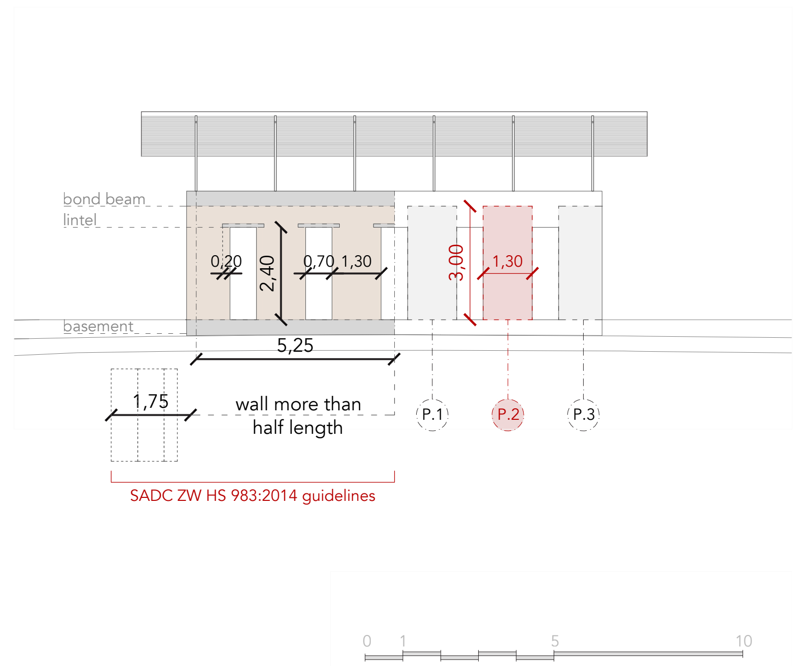
Tab.5.54 - Deformation verification at SLE

$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$ < l/300	
mm	mm	mm	mm	mm
5,60	-5,8	3,9	3,70	33,33
$u_{fin,G}$	$u_{fin,Q1}$	$u_{fin,Qi}$	$u_{fin,TOT}$ < l/200	
mm	mm	mm	mm	mm
16,8	-5,8	0	11	50

5.7.3. RAMMED EARTH WALL DESIGN

NORTH WEST ELEVATION
1:200

rammed earth wall
reinforced concrete
wall sub-panel
analysed wall sub-panel



Walls with openings are considered to form sub-panels either side of the opening. The edges of the sub-panels adjacent to the opening are regarded as being unsupported (ie no lateral support or rotational restraint) with the remaining edges being supported. Generally, to simplify calculations, the openings are assumed to extend for the full height of the wall. The pressure on the opening is considered to be fully transferred to the edge of the two adjoining masonry sub-panels.

The north-west elevation above reported shows the most stressed sub-panel on which compressive and flexural capacity verifications have been performed assuming the sub-panel as supported at top and bottom.

Moreover, according to the guidelines of SADC ZW HS 983:2014, the wall has been assumed 50 cm thick to respect the provided slenderness ratio (1:8) and to guarantee structural safety.

Verifications have been carried out with the hypothesis of unreinforced masonry since the project site is located in an area of very low earthquake risk.

Ultimate limit states verifications

Generally, walls shall be designed to resist compressive forces, with or without simultaneously acting bending moments, and to withstand short-term transient nature forces such as wind and earthquake loads.

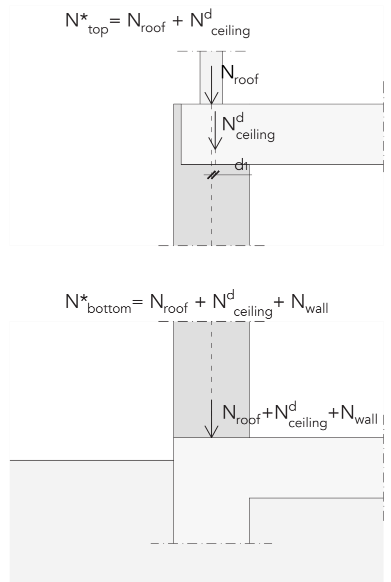
Uniaxial bending and compression

According to NZS 4297:1998, a rammed earth member shall be design such that the following relationship is satisfied:

$$N^* \leq k\Phi N_o$$

where:

- $N_0 = f_e A_b$
- k is the reduction factor depending on slenderness and eccentricity to thickness ratio



Tab.5.55 - Uniaxial bending and compression verifications at ULS

S_r	e/t_w	k	N_0 (N/mm²)		N* (kN)	$k\Phi N_0$ (kN)	
6,00	0,11	0,6	0,5	**top**	32,14	90	**0,36<1**
				bottom	76,83		**0,85<1**

Horizontal bending from transient out-of-plane forces

According to NZS 4297:1998, a wall shall be proportioned so that the following relationship is satisfied under each combination of simultaneously acting design horizontal bending moment (M^*_{dh}):

$$M_{dh}^* \leq M_{ch}$$

where

- $M_{ch} = \Phi f_{et} Z_u$
- $Z_u = \dfrac{bt^2}{6}$

Tab.5.56 - Horizontal bending verifications at ULS

f_{et} (N/mm²)	Φ	Z_u (mm³)	M_{dh}^* (kNm*m)	M_{ch} (kNm*m)	
0,1	0,8	41666667	2,06	3,33	**0,62<1**

Shear verification

Walls aligned in parallel to wind direction contribute to the overall stability of the structure against horizontal lateral forces transmitting these latter forces to the ground by in-plane shear.

According to NZS 4297:1998, the design of an unreinforced earth wall subject to shear forces, with or without simultaneous compressive forces acting across the shear plane, shall be such that the following relationship is satisfied under each combination of simultaneously acting design shear force, V *, and (minimum) compressive stress (f_d) acting at the cross section under consideration:

$$V^* \leq \Phi[\, f_{es}A_b + k_v f_d A_b]$$

Tab.5.57 - *Shear verifications at ULS*

f_{es}	A_b	k_v	Φ	V*	$\Phi\,(f_{es}A_b+k_v f_d A_b)$
N/mm²	m²	m²		kN	kNm*m
0,08	2,20	0	0,7	2,69	138,44

Serviceability limit states verification

According to NZS 4297:1998, to satisfy the verification at serviceability limit states it is sufficient to verify that the assumed thickness of the walls, where they are not supported or attached to partitions or other construction, is at least equal to the minimum thickness provided. For our project, the minimum value corresponds to

Tab.5.58 - *Minimum thickness*

Supports	th_{min}	th_{wall}
Simply supported	h/18 = 16,67 cm	50 cm
One end continuous	h/21	
Both end continuous	h/22	
Cantilever	h/8	

Source: NZS 4297:1998

Hence, the provided thickness satisfies the limit established by NZS 4297:1998 and furthermore it guarantees a contained deflection even when cyclones occur.

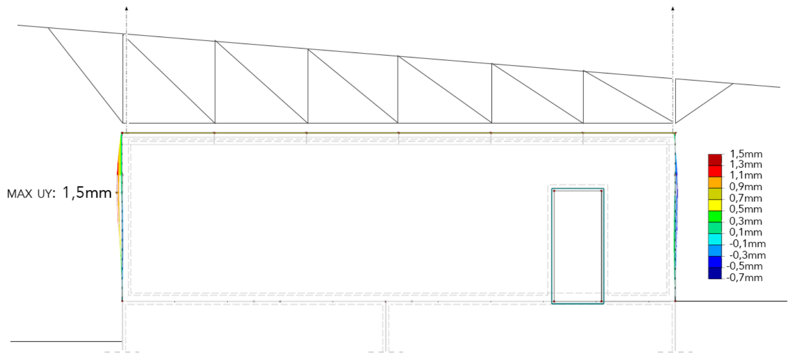
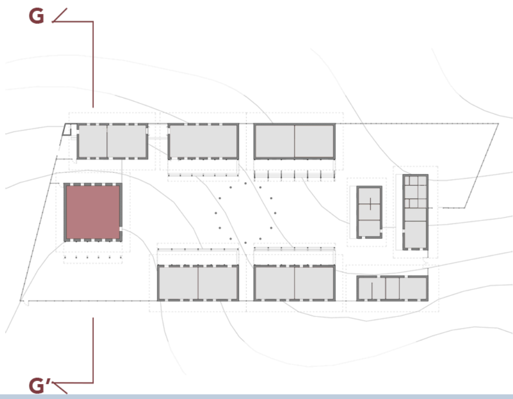
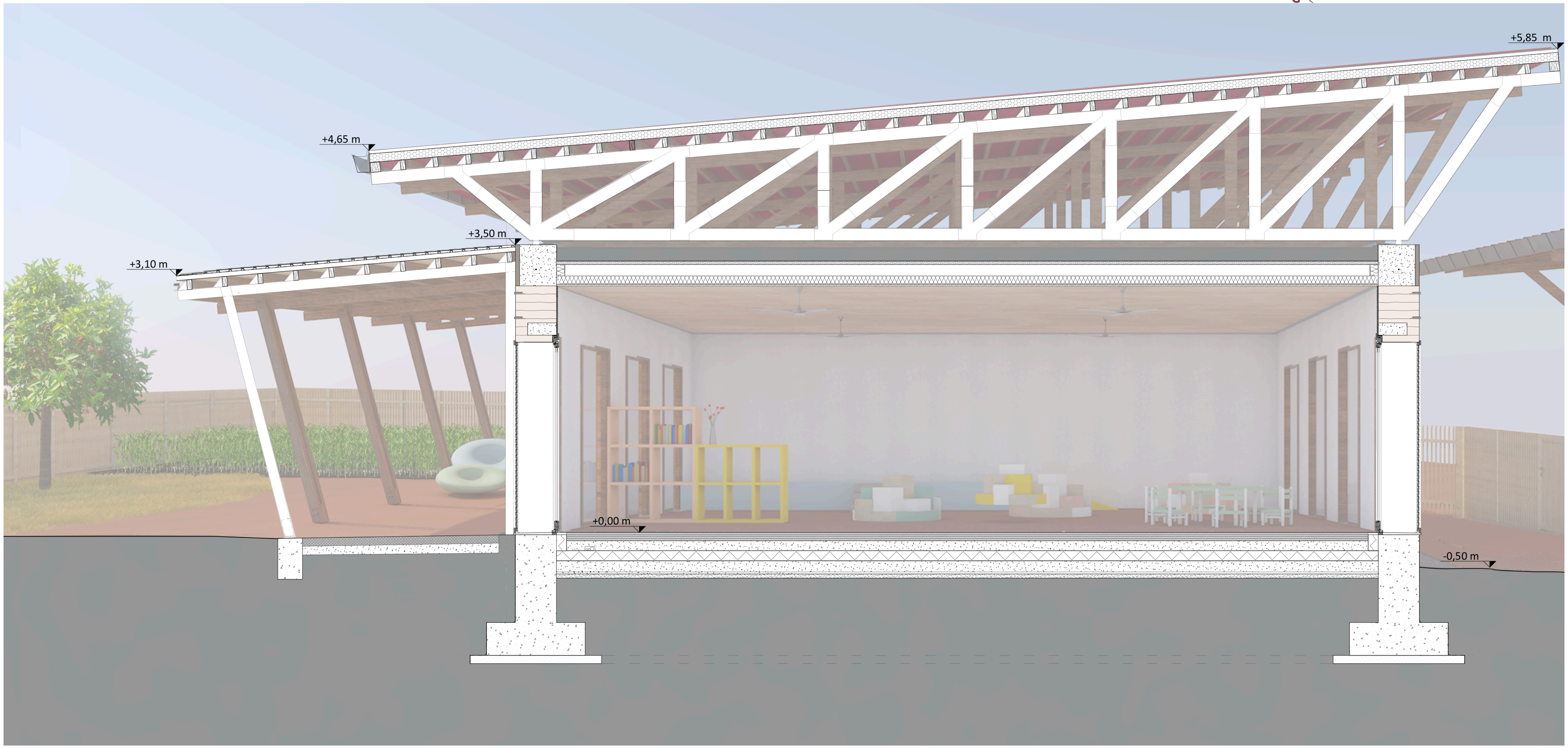


Fig.5.11 -Maximum wall displacement: analysis performed with RFEM.

MULTIFUNCTIONAL SPACE: SECTION GG'
1:50





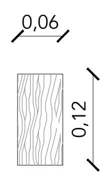
5.8. HORIZONTAL ENCLOSURES

In the previous paragraphs, attention has been focused on the structural elements of the two buildings under study which presented particular differences in terms of dimensions and extension. In the following paragraph, instead, structural verifications and considerations related to horizontal enclosures are presented, for which it is possible to make a common and valid discussion for both the buildings.

5.8.1 ROOF

In both cases, the roof is made up of sandwich panels on timber joists fixed to the truss beams, and with a triangular block of wood in front of it: an economical but efficient solution used in Mozambique to hinder wind suction and therefore prevent the eradication of the joist. Clearly, the extension of the joists in the two buildings is different, but given the same cross-section, spacing and loads (as seen in paragraph 4 of the current chapter), the maximum values of bending moment and shear coincide, while the final deformations are slightly different.

GENERAL DATA



overall length = 18,00 m
s = 0,40 m

A = $7{,}2 \times 10^{-3}$ m^2
W = $1{,}44 \times 10^{-5}$ m^3
I = $8{,}64 \times 10^{-6}$ m^4

Tab.5.59 - *Bending and shear verification verifcation at ULS*

M_{max}	W	$\sigma_{m,d}$	$f_{m,d}$	
kN	mm^3	N/mm^2	N/mm^2	
3,7	144000	25,60	26,92	**0,94<1**
V_{max}	A	τ_d	$f_{v,d}$	
kN	mm^2	N/mm^2	N/mm^2	
4,9	7200	1,02	1,54	**0,66<1**

Tab.5.60 - *Deformation verification at SLE*

$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$ < l/300	
mm	mm	mm	mm	mm
1,20	-30,3	1,8	-27,30	60,00
$u_{fin,G}$	$u_{fin,Q1}$	$u_{fin,Qi}$	$u_{fin,TOT}$ < l/200	
mm	mm	mm	mm	mm
3,6	-30,3	0	-26,7	90

Concerning the sandwich panel, the Eurocinque panel with a thickness of 100 mm produced by Lattonedil s.p.a. has been used. As seen in previous paragraphs, the acting loads reach a maximum value of 2933,22 N/m^2 (approximately 300 Kg/m^2).
According to the product sheet, the panel is able to effectively withstand this load: in fact, it is verified, at service limit state, which is the most severe condition, for spans up to 2,50 m, much greater than the current ones.
However, to apply the verifications carried out, the company recommends a number of fasteners equal to at least 3 per m^2.

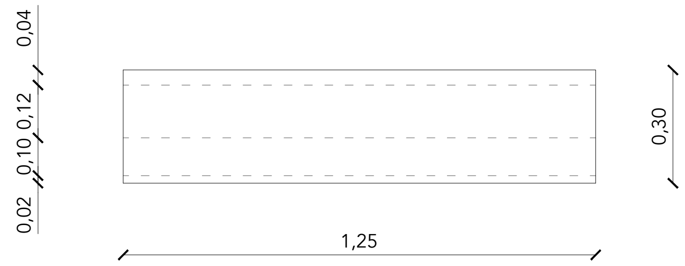
Tab.5.61 - *Eurocinque panel static properties*

th.	distances between supports					
mm	ml					
	1,5	**2,00**	**2,50**	**3,00**	**3,50**	**4,00**
30	276	194	111	48	42	38
40	346	254	159	83	59	55
50	414	313	207	119	87	72
60	482	373	255	156	113	88
80	620	497	352	229	181	121
100	673	575	403	295	168	144
	U=Kg/m² uniformly distributed					
	SLS limit: u < l/200					

Source: www.lattonedil.it

5.8.2 CEILING

The ceiling is in both cases made up of two osb panels enclosing an air cavity, insulated with a straw layer. Since the beams on which it rests have a fairly large interaxle spacing (2,30 m), it is recommended to verify that the structure does not deform under its own weight and under any other imposed loads. To do this, the ceiling was assumed as follow.

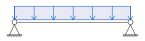


GENERAL DATA

span = 6,00 m
s = 2,20m

A = $3{,}75 \times 10^{-1}$ m^2
W = $1{,}87 \times 10^{-2}$ m^3

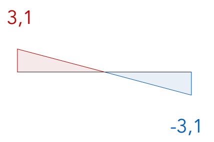
As previously seen in the chapter, the most severe load combination is the one that does not take into account the wind action: the maximum moment and shear are obtained with a load combination entailing permanent loads and imposed load for not accessible roof except for normal maintenance and repair.



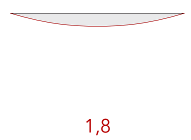
Tab.5.62 - *Load combination*

Permanent loads			Variable loads					
	kN/m	γ_G		kN/m	γ_Q	ψ_0	ψ_1	ψ_2
G_1	2,02	1,35	$Q_{CAT.H}$	4,56	1,50	0	0	0
G_2	1,56							
	F_d =		$\gamma_G(G_1 + G_2) + \gamma_Q Q_{CAT.H}$			**= 11,68 kN/m**		

Shear forces diagram (kN)



Bending moments diagram (kNm)



Deformation diagram

Ultimate limit states verifications required for bending that

$$M_{dh}^* \leq M_{ch}$$

Tab.5.63 - *Bending verification at ULS*

	M_{max}	W	$\sigma_{m,d}$	$f_{m,d}$	
	kN	mm³	N/mm²	N/mm²	
I.	1,75	18750000	0,09	8,83	**0,01<1**

For shear that

$$M_{dh}^* \leq M_{ch}$$

Tab.5.64 - *Shear verification at ULS*

	V_{max}	A	τ_d	$f_{v,d}$	
	kN	mm²	N/mm²	N/mm²	
I.	3,06	375000	0,01	3,66	**0,00<1**

Serviceability limit states verifications have been evaluated calculating both instantaneous and delayed deformation. In the following table, the maximum displacement are reported.

Tab.5.65 - *Deformation verification at SLE*

$u_{inst,G}$	$u_{inst,Q1}$	$u_{inst,Qi}$	$u_{inst,TOT}$ **< l/300**	
mm	mm	mm	mm	mm
5,36	6,81	1,44	7,68	21,00
$u_{fin,G}$	**$u_{fin,Q1}$**	**$u_{fin,Qi}$**	**$u_{fin,TOT}$ < l/200**	
mm	mm	mm	mm	mm
16,08	6,81	0,87	16,08	31,50

Given these consideration, Dataholz osb panels has been used. According to the product sheet, the panels are able to effectively withstand the acting loads.

Tab.5.66 - *Dataholz osb panel static properties*

th.	ρ	$f_{m,K}$	$f_{v,K}$
mm	Kg/m³	N/mm²	N/mm²
> 18-25	550	14,80	6,80

Source: www.galloppinilegnami.it

5.9. CONNECTIONS

As seen in previous paragraphs, the reinforced concrete bond beam has the purpose of connecting the load-bearing rammed earth walls with each other, as well as receiving and distributing the loads coming from the roof and the horizontal enclosure. In both cases, metal fasteners are used to connect the timber elements to the reinforced concrete bond beams.

5.9.1. ROOF TRUSS CONNECTIONS

Fig.5.12 - U-shaped metal bracket. Source: AVSI documentation.

According to AVSI documentation, extruded metal brackets have been used for truss members connection. To fasten the roof structure to the reinforced concrete bond beam, U-shaped terminations have been used for the timber-to-concrete connection. The connection system is embedded into the reinforced concrete bond beam and the anchoring is further strengthened by a steel bar.
The load-bearing capacity of the connection system has been verified according to EN 1995.1.1 (2004). The following calculations concern the roof of a typical classroom.

The characteristic load-carrying capacity for nails, bolts, dowels and screws per shear plane per fastener has been taken as the minimum value calculated from the expressions referring to thin-steel plate as outer members of a double shear connection:

$$F_{v,Rk} = min\begin{cases} 0{,}5 f_{h,2,k} t_2 d \\ 1{,}15\sqrt{2M_{y,Rk} f_{h,2,k} d} + \dfrac{F_{ax,Rk}}{4} \end{cases}$$

- $F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener
- $f_{h,k}$ is the characteristic embedment strength in the timber member
- t_2 is the thickness of the timber middle member
- d is the fastener diameter
- $M_{y,Rk}$ is the characteristic fastener yield moment
- $F_{ax,Rk}$ is the characteristic withdrawal capacity of the fastener. For dowels, this contribution is 0%.

Dealing with bolted connection, 8.5.1 apply. The characteristic value for the yield moment should be calculated as

$$M_{y,Rk} = 0{,}3 f_{u,k} d^{2,6}$$

where:
- $f_{u,k}$ is the characteristic tensile strength (800N/mm^2)
- d is the dowel diameter

To evaluate the characteristic embedment strength the following expression should be used:

$$f_{h,2,k} = 0{,}082(1 - 0{,}01d)\rho_k$$

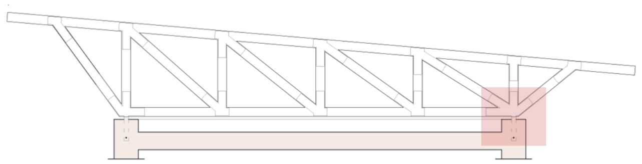
The characteristic resistance $F_{v,Rk}$ evaluated refers to a single bolt. To consider the resistance given by the whole connecting system, this value has to be multiplied by the number of bolts used which is equal to 2. Finally, the design resistance value is evaluated considering the environmental parameters.

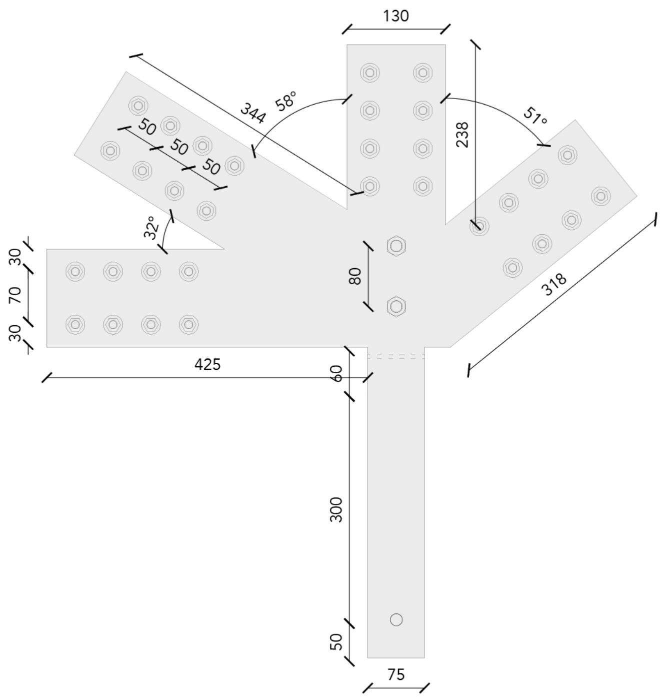
Tab.5.67 - Summary of the parameters calculated

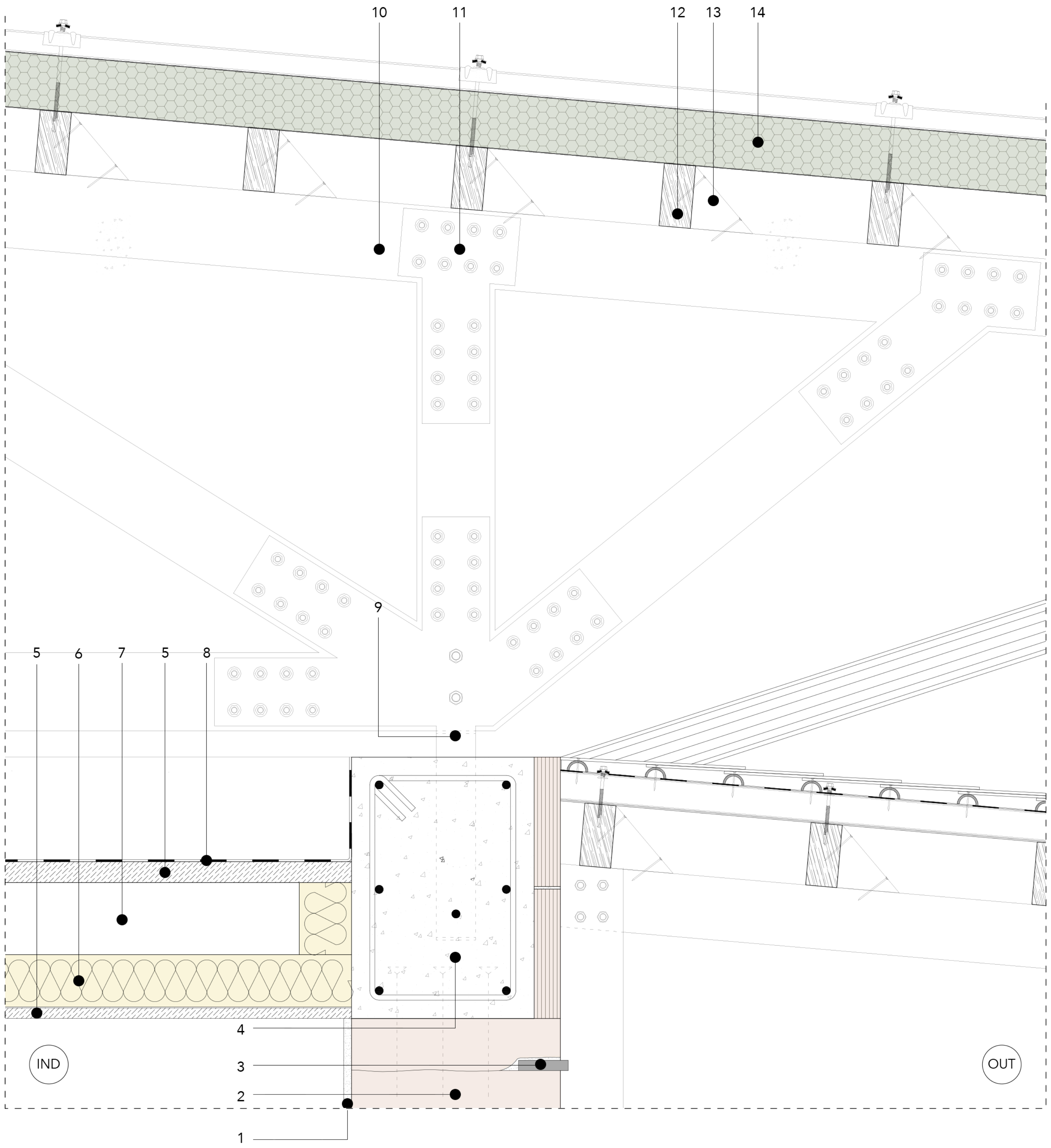
$f_{h,2,k}$	$M_{y,Rk}$	$F_{v,Rk\ (single\text{-}bolt)}$	$F_{v,Rk\ (2\text{-}bolts)}$	R_d
Mpa	Nmm	kN	kN	kN
155,14	229 162,81	54,30	108,60	83,53

TECHNICAL DRAWINGS - ROOF FIXING

1:10







1 internal finishing layer: gypsum plaster.
th.1,5 cm

2 rammed earth loadbearing wall.
th. 40 cm

3 protruding ceramic tile embedded in clay mortar for erosion check.
th. 2 cm

4 bond beam in reinforced concrete hidden behind comrpessed heart block (CEB)
th. 40 cm

5 OSB panel.
th. 2 cm

6 thermal insulation layer of straw panels "Stiferite GT".
ρ 340 Kg/m³; λ 0,09 W/mK; th. 10 cm

7 air cavity.
th. 13,8 cm

8 breathable membrane.
th. 0,4 cm

9 truss beam anchoring system:
U-shaped metal bracket.
th. 0,5 cm

10 roof primary structure: timber truss beam.
cross section: 5x15 cm².

11 truss members connecting system:
concealed metal bracket.
th. 0,5 cm

12 secondary structure: timber joist.
cross section: 6x12 cm².

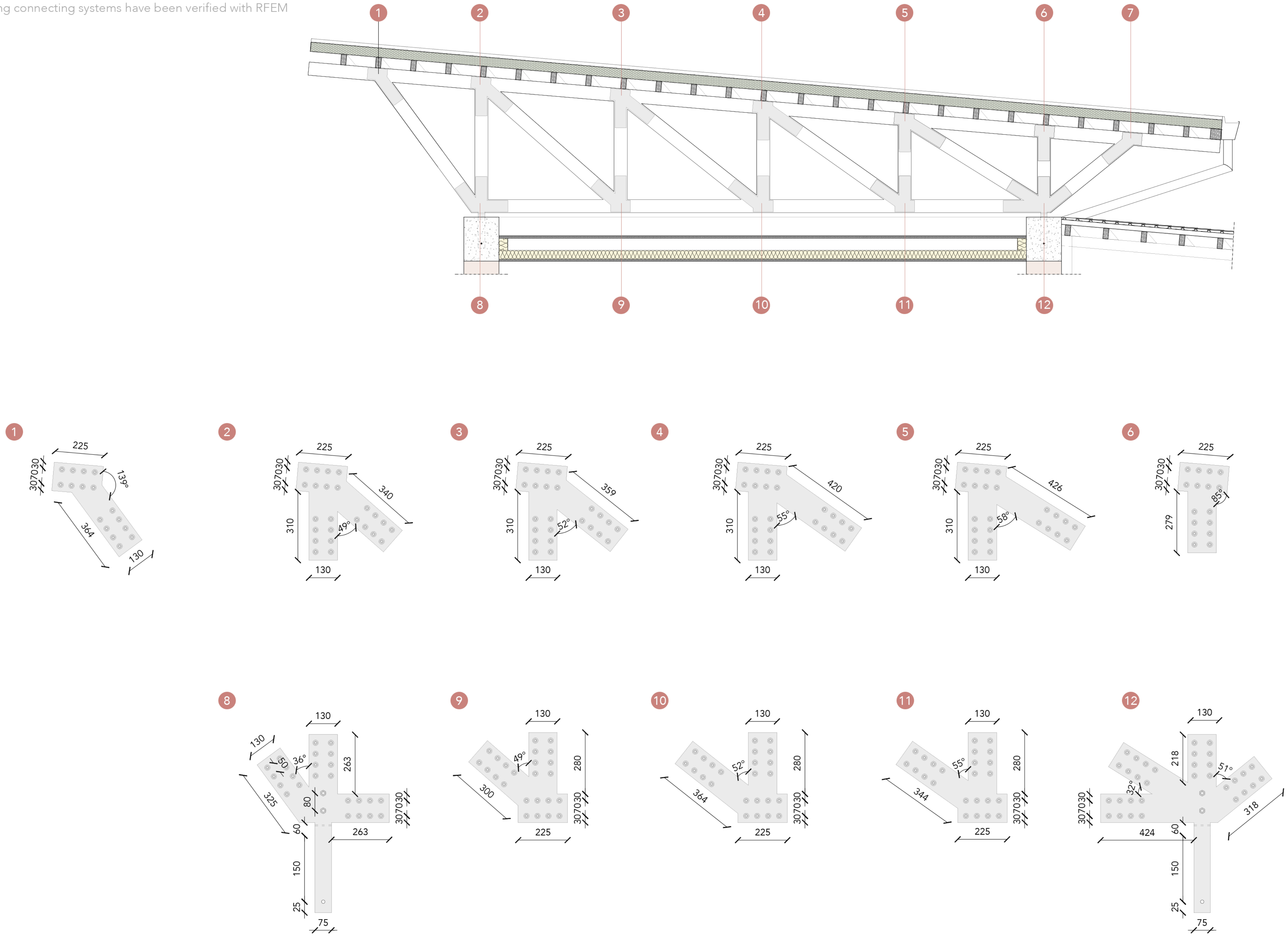
13 timber element to improve joist resistance to cyclones

14 sandwhich panel with double metal lining and polyurethane foam insulating core
ρ 40 Kg/m³; λ 0,22 W/mK; th. 10 cm

TECHNICAL DRAWINGS - TRUSS MEMBERS CONNECTIONS

roof section 1:50; metal bracket 1:20

*the following connecting systems have been verified with RFEM



5.9.2. CEILING BEAM CONNECTIONS

Fig.5.13 - Alumidi concealed bracket. Source: https://www.rothoblaas.com

The Alumidi concealed bracket, produced by Rothoblaas, has been used for the timber-to-concrete connection of the ceiling structure. It is produced by extrusion, therefore without weld.

The load-bearing capacity of the connection system has been verified according to EN 1995.1.1 (2004). The following calculations concern the horizontal enclosure of a typical classroom.

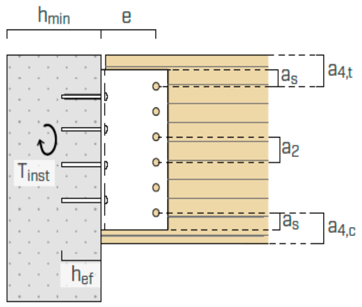


Fig.5.14 -Alumidi minimum distance. Source: https://www.rothoblaas.com

Tab.5.68 - *Installation: minimum distances*

secondary beam - timber		smooth dowel (STA Ø12)
dowel-dowel	a_2	≥36mm
dowel-top of the beam	$a_{4,t}$	≥48mm
dowel-bottom of the beam	$a_{4,c}$	≥36mm
dowel-bracket edge	a_s	≥16mm
dowel-main beam	e	86mm
main beam - concrete		**screw anchor (SKR-E Ø10)**
		mm
minimum support thickness	h_{min}	110mm
concrete hole diameter	d_0	8mm
tightening torque	T_{inst}	50Nm

The characteristic load-carrying capacity for nails, bolts, dowels and screws per shear plane per fastener has been taken as the minimum value calculated from the expressions referring to steel plate of any thickness as the central member of a double shear connection:

$$F_{v,Rk} = min \begin{cases} f_{h,k} t_1 d \\ f_{h,k} t_1 d \left[\sqrt{2 + \dfrac{4 M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right] + \dfrac{F_{ax,Rk}}{4} \\ 2{,}3 \sqrt{M_{y,Rk} f_{h,k} d} + \dfrac{F_{ax,Rk}}{4} \end{cases}$$

where:

- $F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener
- $f_{h,k}$ is the characteristic embedment strength in the timber member
- t_1 is the smaller of the thickness of the timber side member or the penetration depth
- d is the fastener diameter
- $M_{y,Rk}$ is the characteristic fastener yield moment
- $F_{ax,Rk}$ is the characteristic withdrawal capacity of the fastener. For dowels, this contribution is 0%.

For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply. The characteristic value for the yield moment should be calculated as

$$M_{y,Rk} = 0{,}3 f_{u,k} d^{2,6}$$

where:

- $f_{u,k}$ is the characteristic tensile strength (800N/mm^2)
- d is the dowel diameter

To evaluate the characteristic embedment strength the following expression should be used:

$$f_{h,0,k} = 0{,}082(1 - 0{,}01d)\rho_k$$

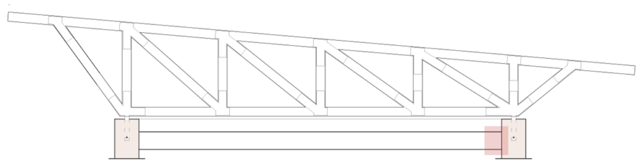
The characteristic resistance $F_{v,Rk}$ evaluated refers to a single dowel. To consider the resistance given by the whole connecting system, this value has to be multiplied by the number of dowels used which is equal to 3. Finally, the design resistance value is evaluated considering the environmental parameters.

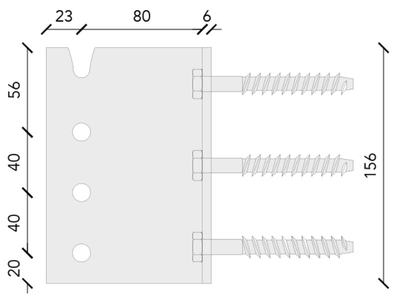
Tab.5.69 - *Summary of the parameters calculated*

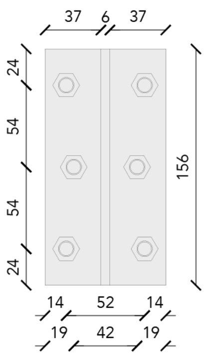
$f_{h,0,k}$	$M_{y,Rk}$	$F_{v,Rk\ (single\text{-}dowel)}$	$F_{v,Rk\ (3\text{-}dowels)}$	R_d
Mpa	Nmm	kN	kN	kN
158,75	153 490,85	39,33	117,98	90,76

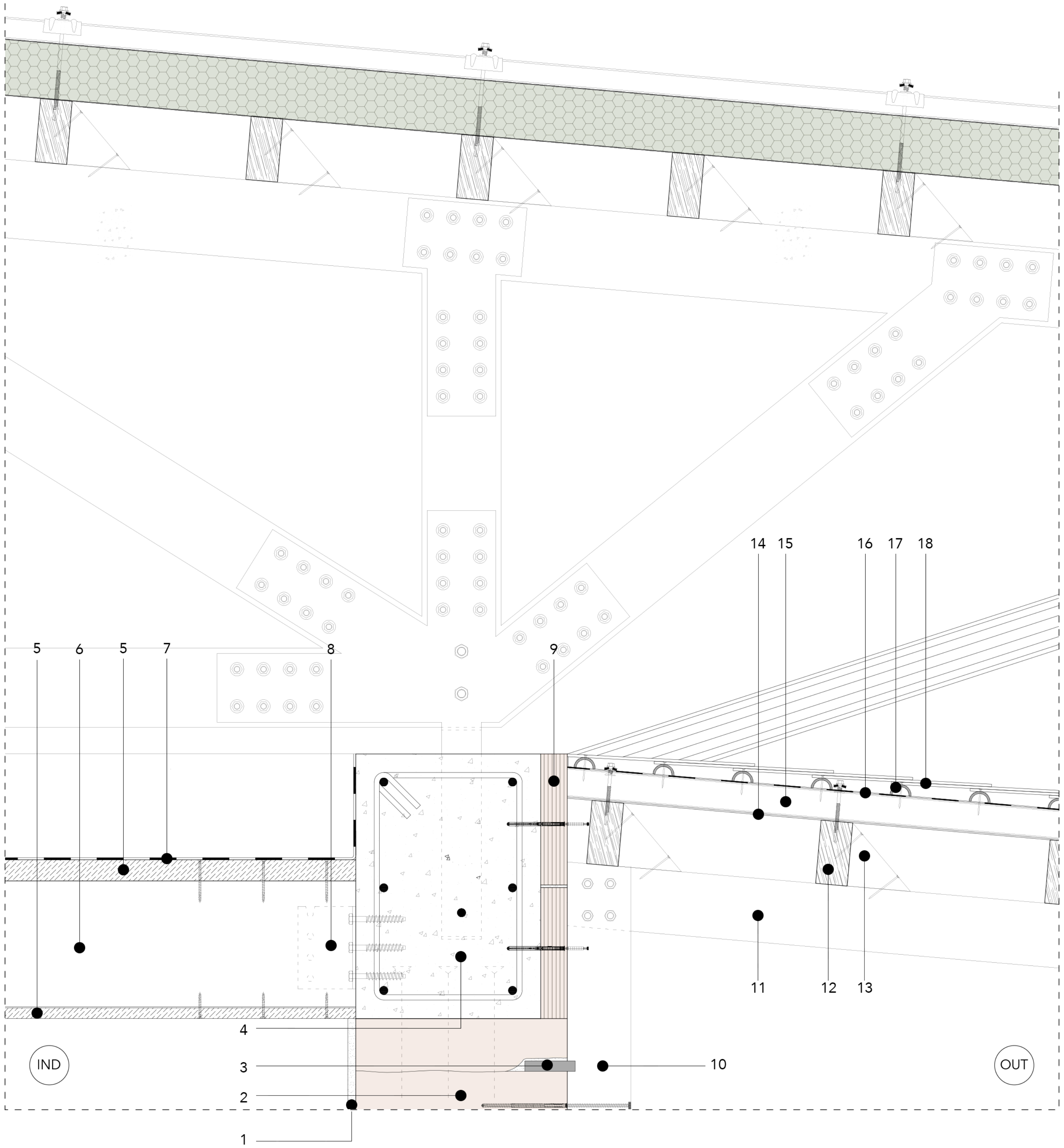
TECHNICAL DRAWINGS - CEILING FIXING

1:10









1 internal finishing layer: gypsum plaster.
th.1,5 cm

2 rammed earth loadbearing wall.
th. 40 cm

3 protruding ceramic tile embedded in clay mortar for erosion check.
th. 2 cm

4 bond beam in reinforced concrete
th. 35 cm

5 OSB panel.
th. 2 cm

6 ceiling primary structure: timber beam.
cross section: 12x24 cm^2

7 breathable membrane.
th. 0,4 cm

8 ceiling beam anchoring system: concealed metal bracket.
th. 0,5 cm

9 comrpessed heart block (CEB.
th. 5 cm

10 veranda structure: double section timber pillar
cross section: 6x12 cm^2

11 veranda structure: primary beam
cross section: 6x12 cm^2

12 secondary structure: timber joist.
cross section: 6x12 cm^2.

13 timber element to improve joist resistance to cyclones

14 layer of interlocking split bamboo stems overlapped to a woven bamboo net
th. 0,2 cm

15 supporting frame of bamboo
th. 6 cm

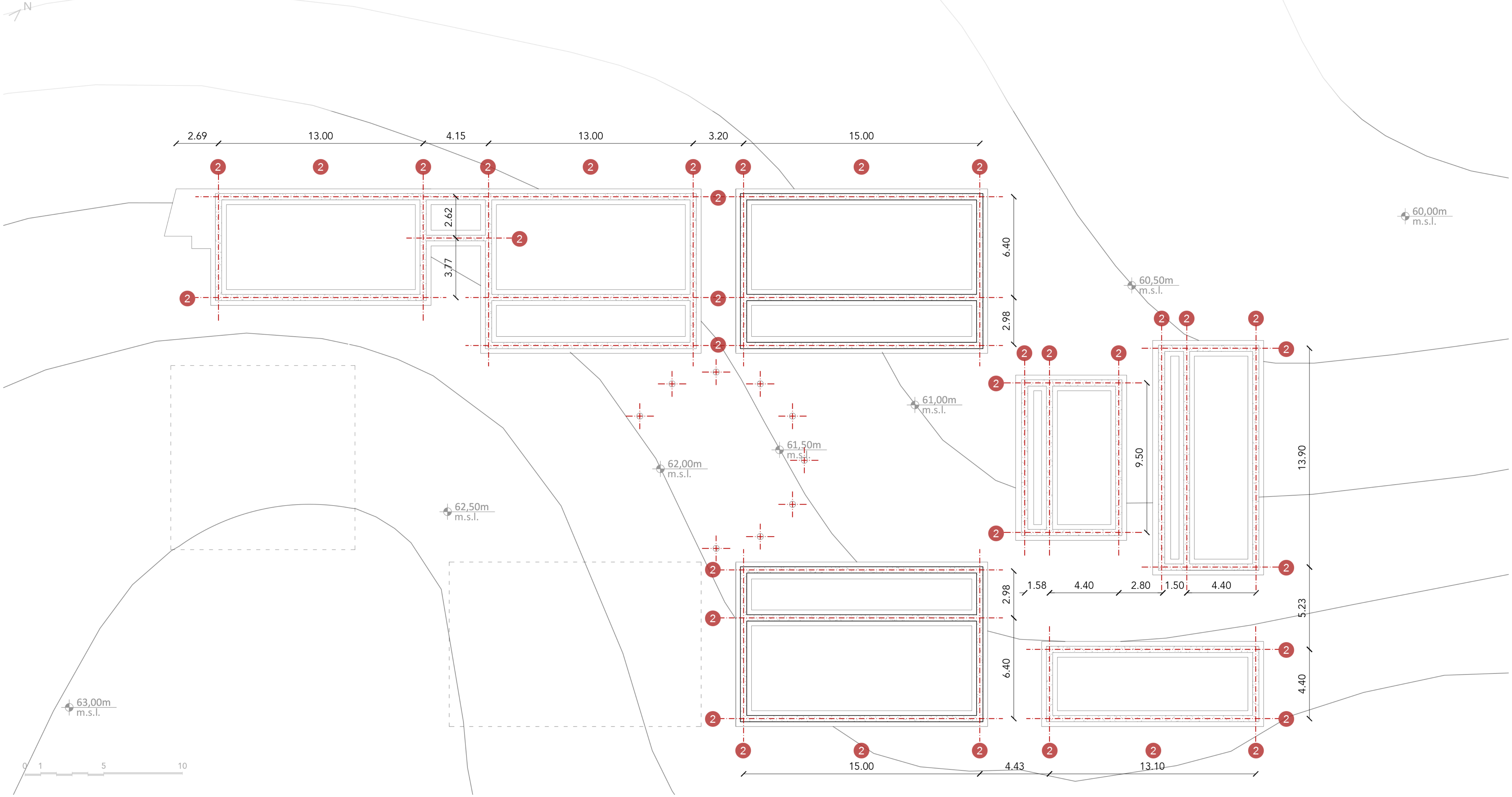
16 waterproof layer of bituminous membrane overalapped to interlocking split bamboo stems
th. 0,3 cm

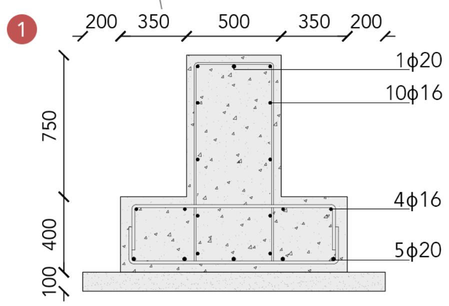
17 supporting layer of bamboo battens
th. 0,5 cm

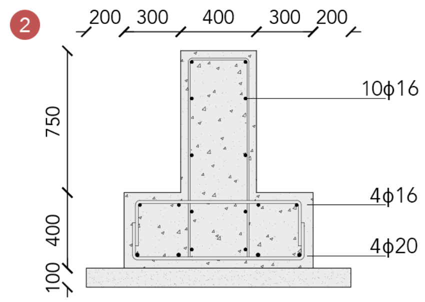
18 external finishing layer of bamboo shingles
th. 0,5 cm

FOUNDATION PLAN: LEVEL -I

1:250







FOUNDATION PLAN: LEVEL I

1:250