School of Building Engineering – Architecture
Master of Science in Architectural Engineering

# Timber in the city

A mid-rise, mixed-use complex in Red Hook, Brooklyn, New York

**Supervisor:**
Prof. Marco Imperadori
**Co supervisor:**
Prof. Gabriele Masera
**Written by:**
Galina Ivanova 781019
Yanina Morozova 782942
Tatjana Somova 780521

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**ABSTRACT (ENGLISH)**

The design challenge was to envision a wood mid-rise, mixed-use complex with housing units, a job training/educational facility, a centre for innovative manufacturing of wood technology and a distribution centre. The project site is in Red Hook which is geographically cut off from much of Brooklyn yet still being - increasingly vibrant. In order to regenerate an urban manufacturing sector and meet the housing needs of New York City, a concept arose to design a place for the creation of creative vocational opportunities embracing new wood technology.

First of all the design was focused on the environmental sustainability of the project, in order to try and minimize its impact on the planet. For this reason numerous passive and active strategies were implemented within the project.

During the design process numerous methods of building systems, with focus on innovation in wood design on a real site had to be interpreted, tweaked, and implemented.

Timber is an ideal green building material as it is well suited for a broad range of structural and aesthetic applications, offers high performance characteristics and finally is an economic driver to maintain forests and protect jobs in our community.

During the urban planning a pedestrian passage, bicycle parking and various paths were designed which will benefit the Red hook community. Moreover a traffic situation was studied and the main heavy transport route was moved to a street with lighter traffic.

The building was designed to occupy the entire plot as an integrated single unit meanwhile consisting of three parts with an internal courtyard. The building was oriented towards the south for a sustainable design and to provide a sea view to the flats. The multifunctional complex has a big parking lot under the entire building which prevents the surrounding streets to be overloaded with cars of indwellers and workers.

**ABSTRACT (ITALIANO)**

La sfida progettuale fu di immaginare un bosco di media altezza, complesso ad uso misto con unità abitative, una struttura di formazione lavoro/educativa, un centro innovativo per la progettazione di tecnologie di legno, e di un centro di distribuzione. Il sito del progetto è a Red Hook, geograficamente tagliato fuori  dalla zona di Brooklyn, ma sempre vibrante.

Cercando di rigenerare un settore manifatturiero urbano e soddisfare le esigenze abitative di New York City, è stato chiesto di progettare un luogo per la creazione di opportunità lavorative per professioni creative che abbracciano le nuove tecnologie del legno.

Innanzitutto  il design è stato orientato alla sostenibilità del progetto, cercando di minimizzare l'impatto sul pianeta. Per questo motivo sono state adottate numerose strategie attive e passive nel progetto.

Durante la progettazione numerosi metodi di costruzione, con particolare attenzione alle innovazioni nel design del legno su un vero e proprio sito, dovevano essere interpretati, inventati, e distribuiti.

Il legno è il materiale ideal per le costruzioni ecologiche: si adatta a una vasta gamma di applicazioni strutturali ed estetiche, offre elevate prestazioni, e incentiva l'economia mantenendo le foreste e assicurando posti di lavoro nelle nostre comunità.

Durante la pianificazione urbana, un passaggio pedonale, un parcheggio per biciclette e dei sentieri sono stati progettati a beneficio della comunità di Red Hook. Inoltre è stata studiata la situazione del traffico e il passaggio dei mezzi pesanti è stato deviato sulla strada poco trafficata.

L'edificio è stato progettato per occupare l'intera area come un'integrata singola unità mentre è costituito da tre parti con cortile interno. Esso è orientato con la sua area massima a sud per favorire la vista sul mare. Il complesso multifunzionale dispone di un grande parcheggio sotto l'intero edificio in modo da non occupare le strade circostanti con le vetture dei residenti e dei lavoratori.

**ABSTRACT (РУССКИЙ)**

Требованием конкурса было спроектировать комплекс из дерева смешанной высоты и смешанных функций, включающий в себя карьерный подготовительно – учебный центр, центр инновационного производства деревянных технологий и центр распространения. Строительный участок в Рэд Хуке географически отрезан от Бруклина, но всё же с растущей динамичностью.

Чтобы восстановить производственный сектор и удовлетворить необходимость в жилье Нью-Йорка было поставлено задание спроектировать место, где будут рождаться возможности для креативных профессий в сфере деревянных технологий.

В первую очередь проект был ориентирован на устойчивость окружающей среды здания, и попытку снизить как можно больше влияние на планету. В этих целях многие пассивные и активные стратегии были применены в проектировании.

Во время проектирования множество новых методов в строительстве, фокусируясь на инновациях в деревянном проектировании на реальном объекте, были изобретены и применены.

Дерево идеальный экологичный материал – оно используется для большого количества конструктивных и эстетических применений, у него множество положительных характеристик, а так же дерево это экономический двигатель для поддержания лесов и защиты труда в обществе.

В городском планировании в пользу района были спроектированы пешеходная галерея, велопарковка и велодорожки. Более того транспортная ситуация была изучена и большегрузный транспорт был отведен с загруженных улиц на менее оживленную.

Здание было спроектировано, занимая весь участок, как единое целое, в то же время, состоя из трёх частей с внутренним двориком. Оно ориентировано максимально возможной площадью на юг и на красивый вид. Под мульти – функциональным комплексом организована парковка, которая позволяет не загружать близлежащие улицы машинами работников и жильцов.

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## CHAPTER 1. INTRODUCTION

### 1. CHOISE OF THE COMPETITION

Choice of the group is based on the common decision to design sustainable residential buildings. Competition interested our group with its location and its climate that is very similar to our home climates. Also we consider that timber is the material of the future, it is sustainable and renewable that makes it very attractive to work with.

### 2. VISION

The program vision is to recreate our existing cities with buildings that are made from renewable resources, furthermore to offer affordable construction, to innovate with wooden materials, and to provide healthy living and working environments.

### 3. GOALS

1. To design a mid-rise, mixed-use complex with affordable housing units.
2. To provide all apartments with natural light and air.
3. To use wood as the primary structural material and include it in the design solution.
4. To interpret, invent, and deploy numerous methods of building systems, with a focus on innovations in wood design.
5. To integrate new mixed-use complex to its surrounding context.
6. To use sustainable and energy-efficient solutions.
7. To provide inhabitants with new bicycle paths and public green area.
8. To make everyone happy

### 4. THE BUILDING PROGRAM REQUIREMENTS

- **RESIDENCE**. Residences in this project are a mix of small units for single or double occupancy and larger.
- **BIKE SHARE + SHOP**. A bicycle share hub and central repair shop is required as an integral component of this new building.
- **WOOD PRODUCTION**. Manufacturing equipment for assembly processes related to the fabrication of elements for the construction of wood buildings.
- **DIGITAL PRODUCTION**. Digitally-based laboratory for the exploration of manufacturing processes.

# 5. RED HOOK

## 5.1. HISTORY

Red Hook is a waterfront neighborhood in Brooklyn so named by 17th century Dutch settlers for the rust colored soil, it has been part of the Town of Brooklyn since it was organized in the 1600's. Red Hook was isolated from the rest of South Brooklyn by wetlands and creeks that were drained and cultivated to use as farmland during the 18th and 19th centuries.

Fig.1. *Red Hook's industrial heyday, c.1875.*

## 5.2. LOCATION OF THE SITE

The site is situated between Dwight and Otsego streets, as well as Beard and Van Dyke Street. It is 7725,7 m² big.


*Fig.2. Red Hook, Brooklyn in New York. USA.*

## 5.3. THE SITE

Red Hook, South Brooklyn Waterfront, New York, is a mixed income, residential and industrial neighbourhood, and an increasingly vibrant community. The project site is across from a big-box furniture store and proximate to Added Value, a major urban agriculture site. It is also a few blocks from the Red Hook Houses, a significant public housing development – one of the largest in the city. Nearby are several major public amenities, including the Red Hook recreational facility, the Red Hook ball fields, and highly trafficked commercial outlets, including a grocery store.

**CHAPTER 2. URBAN DESIGN**

## 1. SITE ANALYSIS

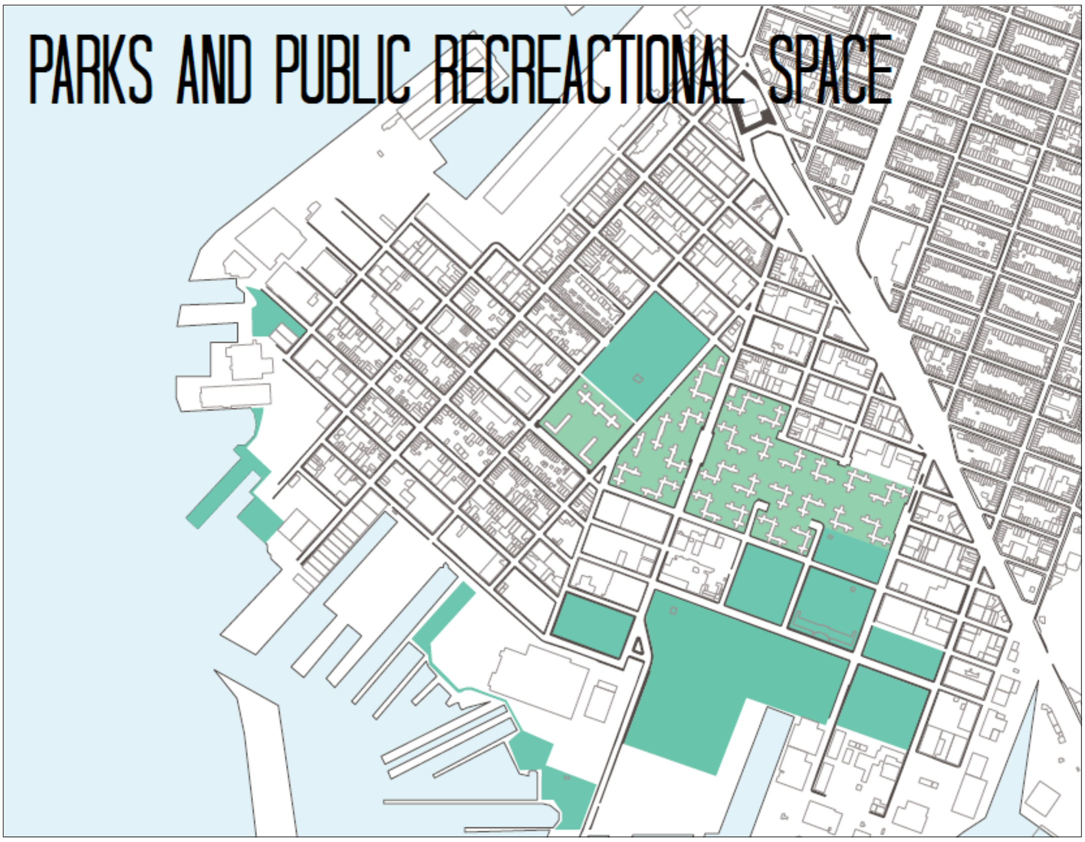


*Fig.3. Green spaces around the site*

During the process of analysis of the site it was found out that a lot of green spaces like parks, playgrounds and internal courtyards of residential complexes are surrounding the site that makes industrial district more pleasant. This makes Red Hook suitable for residential development and family living. As well as it increases value of real estate.

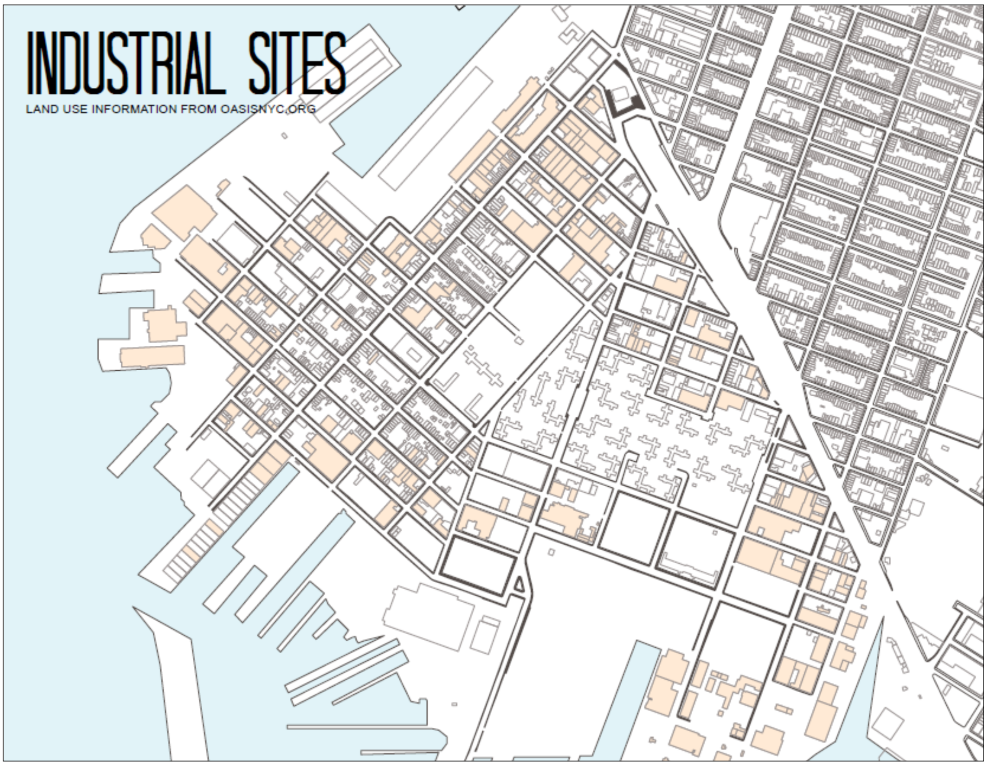


*Fig.4. Industrial sites around the site*

Historically Red Hook is an industrial district with lots of manufacturing buildings and corresponding architecture. These facts give the district opportunities for developing, creation of new residential and working spaces.

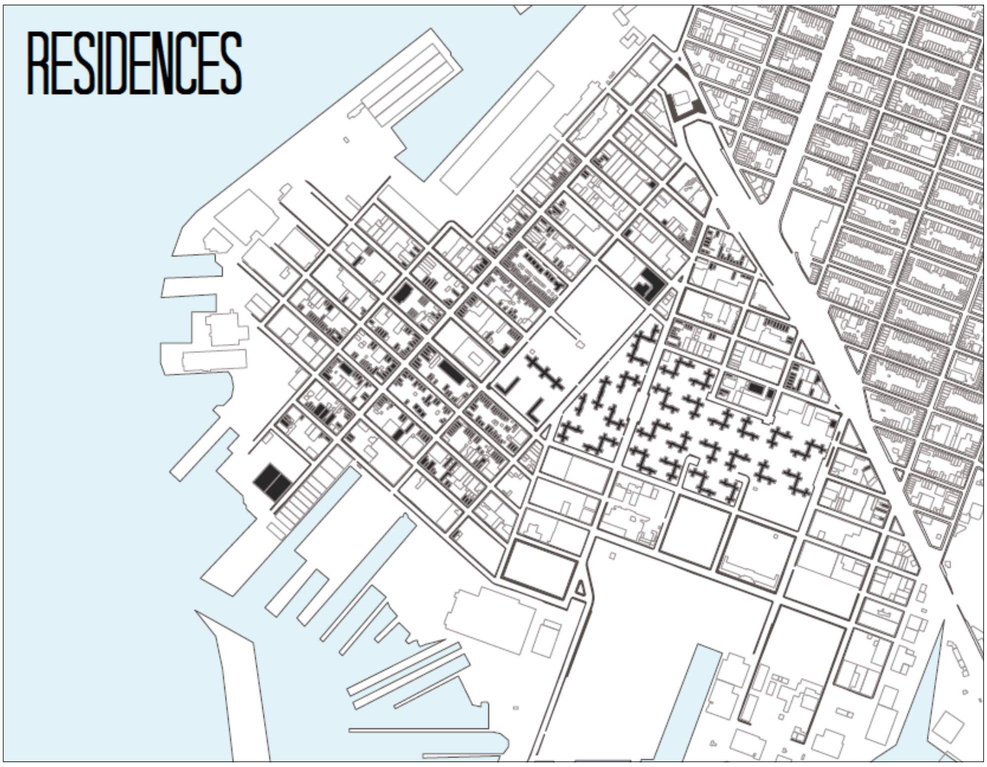


*Fig.5 Residences around the site*

Judging on the spread of residential spaces at the district the site has a big potential in development of residential sphere, because there is no housing next to the coastline with the beautiful view.

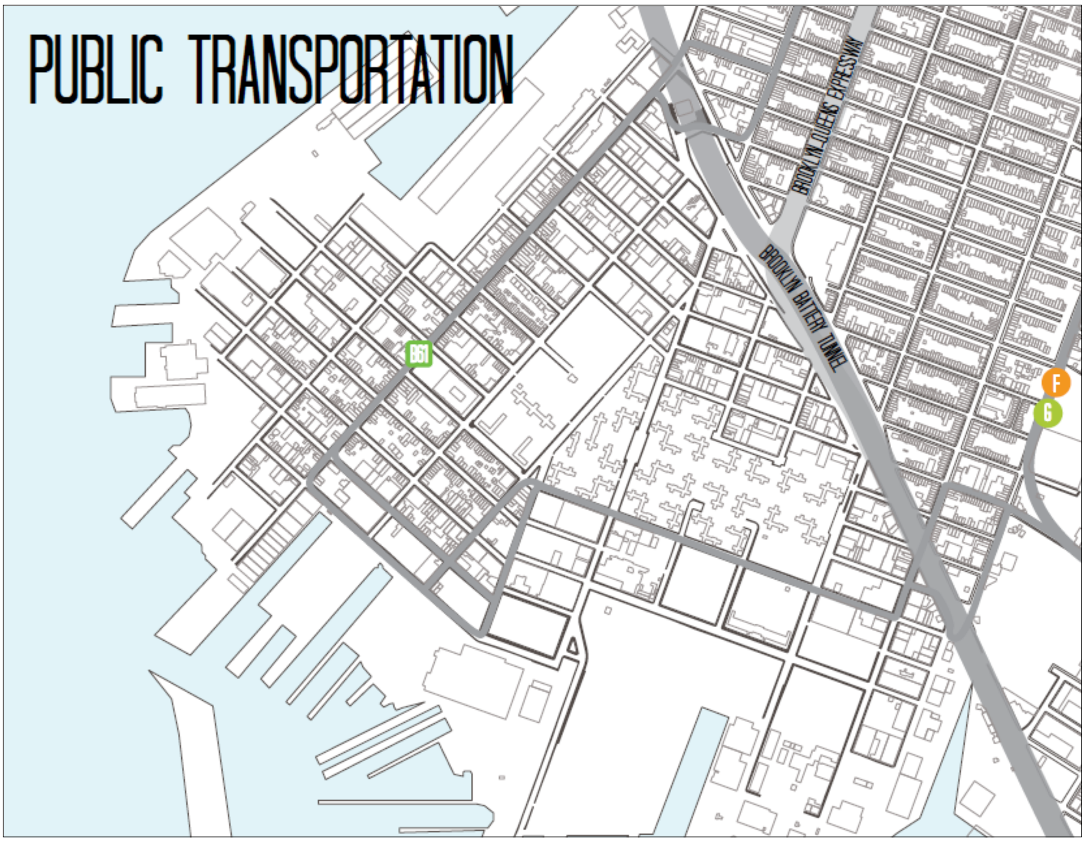


*Fig.6. Public transportation next to the site*

Red Hook has limited public transportation (Subway service is sparse; IKEA provides a shuttle to subway stations. The B61 bus route runs from the Fairway grocery store. Water ferry service is operated by NY Water Taxi). It makes this district not so easy to reach, but quiet and cozy for family life and inspiring for people with creative professions.

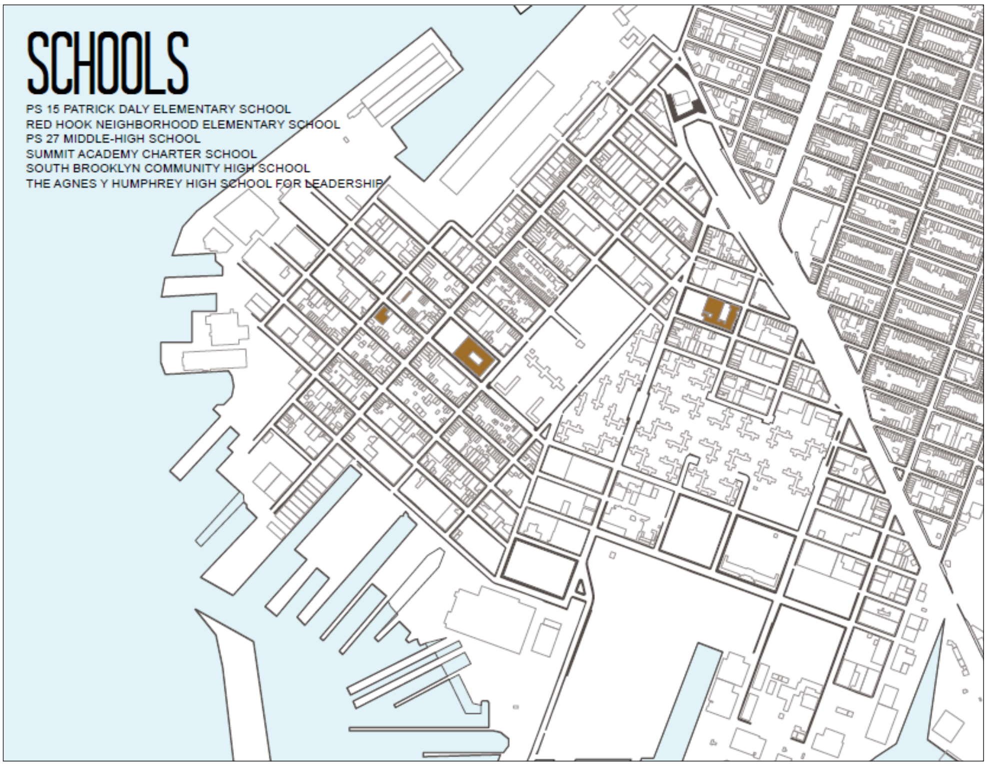


*Fig.7. Available schools next to the site*

Three schools are located next to the site that is very important for families with children especially because the Red Hook has limited transportation.

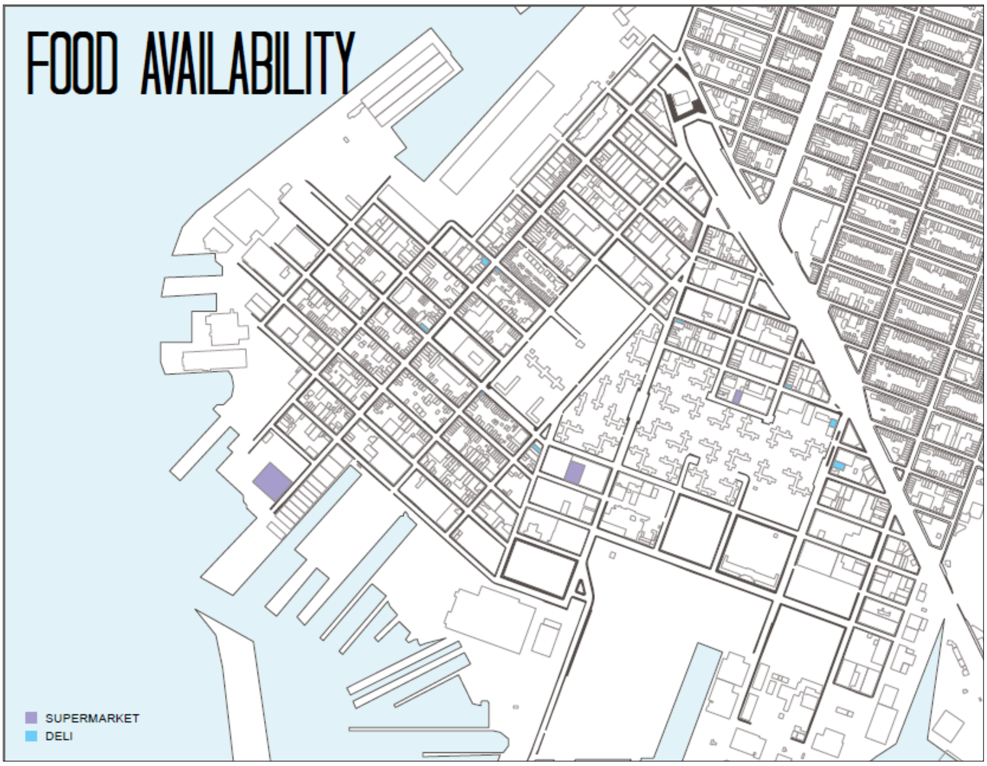


*Fig.8. Food shops next to the site*

The district has limited food availability , just two supermarkets, but still the biggest one is next to the site and also there is an ecological food farm the “Added value”.

## 2. EXISTING URBAN FABRIC


*Fig.9. Existing architecture in Red Hook*

The surrounding architecture and predominant materials determined choise of materials of the future building. Because the main material and focus of the project is timber, at the south façade timber cladding is used, but for the north façade due to fast decay of timber on the north side there was need to use another material.The most popular material in Red Hook is brick, to support it, it was desided to use modern type of brick – terracotta cladding. All the materials used in the project are locally manufactured, in the neighborhood of New York.

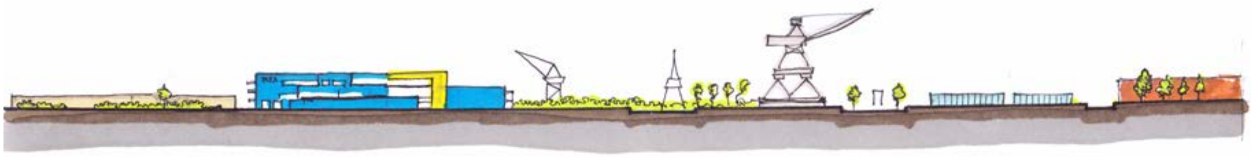

*Fig.10. Beard Street Section*

*Fig.11. Dwight Street Section*

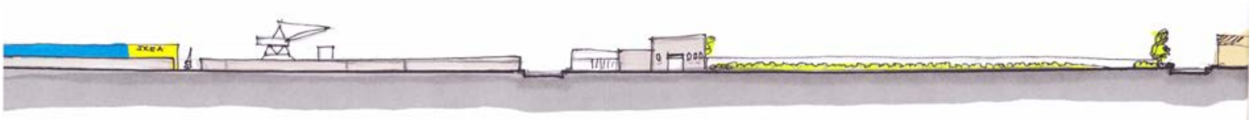

*Fig.12. Otsego Street Section*

*Fig.13. Van Dyke Street Section*

## 3. SWOT ANALYSIS

| STRENGHTS | WEAKNESS |
|---|---|
| + Red Hook is an increasingly vibrant district.<br>+ Nearby are recreational facility, the ball fields, commercial outlets, a grocery store, a major urban agriculture site and is in few blocks from the Red Hook Houses.<br>+ Timber is an ideal green building material. | - Red Hook has limited public transportation (Subway service is sparse; IKEA provides a shuttle to subway stations. The B61 bus route runs from the Fairway grocery store. Water ferry service is operated by NY Water Taxi).<br>- Presence of IKEA store that provoked traffic congestion, decreased property values and historical significance of buildings in the area.<br>- Quite often natural disasters, like floods. |
| **OPPORTUNITIES** | **THREADS** |
| + The Brooklyn Cruise Terminal brings additional tourists.<br>+ Multiple cultural events held in the Red Hook.<br>+ A prospective residential area, especially for artists.<br>+ Fabrication of wood or other components at a smaller scale will occur in Red Hook. | - Red Hook is cut off from much of Brooklyn geographically.<br>- In design the NYC code has to be used, where contemporary timber technology may not yet be anticipated. |

*Tab. 1. SWOT*

The competition site has to include a numerous variety of functions, that might lead to overloading its own area. To solve this problem we divided the site into different zones. Firstly we separated plot area for private and public zones, arranging them independently from each other, locating factories on the back side of the plot and placing residential facilities in the front, to provide them with daylight and a view to the sea.

Our next step was to evaluate all the functions and to find the best possible layout for floor division, as well as to deduce an appropriate number of storeys. The final design includes all facilities joined together in one complex, but dividing them by floors – first double height floor – factory facilities and at the same time its roof will become a courtyard. Residential buildings will be placed over factories and will have atriums, rotated balconies and green terraces.

Then we analysed the possible access to the building, due to high traffic on the IKEA side, we decided to place  car access to the complex from the north side. Pedestrian access to the residence was placed on the other side to make sure that it is separated from the noise pollution of the factories.

To complete new residential complex we decided to add a public green area on a vacant plot by the sea and while keeping new bicycle paths separate from car traffic in the site neighbourhood.  This solution will increase comfort of inhabitants and will be a good addition for existing residential areas.

## 4. PROPOSED SECTIONS OF THE STREETS

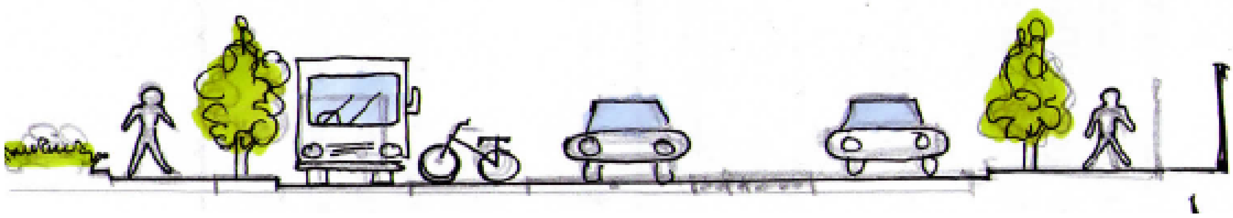

*Fig.14. Otsego Street. New bicycle path*

*Fig.15. Columbia Street. New bicycle paths and trees*

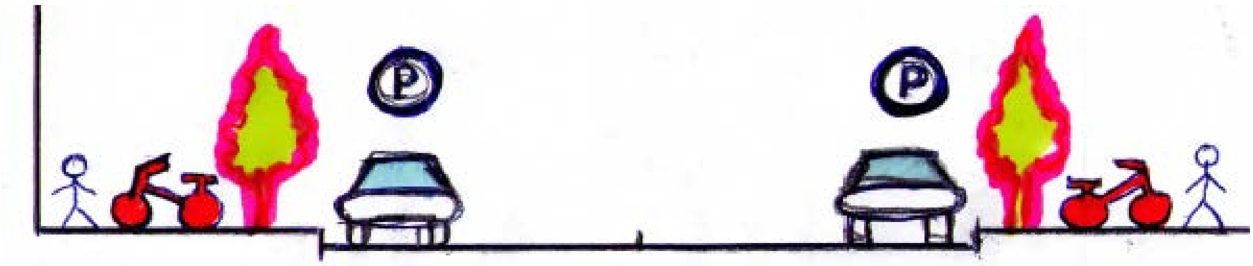

*Fig.16. Dwing Street. New bicycle paths and trees*

## CHAPTER 3. ARCHITECTURAL DESIGN

### 1. SUSTAINABLE CONCEPT

From the start of the design process sustainability has been one of the inspirations for the design of multifunctional building in Brooklyn. Another idea was to design a building which conforms to the surrounding area at the same time being comfortable housing for residents and a place of work for employees. Thus, the purpose was to obtain the convenient location of all required functional areas of the building following the middle-storey.

The project consists of three main units: two storey stylobate and two separate stepped shape volumes. The orientation of the building , its rectangular shape, along with the facade and the internal courtyard are conceived to maximize the natural ventilation and to control exposure to the sun. The South – East oriented terraces are configured as voids in order to obtain an ideal interior climate and optimizes energy consumption. On summer hot noons ,thanks to its movable louvres, it acts as a buffering zone, providing the shade and preventing overheating maximize. During winter days louvers can be fixed in the most opposite position to let the sun in. The door is sliding so that two spaces can be integrated to erase the limits between interior and exterior.

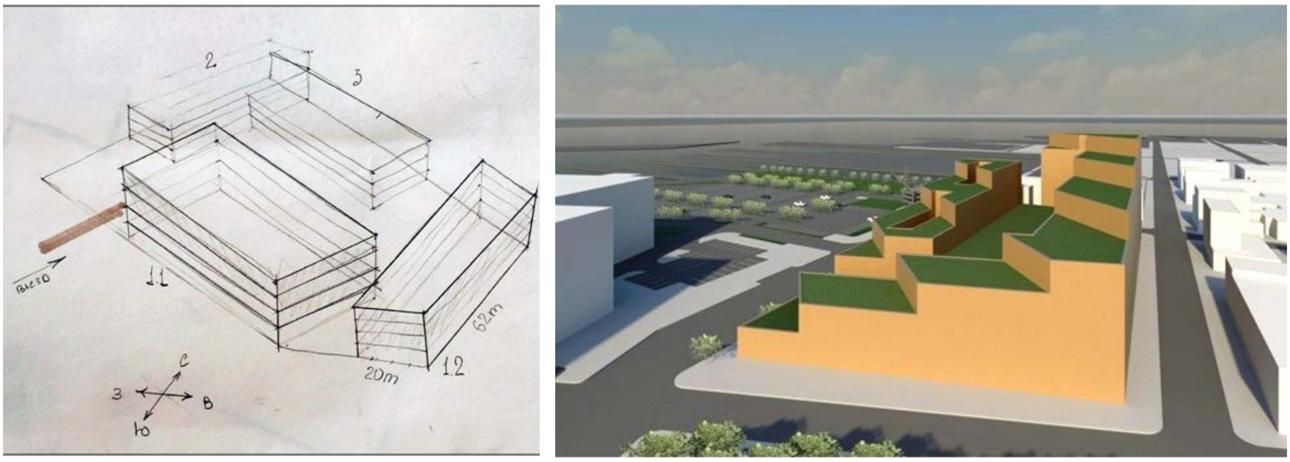


*Fig.17. Conceptual volume*

## 2. MASTER PLAN

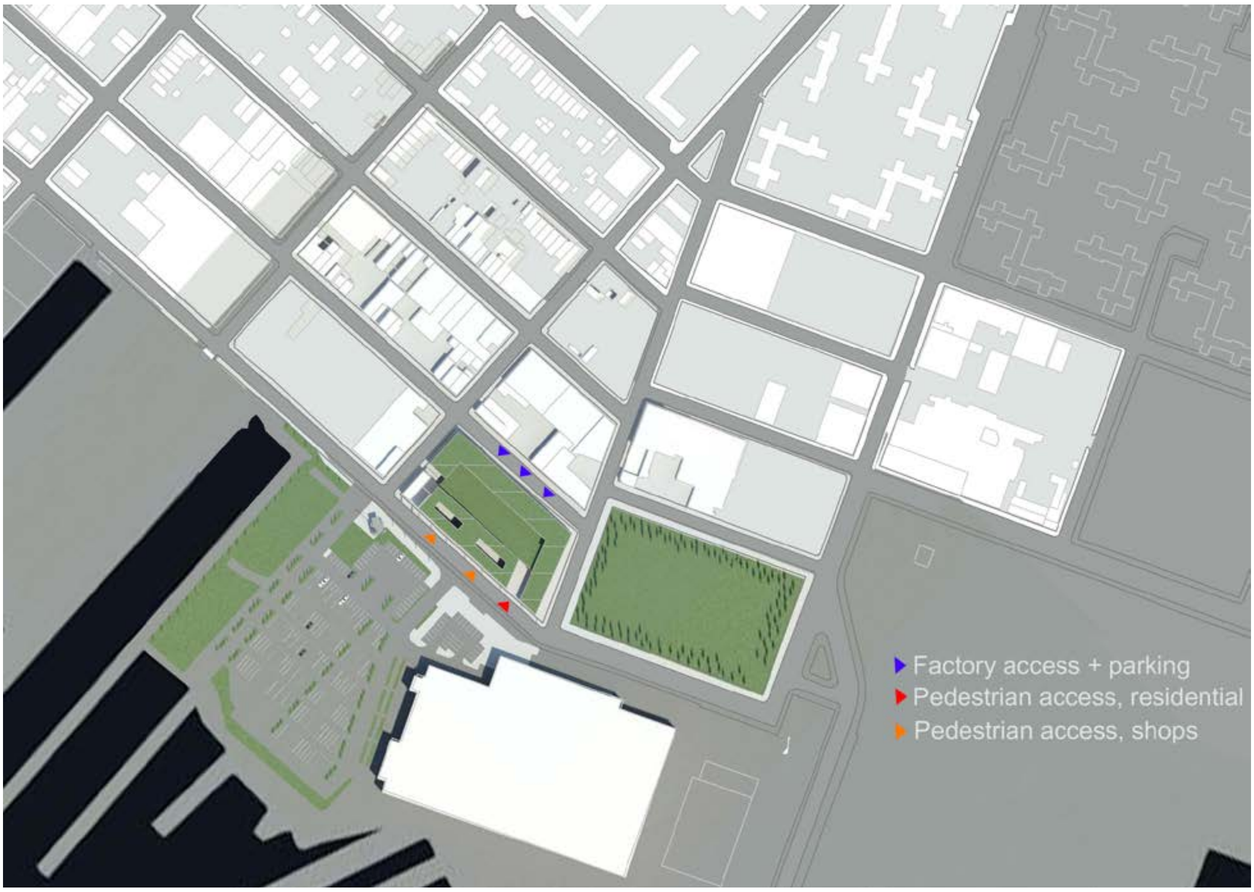


*Fig.18. Master plan*

The entire plot during the design process was totally occupied with unified building consisting of three parts – two tall timber parts and medium concrete part with internal courtyard and roof garden above. The ground floor is occupied with timber factory that requires heavy transport access to the building. It is organized from the north façade, from a quiet street, not to overload the main street with heavy traffic. Access for the cars of residents and workers is also performed from the peripheral streets. Pedestrian accesses in shops and flats are situated on the main street side where the biggest flow of people from IKEA and pedestrian passages occurs.

## 3. FUNCTIONAL PLANNING INSIDE AND OUTSIDE

### 3.1. DESCRIPTION ON THE PLANNING

The stylobate is assigned for commercial zone and factories. The second level of it is cantilevered. Hanging above the Halleck Street it provides the gallery passage along the site. The new bike path is connected with protected bike point which includes bike share, shop and parking. Also the area of wood factory workshop and the factory faces the street.

From the Halleck Street the massive staircase gives an access to a silent space – the long rectangular courtyard. More considered as a private park it contrasts with the crowdy environment. The semantic centers of the courtyard are the big oaks, creating an atmosphere of unity with nature and giving all the advantages of natural forest.

The central part is organized as a "boulevard" – pedestrian zone with long row of benches symmetrically located with respect to skylights.

### 3.2. TYPES OF APARTMENTS

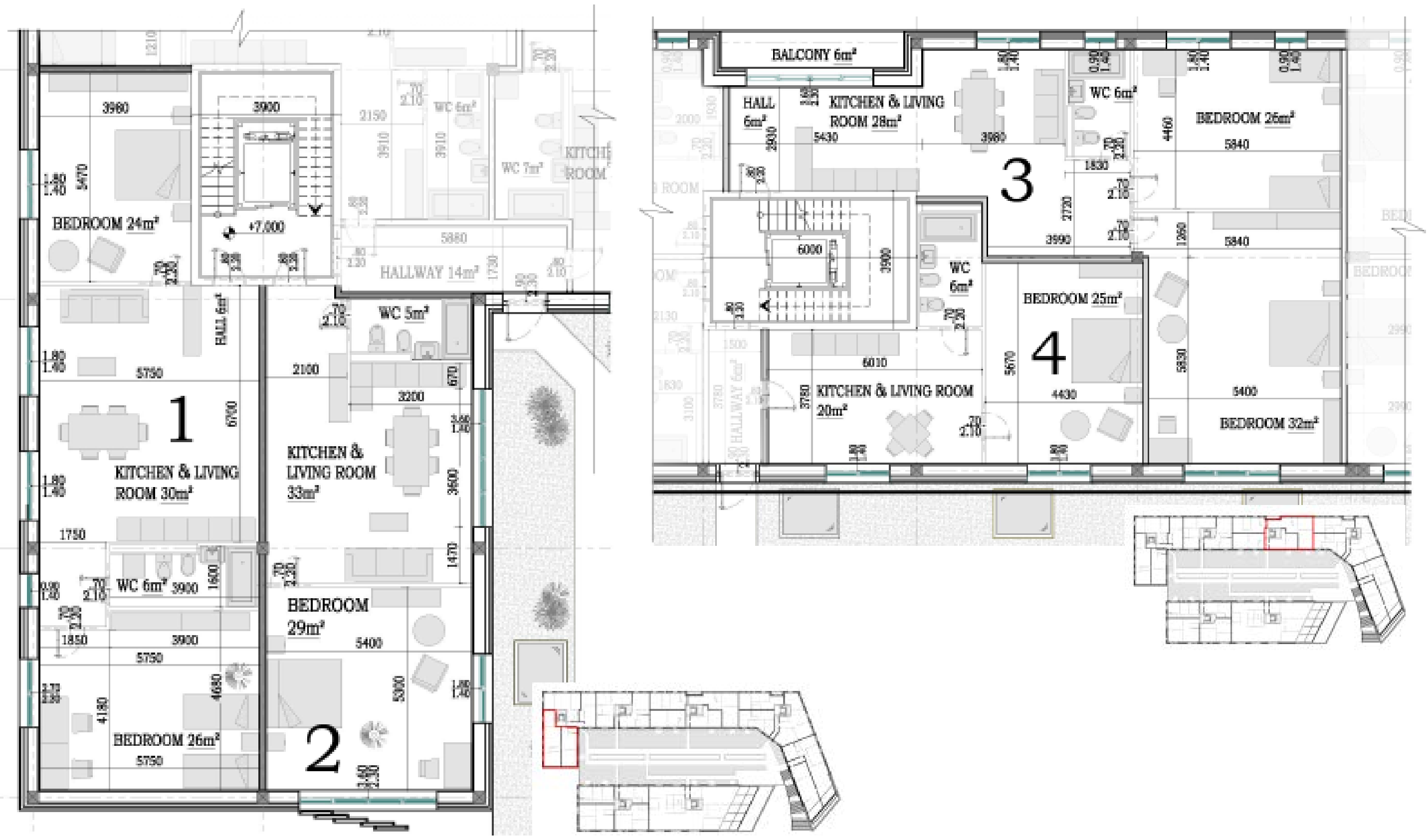


*Fig.19. 1, 2, 3, 4 types of apartments*

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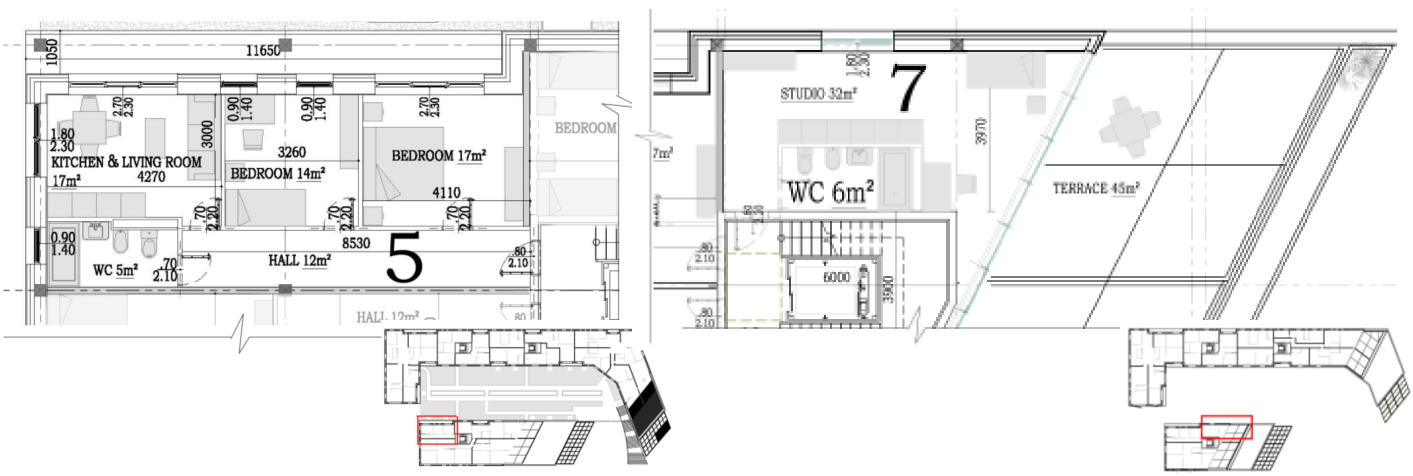



*Fig.20. 5 and 7 types of apartment*

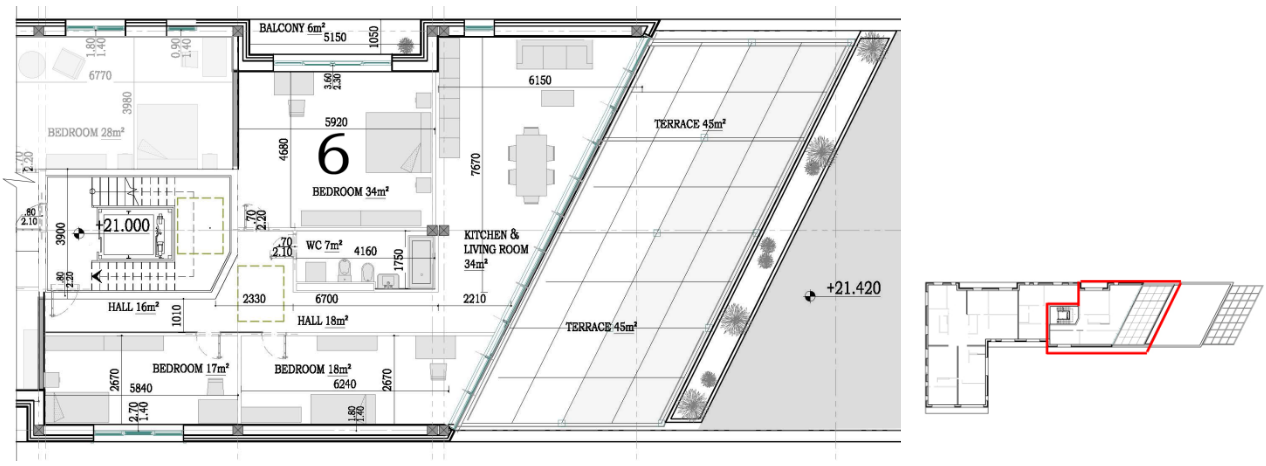
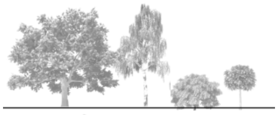
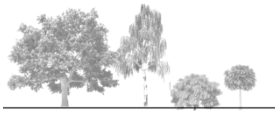
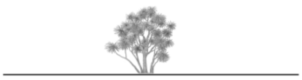



*Fig.21. 6 type of apartment*

| | TYPE OF APARTMENT | LOCATION | AREA | RESIDENTS |
|---|---|---|---|---|
|  | 1st type | 1st floor | 92 $m^2$ | 4 people family |
|  | 2nd type | 1st floor | 60 $m^2$ | 3 people family |
|  | 3rd type | 1st floor | 94 $m^2$ with balcony | 4 people family |
|  | 4th type | 1st floor | 52 $m^2$ | 2 people family |
|  | 5th type | 2nd floor | 52 $m^2$ | 3 people family |
|  | 6th type | 5th floor | 110 $m^2$ with terrace and garden | 5 people family |
|  | 7th type | 3rd floor | 32 $m^2$ with terrace and garden | Studio |

*Tab.2. Types of apartments*

## 4. VIEWS OF THE BUILDING

*Fig. 22. Overall view of the building*

*Fig. 23. Bird's eye view*

*Fig. 24. View from the north side*

*Fig. 25. View from IKEA*

*Fig.26. View from the internal courtyard*

*Fig.27. View of one of the terraces*

**CHAPTER 4. SUSTAINABILITY**

## 1. SUSTAINABILITY FRAMEWORK – ONE LIVING PLANET


As our sustainability framework we chose – **One Living Planet framework.**

This framework seems for us the most reasonable and clear. Our action plan with proposed strategies and targets we based on the 10 principles of the OPL.

### ZERO CARBON

Our climate is changing because of human-induced build up of $CO_2$ in the atmosphere, this is why we are trying to reduce $CO_2$ emissions by designing smaller building footprints and square footage, installing green roof, as well as making building more energy efficient and delivering all energy with renewable technologies.

### ZERO WASTE

Waste from discarded products and packaging creates disposal problems and squanders valuable resources. For this reason we are trying to reduce waste, reusing where possible, and ultimately sending zero waste to landfill.

### SUSTAINABLE TRANSPORT

Travel by car and airplane is contributing to climate change, air and noise pollution, and congestion. We are introducing new bike paths around our site that will connect it to the New York City and provide zero carbon emissions.

### SUSTAINABLE MATERIALS

Destructive resources exploitation increases environmental damage and reduces benefits to local community. That is why we induce sustainable use of materials - lightweight, locally available, recycled materials.

### SUSTAINABLE WATER

Industrial agriculture produces food of uncertain quality, harms local ecosystems, and may have high transport impacts. The site is situated next to the 'Added Value', a major urban agriculture site that provides naturally produced healthy food.

### LAND USE & WILDLIFE

Local supplies of freshwater are often insufficient to meet human needs, due to pollution, disruption of hydrological cycles, and depletion. Due to this problem we are using water more efficiently by installing low water, energy efficient toilets, harvesting rain water with permeable pavement.

Loss of biodiversity due to development in natural areas and over-exploitation of natural resources. Protection and restoration of existing biodiversity and natural habitants we are providing through applying the green roof solution.

### CULTURE & HERITAGE

Local cultural heritage is being lost throughout the world due to globalisation, resulting in loss of local identity and knowledge. Our project goal is to recreate local identity and wisdom, supporting and inspiring the existing art projects by providing spaces designed to gather people.

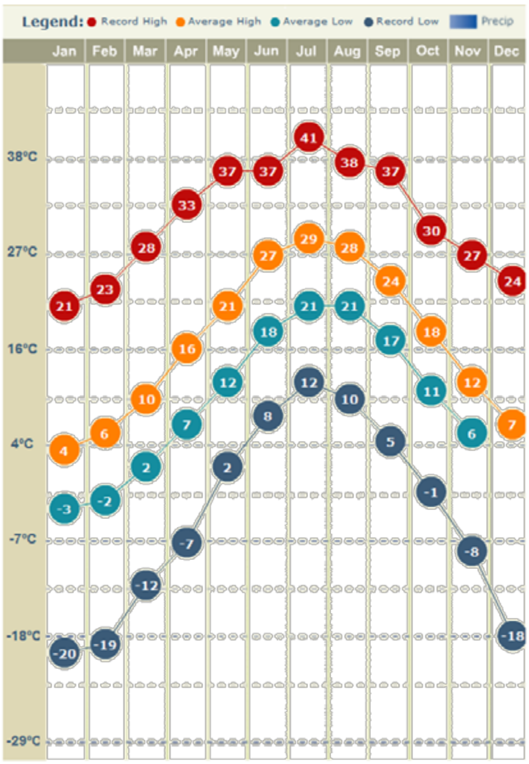
### EQUITY & LOCAL ECONOMY

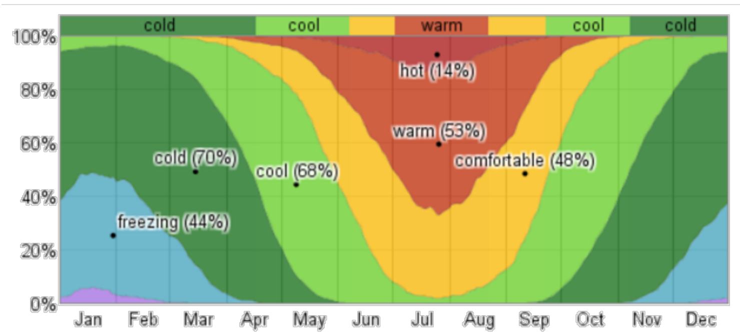
Some in the industrialised world live in relative poverty, while many in the developing world cannot meet their basic needs from what they produce or sell. Creating new wooden and digital production factories that support new employment, providing high quality housing.

### HEALTH & HAPPINESS

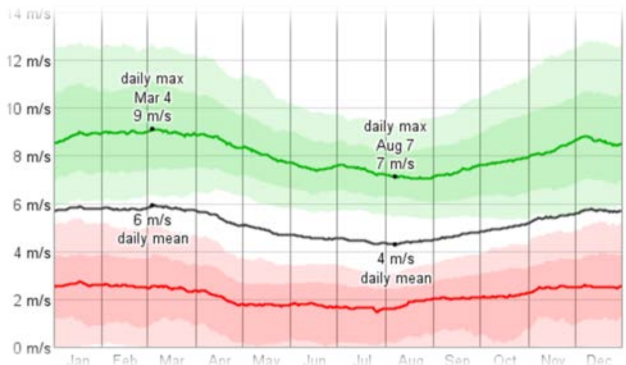
Rising wealth and greater health and happiness increasingly diverge, raising questions about the true basis of well-being and contentment. Encouraging active, sociable, meaningful lives to promote good health and well being by improving pedestrian and bicycle paths, and providing public green spaces.

## 2. ENVIRONMENTAL ANALYSIS




*Gr.2. Fraction of time spent in various temperature bands*

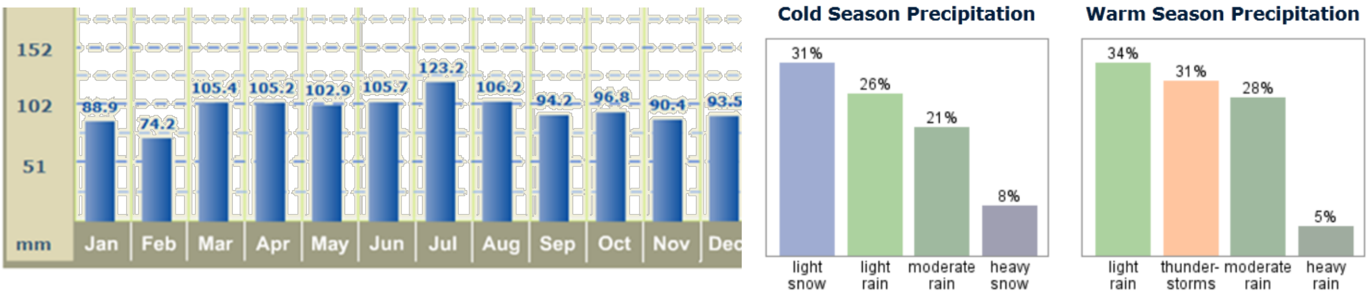



*Gr.1. Thermal fluctuations during the year*

*Gr.3. Wind speed*

Typical wind speed over the year varies from 1 m/s to 9 m/s (light air to fresh breeze), rarely exceeding 13 m/s (strong breeze).

The highest average wind speed of 6 m/s (moderate breeze) occurs around March 4, at which time the average daily maximum wind speed is 9 m/s (fresh breeze).




*Gr.4. Annual precipitations*

| Annual chance of sunshine: 58% | | | | |
|---|---|---|---|---|
| Month | % Sunny | Clear days | Partly cloudy days | Cloudy days |
| January | 51.61% | 8 | 8 | 15 |
| February | 53.57% | 8 | 7 | 13 |
| March | 54.84% | 8 | 9 | 14 |
| April | 55.17% | 7 | 9 | 13 |
| May | 58.06% | 7 | 11 | 13 |
| June | 62.07% | 7 | 11 | 11 |
| July | 64.52% | 7 | 13 | 11 |
| August | 64.52% | 8 | 12 | 11 |
| September | 63.33% | 10 | 9 | 11 |
| October | 64.52% | 11 | 9 | 11 |
| November | 51.72% | 7 | 8 | 14 |
| December | 53.13% | 8 | 9 | 15 |

*Tab.3. Chance of sunshine*

| Month | Heating Degree Days | Cooling Degree Days |
|---|---|---|
| January | 1003 | 0 |
| February | 835 | 0 |
| March | 692 | 0 |
| April | 381 | 1 |
| May | 126 | 52 |
| June | 7 | 205 |
| July | 0 | 379 |
| August | 0 | 338 |
| September | 21 | 130 |
| October | 250 | 11 |
| November | 522 | 0 |
| December | 844 | 0 |

*Tab.4. Potential demand for energy*

From the following climate study we expect that building heating load in Brooklyn area will be much higher than cooling load. Therefore we will consider a solution to open big windows to the south and smaller ones to the north to gain more passive sun energy.

### 3. SUMMER AND MIDTERM SUSTAINABLE STRATEGIES

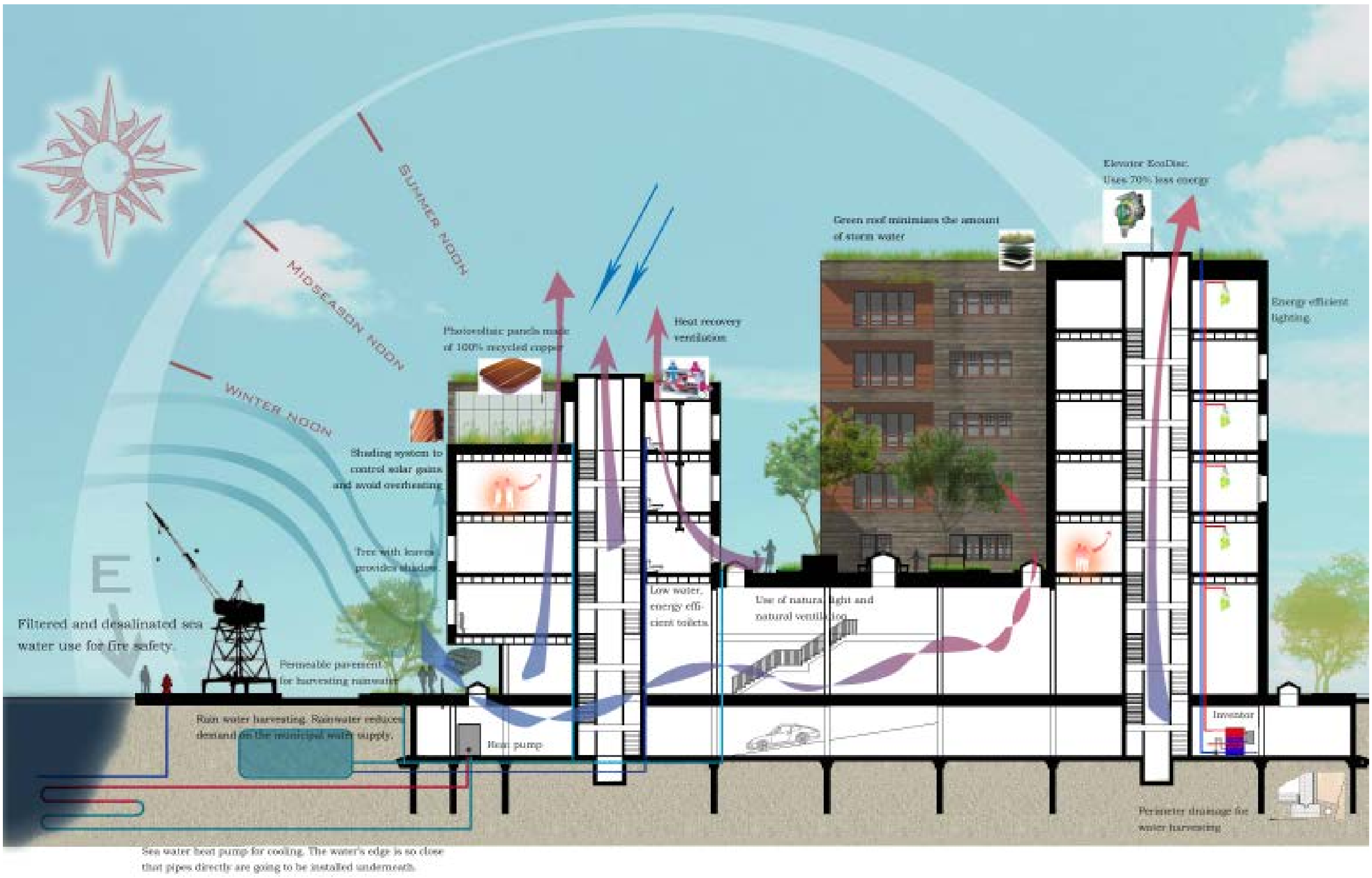



*Fig.28. Summer, midterm sustainable strategies*

First of all passive sustainable strategies are considered. Orientations and narrow floor plans of the buildings allow cross ventilation and proper use of solar energy, natural day lighting allows reducing artificial lighting need during the daytime. Open living areas with high ceilings to maximize air movement and reduce radiant heat to occupants.

Strategically placed high performance windows and shading system to control solar gains and overheating. Big trees in the roof garden with leaves in summer provide shadow and cools down the envelope.

Green roof helps to regulate microclimate, improves air quality and minimizes the amount of storm water.

Sea water heat pump is used as energy source for cooling purposes. The water's edge is so close that pipes directly are going to be installed underneath.

Grey water harvesting using permeable pavement and water collection tank that allows rain water and melting snow to penetrate through the permeable layer and collecting water at the reservoir, which reduces demand on the municipal water supply. Around all the perimeter walls drainage is going to be installed.

Heat recovery ventilation in summer operates as a ventilation system by bypassing the heat transfer system and simply replacing indoor air with outdoor air.

Filtered and desalinated sea water use for toilets and fire safety. Mechanical parts and pipes will be made of stainless steel and plastic to help prevent corrosion.

On demand hot water heaters, use of heating controls (thermostat), low water toilets, energy efficient lighting, elevator and motion sensors.

Photovoltaic panels made of 100% recycled copper oriented on south that provide electricity for the building.

## 4. WINTER SUSTAINABLE STRATEGIES

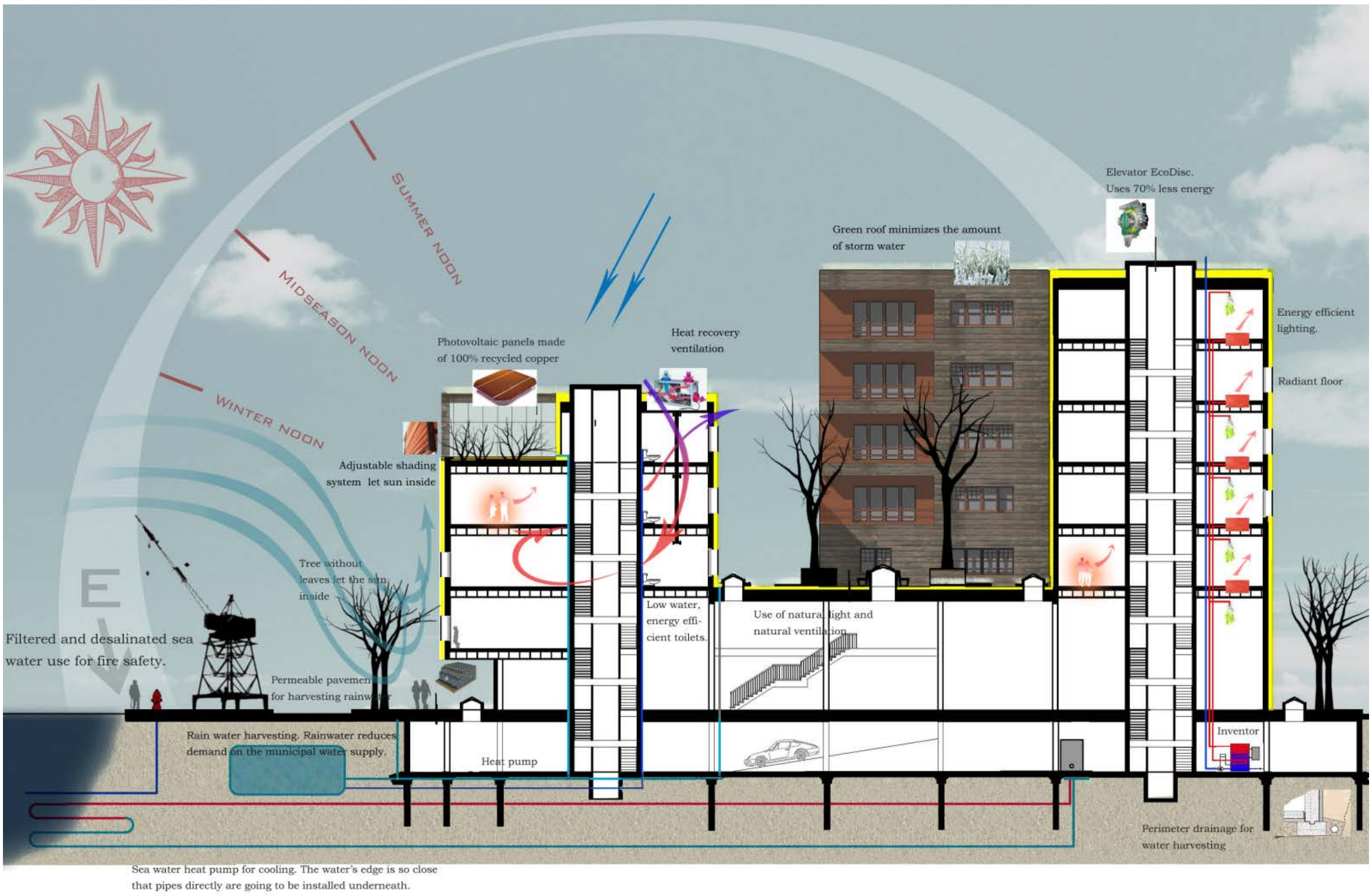



*Fig.29. Winter sustainable strategies*

In winter, passive energy contribution is at maximum. The building is exploiting gains from large windows at the South with shading louvers in a position that allows sunrays to pass. Trees in atrium are losing leaves in the winter, so there is no blocking of sunlight. At the same time, small North windows are minimizing cold bridge and heat loses. Internal gains from people and equipment are also taken into account. All of these are additions to the main source of heating: the radiant floor. With radiant floor heating there is no air movement and the energy sources are the photo voltaic panels.

Highly insulated fiber wood insulation, which is one of the most ecological on the market, and air tight envelope with low U value (0,098 W/m$^2$K) is going to prevent excessive heat loses. High performance and gas-filled windows with double glazing and low-e coatings that reflect part of room's heat in the winter, it reduces heat losses.

Heat recovery ventilation reduces heat loss so less heat input (from another source) is required to raise the indoor temperature to a comfortable level, it is cost-effective, as less energy is required to move air than to heat it, also it reduces indoor moisture in winter, as cooler outdoor air has a lower relative humidity.
Sea water heat pump use for heating the building and hot residential water. Using electricity, the heat pump works to move thermal energy from a cold source to a warmer heat sink. It allows sparing 50 % of $CO_2$ emissions.

Green roof helps to regulate the microclimate and minimizes heat loses.

## 5. ENVELOPE OF THE BUILDING.

For the building envelope as well as for the structure timber materials and materials on a base of wood were used where possible to meet one of the main goals of the project, which is to build the building of timber and ecological materials. For this reason prefabricated wall panels like Rubner Haus Residenz are used that consists of fiber wood and cork insulation, external layer of polyurethane was added to avoid thermal bridges at column regions and for fire safety this material was used.

As for design of the façade it was decided to use ventilated façade system that eliminates condensation on surface of the wall and lets the external insulation layer to last longer because of ventilation gap. At the south façade timber cladding is used to support theme of the project, but for the north façade it was decided to put terracotta cladding due to fast decay of timber on the north side.

As almost all the roof spaces are accessible for this purpose timber paving is used. At the remaining roof spaces green roof is going to be installed.

For all the parts of the envelope thermal resistances and thermal lags were calculated that showed very good values corresponding to Passive house standards. To avoid condensation risks Glaser diagrams were computed for the envelope (Fig.30. and Fig.31.).

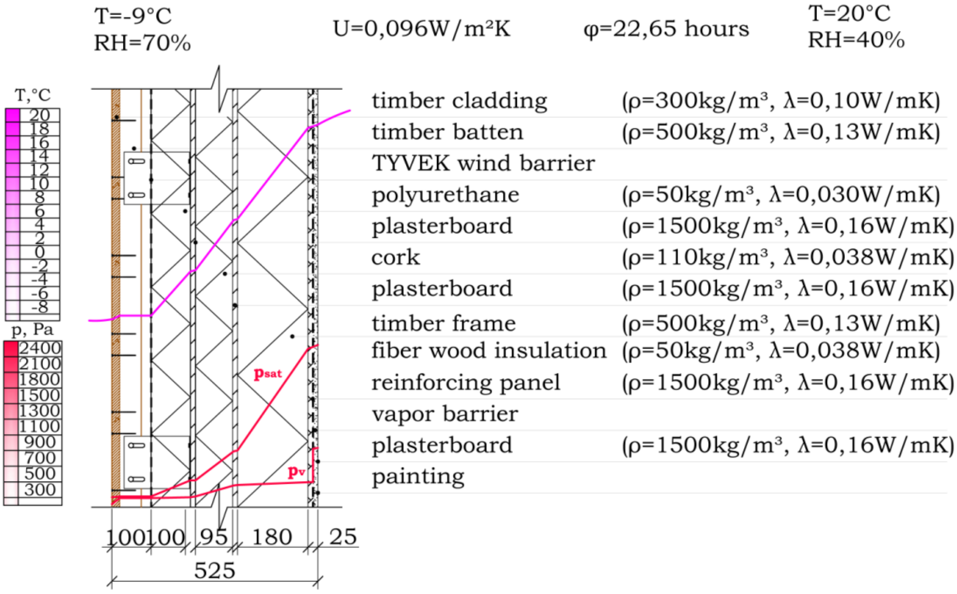



*Fig.30. External wall with timber cladding*

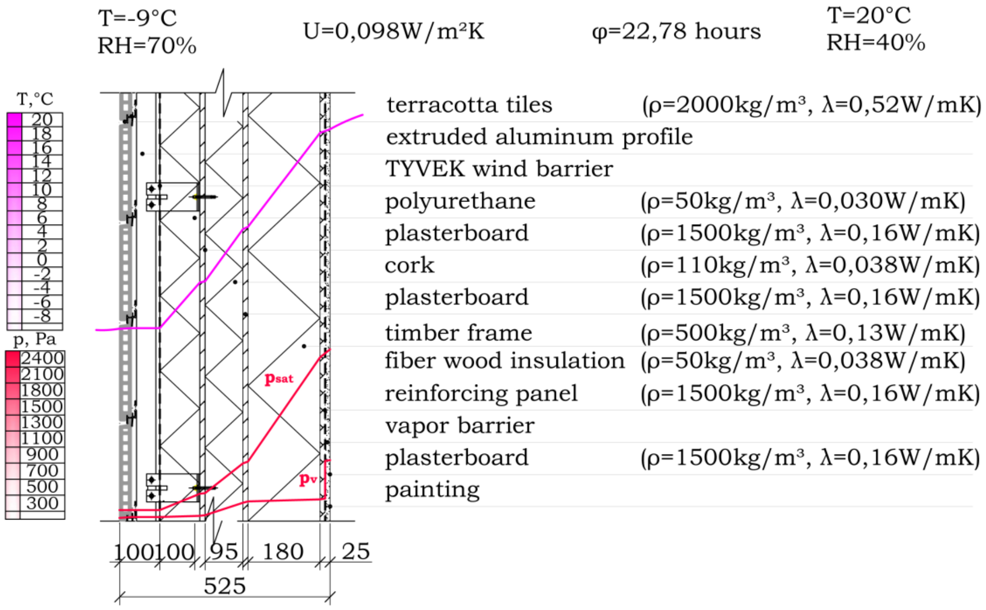



*Fig.31. External wall with terracotta cladding*

## 6. SHADOWS ANALYSIS

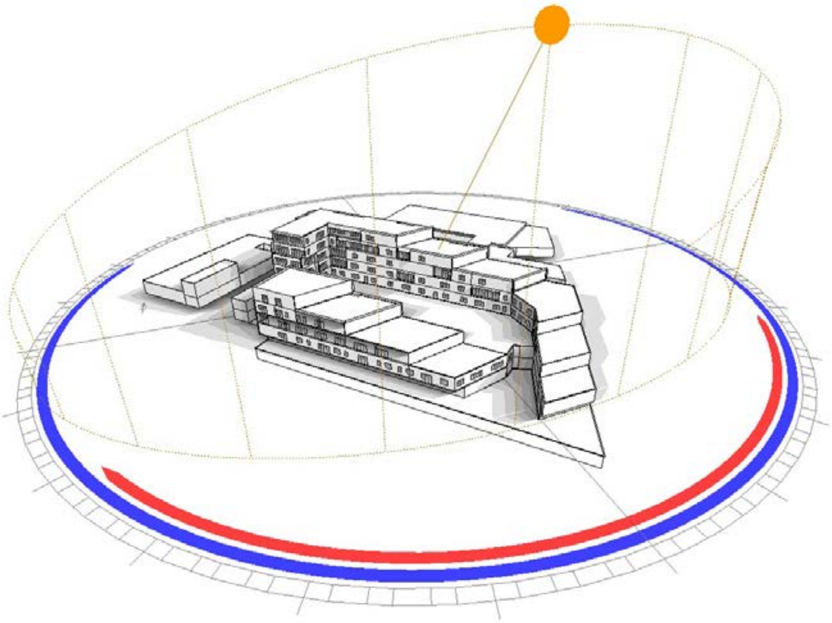


*Fig.32. Sun path*

Shadow analyses were made to better understand the movement of shadows casted by the surrounding buildings due to the sun path in winter and summer seasons.

Times analyzed:

Summer: 08:00 – 17:00

Winter: 09:00 – 16:00

By studying these results, we can see where windows are needed and where they should be avoided. Due to the shape of the building and its orientation to the south in the summer only north facades are shaded. During the summer due to the high sun path the building is exposed to a lot of sunlight (radiation). For this reason, it is necessary to provide shading on highly exposed southern and western windows. At the same time large roof surface can be used for solar panels. Note that during the summer the building casts almost no shadow on the surrounding buildings. In the winter, the building remains in shadow for the entire day and casts long shadows on the surrounding buildings. The large windows oriented on the south provide optimal natural light. The shadow diagrams show the movement of shadows casted by the building.

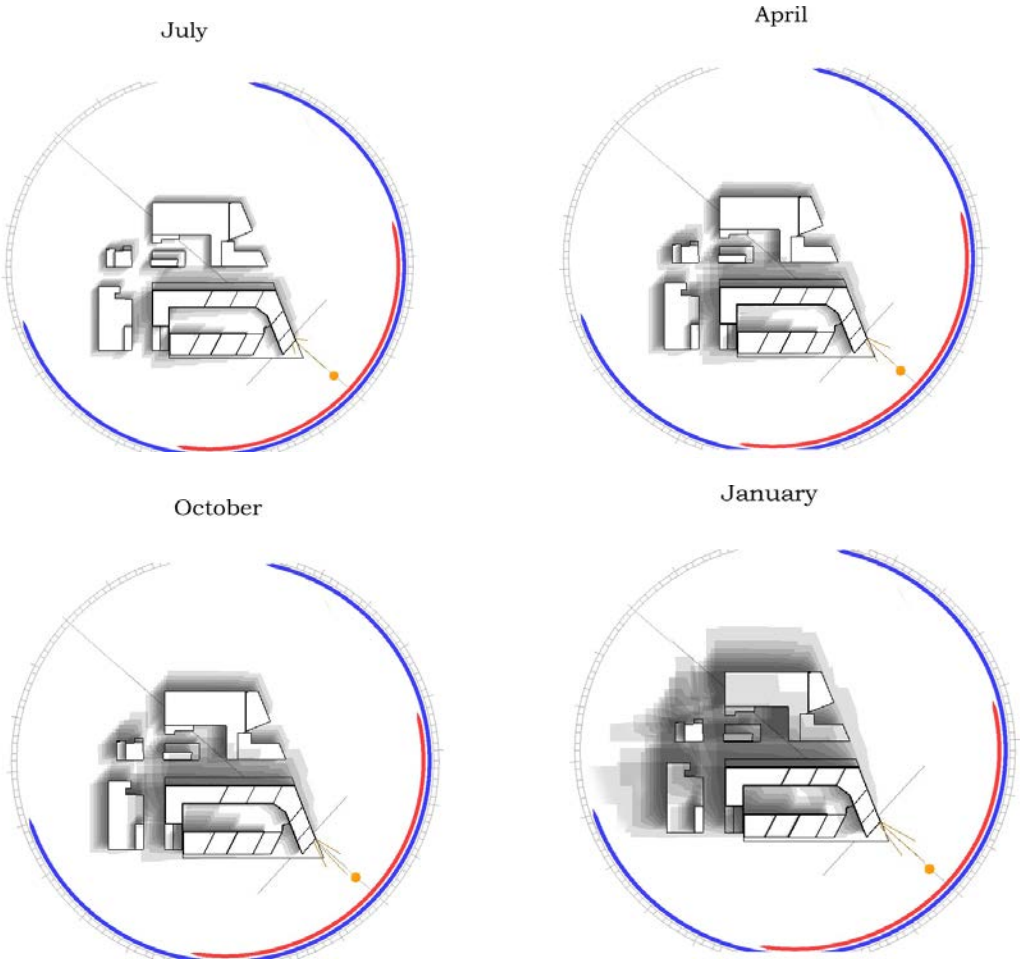



*Fig.33. Sun paths during different months of the year*

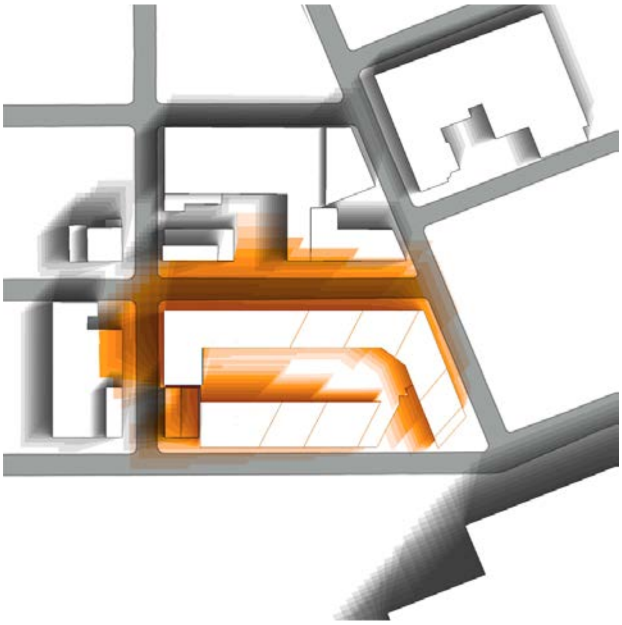


*Fig.34. Shadows casted from the designed building and surrounding buildings*

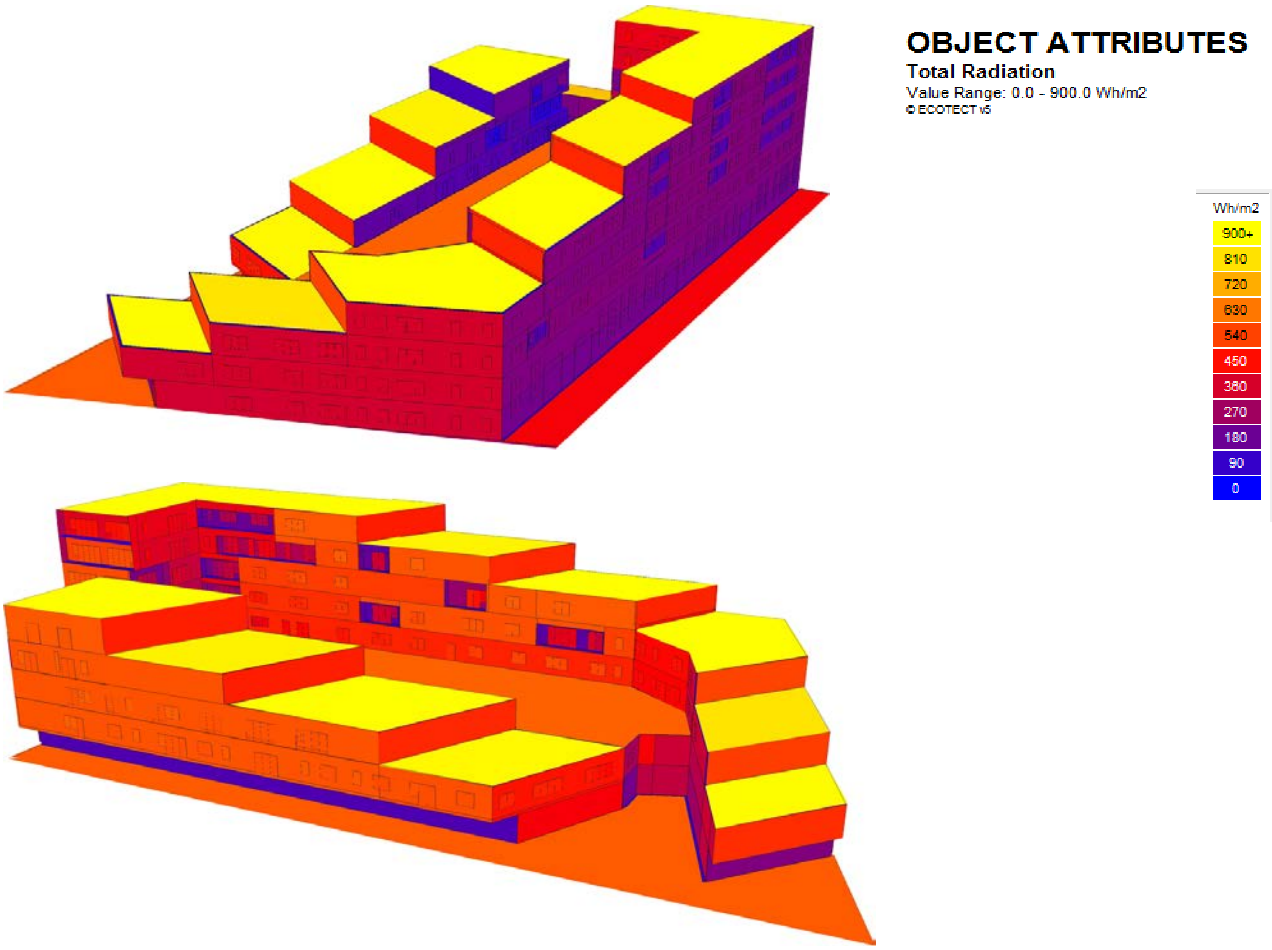
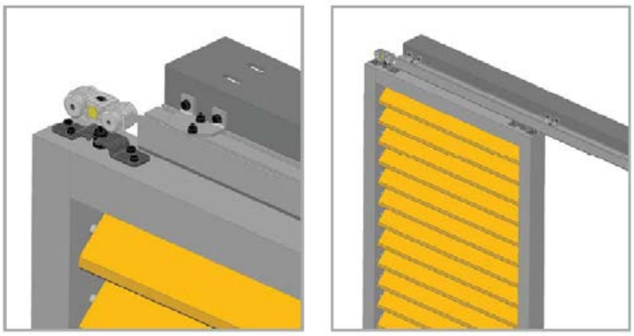



*Fig.35. Solar radiation*

Due to highly exposed southern and western façades with large windows, shading devices is necessary to bring down the cooling loads and intense direct sunlight during the summer.

For those purposes on the inner south facade built in window shadings and balconies were designed to provide nice cool space. But on the south and west facades to avoid overheating movable shading like "Colt Ellisse - sliding panels" were designed. On the accessible roof terraces fixed shadings are provided.



Ellisse sliding panel systems contain louvers which can either rotate or stay fixed. The louvers inside the frames are able to be turned to a maximum of 85° by hand.

*Fig.36. Colt Ellisse - sliding panels*

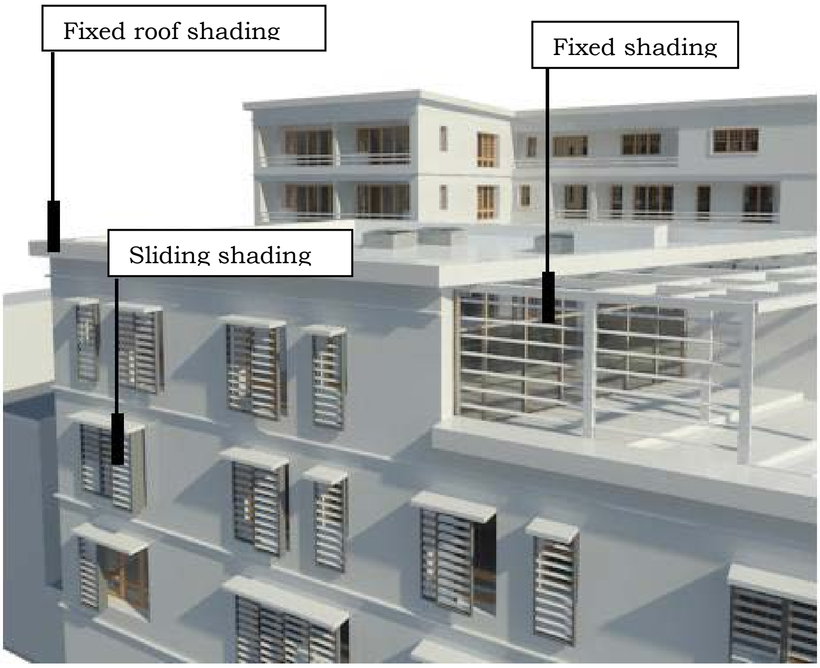



*Fig.37. Types of shadings provided in the building*

## 7. DAYLIGHT FACTOR ANALYSIS

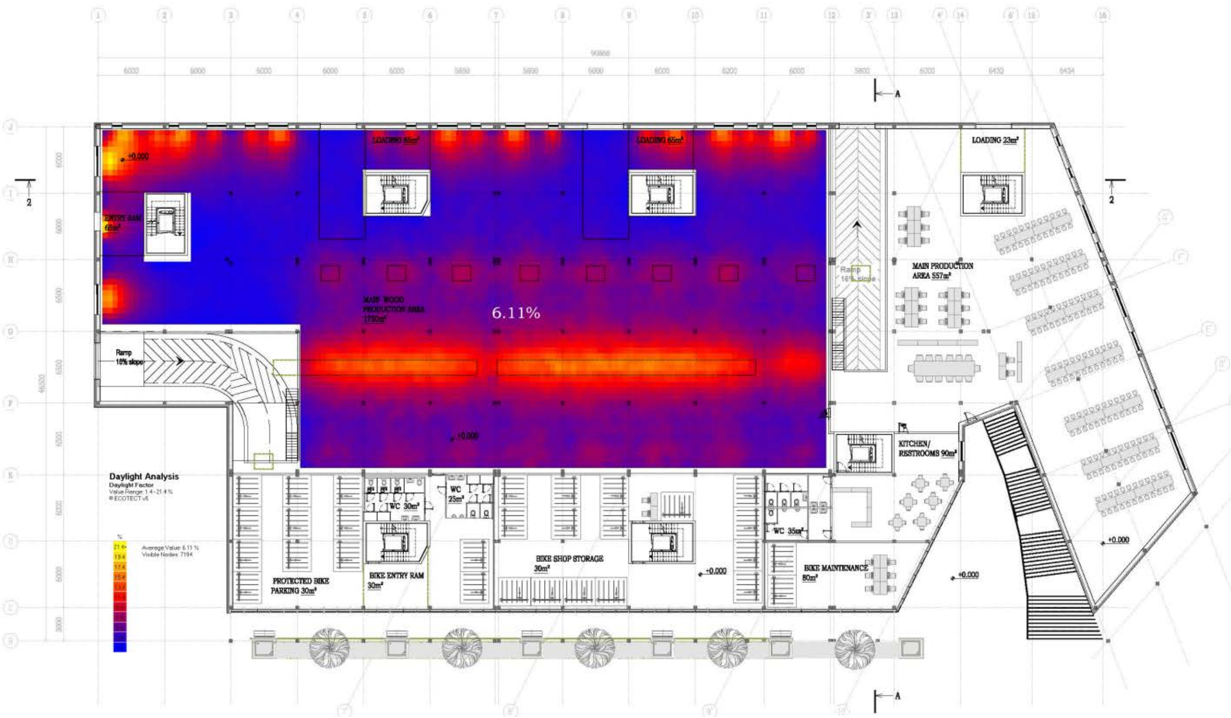



*Fig.38. Natural daylight factor analysis at Manufacturing spaces*


*Fig.39. Natural daylight factor analysis in the loft and on the ground floor*


*Fig.40. Natural daylight factor analysis on the 2nd floor at residential spaces*

In the residential building it is required to have a natural daylight factor not lower than around 2%, outside being 100%. For this reason, using Ecotect, analyses were made to understand the importance of window positioning in the aspect of natural light.

A quick initial overall analyze of the building was made and showed low values in some parts of the building, it was clear skylights were needed in those parts where regular wall – placed windows is not possible. Skylights were added and further, more detailed analysis preformed.

The diagram shows the availability of natural light in the different rooms of the building from different sides of the building and with different windows. From this “single room” analyze it was understood that an acceptable set of result were obtained.

By adding skylights where needed, Daylight Factor requirements of around 2% were met. Adding windows does not only effect the natural lightning but also other aspects like energy demands (more window, higher solar gains during summer and larger cooling loads), U-values (windows generally have much higher values than walls and roofs) which leads to higher heat losses during winter and not the least the architectural point of view.

# CHAPTER 5. STRUCTURAL DESIGN

## 1. CONSTRUCTION TYPE

The design project must be conceived in structural timber. A strategy should be considered that evaluates a method for taking advantage of timber’s properties and characteristics in order to conceptualize and propose a critical evaluation of the design solution.

## 2. TIMBER AS A BUILDING MATERIAL

As a natural material, timber is unique, innovative and easy to handle. It is sustainable, environmentally friendly, can be readily recycled, and as sawn sections or quality controlled engineered products, timber has a large potential market for use as a structural and building material.

### 2.1. BENEFITS OF TIMBER AS BUILDING MATERIAL

+ Natural
  Timber is a natural building material. It is not toxic, does not leak chemical vapour into the building and is safe to handle and touch. It also means that as timber ages, it does so naturally and doesn’t turn into environmentally damaging materials.
+ Renewable
  Timber all the time is being grown in our forests and plantations. As long as new trees are planted, timber will continue to be available.
+ Low in production energy
  It takes very little energy to produce the timber used in building. This means that the embodied energy in timber is very low, the lowest of almost all common building materials.

+ A store for carbon
  Timber is made from carbon drawn from the atmosphere. This carbon would otherwise be adding to the greenhouse effect. Using timber in buildings stores the carbon for as long as the building stands or the timber is used.
+ A very good insulator
  Timber is a natural insulator and can reduce energy needs. Wood is twelve times more insulating than concrete. Reducing the amount of energy used to heat and operate a building, insulation is very important.
+ Thermal Properties
  Many materials change in size and volume as the temperature changes. They expand with increasing of the temperature. This means linear and volumetric expansion. The expansion causes decrease in the strength of materials.
  Wood does not practically expand against heat. On the contrary, by the effect of heat, it dries out and gains strength. The only time wood expands a little is when the humidity level is below 0%, and this is only scientifically significant. In practice, the humidity level of wood does not drop under 5% even in the driest climate.
  The coefficient of thermal conductivity of the wood is very low. For this reason, wood is used for making matches, handles of hardware equipment, ceilings and wall coverings.
  Specific heat of wood is high. That means high amount of energy is needed to increase and decrease the temperature of one kilogram of wood. Wood requires almost twice amount of heat energy than stones and concrete; similarly, three times of energy is needed for heating or cooling steel.
+ Acoustic Properties
  Sound isolation is based on the mass of the surface. Wood, as a light material, is not very perfect for sound isolation, but it is ideal for sound absorption. Wood prevents echo and noise by absorbing sound.
+ Readily available
  Timber is milled all over USA and is often used close to where it is produced, moreover there is a wood manufactory in New York. This promotes local economies and reduces the energy needed to transport materials long distances.
+ Easy to work
  Timber is versatile and can be used in a wide variety of ways. Being light, it is easy to install and can be worked with simple equipment. This reduces the energy needed for construction.
+ Natural Air conditioning effect
  Wood reacts constantly to its surrounding environment. If the humidity is high, wood absorbs moisture. If the humidity is low, it emits moisture. Wood acts as a "natural air conditioner," keeping room temperature constant.
+ Durability of wood in relation to the age of the tree
  The older the tree, the stronger the wood it produces. From the time the tree is harvested, the cellulose molecule structure which gives the wood its strength loses the water in its cells. This causes crystallization, and makes the wood strong. When wood is as old as the tree was at harvest time, it is at its strongest. Wood is indeed a very durable material.
+ Light yet strong

The energy of an earthquake is proportionate to the weight of a building. The heavier the building, the more it is affected by the earthquake. Therefore, a wooden house is more earthquake resistant.

+ Relaxing environment

  It has been scientifically proven that a kind of terpene substance emitted by wood, which is used for protection against invaders, is effective in relaxing the mind and body. This is the so-called "forest bath" effect.

+ Electrical properties

  Resistance to electrical current of a completely dry wood is equal to that of fhenol formaldehit. An oven dried wood is a very good electrical insulator. To some extent air dried wood is the same. Unfortunately electrical resistance of wood is lowered by increasing the moisture content. The resistance of wood saturated with water. Static electricity that is dangerous for human health is not observed in wood. For this reason wood is preferred as a healthy material.

+ Aesthetic properties

  Wood is a decorative material when considered as an aesthetic material. Each tree has its own color, design and smell the design of a tree does change according to the way it is sliced. It is possible to find different wooden materials according to color and design preference. It can be painted to darker colors of varnished, and can be given bright or mat touches.

+ Oxidation properties

  Although wood has oxidation characteristics in some way, it is not the kind of oxidation seen in metals. Metals get rust, wood doesn’t. For such characteristics, use of wood is preferred to avoid rust when necessary.

+ Variation

  There are more than 5000 kinds of woods in the world. Their specific gravity, macroscopic and microscopic structures are different. Accordingly, their physical, thermal, acoustic, electrical and mechanical properties are also different. Because of this variety, it is possible to find wood suitable for needs. For instance, for heat isolation and sound absorption woods in lightweight are used. Similarly, heavy ones are used for construction purposes.

## 2.2. DISADVANTAGES OF TIMBER AS BUILDING MATERIAL

– Shrinkage and swelling of wood

  Wood is a hygroscopic material. This means that it will adsorb surrounding condensable vapors and loses moisture to air below the fiber saturation point.

– Biotic deterioration of wood

  Like any organic good, wood is a nutritional product for some plants and animals. Humans cannot digest cellulose and other fiber ingredients of wood, but some fungi and insects can digest it, and use it as a nutritional product. Insects drill holes and drive lines into wood. Even more dangerously, fungi cause the wood to decay partially and even completely. Biological deterioration of wood due to attack by decay fungi, woodboring insects and marine borers during its processing and in service has technical and economical importance.

- Fungi

  Physiological requirements of wood destroying and wood inhabiting fungi:

A favorable temperature must be 25-30°C for optimum growth of most wood rotting fungi. But some of them can tolerate temperature between 0-45°C.

Oxygen is essential for the growth of fungi. In the absence of oxygen no fungi will grow. It is well known that storage of wood under water will protect them against attacks by fungi. Generally wood will not be attacked by the common fungi at moisture contents below the fiber saturation point. The fiber saturation point (FSP) for different wood lies between 20 to 35% but 30% is accepted generally. It is recommended that wood in service must have a moisture content at least 3% less than FSP to provide desirable safety against fungi.

Wood is an organic compound and consists of 50% carbon. That means that wood is a very suitable nutrient for fungi because fungi derive their energy from oxidation of organic compounds. Decay fungi wood rotters can use polysaccharides while stain fungi evidently require simple forms such as soluble carbohydrates, proteins and other substances present in the parenchyma cell of sapwood. Additionally, the presence of nitrogen in wood is necessary for the growth of fungi in wood.

- Insects

Insects are only second to decay fungi in the economic loss they cause to lumber and wood in service. Insects can be separated into four categories: Termites, powderpost beetles, carpenter ants and marine borers.

- Termites

There are two types of termites: Subterranean termites damage wood that is untreated, moist, in direct contact with standing water, soil, other sources of moisture. Dry wood termites attack and inhabit wood that has been dried to moisture contents as low as 5 to 10%. The damage by dry wood termites is less than subterranean termites.

- Powderpost beetles

Powderpost beetles attack hardwood and softwood. At risk is well seasoned wood as well as freshly harvested and undried wood.

- Carpenter ants

Carpenter ants do not feed on wood. They tunnel through the wood and create shelter. They attack most often wood in ground contact or wood that is intermittently wetted.

- Carpenter bees

They cause damage primarily to unpainted wood by creating large tunnel in order to lay eggs.

- Marine borers

They attack and can rapidly destroy wood in salt water and brackish water.

– Abiotic Deterioration of wood:

- Fire:

Another disadvantage of wood is that it easily catches fire. Wood consists of organic compounds which are composed mainly of carbon and hydrogen. They can combine with oxygen and burns. Because of these properties, wood is classified as a combustible material.

Using thick wood as a structure element is another way of extension of burning point. Outer surface burns and turns into charcoal. Charcoal, which forms on the surface of wood as it burns, is a very effective heat insulator. Therefore large timbers burn very slowly. In addition to this, wood is very good heat insulator too. The outer surface of the wood is 1000°C and the interior part is still 40°C when a piece of thick wood is burning. For this reason, buildings with thick structure elements such as beams and columns do not collapse easily on fire.

## 2.3. MINIMIZING THE PROBLEMS OF WOOD

- Careful selection of wood

Some species have naturally decay resistant heartwood. Such species include sweet chestnut, oak, juniper. Sapwood is never naturally durable species has little or no decay resistance and must be treated if long-term durability is desired.

- Coating

Coating provides protection to wood used both indoors and outdoors. Coating prevents rapid uptake and loss of moisture and reduces shrinking and swelling that can lead to surface cracking and other problems. But coating does not totally prevent changes in moisture content. Coating slows, but does not halt moisture level. Coating with solid color or pigmented stains protects wood against ultraviolet rays.

The addition of fungicides to coating provides some protection against development of decay and mold fungi.

Deteriorating paint film actually increases the decay hazard. Cracked paint allows moisture to come into contact with wood surface, and poses a barrier to rapid and complete redrying.

- Drying

Generally wood will not be attacked by the common fungi at moisture content below the fiber saturation point (FSP). Fungi cannot attack wood used indoor and in heated rooms, since the equilibrium moisture content (EMC) is much more below than FSP.

If wood is soaked in water, wood absorbs water and is saturated with it. Finally there will be no more oxygen in wood. In this situation fungi cannot grow in them. This is the main reason why woods are kept in water for a while. Besides underwater constructions, it is impossible to use woods completely wet; so when they are used out of water, they have to be completely dried out to EMC in order to protect them against fungi attack. In heated rooms, where the EMC lie between 5-10%, fungi cannot survive on them.

One of the most effective ways to prevent degradation of wood is to thoroughly dry it and keep it dry. The last case is very important since even wood that has been klin dried will readily regain moisture if placed in a humid environment.

Wood can be dried in air or in some type of dry klin. Air drying alone is not sufficient for wood items which are used in heated rooms. Therefore klin drying is necessary. Wood that will be used indoor need only be dried to provide for long term protection against rot.

- Treating With Wood Preservatives

Decaying of wood can be prevented by treating it with wood preservatives. But some of the wood preservatives may harm humans and other creatures. For this reason if wood is used outdoor in situations where it is often wet or in close contact to liquid water, then wood must be treated with wood preserving chemicals to achieve long term durability.

Wood preservatives are divided into two groups: waterborne and oilborne chemicals.

About 75% of wood that is commercially treated today is treated with waterborne salts, and CCA is the compound used in treating for the greatest volume of wood.

Oil based or oilborne preservatives are generally used for treating of wood used outdoors in industrial applications; such as ties, piling and poles.

In a serious situation, wood is treated with waterborne.

- Remedial treatment

Wood in service must be periodically retreated by brushing or a variety of other methods.

Retreatment of wood window frames, door frames and timber columns and beams is sometimes carried out by drilling holes in areas where decay has begun and filling these holes with a suitable treating compound. Treating compound in the form of solid rods is mostly preferred since it provides a slow release of active ingredients.

Retreatment of wood used in ground contact must be realized by application of pastes and wrapping with preservative impregnate bandages.

- Fire Retardants

It is impossible to make wood noncombustible like inorganic materials. In order to prevent potential dangers, wood can be processed in some fire retardants.

Fire retardants may be divided into two categories: coating and chemicals – water soluble salts – that are impregnated into the wood structure.

Coatings are used to reduce the formation of volatile, frammable gases by promoting rapid decomposition of the wood surface to charcoal and water. They also protect wood surface against high temperature water soluble salts. Wood can be impregnated by these chemicals. This type of process can contribute to the increase of the burning point and retard spread and penetration of flame.

Fire retardants only reduce the flammability of wood and slow or eliminate progressive combustion. They do not prevent burning totally in the presence of an external source of fire. In this case, wood does not go on burning once an external source of flames is removed.

## 3. DESCRIPTION OF A STRUCTURE OF THE BUILDING

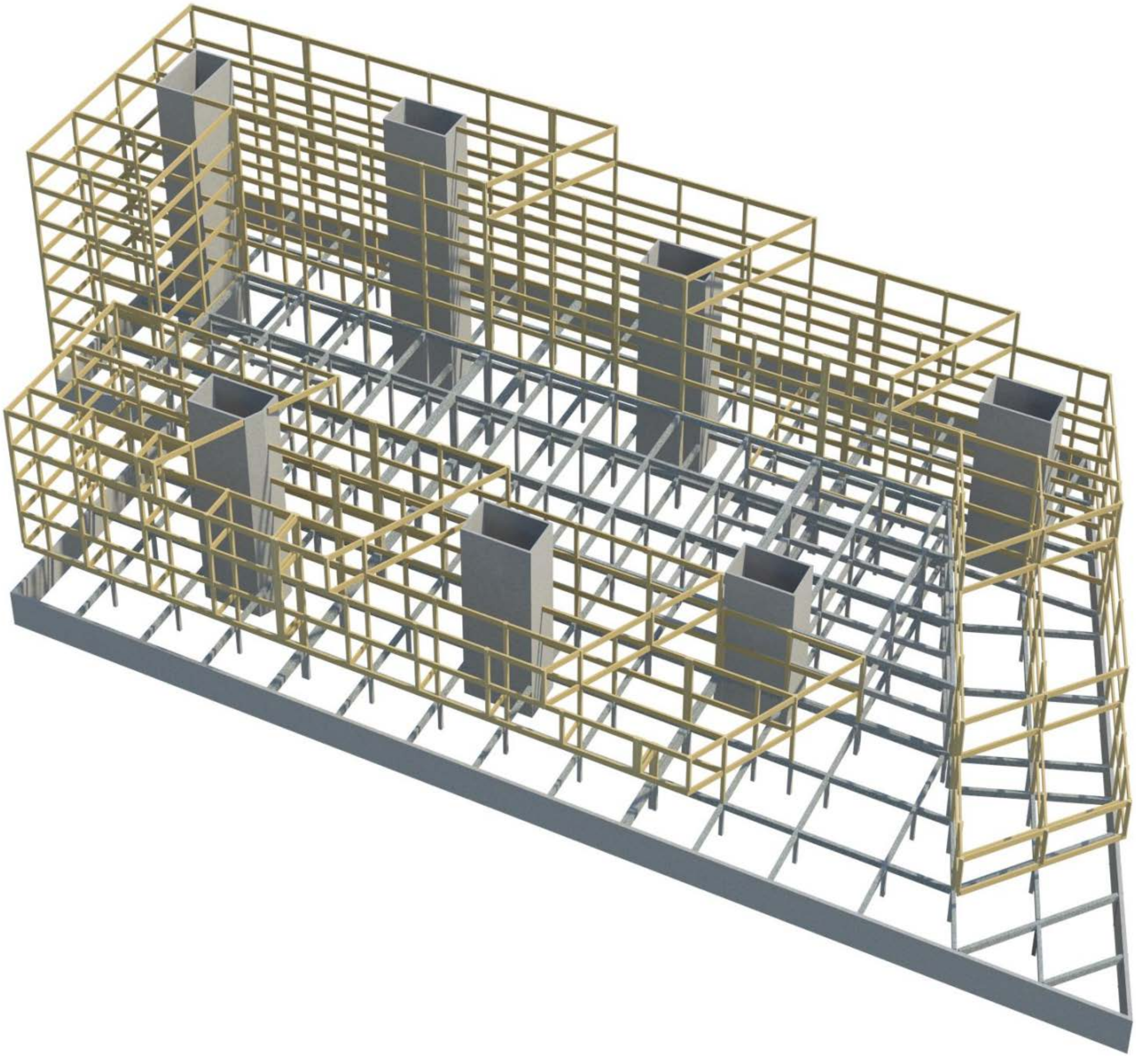


*Fig.41. Axonometric structure*

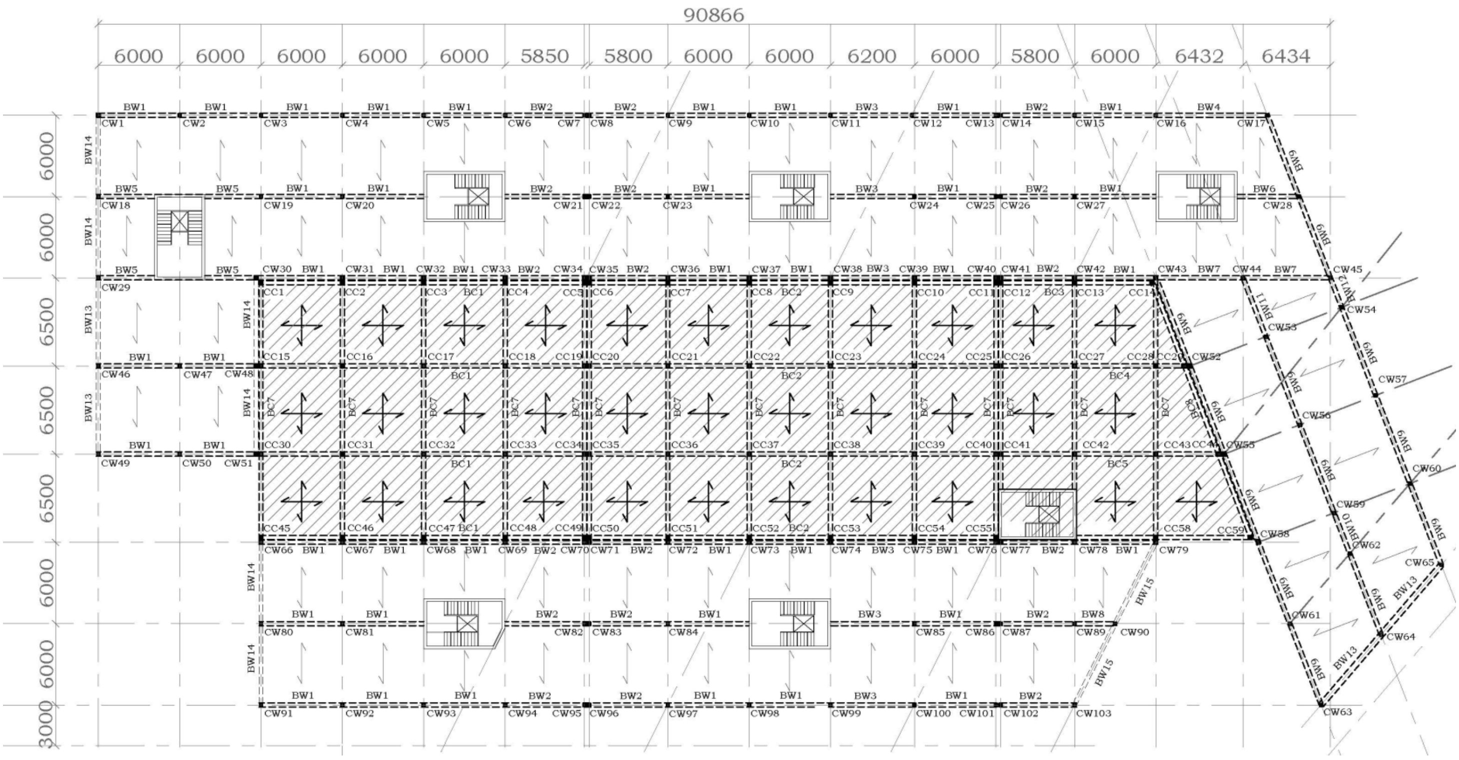


*Fig.42. Structural plan*

*Fig.43. Section of the building*

Structure of the building is composed of two tall timber parts and one part inside made of concrete. A reason to design an inner part of the building in concrete is a presence of a roof garden with massive trees and big amount of soil. This means that structure is going to be subjected to big loads and big amount of moisture that can be harmful for timber. Because of different behaviour of the materials structures are not connected, but are put next to each other leaving an expansion joint. Along the timber parts of the structure are placed bracing structures – concrete core with staircase and elevator. The entire basement with retaining walls and foundation are made of concrete.

Because of a big length of the building expansion joints have to be organised. For this reason elongation of the structure should be calculated.

$$\Delta l = \alpha \cdot \Delta T \cdot l$$

where Δl – elongation of a structure [mm]

α – linear expansion coefficient [1/K]

ΔT – difference of temperatures [K]

L – length of a structure [mm]

Elongation of timber structure due to thermal fluctuations from summer to winter:

$$\Delta l = \alpha \cdot \Delta T \cdot l = 5 \cdot 10^{-6} \cdot \big(38 - (-7)\big) \cdot 90866 = 20{,}4\, mm$$

Elongation of timber structure due to thermal fluctuations during a winter day:

$$\Delta l = \alpha \cdot \Delta T \cdot l = 5 \cdot 10^{-6} \cdot \big(2 - (-5)\big) \cdot 90866 = 3{,}2\, mm$$

Elongation of timber structure due to thermal fluctuations during a summer day:

$$\Delta l = \alpha \cdot \Delta T \cdot l = 5 \cdot 10^{-6} \cdot (29 - 20) \cdot 90866 = 4{,}1\, mm$$

Elongation of concrete structure due to thermal fluctuations from summer to winter:

$$\Delta l = \alpha \cdot \Delta T \cdot l = 12 \cdot 10^{-6} \cdot \big(38 - (-7)\big) \cdot 72082 = 38{,}9\, mm$$

Elongation of concrete structure due to thermal fluctuations during a winter day:

$$\Delta l = \alpha \cdot \Delta T \cdot l = 12 \cdot 10^{-6} \cdot \big(2 - (-5)\big) \cdot 72082 = 6{,}1\, mm$$

Elongation of concrete structure due to thermal fluctuations during a summer day:

$$\Delta l = \alpha \cdot \Delta T \cdot l = 12 \cdot 10^{-6} \cdot (29 - 20) \cdot 72082 = 7{,}8\, mm$$

To accumulate these variations of length two expansion joints are introduced.

## 4. DESIGN OF STRUCTURAL ELEMENTS.

Detailed calculations and analysis of the structures are obtained in Annex 4.

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# ANNEX 4

**STRUCTURAL DESIGN**

## 1. MATERIALS

### 1.1. CONCRETE

Concrete strength class: C40/50 (according to exposure class XS1)

Characteristic cylinder compressive strength

$$f_{ck} = 40\ \text{N/mm}^2$$

Design compressive strength [EC2 – 3.1.6(1) and Table 2.1N for $\gamma_C$]

$$f_{cd} = \alpha_{cc}\frac{f_{ck}}{\gamma_c} = 0{,}85 \cdot \frac{40}{1{,}5} = 22{,}67\frac{N}{mm^2}$$

Allowable compressive stress under characteristic combination of actions [EC2 – 7.2(2)]

$$\sigma_{c,adm} = k_1 f_{ck} = 0{,}6 \cdot 40 = 24\frac{N}{mm^2}$$

Medium tensile strength [EC2 – Table 3.1]

$$f_{ctm} = 0{,}3(f_{ck})^{\frac{2}{3}} = 0{,}3 \cdot (40)^{\frac{2}{3}} = 3{,}51\frac{N}{mm^2}$$

Characteristic tensile strength [EC2 – Table 3.1]

$$f_{ctk;0,05} = 0{,}7 f_{ctm} = 0{,}7 \cdot 3{,}51 = 2{,}46\frac{N}{mm^2}$$

Design tensile strength [EC2 – 3.1.6(2) and Table 2.1N for $\gamma_C$]

$$f_{ctd} = \alpha_{ct}\frac{f_{ctk;0,05}}{\gamma_c} = 1{,}0 \cdot \frac{2{,}46}{1{,}5} = 1{,}64\frac{N}{mm^2}$$

Secant modulus of elasticity [EC2 – Table 3.1]

$$E_{cm} = 22\left(\frac{f_{cm}}{10}\right)^{0,3} = 22\left(\frac{f_{ck}+8}{10}\right)^{0,3} = 22\left(\frac{40+8}{10}\right)^{0,3} = 35000\frac{N}{mm^2}$$

### 1.2. HIGH DUCTILITY STEEL TYPE B450C

Characteristic yield strength

$$f_{yk} \geq 450\frac{N}{mm^2}$$

Design yield strength [EC2 – 3.2.7 and Table 2.1N for $\gamma_S$]

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{450}{1{,}15} = 391\frac{N}{mm^2}$$

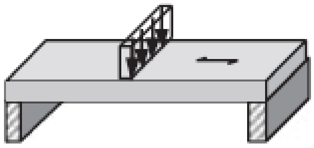
Admissible stress under characteristic combination of actions [EC2 – 7.2(5)]

$$\sigma_{s,adm} = k_3 \cdot f_{yk} = 0{,}8 \cdot 450 = 360\frac{N}{mm^2}$$

Modulus of elasticity [EC2 – 3.2.7(4)]

$$E_s = 200000\frac{N}{mm^2}$$

### 1.3. CLT panel



Technical characteristics perpendicular to the span of the panel.

Characteristic strength in bending parallel to grain

$f_{m,k}$ = 24,0 N/mm²

Characteristic strength in tension perpendicular to grain

$f_{t,90,k}$ = 0,12 N/mm²

Characteristic strength in compression parallel to grain

$f_{c,0,k}$ = 21,0 N/mm²

perpendicular to grain

$f_{c,90,k}$ = 2,7 N/mm²

Characteristic strength in shear parallel to grain

$f_{v,k}$ = 2,7 N/mm²

perpendicular to grain

$f_{R,v,k}$ = 1,5 N/mm²

Modulus of elasticity parallel to grain

$E_{0,mean}$ = 12000 N/mm²

perpendicular to the grain

$E_{90,mean}$ = 370 N/mm$^2$

Shear modulus parallel to grain

$G_{mean}$ = 690 N/mm$^2$

perpendicular to the grain

$G_{R,mean}$ = 50 N/mm$^2$

### 1.4. GLULAM

Characteristic bending strength $f_{m,g,k}$ = 32 N/mm$^2$

Characteristic shear strength $f_{v,g,k}$ = 2,7 N/mm$^2$

Characteristic bearing strength $f_{c,90,g,k}$ = 2,7 N/mm$^2$

Characteristic compression strength parallel to the grain $f_{c.0.g.k}$ = 29 N/mm$^2$

Mean modulus of elasticity parallel to grain $E_{0,g,mean}$ = 13,7 kN/mm$^2$

Mean shear modulus $G_{0.g.mean}$ = 0,78 kN/mm$^3$

## 2. LOADS

### 2.1. SELF-WEIGHT OF STRUCTURAL AND NON-STRUCTURAL ELEMENTS

The following loads refer to a square meter of a typical floor.

#### 2.1.1. VERTICAL CLOSURES

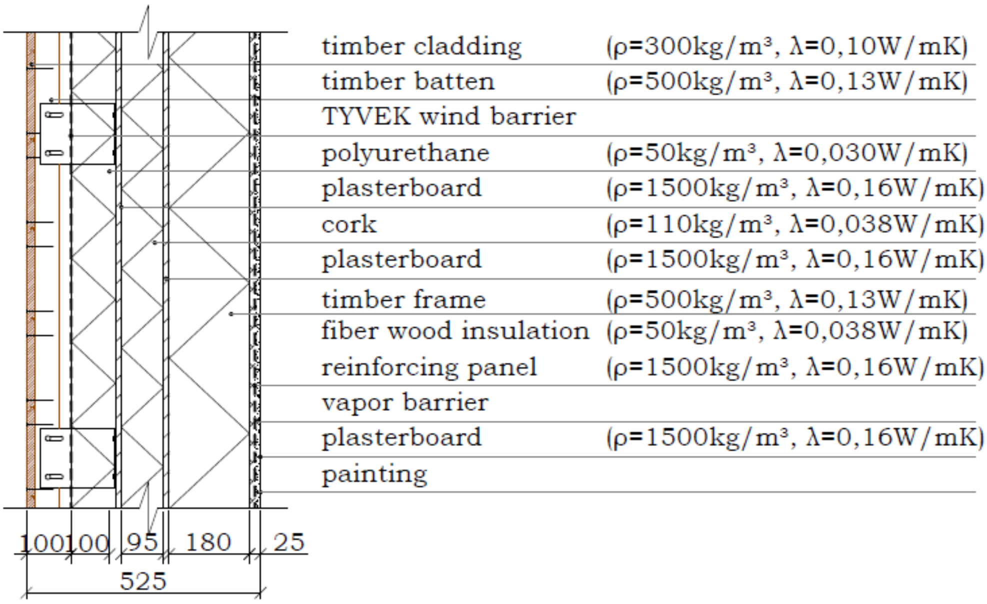



*Fig. 1. Typical external wall with timber cladding*

| Layer | Thickness [cm] | Specific weight [kN/m³] | Weight [kN/m²] |
|---|---|---|---|
| Timber cladding | 2 | 4,7 | 0,006 |
| Timber battens (every 60 cm) | 0,55 | 5,1 | 0,028 |
| Polyurethane | 10 | 0,5 | 0,050 |
| Plasterboard | 1,25 | 15,0 | 0,188 |
| Cork | 9,5 | 1,1 | 0,105 |
| Plasterboard | 1,25 | 15,0 | 0,188 |
| Timber frame (every 60 cm) | 18 | 5,1 | 0,184 |
| Fiber wood insulation | 18 | 0,8 | 0,144 |
| Double layer of plasterboard | 2,5 | 15,0 | 0,375 |
| | | Total: | 1,267 |

*Tab. 1. Self – weight of external wall with timber cladding*

For a floor height of (4,00 – 0,183) m = 3,817 m (0,183 m = slab thickness) the linear weight of the wall is

$$1{,}267 \cdot 3{,}817 = 4{,}836\ kN/m$$

Assuming 20% of openings the total linear weight of the wall is

$$4{,}836 - 0{,}2 \cdot 4{,}836 = 3{,}869\ kN/m$$

The load of the external walls is directly applied on the slabs along the perimeter.

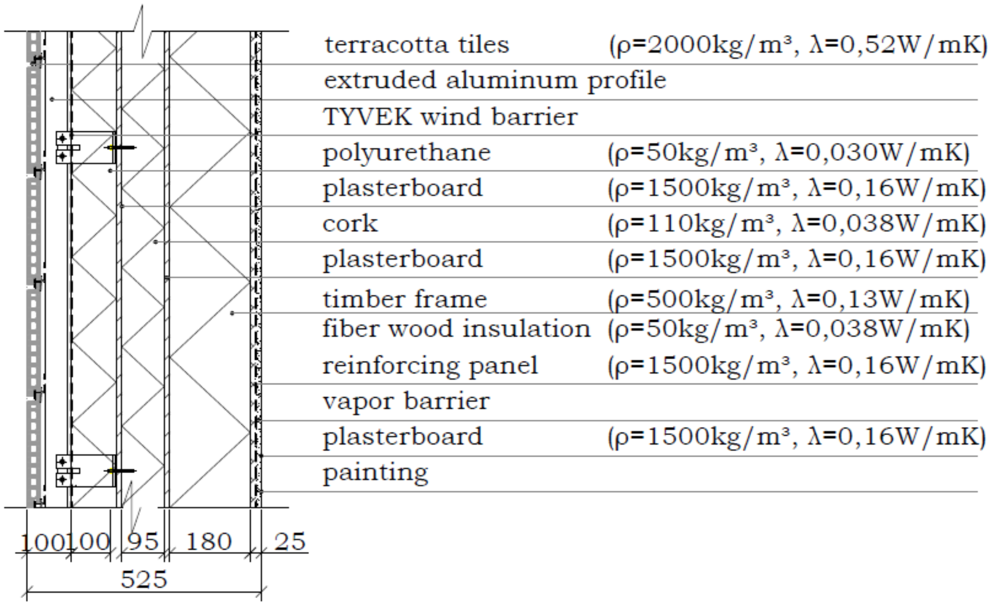



*Fig.2. Typical external wall with different cladding*

| Layer | Thickness [cm] | Specific weight [kN/m³] | Weight [kN/m²] |
|---|---|---|---|
| Ventilated terracotta façade system | 3 | 20,0 | 0,600 |
| Polyurethane | 10 | 0,5 | 0,050 |
| Plasterboard | 1,25 | 15,0 | 0,188 |
| Cork | 9,5 | 1,1 | 0,105 |
| Plasterboard | 1,25 | 15,0 | 0,188 |
| Timber frame (every 60 cm) | 18 | 5,1 | 0,184 |
| Fiber wood insulation | 18 | 0,8 | 0,144 |
| Double layer of plasterboard | 2,5 | 15,0 | 0,375 |
| | | Total: | 1,832 |

*Tab.2. Self – weight of external wall with timber cladding*

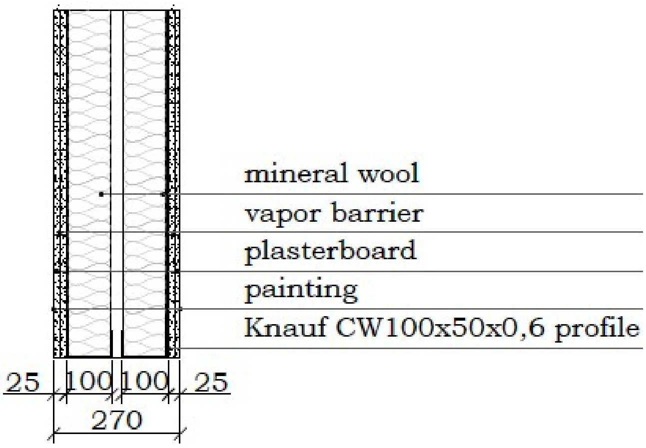
For a floor height of (4,00 – 0,183) m = 3,817 m (0,183 m = slab thickness) the linear weight of the wall is

$$1{,}832 \cdot 3{,}817 = 6{,}993\ kN/m$$

Assuming 20% of openings the total linear weight of the wall is

$$6{,}993\ - 0{,}2 \cdot 6{,}993\ = 5{,}594\ kN/m$$

The load of the external walls is directly applied on the slabs along the perimeter.




| Layer | Thickness | Specific weight | Weight |
|---|---|---|---|
| | [cm] | [kN/m³] | [kN/m²] |
| Double layer of plasterboard | 2,5 | 15,0 | 0,375 |
| Knauf drywall system | 10 | 1,2 | 0,12 |
| Knauf drywall system | 10 | 1,2 | 0,12 |
| Double layer of plasterboard | 2,5 | 15,0 | 0,375 |
| | | Total: | 0,99 |

*Tab.3. Self-weight of internal partition*

*Fig.3. Typical internal partition (Knauf drywall system)*

Assuming a net floor height of 2,82 m the linear weight of the wall is

$$0{,}99 \cdot 2{,}82 = 2{,}792\ kN/m$$

EN 1991-1-1 [§ 6.3.1.2(8)] permits to consider an equivalent uniformly distributed load all over the floor, instead of the free action of movable partitions, if the slab can well redistribute the load transversally. The nominal value of this uniform load is given in function of the linear self-weight of the wall considered for movable partitions with a self-weight ≤ 3,0 kN/m wall length - $q_k$ = 1,2 kN/m²

It has to be specified that this equivalent uniform load has to be considered as a live load with partial safety factor $\gamma_Q = 1{,}5$ (=0 where favourable) for Ultimate Limit State (ULS) combinations and coefficients $\psi_0 = \psi_1 = \psi_2 = 1{,}0$ for Serviceability Limit State (SLS) combinations.

### 2.1.2. HORIZONTAL CLOSURES

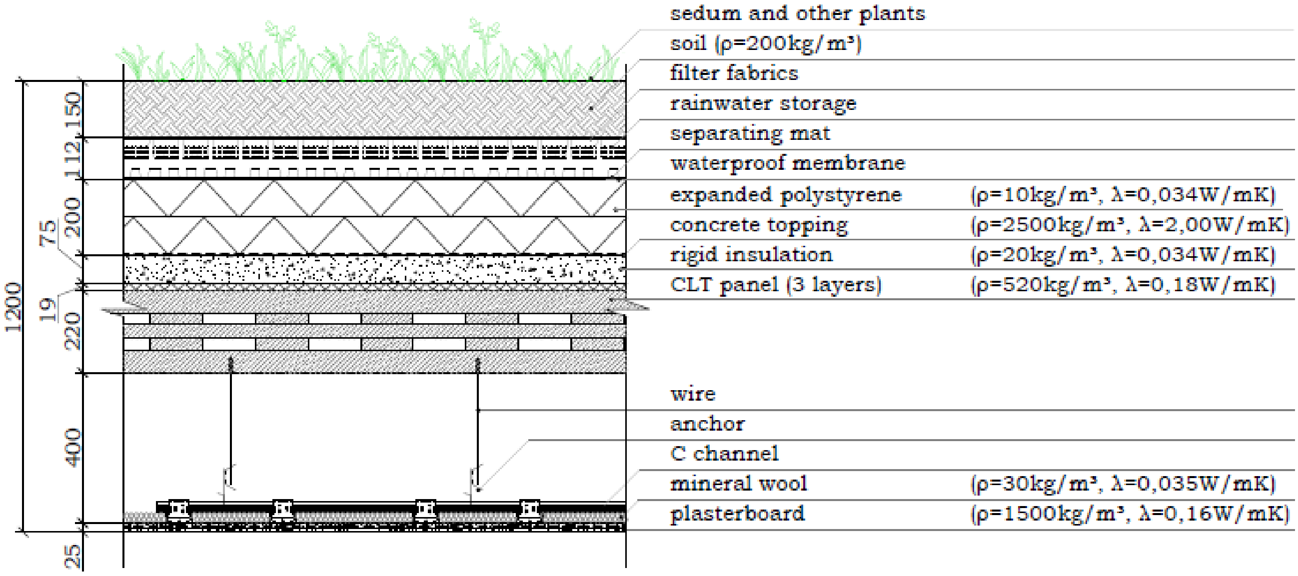



*Fig.4. Typical green roof solution over the timber part of the building*

| Layer | Thickness | Specific weight | Weight |
|---|---|---|---|
| | [cm] | [kN/m³] | [kN/m²] |
| Soil | 15,0 | 16,0 | 2,400 |
| Rainwater storage (saturated) | 10,0 | | 0,09 |
| Separating mat | 1,2 | | 0,45 |
| Expanded polystyrene | 20,0 | 0,3 | 0,06 |
| Concrete topping | 7,5 | 25,0 | 1,875 |
| Rigid insulation | 1,25 | 0,3 | 0,004 |
| CLT panel | 22,0 | 5,2 | 1,144 |
| Mineral wool | 2,5 | 0,8 | 0,020 |
| Double layer of plasterboard | 2,5 | 15,0 | 0,375 |
| | | Total: | 6,418 |

*Tab.4. Self-weight of the green roof*

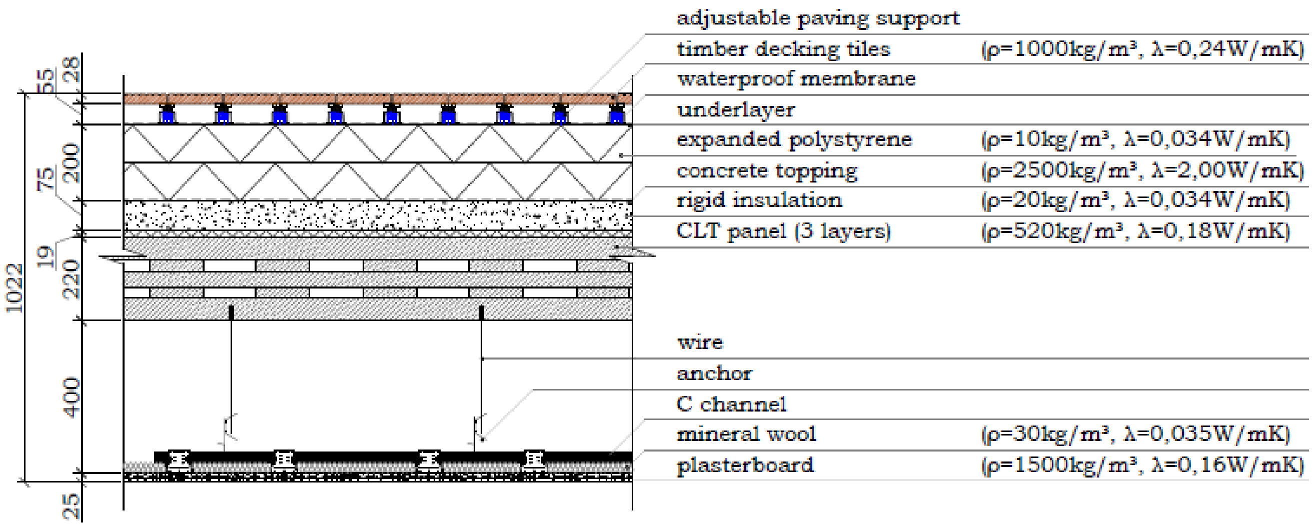



*Fig.5. Typical accessible roof solution over the timber part of the building*

| Layer | Thickness | Specific weight | Weight |
|---|---|---|---|
| | [cm] | [kN/m³] | [kN/m²] |
| Timber paving | 2,8 | 7,5 | 0,21 |
| Expanded polystyrene | 20,0 | 0,3 | 0,06 |
| Concrete topping | 7,5 | 25,0 | 1,875 |
| Rigid insulation | 1,25 | 0,3 | 0,004 |
| CLT panel | 22,0 | 5,2 | 1,144 |
| Mineral wool | 2,5 | 0,8 | 0,020 |
| Double layer of plasterboard | 2,5 | 15,0 | 0,375 |
| | | Total: | 3,688 |

*Tab.5 Self – weight of the accessible roof*

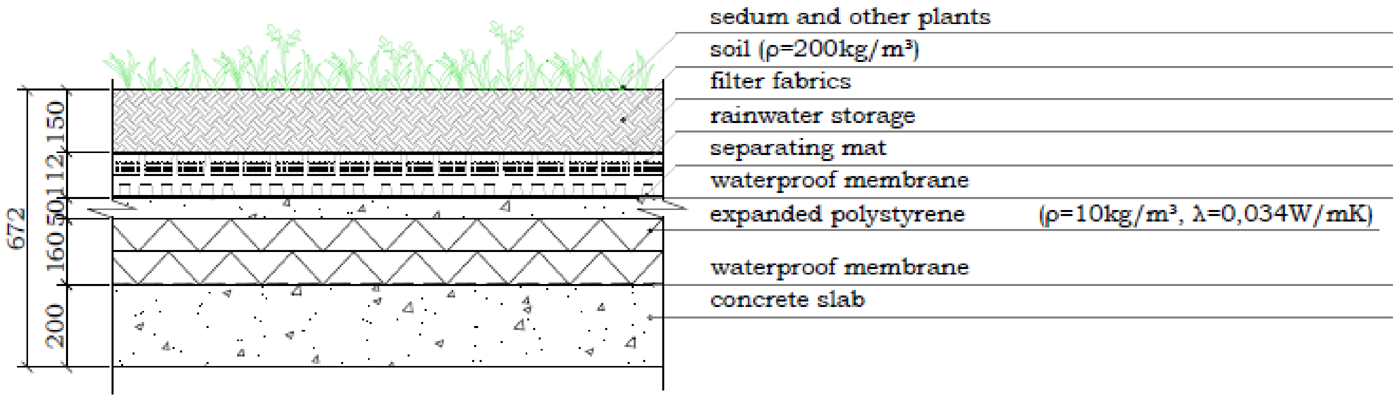



*Fig.6. Typical green roof solution over the concrete part of the building*

| Layer | Thickness [cm] | Specific weight [kN/m³] | Weight [kN/m²] |
|---|---|---|---|
| Soil | 15,0 | 16,0 | 2,40 |
| Rainwater storage (saturated) | 10,0 | | 0,09 |
| Separating mat | 1,2 | | 0,45 |
| Concrete topping | 5,0 | 25,0 | 1,25 |
| Expanded polystyrene | 16,0 | 0,3 | 0,048 |
| Concrete slab | 20,0 | 25,0 | 5,00 |
| | | Total: | 8,788 |

*Tab. 6.. Self – weight of the green roof*

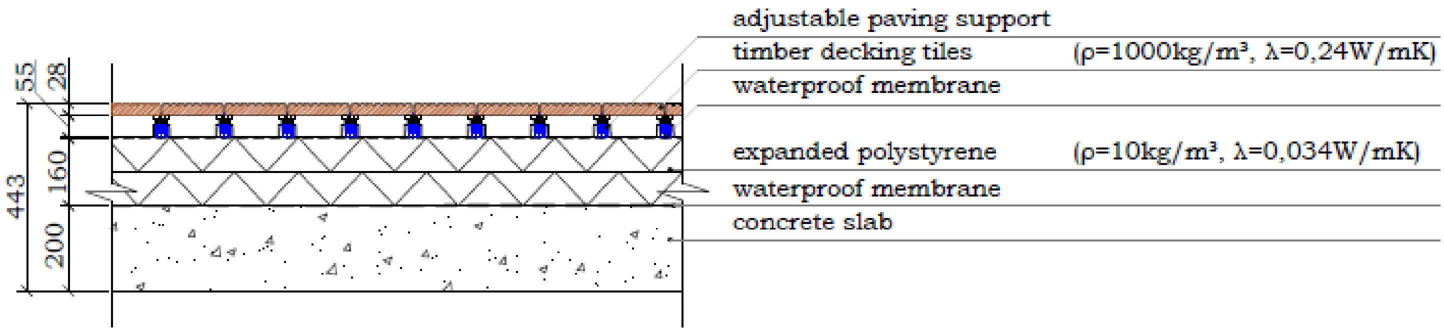



*Fig.7. Typical accessible roof solution over the concrete part of the building*

| Layer | Thickness [cm] | Specific weight [kN/m³] | Weight [kN/m²] |
|---|---|---|---|
| Timber paving | 2,8 | 7,5 | 0,210 |
| Expanded polystyrene | 2,0 | 0,3 | 0,006 |
| Concrete slab | 20,0 | 25,0 | 5,00 |
| | | Total: | 5,216 |

*Tab.7. Self-weight of the accesible roof*

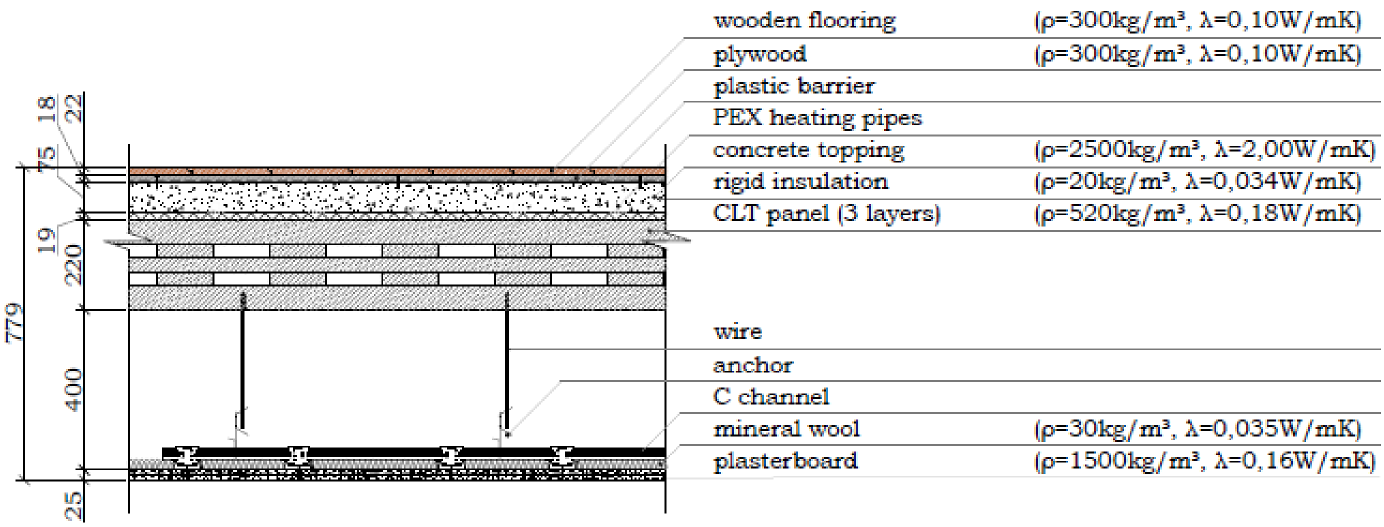



*Fig.8. Typical floor slab in the timber part of the building*

| Layer | Thickness [cm] | Specific weight [kN/m³] | Weight [kN/m²] |
|---|---|---|---|
| Wooden flooring | 2,2 | 7,2 | 0,156 |
| Plywood | 1,8 | 5,8 | 0,104 |
| Concrete topping | 7,5 | 25,0 | 1,875 |
| Rigid insulation | 1,25 | 0,3 | 0,004 |
| CLT panel | 22,0 | 5,2 | 1,144 |
| Mineral wool | 2,5 | 0,8 | 0,020 |
| Double layer of plasterboard | 2,5 | 15,0 | 0,375 |
| | | Total: | 3,681 |

*Tab.8. Self – weight of the typical floor slab*

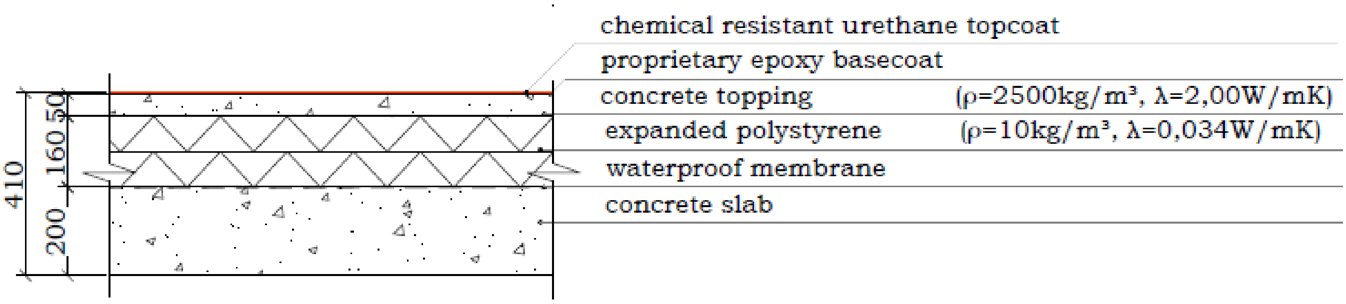



*Fig.9. Typical floor slab in the concrete part of the building*

| Layer | Thickness [cm] | Specific weight [kN/m³] | Weight [kN/m²] |
|---|---|---|---|
| Concrete topping | 5,0 | 25,0 | 1,250 |
| Expanded polystyrene | 16,0 | 0,3 | 0,048 |
| Concrete slab | 20,0 | 25,0 | 5,00 |
| | | Total: | 6,298 |

*Tab.9. Self – weight of the typical concrete slab*

### 2.2. IMPOSED LOADS

The imposed load for floors in residential buildings for category A is [EN 1991-1-1 §6.3.1.2, Tables 6.1 and 6.2, in accordance with National Annex]

2,00 kN/m$^2$

The imposed load for floors in industrial buildings for category E2 is [EN 1991-1-1 §6.3.1.2, Tables 6.3 and 6.4, in accordance with National Annex]

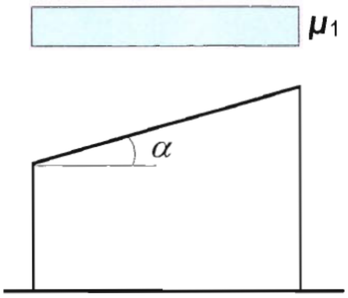
7,50 kN/m$^2$

### 2.3. SNOW LOADS

EN 1991-1-3 with specifications according to the National Annex dated 24-11-2004 apply for the persistent design situation the snow load on the roof is expressed by the formula [Expression 5.1-EC1-1-3]:

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k$$

where $\mu_i$ is the snow load shape coefficient equal to 0,8 for a flat roof with an angle of the pitch less than 30° [EN1991-1-3 §5.3.2 e 5.3.3 - Figure 5.1]




According to EN 1991-1-3 load combinations to be considered in the design are shown in Fig.10. This load combination will be considered in order to estimate the maximum load on columns coming from the roof.

*Fig.10. Snow load combination*

$C_e$ is the exposure coefficient function of the topography of the site. $C_e$ = 0,8 for Windswept topography, that is: “flat unobstructed areas exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees .” [EN1991-1-3 § 5.2.(7) – Table 5.1 in accordance with National Annex]

$C_t$ is the thermal coefficient that should be used to account for the reduction of snow loads on roofs with high thermal transmittance (> 1 W/m$^2$K). $C_t$ = 1,0 unless otherwise specifications [EN1991-1-3 § 5.2.(8) and National Annex].

$s_k$ = 25 lb/ft$^2$ = 25 · 0,0479 kN/m$^2$ ≈ 1,2 kN/m$^2$ is the characteristic value of snow load on the ground for Red Hook, Brooklyn, New York for a design working life of the structure of 50 years in accordance with the initial design assumptions.


*Fig.11. Characteristic value of snow*

At the end the value of the snow load is

$$s = 0,8 \cdot 0,8 \cdot 1,0 \cdot 1,2 = 0,768\ \mathrm{kN/m2}$$

## 2.4. WIND LOADS

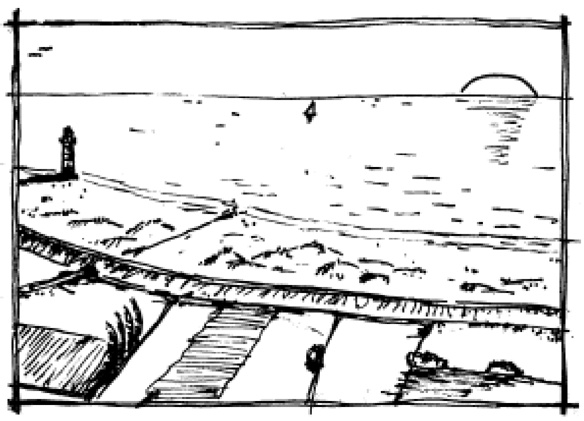


*Fig.12. Terrain category*

According to EN 1991-1-4 the following procedure applies.

The fundamental value of the basic wind velocity, $v_{b,0}$, is evaluated. $v_{b,0}$ is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in sea or coastal area exposed to the open sea (terrain category 0).

Basic values are characteristic values having annual probabilities of exceedence of 0,02, which is equivalent to a mean return period of 50 years.

Values are nominal design 3-second gust wind speeds in miles per hour.

$$v_{b,0} = 110 \cdot 0{,}447 \text{ m/s} = 49{,}2 \text{ m/s}$$

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


*Fig.13. Basic wind velocity*

where

$v_b$ is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category 0;

$v_{b,0}$ is the fundamental value of the basic wind velocity,

$c_{dir}$ is the directional factor,

$c_{dir}$ = 1,0,

$c_{season}$ is the season factor,

$c_{season}$ = 1,0.

$$v_b = 1{,}0 \cdot 1{,}0 \cdot 49{,}2 = 49{,}2\ m/s$$

Wind direction is parallel to the longest side of the building (X direction).

According to EN1991-1-4 § 7.2.2 a building, whose height h (28,0 m) is less than windward wall length b (46,5 m), the reference height $z_e$ should be considered to be one part.

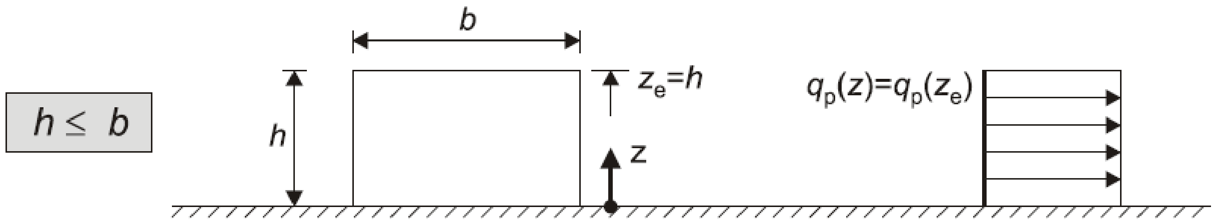



*Fig.14. Reference height*

The mean wind velocity $v_m(z)$ at a height $z$ above the terrain depends on the terrain roughness and orography and on the basic wind velocity, $v_b$, and should be determined:

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b$$

where $c_r(z)$ is the roughness factor;

$c_o(z)$ is the orography factor, taken as 1,0;

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

$$c_r(z) = c_r(z_{min}) \qquad for\ z \le z_{min}$$

where $z_e$ = 28,00 m

$z_0$ = 0,003 m (terrain category 0) is the roughness length;

$z_{0,II}$ = 0,05 m (terrain category II)

$z_{min}$ = 1,00 m (terrain category 0)

$z_{max}$ = 200 m

$k_r$ is a terrain factor depending on the roughness length $z_0$:

$$k_r = 0{,}19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0{,}07} = 0{,}19 \cdot \left(\frac{0{,}003}{0{,}05}\right)^{0{,}07} = 0{,}156$$

$$c_r(z_{min}) = k_r \cdot ln\left(\frac{z_{min}}{z_0}\right) = 0{,}156 \cdot ln\left(\frac{1{,}00}{0{,}003}\right) = 0{,}91$$

$$c_r(z_e) = k_r \cdot ln\left(\frac{z_e}{z_0}\right) = 0{,}156 \cdot ln\left(\frac{28{,}0}{0{,}003}\right) = 1{,}43$$

$$v_m(z_{min}) = 0{,}91 \cdot 1{,}0 \cdot 49{,}2 = 44{,}77\frac{m}{s}$$

$$v_m(z_e) = 1{,}43 \cdot 1{,}0 \cdot 49{,}2 = 70{,}36\frac{m}{s}$$

The turbulence intensity $I_v(z)$ (Expression 4.7-EC1-1-4) at height $z$ is defined as the standard deviation of the turbulence divided by the mean wind velocity.

The turbulent component of wind velocity has a mean value of 0 and a standard deviation $\sigma_v$. The standard deviation of the turbulence $\sigma_v$ may be determined:

$$\sigma_v = k_r \cdot v_b \cdot k_I$$

where $k_I$ is the turbulence factor, the recommended value for $k_I$ is 1,0.

$$\sigma_v = 0{,}156 \cdot 49{,}2 \cdot 1{,}0 = 7{,}68\, m/s$$

$$I_v(z_e) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_0(z) \cdot \ln\left(\frac{z}{z_0}\right)} = \frac{1{,}0}{1{,}0 \cdot \ln\left(\frac{28{,}00}{0{,}003}\right)} = 0{,}109 \qquad for\ z_{min} \leq z \leq z_{max}$$

$$I_v(z_{min}) = \frac{1{,}0}{1{,}0 \cdot \ln\left(\frac{1{,}00}{0{,}003}\right)} = 0{,}172 \qquad for\ z \leq z_{min}$$

The peak velocity pressure $q_p(z)$ at height z, which includes mean and short-term velocity fluctuations, can then be determined [Expression 4.8-EC1-1-4]:

$$q_p(z) = \left(1 + 7 \cdot I_v(z)\right) \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b$$

where $\rho$ = 1,25 kg/m$^3$ is the air density (raccomended value).

$$q_p(z_e) = (1 + 7 \cdot 0{,}109) \cdot \frac{1}{2} \cdot 1{,}25 \cdot 70{,}36^2 = 5454{,}87\ \frac{N}{m^2} = 5{,}45\ \frac{kN}{m^2}$$

$q_b = \frac{1}{2} \rho\, v_b^2$ is the basic velocity pressure [Expression 4.10-EC1-1-4]

$$q_b = \frac{1}{2} \cdot 1{,}25 \cdot 49{,}2^2 = 1512{,}9\ N/m^2 = 1{,}51\ kN/m^2$$

The exposure coefficient [Expression 4.9 and Figure 4.2 – EC1-1-4]:

$$c_e(z) = \left(1 + 7 \cdot I_v(z)\right) \cdot c_r(z)^2 \cdot c_0(z)^2 = (1 + 7 \cdot 0{,}109) \cdot 1{,}43^2 \cdot 1{,}0^2 = 3{,}59$$

The wind pressure acting on the external surfaces, $w_e$, can be obtain by the folllowing expression [Expression 5.1-EC1-1-4]:

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

where $q_p(z_e)$ is the peak velocity pressure

$z_e$ is the reference height for the external pressure;

$c_{pe}$ is the pressure coefficient for the external pressure that will be specified later on.

The wind force, $F_w$ acting on a structure or a structural element may be determined by vectorial summation of the forces acting on their reference surfaces [Expression 5.5-EC1-1-4]:

$$F_w = c_s \cdot c_d \cdot \sum_i w_{ei} \cdot A_i$$

where the structural factor $c_s c_d$ (separeted into a size factor $c_s$ and a dynamic factor $c_d$) is taken as 1,0 as reccomended for framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth [EC1-1-4 §6.2.(1) c].

The values of the pressure coefficients for different walls are evaluated looking at Table 7.1 – EC1-1-4 with a ratio h/d = 28,0/100,2 ≅ 0,28, where d is the parallel dimension of the building to the wind direction:

windward wall: $c_{pe}$ = 0,7;

leeward wall: $c_{pe}$ ≅ -0,31.

The lack of correlation of wind pressures between the windward and leeward side can be considered as follows. For buildings with h/d ≤ 1, the resulting force is multiplied by 0,85 [EN 1991-1-4 - 7.2.2 (3)]. Being h/d = 0,28 the value 0,85 is used.

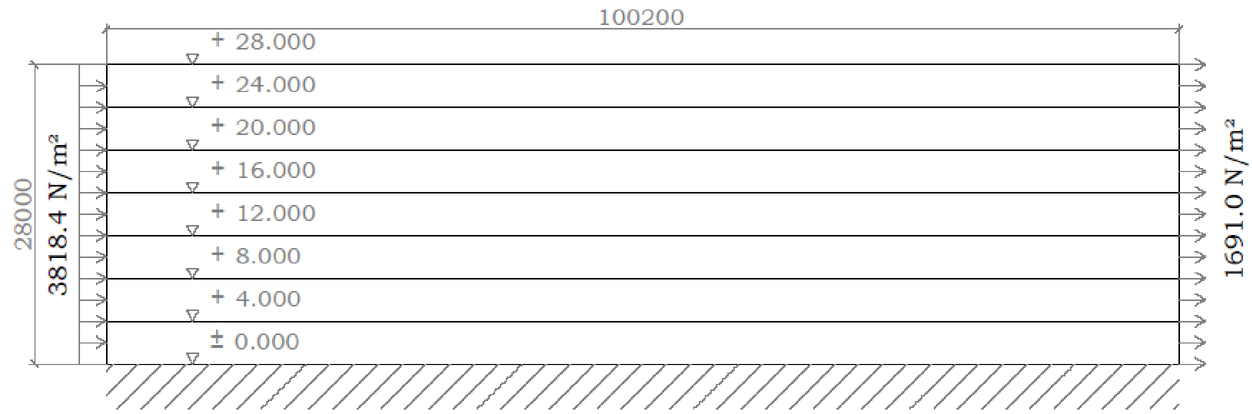



*Fig.15. Wind pressure and depressure (longest side)*

Wind direction is parallel to the shortest side of the building (Y direction).

The building height (28 m) is less than the dimension of the building in the hortogonal direction to the wind direction (100,2 m) and consequently an uniform distribution of pressure will be applied both on the windward and the leeward side. The reference height is assumed to be the maximum height of the bilding.

For a ratio h/d = 28/46,5 = 0,602 the pressure coefficients are [Table 7.1- EN 1991-1-4]:

windward wall: $c_{pe}$ = 0,747;

leeward wall: $c_{pe} \cong$ - 0,394.

Wind actions are therefore uniform distribution of pressure and depressure.

The coefficient that takes into account the lack of correlation of wind pressures between the windward and leeward side is taken equal to 0,85 with h/d =0,602 ≤ 1.

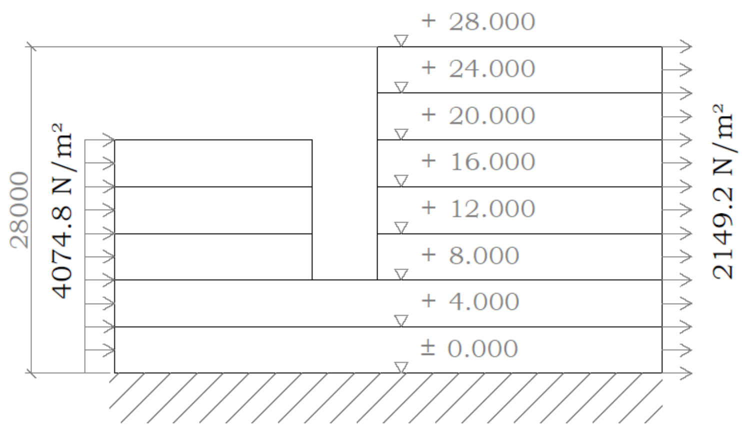



*Fig.16. Wind pressure and depressure (shortest side)*

## 2. SLABS

### 2.1. CLT PANEL.

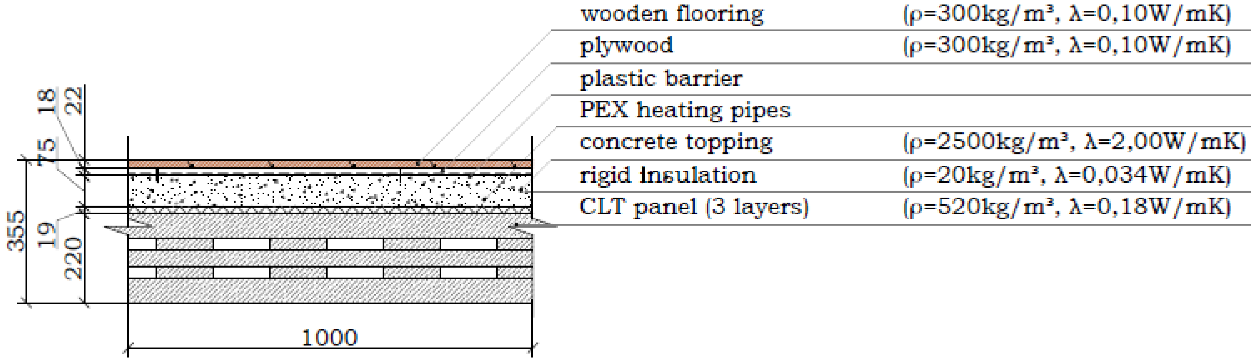



*Fig.17. Typical floor slab in timber part of the building*

APPLIED LOADS ARE:

<u>permanent actions:</u>

CLT slab self-weight: $G_1$ = 1,14 kN/m

other permanent loads (finishes…) $G_2$ = 2,14 kN/m

vertical closures self-weight: $G_3$ = 5,59 kN/m

<u>variable actions</u>

live load: $Q_1$ = 2,00 kN/m

inside partitions self-weight: $Q_2$ = 1,20 kN/m

STRUCTURAL ANALYSIS

The structural analysis will be carried out using linear analysis based on the theory of elasticity, considering the combination of actions for Ultimate Limit States [EC2 – 5.1.3(1)P] that is [EC0 – Expression 6.10]

$$\sum_{j\geq 1}\gamma_{Gj}G_{kj}+\gamma_{Q1}Q_{k1}+\sum_{i>1}\gamma_{Qi}\psi_{0i}Q_{ki}$$

Uniformly distributed design load for 1 m of the slab:

$$q_d = 1{,}35\cdot(1{,}14+2{,}14+5{,}59)+1{,}5\cdot(2{,}0+1{,}2)=16{,}77\,\frac{kN}{m}$$

Bending moment and shear force.

$$M_{Ed}=\frac{q_d\cdot l^2}{8}=\frac{16{,}77\cdot 6{,}0^2}{8}=75{,}49\ kNm$$

$$V_{Ed}=\frac{q_d\cdot l}{2}=\frac{16{,}77\cdot 6{,}0}{2}=50{,}31\ kN$$

Material factor for CLT at ULS, $\gamma_M$ = 1,2

Factor for medium-duration loading $k_{mod} = 0,8$

Comparative E-modulus:

$$n = \frac{E_{90,mean}}{E_{0,mean}} = \frac{370}{12000} = 0,03$$

Effective cross-area:

$A_{eff} = 16000\ cm^2$ Cross layers are not taken into account.

Compliance of γ-factor:

$$\gamma_1 = \frac{1}{1 + \left(\frac{\pi^2 \cdot E_1 \cdot A_1}{l^2} \cdot \frac{d_{crosslayer}}{G \cdot b}\right)} = \frac{1}{1 + \left(\frac{3,14^2 \cdot 12000 \cdot 60000}{6000^2} \cdot \frac{30}{50 \cdot 1000}\right)} = 0,89$$

Effective moment of inertia:

$$I_{eff} = I_1 + 2 \cdot I_2 = \frac{b \cdot h_1^3}{12} + 2 \cdot \left(\frac{b \cdot h_2^3}{12} + \gamma_1 \cdot A_2 \cdot a^2\right) = \frac{100 \cdot 4^3}{12} + 2 \cdot \left(\frac{100 \cdot 6^3}{12} + 0,89 \cdot 600 \cdot 8^2\right)$$
$$= 72024,3\ cm^4$$

Resisting moment:

$$W_{eff} = \frac{I_{eff}}{\gamma_1 \cdot a_1 + \frac{d_1}{2}} = \frac{72024,3}{0,89 \cdot 8 + \frac{6}{2}} = 7117,1\ cm^3$$

Static moment:

$$S_{eff} = 2 \cdot z_1 \cdot d_1 \cdot b = 2 \cdot 8 \cdot 6 \cdot 100 = 9600\ cm^3$$

Bending resistance along the fibers:

$$f_{c,0,d} = \frac{f_{c,0,k} \cdot k_{mod}}{\gamma_m} = \frac{21,0 \cdot 0,8}{1,2} = 14\ \frac{N}{mm^2}$$

Bending stress:

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{7549}{7117,1} = 10,6\ \frac{N}{mm^2}$$

Allowable stress:

$$f_{m,d} = \frac{f_{m,k} \cdot k_{mod}}{\gamma_m} = \frac{24,0 \cdot 0,8}{1,2} = 16\ \frac{N}{mm^2}$$

$$\sigma_{m,d} = 10,6\ \frac{N}{mm^2} < f_{m,d} = 16\ \frac{N}{mm^2}$$

$$\frac{\sigma_{m,d}}{f_{m,d}} = \frac{10,6}{16} = 66,3\ \%$$

Shear stress:

$$\tau_{v,d} = \frac{V_d \cdot S_{eff}}{I_{eff} \cdot b} = \frac{50{,}31 \cdot 9600}{72024{,}3 \cdot 100} = 0{,}67\ \frac{N}{mm^2}$$

Allowable stress:

$$f_{v,d} = \frac{f_{v,k} \cdot k_{mod}}{\gamma_m} = \frac{2{,}5 \cdot 0{,}8}{1{,}2} = 1{,}67 \frac{N}{mm^2}$$

$$\tau_{v,d} = 0{,}67\ \frac{N}{mm^2} < f_{v,d} = 1{,}67\ \frac{N}{mm^2}$$

$$\frac{\tau_{v,d}}{f_{v,d}} = \frac{0{,}67}{1{,}67} = 40\ \%$$

Shear stress between the layers:

$$f_{R,v,d} = \frac{f_{R,v,k} \cdot k_{mod}}{\gamma_m} = \frac{1{,}5 \cdot 0{,}8}{1{,}2} = 1{,}0\ \frac{N}{mm^2}$$

$$\tau_{v,d} = 0{,}67\ \frac{N}{mm^2} < f_{R,v,d} = 1{,}0\ \frac{N}{mm^2}$$

$$\frac{\tau_{v,d}}{f_{R,v,d}} = \frac{0{,}67}{1{,}0} = 67\ \%$$

Deflection from $g_k$ and $q_k$:

$$w_{g,inst} = \frac{5}{384} \cdot \frac{g_k l^4}{E_{0,mean} I_{eff}} = \frac{5}{384} \cdot \frac{0{,}089 \cdot 600^4}{1200 \cdot 72024{,}3} = 1{,}74\ cm$$

$$w_{q,inst} = \frac{5}{384} \cdot \frac{q_k l^4}{E_{0,mean} I_{eff}} = \frac{5}{384} \cdot \frac{0{,}032 \cdot 600^4}{1200 \cdot 72024{,}3} = 0{,}63\ cm$$

$$w_{fin} = 1{,}74 + 0{,}63 = 2{,}37\ cm$$

Allowable deflection:

$$w_{zul} = \frac{l}{250} = \frac{600}{250} = 2{,}4\ cm$$

$$w_{fin} = 2{,}37\ cm < 2{,}4\ cm$$

## 2.2. REINFORCED CONCRETE SLABS

The typical slab is concrete slab reinforced in both directions with insulation and a concrete topping.

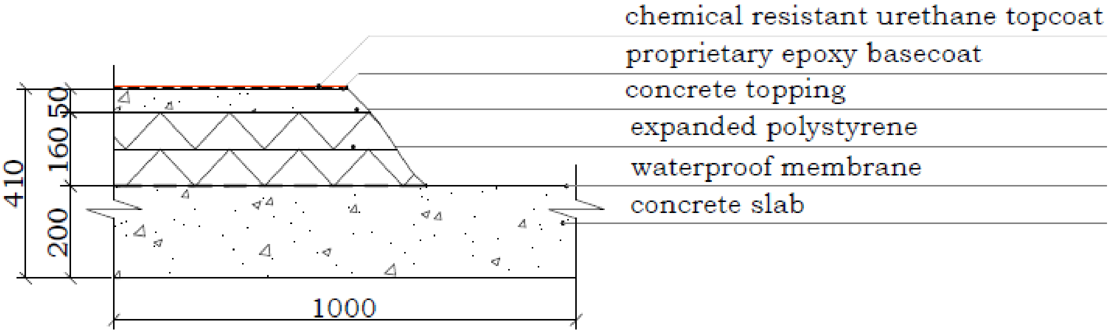



*Fig.18. Typical floor slab in concrete part of the building*

The design of the slab is made considering a 1 m wide strip.

APPLIED LOADS ARE:

<u>permanent actions:</u>

concrete slab self-weight: $G_1$ = 5,0 kN/m

other permanent loads (finishes…) $G_2$ = 1,3 kN/m

<u>variable actions</u>

live load: $Q_1$ = 7,5 kN/m

STRUCTURAL ANALYSIS.

The structural analysis will be carried out using linear analysis based on the theory of elasticity, considering the combination of actions for Ultimate Limit States [EC2 – 5.1.3(1)P] that is [EC0 – Expression 6.10]

$$\sum_{j\geq 1} \gamma_{Gj} G_{kj} + \gamma_{Q1} Q_{k1} + \sum_{i>1} \gamma_{Qi} \psi_{0i} Q_{ki}$$

Uniformly distributed load:

$$n = 1{,}35 \cdot g_k + 1{,}5 \cdot q_k = 1{,}35 \cdot (5{,}0 + 1{,}3) + 1{,}5 \cdot 7{,}5 =$$

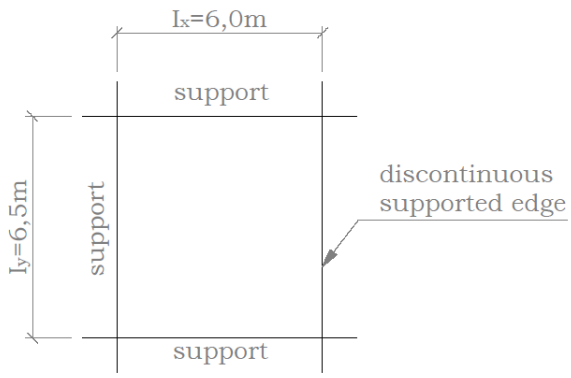
$$= 19{,}8\ kN/m^2$$

$$\frac{l_y}{l_x} = \frac{6{,}5}{6{,}0} = 1{,}08$$

$\beta_{sx}$ and $\beta_{sy}$ are the moment coefficients taken from tables based on previous experience.

Positive moments at mid – span:

$M_{sx} = \beta_{sx} \cdot n \cdot l_x^2 = 0{,}035 \cdot 19{,}8 \cdot 6^2 = 24{,}9\ kNm$ in $l_x$ direction
$M_{sy} = \beta_{sy} \cdot n \cdot l_x^2 = 0{,}028 \cdot 19{,}8 \cdot 6^2 = 20{,}0\ kNm$ in $l_y$ direction




Negative moments at the supports:

$M_x = 0{,}045 \cdot 19{,}8 \cdot 6^2 = 32{,}1\ kNm$
along the long side

$M_y = 0{,}037 \cdot 19{,}8 \cdot 6^2 = 26{,}4\ kNm$
along the short side

Torsion reinforcement should be provided at the discontinuous supported edge corners.

*Fig.19. Restrained slab spanning in two directions*

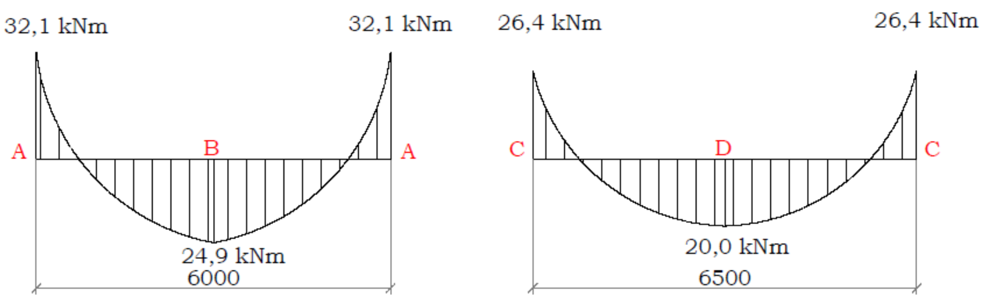



*Fig.20. Bending moment diagrams*

REINFORCEMENT PRE-DIMENSIONING.

In order to determine the cover the prescriptions in EN 1992-1-1 §4.4.1 apply.

The nominal cover is defined as a minimum cover, $c_{min}$, plus an allowance in design for deviation, $\Delta c_{dev}$ [Expression 4.1-EC2]

$$c_{nom} = c_{min} + \Delta c_{dev}$$

where [Expression 4.2-EC2]

$c_{min} = \max (c_{min,b};\ c_{min,dur} + \Delta c_{\gamma} - \Delta_{cdur,st} - \Delta c_{dur,add};\ 10\ mm)$

where $c_{min,b} = \phi = 10$ mm

$c_{min,dur} = 25$ mm [§ 4.4.1.2(5)-EC2 and Table 4.4 N-EC2 for exposure class XS1 (exposed to airborne salt but not in direct contact with sea water) and structural class S2, being used concrete of strength class C40/50]

$\Delta_{c,\gamma} = 0$ mm (recommended value) [§4.4.1.2(6)-EC2]

$\Delta c_{dur,st} = \Delta c_{dur,add} = 0$, since no stainless steel bar or other special measures will be taken [§4.4.1.2(7-8)-EC2].

In the end the minimum concrete cover is:

$c_{min}$ = max (10mm, 25mm, 10mm) = 25 mm

Assuming $\Delta c_{dev}$ = 10 mm, as recommended by EC2 [§4.4.1.3], the nominal concrete cover is:

$c_{nom}$ = 25 +10 = 35 mm

and the effective depth of the slab is:

d = h – c - $\phi$/2 = 200 - 35 - 10/2 = 160 mm.

Since long span bars are placed above short span bars d' = 160 – 10 = 150 mm

The longitudinal reinforcing bars will be pre-dimensioned using the same formulas that will be used for further verifications.

The following assumptions are then made:

- only tension reinforcement is considered
- plane sections remain plane
- the strain in bonded reinforcement is the same as the surrounding concrete
- the tensile strength of the concrete is ignored
- a rectangular stress distribution is assumed for the concrete in compression [EC2 – 3.1.7(3)] where the factor $\lambda$ is equal to 0,8 [EC2 – Expression 3.19] and the factor $\eta$ is equal to 1,0 [EC2 – Expression 3.21] for a concrete strength class C25/30.

An elastic-perfectly plastic stress/strain relationship is assumed for reinforcing bars without the need to check the strain limit [EC2 – 3.2.7(2)b]

$$K = \frac{M_{sx}}{bd^2 f_{ck}}$$

From the lever-arm curve:

$$l_a = \frac{z}{d}$$

Required bending reinforcement:

$$A_s = \frac{M}{0{,}87 f_{yk} z}$$

$$\frac{A_{s,min}}{b \cdot d} = 0{,}26 \cdot \frac{f_{ctm}}{f_{yk}} = 0{,}26 \cdot \frac{3{,}51}{500} > 0{,}0018$$

| Section | $M_{Ed}$ [kNm] | b [mm] | K | z/d | z [mm] | $A_{s,req}$ [mm²] | $A_s$ [mm²] n° ϕ |
|---|---|---|---|---|---|---|---|
| A | 32,1 | 1000 | 0,03 | 0,95 | 152 | 485,5 | 526,7 Ø10 – 150 |
| B | 24,9 | 1000 | 0,02 | 0,95 | 152 | 376,6 | 395,0 Ø10 – 200 |
| C | 26,4 | 1000 | 0,03 | 0,95 | 142,5 | 425,9 | 451,4 Ø10 – 175 |
| D | 20,0 | 1000 | 0,02 | 0,95 | 142,5 | 322,6 | 351,1 Ø10 – 225 |

*Tab.10. Reinforcement of the slab*

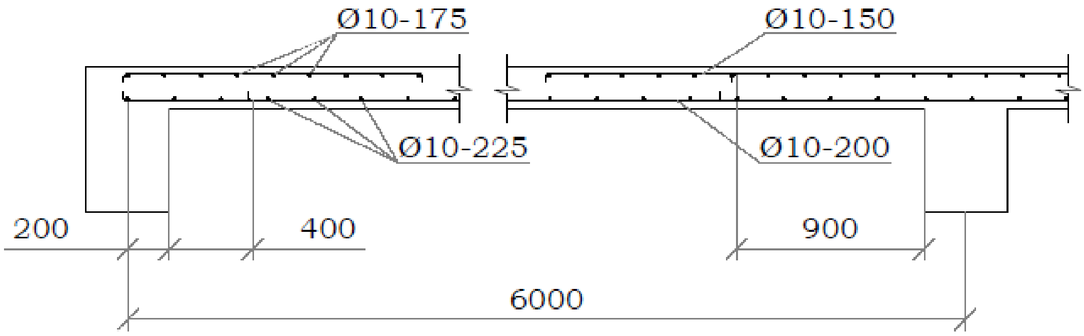



*Fig.21. The layout of reinforcement*

DEFLECTION CONTROL, SPAN – EFFECTIVE DEPTH RATIO.

$$\rho = \frac{100A_{s,req}}{bd}$$

$$\rho_0 = \frac{\sqrt{f_{ck}}}{10} = \frac{\sqrt{40}}{10} = 0{,}6$$

The actual ratio:

$$\frac{l_x}{d} = \frac{6000}{160} = 37{,}5 \qquad \frac{l_y}{d'} = \frac{6500}{150} = 43{,}3$$

Basic span – effective depth ratio:

$$\frac{l}{d} = K \cdot \left[11 + \frac{1{,}5\sqrt{f_{ck}} \cdot \rho_0}{\rho} + 3{,}2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{1{,}5}\right] \qquad \rho_0 > \rho$$

| Section | $M_{Ed}$ [kNm] | $A_{s,req}$ [mm²] | ρ [%] | $\left(\frac{l}{d}\right)_{lim}$ | $\frac{l}{d}$ |
|---|---|---|---|---|---|
| A | 32,1 | 485,5 | 0,25>0,18 | 64,1 | 37,5 |
| B | 24,9 | 376,6 | 0,19>0,18 | 96,5 | 37,5 |
| C | 26,4 | 425,9 | 0,35>0,18 | 71,3 | 43,3 |
| D | 20,0 | 322,6 | 0,27>0,18 | 111,7 | 43,3 |

*Tab.11. Span – effective depth ratio*

The actual span depth ration is smaller than the effective one therefore the direct calculation of deflections can therefore be omitted.

Check for shear.

It is not usual for a slab to contain shear reinforcement and check for shear is generally satisfied and hence they are not checked.

## 3. BEAMS

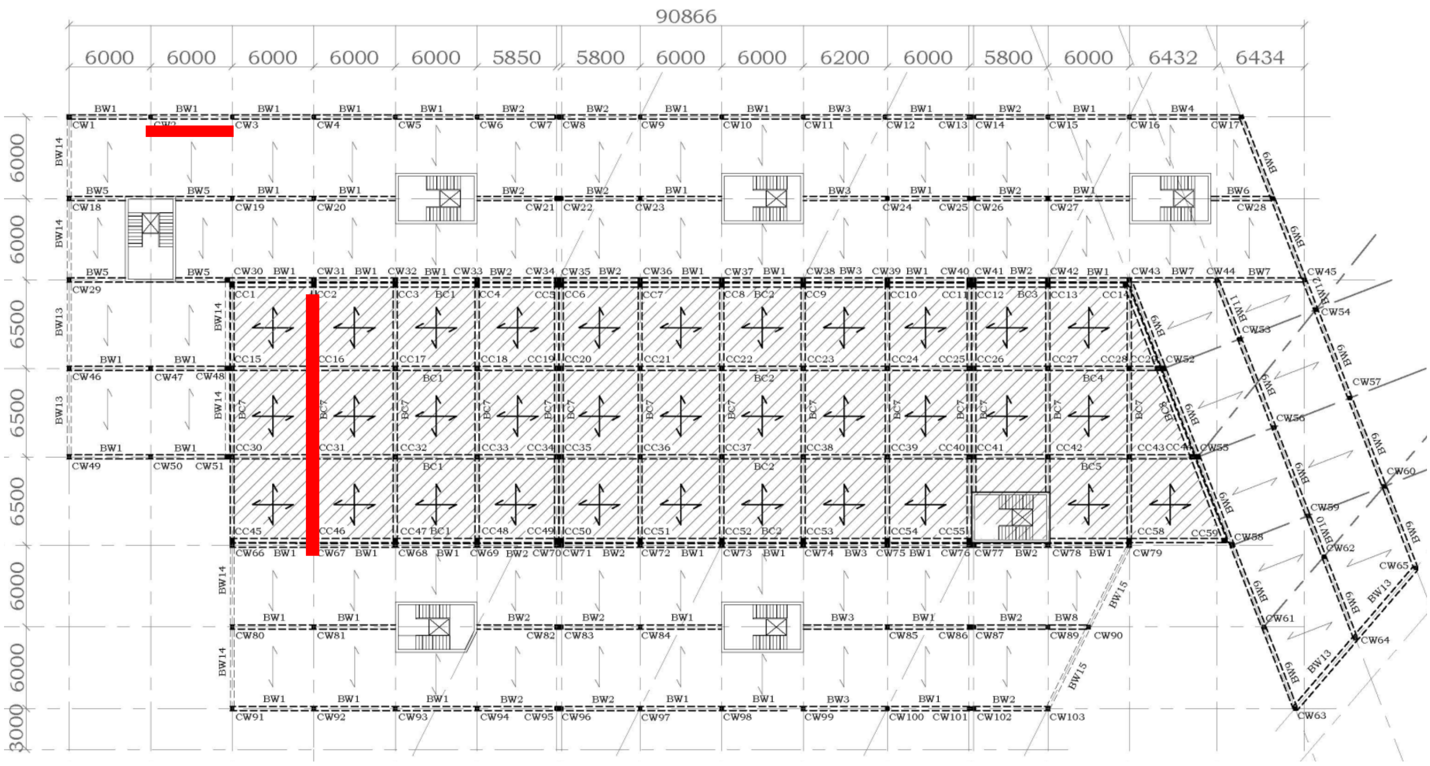
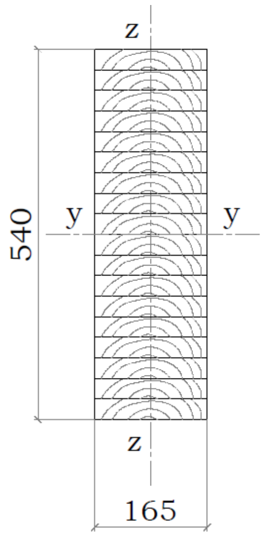



*Fig.22. Beam design*

### 3.1. GLULAM BEAM

The design is going to be performed on the beam that runs between columns CW17 and CW18.




| | |
|---|---|
| Bearing length at each end of a beam | $l_b$ = 200 mm |
| Material factor for glulam at ULS | $\gamma_M$ = 1,25 |
| Factor for medium-duration loading | |
| and service class 1 | $k_{mod}$ = 0,8 |
| Size factor for depth more than 600 mm | $k_h$ =1,0 |
| Lateral stability of a beam | $k_{crit}$ = 1,0 |
| Bearing factor | $k_{c.90}$ = 1,0 |

Deformation factor for service class 1 $k_{def} = 0,6$

Factor for quasi-permanent value

of variable action $\psi_2 = 0,0$

Load sharing factor $k_{sys} = 1,0$

APPLIED LOADS ARE:

permanent actions:

self-weight of a beam

$$G_1 = b \cdot h \cdot \rho_m = 0,165 \cdot 0,54 \cdot 4,56 = 0,4\ \frac{kN}{m}$$

CLT slab with finishes weight: $G_2 = 3,28$ kN/m

vertical closures self-weight: $G_3 = 5,59$ kN/m

variable actions

live load: $Q_1 = 2,00$ kN/m

inside partitions self-weight: $Q_2 = 1,20$ kN/m

STRUCTURAL ANALYSIS

Uniformly distributed design load for 1 m of the beam:

$$q_d = 1,35 \cdot (0,4 + 3,28 + 5,59) + 1,5 \cdot (2,0 + 1,2) = 17,3\,\frac{kN}{m}$$

Bending moment and shear force:

$$M_{Ed} = \frac{q_d \cdot l^2}{8} = \frac{17,3 \cdot 6,0^2}{8} = 77,92\ kNm$$

$$V_{Ed} = \frac{q_d \cdot l}{2} = \frac{17,3 \cdot 6,0}{2} = 51,9\ kN$$

$$W = \frac{bh^2}{6} = \frac{16,5 \cdot 54^2}{6} = 8019 cm^3$$

Design bending stress:

$$\sigma_{m,y,d} = \frac{M_{Ed}}{W} = \frac{7792}{8019} = 9,72\ \frac{N}{mm^2}$$

Design bending strength:

$$f_{m,y,d} = \frac{k_{mod} \cdot k_{sys} \cdot k_h \cdot f_{m,y,k}}{\gamma_M} = \frac{0,8 \cdot 1,0 \cdot 1,0 \cdot 32,0}{1,25} = 20,48\ \frac{N}{mm^2}$$

Design bending strength taking lateral torsional buckling effect into account (equation (4.13); EC5, *equation (6.33)*):

$$f_{mr,y,d} = k_{crit} \cdot f_{m,y,d} = 1{,}0 \cdot 20{,}48 = 20{,}48\ \frac{N}{mm^2}$$

$$\sigma_{m,y,d} = 9{,}72\ \frac{N}{mm^2} < f_{m,y,d} = f_{mr,y,d} = 20{,}48 \frac{N}{mm^2}$$

Bending strength of a glulam beam greater than the bending stress and is satisfactory for this loading condition.

Design shear stress:

$$\tau_{v,d} = \frac{3}{2} \cdot \frac{V_{Ed}}{b \cdot h} = \frac{3}{2} \cdot \frac{51{,}9}{165 \cdot 540} = 0{,}87\ \frac{N}{mm^2}$$

Design shear strength:

$$f_{v,d} = \frac{k_{mod} \cdot k_{sys} \cdot f_{v,g,k}}{\gamma_M} = \frac{0{,}8 \cdot 1{,}0 \cdot 2{,}7}{1{,}25} = 1{,}73\ \frac{N}{mm^2}$$

$$\tau_{v,d} = 0{,}87\ \frac{N}{mm^2} < f_{v,d} = 1{,}73\ \frac{N}{mm^2}$$

Design value of the end reaction:

$$R_{Ed} = V_{Ed} = 51{,}9\ kN$$

Design bearing stress:

$$\sigma_{c,90,d} = \frac{R_{Ed}}{b \cdot l_b} = \frac{51{,}9}{200 \cdot 165} = 1{,}57\ \frac{N}{mm^2}$$

Design bearing strength:

$$f_{c,90,d} = \frac{k_{mod} \cdot k_{sys} \cdot f_{c,90,g,k}}{\gamma_M} = \frac{0{,}8 \cdot 1{,}0 \cdot 2{,}7}{1{,}25} = 1{,}73 \frac{N}{mm^2}$$

Factored design bearing strength:

$$k_{c,90} \cdot f_{c,90,d} = 1{,}0 \cdot 1{,}73 = 1{,}73 \frac{N}{mm^2}$$

$$\sigma_{c,90,d} = 1{,}57\ \frac{N}{mm^2} < f_{c,90,d} = 1{,}73 \frac{N}{mm^2}$$

Bearing strength is OK without need to use a higher value for $k_{c.90}$.

Beam deflection:

At the SLS the partial safety factor is 1.

As the member is material having the same creep properties, the mean value of stiffness will be used to derive the instantaneous and the final deflection of the beam.

Instantaneous deflection due to loading on the beam:

$$u_{inst,dl} = \frac{5 \cdot G_k \cdot l^4}{32 \cdot E_{o,g,mean} \cdot b \cdot h^3}\left[1 + 0,96 \frac{E_{o,g,mean}}{G_{o,g,mean}}\left(\frac{h}{l}\right)^2\right]$$
$$= \frac{5 \cdot 9,27 \cdot 10^{-3} \cdot 6000^4}{32 \cdot 11,6 \cdot 165 \cdot 540^3}\left[1 + 0,96 \cdot \frac{11,6}{0,72} \cdot \left(\frac{540}{6000}\right)^2\right] = 7,0\ mm$$

$$u_{inst,Q} = \frac{5 \cdot Q_k \cdot l^4}{32 \cdot E_{o,g,mean} \cdot b \cdot h^3}\left[1 + 0,96 \frac{E_{o,g,mean}}{G_{o,g,mean}}\left(\frac{h}{l}\right)^2\right]$$
$$= \frac{5 \cdot 3,2 \cdot 10^{-3} \cdot 6000^4}{32 \cdot 11,6 \cdot 165 \cdot 540^3}\left[1 + 0,96 \cdot \frac{11,6}{0,72} \cdot \left(\frac{540}{6000}\right)^2\right] = 2,4\ mm$$

Instantaneous deflection at the mid-span of a beam:

$$u_{inst} = u_{inst,dl} + u_{inst,Q} = 7,0 + 2,4 = 9,4\ mm$$

Limitation on deflection at the instantaneous state:

$$w_{inst} = \frac{l}{300} = \frac{6000}{300} = 20\ mm$$

$$u_{inst} = 9,4\ mm < w_{inst} = 20\ mm$$

Final deflection due to permanent actions:

$$u_{fin,G} = u_{inst,dl} \cdot \left(1 + k_{def}\right) = 7,0 \cdot (1 + 0,6) = 11,2\ mm$$

Final deflection due to variable and quasi-permanent actions:

$$u_{fin,Q} = u_{inst,Q} \cdot \left(1 + \psi_2 \cdot k_{def}\right) = 2,4 \cdot (1 + 0,0 \cdot 0,6) = 2,4\ mm$$

Final deflection due to permanent and quasi-permanent actions:

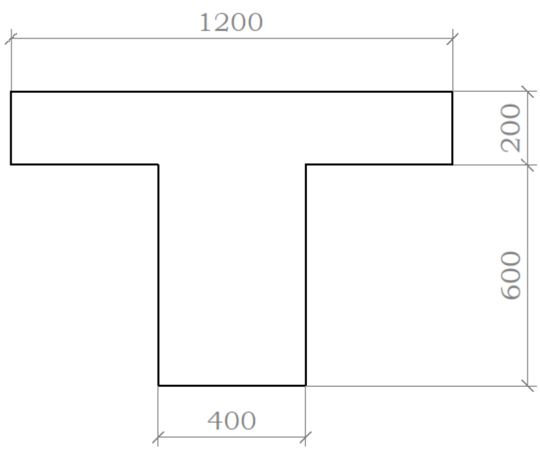
$$u_{net,fin} = u_{fin,G} + u_{fin,Q} = 11,2 + 2,4 = 13,6\ mm$$

Adopting EC5 limitation on deflection:

$$w_{net,fin} = \frac{l}{250} = \frac{6000}{250} = 24\ mm$$

$$u_{net,fin} = 13,6\ mm < w_{net,fin} = 24\ mm$$

## 3.2. REINFORCED CONCRETE BEAM.




The design is going to be performed on the continuous beam that runs from column CC2 to column CC39. The beam height is greater than the slab thickness, with a superior flange whose height is the same as the depth of the slab and width equal to 1200 mm and an inferior rib/web whose height is 600 mm and width is 400 mm.

APPLIED LOADS ARE:

<u>permanent actions:</u>

structural self-weight

$$X_{G_{1slab}} = 0{,}045 \cdot 5{,}0 \cdot 6^2 = 8{,}1\ kNm$$

$$G_{1,flange} = G_{1,slab} \cdot l_{slab} + 2 \cdot \frac{X_{G_{1,slab}}}{l_{slab}} = 5{,}0 \cdot 6{,}0 + 2 \cdot \frac{8{,}1}{6{,}0} = 32{,}7 \frac{kN}{m}$$

$$G_{1,rib} = b \cdot h \cdot \rho_m = 0{,}4 \cdot 0{,}6 \cdot 25 = 6{,}0\ \frac{kN}{m}$$

$$G_1 = 32{,}7 + 6{,}0 = 38{,}7 \frac{kN}{m}$$

other permanent load

$$X_{G_{2,slab}} = 0{,}045 \cdot 1{,}3 \cdot 6^2 = 2{,}1\ kNm$$

$$G_2 = G_{2,slab} \cdot l_{slab} + 2 \cdot \frac{X_{G_{2,slab}}}{l_{slab}} = 1{,}3 \cdot 6{,}0 + 2 \cdot \frac{2{,}1}{6{,}0} = 8{,}5 \frac{kN}{m}$$

<u>variable actions</u>

live load

$$X_{Q_{1,slab}} = 0{,}045 \cdot 7{,}5 \cdot 6^2 = 12{,}2\ kNm$$

$$Q_1 = G_{1,slab} \cdot l_{slab} + 2 \cdot \frac{X_{Q_{1,slab}}}{l_{slab}} = 7{,}5 \cdot 6{,}0 + 2 \cdot \frac{12{,}2}{6{,}0} = 49{,}1 \frac{kN}{m}$$

STRUCTURAL ANALYSIS

The stress analysis will be carried out refering to the static schemes shown Fig.23. and Fig.24. for the appropriate ULS load combination [EC0 – Expression 6.10]

$$\sum_{j\geq 1} \gamma_{Gj}\ G_{kj} + \gamma_{Q1}\ Q_{k1} + \sum_{i>1} \gamma_{Qi}\ \psi_{0i}\ Q_{ki}$$

The most unfavorable combination of actions can be achieved considering the live load $Q_1$ as the leading action.

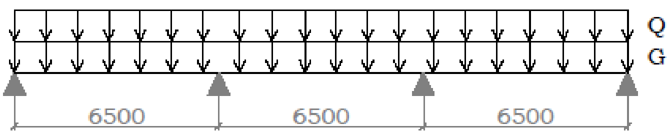



*Fig.23. Continuous beam - maximum moment in the spans and minimum moment at the continuity supports*

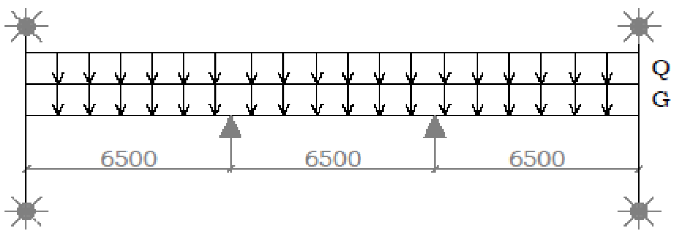



*Fig.24. Frame - negative moments at the ends*

Load Combination 1 (maximum moment at the continuity supports).

Permanent loads on three spans: $\gamma_G(G_1 + G_2) = 1{,}35 \cdot (38{,}7 + 8{,}5) = 63{,}72\,\frac{kN}{m}$

Variable load on three spans: $\gamma_Q \cdot Q_1 = 1{,}5 \cdot 49{,}1 = 73{,}65\,\frac{kN}{m}$

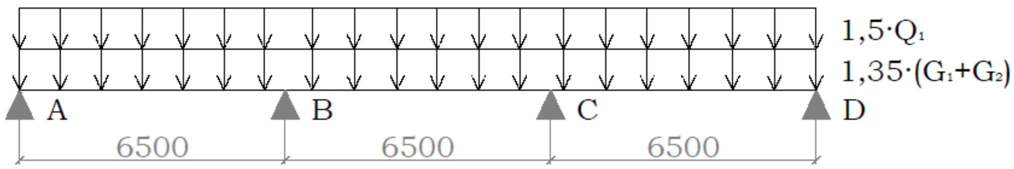



*Fig.25. Continuous beam scheme – load combination 1*

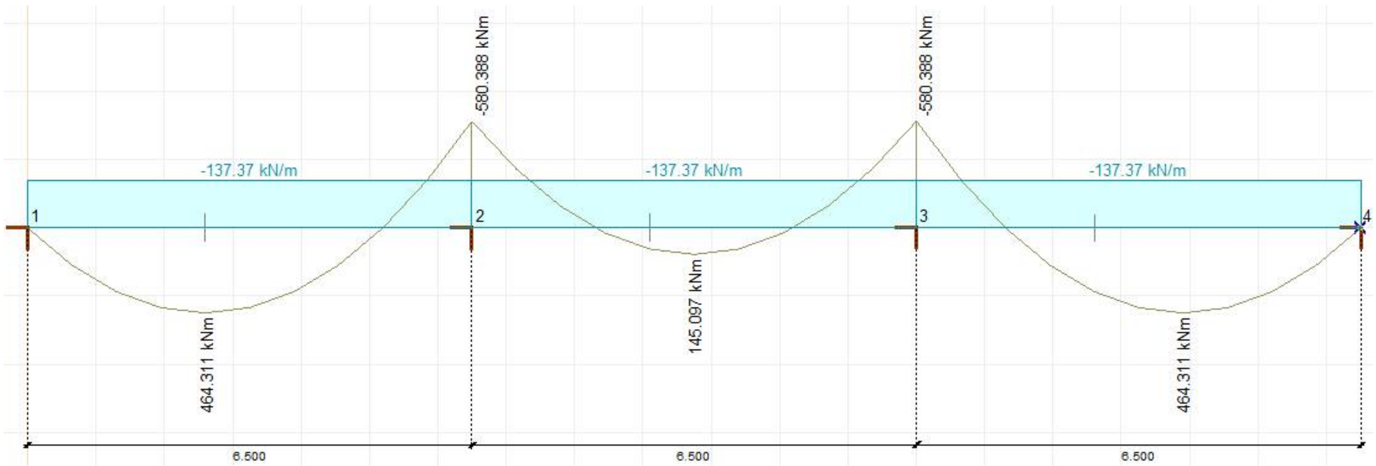



*Fig.26. Bending moment diagram computed with Axis VM Software*

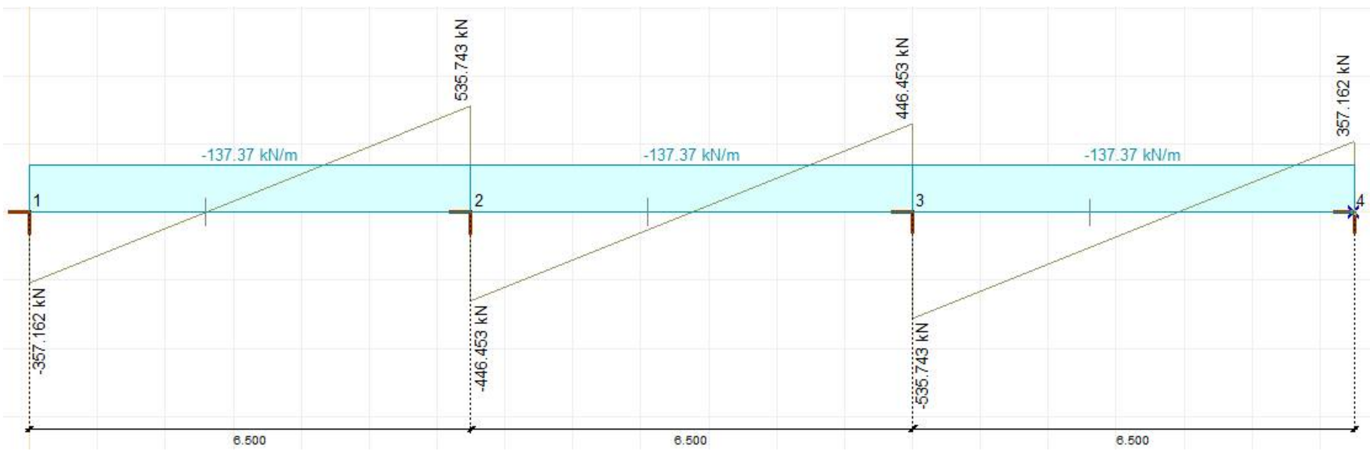



*Fig.27. Shear force diagram computed with Axis VM Software*

Load Combination 2 (maximum moment in the left span).

Permanent loads on three spans: $\gamma_G(G_1 + G_2) = 1{,}35 \cdot (38{,}7 + 8{,}5) = 63{,}72\,\frac{kN}{m}$

Variable load on the left span: $\gamma_Q \cdot Q_1 = 1{,}5 \cdot 49{,}1 = 73{,}65\,\frac{kN}{m}$

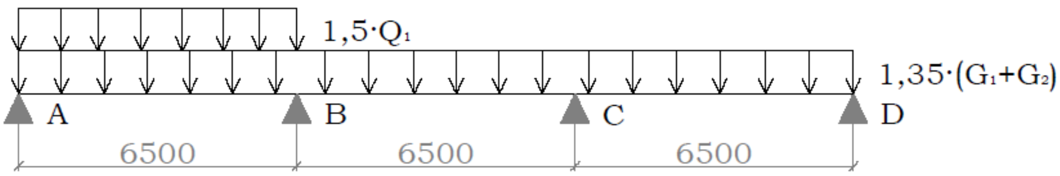



*Fig.28. Continuous beam scheme – load combination 2*

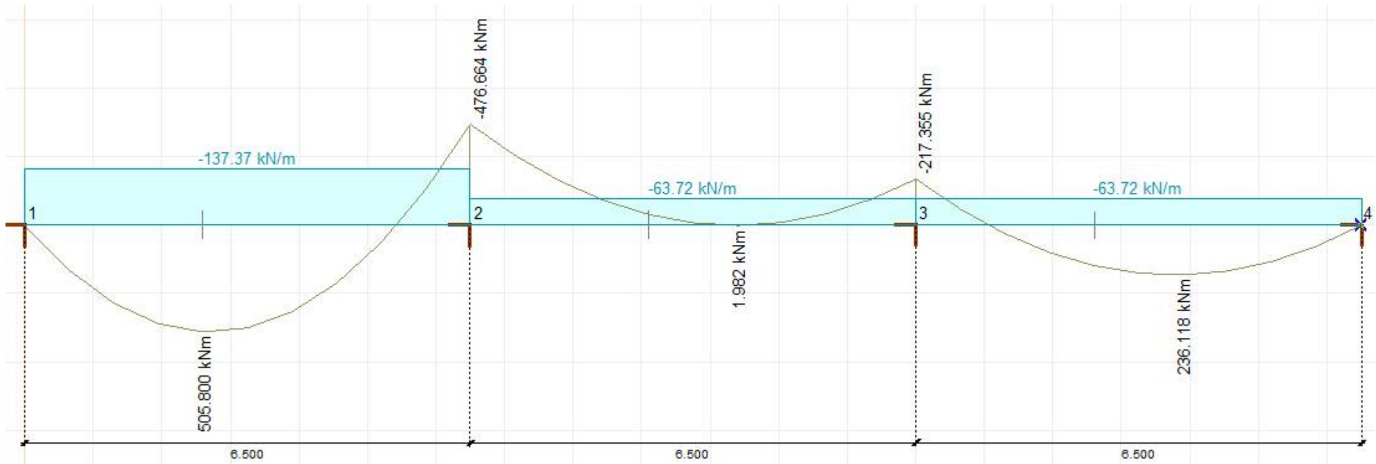



*Fig.29. Bending moment diagram computed with Axis VM Software*

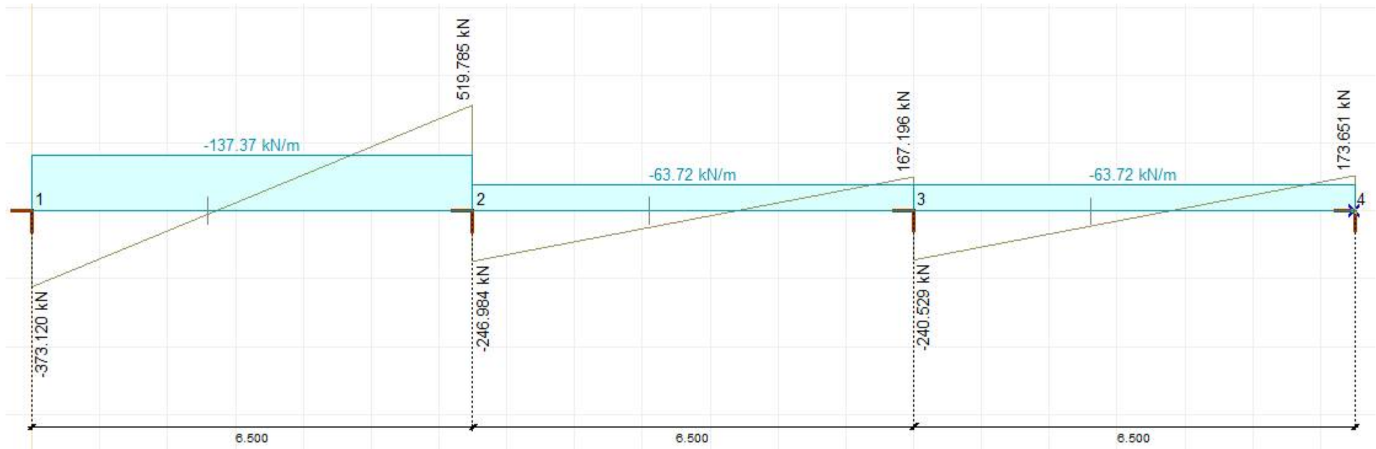



*Fig.30. Shear force diagram computed with Axis VM Software*

*Load Combination 3 (maximum moment in the middle span).*

Permanent loads on three spans: $\gamma_G(G_1 + G_2) = 1{,}35 \cdot (38{,}7 + 8{,}5) = 63{,}72\,\frac{kN}{m}$

Variable load on the middle span: $\gamma_Q \cdot Q_1 = 1{,}5 \cdot 49{,}1 = 73{,}65\,\frac{kN}{m}$

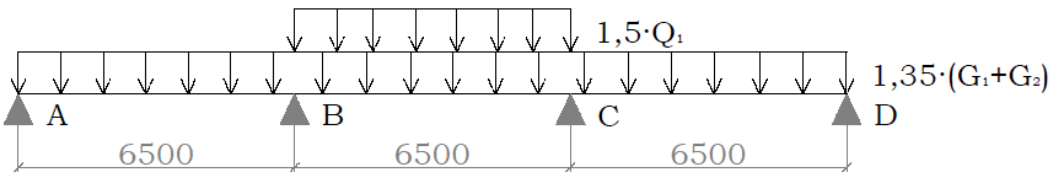



*Fig.31. Continuous beam scheme – load combination 3*

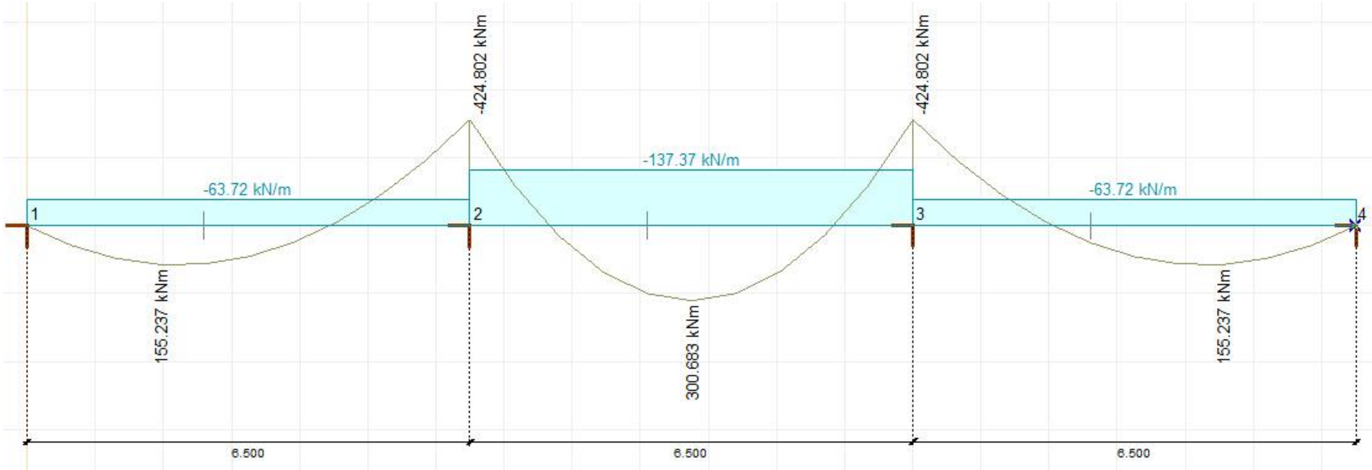



*Fig.32. Bending moment diagram computed with Axis VM Software*

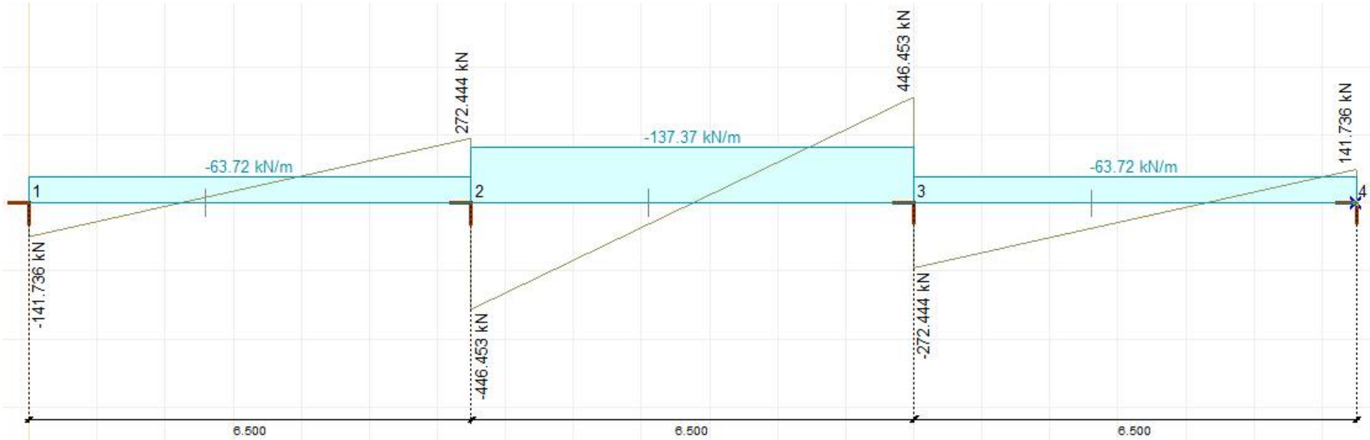



*Fig.33. Shear force diagram computed with Axis VM Software*

<u>*Load Combination 4 (maximum moment in the right span).*</u>

Permanent loads on three spans: $\gamma_G(G_1 + G_2) = 1{,}35 \cdot (38{,}7 + 8{,}5) = 63{,}72\,\frac{kN}{m}$

Variable load on the right span: $\gamma_Q \cdot Q_1 = 1{,}5 \cdot 49{,}1 = 73{,}65\,\frac{kN}{m}$

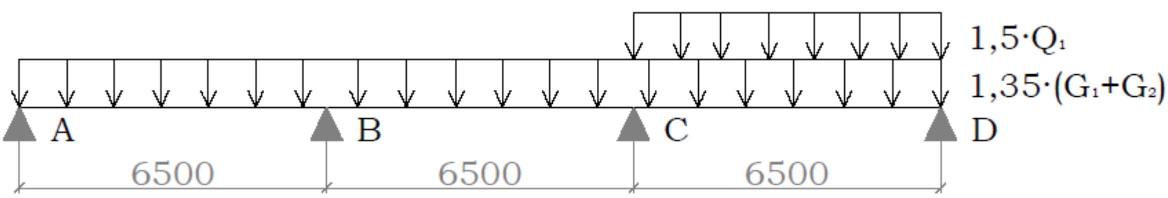


*Fig.34. Continuous beam scheme – load combination 4*

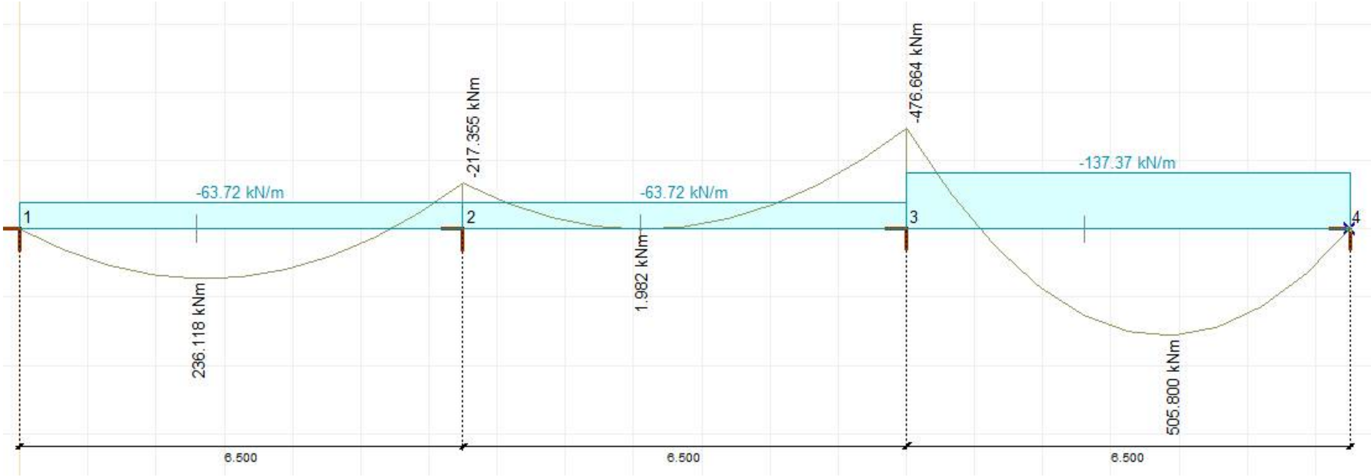



*Fig.35. Bending moment diagram computed with Axis VM Software*

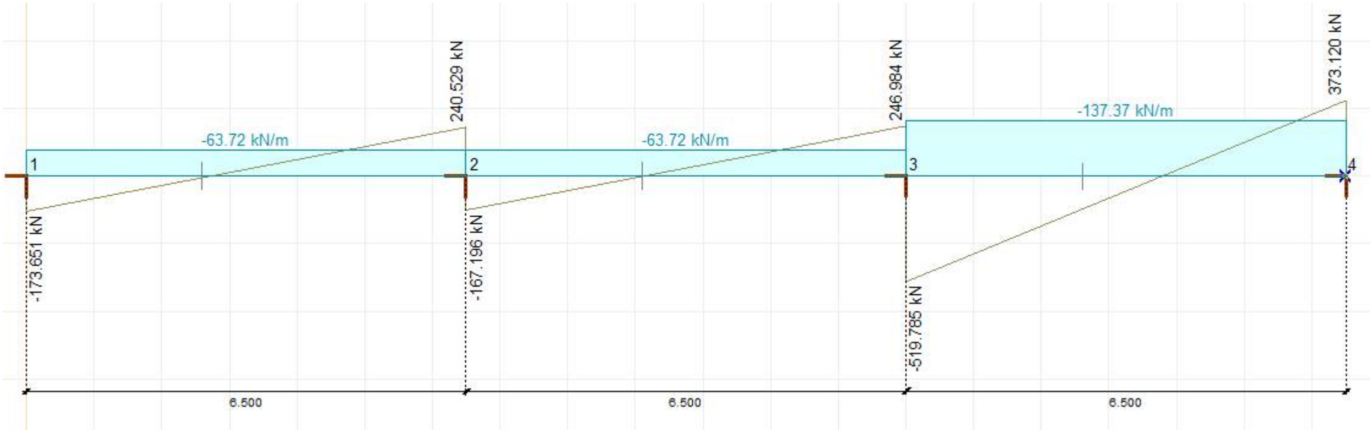
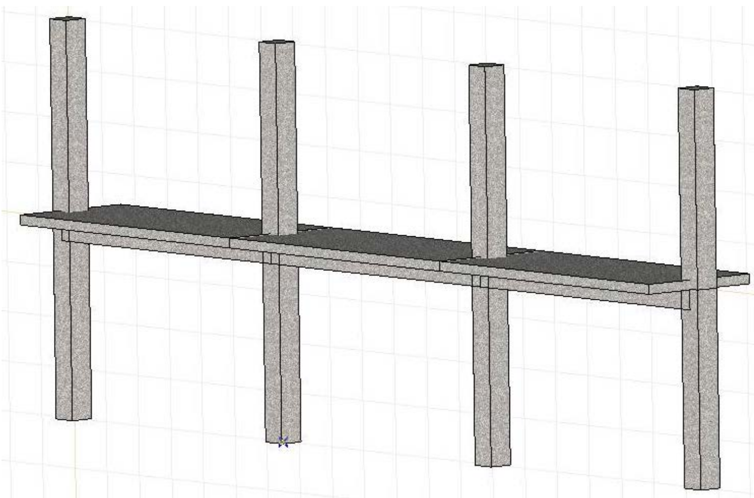



*Fig.36. Shear force diagram computed with Axis VM Software*

Load Combination 5 (maximum moment in the left span).

Permanent loads on three spans: $\gamma_G(G_1 + G_2) = 1{,}35 \cdot (38{,}7 + 8{,}5) = 63{,}72\,\frac{kN}{m}$

Variable load on the left span: $\gamma_Q \cdot Q_1 = 1{,}5 \cdot 49{,}1 = 73{,}65\,\frac{kN}{m}$



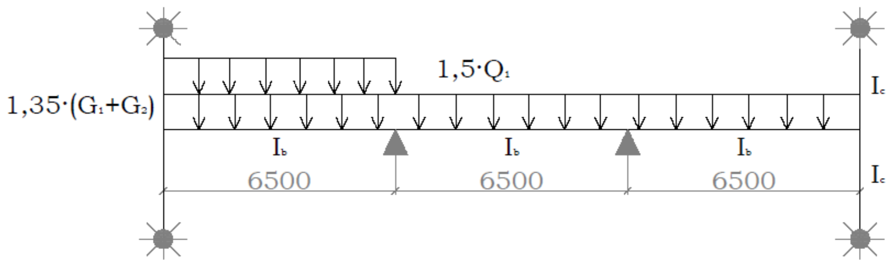



*Fig.37. Frame scheme – load combination 5*

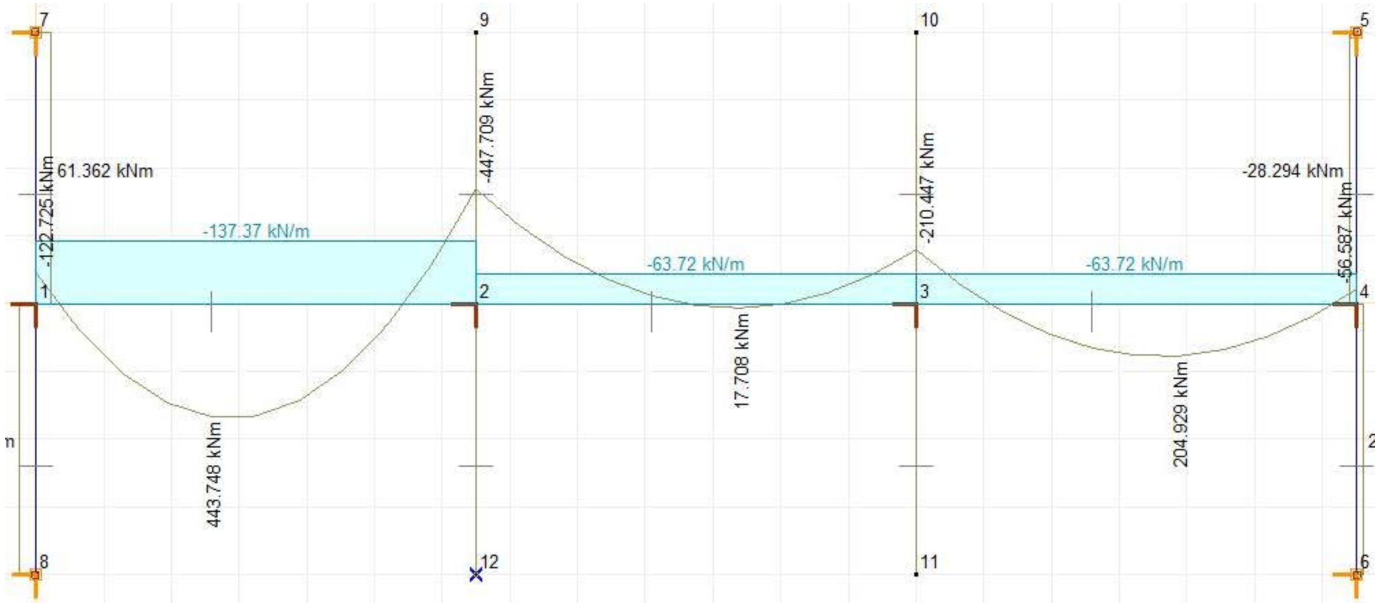



*Fig.38. Bending moment diagram computed with Axis VM Software*

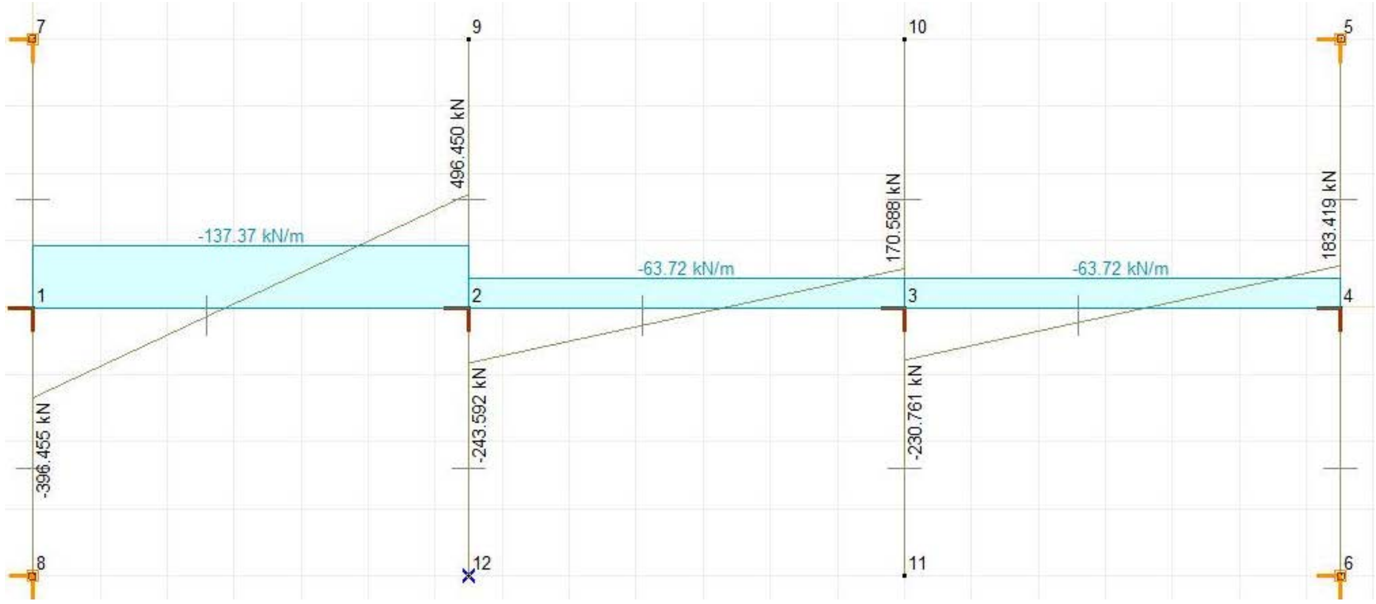



*Fig.39. Shear force diagram*

<u>Load Combination 6 (maximum moment in the right span).</u>

Permanent loads on three spans: $\gamma_G(G_1 + G_2) = 1{,}35 \cdot (38{,}7 + 8{,}5) = 63{,}72\,\frac{kN}{m}$

Variable load on the left span: $\gamma_Q \cdot Q_1 = 1{,}5 \cdot 49{,}1 = 73{,}65\,\frac{kN}{m}$

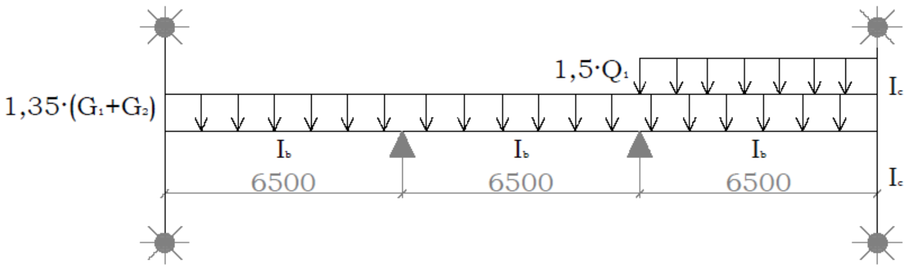



*Fig.40. Frame scheme – load combination 6*

The frame is symmetrical, so the internal force diagrams are symmetrical to load combination 5.

<u>Load Combination 7 (maximum moment in the middle span).</u>

Permanent loads on three spans: $\gamma_G(G_1 + G_2) = 1{,}35 \cdot (38{,}7 + 8{,}5) = 63{,}72\,\frac{kN}{m}$

Variable load on the left span: $\gamma_Q \cdot Q_1 = 1{,}5 \cdot 49{,}1 = 73{,}65\,\frac{kN}{m}$

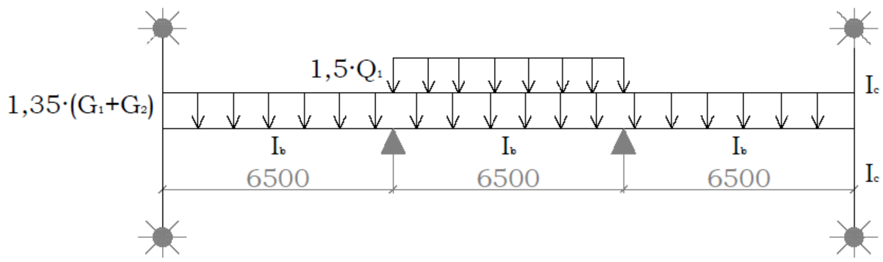



*Fig.41. Frame scheme – load combination 6*

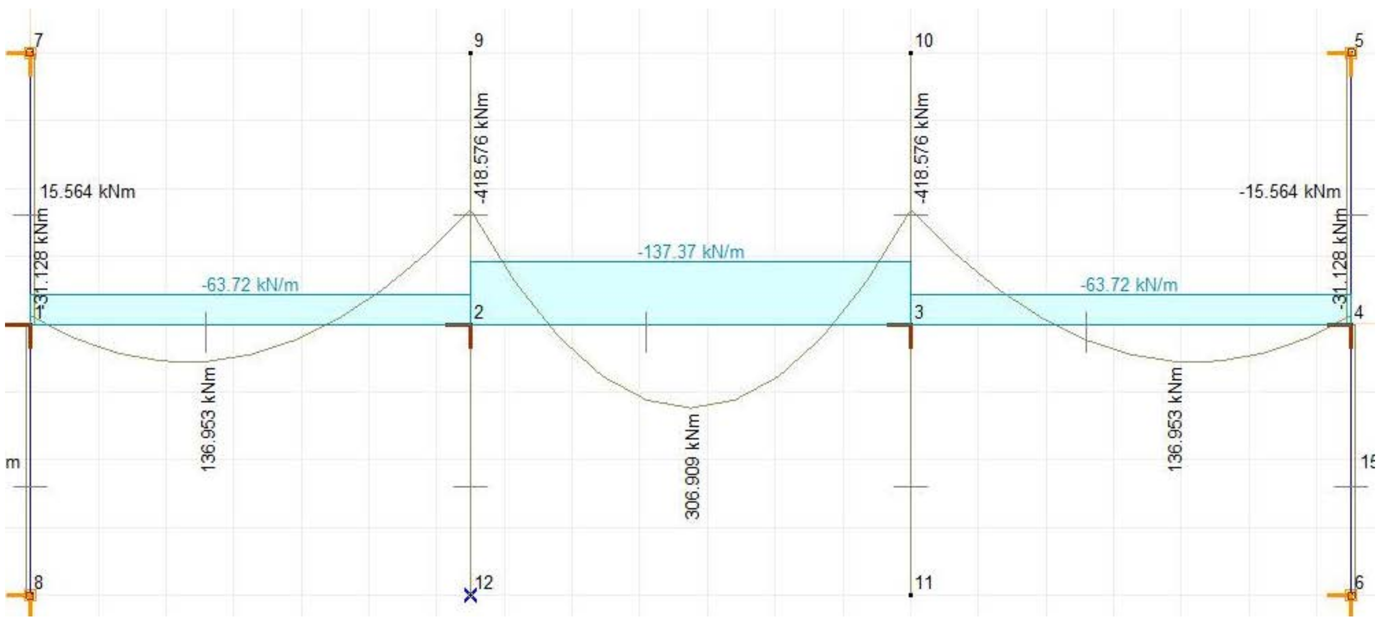
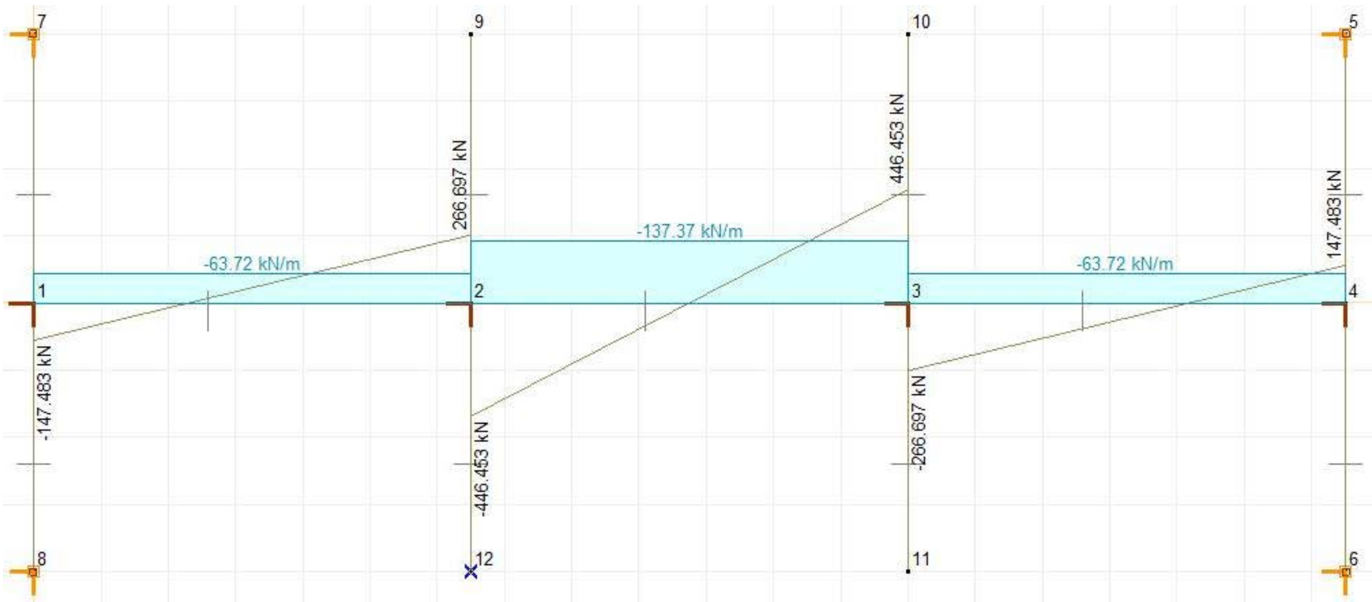


*Fig.42. Bending moment diagram*

*Fig.43. Shear force diagram*

ENVELOPE OF THE BENDING MOMENT DIAGRAM AND PRE-DIMENSIONING OF REINFORCEMENT.



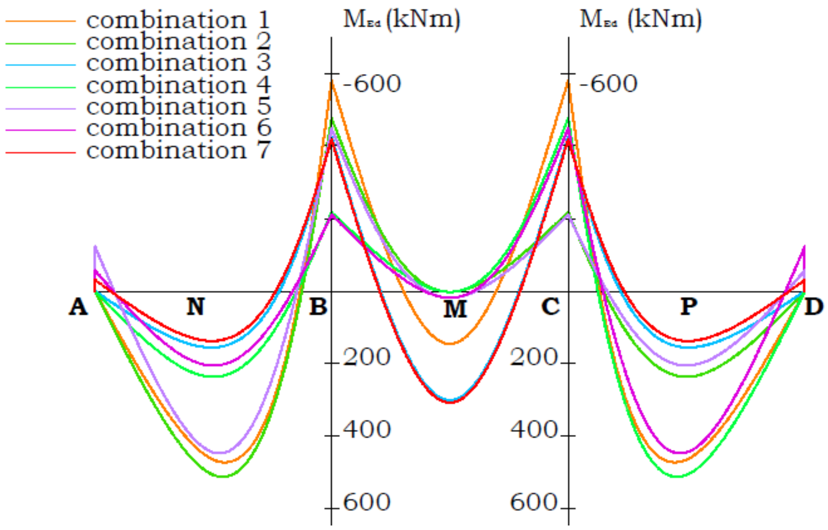




*Fig.44. Envelope of the bending moments*

| Section | Position (m) | $M_{Ed}$ (kNm) |
|---|---|---|
| A (left end column) | 0,00 | -122,7 |
| N ($M_{max}$ left span) | 2,70 | 505,8 |
| B (continuity support) | 6,50 | -580,4 |
| M ($M_{max}$ middle span) | 9,75 | 306,9 |
| C (continuity support) | 13,00 | -580,4 |
| P ($M_{max}$ right span) | 16,80 | 505,8 |
| D (right end column) | 19,50 | -122,7 |

*Tab.12. Significant sections and corresponding bending moment*

The pre-dimensioning of longitudinal reinforcement is carried out through the same expressions used for verifications as well as was previously done for slabs and the same assumptions on the behavior of the materials are made.

The effective depth of the section, d, can be calculated after the evaluation of the concrete cover, similarly to what previously done for slabs.

According to EC2 – 4.4.1 the nominal concrete cover follows from [EC2 – Expression 4.1 and 4.2]

In order to determine the cover the prescriptions in EN 1992-1-1 §4.4.1 apply.

The nominal cover is defined as a minimum cover, $c_{min}$, plus an allowance in design for deviation, $\Delta c_{dev}$ [Expression 4.1-EC2]

$c_{nom} = c_{min} + \Delta c_{dev}$

where [Expression 4.2-EC2]

$c_{min} = \max (c_{min,b};\ c_{min,dur} + \Delta c_{\gamma} - \Delta_{cdur,st} - \Delta c_{dur,add};\ 10\text{ mm})$

For transversal shear reinforcement (stirrups):

$c_{min,b} = \phi = 8$ mm

$c_{min,dur} = 25$ mm [§ 4.4.1.2(5)-EC2 and Table 4.4 N-EC2 for exposure class XS1 (exposed to airborne salt but not in direct contact with sea water) and structural class S2, being used concrete of strength class C40/50]

$\Delta_{c,\gamma} = 0$ mm (recommended value) [§4.4.1.2(6)-EC2]

$\Delta c_{dur,st} = \Delta c_{dur,add} = 0$, since no stainless steel bar or other special measures will be taken [§4.4.1.2(7-8)-EC2].

In the end the minimum concrete cover is:

$c_{min} = \max (8\text{mm}, 25\text{mm}, 10\text{mm}) = 25$ mm

Assuming $\Delta c_{dev}$ = 10 mm, as recommended by EC2 [§4.4.1.3], the nominal concrete cover is:

$c_{nom,\ transv}$ = 25 +10 = 35 mm

For transversal longitudinal reinforcement:

$c_{min,b}$ = $\phi$ = 20 mm

$c_{min,dur}$ = 25 mm [§ 4.4.1.2(5)-EC2 and Table 4.4 N-EC2 for exposure class XS1 (exposed to airborne salt but not in direct contact with sea water) and structural class S2, being used concrete of strength class C40/50]

$\Delta_{c,\gamma}$ = 0 mm (recommended value) [§4.4.1.2(6)-EC2]

$\Delta c_{dur,st}$ = $\Delta c_{dur,add}$ = 0, since no stainless steel bar or other special measures will be taken [§4.4.1.2(7-8)-EC2].

In the end the minimum concrete cover is:

$c_{min}$ = max (20mm, 25mm, 10mm) = 25 mm

Assuming $\Delta c_{dev}$ = 10 mm, as recommended by EC2 [§4.4.1.3], the nominal concrete cover is:

$c_{nom,\ long}$ = 25 +10 = 35 mm

From the previous calculations, it appears that the concrete cover for stirrups is dominant. As a matter of fact, assuming $c_{nom,\ transv}$ = 35 mm, the longitudinal reinforcement cover is $c_{long}$ = 43 mm > $c_{nom,\ long}$ = 35 mm.

And the effective depth of the slab is:

d = h – $c_{nom,\ transv}$ – $\phi_{transv}$ – $\phi_{long}$/2 = 800 – 35 – 8 – 20/2 = 747 mm

Through the translational equilibrium the required reinforcement area can be evaluated:

$$A_s = \frac{0{,}8 \cdot b \cdot x \cdot f_{cd}}{f_{yd}}$$

The following limit for the depth of the neutral axis applies.

$$\xi = \frac{x}{d} \leq \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{yd}} = \frac{3{,}5}{3{,}5 + 1{,}96} = 0{,}641$$

The so determined reinforcement needs to be not less than the minimum recommended [EC2 – 9.2.1.1, Expression 9.1N]

| Section | $M_{Ed}$ [kNm] | b [mm] | x [mm] | x/d | $A_{s,req}$ [mm$^2$] | $b_t$ [mm] | $A_{s,min}$ [mm$^2$] | $A_s$ [mm$^2$] n° $\phi$ |
|---|---|---|---|---|---|---|---|---|
| A (column) | -122,7 | 400 | 22,4 | 0,03 | 415,6 | 1200 | 1817,9 | 1885 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | 6Ø20 |
| N ($M_{max}$ left) | 505,8 | 1200 | 31,7 | 0,04 | 1764,4 | 400 | 606,0 | 2275<br>4Ø20+4Ø18 |
| B (support) | -580,4 | 400 | 101,6 | 0,14 | 1885,0 | 1200 | 1817,9 | 2275<br>4Ø20+4Ø18 |
| M ($M_{max}$ middle) | 306,9 | 1200 | 19,1 | 0,03 | 1063,1 | 400 | 606,0 | 1137<br>2Ø18+2Ø20 |
| C (support) | -580,4 | 400 | 101,6 | 0,14 | 1885,0 | 1200 | 1817,9 | 2275<br>4Ø20+4Ø18 |
| P ($M_{max}$ right) | 505,8 | 1200 | 31,7 | 0,04 | 1764,4 | 400 | 606,0 | 2275<br>4Ø20+4Ø18 |
| D (column) | -122,7 | 400 | 22,4 | 0,03 | 415,6 | 1200 | 1817,9 | 1885<br>6Ø20 |

$$A_{s,min} = 0{,}26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d$$

*Tab.13. ULS pre-dimensioning of longitudinal reinforcement*

BENDING ULTIMATE LIMIT STATE VERIFICATION

The same assumptions on the behavior of the structural member as the previous chapter apply.

Assuming yielded steel and a rectangular stress block for concrete the translational equilibrium is

$$0{,}8 \cdot b \cdot x \cdot f_{cd} = A_s \cdot f_{yd}$$

The neutral axis then is

$$x = \frac{A_s \cdot f_{yd}}{0{,}8 \cdot b \cdot f_{cd}}$$

Through the rotational equilibrium about the barycentre either of the compressions or tensions, the resisting moment can be easily determined.

$M_{Rd} = A_s \cdot f_{yd} (d - 0{,}4 \cdot x) = 0{,}8 \cdot b \cdot x \cdot f_{cd} (d - 0{,}4 \cdot x) \geq M_{Ed}$

Results of such calculations are shown in Tab.14.

| Sec. | $A_s$ (mm$^2$) | b (mm) | x (mm) | ξ | $M_{Rd}$ (kNm) | $\lvert M_{Ed} \rvert$ (kNm) | $M_{Rd}/M_{Ed}$ |
|---|---|---|---|---|---|---|---|
| A | 1885 | 400 | 101,6 | 0,14 | 520,6 | 122,7 | 4,24 |
| N | 2275 | 1200 | 40,9 | 0,04 | 649,9 | 505,8 | 1,28 |
| B | 2275 | 400 | 101,6 | 0,16 | 620,8 | 580,4 | 1,07 |
| M | 1137 | 1200 | 20,4 | 0,03 | 328,5 | 306,9 | 1,07 |
| C | 2275 | 400 | 101,6 | 0,16 | 620,8 | 580,4 | 1,07 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| P | 2275 | 1200 | 40,9 | 0,05 | 649,9 | 505,8 | 1,28 |
| D | 1885 | 400 | 101,6 | 0,14 | 520,6 | 122,7 | 4,24 |

*Tab.14. Neutral axis depth*

SHEAR ULTIMATE LIMIT STATE VERIFICATION

In Fig.45. the design shear diagram obtained through the previous structural analysis are shown.

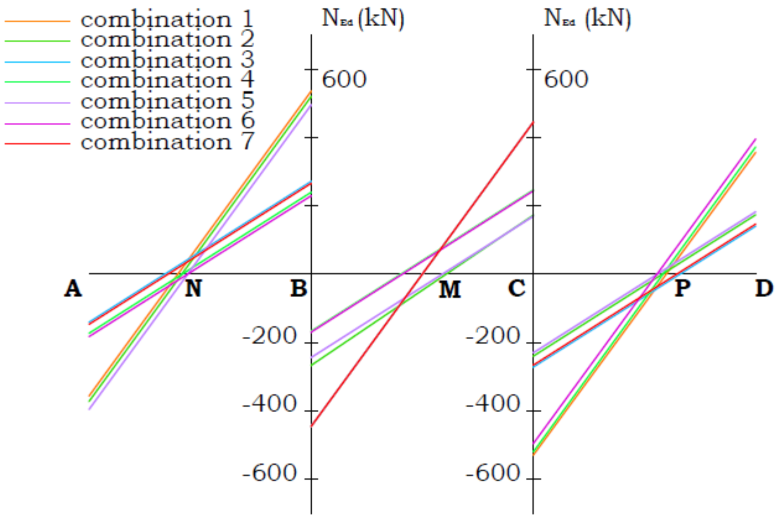



*Fig.45. Design shear diagram*

The transversal reinforcement ratio needs to comply with the following limit [EC2 – 9.2.2(5) and Expression 9.5N]:

$$\rho_{sw} = \frac{A_{sw}}{b_w s} \geq 0{,}08 \frac{\sqrt{f_{ck}}}{f_{yk}} = 0{,}0011$$

where appropriate numeric value for concrete and steel strengths have been adopted (C40/50, B450C).

The maximum longitudinal spacing between stirrups is [EC2 – 9.2.2(6) and Expression 9.6N]

$$s_{t,max} = 0{,}75 \cdot d \cdot (1 + \cot\alpha) = 0{,}75 \cdot 747 = 560\ mm$$

whereas the transverse spacing of the legs of stirrups needs not to exceed the value [EC2 – 9.2.2(8) and Expression 9.8N]

$$s_{i,max} = 0{,}75 \cdot d = 0{,}75 \cdot 747 = 560\ mm \leq 600\ mm$$

Assuming stirrups ϕ8/250 mm with two legs it is

$$\rho_{sw} = \frac{100}{400 \cdot 250} = 0{,}001 > \rho_{sw,min}$$

The corresponding value of the shear resistance then is [EC2 – Expression 6.8]

$$V_{Rd,s} = 0,9 \cdot d \frac{A_{sw}}{s} \cdot f_{ywd} \cdot \cot\theta = 0,9 \cdot 747 \cdot \frac{100}{250} \cdot 391 \cdot 2 = 210,3\ kN$$

where ctgθ = 2 has been assumed.

Overlapping the design shear with the resistant shear diagram Fig.46. it can be easily noted that the provided shear reinforcement is not sufficient in a 1,35 m long portion of beam near the external column as well as two parts at both continuity supports, respectively 1,7 m and 2,4 m long.

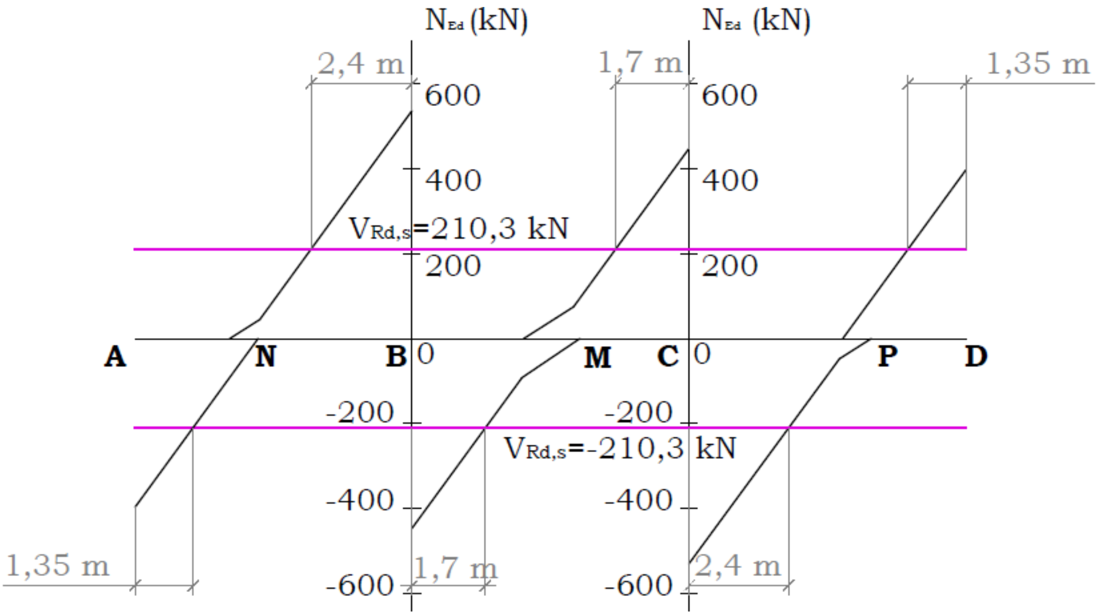



*Fig.46. Design shear and resistant shear*

The required trasversal reinforcement can be then evaluated with the following expression:

$$\rho_{sw} \geq \rho_{sw,rqd} = \frac{V_{Ed}}{0,9 \cdot d \cdot b_w \cdot f_{yd} \cdot \cot\theta}$$

where the design shear $V_{Ed}$ can be computed at a distance $\overline{x}$ from the ideal support equal to the semi-width of the column. The results of such calculations and verifications in accordance with EC2 – Expression 6.8 are reported in Tab. 15

| Zone | $\overline{x}$ (m) | $\lvert V_{Ed}(\overline{x})\rvert$ (kN) | $\rho_{sw\ rqd}$ | ϕ/spacing $\rho_{sw}$ | $V_{Rd,s}$ (kN) |
|---|---|---|---|---|---|
| Left end Combination 5 | 0,2 | 369 | 0,0018 | ϕ8/125mm 0,002 | 420,6 |
| Continuity B (left) Combination 1 | 6,5 – 0,2 = 6,3 | 509 | 0,0024 | ϕ8/100mm 0,0025 | 525,7 |
| Continuity B (right) Combination 1 | 6,5 + 0,2 = 6,7 | 419 | 0,0020 | ϕ8/125mm 0,002 | 420,6 |
| Continuity C (left) Combination 1 | 13,0 – 0,2 =12,8 | 419 | 0,0020 | ϕ8/125mm 0,002 | 420,6 |
| Continuity C (right) Combination 1 | 13,0 + 0,2 =13,2 | 509 | 0,0024 | ϕ8/100mm 0,0025 | 525,7 |
| Right end Combination 6 | 19,5 – 0,2 =19,3 | 369 | 0,0018 | ϕ8/125mm 0,002 | 420,6 |

*Tab.15. ULS dimensioning and verification of shear reinforcement*

According to EC2 – Expression 6.9

$$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu_1 \cdot f_{cd} \cdot \frac{\cot\theta + \cot\alpha}{1 + \cot\theta^2} =$$

$$= 400 \cdot 0{,}9 \cdot 747 \cdot 0{,}59 \cdot 22{,}67 \cdot \frac{2}{1+4} = 1438{,}8\ kN > V_{Ed}$$

where $\nu_1 = \nu = 0{,}7 \cdot (1 - f_{ck}/250) = 0{,}59$ [EC2 – Expression 6.6N – National Annex].

SERVICEABILITY LIMIT STATES: STRUCTURAL ANALYSIS

The bending moments under the characteristic combination of actions [EC2 – Expression 6.14b] are calculated in order to verify the serviceability limit states.

$$\sum_{j \geq 1} G_{kj} + Q_{1k}$$

The most unfavourable situation implies that the leading action is the imposed load for buildings; $Q_{1k}$. Referring to the following combinations of actions the analytical expression of the bending moment diagrams can be calculated.

*Load Combination 1' (maximum moment at the continuity supports).*

Permanent loads on three spans: $G_1 + G_2 = 38{,}7 + 8{,}5 = 47{,}2 \frac{kN}{m}$

Variable load on three spans: $Q_1 = 49{,}1 \frac{kN}{m}$

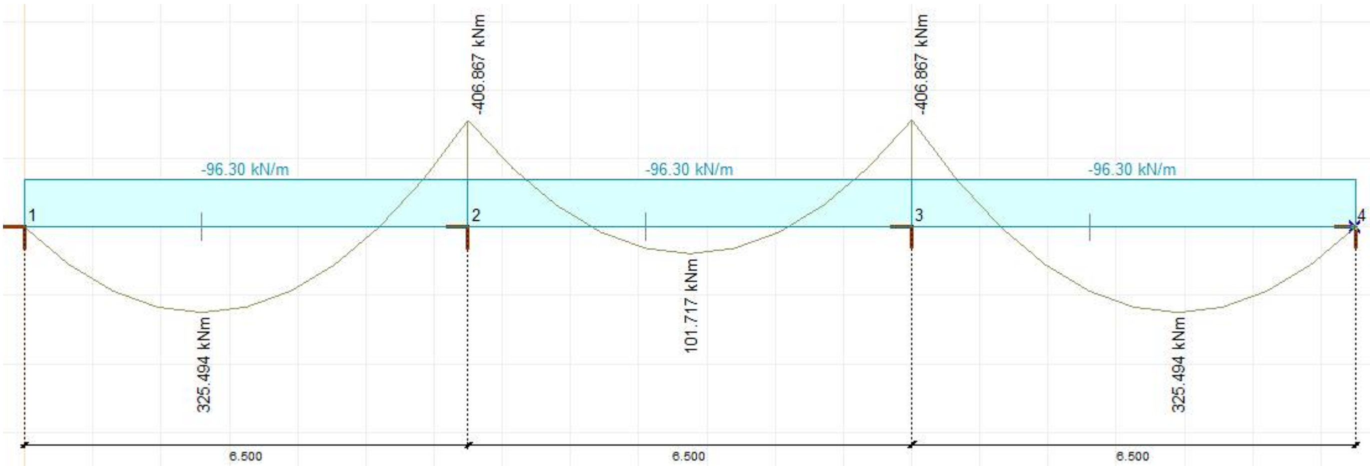



*Fig.47. Bending moment diagram* computed with Axis VM Software

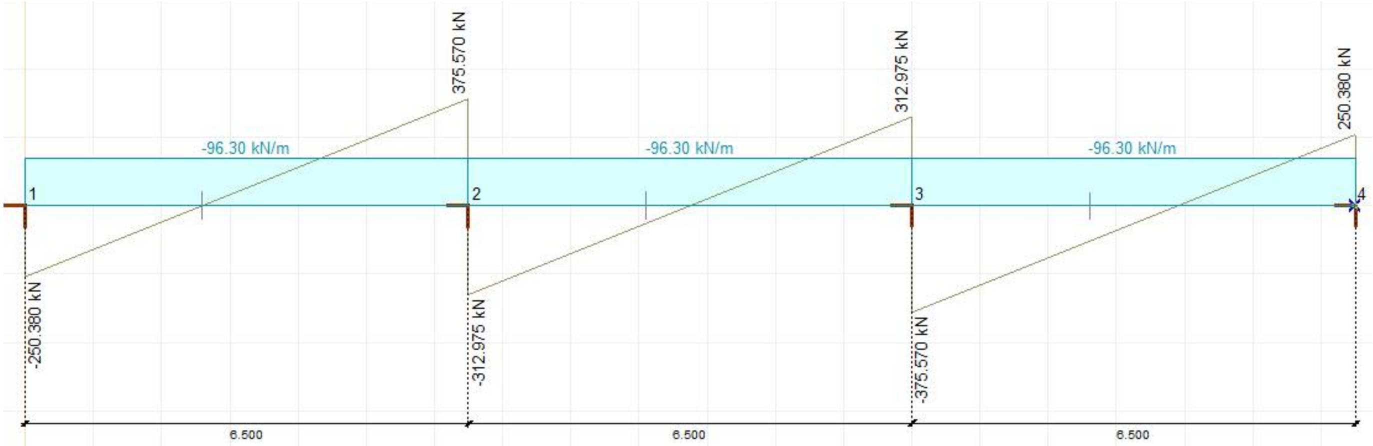


*Fig.48. Shear force diagram computed with Axis VM Software*

<u>Load Combination 2' (maximum moment in the left span).</u>

Permanent loads on three spans: $G_1 + G_2 = 38{,}7 + 8{,}5 = 47{,}2\,\frac{kN}{m}$

Variable load on three spans: $Q_1 = 49{,}1\,\frac{kN}{m}$

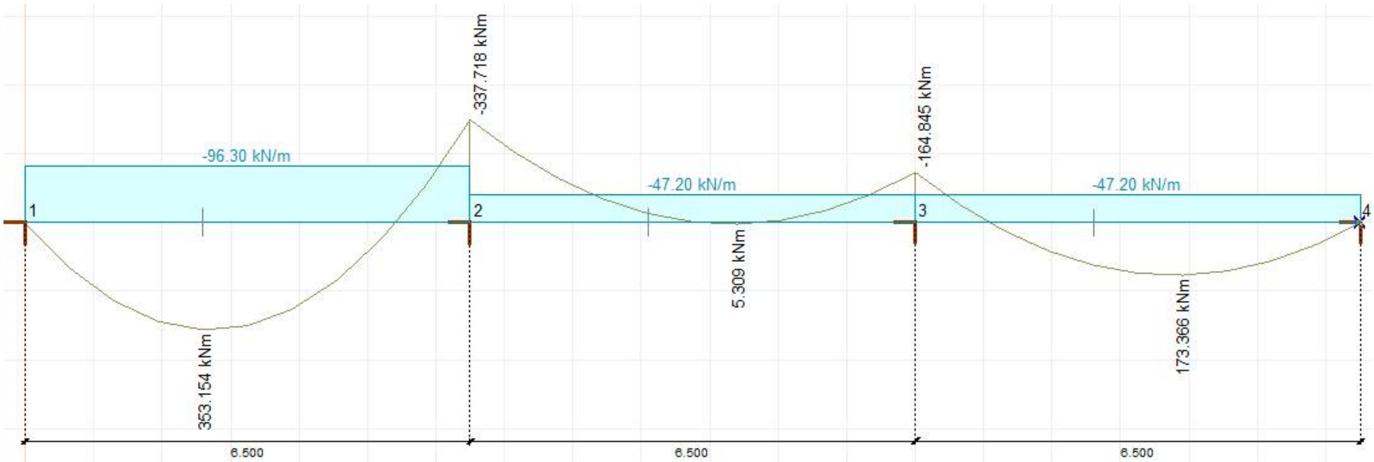


*Fig.49. Bending moment diagram computed with Axis VM Software*

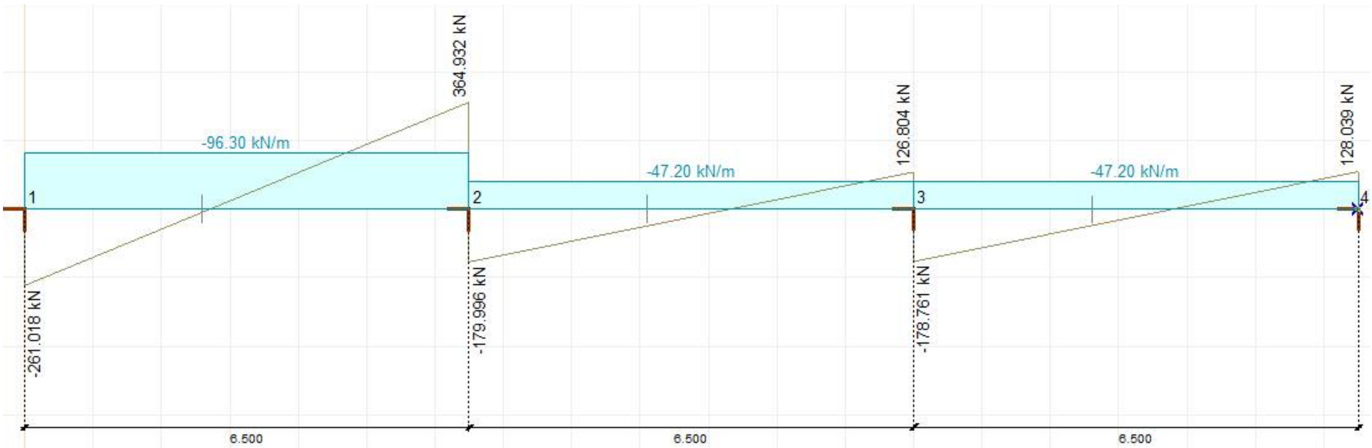


*Fig.50. Shear force diagram computed with Axis VM Software*

*Load Combination 3' (maximum moment in the middle span).*

Permanent loads on three spans: $G_1 + G_2 = 38{,}7 + 8{,}5 = 47{,}2 \frac{kN}{m}$

Variable load on three spans: $Q_1 = 49{,}1 \frac{kN}{m}$

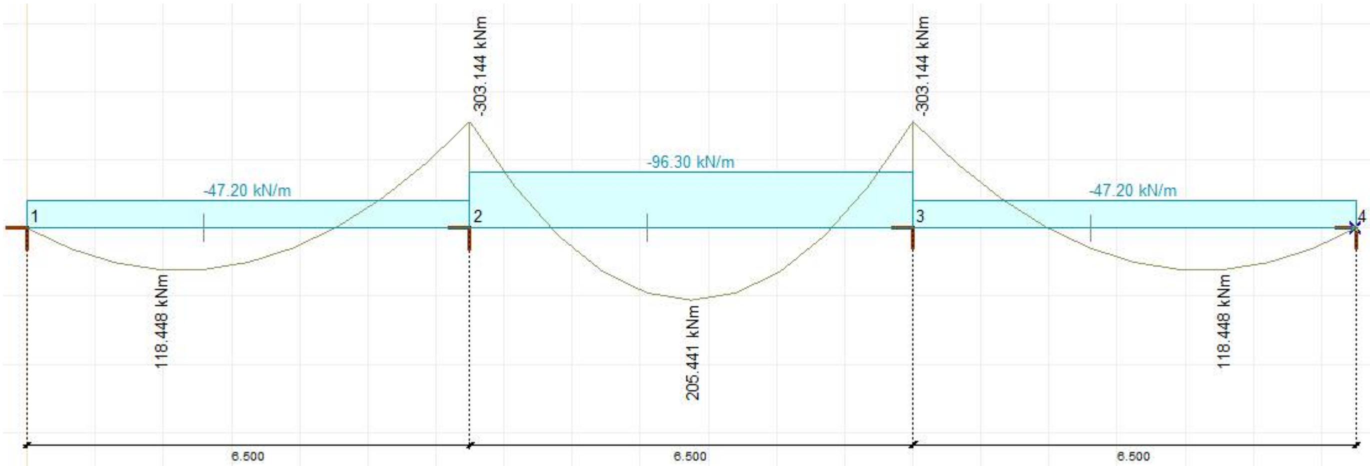



*Fig.51. Bending moment diagram computed with Axis VM Software*

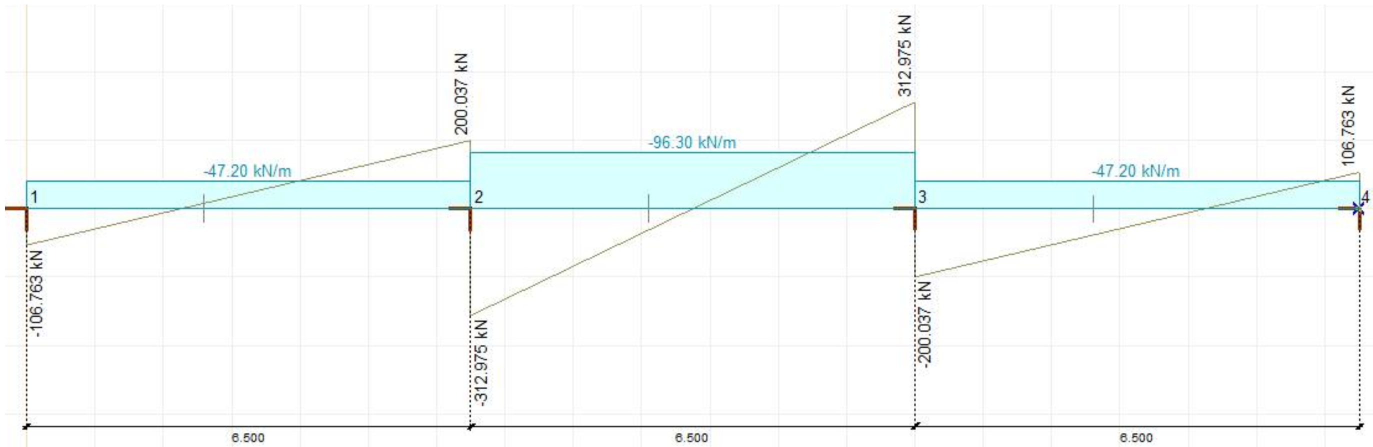



*Fig.52. Shear force diagram computed with Axis VM Software*

*Load Combination 4' (maximum moment in the right span).*

Permanent loads on three spans: $G_1 + G_2 = 38{,}7 + 8{,}5 = 47{,}2 \frac{kN}{m}$

Variable load on three spans: $Q_1 = 49{,}1 \frac{kN}{m}$

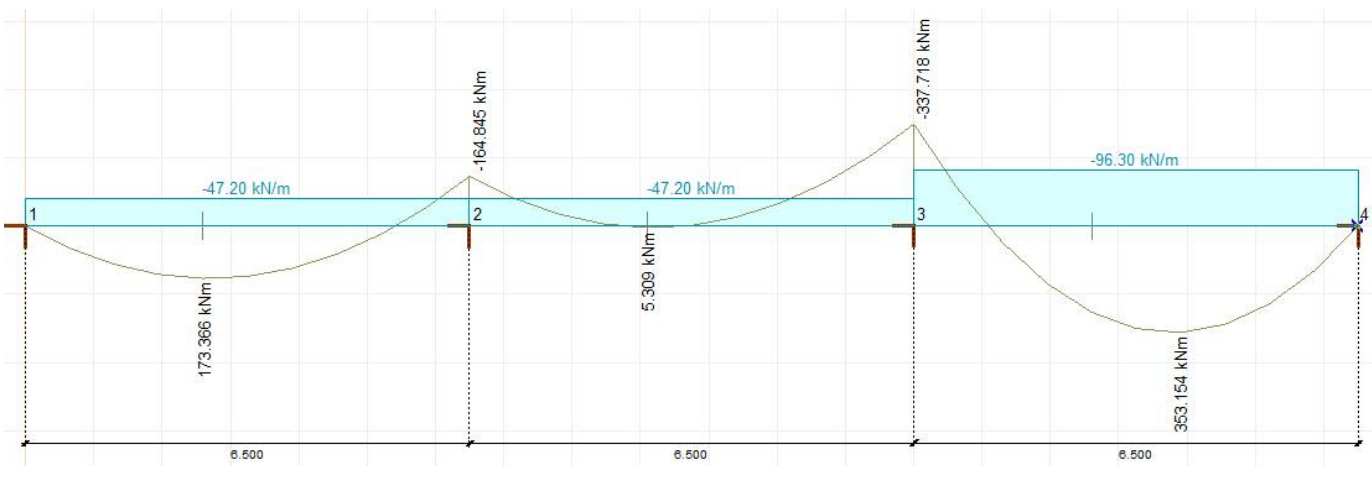


*Fig.53. Bending moment diagram computed with Axis VM Software*

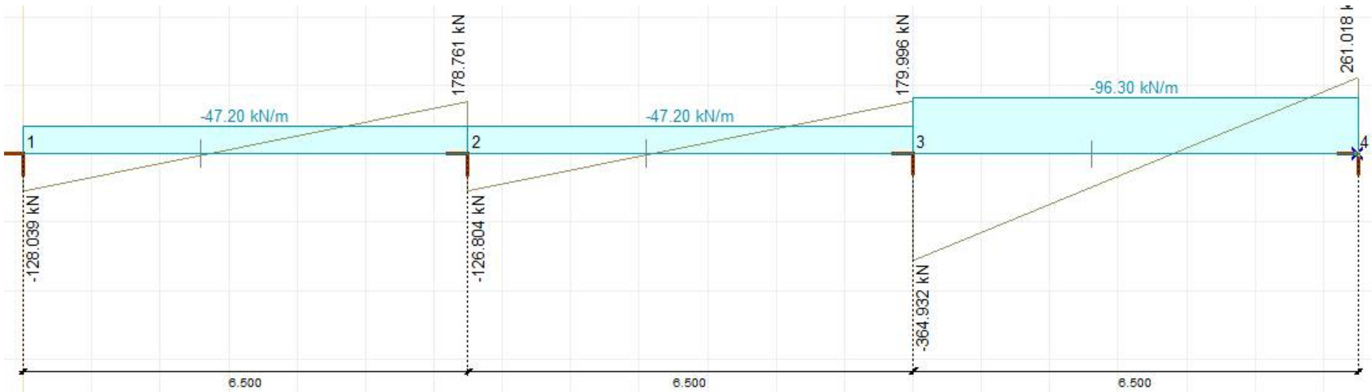



*Fig. 54. Shear force diagram computed with Axis VM Software*

Load Combination 5' (maximum moment in the left span).

Permanent loads on three spans: $G_1 + G_2 = 38{,}7 + 8{,}5 = 47{,}2\,\frac{kN}{m}$

Variable load on three spans: $Q_1 = 49{,}1\,\frac{kN}{m}$

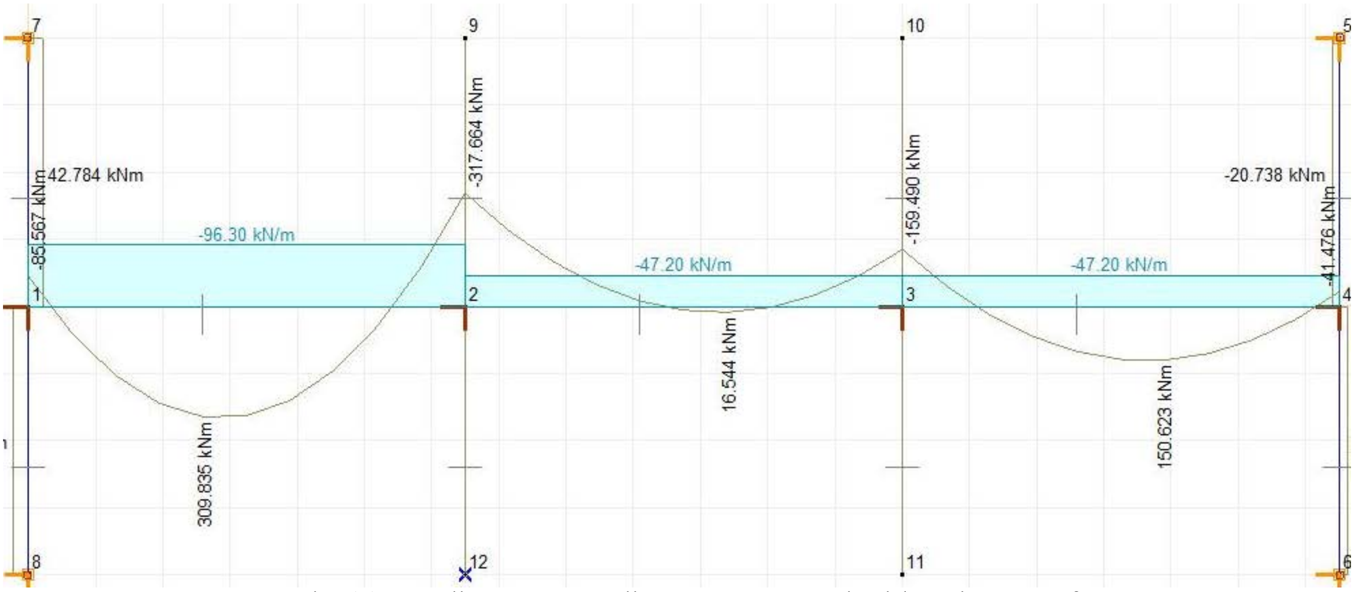



*Fig.55. Bending moment diagram computed with Axis VM Software*

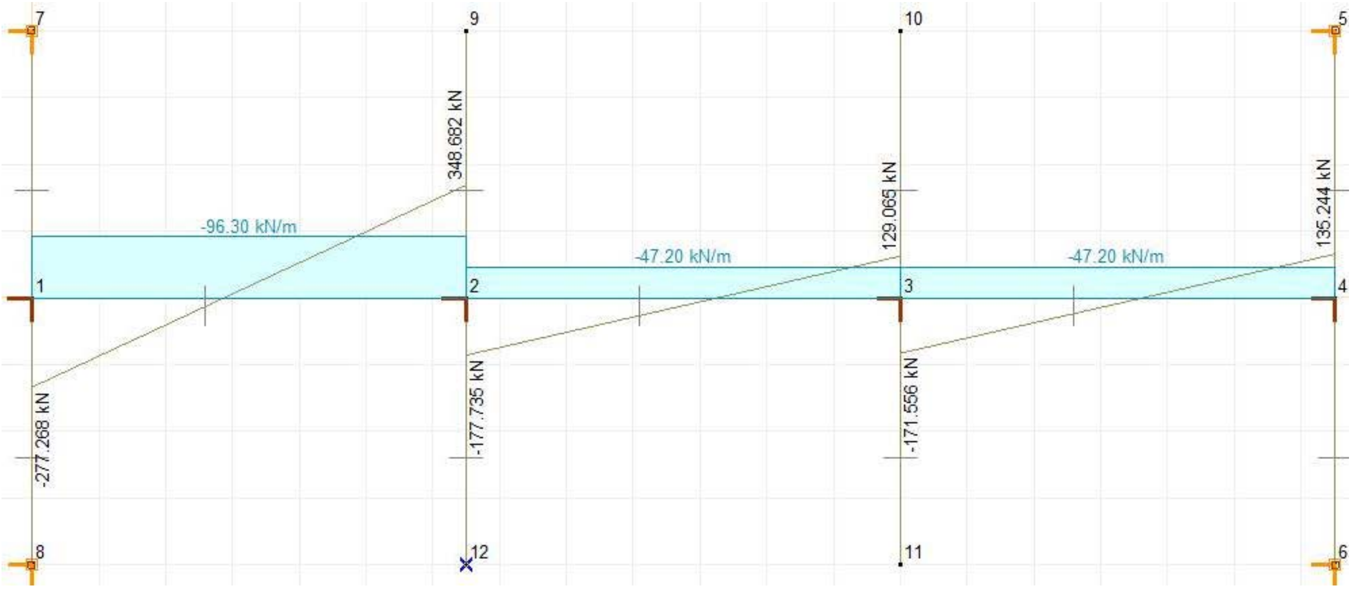



*Fig.56. Shear force diagram computed with Axis VM Software*

<u>Load Combination 6' (maximum moment in the right span).</u>

Permanent loads on three spans: $G_1 + G_2 = 38{,}7 + 8{,}5 = 47{,}2\,\frac{kN}{m}$

Variable load on three spans: $Q_1 = 49{,}1\,\frac{kN}{m}$

The frame is symmetrical, so the internal force diagrams are symmetrical to load combination 5'.

<u>Load Combination 7' (maximum moment in the middle span).</u>

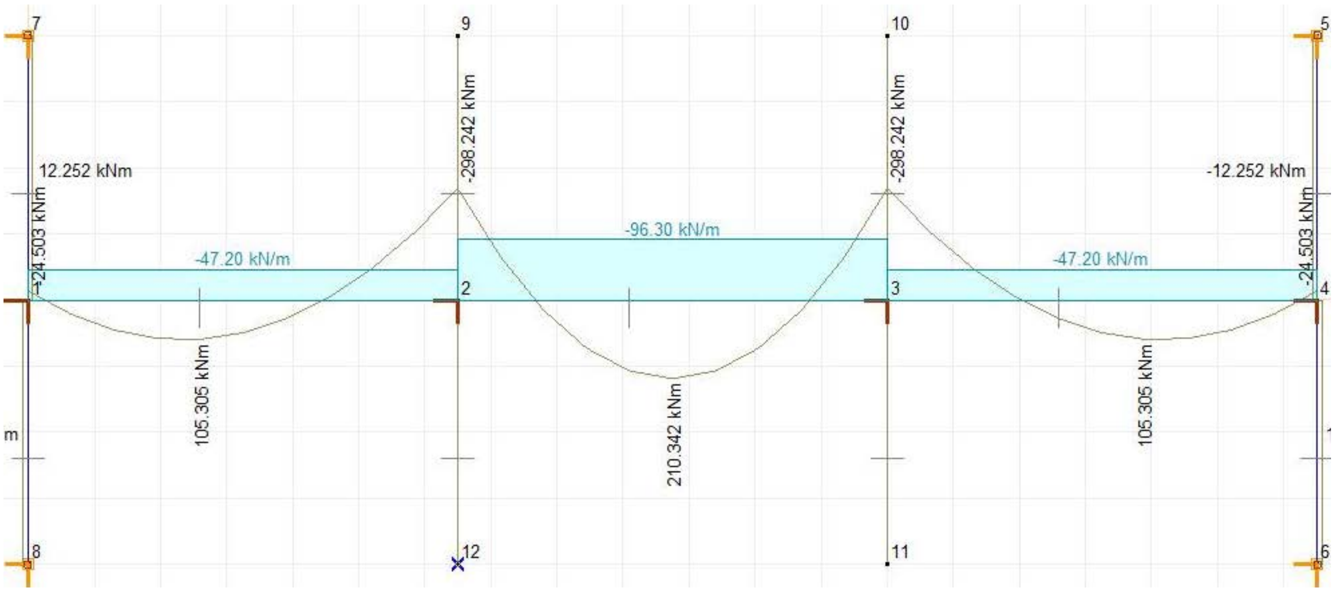



*Fig.57. Bending moment diagram computed with Axis VM Software*

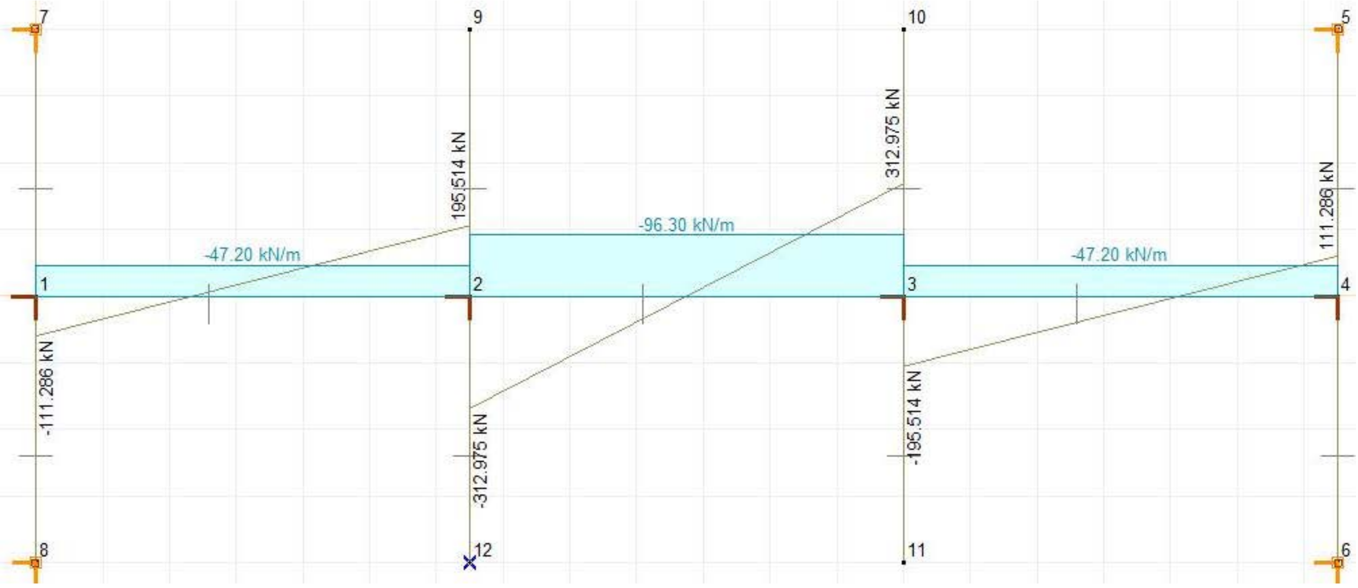



*Fig.58. Shear force diagram computed with Axis VM Software*

SERVICEABILITY LIMIT STATES VERIFICATIONS: STRESS LIMITATION AND CRACK CONTROL

The translational equilibrium equation is

$$\frac{1}{2}\sigma_c \cdot b \cdot x + \sigma_s^{'} \cdot A_s^{'} - A_s \cdot \sigma_s$$

Assuming an elastic behaviour for both concrete and steel ($\sigma = E\ \varepsilon$) and plane diagram of strains

$$\sigma_s = \alpha_e \cdot \sigma_c \cdot \frac{d-x}{x}$$

$$\sigma_s^{'} = \alpha_e \cdot \sigma_c \cdot \frac{x-d'}{x}$$

where the coefficient $\alpha_e = E_s/E_c$ is assumed equal to 15.

$$b \cdot \frac{x^2}{2} + \alpha_e \cdot (A_s + A_s^{'})x - \alpha_e \cdot (A_s \cdot d + A_s^{'}) = 0$$

The position of the neutral axis, x, can be obtained from the previous equation (null static moment of the homogenised cross-section).

Through the rotational equilibrium about the barycentre of the tensions, the maximum compression in concrete can be calculated and compared to the allowable value $\sigma_{c,adm} = 0{,}6\ f_{ck} = 24$ MPa [EC2 – 7.2(3)]

$$\frac{1}{2} \cdot \sigma_c \cdot b \cdot x \cdot \left(d - \frac{x}{3}\right) = M$$

$$\sigma_c = \frac{2 \cdot M}{b \cdot x \cdot \left(d - \frac{x}{3}\right)} < \sigma_{c,adm}$$

The stress in the reinforcement can be easily obtained from the previous formula

$$\sigma_s = \alpha_e \cdot \sigma_c \cdot \frac{d-x}{x} < \sigma_{s,adm}$$

and then compared to the admissible value $\sigma_{s,adm}$ = 0.8 $f_{yk}$ = 360 MPa [EC2 – 7.2(5)]. In Tab.16. the results of such calculations are shown.

| Sec. | $M_{Ed}$ (kNm) | $A_s$ (mm²) | b (mm) | x (mm) | $\sigma_c$ (N/mm²) | <0,6$f_{ck}$? | $\sigma_s$ (N/mm²) | <0,8$f_{yk}$? |
|---|---|---|---|---|---|---|---|---|
| A | -85,6 | 1885 | 400 | 261,9 | 2,5 | OK! | 68,8 | OK! |
| N | 353,2 | 2275 | 1200 | 179,6 | 4,8 | OK! | 226,0 | OK! |
| B | -406,9 | 2275 | 400 | 281,8 | 11,1 | OK! | 273,7 | OK! |
| M | 210,3 | 1137 | 1200 | 132,2 | 3,8 | OK! | 263,1 | OK! |
| C | -406,9 | 2275 | 400 | 281,8 | 11,1 | OK! | 273,7 | OK! |
| P | 353,2 | 2275 | 1200 | 179,6 | 4,8 | OK! | 226,0 | OK! |
| D | -85,6 | 1885 | 400 | 261,9 | 2,5 | OK! | 68,8 | OK! |

*Tab.16. Tension limitation*

In order to verify whether the crack control without direct calculation is sufficient, it is necessary to compare the tension in the reinforcement with the limits given in EC2 – Table 7.2 in function of the bar size and the admissible crack width. Assuming Ø =20 mm and $w_k$ = 0,4 mm, the maximum stress in the steel is 240 N/mm², which is always greater than the values shown in Tab.16.

Serviceability Limit States: control of cracking with direct calculation

The control of cracking is hereafter performed through a direct calculation of crack width in accordance with EC2 – 7.3.4.

The characteristic value of the crack width is [EC2 – Expression 7.8]

$$w_k = s_{max}(\varepsilon_{sm} - \varepsilon_{cm})$$

The maximum crack spacing can be calculated with the following expression [EC2 – Expression 7.11]

$$s_{rmax} = 3{,}4 \cdot c + 0{,}425 \cdot k_1 \cdot k_2 \cdot \frac{\emptyset}{\rho_{eff}}$$

where $k_1$ = 0,8 for high bond bars

$k_2$ = 0,5 for bending

$$\rho_{eff} = \frac{A_s}{A_{c,eff}} = \frac{A_s}{b_t h_{eff}}$$

$h_{eff}$ = min [2,5(h – d), (h – x)/3, h/2]

$b_t$ is the width of the cross-section in the tension zone

The mean strain between cracks is [EC2 – Expression 7.9]

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s - k_t \frac{f_{ct}}{\rho_{eff}} \cdot \left(1 + \frac{E_s}{E_{cm}} \cdot \rho_{eff}\right)}{E_s} \geq 0{,}6 \cdot \frac{\sigma_s}{E_s}$$

where $k_t$ = 0,4 is assumed according to long term loading specifications.

In Tab.17. the control of cracking with direct calculation is shown.

| Sec. | x (mm) | 2,5·(h – d) (mm) | (h – x)/3 (mm) | h/2 (mm) | $b_t$ (mm) | $h_{eff}$ (mm) | $A_s$ (mm²) | $\rho_{eff}$ (%) |
|---|---|---|---|---|---|---|---|---|
| A | 261,9 | 132,5 | 179,4 | 400 | 1200 | 132,5 | 1885 | 1,2 |
| N | 179,6 | 132,5 | 206,8 | 400 | 400 | 132,5 | 2275 | 4,3 |
| B | 281,8 | 132,5 | 172,7 | 400 | 1200 | 132,5 | 2275 | 1,4 |
| M | 132,2 | 132,5 | 222,6 | 400 | 400 | 132,5 | 1137 | 2,1 |
| C | 281,8 | 132,5 | 172,7 | 400 | 1200 | 132,5 | 2275 | 1,4 |
| P | 179,6 | 132,5 | 206,8 | 400 | 400 | 132,5 | 2275 | 4,3 |
| D | 261,9 | 132,5 | 179,4 | 400 | 1200 | 132,5 | 1885 | 1,2 |

| Sec. | $\rho_{eff}$ (%) | $s_{rmax}$ (mm) | $\sigma_s$ (N/mm²) | ($\varepsilon_{sm}$-$\varepsilon_{cm}$) | 0,6·$\sigma_s$/$E_s$ | $w_k$ (mm) | <0,4 mm |
|---|---|---|---|---|---|---|---|
| A | 1,2 | 126,1 | 68,8 | 0,000298 | 0,000206 | 0,04 | OK! |
| N | 4,3 | 121,3 | 226,0 | 0,001088 | 0,000678 | 0,13 | OK! |
| B | 1,4 | 125,1 | 273,7 | 0,001323 | 0,000821 | 0,17 | OK! |
| M | 2,1 | 123,2 | 263,1 | 0,001272 | 0,000789 | 0,16 | OK! |
| C | 1,4 | 125,1 | 273,7 | 0,001323 | 0,000821 | 0,17 | OK! |
| P | 4,3 | 121,3 | 226,0 | 0,001088 | 0,000678 | 0,13 | OK! |
| D | 1,2 | 126,1 | 68,8 | 0,000298 | 0,000206 | 0,04 | OK! |

*Tab.17. SLS - Control of cracking with direct calculation*

SERVICEABILITY LIMIT STATES VERIFICATIONS: DEFLECTION CONTROL

Eurocode 2 permits not to calculate explicitly the deflection of members if the span/effective depth ratio, l/d, is less than the following limit [EC2 – Expression 7.16a]

$$\frac{l}{d} = K\left[11 + 1{,}5 \cdot \sqrt{f_{ck}} \cdot \frac{\rho_0}{\rho} + 3{,}2 \cdot \sqrt{f_{ck}} \cdot \left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \qquad \rho \leq \rho_0$$

The reinforcement ratio at left mid-span is

$$\rho = \frac{A_s}{b \cdot d} = \frac{2275}{1200 \cdot 747} = 0{,}0025$$

whereas the reference reinforcement ratio is

$$\rho_0 = \sqrt{f_{ck}} \cdot 10^{-3} = 0{,}0063 > \rho$$

Assuming [EC2 – Expression 7.17]

$$\frac{\sigma_s}{310} = \frac{500}{f_{yk}} \cdot \frac{A_{s,prov}}{A_{s,req}} = \frac{500}{450} \cdot \frac{2275}{1764{,}4} = 1{,}43$$

and K = 1,3 for end spans of continuous beams the result is $(l/d)_{max} \cong 97{,}96$.

Being l = 6,5 m and d = 0,747 m, l/d = 8,7 << 97,96 and the direct calculation of deflections can be omitted.

*Reinforcement arrangement*

According to EC2 – 9.2.1.3(1) the reinforcement arrangement needs to be determined in order to resist the envelope of the acting tensile force including the effect of inclined cracks in webs or flanges. This additional tensile force is estimated by shifting in the most unfavourable direction the moment curve a distance

$$a_l = \frac{z \cdot (\cot\theta - \cot\alpha)}{2} = 0{,}9 \cdot d \cdot \frac{(\cot\theta - \cot\alpha)}{2} = 0{,}9 \cdot 747 = 672{,}3\,mm$$

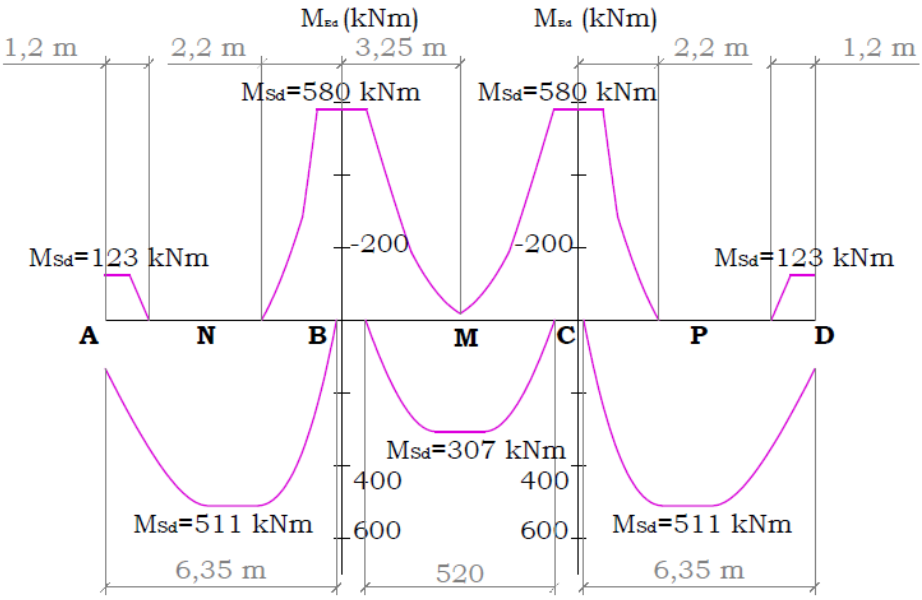



*Fig.59. Shifted envelope of bending moments*

In order to determine the total length of the reinforcing bars it is necessary to evaluate the anchorage length. The basic required anchorage length in a straight bar is [EC2 – Expression 8.3]

$$l_{b,rgd} = \frac{\emptyset}{4} \cdot \frac{\sigma_s}{f_{bd}}$$

where $\sigma_s$ is the design stress of the bar at the position from where the anchorage is measured from (= $f_{yd}$ in this case)

$f_{bd} = 2{,}25\ \eta_1\ \eta_2\ f_{ctd}$ is the design value of the ultimate bond stress for ribbed bars [EC2 – Expression 8.2]

Assuming $\eta_1 = 1{,}0$ (good bond conditions according to EC2 – 8.4.2 and Figure 8.2) and $\eta_2 = 1{,}0$ ($\phi < 32$ mm) and being $f_{ctd} = 1{,}2$ N/mm$^2$ the design ultimate bond strength is

$f_{bd} = 2{,}7$ N/mm$^2$

and the basic anchorage length is

$l_{b,rqd} \cong 36{,}2\ \phi$

$l_{b,rqd} \cong 725$ mm for $\phi 20$ bars

$l_{b,rqd} \cong 650$ mm for $\phi 18$ bars

The design value of the anchorage length is calculated through the following [EC2 – Expression 8.4]

$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,rqd}$

where

$\alpha_1 = 1{,}0$ for straigth bars both in tension and compression

for bars in tension:

$$\alpha_2 = 1 - 0{,}15 \cdot \frac{c_d - \emptyset}{\emptyset} \leq 1{,}0$$

Looking at Fig.60 and assuming:

$a = \max\ (20\ mm, \emptyset, d_g + 5\ mm)$ [EC2 – 8.2(2)]

for $d_g = 20$ mm – maximum aggregate dimension

$a = 25$ mm (bar clear spacing)

$c_1 = c = 35$ mm

*$c_d = \min\ (a/2, c, c_1) = 35$ mm*

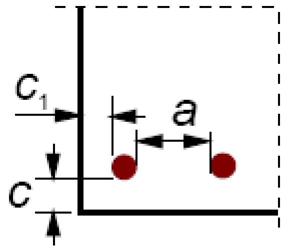



*Fig.60 Values of $c_d$ [EC2 - Figure 8.3a]*

$\alpha_2 = 0{,}89$ for $\emptyset 20$ bars

$\alpha_2 = 0{,}86$ for $\emptyset 18$ bars

for bars in compression $\alpha_2 = 1{,}0$

$\alpha_3 = 1 - K \cdot \lambda = 1 - 0{,}05 \cdot 1{,}02 = 0{,}949$

where $K = 0{,}05$ [EC2 – Figure 8.4]

$\lambda = (\Sigma A_{st} - \Sigma A_{stmin})/A_s = (400 - 78{,}5)/314 = 1{,}02$

$\Sigma A_{st} = 8 \cdot 50 = 400\ mm^2$ cross-sectional area of the transverse reinforcement along the anchorage length (stirrups $\phi 8/150$)

$\Sigma A_{stmin} = 0{,}25 \cdot A_s = 0{,}25 \cdot 315\ mm^2 = 78{,}5\ mm^2$

$\alpha_4 = 1{,}0$ in absence of welded transverse bars along the design ancorage length

$\alpha_5 = 1{,}0$ in absence of further evaluation of the pressure transverse to the plane of splitting along the design anchorage length.

The design anchorage of tension reinforcing bars in the end is

$l_{bd} = 0{,}89 \cdot 0{,}949 \cdot l_{b,rqd} \cong 585$ mm (for $\phi$ 20 bars)

$l_{bd} = 0{,}86 \cdot 0{,}949 \cdot l_{b,rqd} \cong 530$ mm (for $\phi$ 18 bars)

The so-determined values satisfy the minimum anchorage length in accordance with EC2 – Expression 8.6.

$l_{bd} \geq l_{b,min}$ = max (0,3 $l_b$, 10Ø, 100 mm), where $l_b = \dfrac{\phi}{4}\dfrac{f_{yd}}{f_{bd}}$

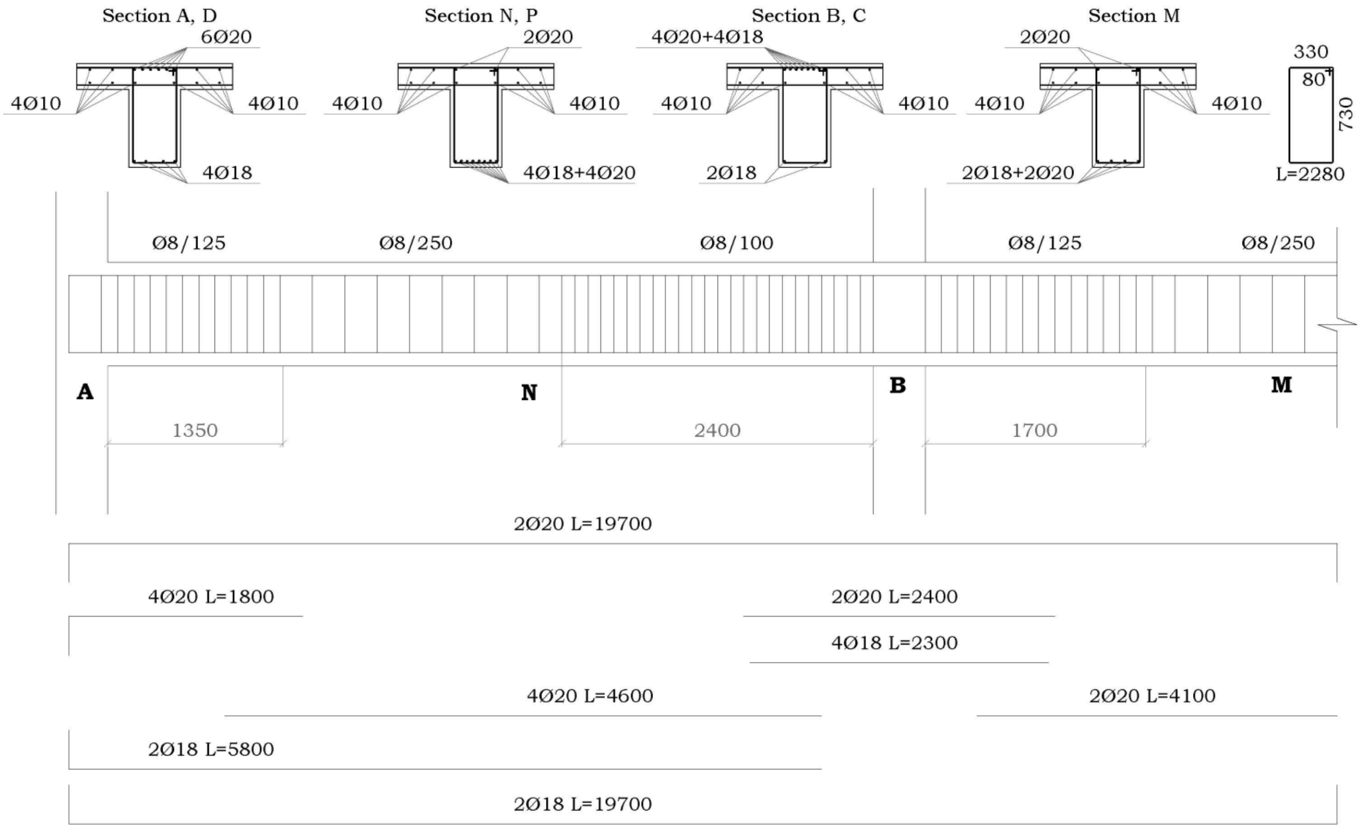
$l_{b,min}$ = 260 mm (for Ø 20 bars)

$l_{b,min}$ = 235 mm (for Ø 18 bars)

The following values of the design anchorage length are then adopted.

$l_{bd}$ = 600 mm for Ø 20 bars

$l_{bd}$ = 550 mm for Ø 18 bars




The final reinforcement arrangement is shown in Fig.61.

*Fig.61. Reinforcement arrangement of symmetrical half of the building*

## 4. COLUMNS

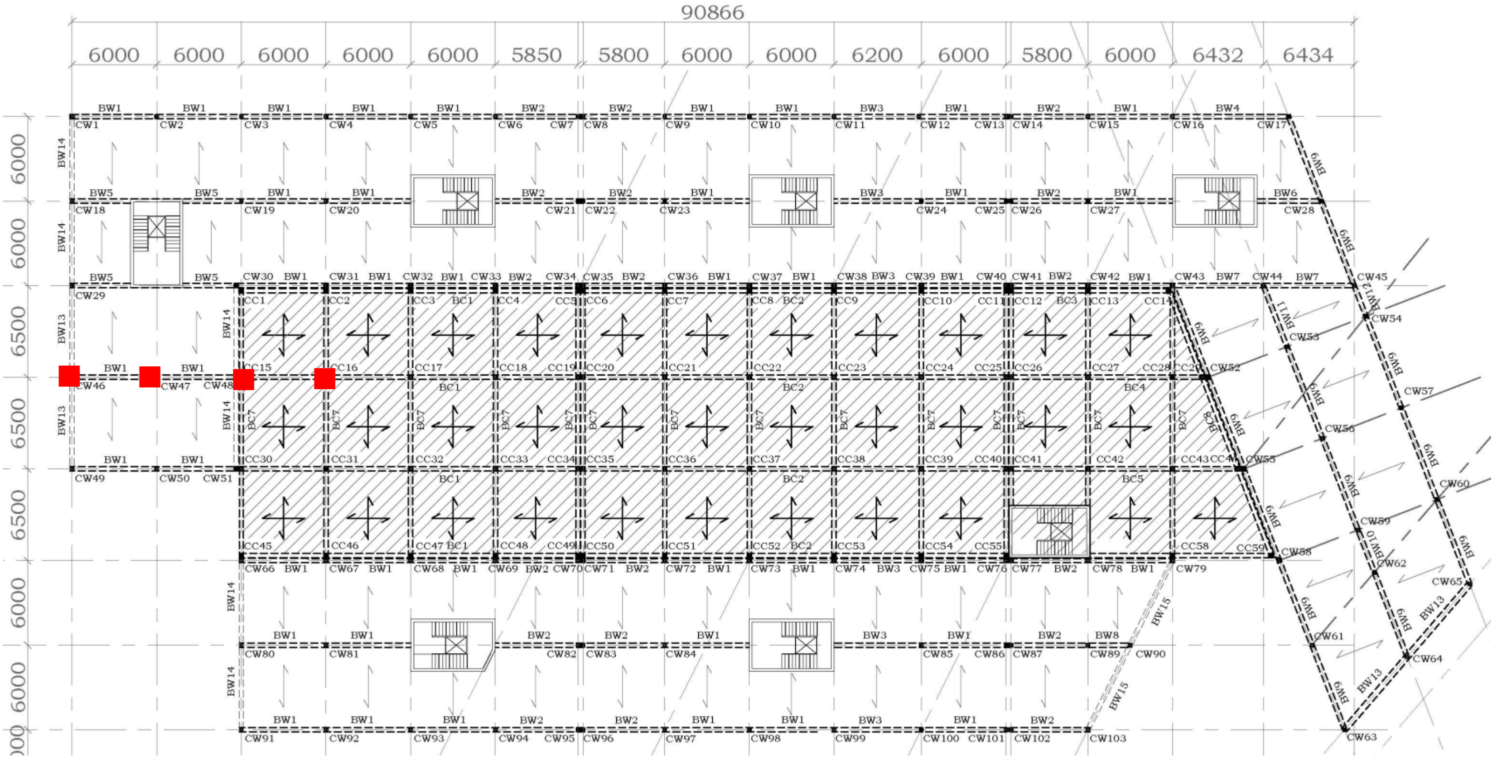


*Fig.62. Column design*

### 4.1. GLULAM COLUMN

An inside column and an external column will be analysed in this section. The first one is mainly subjected to axial load whereas for the second one bending moments are significant. The "influence areas" method will be used in both cases in order to determine the axial load.

COLUMN CW47 CENTRED AXIAL FORCE

Influence area: $6{,}0 \cdot 6{,}5\ m^2 = 39{,}0\ m^2$

Roof loads:

roof slab $6{,}48\ kN/m^2 \cdot 39{,}0\ m^2 = 252{,}72\ kN$

weight of the beam

$0{,}165\ m \cdot 0{,}54\ m \cdot 6{,}5\ m \cdot 4{,}56\ kN/m^3 = 2{,}64\ kN$

snow $0{,}768\ kN/m^2 \cdot 39{,}0\ m^2 = 29{,}95\ kN$

Typical floor loads:

Slab self-weight $3{,}28\ kN/m^2 \cdot 39{,}0\ m^2 = 127{,}92\ kN$

vertical closures self-weight $5{,}59\ kN/m \cdot 6{,}5\ m = 36{,}34\ kN$

Variable loads (inside partitions included): $3{,}2\ kN/m^2 \cdot 39{,}0\ m^2 = 124{,}8\ kN$

Weight of the beam

$0{,}165\ m \cdot 0{,}54\ m \cdot 6{,}5\ m \cdot 4{,}56\ kN/m^3 = 2{,}64\ kN$

For the calculation of columns only, a reduction factor can be applied to variable loads [EC1-1 – 6.3.1.2(10) – National Annex]

$$\alpha_A = \frac{5}{7} \cdot \psi_0 + \frac{A_0}{A} \leq 1{,}0$$

where: $\psi_0 = 0{,}7$

$A_0 = 10\ m^2$

A is the influence area of the column considered

For column CW47 $\alpha_A \cong 0{,}76$.

Loads on every storey are:

- roof + roof floor
  - ° permanent loads 255,36 kN
  - ° variable loads 29,95 kN
- 6° floor

° permanent loads 166,9 kN

° variable loads 94,85 kN (= 0,76·124,8 kN)

- 5° floor

° permanent loads 166,9 kN

° variable loads 94,85 kN (= 0,76·124,8 kN)

- 4° floor

° permanent loads 166,9 kN

° variable loads 94,85 kN (= 0,76·124,8 kN)

- 3° floor

° permanent loads 166,9 kN

° variable loads 94,85 kN (= 0,76·124,8 kN)

- 2° floor

° permanent loads 166,9 kN

° variable loads 94,85 kN (= 0,76·124,8 kN)

- 1° floor

° permanent loads 166,9 kN

° variable loads 94,85 kN (= 0,76·124,8 kN)

- ground floor

° permanent loads 166,9 kN

° variable loads 94,85 kN (= 0,76·124,8 kN)

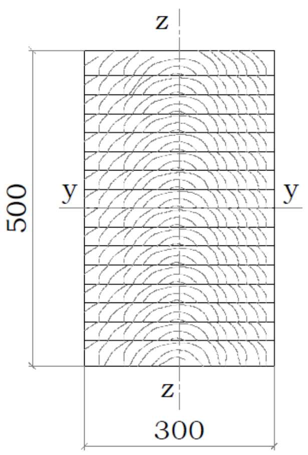
For the ULS combination of actions, a single multiplicative factor will be referred to, as a simplification: $\gamma_F$* is obtained as weighted mean of the coefficients $\gamma_G = 1{,}35$ and $\gamma_Q = 1{,}5$, respectively concerning permanent actions and variable actions.

$$\gamma_f^* = \frac{\gamma_G G_k + \gamma_Q Q_k}{G_k + Q_k} = \frac{1{,}35 \cdot 166{,}9 + 1{,}5 \cdot 94{,}85}{166{,}9 + 94{,}85} = 1{,}4$$

| Column | $F_k$ (kN) | Column self-weight (kN) | N (kN)<br>$N = \sum F_{kj}$ | $N_{Ed} = \gamma_f \cdot N$ (kN) |
|---|---|---|---|---|
| 6° floor | 285,31 | | 285,31 | 399,43 |

| | | | | |
|---|---|---|---|---|
| 5° floor | 261,75 | 2,39 | 549,45 | 769,23 |
| 4° floor | 261,75 | 2,39 | 813,59 | 1139,03 |
| 3° floor | 261,75 | 2,39 | 1077,73 | 1508,82 |
| 2° floor | 261,75 | 2,39 | 1341,87 | 1878,62 |
| 1° floor | 261,75 | 2,39 | 1606,01 | 2248,41 |
| Ground floor | 261,75 | 2,39 | 1870,15 | 2618,21 |

*Tab.18. Axial load calculation*




GLULAM COLUMN GEOMETRIC PROPERTIES

Material factor for glulam $\gamma_M = 1,25$

Size factor $k_h = 1,0$

Factor for medium-duration loading and service class 2 $k_{mod,med} = 0,8$

Load sharing factor $k_{sys} = 1,0$

Actual column length $L = 3,5$ m

Effective length of the column buckling about the $y$–$y$ axis

$L_{e.y} = 1,0 \cdot L = 3,5$ m

Effective length of the column buckling about the $z$–$z$ axis

$L_{e.z} = 1,0 \cdot L = 3,5$ m

Effective length of the member acting as a beam with a constant moment along its length (Table 4.2 (EC5, *Table 6.1*))

$l_{ef} = L = 3,5$ m

Width of the column $b = 300$ mm

Depth of the column $h = 500$mm

Cross-sectional area of the column

$$A = b \cdot h = 150 \cdot 10^3\ mm^2$$

Second moment of area of the column about the $y$–$y$ axes

$$I_y = \frac{b \cdot h^3}{12} = 31,25 \cdot 10^8\ mm^4$$

Section modulus about the $y$–$y$ axes

$$W_y = \frac{2 \cdot I_y}{h} = 12,5 \cdot 10^6\ mm^3$$

Radius of gyration about the $y$–$y$ axis

$$i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{31{,}25 \cdot 10^8}{150 \cdot 10^3}} = 144{,}34\, mm$$

Slenderness ratio about the *y–y* axis

$$\lambda_y = \frac{L_{e.y}}{i_y} = \frac{3500}{144{,}34} = 24{,}25$$

Second moment of area of the column about the *z–z* axis

$$I_z = \frac{h \cdot b^3}{12} = 11{,}25 \cdot 10^8\, mm^4$$

Radius of gyration of the column about the *z–z* axis

$$i_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{11{,}5 \cdot 10^8}{150 \cdot 10^3}} = 86{,}6\, mm$$

Slenderness ratio about the *z–z* axis

$$\lambda_z = \frac{L_{e.z}}{i_z} = \frac{3500}{86{,}6} = 40{,}42$$

AXIAL COMPRESSION CONDITION

Design compression stress:

$$\sigma_{c,0,d} = \frac{N_{Ed}}{A} = \frac{2618{,}21 \cdot 10^3}{150 \cdot 10^3} = 17{,}45 \frac{N}{mm^2}$$

Design compression strength:

$$f_{c,0,d} = \frac{k_{mod,med} \cdot k_{sys} \cdot f_{c,0,g,k}}{\gamma_M} = \frac{1{,}0 \cdot 0{,}8 \cdot 29{,}0}{1{,}25} = 18{,}56\ \frac{N}{mm^2}$$

Buckling resistance condition (5.4.1 (EC5, *6.3.2*)):

Relative slenderness about the *y–y* axis (EC5, equation (6.21)):

$$\lambda_{rel.y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c,0,g,k}}{E_{0,05,g}}} = \frac{24{,}25}{3{,}14} \cdot \sqrt{\frac{29{,}0}{13700}} = 0{,}36$$

Relative slenderness about the*z–z* axis, (EC5, equation (6.22)):

$$\lambda_{rel.z} = \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,0,g,k}}{E_{0,05,g}}} = \frac{40{,}42}{3{,}14} \cdot \sqrt{\frac{29{,}0}{13700}} = 0{,}59$$

Factor $\beta_c$ = 0,1 for glulam.

Factor $k_y$ (EC5, equation (6.27)):

$$k_y = 0{,}5 \cdot \left[1 + \beta_c \cdot \left(\lambda_{rel,y} - 0{,}3\right) + \lambda_{rel,y}^2\right] = 0{,}5 \cdot [1 + 0{,}1 \cdot (0{,}36 - 0{,}3) + 0{,}36^2] = 0{,}57$$

Instability factor about the $y$–$y$ axis (EC5, equation (6.25)):

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} = \frac{1}{0{,}57 + \sqrt{0{,}57^2 - 0{,}36^2}} = 0{,}99$$

Factor $k_z$ (EC5, equation (6.28)):

$$k_z = 0{,}5 \cdot \left[1 + \beta_c \cdot \left(\lambda_{rel,z} - 0{,}3\right) + \lambda_{rel,z}^2\right] = 0{,}5 \cdot [1 + 0{,}1 \cdot (0{,}59 - 0{,}3) + 0{,}59^2] = 0{,}69$$

Instability factor about the $z$–$z$ axis (EC5, equation (6.25)):

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} = \frac{1}{0{,}69 + \sqrt{0{,}69^2 - 0{,}59^2}} = 0{,}95$$

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} = \frac{17{,}45}{0{,}99 \cdot 18{,}56} = 0{,}95 < 1$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} = \frac{17{,}45}{0{,}95 \cdot 18{,}56} = 0{,}99 < 1$$

As all relationships are less than unity, the glulam member will meet the ULS requirements of EC5 (equation (6.23) and (6.24)).

COLUMN CW46 CENTRED AXIAL FORCE AND BENDING MOMENT.

Influence area: $3{,}0 \cdot 6{,}5\ m^2 = 19{,}5\ m^2$

<u>Roof loads:</u>

roof slab $6{,}48\ kN/m^2 \cdot 19{,}5\ m^2 = 126{,}36\ kN$

weight of the beam

$0{,}165\ m \cdot 0{,}54\ m \cdot 6{,}5\ m \cdot 4{,}56\ kN/m^3 = 2{,}64\ kN$

snow $0{,}768\ kN/m^2 \cdot 19{,}5\ m^2 = 14{,}98\ kN$

<u>Typical floor loads:</u>

Slab self-weight $3{,}28\ kN/m^2 \cdot 19{,}5\ m^2 = 63{,}96\ kN$

vertical closures self-weight $5{,}59\ kN/m \cdot 6{,}5\ m = 36{,}34\ kN$

<u>Variable loads (inside partitions included):</u>

$3{,}2\ kN/m^2 \cdot 19{,}5\ m^2 = 62{,}4\ kN$

Weight of the beam

$0{,}165\ m \cdot 0{,}54\ m \cdot 6{,}5\ m \cdot 4{,}56\ kN/m^3 = 2{,}64\ kN$

For the calculation of columns only, a reduction factor can be applied to variable loads [EC1-1 – 6.3.1.2(10) – National Annex]

$$\alpha_A = \frac{5}{7} \cdot \psi_0 + \frac{A_0}{A} \leq 1{,}0$$

where: $\psi_0 = 0{,}7$

$A_0 = 10\ m^2$

A is the influence area of the column considered

For column CW47 $\alpha_A \cong 1{,}0$.

Loads on every storey are:

- roof + roof floor
  - permanent loads 124,0 kN
  - variable loads 14,98 kN
- 6° floor – ground floor
  - permanent loads 102,94 kN
  - variable loads 62,4 kN

For the ULS combination of actions, a single multiplicative factor will be referred to, as a simplification: $\gamma_F^*$ is obtained as weighted mean of the coefficients $\gamma_G = 1{,}35$ and $\gamma_Q = 1{,}5$, respectively concerning permanent actions and variable actions.

$$\gamma_f^* = \frac{\gamma_G G_k + \gamma_Q Q_k}{G_k + Q_k} = \frac{1{,}35 \cdot 102{,}94 + 1{,}5 \cdot 62{,}4}{102{,}94 + 62{,}4} = 1{,}4$$

| Column | $F_k$ (kN) | Column self-weight (kN) | N (kN) $N = \sum F_{kj}$ | $N_{Ed} = \gamma_f \cdot N$ (kN) |
|---|---|---|---|---|
| 6° floor | 138,98 | | 138,98 | 194,57 |
| 5° floor | 165,34 | 2,39 | 306,71 | 429,39 |
| 4° floor | 165,34 | 2,39 | 474,44 | 664,22 |
| 3° floor | 165,34 | 2,39 | 642,17 | 899,04 |
| 2° floor | 165,34 | 2,39 | 809,9 | 1133,86 |
| 1° floor | 165,34 | 2,39 | 977,63 | 1368,68 |
| Ground floor | 165,34 | 2,39 | 1145,36 | 1603,50 |

*Tab.19. Axial load calculation*

GLULAM COLUMN GEOMETRIC PROPERTIES

Effective length of the member acting as a beam with a constant moment along its length (Table 4.2 (EC5, Table 6.1))

$l_{ef} = L = 3{,}5$ m

Width of the column $b = 300$ mm

Depth of the column $h = 500$mm

Cross-sectional area of the column

$$A = b \cdot h = 150 \cdot 10^3\, mm^2$$

Second moment of area of the column about the $y$–$y$ axes

$$I_y = \frac{b \cdot h^3}{12} = 31{,}25 \cdot 10^8\, mm^4$$

Section modulus about the $y$–$y$ axes

$$W_y = \frac{2 \cdot I_y}{h} = 12{,}5 \cdot 10^6\, mm^3$$

Radius of gyration about the $y$–$y$ axis

$$i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{31{,}25 \cdot 10^8}{150 \cdot 10^3}} = 144{,}34\, mm$$

Slenderness ratio about the $y$–$y$ axis

$$\lambda_y = \frac{L_{e.y}}{i_y} = \frac{3500}{144{,}34} = 24{,}25$$

Second moment of area of the column about the $z$–$z$ axis

$$I_z = \frac{h \cdot b^3}{12} = 11{,}25 \cdot 10^8\, mm^4$$

Radius of gyration of the column about the $z$–$z$ axis

$$i_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{11{,}5 \cdot 10^8}{150 \cdot 10^3}} = 86{,}6\, mm$$

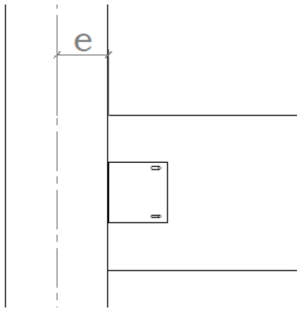
Slenderness ratio about the $z$–$z$ axis

$$\lambda_z = \frac{L_{e.z}}{i_z} = \frac{3500}{86{,}6} = 40{,}42$$

STRENGTH OF COLUMN

The critical design load case at the ULS will be due to the combination of permanent and unfavourable medium-term duration variable action.

MOMENT CONDITION




Design compressive action for the critical load combination from the upper stories on the ground floor column:

$$N_{Ed} = 1603{,}5\text{kN}$$

Moment induced by ground floor beam, coming from one side of the column:

Eccentricity $e_z = 250$ mm

Design moment about the $y$–$y$ axis

$$M_{Ed} = N_{Ed} \cdot e_z = 167{,}73 \cdot 0{,}25 = 41{,}93\ kNm$$

Design bending stress about the $y$–$y$ axis

$$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = \frac{41{,}93 \cdot 10^6}{12{,}5 \cdot 10^6} = 3{,}35\ \frac{N}{mm^2}$$

Design bending strength about the $y$–$y$ axis, $f_{m.y.d}$:

$$f_{m,y,d} = \frac{k_{mod,med} \cdot k_{sys} \cdot k_h \cdot f_{m,y,g,k}}{\gamma_M} = \frac{1{,}0 \cdot 0{,}8 \cdot 1{,}0 \cdot 32{,}0}{1{,}25} = 20{,}48\ \frac{N}{mm^2}$$

Redistribution factor for a rectangular section (4.5.1 (EC5, *6.1.6*)) $k_m = 0{,}7$

Buckling resistance condition – lateral torsional buckling under major axis bending

(4.5.1.2 (EC5, *6.3.3*))

Lateral stability factor (Table 4.3 (EC5, equation (6.34))) $k_{crit} = 1$

Critical bending stress (equation (4.7c); EC5, equation (6.31)):

$$\sigma_{m,crit} = \frac{\pi \cdot b^2 \cdot \left(E_{0,05,g} \cdot G_{0,05,g} \cdot \left(1 - 0{,}63 \cdot \frac{b}{h}\right)\right)^{0,5}}{h \cdot l_{ef}}$$

$$= \frac{3{,}14 \cdot 300^2 \cdot \left(13700 \cdot 780 \cdot \left(1 - 0{,}63 \cdot \frac{300}{500}\right)\right)^{0,5}}{500 \cdot 3500} = 416{,}33\frac{N}{mm^2}$$

Relative slenderness for bending (equation (4.10); EC5, equation (6.30)):

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,y,g,k}}{\sigma_{m,crit}}} = \sqrt{\frac{32{,}0}{416{,}33}} = 0{,}28$$

AXIAL COMPRESSION CONDITION

Design compression stress:

$$\sigma_{c,0,d} = \frac{N_{Ed}}{A} = \frac{1603{,}5 \cdot 10^3}{150 \cdot 10^3} = 10{,}69 \ \frac{N}{mm^2}$$

Design compression strength:

$$f_{c,0,d} = \frac{k_{mod,med} \cdot k_{sys} \cdot f_{c,0,g,k}}{\gamma_M} = \frac{1{,}0 \cdot 0{,}8 \cdot 29{,}0}{1{,}25} = 18{,}56 \ \frac{N}{mm^2}$$

Buckling resistance condition (5.4.1 (EC5, *6.3.2*)):

Relative slenderness about the $y$–$y$ axis (EC5, equation (6.21)):

$$\lambda_{rel.y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c,0,g,k}}{E_{0,05,g}}} = \frac{24{,}25}{3{,}14} \cdot \sqrt{\frac{29{,}0}{13700}} = 0{,}36$$

Relative slenderness about the$z$–$z$ axis, (EC5, equation (6.22)):

$$\lambda_{rel.z} = \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,0,g,k}}{E_{0,05,g}}} = \frac{40{,}42}{3{,}14} \cdot \sqrt{\frac{29{,}0}{13700}} = 0{,}59$$

Factor $\beta_c$ = 0,1 for glulam.

Factor $k_y$ (EC5, equation (6.27)):

$$k_y = 0{,}5 \cdot \left[1 + \beta_c \cdot \left(\lambda_{rel,y} - 0{,}3\right) + \lambda_{rel,y}^2\right] = 0{,}5 \cdot [1 + 0{,}1 \cdot (0{,}36 - 0{,}3) + 0{,}36^2] = 0{,}57$$

Instability factor about the $y$–$y$ axis (EC5, equation (6.25)):

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} = \frac{1}{0{,}57 + \sqrt{0{,}57^2 - 0{,}36^2}} = 0{,}99$$

Factor $k_z$ (EC5, equation (6.28)):

$$k_z = 0{,}5 \cdot \left[1 + \beta_c \cdot \left(\lambda_{rel,z} - 0{,}3\right) + \lambda_{rel,z}^2\right] = 0{,}5 \cdot [1 + 0{,}1 \cdot (0{,}59 - 0{,}3) + 0{,}59^2] = 0{,}69$$

Instability factor about the $z$–$z$ axis (EC5, equation (6.25)):

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} = \frac{1}{0{,}69 + \sqrt{0{,}69^2 - 0{,}59^2}} = 0{,}95$$

COMBINED STRESS CONDITIONS

Compression stress condition about the $y$–$y$ axis (equation (5.35) (EC5, 6.3.2(3)))

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} = \frac{10{,}69}{0{,}99 \cdot 18{,}56} + \frac{3{,}35}{20{,}48} = 0{,}75 < 1$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} = \frac{10{,}69}{0{,}95 \cdot 18{,}56} + 0{,}7 \cdot \frac{3{,}35}{20{,}48} = 0{,}71 < 1$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + \left(\frac{\sigma_{m,y,d}}{k_{crit} \cdot f_{m,y,d}}\right)^2 = \frac{10,69}{0,95 \cdot 18,56} + \left(\frac{3,35}{1 \cdot 20,48}\right)^2 = 061 < 1$$

As all relationships are less than unity, the glulam member will meet the ULS requirements of EC5 (equation (6.23) and (6.24)).

## 4.2. REINFORCED CONCRETE COLUMNS

An inside column and an external column will be analysed in this section. The first one is mainly subjected to axial load whereas for the second one bending moments are significant. The "influence areas" method will be used in both cases in order to determine the axial load to be taken into account during the pre-dimensioning of reinforcement.

COLUMN CC16 CENTRED AXIAL FORCE

Influence area: $6,0 \cdot 6,5\ m^2 = 39,0\ m^2$

Modified influence area (redundancy coefficient = 1,4): $39,0 \cdot 1,4 = 54,6\ m^2$

Roof loads:

roof slab with green roof $8,79\ kN/m^2 \cdot 54,6\ m^2 = 479,93\ kN$

tree box with soil above each column $(2,3\ m)^2 \cdot 1,0\ m \cdot 2,4\ kN/m^3 = 12,7\ kN$

big tree in each box 50 kN

weight of the beam (redundancy coefficient = 1,2)

$$1,2 \cdot 0,6\ m \cdot 0,4\ m \cdot 6,5\ m \cdot 25\ kN/m^3 = 46,8\ kN$$

snow $0,768\ kN/m^2 \cdot 54,6\ m^2 = 41,93\ kN$

live load $2,0\ kN/m^2 \cdot 54,6\ m^2 = 109,2\ kN$

Typical floor loads:

Slab self-weight $6,3\ kN/m^2 \cdot 54,6\ m^2 = 343,98\ kN$

Live load $7,5\ kN/m^2 \cdot 54,6\ m^2 = 409,5\ kN$

Weight of the beam (redundancy coefficient = 1,2)

$$1,2 \cdot 0,6\ m \cdot 0,4\ m \cdot 6,5\ m \cdot 25\ kN/m^3 = 46,8\ kN$$

For the calculation of columns only, a reduction factor can be applied to variable loads [EC1-1 – 6.3.1.2(10) – National Annex]

$$\alpha_A = \frac{5}{7} \cdot \psi_0 + \frac{A_0}{A} \leq 1,0$$

where: $\psi_0 = 0,7$

$A_0 = 10\ m^2$

A is the influence area of the column considered

For column CC16 $\alpha_A \cong 0{,}68$.

Loads on every storey are:

- roof + ground floor
  - permanent loads 589,43 kN
  - variable loads 151,13 kN
- basement
  - permanent loads 390,78 kN
  - variable loads 278,46 kN (= 0,68 · 409,5 kN)

For the ULS combination of actions, a single multiplicative factor will be referred to, as a simplification: $\gamma_F$* is obtained as weighted mean of the coefficients $\gamma_G$ = 1,35 and $\gamma_Q$ = 1,5, respectively concerning permanent actions and variable actions.

$$\gamma_f^* = \frac{\gamma_G G_k + \gamma_Q Q_k}{G_k + Q_k} = \frac{1{,}35 \cdot 390{,}78 + 1{,}5 \cdot 278{,}46}{390{,}78 + 278{,}46} = 1{,}4$$

| Column | $F_k$ (kN) | N (kN) $N = \sum F_{kj}$ | $N_{Ed} = \gamma_f \cdot N$ (kN) | $A_{c0} = \frac{N_{Ed}}{f_{cd}}$ (mm$^2$) | b x h (mm) | $A_c$ (mm$^2$) |
|---|---|---|---|---|---|---|
| Ground floor | 740,56 | 740,56 | 1036,78 | 45733,7 | 300 x 400 | 120000 |
| Basement | 669,24 | 1409,8 | 1973,72 | 87063,1 | 300 x 400 | 120000 |

*Fig.20. Axial load calculation*

$F_k$ represents the characteristic value of the axial load for every storey, whereas N is the nominal value of the axial load to be considered in the pre-dimensioning of the column. The design value, $N_{Ed}$, can easily be obtained multiplying N for the coefficient previously determined $\gamma_f$*. Dividing the design axial load for the design compressive strength of the concrete, $f_{cd}$, the needed area of concrete only can be obtained. The dimensions (b x h) of the cross-section can then easily be determined and the column self-weight can be computed and added to the design axial load at the bottom of the column.

<u>Column self-weight:</u>

ground floor 0,3 m · 0,4 m · 7,0 m · 25 kN/m$^3$ = 21,0 kN

basement 0,3 m · 0,4 m · 3,5 m · 25 kN/m$^3$ = 10,5 kN

In Tab. 21. the design axial load is modified in order to take into account the self-weight of the column at each floor.

| Column | $F_k$ (kN) | N (kN) $N = \sum F_{kj}$ | $N_{Ed} = \gamma_f \cdot N$ (kN) | $\boldsymbol{A_{c0} = \frac{N_{Ed}}{f_{cd}}\ (mm^2)}$ | b x h (mm) | $A_c$ (mm$^2$) |
|---|---|---|---|---|---|---|
| Ground floor | 761,56 | 761,56 | 1066,18 | 47030,6 | 300 x 400 | 120000 |
| Basement | 679,74 | 1441,3 | 2017,82 | 89008,4 | 300 x 400 | 120000 |

*Tab.21. Column CC15 - self-weight influence*

It is then necessary to dimension the longitudinal reinforcement. According to EC2 the following limits apply:

- technological limit: at least one bar needs to be placed at each corner of a polygonal column, whose diameter needs to be not less than 12 mm [EC2 – 9.5.2(4) and 9.5.2(1) – National Annex]
- geometrical limit: $A_s \geq 0{,}003\ A_c$ [EC2 – 9.5.2(2) – National Annex]
- static limit: $A_s \geq 0{,}10\ N_{Ed}/f_{yd}$ [EC2 – 9.5.2(2)]

| Column | $A_c$ ($mm^2$) | $A_{s,min}$ ($mm^2$) $\rho_s$ = 0,3 % | $A_{s,min}$ ($mm^2$)= $0{,}1 \cdot N_{Ed}/f_{yd}$ | 4 Ø 12 ($mm^2$) | n° x Ø | $A_s$ ($mm^2$) |
|---|---|---|---|---|---|---|
| Ground floor | 120000 | 360 | 275,8 | 452,0 | 4 x 12 | 452,0 |
| Basement | 120000 | 360 | 517,7 | 452,0 | 4 x 14 | 616,0 |

*Tab.22. Column CC16- pre-dimensioning of longitudinal reinforcement*

Both Ultimate Limit States and Serviceability Limit States verifications can then be performed.

The translational equilibrium of the cross-section for SLS is

$$N = \sigma_c \cdot A_c + \sigma_s A_s$$

Under the hypothesis of plane sections (Eulero-Bernoulli), same strain in steel and surrounding concrete ($\varepsilon_c = \varepsilon_s$) and elastic materials, it is $\sigma_s = \alpha_e\ \sigma_c$, where the ratio between the modulus of elasticity $\alpha_e$ is assumed equal to 15 in order to take into account the time-dependent behaviour of concrete.

$$N = \sigma_c \cdot (A_c + \alpha_e \cdot A_s) = \sigma_c \cdot A_{ie}$$

Obviously it needs to be

$$\sigma_c = \frac{N}{A_{ie}} \leq \sigma_{c\ adm} = 0{,}6 \cdot f_{ck} = 24 \frac{N}{mm^2}$$

| Column | $A_c$ ($mm^2$) | $A_s$ ($mm^2$) | $A_{ie}$ ($mm^2$) | N (kN) | $\sigma_c$ ($N/mm^2$) | < $\sigma_{c\ adm}$ |
|---|---|---|---|---|---|---|
| Ground floor | 120000 | 452,0 | 126780 | 761,56 | 6,0 | OK! |
| Basement | 120000 | 616,0 | 129240 | 1441,3 | 11,15 | OK! |

*Tab.23. Column CC16 - SLS verification*

The translational equilibrium for ULS is

$$N_{Rd} = A_c \cdot f_{cd} + A_s \cdot f_{yd}$$

| Column | $A_c$ ($mm^2$) | $A_s$ ($mm^2$) | $N_{Ed}$ (kN) | $N_{Rd}$ (kN) | $N_{Rd}/\ N_{Ed}$ |
|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| Ground floor | 120000 | 452,0 | 1066,18 | 2897,13 | 2,72 |
| Basement | 120000 | 616,0 | 2017,82 | 2961,26 | 1,47 |

*Tab.24. Column CC16 - ULS verification*

In Eurocode 2 some prescriptions on transversal reinforcement are outlined.

The minimum diameter of transversal bars needs to be not less than ¼ of the longitudinal diameter and however not less than 6 mm.

The spacing of the transverse reinforcement along the column needs not to exceed the following limits:

- 20 times the longitudinal bar size ($20 \cdot 12 = 240$ mm; $20 \cdot 14 = 280$ mm)
- the smaller dimension of the column (at most, 300 mm)
- 400 mm

In those sections within a distance equal to the larger dimension of the column cross-section above and below beams and slabs the previous limits are reduced by a factor 0,6 ($0{,}6 \cdot 240 = 144$ mm).

Stirrups $\phi 8/200$ will be provided along all the columns, whereas at the bottom and the top of the columns for a distance equal to 500 mm stirrups $\phi 8/125$ will be provided.

COLUMN CC15 CENTRED AXIAL FORCE AND BENDING MOMENT.

Influence area: $3{,}0 \cdot 6{,}5\ m^2 = 19{,}5\ m^2$

Modified influence area (redundancy coefficient = 1,4): $19{,}5 \cdot 1{,}4 = 27{,}3\ m^2$

<u>Roof loads:</u>

roof slab with green roof $8{,}79\ kN/m^2 \cdot 27{,}3\ m^2 = 239{,}97\ kN$

tree box with soil above each column $(2{,}3\ m)^2 \cdot 1{,}0\ m \cdot 2{,}4\ kN/m^3 = 12{,}7\ kN$

big tree in each box 50 kN

weight of the beam (redundancy coefficient = 1,2)

$$1{,}2 \cdot 0{,}6\ m \cdot 0{,}4\ m \cdot 6{,}5\ m \cdot 25\ kN/m^3 = 46{,}8\ kN$$

snow $0{,}768\ kN/m^2 \cdot 27{,}3\ m^2 = 20{,}97\ kN$

live load $2{,}0\ kN/m^2 \cdot 27{,}3\ m^2 = 54{,}6\ kN$

<u>Typical floor loads:</u>

Slab self-weight $6{,}3\ kN/m^2 \cdot 27{,}3\ m^2 = 171{,}99\ kN$

Live load $7{,}5\ kN/m^2 \cdot 27{,}3\ m^2 = 204{,}75\ kN$

Weight of the beam (redundancy coefficient = 1,2)

$$1{,}2 \cdot 0{,}6\ m \cdot 0{,}4\ m \cdot 6{,}5\ m \cdot 25\ kN/m^3 = 46{,}8\ kN$$

For the calculation of columns only, a reduction factor can be applied to variable loads [EC1-1 – 6.3.1.2(10) – National Annex]

$$\alpha_A = \frac{5}{7} \cdot \psi_0 + \frac{A_0}{A} \leq 1{,}0$$

where: $\psi_0 = 0{,}7$

$A_0 = 10\ m^2$

A is the influence area of the column considered

For column CC15 $\alpha_A \cong 0{,}87$.

Loads on every storey are:

- roof + ground floor
  - permanent loads 349,47 kN
  - variable loads 75,57 kN
- basement
  - permanent loads 218,79 kN
  - variable loads 178,13 kN (= 0,87 · 204,75 kN)

For the ULS combination of actions, a single multiplicative factor will be referred to, as a simplification: $\gamma_F$* is obtained as weighted mean of the coefficients $\gamma_G = 1{,}35$ and $\gamma_Q = 1{,}5$, respectively concerning permanent actions and variable actions.

$$\gamma_f^* = \frac{\gamma_G G_k + \gamma_Q Q_k}{G_k + Q_k} = \frac{1{,}35 \cdot 390{,}78 + 1{,}5 \cdot 278{,}46}{390{,}78 + 278{,}46} = 1{,}4$$

The same procedure followed for column CC16 is adopted for both pre-dimensioning and verification of column CC15.

| Column | $F_k$ (kN) | N (kN) $N = \sum F_{kj}$ | $N_{Ed} = \gamma_f \cdot N$ (kN) | $A_{c0} = \frac{N_{Ed}}{f_{cd}}$ $(mm^2)$ | b x h (mm) | $A_c$ $(mm^2)$ |
|---|---|---|---|---|---|---|
| Ground floor | 425,04 | 425,04 | 595,06 | 26248,6 | 300 x 400 | 120000 |
| Basement | 396,92 | 821,96 | 1150,74 | 50760,7 | 300 x 400 | 120000 |

*Tab.25. Axial load calculation*

Column self-weight:

ground floor $0{,}3\ m \cdot 0{,}4\ m \cdot 7{,}0\ m \cdot 25\ kN/m^3 = 21{,}0\ kN$

basement $0{,}3\ m \cdot 0{,}4\ m \cdot 3{,}5\ m \cdot 25\ kN/m^3 = 10{,}5\ kN$

In Tab.26. the design axial load is modified in order to take into account the self-weight of the column at each floor.

| Column | $F_k$ (kN) | N (kN) $N = \sum F_{kj}$ | $N_{Ed} = \gamma_f \cdot N$ (kN) | $A_{c0} = \frac{N_{Ed}}{f_{cd}}$ $(mm^2)$ | b x h (mm) | $A_c$ $(mm^2)$ |
|---|---|---|---|---|---|---|
| Ground floor | 446,04 | 446,04 | 624,46 | 27545,5 | 300 x 400 | 120000 |
| Basement | 407,42 | 853,46 | 1194,84 | 52706,0 | 300 x 400 | 120000 |

*Tab.26. Column CC15 - self-weight influence*

| Column | $A_c$ $(mm^2)$ | $A_{s,min}$ $(mm^2)$ $\rho_s = 0{,}3$ % | $A_{s,min}$ $(mm^2) = 0{,}1 \cdot N_{Ed}/f_{yd}$ | 4 Ø 12 $(mm^2)$ | n° x Ø | $A_s$ $(mm^2)$ |
|---|---|---|---|---|---|---|
| Ground floor | 120000 | 360 | 160,8 | 452,0 | 4 x 12 | 452,0 |
| Basement | 120000 | 360 | 307,2 | 452,0 | 4 x 12 | 452,0 |

*Tab.27. Column CC15- pre-dimensioning of longitudinal reinforcement*

| Column | $A_c$ $(mm^2)$ | $A_s$ $(mm^2)$ | $A_{ie}$ $(mm^2)$ | N (kN) | $\sigma_c$ $(N/mm^2)$ | $< \sigma_{c\ adm}$ |
|---|---|---|---|---|---|---|
| Ground floor | 120000 | 452,0 | 126780 | 446,04 | 3,52 | OK! |
| Basement | 120000 | 452,0 | 126780 | 853,46 | 6,73 | OK! |

*Tab.28. Column CC15 - SLS verification*

| Column | $A_c$ $(mm^2)$ | $A_s$ $(mm^2)$ | $N_{Ed}$ (kN) | $N_{Rd}$ (kN) | $N_{Rd}/N_{Ed}$ |
|---|---|---|---|---|---|
| Ground floor | 120000 | 452,0 | 624,46 | 2897,13 | 4,64 |
| Basement | 120000 | 452,0 | 1194,84 | 2897,13 | 2,42 |

*Tab.29 Column CC15 - ULS verification*

The same transversal reinforcement determined for column CC16 applies.

In order to evaluate the axial loads and bending moments acting on the columns a partial scheme can be adopted.

The stiffness of columns and beams are:

- column CC15 and column CC16:

$$I_c = \frac{300 \cdot 400^3}{12} = 16 \cdot 10^8\, mm^4$$

- beams:

$$I_b = \frac{400 \cdot 800^3}{12} = 171 \cdot 10^8\, mm^4 \cong 10 \cdot I_c$$

Loads that act on the structure are:

| | |
|---|---|
| structural self-weight | $G_1$ = 32,7 kN/m |
| other permanent loads | $G_2$ = 8,5 kN/m |
| snow load (roof) | $Q_s$ = 4,61 kN/m |
| imposed variable load | $Q_1$ = 49,1 kN/m |
| imposed variable load on the roof | $Q_2$ = 12,0 kN/m |

The following combinations of actions are considered.

ULTIMATE LIMIT STATES

<u>Combination 1ULS: snow as leading action</u> – significant just for actions at the top of the last storey columns

$\gamma_G$ ($G_1$+$G_2$) = 1,35 ($G_1$+$G_2$) = 55,62 kN/m on every storey, on all the spans

$\gamma_{Qs}$ ($Q_s$ + $Q_2$) = 1,5 ($Q_s$ + $Q_2$) = 24,92 kN/m on roof floor

$\gamma_{Q1}$ $\psi_{01}$ $Q_1$ = 1,5 · 0,7 $Q_1$ = 51,56 kN/m on all other floors

<u>combination 2ULS: imposed variable load as leading action</u>

$\gamma_G$ ($G_1$+$G_2$) = 1,35 ($G_1$+$G_2$) = 55,62 kN/m on every storey, on all the spans

$\gamma_{Qs}$ $\psi_{0s}$ ($Q_s$ + $Q_2$) = 1,5 · 0,7 ($Q_s$ + $Q_2$) = 17,44 kN/m on roof floor

$\gamma_{Q1}$ $Q_1$ = 1,5 · $Q_1$ =73,65 kN/m on all other floors

SERVICEABILITY LIMIT STATES

<u>combination 1SLS: snow as leading action</u>

($G_1$+$G_2$) = 41,2 kN/m on every storey, on all the spans

($Q_s$ + $Q_2$) = 16,61 kN/m on roof floor

$\psi_{01}$ $Q_1$ = 0,7 $Q_1$ = 34,37 kN/m on all other floors

<u>combination 2SLS: imposed variable load as leading action</u>

$\gamma_G$ $(G_1+G_2)$ = 55,62 kN/m on every storey, on all the spans

$\psi_{0s}$ $(Q_s + Q_2)$ = 0,7 $Q_s$ = 11,63 kN/m on roof floor

$Q_1$ = 49,1 kN/m on all other floors

Variable loads are considered acting either on both the spans or just on left or right span. The structural analysis results, obtained through appropriate computer programs, in terms of axial load and bending moments at the top and at the bottom of each column are reported in the following tables.

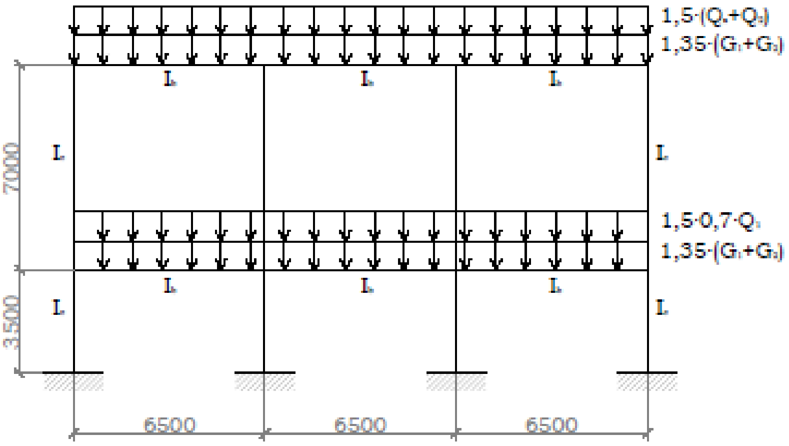



*Fig.63. Combination 1ULS*

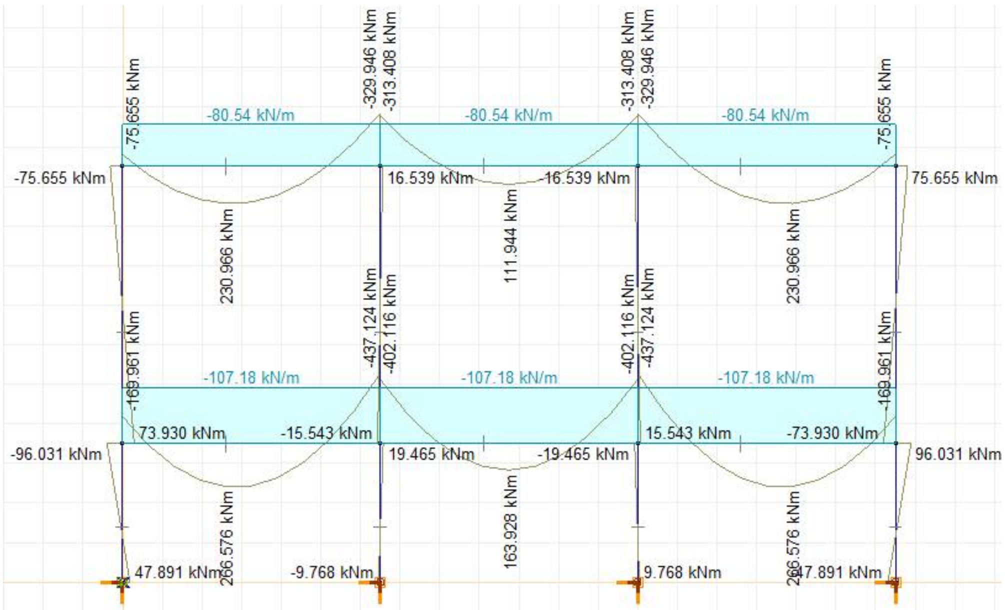



*Fig.64.Moment diagram Combination 1ULS* computed with Axis VM Software

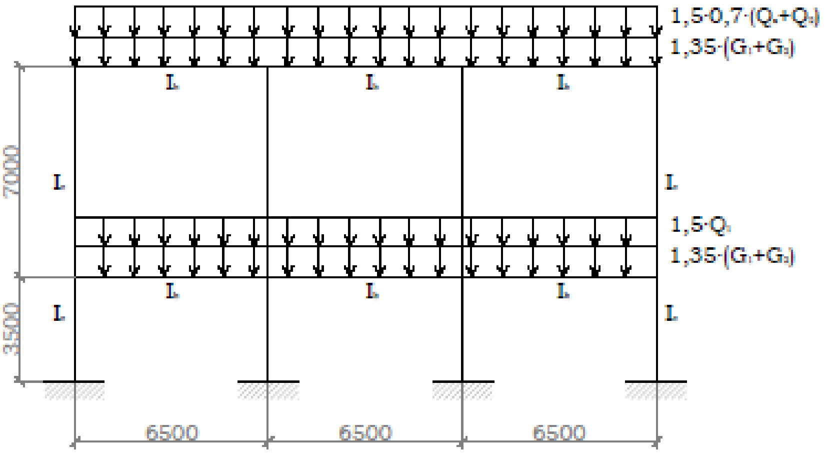



*Fig.65. Combination 2ULS*

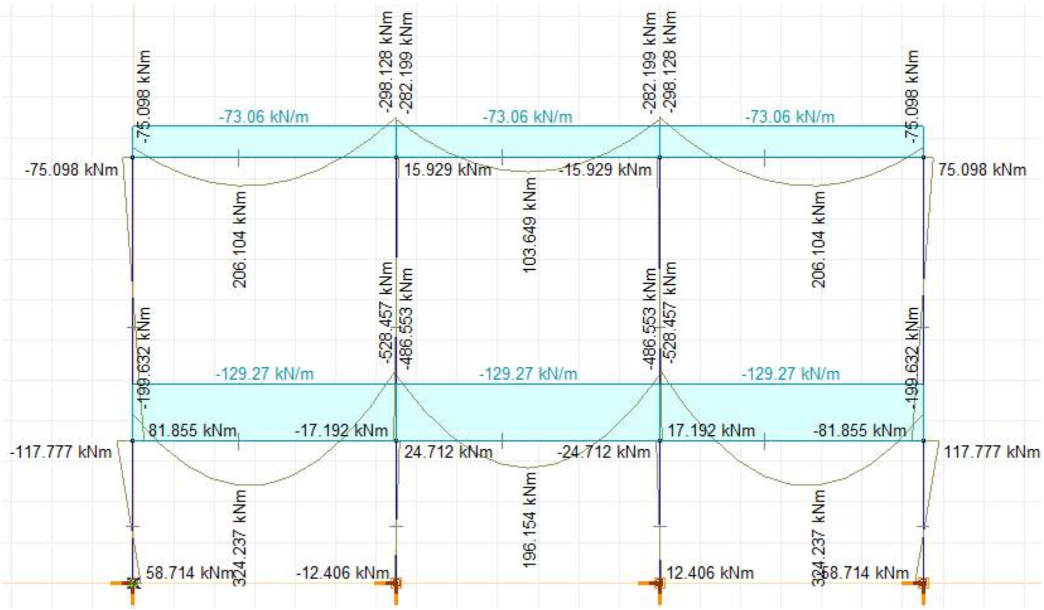



*Fig.66. Moment diagram Combination 2ULS* computed with Axis VM Software

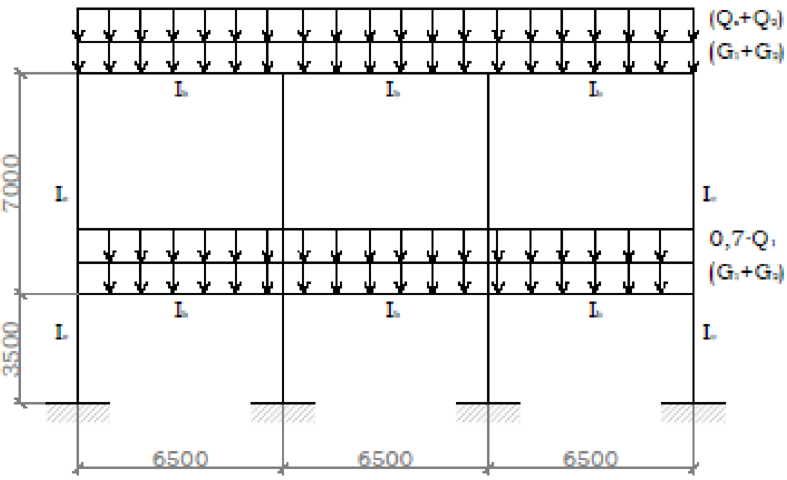



*Fig.67. Combination 1SLS*

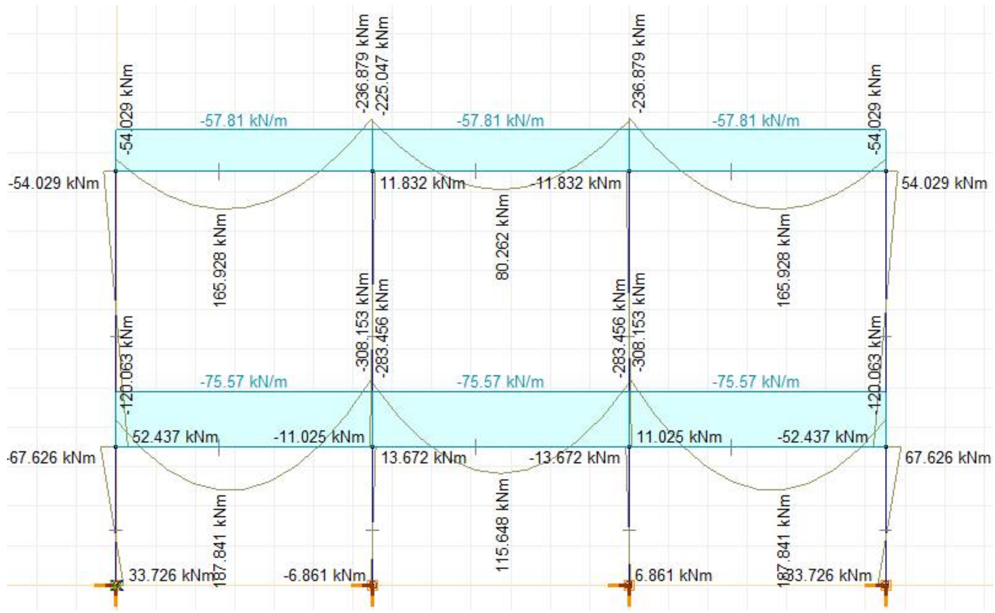



*Fig. 68. Moment diagram Combination 1SLS computed with Axis VM Software*

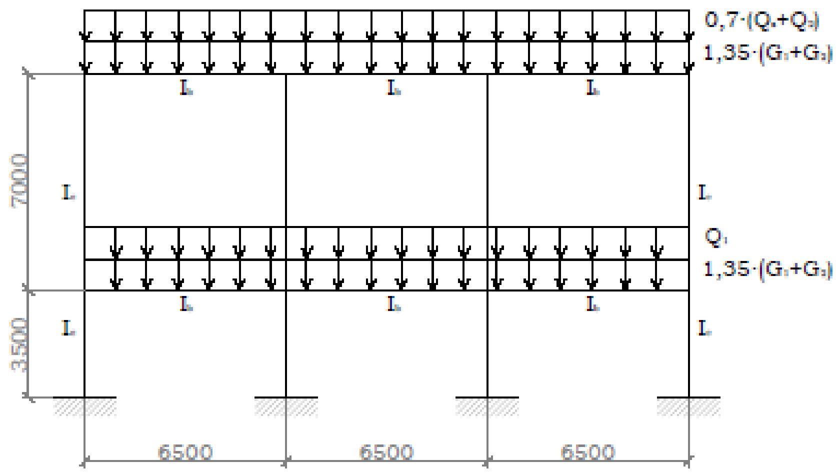



*Fig.69. Combination 2SLS*

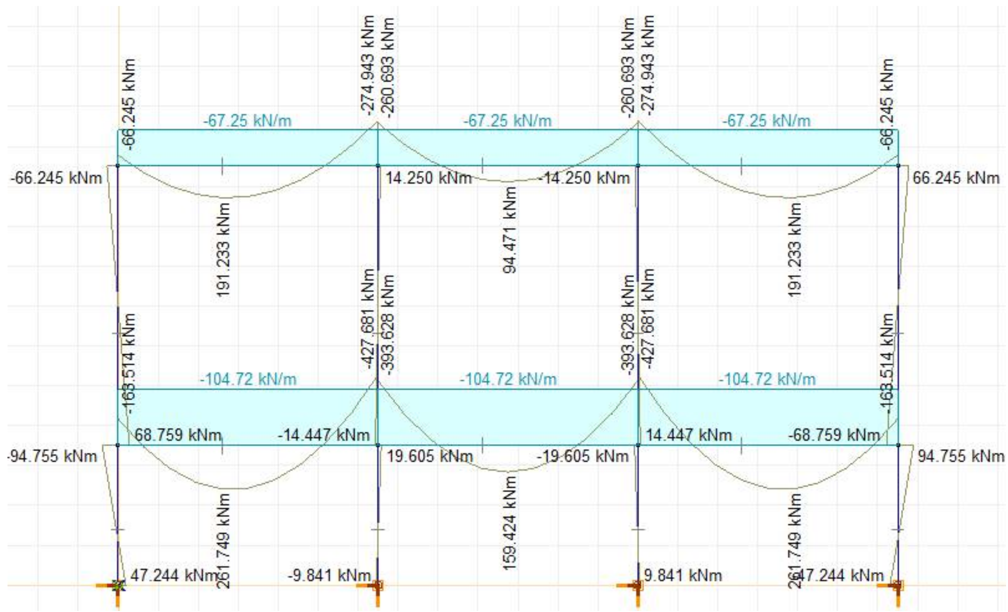



*Fig.70. Moment diagram Combination 2SLS computed with Axis VM Software*

ULS AND SLS VERIFICATIONS

Verifications of top section and bottom section for every column are performed.

*ULS verifications*

The following three hypothesis are made, assuming the rectangular stress-block for concrete [EC2 – 3.1.7(3)] and elastic-perfectly plastic behaviour for steel [EC2 – 3.2.7(2)b]:

breaking of concrete in compression due to ultimate strain $\varepsilon_{cu}$ = 0,0035 and both superior and inferior steel yielded.

The translational equilibrium gives the position of the neutral axis

$$0{,}8 \cdot b \cdot x \cdot f_{cd} + f_{yd} \cdot A_s' - f_{yd} \cdot A_s = N_{Ed}$$

whereas the rotational equilibrium about the barycentre of the concrete gross area gives the value of the corresponding resisting moment

$$0{,}8 \cdot b \cdot x \cdot f_{cd} \cdot (0{,}5 \cdot h - 0{,}4 \cdot x) + f_{yd} \cdot A_s' \cdot (0{,}5 \cdot h - d') + f_{yd} \cdot A_s \cdot (d - 0{,}5h) = M_{Rd}$$

breaking of concrete in compression due to ultimate strain $\varepsilon_{cu}$ = 0,0035 with superior elastic steel and inferior yielded steel.

The translational equilibrium is

$$0{,}8 \cdot b \cdot x \cdot f_{cd} + \sigma_s' \cdot A_s' - f_{yd} \cdot A_s = N_{Ed}$$

$$\sigma_s' = E_s \cdot \varepsilon_{cu} \cdot \frac{x - d'}{x}$$

and the rotational equilibrium is

$$0{,}8 \cdot b \cdot x \cdot f_{cd} \cdot (0{,}5 \cdot h - 0{,}4 \cdot x) + \sigma_s' \cdot A_s' \cdot (0{,}5 \cdot h - d') + f_{yd}.A_s \cdot (d - 0{,}5 \cdot h) = M_{Rd}$$

breaking of concrete in compression due to ultimate strain $\varepsilon_{cu}$ = 0,0035 with superior yielded steel and inferior elastic steel.

Translational equilibrium:

$$0{,}8 \cdot b \cdot x \cdot f_{cd} + f_{yd} \cdot A_s' - \sigma_s \cdot A_s = N_{Ed}$$

$$\sigma_s = E_s \cdot \varepsilon_{cu} \cdot \frac{d - x}{x}$$

Rotational equilibrium:

$$0{,}8 \cdot b \cdot x \cdot f_{cd} \cdot (0{,}5h - 0{,}4x) + f_{yd} \cdot A_s' \cdot (0{,}5h - d') + \sigma_s \cdot A_s \cdot (d - 0{,}5h) = M_{Rd}$$

The inferior reinforcement is yielded if

$$x \leq \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{yd}} \cdot d = \frac{3{,}5}{3{,}5 + 1{,}96} \cdot 747 = 478{,}8\, mm$$

whereas the superior reinforcement is yielded if

$$x \geq \frac{\varepsilon_{cu}}{\varepsilon_{cu} - \varepsilon_{yd}} \cdot d' = \frac{3{,}5}{3{,}5 - 1{,}96} \cdot 53 = 120{,}5\, mm$$

If the neutral axis is outside the section (uncracked section) the resultant of compression stresses in the concrete and its position can be evaluated using the stress-block with an equivalent height related to the neutral axis depth

$$h^* = \frac{x - \lambda \cdot h}{x - k \cdot h} \cdot h = \frac{x - 0{,}8 \cdot h}{x - 0{,}75 \cdot h} \cdot h$$

where λ = 0,8 and k = 0,75 for concrete strength class $f_{ck} \leq 50$ MPa.

The resistance verification can then be performed

$$M_{Rd} \geq M_{Ed}$$

$$M_{Rd} \geq N_{Ed} \cdot e_0$$

where $e_0$ = max (20 mm; h/30) and h is the height of the cross-section of the column [EC2 – 6.1(4)]. In this case it is $e_0$ = 20 mm.

*SLS verifications*

If the axial load is inside the inertia ellipsoid, the classical De Saint Venant formulas with homogenized geometrical characteristics of the cross-section are adopted in order to evaluate the stresses in the materials.

If the axial load is outside the inertia ellipsoid (cracked section), the position of the neutral axis can be easily found writing the rotational equilibrium about the point of application of the eccentric axial force (e = M/N)

$$x^3 + 3 \cdot \left(e - \frac{h}{2}\right) \cdot x^2 + \frac{6\alpha_c}{b} \cdot \left(A_s \cdot (e - 0{,}5h + d')\right) \cdot x - \frac{6\alpha_c}{b} \cdot \left(A_s \cdot d \cdot (e - 0{,}5h + d)\right) +$$

$$+ A_s \cdot d' \cdot (e - 0{,}5h + d') = 0$$

where $\alpha_e = 15$ represents the ratio between the modulus of elasticity of the two materials, taking into account the long term behaviour of concrete (creep and shrinkage).

Stresses in the materials in the end can be calculated as follows

$$\sigma_c = \frac{N_{Ed}}{\frac{bx^2}{2} + \alpha_e \cdot A'_s \cdot (x - d') - \alpha_e \cdot A_s \cdot (d - x)} \cdot x$$

$$\sigma_s = \alpha_e \cdot \sigma_c \cdot \frac{d - x}{x} \qquad \textit{positive if traction}$$

$$\sigma'_s = \alpha_e \cdot \sigma_c \cdot \frac{x - d'}{x} \qquad \textit{positive if compression}$$

A summary of the previous verifications is reported in the following tables.

*Column CC16: maximum axial load and moment – combination 2ULS and 2SLS*

| Column | $N_{Ed}$ | $M_{Ed}$ | b x h ($mm^2$) |
|---|---|---|---|
| Ground floor | 437,13 | 14,25 | 400 x 300 |
| | 469,53 | 14,45 | 400 x 300 |
| Basement | 1150,21 | 19,61 | 400 x 300 |
| | 1166,41 | 9,84 | 400 x 300 |

*Tab.30. Serviceability Limit States*

| Column | e = M/N (mm) | < h/6? | $\sigma_c$ ($N/mm^2$) | < 24 $N/mm^2$? | $A_s$ = $A'_s$ ($mm^2$) | $\sigma_s$ ($N/mm^2$) | $\sigma'_s$ ($N/mm^2$) |
|---|---|---|---|---|---|---|---|
| Ground floor | 32,6 | OK! | 5,5 | OK! | 226 | 27,9 | 71,4 |
| | 30,8 | OK! | 5,8 | OK! | 226 | 30,2 | 76,0 |
| Basement | 17,0 | OK! | 12,5 | OK! | 308 | 78,4 | 165,92 |
| | 8,4 | OK! | 12,0 | OK! | 308 | 81,1 | 160,56 |

*Tab. 31. Serviceability Limit States*

| Column | $N_{Ed}$ (kN) | $M_{Ed}$ kNm) | $N_{Ed}\cdot e_0$ (kNm) | x (mm) | $\varepsilon'_s$ ($\varepsilon_{yd}$ = 0,196%) | $\varepsilon_s$ ($\varepsilon_{yd}$ = 0,196%) | $M_{Rd}$ (kNm) | $\gamma$ |
|---|---|---|---|---|---|---|---|---|
| Ground floor | 474,89 | 15,93 | 9,5 | 65,5 | 0,0083 | -0,0007 | 76,5 | 4,8 |
| | 507,29 | 17,19 | 10,1 | 69,9 | 0,0069 | -0,0008 | 79,6 | 4,6 |
| Basement | 1347,55 | 24,71 | 27,0 | 185,8 | 0,0027 | -0,0057 | 126,1 | 4,7 |
| | 1363,75 | 12,41 | 27,3 | 188,0 | 0,0027 | -0,0059 | 126,1 | 4,7 |

*Tab.32. Ultimate Limit States*

*Column CC15: maximum axial load and moment – combination 2ULS and 2SLS*

| Column | $N_{Ed}$ | $M_{Ed}$ | b x h ($mm^2$) |
|---|---|---|---|
| Ground floor | 218,56 | 66,25 | 400 x 300 |
| | 250,96 | 68,76 | 400 x 300 |
| Basement | 591,3 | 94,76 | 400 x 300 |
| | 607,5 | 47,24 | 400 x 300 |

*Tab.33. Serviceability Limit States*

| Column | e = M/N (mm) | < h/6? | $\sigma_c$ ($N/mm^2$) | < 24 $N/mm^2$? | $A_s$ = $A'_s$ ($mm^2$) | $\sigma_s$ ($N/mm^2$) | $\sigma'_s$ ($N/mm^2$) |
|---|---|---|---|---|---|---|---|
| Ground floor | 303,1 | NO! | 12,0 | OK! | 226 | 248,6 | 94,76 |
| | 274,0 | NO! | 10,5 | OK! | 226 | 155,4 | 95,03 |
| Basement | 160,3 | NO! | 23,3 | OK! | 226 | 312,9 | 216,52 |
| | 77,8 | NO! | 12,0 | OK! | 226 | 7,4 | 142,69 |

*Tab.34. Serviceability Limit States*

| Column | $N_{Ed}$ (kN) | $M_{Ed}$ kNm) | $N_{Ed}\cdot e_0$ (kNm) | x (mm) | $\varepsilon'_s$ ($\varepsilon_{yd}$ = 0,196%) | $\varepsilon_s$ ($\varepsilon_{yd}$ = 0,196%) | $M_{Rd}$ (kNm) | $\gamma$ |
|---|---|---|---|---|---|---|---|---|
| Ground floor | 237,45 | 75,1 | 4,8 | 105,2 | 0,0083 | -0,0007 | 76,5 | 1,02 |
| | 269,85 | 81,9 | 5,4 | 125,9 | 0,0069 | -0,0008 | 79,6 | 1,03 |
| Basement | 689,98 | 117,8 | 13,8 | 131,8 | 0,0027 | -0,0057 | 126,1 | 1,08 |
| | 706,18 | 58,7 | 14,1 | 240,1 | 0,0027 | -0,0059 | 126,1 | 2,15 |

*Tab.35. Ultimate Limit States*

REINFORCEMENT ARRANGEMENT

In Fig.71. the reinforcement arrangement is displayed.

The anchorage lengths have been assumed in accordance to the same procedure used for beams.The following values then apply:

$l_{bd}$ = 450 mm for $\phi$12 bars

$l_{bd}$ = 500 mm for $\phi$14 bars

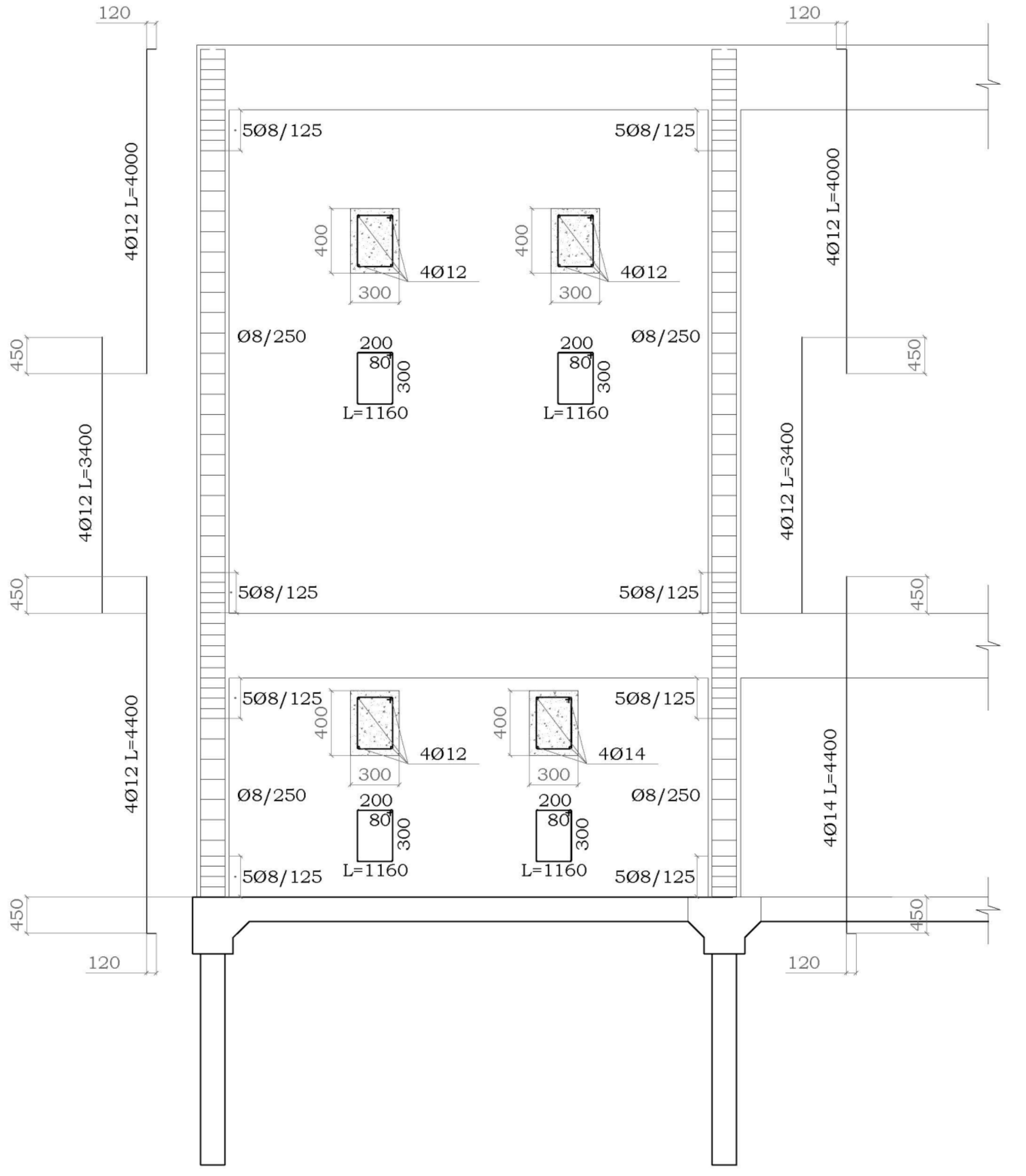



*Fig.71. Reinforcement arrangement*

## 5. FOUNDATION

After a research was made and all the available information was studied, it was determined that soil in Red Hook is formed by clays (Fig.72.).

For this type of soil that is very weak and unstable the most appropriate solution for foundation is a small diameter bored cast-in-place piles.

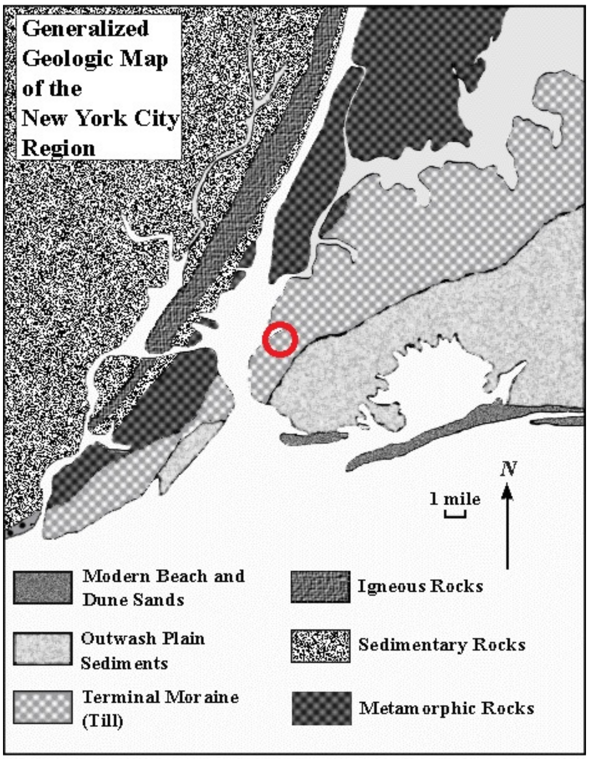



*Fig.72. Types of soil*

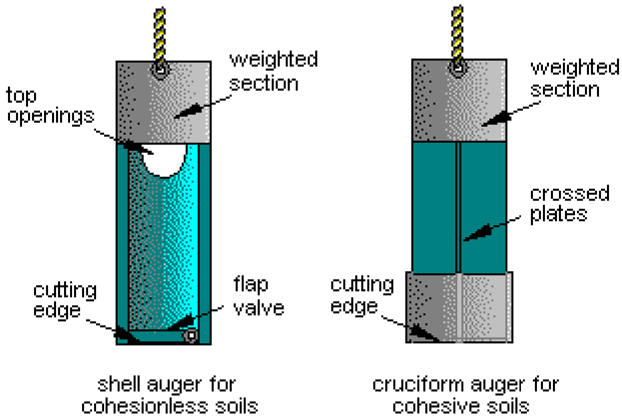



*Fig.73. A tripod rig*

With non-displacement piles soil is removed and the resulting hole filled with concrete. Clays are especially suitable for this type of pile formation as in clays the bore hole walls only require support close to the ground surface.

This type of piles usually is constructed by using a tripod rig (Fig.). The equipment consists of a tripod, a winch and a cable operating a variety of tools. In cohesive soils, the borehole is advanced by repeatedly dropping a cruciform-section tool with a cylindrical cutting edge into the soil and then winching it to the surface with its burden of soil. Once at the surface the clay which adheres to the cruciform blades is paired away.

For dimensioning of the piles a picture of existing building in Red Hook was used (Fig.74.), it can be seen that depth of the piles is approximately 3,0 m. in a project 3,0 m long 300 x 300 mm square piles are going to be used.

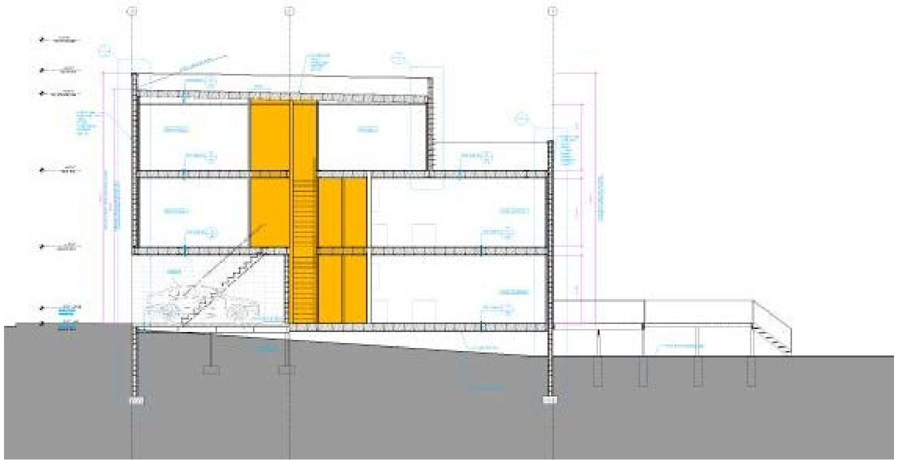


*Fig.74. Existing building in Red Hook*