A Study of Bidirectional Lightened Slabs with the Finite Element Method

Civil Engineering Final Thesis

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Preface

This thesis was realized as part of the Civil Engineering program at the HZ University of Applied Sciences in the Netherlands. It was developed in cooperation with and under the supervision of the company Aig Associati e Partner based in Bolzano, Italy.

This thesis reflects my passion for the field of structural engineering and the search for new challenges. Indeed, this project provided me with both an insight into the work of a structural engineer within an architectural and engineering environment and the challenge to immerse myself in new topics.

Not only have I never had the opportunity to model and verify large structures such as the one presented in this paper, but I have never worked extensively with finite element method approaches.

I would never have been able to tackle such a complex subject without the help and supervision of engineer Leonardo Mattei. I am very grateful to have been his intern and sidekick during the short period of my internship, and I am happy to say that Engineer Mattei ignited my passion for the world of structural engineering.

I would like to thank the entire team at Aig Associates and Partners for accepting me as an intern, for providing me with useful information and for helping me grow not only as a student but also as a young professional.

Special thanks go to my family and friends who have always supported me during this difficult but stimulating phase of my education.

Costanza Mariotti, Bolzano, 5th of June 2023

Abstract

The following paper refers to the final thesis for the curriculum at HZ Univertiy of Applied Sicence and was carried out at the request of the Italian firm Aig Associati and Partner.

The structural team of the architecture and engineering firm requested a study on the most effective and least time-consuming expedient to be applied in modelling a reinforced concrete flat slab using the finite element method.

Although the most important requirement in this subject is time reduction, the application of the solution has been considered. As a matter of fact, with the fundamental idea that the final approach should demonstrate ease of application for every engineer and technician in the company and beyond, convenience of modelling and recreation along with the added skills required became important criteria of this project.

In addition, as will be explored further in this report, the lack of the ability to include specific bodies, describing the hollow plastic casing identified as a lightening body, posed a challenge. Thus, expedients that would recreate the physical element and its behavioural pattern in real life were carefully considered. Therefore, it was essential to apply a model that would accurately recreate the real-life element, naturally within the established limits.

Three concepts were identified and, with the help of a hierarchical analytical process of the multiple-choice criteria analysis type, the final solution was chosen. After the decision-making phase, the winning approach was implemented in the structural method of analysis of the building of this development, performed with the finite element program "SOFiSTiK."

The conclusion shows that the choice of implementing a system of riveted elements in the structure brought many advantages in terms of cost and safety requirements. In fact, by reducing the overall self-weight of the building, a reduction in construction material was identified for both the upper part of the structure and the underground construction elements.

A great benefit was also delivered with regard to the action of a seismic event on the structure. In addition, the final design considered all requirements for the following research, meeting the minimum requirements imposed by the Eurocodes and the Italian annexes.

Table of Contents

1. Introduction

1.1. Background

The following graduation research paper is done in collaboration with the company Aig Associati and Partner, based in Bolzano, Trentino Alto-Adige, Italy. The agency has been active in the civil engineering sector since 1991, offering multidisciplinary experience. In fact, it covers various fields of work including architecture, engineering, geotechnics and other services.

The company processes customer requirements with creative solutions and translates them into optimised solutions with a high level of sustainability through integrated planning. The synergetic approach with all stakeholders allows the engineering firm to find innovative solutions, which result in quality constructions. Moreover, the multi-disciplinary approach allows projects to be tackled by considering issues related to the dimensioning of structures and interaction with plant technologies, guaranteeing the client reduced costs during both the design and construction phases.

One of the main specializations of the company is the design and construction of urban structures, such as hospitals enlargements, shopping malls, residential apartment blocks and other urban structures required in the region of Trentino Alto-Adige.

The intent to help people and provide them with an effective and useful environment led to the discussion of the case study for this research paper. Aig Associates and Partners is working on a nursing home (see Figure 1) for elderly people with cognitive disorders.

The agency requires a structural analysis and feasibility study of the modelling approach for the structure's lightened flat slabs.

Figure 1: Rendering of the nursing home

1.2. Problem Statement

The use of bidirectional slabs offers multiple advantages for residential buildings, such as the nursing home in this paper; nonetheless, its self-weight can result critical in structures of large dimensions.

The structure in question will have a layout depth of an estimated 100 metres and a width of 45 metres. Although the height of the building will only be two storeys, the mass of the building will be critical due to its sturdiness.

The Italian anti-seismic legislation – aligned with the most modern regulations at international level – prescribes technical standards according to which a building must withstand less severe earthquakes without serious damage and without collapsing in stronger earthquakes, safeguarding human lives. However, a greater mass of the structure will result in a lower resistance, hence safety, in case of seismic events.

Moreover, Aig Associati and Partners requires a feasibility study on the most optimum finite element method (FEM) approach to reduce the intrinsic weight of the structure as a whole, without compromising its stability and structural strength.

The prerequisite of using a finite element analysis imposes the restriction of being able to recreate detailed and accurate models of the structure. In fact, most FEM software do not have the prerogative of incorporating particular geometric shapes and solids. This results in the requirement to adopt alternative strategies when modelling the structure.

1.3. Goals and Objectives

The research will mainly focus on the details of the bidirectional slab and its relieving system. Furthermore, a finite element method will be applied for the feasibility study, dimensioning and requirements of the structure. The optimal FEM model to be implemented for the lightened flat slab will accordingly come under discussion. The technical and functional requirements, installation time and costs, and the overall cost of the structure will likewise be considered. While considering all the above-mentioned criteria, a great focus will be put on safety and risks of the model, in fact in construction providing a structure that complies with the codes automatically results in a theoretically safe structure for human lives. Nonethelss, the implementation of FEM modelling in this research paper is not considered in the codes, as these do not touch upon this topic. For this reason, great regard must be put when analysing different the FEM approaches.

1.3.1. Main Goal

The main goal of this graduation paper is to determine the most optimum FEM method to model the relived flat slab. The solution will be analysed on the basis of:

a. Designing time needed to create the relived model;
b. Reliability and accuracy of the model compared to the real life element;
c. Modelling and replication ease for future applications and
d. The safety and risk repercussions of the FEM approach.

1.4. Research Question

Based on the given data, the problem statement and objectives of the study case, the following research title has been outlined.

A study of bidirectional lightened slabs with the finite element method.

This can be translated into the following research question.

Why the preference for lightweight slabs in construction and how to model them in the most efficient and least time-consuming way using finite elements?

1.4.1. Research Sub-Questions

To provide an extensive answer to the main research question, the aid of sub-questions has been employed.

1. **What are the differences between a full bidirectional flat slab and a lightened one?**
2. **What are its effects on the structure?**
3. **What type of weight-relieving systems can be used? And why?**
4. **In which ways can the lightened slab be modelled in finite elements?**
5. **Which way is the most effective and least expensive approach?**
6. **To what extent does the approach reflect the real-life situation?**
7. **Which way is the most considerate towards lowering risks during the construction phase as well as once the structure is constructed?**
8. **What is the most consistent and fair method to compare alternatives?**

1.5. Research Methodology

1.5.1. Literature Review and Data Gathering

The first part of the graduation thesis will consist of various literature reviews to obtain in-depth information and previous applications and outcomes of a voided bidirectional flat slab. Moreover, data will be gathered from the host company, Aig Associati and Partners, to better understand the technical and functional requirements of a nursing home. This will be fulfilled by taking part to the meetings with the clients and stakeholders as well as by consulting the engineers at the firm.

1.5.2. Concept Design Phase: Alternatives and Multiple Criteria Analysis

Based on the literature study and information gathered during the graduation internship, a study of the potential solutions will be carried out. In order to ease the workload and reduce the time of this phase, only one section of the structure will be analysed. The three different methods will be applied and through a multi-criteria analysis, the most favourable solution will be determined.

1.5.3. Final Design Phase: Finite Element Method

As mentioned in paragraph 1.3, a FEM study will be utilised to carry out the structural analysis and find the optimum solution.

FEM is a numerical technique used to analyse and solve engineering constructs, not only in the field of civil engineering. This technique will be applied in this study case due to its extensive advantages.

Modelling. FEM facilitates the modelling of complex and irregular geometric shapes. Since the designer is able to model both internal and external properties, how critical factors can affect the entire structure and why failures occur can be determined.

Adaptability. The FEM can be adapted to meet certain accuracy specifications in order to reduce the design process phase. Creating multiple iterations of initial prototypes is usually an expensive and time-consuming process. Instead of spending weeks on physical prototyping, the designer can model several concepts and materials in a few hours using software.

Accuracy. While manual modelling of a complex physical deformation may be impractical, a computer using FEM can solve the problem with a high margin of accuracy.

Time-dependent simulation. FEM is very useful for time-dependent simulations, such as accident simulations, where deformations in one area depend on deformations in another area.

Boundaries. With FEM, designers can use boundary conditions to define which conditions the model must respond to. Boundary conditions can include point forces, distributed forces, thermal effects, seismic effects and position constraints.

Visualisation. Thanks to the detailed visualisations produced by the FEM, engineers can easily identify any vulnerabilities in the design and use the new data to optimise and update the design (Ikponmwosa, 2017).

In this research paper the following FEM software for structural calculations will be adopted SOFiSTiK 2023.

SOFiSTiK is Europe's leading manufacturer of construction software for analysis, design and detailing. More than 2,000 customers worldwide use the company's solutions to construct complex and large infrastructures and buildings or to realise extraordinary projects.

As one of the pioneers in the use of technology in the construction industry, SOFiSTiK has significantly improved the capabilities available for design and construction, such as creating new methods for structural engineering and reinforcement detailing.

Inputs will be calculated and estimated via the literature reviews, the engineers' consult as well as the Italian construction codes issued in the Official Journal of the Italian ministry of infrastructures and transport. These are:

- the update of the Technical Standards for Construction dated 17 January 2018 (NTC 2018)
- the memorandum dated 21 January 2019, no. 7 (Memorandum 2019 to the NTC 2018)

To note that the Italian codes are based on the Eurocodes, hence these will be adopted too when analysing and interpreting data.

The Eurocodes of most relevance in this paper will be:

- Eurocode – Basis of structural design	EN 1990
- Eurocode 1 – Actions on structures	EN 1991
- Eurocode 2 – Design of concrete structures	EN 1992
- Eurocode 8 – Design of structures for earthquake resistance	EN 1998

2. Theoretical Framework

2.1. Current Situation

Griesfeld Nursing Home Foundation is the client for the building described and analysed in this graduation project.

The foundation firmly adheres to the indications contained in the European Charter of the Rights of Elderly People in Residential Institutions. Openness to the outside world, in particular the optimisation of integration in the local context and the opening of facilities to the wider community for public use, aim to require viable institutions.

With the aim of providing care for elderly people with cognitive difficulties, the Griesdeld Nursing Home Foundation is in the process of expanding its facilities. This will be done through the construction of a new facility in the location of 'Magrè on the Wine Road'.

2.2. Project Location

Magrè sulla Strada del Vino (Magrè on the Wine Route in English) is an Italian municipality of 1272 inhabitants in the autonomous province of Bolzano in Trentino-Alto Adige. Its location is indicated by the red dot in Figure 2.

Figure 2: Magrè on the Wine Route

Surrounded by vineyards and orchards, at the base of the mountains and close to the embankment of the Adige River, it is the perfect and peaceful location (see Figure 3) for a future nursing home.

Figure 3: Peaceful Surroundings and Environment

2.3. Boundary Conditions and Limitations

2.3.1. Boundary Conditions

Due to the particular geodynamic conditions, the Italian territory is frequently subject to earthquakes, which gives it the record in Europe for these events; out of 1.300 destructive earthquakes that occurred in the 2^{nd} millennium in the central Mediterranean, as many as 500 affected Italy.

Analysis of focal movements indicates that they are mainly distributed along the areas affected by Alpine and Apennine tectonics, where they are caused respectively by movements along faults. It is therefore necessary to consider the possibility of potential seismic occurrence at the location of construction.

In Trentino-Alto Adige, the climate is Alpine, with cold, snowy winters and short, warm summers. In winter, snowfalls are abundant and frequent, while in spring and autumn it often rains. Furthermore, depending on the orography, the exposure of the location and the altitude, strong winds result prevailing. From a construction point of view, it is of utmost importance to take into account the effect of snow and wind on the structure.

2.3.2. Limitations

As mentioned in the previous paragraph, the location of the site requires that account be taken of the loads applied to the structure by seismic, wind and snow actions. Furthermore, great consideration will be given to the client's wishes and importance will be attached to the architectural design, making the structural design reliant to some extent on the architect's proposals. All this within the budget identified at the preliminary stage.

All the above-mentioned limitations reflect into the layout of the structure's floor plans, which determine the model to be used when preforming the structural analysis. In fact, according to the client's wish and the architect's adaptation of those desires, the layout, dimensioning and positioning of the spaces within the structure differ. This reflects into the positioning of the bearing walls and columns as well as into the loads applied to the floor.

2.3.2.1. Description of Finite Element Software: SOFiSTiK

Lastly, other limitations apply to the FEM software in the sense that, the firm previously determined the most feasible and optimum FEM software to be used in every project, being 'SOFiSTiK 2023'.

SOFiSTiK develops software for every domain of structural engineering. The solutions provide the basis for innovative work practices and the complete digitisation of the construction industry. Structural analysis and building design demand powerful and versatile software. Some of the essential features of SOFiSTiK's FEM packages include seismic design for 3D models, reliable slab design with punching checks and design of columns and foundations. These are all necessary functions in any structural engineering company.

SOFiSTiK FEM packages are tailor-made solutions based on proven finite element technology for structural analysis using the finite element method. Based on an open system architecture, the powerful FE programmes offer seamless workflows for all requirements of modern civil engineering. The application area of finite element solutions ranges from simple 2D slab design to 3D building and bridge design; from open, native and integrated BIM to large and small BIM.

All FEM packages include code verification modules suitable for Eurocodes and various annexes, as well as comprehensive and interactive post-processing and high-performance solutions. The input options are versatile; from parametric input via the AutoCAD SOFiPLUS add-on to BIM planning with Autodesk Revit. These options provide civil engineers with the right tools to effectively realise construction projects of any scale.

This software regardless its powerful calculation and analysis engine, has the limitation of not being able to support tetrahedral shapes.[1]

In geometry, a tetrahedron is simply a pyramid-shaped solid with a triangular base. This element is essential for modelling the lightening systems chosen by the company. Therefore, the slab cannot be modelled in a realistic manner as seen in Figure 4.

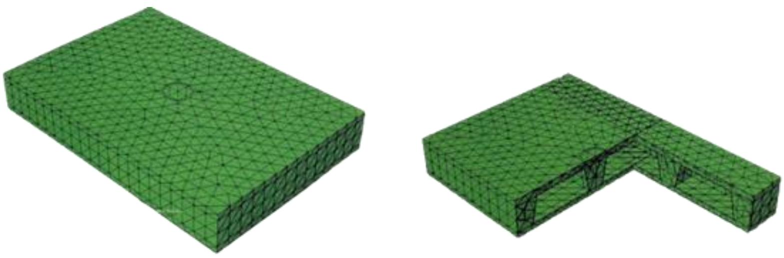

Figure 4: FEM approach with tetrahedral shapes

2.4. Schedule of Requirements

2.4.1. Functional Requirements

As a result of several meetings with the client, different prerequisites were identified. To facilitate the understanding of such requirements, categories were developed as follows.

2.4.1.1. For the Client

The Griesfeld Nursing Home Foundation offers its guests 'a new home, with additional services', a familiar place where it is possible to establish new social relationships and participate in community events.

[1] This is a feature that lacks in many other FEM software. As a matter of fact, the ones that do support tetrahedral shapes – such as ABAQUS – result highly expensive and not in the budget of many small to medium scale engineering companies.

In the words of the client: 'various initiatives give meaning and "colour" to the daily life of the guests', taking into account their needs and habits and respecting the uniqueness of each individual.

This is reflected in the design of single and double living units, with the aim of conveying a familiar and homely space. In addition, the community spaces requested by the client include gardens, rooms for both alternative and traditional therapies, and communal spaces for the client's comfort.

2.4.1.2. For the Staff

Nursing homes offer medical services to residents in need of comprehensive care. Most nursing homes have nursing assistants and specialist nurses available 24 hours a day. The main difference from nursing homes is that nursing homes provide 24-hour medical care and assistance in daily living. The main difference from assisted living facilities is that nursing homes provide 24-hour medical care and assistance with activities of daily living, whereas assisted living facilities encourage residents to remain as independent as possible and offer help when needed.

This manifests itself in the need to provide dedicated space for the different medical facilities, in compliance with health and safety regulations. In addition, staff will be provided with specially designated spaces to facilitate their work experience; spaces such as changing rooms with showers and rest areas located within the facility.

Finally, the facilities entrusted to the Griesfeld Nursing Home Foundation are subject to a single administration to facilitate the management of staff and situations that may arise. This feature will be taken into account when planning the layout of the various areas of the building.

2.4.1.3. For the Visitors

The client's vision leads to considering the guests' relatives and mutual bonding as a naturally essential part of their ongoing life. For this reason, the facility is designed to welcome both guests and visitors, thus offering warm and welcoming meeting areas, which will be ensured by the inclusion of a bar and restaurant in the facility and the creation of other ideal spaces for these occasions.

2.4.2. Technical Requirements

The technical requirements for this building are given by compliance with the Italian building codes and their annexes. These refer to the safety and stability of the overall structure, considering external loads such as seismic, wind and snow loads.

2.4.2.1. Ductility

As per the new Italian technical standards for construction (NTC 2018) a ductility check in specific construction details is necessary to ensure an adequate level of ductility. This applies to relevant seismic zones.

A structure has the necessary capacity to withstand a seismic event if it has the ability to dissipate the seismic action. This can only occur if the structure is dissipative, thus when it enters the post-elastic field. This occurs with the formation of permanent plastic deformations, called plastic hinges, in critical areas (Furcolo, 2019).

Critical zones are defined as the ends of the main and secondary columns, so verification is necessary at the abutments of the columns to ensure the required ductility under seismic conditions.

In the case where there is no beam at the abutments of the columns, but rather a slab, the verification can be carried out using the plate model.

2.4.2.2. Flat Slab

The plate model has the advantage of eliminating emerging beams. In fact, a slab floor transfers loads in two directions. In contrast to a traditional one-directional floor, which, as the name suggests, transfers the load in one direction. This feature allows larger spans to be used without increasing the thickness of the floor itself (Pisapia, Solai a piastra: perchè usarli e come modellarli – con Marco Calvi [Brain Hunter Podcast], 2019).

In addition, flat slab floors have excellent rigidity due to their prevented lateral deformation. This allows these elements to reduce their deformations due to loads and design a reduced thickness.

Lack of Beams

In a one-way slab system, the stresses are transferred from the slab to the beam and finally to the column.

By using a slab floor, the load is transferred directly to the column. The absence of beams reduces both the time and cost of installation; on the other hand, this makes punching shear checks essential.

Punching Shear

The punching shear in flat slabs, as previously mentioned, is a dimensioning parameter and decisive for characteristics such as spans and slab thickness.

The punching checks are performed in accordance with the technical standards for construction (NTC 2018).

In lightened slabs, it is important to verify that the load-relieving system is not positioned within the punching verification perimeter.

Global Behaviour in Presence of Seismic Actions

Following certain parameters and having the certainty that the structure is designed in a low seismicity zone, a plate-and-pillar structure can be considered a primary structure. This is only permitted, according to Italian regulations (Redazione DEI, 2019), if the structure exhibits non-dissipative or low-dissipative behaviour.

On the other hand, in areas of medium to high seismicity, the structure must be considered dissipative, and therefore a system of beams and columns is necessary (Redazione DEI, 2019).

Behaviour Factor

A slab-on-column system has low ductile reserves; this results in a behaviour factor (q) of 1.5 or less.

Choosing the behaviour parameter of 1.5 results in low intrinsic ductility relative to the materials in question.

In contrast, a slab system on walls or cores must offer a horizontal resistance of more than 65%, thus acting as a brace for torsional effects. This allows the behaviour factor to be raised, entering the ductile zone (q greater than 1.5).

Ductility Class

In the case of a flat slab design, as mentioned in the previous paragraph, the behaviour factor will be no greater than 1.5. This means that the ductility class is a low class; the ductility margin is due to the characteristics of the design materials.

It should be remembered that this concept can be applied to areas of low seismicity and/or to structures that have favourable geometric characteristics to guarantee calculation under non-dissipative conditions.

In the second case, namely of a flat slab on walls or cores, the ductility class will be medium to high. In fact, the load-bearing elements act as primary structural elements in situations of seismic activity. The plate will therefore be considered as a secondary structural element (Pisapia, Solai a piastra: perchè usarli e come modellarli – con Marco Calvi [Brain Hunter Podcast], 2019).

2.4.2.3. Lightened Flat Slab

Lightened slabs have cavities within the section of the plate itself, altering its main resistance property. For this reason, a study of both the form of the type of lightening and the most suitable arrangement must be carried out.

The choice of whether to lighten a plate is given by the mass of a standard full concrete plate. As this is a massive structure, its own weight, in certain cases, prevails over the service loads (Geoplast S.p.A. , 2019). By lightening the self-weight, the load in the foundation as well as the excavation volume are reduced.

Type of Relieving Formwork

Reticular Slabs

By laying the relieving blocks at the lower limit of the slab, a grid of orthogonal ribs is created that is able to maintain the bi-directional nature of slab slabs. This solution offers a reduction in the amount of concrete required, decreases the use of steel reinforcement and in some cases the blocks used to create the ribs are recoverable (if plastic or fibreglass) (Geoplast S.p.A. , 2019).

On the other hand, in order to guarantee the characteristics given by a standard flat slab, the design of a gridded slab has limitations in geometry. The geometric parameters ensure sufficient torsional stiffness; if they are not met, the structure offers reduced performance compared to a solid slab.

Furthermore, to guarantee the necessary stiffness of the building, each rib requires a reinforcement similar to the beams, which slows down the installation time.

Lightened Slabs with Hollow Articles

The lightened slab with hollow sections has low-density polystyrene lightening systems embedded in the concrete section, creating a network of ribs enclosed in two solid slabs. In contrast to the reticulated slab, this lightening system offers a section that is considered a slab in its own right, given the presence of the bottom slab (Geoplast S.p.A. , 2019).

The presence of a lower and upper slab allows the element to be reinforced using the same methodology as solid plates, while reducing the amount of steel.

Lightened Slabs with Hollow Articles in Plastic

This solution is a variation of the alternative described in the previous paragraph; the difference lies in the shape and material of the hollow bodies used. In fact, the lightened body consists of recycled polypropylene boxes.

Using this material and shape results in greater robustness than polystyrene caissons, which leads to optimised logistics and installation. In fact, polystyrene structures are bulky and take up a lot of space on the construction site (Geoplast S.p.A. , 2019).

Furthermore, in the event of a fire, polystyrene has been confirmed to cause slab explosions and release toxic gases. This is due to the lack of appropriate vents that can lead to explosive ruptures of concrete sections, compromising the stability of the building itself (Franchi, 2008).

The plastic blocks proposed by Geoplast override the limitations found in polystyrene bodies.

Modelling Lightened Flat Slabs using Finite Element

The use of lightened plates makes it possible to reduce the load in the foundation as well as the excavation volume, this is due to the reduction of its own weight. In fact, as previously mentioned, a lightened plate section presents cavities.

Chosen Types of Relieving Formwork

For the ‘Casa Haus inge’ project, the plates will be lightened with prismatic plastic hollow blocks, manufactured by Geoplast. The company offers recycled polypropylene boxes with plan dimensions of 52 x 52 cm and variable height. In addition, depending on requirements, Geoplast presents a 'single' and a 'double' version (by coupling two 'singles') (Geoplast S.p.A. , 2019).

Figure 5: Plastic Lightening 'Single' Type

Figure 6: Plastic Lightening 'Double' Type

Preliminary Description of FEM Modelling

The Finite Element Method (FEM) technique is adopted for the study of structural phenomena related to the stiffness and strength of bodies. This analysis also is applicable to lightened slabs with caissons.

Reinforced concrete slabs lightened with 'New Nautilus EVO'[2] elements can be modelled as solid slabs with reduced stiffness and self-weight. Three possible approaches are proposed in the Geoplast Calculation Manual (Geoplast S.p.A. , 2019):

a. Modelling the lightened plate as a slab with construction thickness, but including multiplying coefficients to reduce weights and inertias;
b. Modelling the lightened plate as a solid slab with reduced thickness, in order to obtain equivalent stiffness and self-weights;
c. Modelling the lightened plate as a slab with construction thickness, but including coefficients on the reinforced concrete, reducing Young's modulus and self-weight.

Furthermore, based on the 'BubbleDeck' lighteners and their modelling,[3] a fourth expedient for modelling lightened plates has been found (Sabah Mahdi & Shatha, 2021):

d. Modelling the lightened plate with a material package, namely a slab and a counter slab framing a third intermediate layer modelled with a material – concrete – with reduced elastic modulus, self-weight and shear strength. The material for the lightened slab used in the finite element modelling will therefore be a package of three materials, referred to in this report as 'sandwich' material.

The moment of inertia will be considered for each of the systems proposed above; therefore, a study of the moment of inertia will be outlined in paragraph 3.2.

[2] Type of Geoplast relieving body adopted for the 'Casa Haus inge' project.

[3] In parallel to a study on the modelling of spherical lightening elements, the approach was also discussed with Engineer Emanuele Agostini, an expert in the modelling of structural elements with the software 'SOFiSTiK'. Engineer Agostini, in fact, already applied modelling the slab as a package of materials in other project, with the difference of the formwork utilized.

3. Methodology

3.1. Research Strategy

As mentioned in section 1.5, the graduation thesis will consist of three phases:

1. Literature review and data collection — this phase will be set out in the introductory chapters and will serve to provide adequate information to begin the second phase.

2. Conceptual design phase — this involves the study of the three possible alternatives on the basis of the results studied. To facilitate the work, only a portion of the structure will be analysed.

3. Final design phase — in contrast to the conceptual design phase, in which only a section of the structure is analysed using three different methods, in this phase the detailing of the entire structure will be presented. In fact, the structural analysis will be performed with the selected finite element method.

3.2. Moment of Inertia of the Relieved Section

As mentioned in paragraph 2.4.2.3, a study of the moment of inertia will be laid out below.

3.2.1. Specifications of the Section

The section being analysed is given by the lightening model chosen for the plate. In fact, each model varies in height, modifying the thickness of the plate itself. It is therefore optimal to pre-dimension the plate thickness according to the standards and then choose a model to derive the section for the study of the inertia moments.

The predimensioning of the thickness (H) is based on the following structural types together with the required spans (L):

- Full plate on pillars $H = \frac{L}{25}$
- Lightened plate on pillars $H = \frac{L}{28}$
- Full plate on beams $H = \frac{L}{30}$
- Lightened plate on beams $H = \frac{L}{32}$

In the study case of the 'Casa Haus inge', the plates to be analysed are solid plates on columns, with variable spans.

The lightening models and their characteristics are provided by the Geoplast technical data sheets.[4]

[4] See Appendix 1 – Geoplast Technical Data Sheet.

Once the thickness of the plate has been determined the following cross-section can be extrapolate for the analysis of the lightened slab (refer to Figure 7).

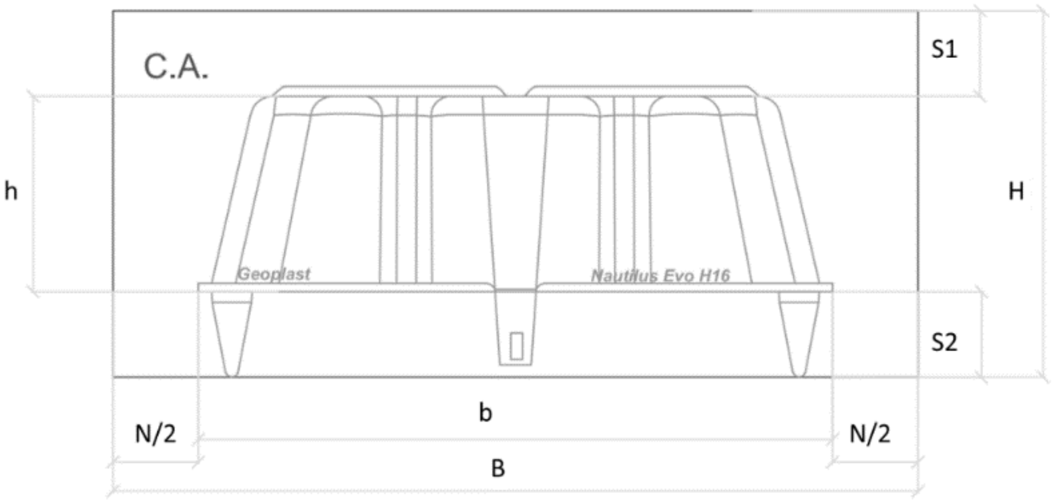

Figure 7: Lightened Slab Section

The base (B) of the section to be analysed is obtained by adding the size of the chosen beams (N) to the width of the lightening element (52 cm). Note that Geoplast offers six different rib widths (see Table 1)to be used on any chosen model.

Table 1: Beam Width for each Model

OPTIONS PROVIDED	BEAM WIDTH (N) [mm]
option 1	100
option 2	120
option 3	140
option 4	160
option 5	180
option 6	200

On the other hand, the overall height of the section (H) is taken from the pre-dimensioning; the lower (S2) and upper (S1) spacing are chosen by the designer symmetrically or by placing the larger spacing at the underside. The choice of placing the greater spacing at the base of the section is given by fire safety considerations; in fact, greater thicknesses result in better resistance in the event of fire (Geoplast S.p.A. , 2019).

In short, the designer is instructed to choose the following values to determine the section for the analysis of the moment of inertia:

1. Construction thickness of the floor
2. Lightening type
3. Upper and lower spacing
4. Rib width

The characteristics listed above lead to the determination of the floor cross-section, as in Figure 7.

3.2.2. Simplification of the Lightening Model

To study the moment of inertia of the lightened section, the moment of inertia of the lightening system must also be analysed. Due to the complexity of the profile of the New Nautilus EVO, a simplified profile shape is used.

The simplified shape that best fits the real shape (excluding the feet) is that of a trapezoid (shown in blue in Figure 8); however, for safety reasons the trapezoid will have a larger area than the real area. This will result in a smaller moment of inertia than the real so that there is a margin of error, in this case of 20%.

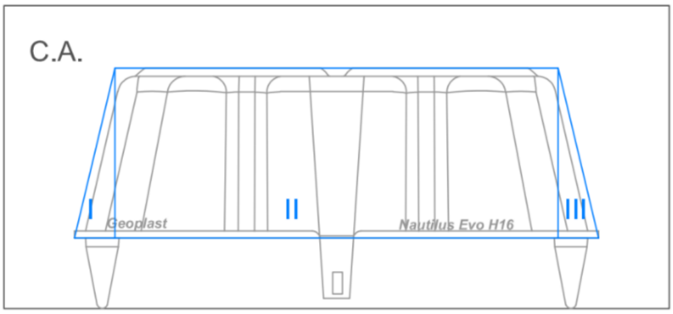

Figure 8: Simplification of the Lightening Model

3.2.3. Dimensions of the Simplified Model

It must be noted that the simplified profile is itself divided into three simple elements to facilitate calculations:

- → two right-angled triangles (I & III)
- → a rectangle (II)

The height (h) of the three elements listed above is determined from the height of the model while adding the size of the upper spacers (Geoplast S.p.A. , 2019). In fact, each formwork is equipped at the top with uniformly distributed 8 mm thick elements. These are used for the positioning of the upper reinforcement, which is laid directly onto them, guaranteeing the appropriate concrete cover (Figure 9).

Figure 9: Upper Spacers

To derive the horizontal dimensions of the elements, the volume specified in the technical tables (Geoplast S.p.A. , 2019) is used.

The base of the two triangles (I & III) is determined by taking the difference between the area of a rectangle with base and height given by the model[5] and the area derived from the volume (refer to equation (1) and Figure 10).

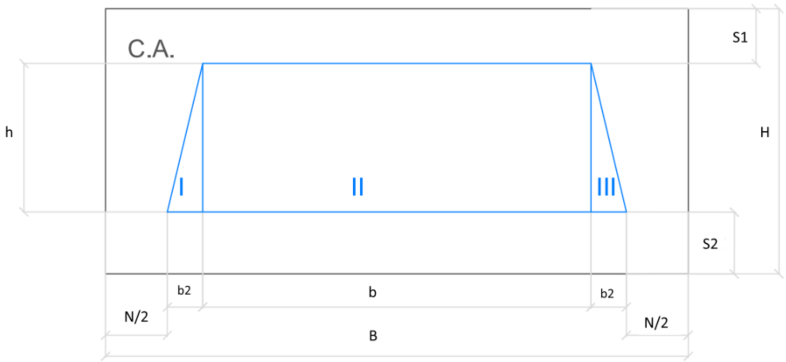

Figure 10: Dimensions of simplified model

$$b'1 = b'2 = \frac{\left(52\ cm * h_{model} - \frac{V}{52\ cm}\right)}{h} \tag{1}$$

To meet the safety requirements, the value found with formula (1) is reduced by 50 % while enlarging the size of the base of the rectangle, see equations (2) and (3).

$$b1 = b2 = b'1 * 0.5 \tag{2}$$

$$b = 52\ cm - (b1 + b2) \tag{3}$$

3.2.4. Moment of Inertia

The moment of inertia relates to the geometric properties of a body; in this case, the moment of inertia of a flat surface will be considered. In fact, the study of the inertia of the hollow slab section is interesting to conduct, since it relates directly to the resistance of the section subjected to bending (Edutecnica, 2018).

With the aid of an Excel spreadsheet,[6] the moment of inertia of the simplified hollow section can be derived. In Table 2 follow the results for the study of a section using New Nautilus Evo Single H16.

Table 2: Moment of Inertias

	Full Section	**Void**	**Relived Section**
Iy [mm^4]	1.80E+09	2.04E+08	1.60E+09

[5] Note that each model features a 52 x 52 cm floor plan.

[6] Refer to Appendix 2 – Lightened Slab Specifications Excel Sheet.

On the other hand, for the study of the middle section of the slab (see section in red in Figure 11) the upper and lower spacings have been excluded. As done previously, the results have been calculated with the aid of an Excel spreadsheet.[7]

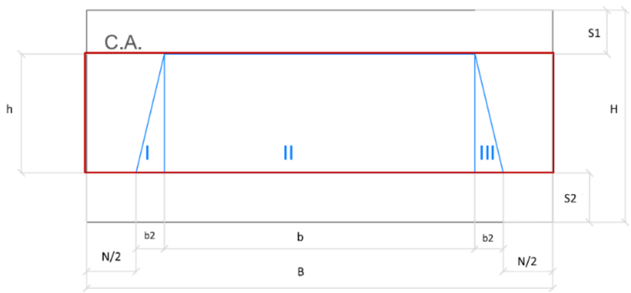

Figure 11: Middle Section for the Inertia Study

Herby the moments of inertia calculated.

Table 3: Moment of Inertias - middle section

	Full Section	Void	Relived Section
Iy [mm^4]	2.61E+08	1.95E+08	6.94E+07

3.3. Finite Element Method Modelling

As mentioned in the section 2.4.2.3, four strategies can be adopted for modelling lightweight plates in finite elements. The four strategies make use of different reduction coefficients, with the exception of self-weight, used in all four. See below the reductive coefficients used for each solution studied:

- → Solution a   reductive coefficients for weights and inertias
- → Solution b   reductive coefficients for stiffness and own weights
- → Solution c   reductive coefficients for Young's modulus and own weights
- → Solution d   reductive coefficients for Young's modulus, own weights and shear strength[8]

3.3.1. Reductive Coefficient for Inertia and Young's Modulus

The reduction coefficient in question is applied to a solid plate to obtain the equivalent bending stiffness of the plate lightened with 'New Nautilus EVO' formwork.

Since the flexural strength[9] is closely related to both the moment of inertia (I) and Young's modulus – also known as the modulus of elasticity – (E), the reductive coefficient (R_f) can be applied in both solutions a and solution c and solution d by only considering the inertia of the middle section (excluding the top and bottom slabs).

The coefficient is calculated as the ratio between the moment of inertia of the hollow section and the moment of inertia of the solid section.

[7] Refer to Appendix 2 – Lightened Slab Specifications Excel Sheet.

[8] Note that the reduction will be applied and studied only for the middle section between the two slabs.

[9] $flexural\ strenght = EI$

$$R_f = \frac{I_{y,void}}{I_{y,full}} \tag{4}$$

For solution a and solution c the following value has been calculated.

$$R_f = \frac{I_{y,void}}{I_{y,full}} = \frac{1.6 * 10^9}{1.8 * 10^9} = 0.89$$

Whereas for solution d the reductive coefficient results as follows.

$$R_f = \frac{I_{y,void}}{I_{y,full}} = \frac{6.94 * 10^7}{2.61 * 10^8} = 0.27$$

3.3.2. Reductive Coefficient for Stiffnesses

Correspondingly to the previous procedure, the torsional and shear stiffnesses must be reduced in order to correctly model the behaviour of the lightened section.

3.3.2.1. Reduction Coefficient for Torsion

This coefficient is used for the modelling strategy of a lightened plate as a solid slab with reduced thickness (H_f), consequently the thickness must be calculated as follows.

$$H_f = \sqrt[3]{12 * I_{y,void}} = \sqrt[3]{12 * 1.6 * 10^9} = 308\, mm \tag{5}$$

The ratio of the fictitious thickness calculated using formula (5) to the total thickness of the concrete section results in the reductive coefficient for torsion (6).

$$R_t = \frac{H_f}{H_{tot}} = \frac{308}{320} = 0.96 \tag{6}$$

3.3.2.2. Reduction Coefficient for Shear

The multiplying factor to be used for shear strength reduction is derived from a correlation between the area of the hollow section and the area of the equivalent solid section.

$$R_s = \frac{A_{void}}{A_{full}} = \frac{130295}{211200} = 0.62 \tag{7}$$

This reduction coefficient will be applied to solution d as well.

3.3.3. Reductive Coefficient for Own Weight

When referring to the reduction of the self-weight of the slab, it must be noted that the different approaches should reach the same reduced weight. This means that adjustments have to be applied when reducing the thickness of the slab for solution b.

In fact, as the self-weight of the slab is determined by multiplying the density of the material by its thickness, solution b will result in a lower self-weight compared to the other solutions. Therefore, adjustments must be implemented.

3.3.3.1. Whole Slab Section

With regard to the weight of the lightened slab, the reduction factor (R_w) can be derived by subtracting the volume of the chosen formwork per square metre from the corresponding weight of the full slab (W_{full}).

$$W_{full} = \gamma_c * H_{tot} \tag{8}$$

$$W_{void} = W_{full} - \left(\frac{1}{B^2} * V_{void}\right) * \gamma_c \tag{9}$$

$$R_w = \frac{W_{void}}{W_{full}} = \frac{6.16}{8} = 0.77 \tag{10}$$

In which:

$\gamma_c = 25 \frac{kg}{m^3}$

3.3.3.2. Middle Slab Section

On the other hand, since the fourth solution will study only the reductions in the middle section, the calculation for the reduction of the self-weight will be performed as follows.

$$V'_{full} = H_{tot} * 1m^2 \tag{11}$$

$$V'_{void} = V'_{full} - n * V'_{void} \tag{12}$$

$$R_w = \frac{V_{void}}{V_{full}} = \frac{0.087}{0.168} = 0.54 \tag{13}$$

In which:

V'_{full} is the volume of the middle section

n is the number of formworks in $1m^2$

$n = \frac{1m^2}{spacing\ between\ fromwork^2}$

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3.4. Assessment of Wind or Earthquake Pressure Predominance

In this chapter, an assessment of which of the stresses, from wind or from a potential earthquake, prevails will be presented, so as to maximise design time. In fact, based on the results of this report, it is possible to exclude either stress, depending on the situation.

The load outcomes will then be applied when performing the structural analysis of the whole structure of the development 'Casa Haus inge'.

3.4.1. Wind Pressure

According to the NTC 2018 (Redazione DEI, 2019), the wind direction on a structure should generally be assumed to be horizontal; this results in dynamic effects occurring.

3.4.1.1. Zone Definition

Each zone in Italy is characterised by a reference speed and altitude, as seen in Figure 12.

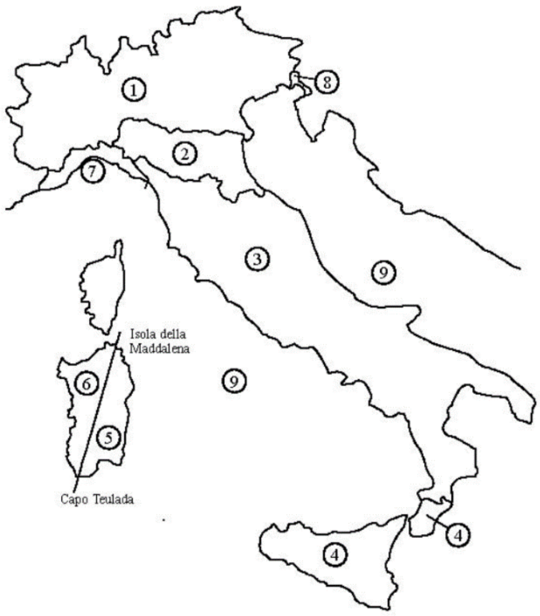

Figure 12: Division of the Italian territory according to wind

In this case, the development of 'Casa Haus inge' is designed to be constructed in the location of 'Magrè on the Wine Road', in the province of Bolzano, Trentino Alto Adige, at an altitude of 241 m above sea level. According to the Italian construction codes (see Table 4), this results in a structure in zone 1 of the Italian territory, with a reference wind speed of 25 m/sec and a reference altitude of 1000 m above sea level (Redazione DEI, 2019).

Table 4: Parameter values for 'Casa Haus inge'

zone	description	$v_{b,0}$ [m/s]	a_0 [m]	k_s
1	Valle d'Aosta, Piemonte, Lombardia, **Trentino Alto Adige**, Veneto, Friuli Venezia Giulia (with the exception of the province of Trieste)	25	1000	0.4

A reductive altitude coefficient (C_a) is applied to areas where the altitude above sea level of the construction site (a_s) is higher than the reference altitude provided by the NTC 2018 (Redazione DEI, 2019). In this case, the construction site altitude (241 m above sea level) is lower than the reference altitude (1000 m above sea level), resulting in an unchanged reference wind speed.

$$C_a = 1 \tag{14}$$

$$v_b = C_a * v_{b,0} \tag{15}$$

Where:

$$a_s \leq a_0$$

3.4.1.2. *Definition of the Kinetic Reference Pressure*

The reference kinetic pressure is calculated according to the following formula (16) (Redazione DEI, 2019).

$$q_r = \frac{1}{2}\rho v_r \tag{16}$$

In which:

$$\rho = density\ of\ air = 1.25\frac{kg}{m^3}$$

Note that in formula (16), the reference speed is taken as a function of the return time.

The return time is generally taken between 10 years and 500 years, based on the extensive data archive in Italy. For the project 'Casa Haus inge', a reference return time of 50 years was chosen.

The reference velocity as a function of the return period is derived according to the formula (17).

$$v_r = v_{b,0} * 0.75\sqrt{1 - 0.2 * \ln\left(-\ln\left(1 - \frac{1}{T_R}\right)\right)} \tag{17}$$

$$v_r = 25 * 0.75\sqrt{1 - 0.2 * \ln\left(-\ln\left(1 - \frac{1}{50}\right)\right)} = 25.018\frac{m}{s}$$

With the wind speed value just obtained, the reference kinetic pressure can be derived using formula (16).[10]

$$q_r = \frac{1}{2}\rho v_r = \frac{1}{2} * 1.25 * 25.018 = 391.200\frac{N}{m^2} = 39.12\frac{kg}{m^2}$$

3.4.1.3. *Exposure and Topographical Coefficients*

The coefficients of exposure (C_e) and topography (C_t) depend on the chosen height of wind application on the building and on the topography of the terrain and the exposure category of the site where the construction is designed. These values are extrapolated from the NTC 2018.

[10] To be noted that all calculations are based on previously prepared calculation spread sheets. In this case refer to Appendix 5 – Wind Load Excel Sheet.

A roughness class 'type A' (see Table 5 for further details) was assigned for the development under analysis because the building will be constructed in the vicinity of the municipality of Magrè, which is considered an urban area.

Table 5: Terrain Roughness Class

Roughness Class	description
A	Urban areas in which at least 15% of the surface area is covered by buildings whose average height exceeds 15 m

Remembering that 'Magrè on the Wine Road' is located at an altitude of 241 m above sea level at an average distance of 140 km from the coast, in zone 1 of the Italian territory, the exposure category of the site can be extrapolated, following the indications of the NTC 2018 (see Figure 13).

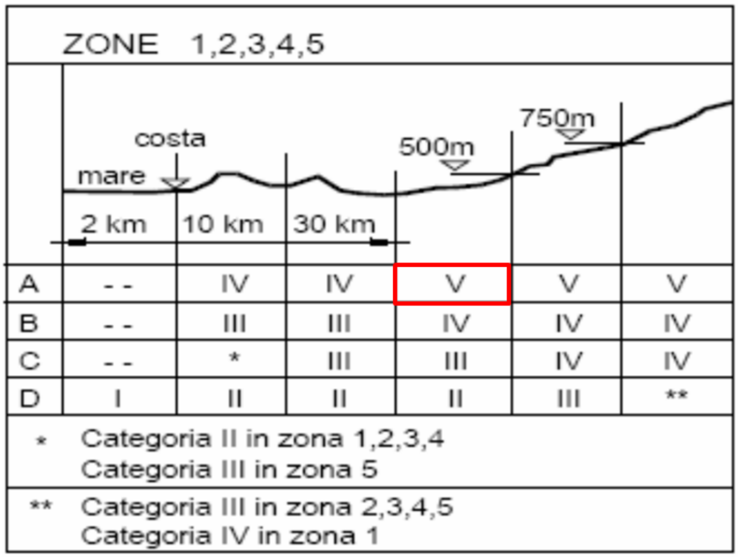

Figure 13: Definition of Exposure Categories

A 'type V' site exposure category can be used for the 'Haus inge' project. According to NTC 2018, the following values (see Table 6) are assigned to a 'V' exposure category (Redazione DEI, 2019).

Table 6: Parameter Values by Exposure Category

Site exposure category	K_r	z_0 [m]	z_{min} [m]	$c_e(z_{min})$
V	0.2	0.1	5	1.7

The exposure coefficient (C_e) is determined, as mentioned above, by the height (z) above ground of the application point considered and varies in accordance with whether it exceeds the minimum value(z_{min}) or not.

In the case of 'Casa Haus inge', the application point was chosen based on the overall height of the building (see Figure 14) with a tolerance margin of 10%.

$$z = 7.1 * 1.1 = 7.81\ m\ \therefore z < z_{min}$$

$$C_e = C_e(z_{min}) \tag{18}$$

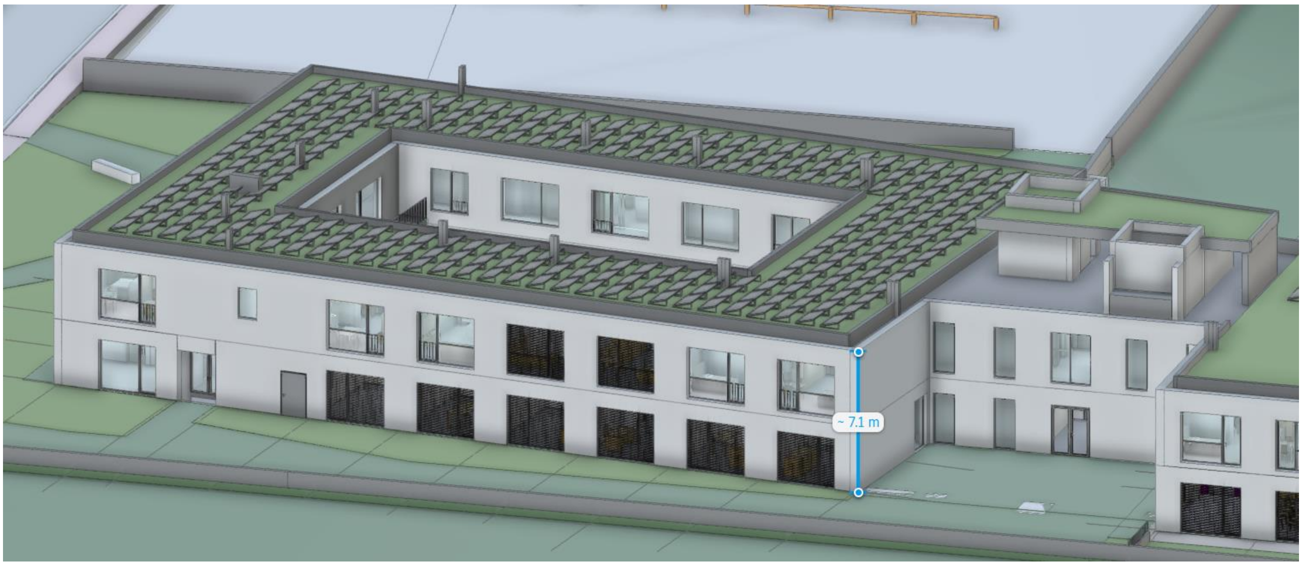

Figure 14: Height of the building

On the other hand, the topographical coefficient (C_t) can be assumed to be 1, owing to the fact that the construction location lies in a flat area.

3.4.1.4. Static Equivalent Action Calculation

The wind action is assessed as a static load and is determined by combining the reduction coefficients with the reference kinetic pressure (see formula (19)).

The reduction coefficients refer to the site exposure (C_e); the pressure on the building surfaces (C_p) and the dynamic coefficient (C_d). The pressure coefficient depends on the type and geometry of the building, therefore for the preliminary calculation it will be assumed to be 1; in the following paragraphs a further study of the building will be set out with the aim of determining the pressure coefficients on the external walls of the building. On the other hand, the dynamic coefficient considers the amplifying effects due to the dynamic response of the building (Redazione DEI, 2019) and in this case it can be assumed to be equal to 1. This is due to the type of the building which according to NTC 2018 is of 'recurring type with regular shape and not exceeding 80 m in height' (Redazione DEI, 2019).

$$p = q_r * C_e * C_p * C_d \tag{19}$$

$$p = 39.12 * 2.67 * 1 * 1 = 104.6 \frac{kg}{m^2}$$

3.4.1.5. Tangent Action of the Wind

The wind friction force is calculated by multiplying the static wind value (19) by a friction coefficient. This depends on the roughness of the building surface; in the case of ‘Casa Haus inge' the material used is reinforced concrete. According to NTC 2018, reinforced concrete is considered a rough surface material, resulting in a coefficient (C_f) of 0.02 (Redazione DEI, 2019).

This corresponds to a wind friction force (p_f) of 2.09 kg/m^2.

3.4.1.6. *Pressure Value on Vertical Walls*

For the evaluation of the external pressure on the vertical walls of the building, the latter will be considered a construction with a rectangular plan, as seen in Figure 15 below.

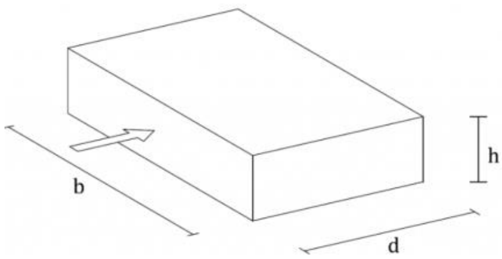

Figure 15: Dimensions of a Rectangular Plan Building

For the 'Casa Haus inge' case, the larger dimensions with an overestimation for safety reasons, will be taken into account.

The height is taken to be equal to the height established in the section 3.4.1.3, therefore equal to 7.81 m.

On the other hand, the base (b) and depth (d) of the building are established according to the plan of the upper floor, shown in Figure 16.

Figure 16: Dimensions in Plan of Upper Floor

Note that for safety reasons, a margin of error of 10% will be applied to the dimensions for dimensions less than 100 m, and 5% for those greater than or equal to 100 m.

$$d = 36.8 * 1.1 = 40.48\ m$$

$$b = 100 * 1.05 = 105\ m$$

As stated in NTC 2018 (see Table 7 extrapolated from the codes), the external pressure coefficients are set according to the value of the ratio between the height and depth of the building.

Table 7: Global Coefficients for Vertical Walls

Upwind surface		Downwind surface		Side surface	
h/d≤1	$C_{pe} = 0.7 + 0.1 * \frac{\mathrm{h}}{\mathrm{d}}$	**h/d≤0,5**	$C_{pe} = -0.5 - -0.8 * \frac{\mathrm{h}}{\mathrm{d}}$	**h/d≤1**	$C_{pe} = -0.3 - -0.2 * \frac{\mathrm{h}}{\mathrm{d}}$
h/d>1	0.80	**h/d>0,5**	-0.90	**1<h/d≤5**	$C_{pe} = -0.5 - -0.05 * \left(\frac{\mathrm{h}}{\mathrm{d}} - 1\right)$

In this case the ratio is equal to 0.19.

$$\frac{h}{d} = \frac{7.81\ m}{40.48\ m} = 0.19 < 1$$

Thus, the global coefficients and the respective wind pressure values are represented in the table below.

Table 8: Values of Wind Pressure on Vertical Walls

	C_e	P_s [kg/m²]
Upwind surface	0.72	$p_s = p * C_e = 104.6 * 0.72 = 75.24$
Downwind surface	-0.65	$p_s = p * C_e = 104.6 * -0.65 = -68.45$
Side surface	-0.34	$p_s = p * C_e = 104.6 * -0.34 = -65.42$

3.4.1.7. Wind Force

The wind force is assumed to be perpendicular to the vertical walls of the building and only the critical value between the above figures is used.

In this case, the wind force is determined by the pressure for the upwind elements together with the value of the grazing action on the roof of the designed building, expressed in kN, for both walls of the building (see formula (20) and (21)).[11]

$$F_{against\ wall\ b} = p_s * b * h + p_f * b * d = 705.92\ kN \tag{20}$$

$$F_{against\ wall\ d} = p_s * d * h + p_f * b * d = 326.79\ kN \tag{21}$$

3.4.2. Earthquake Load

3.4.2.1. Zone Definition

The constructions in the Italian territory are all to be considered subject to seismic actions of intensity according to the site of the work (Furiozzi, Messina, & Paolini, 2019). Four zones have been identified in the Italian territory; each zone defines the maximum horizontal acceleration that can occur in the event of an earthquake, along with other fundamental parameters. These values can be defined with response spectra.

3.4.2.2. Response Spectrum Description

The response spectrum determines the maximum acceleration to which the structure will be subjected during a seismic event with a given probability of occurrence.

[11] For the full results refer to Appendix 5 – Wind Load Excel Sheet.

his graph is essential for the design of structures in seismic zones and takes into consideration the following characteristics:

a. the subsurface hazard
b. the category of the subsurface
c. the topographical conditions
d. the probability of occurrence
e. the nominal life of the structure
f. the damping value of the construction

The spectra used in the normative (NTC 2018) were obtained based on historical accelerero-grams recorded by dedicated seismic stations. An evaluation of the simple oscillator response was then performed, in terms of relative displacement, acceleration and velocity. The maximum response was then evaluated and reported in the response spectra used in the codes (NTC 2018). In fact, for the purpose of designing and verifying a structure in a seismic zone, it is not necessary to know the response for each instant. On the other hand, the peak acceleration is of great importance, hence the response spectra are used (Pisapia, Spettri di risposta elastici: come sono stati ottenuti grazie a un lollipop (in 4 step), 2019).

3.4.2.3. Study Case 'Casa Haus inge'

In the case study of 'Casa Haus inge', the construction will be planned in the municipality of 'Magrè on the Wine Road', in the province of Bolzano (indicated by the red arrow on Figure 17, which represents the fourth seismic zones in Italy). The site has longitude 11.20985 and latitude 46.28739, placing it in the so-called 'Zone 4 of low seismicity' in Italy.

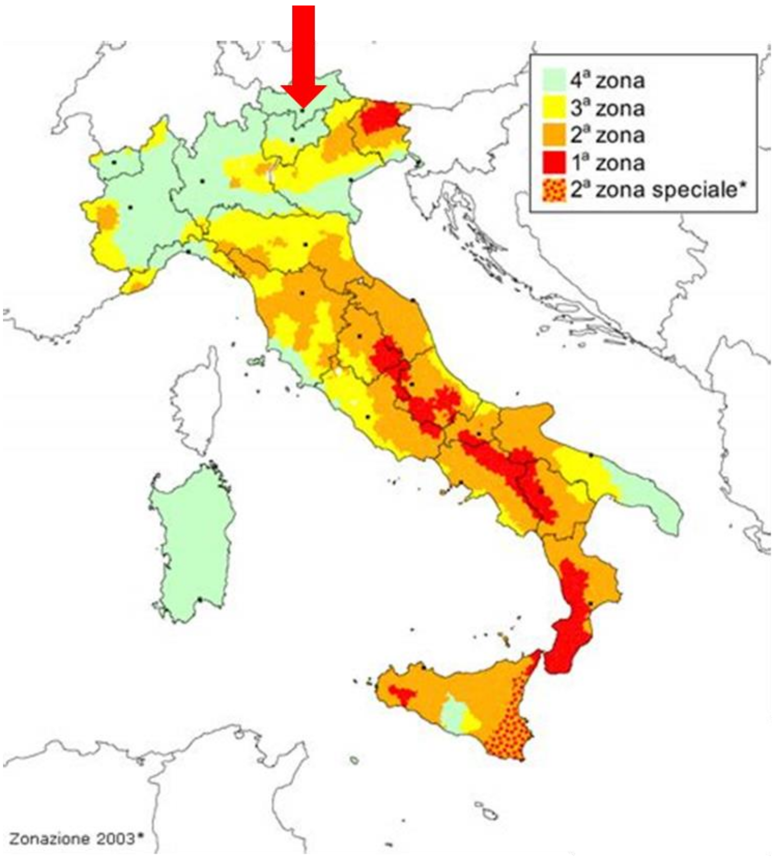

Figure 17: Location of Construction in Italian Seismic Zones

The response spectra will be determined with the automatic programme of the Upper Council of Public Works. Using the automatic programme, the response spectra for the Magrè on the Wine Road site can be obtained.

As can be seen from Figure 18, the response spectra describe the change in acceleration over a two-second time period. In addition, nine spectra are shown, indicating nine different return periods. However, only four return periods will be analysed, based on the fundamental parameters of the response spectra.

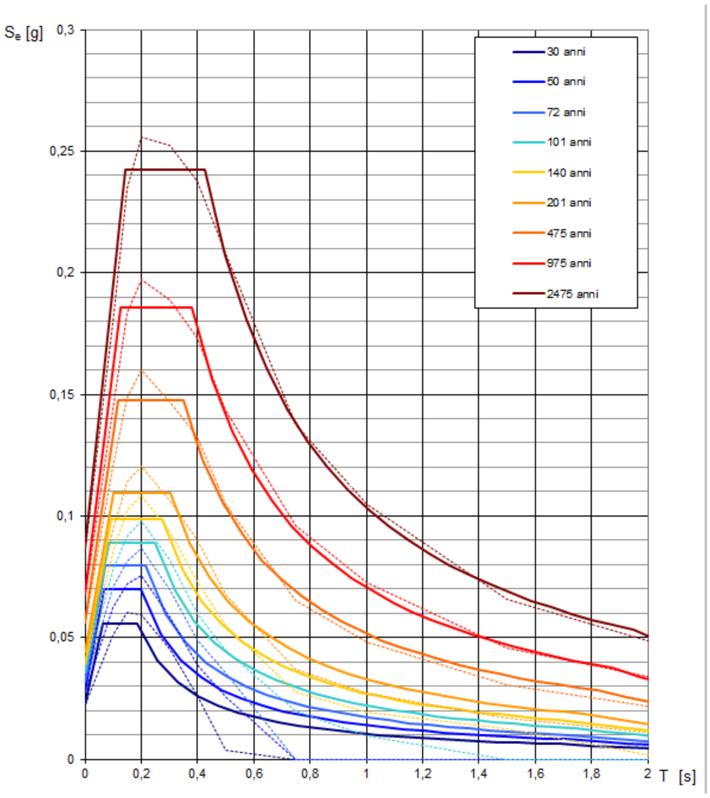

Figure 18: Response spectra for 'Magrè on the Wine Road'

3.4.2.4. Design Strategy

The spectral patterns shown in Figure 18 refer to a return time (T_R), established on the basis of a predetermined probability of passing the event (P_{VR}) and the reference life of the building itself (V_R).

$$T_R = -\frac{V_R}{\ln(1 - P_{RV})} \tag{22}$$

3.4.2.5. Reference Life of the Building

The reference life of the building is calculated as follows.

$$V_R = V_N * C_U \tag{23}$$

In which:

V_N is the nominal life of the construction and

C_U is the wear coefficient

For the 'Casa Haus inge' project, the following characteristics were specified as per NTC 2018 (see Table 9).

Table 9: Building's characteristics

Type of construction	**V_N [anni]**
2 – ordinary constructions	50
Type of works	**C_U**
II – buildings whose use involves normal crowding, with no environmentally hazardous contents and no essential public and social functions.	1.0

Therefore, the reference life of the construction, as per formula (23), has an unchanged value of 50 years.

3.4.2.6. Probability of Exceedance of the Seismic Event

The exceedance probability values are closely linked to the limit states of the building design.

The limit states for the design of a building in the event of seismic actions and their corresponding probability of exceedance values are as follows.

Table 10: Limit States and Probability of Exceedance

Limit state		**Probability of exceedance (P_{VR}) [%]**
Serviceability Limit State (SLS)	Immediate Operational Limit State (SLO)	0.81
	Damage Limit State (SLD)	0.63
Ultimate Limit State (ULS)	Live preservation Limit State (SLV)	0.10
	Collapse Prevention Limit State (SLC)	0.05

3.4.2.7. Return Time

As with formula (23), the return period is calculated for each limit state and reported in the following table (Table 11).

Table 11: Return time

		T_R [anni]
SLS	SLO	30
	SLD	50
ULS	SLV	475
	SLC	975

Based on the previously individualised response spectra, four response spectra can be extrapolated for each limit state (refer to Figure 19).

Response spectra are necessary for the determination of three fundamental parameters that define the basic seismic severity:

1. maximum horizontal acceleration with respect to the site — a_g
2. Maximum value of the amplification factor of the spectrum in horizontal acceleration — F_0
3. Reference value for determining the start period of the constant velocity section of the horizontal acceleration spectrum — T^*_c

For a designer, the values of the fundamental parameters as a function of return times are the most interesting to analyse and the most important to use.

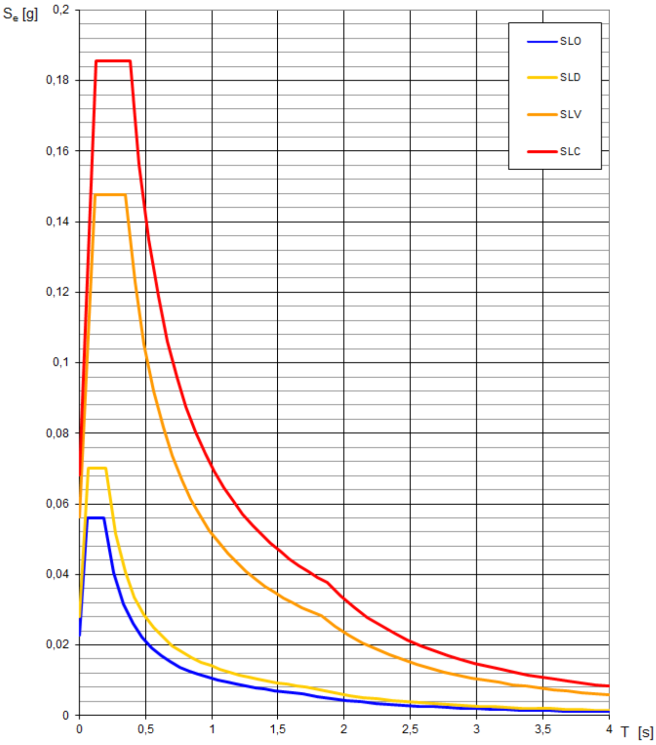

Figure 19: Design response spectra

The values are shown in the table below (Table 12).

Table 12: Core Parameter Values for Return Periods

LIMIT STATE		a_g [g/10]	F_0	T^*_c [s]	Return Time T_R [anni]
SLS	**SLO**	0.023	2.473	0.187	30
	SLD	0.028	2.498	0.199	50
SLU	**SLV**	0.056	2.628	0.349	475
	SLC	0.068	2.724	0.381	975

3.4.2.8. Design Strategy Choice

For the purpose of this report, only the life-saving limit state verification will be performed; in fact, this limit state defines failures and collapses of the non-structural and system components together with significant damage to the structural components. However, the structure retains part of the stiffness and strength for vertical actions and provides a margin of safety against collapse (Furiozzi, Messina, & Paolini, 2019).

3.4.2.9. Definition of the Design Action

Local Seismic Response

The local seismic response is given by the site conditions such as soil characteristics and topographical surface.

For the case of 'House Haus inge' the soil characteristics are given in Table 13.

Table 13: Soil Characteristics

Soil Category	**Description**
C	Deposits of medium coarse-grained soils or medium coarse-grained soils, namely moderately thickened sands and gravels or medium clays
Topographical surface	**Description**
T1	Flat surface

Using the automatic calculation tool,[12] stratigraphic category coefficients as a function of topographic category (C_C) and stratigraphic amplification coefficients (S_S) can be found (see Table 14).

Table 14: Soil Coefficient Values

LIMIT STATE		**S_S**	**C_C**
SLS	**SLO**	1.5	1.826
	SLD	1.5	1.790
ULS	**SLV**	1.5	1.486
	SLC	1.5	1.444

Furthermore, the topographical surface determines topographical amplification values (S_T) which when multiplied by the stratigraphic amplification coefficients (S_S) result in the dependent factor of the foundation soil (S). However, since the work is on a flat surface, the multiplicative value (S_T) is equal to 1 and therefore the factor of the foundation soil will be the same as the stratigraphic amplification coefficient.

$$S = S_S * S_T = 1.5 * 1 = 1.5 \tag{24}$$

3.4.2.10. Design Response Spectrum

For the determination of the design response spectrum, the dependent parameters and the behaviour factor of the structure are required.

3.4.2.10.1. Dependent Parameters

These refer to the factor 'η' which alters the elastic spectrum for damping of 5% (in the case of 'Casa Haus inge') and three significant times (see equations (25), (26) and (27)), which determine three values of the elastic response spectrum under acceleration.

[12] See Appendix 3 – Automated Excel Sheet for Seismic Spectrum (NTC 2018).

$$T_C = C_C * T_C^* \tag{25}$$

$$T_B = \frac{T_C}{3} \tag{26}$$

$$T_D = 4.0 * \frac{a_g}{g} + 1.6 \tag{27}$$

In which:

$$g = 9.81 \frac{m}{s^2}$$

The values of the above-mentioned dependent parameters and the respective values of the elastic response spectrum were obtained with the automatic calculation tool[13] and are shown in the table below.

Table 15: Dependent Parameters and Elastic Response Spectrum

DEPENDENT PARAMETERS		Design Response spectra (S_e) [m/s^2]
SLV		
η	0.667	
TB [s]	0.173	1.48
TC [s]	0.519	1.48
TD [s]	1.825	0.042

3.4.2.10.2. Structural Behaviour Factor

Constructions subject to seismic action, which are not equipped with adequate isolation and/or dissipation devices, must be designed according to one of the following structural behaviours:

1. non-dissipative structural behaviour
2. dissipative structural behaviour

For a non-dissipative structural behaviour, all members and connections remain in the elastic field; the demand resulting from seismic and other actions is calculated on the basis of the limit state to which it refers, but independently of the type of structure and without taking into account material non-linarites, by means of an elastic model.

For dissipative structural behaviour, when assessing the demand, a large number of members and/or connections evolve in the plastic field, while the rest of the structure remains in the elastic field. The demand resulting from seismic and other actions is calculated, depending on the limit state to which it refers and the type of structure, taking into account the dissipative capacity of the materials (Redazione DEI, 2019).

3.4.2.10.3. Non-Dissipative Structural Behaviour

The choice of analysing the structure as a structure with non-dissipative behaviour has been made because in this way a simplification of the verifications will be adopted. In fact, in a reinforced concrete structure with dissipative behaviour, the construction requirements are specified in terms of geometric and reinforcement limitations. This is not the case in a structure with non-dissipative behaviour, saving time in the design phase.

[13] See Appendix 3 – Automated Excel Sheet for Seismic Spectrum (NTC 2018).

Another characteristic of non-dissipative structural behaviour is the reduction of the behaviour factor, following the formula below (28).

$$1 \leq q_{DN} = \frac{2}{3} q_{CD-"B"} \leq 1.5 \tag{28}$$

The reduction in the behaviour factor results in a seismic action with a higher value than for dissipative structures. In fact, a non-dissipative structure is characterised by greater strength than its ductility. This results in a seismic response purely dependent on the stiffness and strength of the structure itself (Pisapia, Comportamento strutturale non dissipativo: il calcolo elastico torna alla ribalta [NTC2018], 2019).

The choice to use non-dissipative structural behaviour is given by the complexity of the requirements and conditions. As referred to in the Italian Technical Standards for Construction (NTC 2018), in the case of non-dissipative structural behaviour, reference is made exclusively to the regulations for concrete constructions, without any additional requirements. The only condition to be maintained is the design of the elements in such a way as to remain in the essentially elastic domain (Redazione DEI, 2019).

On the other hand, in the case of dissipative structural behaviour, the design principles and criteria are applied extensively for all structural elements, so that they contribute to the realisation of the cyclic inelastic dissipative and globally stable mechanisms.

Furthermore, the location of the structure in the various Italian seismic zones influences the choice of behaviour factor.

All buildings on Italian territory are to be considered affected by possible seismic actions with different intensities according to their geographical location. As can be seen from Figure 17, the structure will be built in a zone with a minimum level of danger.

As mentioned in the regulations, constructions in zone 4 admit safety verification criteria with simplified methods (Furiozzi, Messina, & Paolini, 2019).

3.4.2.10.4. Behaviour Factor

As previously established, the choice to use a non-dissipative construction scheme leads to the use of a low behaviour factor (28).

I The minimum value for the low ductility class ($q_{CD-"B"}$) was chosen on the basis of Italian regulations and with the assumption that the structure consists of reinforced concrete with non-coupled walls.

$$1 \leq q_{DN} = \frac{2}{3} q_{CD-"B"} \leq 1.5$$

$$q_{DN} = \frac{2}{3} * 3 = 2 \therefore q_{DN} = 1.5$$

With:

$$q_{CD-"B"} = 3$$

Due to the fact that the value given by the formula exceeds the imposed limit, the factor of 1.5 was assigned to make the structure not entirely elastic, but substantially elastic. In fact, to make the structure and its elements entirely elastic, a behaviour factor of 1 must be used.

3.4.2.10.5. Value of the Design Response Spectrum

The value of the design response spectrum depends not only on the parameters and factors mentioned above but also on the period of the structure's own vibration. This can be estimated as follows.

$$T = 0.075 * H^{\frac{3}{4}} = 0.0755 * 7.81^{\frac{3}{4}} = 0.35 \tag{29}$$

Four situations are proposed by the NTC 2018 for determining the design response spectrum:

1. *$0 \leq T < T_B$*
2. *$T_B \leq T < T_C$*
3. *$T_C \leq T < T_D$*
4. *$T_D \leq T$*

With reference to the building's own vibration period (29), the second situation is the case for the 'Casa Haus inge' project.

$$T_B \ \leq \ T \ < \ T_C \quad \therefore 0.173s < 0.35s < 0.519s \tag{30}$$

The value of the response spectrum is therefore calculated according to the following expression and applies to the second situation (31).

$$S_d(T) = \ a_g * S * \frac{1}{q} * F \tag{31}$$

In which:

$$F = F_0$$

Therefore

$$S_d(T) = \ 0.056 * 1.5 * \frac{1}{1.5} * 2.628 = 1.477 \frac{m}{s^2}$$

3.4.2.11. Vertical Seismic Action

Only the horizontal seismic action is taken into account, since as established by NTC 2018 the vertical seismic action can be neglected if the work does not present the following characteristics:

- horizontal slabs with a span of more than 20 m
- prestressed elements (excluding slabs with a span of less than 8 m);
- corbelled elements with a span exceeding 4 m;
- thrust type structures;
- pillars in false and/or
- buildings with suspended floors.

For the case of 'Casa Haus inge' this requirement is fulfilled, therefore the vertical seismic action is to be neglected.

3.4.2.12. Horizontal Seismic Action

Total Horizontal Seismic Action

The total horizontal seismic action is determined according to equation (32), as follows.[14]

$$F_h = S_d(T)\frac{W\lambda}{g} \tag{32}$$

In which:

W is the whole weight of the construction and

$\lambda = 1$

The total building weight for 'Casa Haus inge' is calculated considering the secondary structural elements, hence the floors of the designed storeys.

The building was designed on two floors and with two equivalent floors, each with an area of 2051.27 m^2 and a thickness of 32 cm.

$$G_1 = \left(2051.27\, m^2 * 0.32\, m * 25\frac{kN}{m^3}\right) * 2 = 32820.32\, kN \tag{33}$$

Moreover, as stated by the Italian codes (NTC 2018) the combination of the loads according to the seismic combination needs to be included (Redazione DEI, 2019).

$$W = G_1 + G_n + Q_i * \psi_{2,i} \tag{34}$$

The additional permanent load (G_2) considered in this case is the sum of

- → 320 kg/m^2 describing the pavement's load on the first-floor slab and
- → 200 kg/m^2 describing the partition walls in the first-floor slab.

Thus, a total additional load of 520 kg/m^2 will be considered for the permanent non-structural load on the first floor. On the other hand, the roof is expected to accommodate solar panels, characterised with a weight of 70 kg/m^2.

Additionally, it is foreseen by the construction codes (NTC 2018) that residential buildings experience an incidental load ranging between 200 and 400 kg/m^2, hence 300 kg/m^2 will be considered on the first-floor slab.[15]

The additional values are implemented according to the calculation combination shown in formula (34). The load combination takes into account the probability of the presence of the accidental load, using a combination coefficient (ψ_2) for residential structures, such as for the 'Casa Haus inge' case, of 0.3 (Furiozzi, Messina, & Paolini, 2019).

$$W = G_1 + G_2 + G_3 + Q_1 * \psi_{2,1} = 32820.3 + 5.2 * 2051.3 + 0.7 * 2051.3 + 3 * 2051.3 * 0.3 = 46769\, kN$$

[14] Note that the aid of a previously prepared calculation sheet has been utilized. Refer to Appendix 4 – Seismic Action Excel Sheet.

[15] The values indicated for the additional permanent loads as well as the accidental loads have been estimated. The determination of all the exact loads acting on the structure can be found in a later chapter (see section 3.5.10).

The horizontal seismic action can finally be calculated and is found to have a magnitude of approximately 7039.5 kN orthogonally to the building.

$$F_h = 1.48\frac{m}{s^2}\frac{46769\ kN * 1}{9.81\frac{m}{s^2}} = 7039.5\ kN \tag{35}$$

Distribution of Total Action to Individual Levels

The action derived from expression (35) is equivalent to the total action applied at the midpoint of the building height. However, it is assumed that with the height of the foundation floor, the displacements increase linearly. This results in a distribution of the total action on each level of the building, in this case with an inter-storey height of 3.3 m (see Figure 20).

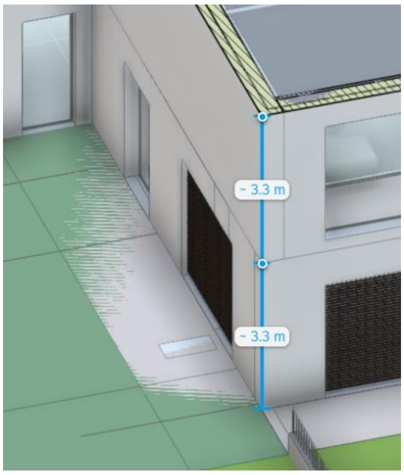

Figure 20: Inter-storey height

The forces to be applied per storey are defined by the formula below (36).

$$F_i = F_h * \rho_i \tag{36}$$

In which:

$\rho_i = \frac{z_i W_i}{z_1 W_1 + z_1 W_1}$

z_i is the inter-storey height from the foundation level

By applying formula (36), the following values were extrapolated.

$$F_1 = F_h * \rho_1 = 7039.5 * 0.3 = 2091\ kN$$

$$F_2 = F_h * \rho_2 = 7039.5 * 0.7 = 4948.5\ kN$$

3.4.3. Conclusion

To assess the prevalence of wind or earthquake pressure, the forces applied to the upper floor will be taken into account. Furthermore, the prevailing forces will be preferred in favour of design safety.

Thus, the values for wind pressure and earthquake pressure, both expressed in kN, are as follows:

→ Wind pressure 705.9 kN

→ Earthquake pressure 4948.5 kN

Due to the fact that the earthquake pressure exceeds the wind pressure, the latter can be excluded.

3.5. Full Flat Slab Analysis

3.5.1. Foreword

As mentioned in paragraph 1.1, this project involves the construction of a two-storey structure in Magrè on the Wine Road. The structure has a relatively large floor plan with various features, such as indoor and outdoor gardens and balconies (see Figure 21).

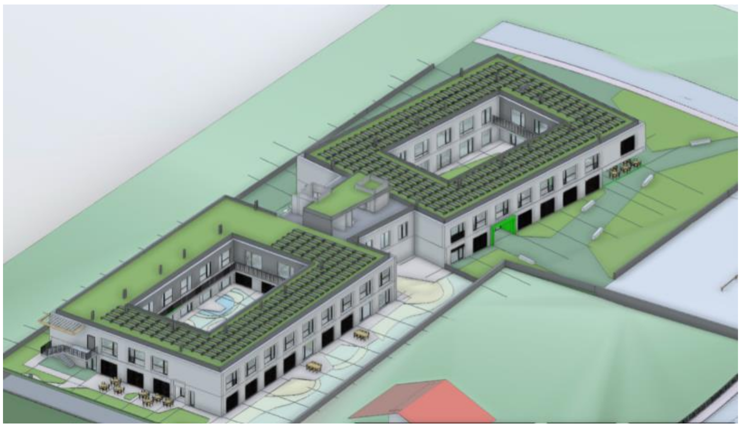

Figure 21: Rendering of 'Casa Haus inge'

With the intention of studying the best method to model the lightened slab with finite elements, a section of the building was chosen and will be analysed in this chapter.

The section (see Figure 22) was determined taking into account demanding and critical situations, thus a section with balcony and photovoltaic panels on the roof was preferred.

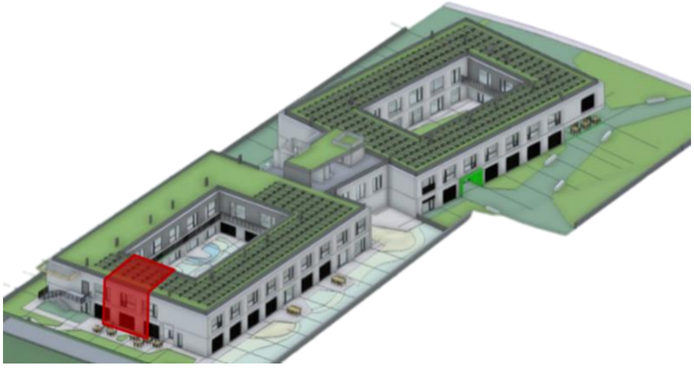

Figure 22: Chosen section to analyse

It is important to note that the section will not be braced, so the earthquake resistance check will not be performed. Furthermore, it was necessary to constrain the moments on both edges of the two slabs to take into account the continuity of the floors. These two characteristics modify the behaviour of the concrete slabs in such a way that they are considered unidirectional. Although the slabs of the entire structure are bidirectional, the concept was accepted for this particular study.

Since the purpose of this analysis is to determine and compare the results of different FEM approaches for the slab, it is relevant to study only the slabs, neglecting the septa.

It should be noted that a preliminary pre-static analysis of the building has already been carried out, it can therefore be assumed that the septa are sufficiently stable and resistant to loads from its operation.

3.5.2. Description of the Chosen Section

The structure has two unidirectional slabs with two distinct surfaces; in fact, the first-floor slab extends further in one direction to simulate the balcony (refer to Figure 23).

As for the walls, the four load-bearing septa all have the same geometric configuration, namely a thickness of 0.25 m and a width of 1.7 m.

Their height, however, varies. The wall extending from the foundation level (-1 m from ground level) to the first-floor slab has a height of 4.14 m, while the wall extending from the first floor to the roof has a height of 3.63 m.

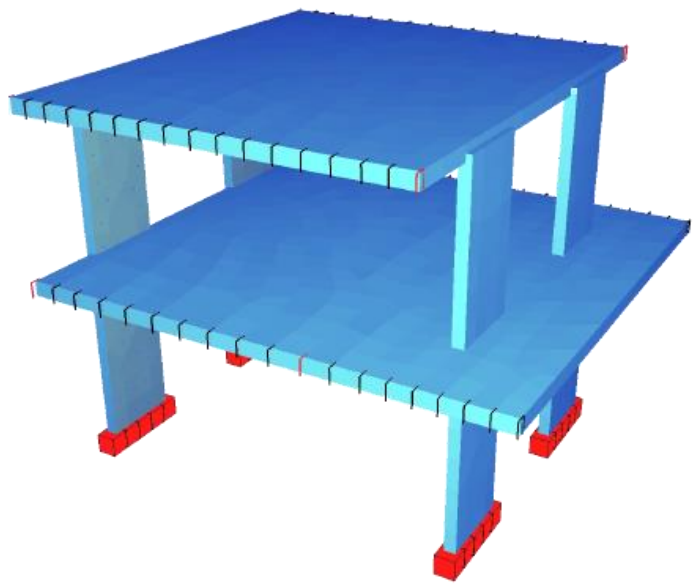

Figure 23: Finite Element Model

3.5.2.1. First Floor Slab

As mentioned previously, the first-floor slab extends 1.8 m to simulate the presence of the balcony. Thus, its floor dimensions present a span of 10.1 m in one direction and 9.5 m in the other, with a total surface area of approximately 96 m^2.

Its thickness is 32 cm.

3.5.2.2. Roof Slab

The covering slab has a span of 8.3 m in one direction and 9.5 m in the other, resulting in a surface area of approximately 79 m^2.

Its thickness is 32 cm.

3.5.3. Calculation Method

The study of the structures was conducted according to the methods of building science assuming elastic, homogeneous and isotropic materials.

The search for stress parameters was carried out considering the heaviest load arrangements and using automatic calculation codes for structural analysis.

Strength verifications of the sections were carried out according to the semi-probabilistic limit state method in accordance with the regulations in force.

All the automatic calculation codes used for the verification of the structures are of safe and proven validity. The software used for the analysis and verification of the structure is described below.

3.5.3.1. SOFiSTiK

Numerical modelling was conducted with the aid of the commercial finite element software SOFiSTiK. Within the design environment of the software, the three-dimensional model was created, the design loads were assigned, the load combinations were defined, and finally the ultimate limit state and serviceability limit state verifications of the structure are performed.

3.5.3.2. Reliability

All the automatic calculation codes used for the calculation and verification of structures are of proven and reliable validity and have been used in accordance with their characteristics. This assertion is supported by a large number of calculation codes on the market with several years of use and continuous updating. All the outputs obtained are validated by comparison with the results of manual calculation methods carried out using traditional methods and adopted during the initial pre-dimensioning of the structure.

3.5.4. Analysis Type

The type of structural analysis conducted was linear elastic anlaysis, in other words for the determination of the stress state of horizontal and vertical structural elements.

The linear elastic structural analysis was conducted using the displacement method for the evaluation of the stress-strain state induced by static loads.

3.5.5. Characteristics of the Materials

The materials used in the structure as shown in Table 16.

Table 16: Materials Used

Mat	Classification
1	CA 30/37 (Italia)
2	B 450 C (Italia)

3.5.5.1. Concrete

The type of concrete chosen for the structure 'Casa Haus inge' is type C30/37 and presents the following characteristics (see Table 17).

Table 17: Characteristics C30/37

Young's modulus E	33019	[N/mm2]	Safetyfactor	1.50	[-]
Poisson's ratio µ	0.20	[-]	Strength fc	26.10	[MPa]
Shear modulus G	13758	[N/mm2]	Nominal strength fck	30.71	[MPa]
Compression modulus K	18344	[N/mm2]	Tensile strength fctm	2.94	[MPa]
Nominal Weight γ	25.0	[kN/m3]	Tensile strength fctk,05	2.06	[MPa]
Mean density ρ	2400.0	[kg/m3]	Tensile strength fctk,95	3.82	[MPa]
Elongation coefficient α	1.00E-05	[1/K]	Bond strength fbd	3.09	[MPa]
			Service strength fcm	38.71	[MPa]
			Fatigue strength fcd,fat	15.26	[MPa]
			Tensile strength fctd	1.37	[MPa]
			Tensile failure energy Gf	0.14	[N/mm]

Moreover, the stress-strain deformation accounting for the serviceability limit state (SLS) of such material can be visualized in Table 18.

Table 18: Stress-strain deformations for SLS

Stress-Strain for serviceability	**ε[o/oo]**	**σ-m[MPa]**	**E-t[N/mm2]**
Is only valid within the defined	0.000	0.00	33019
stress range	-0.544	-16.11	26145
	-1.087	-28.27	18446
	-1.631	-35.99	9787
	-2.174	-38.71	0
	-3.500	-19.93	-29977
	Safetyfactor	1.20	

On the other hand, its deformations for the ultimate limit state (ULS) are as follows (see Table 19).

Table 19: Stress-strain deformations for ULS

Stress-Strain for ultimate load	**ε[o/oo]**	**σ-u[MPa]**	**E-t[N/mm2]**
Is only valid within the defined	0.000	0.00	26103
stress range	-2.000	-26.10	0
	-3.500	-26.10	0
	Safetyfactor	1.50	

Lastly, the stress-strain curve is shown in Figure 24 below, combining all the values seen before.

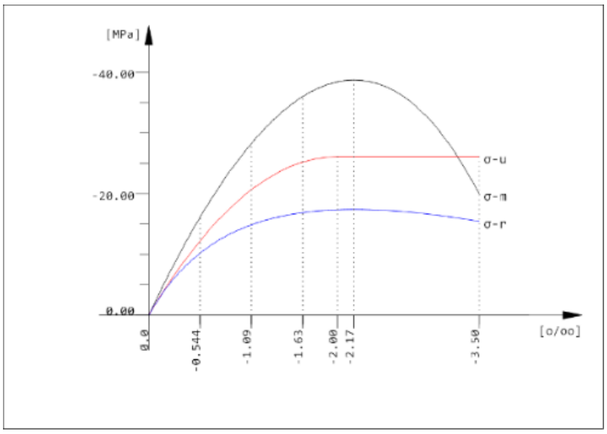

Figure 24: Stress-strain curve - C30/37

3.5.5.2. Reinforcement Steel

The type of reinforcement steel designed to be used in this development is B 450 C; this presents the following characteristics (see Table 20).

Table 20: Characteristics B 450 C

Young's modulus E	**200000**	**[N/mm2]**	**Safetyfactor**	**1.15**	**[-]**
Poisson's ratio µ	0.30	[-]	Yield stress fy	450.00	[MPa]
Shear modulus G	76923	[N/mm2]	Compressive yield fyc	450.00	[MPa]
Compression modulus K	166667	[N/mm2]	Tensile strength ft	540.00	[MPa]
Nominal Weight γ	78.5	[kN/m3]	Compressive strength fc	540.00	[MPa]
Mean density ρ	7850.0	[kg/m3]	Ultimate strain	67.50	[o/oo]
Elongation coefficient α	1.20E-05	[1/K]	relative bond coeff.	1.00	[-]
max. thickness t-max	32.00	[mm]	EN 1992 bond coeff. k1	0.80	[-]
			Hardening modulus Eh	0.00	[MPa]
			Proportional limit fp	450.00	[MPa]
			Dynamic allowance σ-dyn	152.17	[MPa]

Its stress-strain deformation accounting for the serviceability limit state (SLS) of such material can be visualized in Table 21.

Table 21: Stress-strain deformation for SLS

Stress-Strain for serviceability	**ε[o/oo]**	**σ-m[MPa]**	**E-t[N/mm2]**
Is also extended beyond the	1000.000	540.00	0
defined stress range	67.500	540.00	0
	2.250	450.00	1379
	0.000	0.00	200000
	-2.250	-450.00	1379
	-67.500	-540.00	0
	-1000.000	-540.00	0
	Safetyfactor	1.15	

On the other hand, its deformations for the ultimate limit state (ULS) are as follows.

Table 22: stress-strain deformations for ULS

Stress-Strain for ultimate load	ε[o/oo]	σ-u[MPa]	E-t[N/mm2]
Is also extended beyond the	1000.000	469.57	0
defined stress range	67.500	469.57	0
	1.957	391.30	1194
	0.000	0.00	200000
	-1.957	-391.30	1194
	-67.500	-469.57	0
	-1000.000	-469.57	0
	Safetyfactor	(1.15)	

Lastly, the stress-strain curve for this type of reinforcement steel is shown in Figure 25.

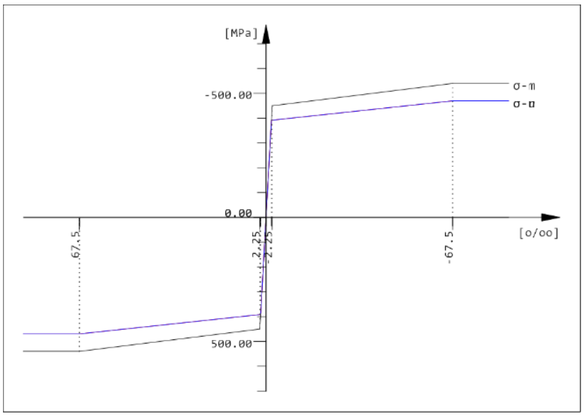

Figure 25: Stress-strain curve - B 450 C

3.5.6. Exposure Class

The following exposure class (see Table 23) was established for the elements of the structure in reinforced concrete.

Table 23: Exposure class

EXPOSURE CLASS	ENVIRONMENT DESCRIPTION	INFORMATIVE EXAMPLES OF SITUATIONS TO WHICH EXPOSURE CLASSES MAY APPLY
Carbonation-induced corrosion (where the concrete contains metal reinforcements or inserts and is exposed to air and moisture).		
XC1	Dry or permanently wet concrete	Concrete inside buildings with low humidity Concrete permanently immersed in water

The durability of reinforced concrete structures will be ensured by adherence to the minimum concrete coverings required by the standards (Redazione DEI, 2019), with mix-design prescriptions for concretes and through the control of cracking according to the different load combinations envisaged in the design (European Commission, 2006).

3.5.7. Combination of Loads

3.5.7.1. Checks at Ultimate Limit State (ULS)

Structure Resistance Limit State (STR)

Limit state verifications must be carried out for all the most severe loading conditions that may occur on the structure. Structural limit state verifications are performed by applying the reductive coefficients given in column A1 of Table 24.

Table 24: Partial factors: ultimate limit states for buildings

		Coefficients γ_F	**EQU**	**A1**	**A2**
Permanent loads G_1	Favourable	γ_{G1}	0.9	1.0	1.1
	Unfavourable		1.1	1.3	1.1
Non-structural permanent loads G_2	Favourable	γ_{G2}	0.8	0.8	0.8
	Unfavourable		1.5	1.5	1.3
Variable actions Q	Favourable	γ_{Qi}	0.0	0.0	0.0
	Unfavourable		1.5	1.5	1.5

3.5.7.2. Checks at Serviceability Limit State (SLS)

The load combinations are evaluated by applying various reductive coefficients (see Table 25) to the variable actions.

Table 25: ψ factors for buildings

Action	**Ψ_{0j}**	**Ψ_{1j}**	**Ψ_{2j}**
Category A: domestic, residential	0.7	0.5	0.3
Category B: offices	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping	0.7	0.7	0.6
Category E: storage	1.0	0.9	0.8
Category F: vehicle weight (≤ 30 kN)	0.7	0.7	0.6
Category G: 30 kN < vehicle weight ≤ 160 kN	0.7	0.5	0.3
Category H: roofs accessible for maintenance	0.0	0.0	0.0
Wind loads on buildings	0.6	0.2	0.0
Snow loads on buildings (≤ 1000 m above sea level)	0.5	0.2	0.0
Snow loads on buildings (> 1000 m above sea level)	0.7	0.5	0.2
Temperature (non-fire) in buildings	0.6	0.5	0.0

Crack Control

The verification at the crack limit state depends on the class of concrete, the type of reinforcement steel chosen and the environmental conditions. In this case, the concrete is class C30/37 with a low-sensitive steel type with an exposure class XC1, which according to the standards (Redazione DEI, 2019) belongs to an ordinary environment.

These conditions result in a maximum cracking value:

- → For frequent combination of actions *≤ 0.40 mm*
- → For quasi-permanent combination of actions *≤ 0.30 mm*

Tensions Checks

As stipulated in the regulations (Redazione DEI, 2019), it is necessary to check that the stresses in both the concrete and the reinforcement steel do not exceed the maximum permitted values.

Maximum Compressive Tension of Concrete

The maximum concrete tension must comply with the following values:

- → For the characteristic SLS combination $\sigma_{c,max} \leq 0.60\, f_{ck}$
- → For the long-term SLS combination $\sigma_{c,max} \leq 0.45\, f_{ck}$

Maximum Steel Tension

The maximum steel tension must comply with the following values (European Commission, 2006):

$$\sigma_{s,max} \leq 0.80\, f_{yk}$$

Deformation Checks

Deformability limits are set according to the functionality of the structure with reference to use, static requirements and aesthetic functions (Redazione DEI, 2019).

According to Eurocode 2 (European Commission, 2006), the overall appearance and utility of the structure could be compromised when the calculated failure of a beam, slab or cantilever beam subjected to quasi-permanent loads exceeds the value of *span/250*.

Deformations that could damage adjacent parts of the structure must also be limited. For deflection after construction, the limit of *span/500* is usually considered accurate for quasi-permanent loads.

3.5.8. Load analysis

The loads due to the future use of the structure are given in the following paragraphs.

3.5.8.1. Self-Weight

The structure's own weight is given by the two floors with a thickness of 32 cm and density of 25 kN/m^3. In addition, the four load-bearing walls present the same density, but with a thickness of 25 cm.

In the 'Haus inge' project, 'Thermowand' walls will be used. These consist of two prefabricated crusts with an insulation layer as illustrated in Figure 26.

Figure 26: Thermowand

The hollow core will be filled with in-situ concrete; this will constitute the structural septum measuring 25 cm thick.

3.5.8.2. Non-Structural Permanent Weight

A distinction between the loads applied on the two floors is necessary; in fact, the two slabs serve different purposes.

Roof Slab

The roof slab has been designed as a 'green roof' so a layer of soil (green bundle) will be placed. In addition, photovoltaic panels have been designed as part of the building's energy production for greater eco-sustainability.

Green Bundle

The green package was designed as a soil layer with a density of 3.2 kN/m^3 and a thickness of 20 cm. Additionally, the distributed load was increased by 10 per cent for safety, thus amounting to 0.7 kN/m^2.

Solar Panels

A portion of the slab will be occupied by photovoltaics with a weight of 0.5 kN/m^2; however, for safety's sake their weight was taken over the entire roof slab.

First-Floor Slab

Pavement

The floor for the 'Casa Haus inge' building was designed according to the client's specifications with 2 cm thick tiles with a weight of 22 kN/m^3. On the other hand, two further layers will be placed between the reinforced concrete slab and the tiles as shown in Figure 27:

→ An 8 cm thick screed infill with a weight of 24 kN/m^3
→ A 14 cm thick layer of lightened sub-base with a weight of 8 kN/m^3

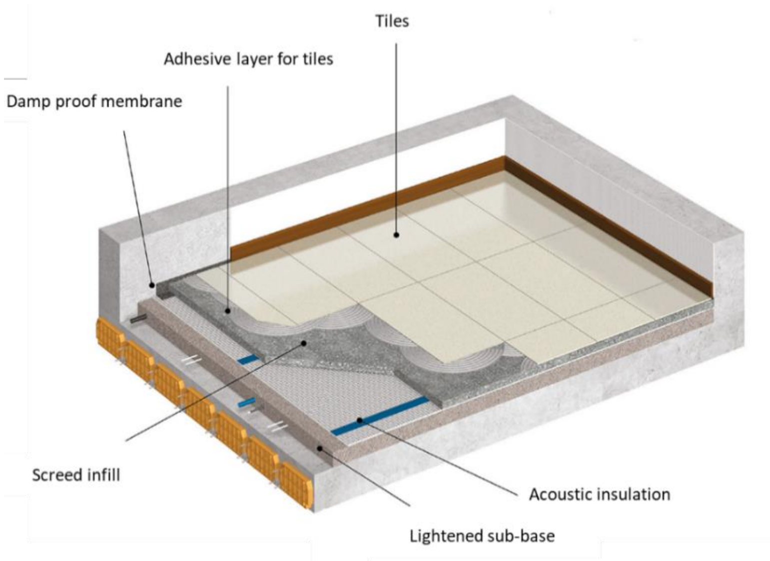

Figure 27: Paving scheme

The pavement will therefore reach a value of 3.5 kN/m^2.

Partitions

As explicated in the Technical Regulations for Construction (NTC 2018) "partitions and lightweight systems in residential and office buildings may be assumed, in general, to be equivalent distributed loads, provided that the floors have adequate transverse distribution capacity" (Redazione DEI, 2019).

In the case of the design of this report, the partitions will have two 2.5 cm thick layers of gypsum board with an own weight of 22 kN/m^3 and a layer of rock heath (density 1.5 kN/m^3) on the inside with a thickness of 10 cm. This therefore contributes to a linear weight of 4.5 kN/m, which according to the Technical Standards (NTC 2018) can be considered as a uniformly distributed load of 2 kN/m^2 (Redazione DEI, 2019).

Windows

The windows and balcony doors in the building were designed with a wooden frame (7 cm thick with a density of 6 kN/m^3) covered with aluminium (1.8 cm thick with a density of 27 kN/m^3) and three layers of 6 mm glass with a weight of 0.20 kN/m^2. Its scheme can be visualised in Figure 28.

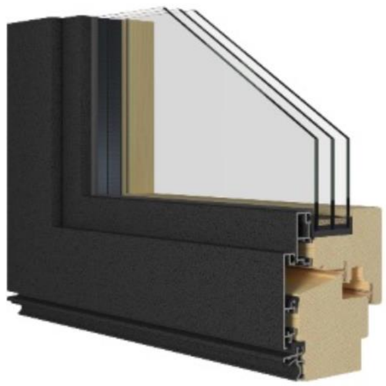

Figure 28: Typical window section

In addition, an upper concrete panel will be placed for protection against rain and snow as well as for aesthetic reasons, this will have a density of 24 kN/m^3, a height of 1 m and a thickness of 0.06 m.

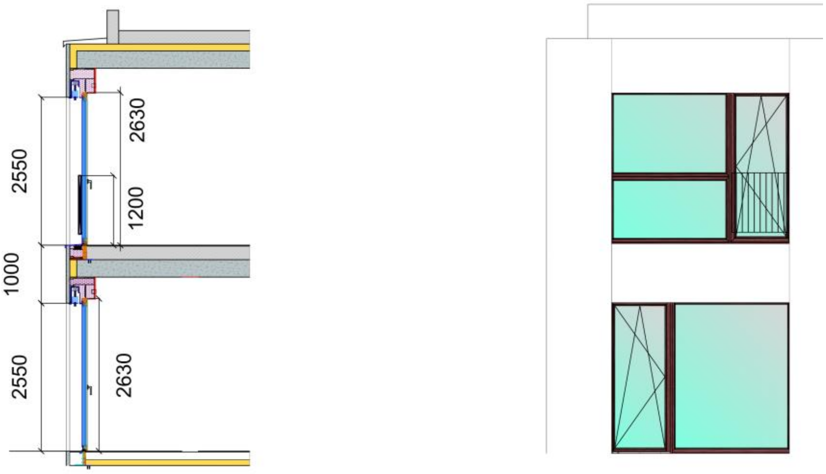

Figure 29: Window and Balcony door scheme

As can be seen from Figure 29, the windows are distinguished by storey; however, for the sake of safety, one type of diagram was used to calculate their weight.

The scheme considers a window extended over 3 m, including the concrete panel, the three panes of glass, the aluminium-clad wooden frame and the metal railing (with a height of 1.2 m, a base of 0.95 m and a linear weight of 0.2 kN/m). This results in a linear load of 3.2 kN/m.

Parapet

The metal railing on the balcony will be considered as a linear load on the outer edge amounting to 2 kN/m.

3.5.8.3. Variable Loads

The accidental loads are dictated by the technical regulations (Redazione DEI, 2019) as reported in Table 26.

Table 26: Variable Loads

Category	Envorments	q_k [kN/m^2]
A	**Environments for residential use**	
	Areas for domestic and residential activities; this category includes living quarters and related services, hotels (excluding crowded areas), hospital rooms	2.00
	Common staircases, balconies and hallways	4.00
H	**Roofings**	
	Cat. H roofs accessible for maintenance and repairs only	1.00

3.5.8.4. Snow Loads

The snow load is considered to be applied vertically on the roofs of the structure by means of the following expression.

$$q_s = q_{sk} * \mu_i * C_E * C_t \tag{37}$$

In which:

q_{sk}	is the reference value of the snow load on the ground
μ_i	is the cover shape coefficient
C_E	is the exposure coefficient
C_t	is the thermal coefficient

Snow Ground Load Reference Value

As established by the (NTC 2018) the snow load is closely related to the local climate and exposure conditions, hence the project site. To better establish the characteristic values of this load, the Italian territory has been divided into zones shown in Figure 30.

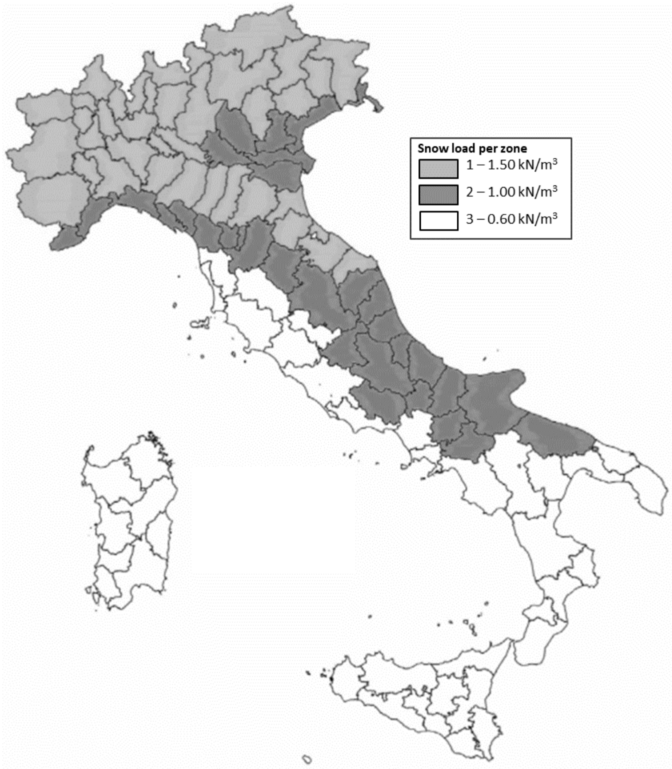

Figure 30: Italian zones according to snow loads

The construction is planned in the locality of 'Magrè in the Wine Road' at an altitude (a_s) of 241 metres above sea level. Given the location of the municipality, the area has been determined as an alpine zone with a reference value of the snow load on the ground as shown in Table 27.

Table 27: Zone and reference snow load

zone	description	q_{sk} [kN/m²]
a_s > 200m		
I - Aplin	Aosta, Belluno, Bergamo, Biella, **Bolzano,** Brescia, Como, Cuneo, Lecco, Pordenone, Sondrio, Torino, Trento, Udine, Verbano Cusio-Ossola, Vercelli, Vicenza	$q_{sk} = 1.39\left[1+\left(\frac{a_s}{758}\right)^2\right] = 1.54\frac{kN}{m^2}$

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Roofing Shape Coefficient

As can be deduced from the name of the coefficient, the roof shape coefficient takes into account the type of roofing designed for the building. In the case of 'Casa Haus inge', the roof will be a flat pitch; furthermore, there is a distinction between extended and non-extended flat pitch roofing.

To distinguish the flat roofing type, the equivalent plan dimension is calculated.[16]

$$L_c = 2W - \frac{W^2}{L} = 2 * 9.4 * \frac{9.4^2}{8.3} = 8.15\, m \tag{38}$$

In which:

W is the minimum floor dimension of the roof

L is the maximum floor dimension of the roof

Roofs with an equivalent size (L_C) greater than 50 m are considered 'extended' (Redazione DEI, 2019). Consequently, the shape coefficient of the canopy remains unchanged with a value of $\mu_1 = 0.80$.

Exposure Coefficient

The exposure coefficient considers the topography of the location. In fact, depending on the presence of trees, taller buildings or other terrain features that obstruct the wind in the area, the snow load may decrease or increase.

In this case, construction will take place in a flat area, which means that the coefficient will be considered equal to 1.

Thermal Coefficient

The thermal coefficient takes into account the reduction in snow load due to snow melting. This can be caused by heat loss from the construction.

This coefficient depends on the thermal insulation properties of the material used for the roof.

In the absence of a specific and documented study, as in this case, the coefficient should be set equal to 1, for safety's sake.

Snow Load Value

The snow load is therefore calculated using formula (39) as follows.

$$q_s = q_{sk} * \mu_1 * C_E * C_t = 1.54 * 0.80 * 0.9 * 1 = 1.11 \frac{kN}{m^2} \tag{39}$$

3.5.8.5. Sum of Loads

The loads due to the intended future operation of the structure and its own weight are shown in Table 28 and their positioning on the structure can be viewed in chapter 3.5.10.

[16] Calculations have been performed with the aid of a previously prepared spread sheet. See Appendix 6 – Snow Load Excel Sheet.

Table 28: Loads

Load Type	Description	Vaule
G	Own weight due to reinforced concrete elements	**1728.7 kN**
G_2	Permanent non-structural load	
	Green Bundle	0.7 kN/m^2
	Photovoltaic panels	0.5 kN/m^2
	Partitions	2 kN/m^2
	Windows and balcony doors	3.2 kN/m
	Parapet	2 kN/m
	Pavement	3.2 kN/m^2
	Sum	**606.7 kN**
Q_{A1}	Accidental load	
	Residential areas (in rooms)	2 kN/m^2
	Crowded areas (in corridors)	4 kN/m^2
	Sum	**198 kN**
Q_{A2}	Accidental load in balcony	4 kN/m^2
		68.8 kN
Q_H	Accidental load due to roof maintenance	1 kN/m^2
		78.8 kN
S	Distributed load due to snow	1.11 kN/m^2
		87.5 kN

The load combinations for the rare operating limit states and ultimate limit states are calculated as follows and the results are indicated in Table 29.

$$SLS_{chara.} = G + G_2 + Q_{A1} + Q_{A2} + Q_H + S \tag{40}$$

$$ULS = \gamma_{g1} * G + \gamma_{g2} * G_2 + \gamma_{q1} * Q_{A1} + \gamma_{q2} * Q_{A2} + \gamma_{qh} * Q_H + \gamma_{qS} * S \tag{41}$$

Table 29: Combination of loads

	γ_{g1}	γ_{g2}	γ_{qi}	**SLE rara**	2768.5 kN
Tot	1.3	1.5	1.5	**SLU**	3807 KN

3.5.9. Finite Element Model

The section of the structure analysed in this chapter can be visualised in its finite element shape as follows.

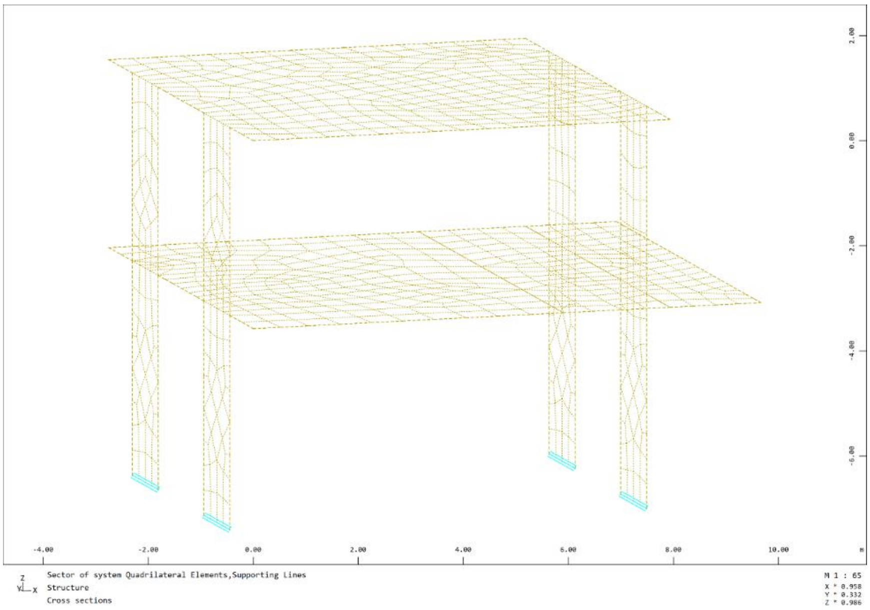


Figure 31: Basic finite element model

Figure 23 depicts the model showing the finite element modelling representation and thickness of each element. On the other hand, Figure 31 shows the counter and mesh size generated in the FEM software.

3.5.10. Loading Diagram

The loads calculated and described in chapter 3.5.8 can in this chapter be visualized.

3.5.10.1. Self-Weight

The self-weight accounts for the structure's concrete elements, hence the septa and the floors as indicated in Figure 32.

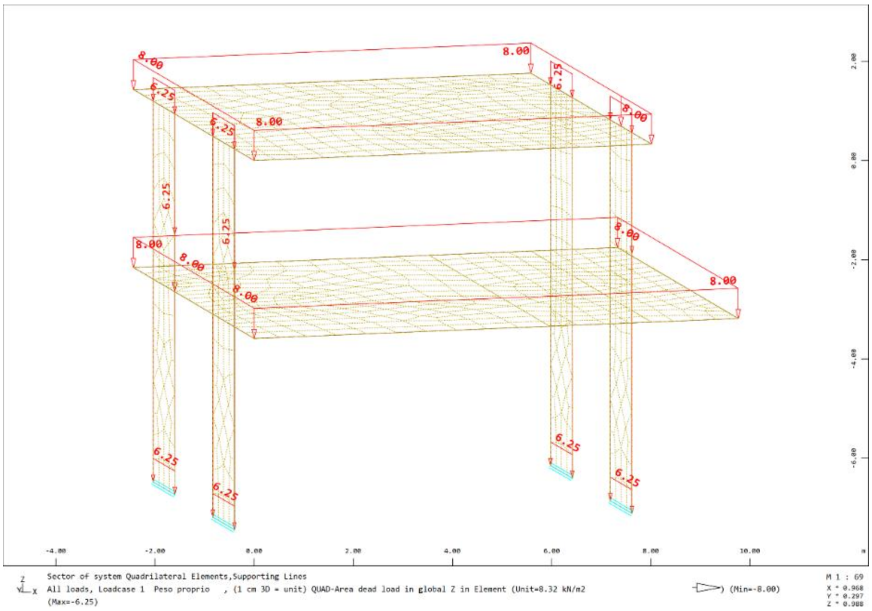


Figure 32: Self-weight

3.5.10.2. Permanent Non-Structural Weight

The permanent non-structural loads derive from the green bundle on the roof and from windows, pavement, partitions and parapet on the first-floor slab and have been positioned accordingly as seen in Figure 33.

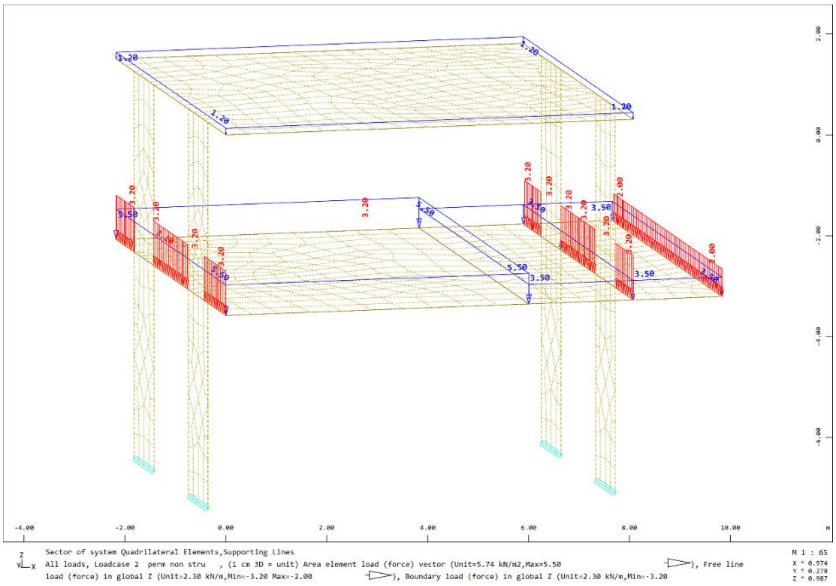


Figure 33: Permanent non-structural loads

3.5.10.3. Variable Loads

Variable Loads on the Intern

Variable loads in the structure on the first floor are devised into loads in residential zones (in this case the rooms for the clients of the building) and into loads in crowded zones, such as the corridor (see Figure 34).

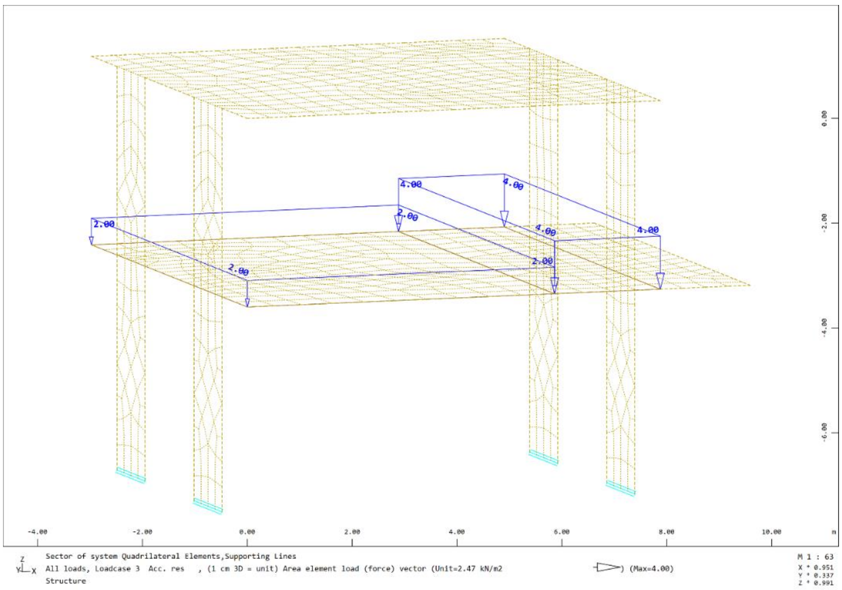


Figure 34: Variable loads on the intern

Variable Loads on the Extern

Similarly, as done on the inside portion of the structure, the balcony requires a variable load accounting for crowding (refer to Figure 35).

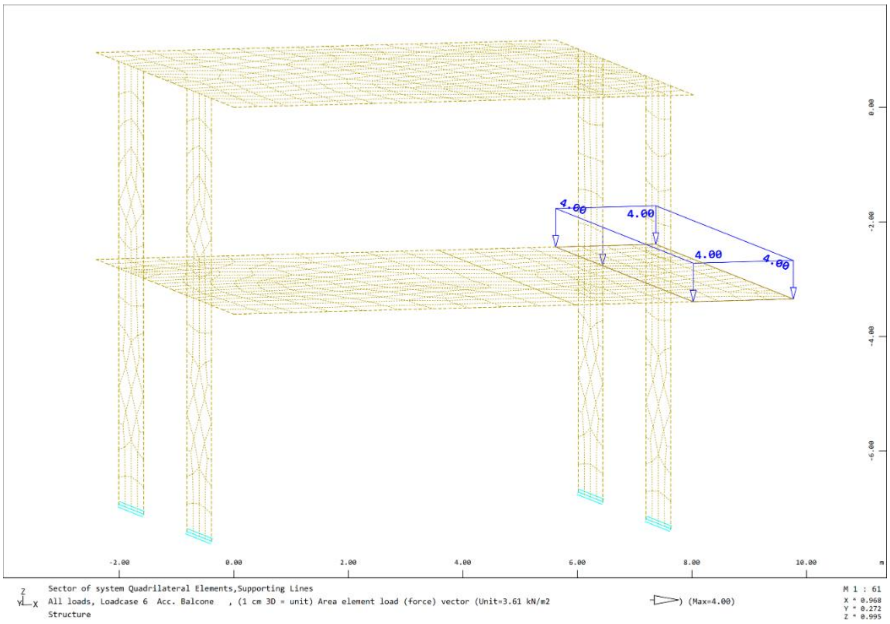


Figure 35: Variable loads on the extern

Variable Loads on the Roof

As mentioned previously, maintenance operations are to be expected on the roof, this has been considered by considering a uniformly distributed load spread over the whole roof section, as depicted in Figure 35.

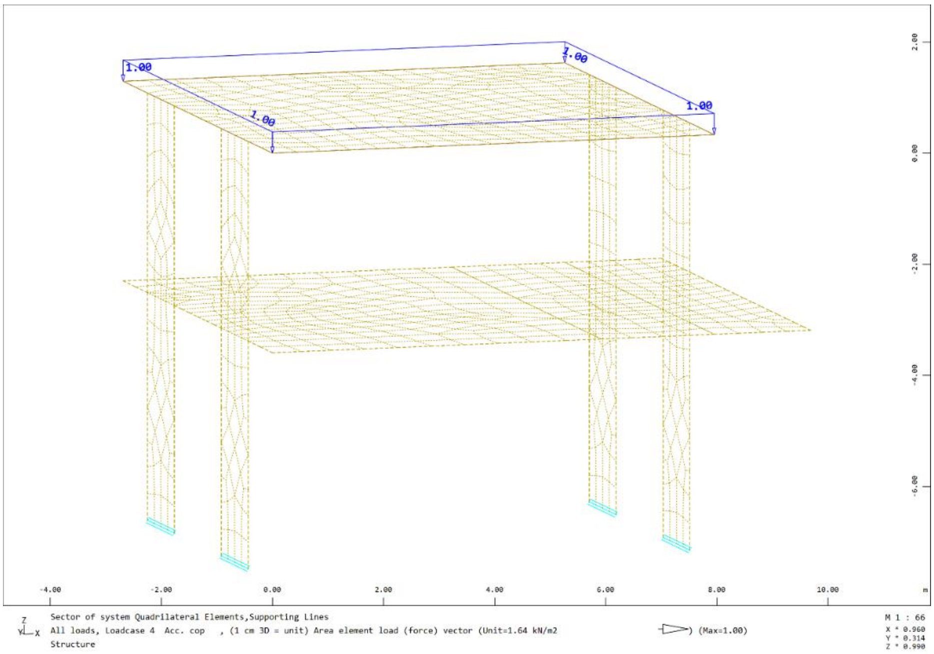


Figure 36: variable load on roof

3.5.10.4. Snow Load

Lastly, the snow load is considered on the whole surface of the roof (see Figure 37).

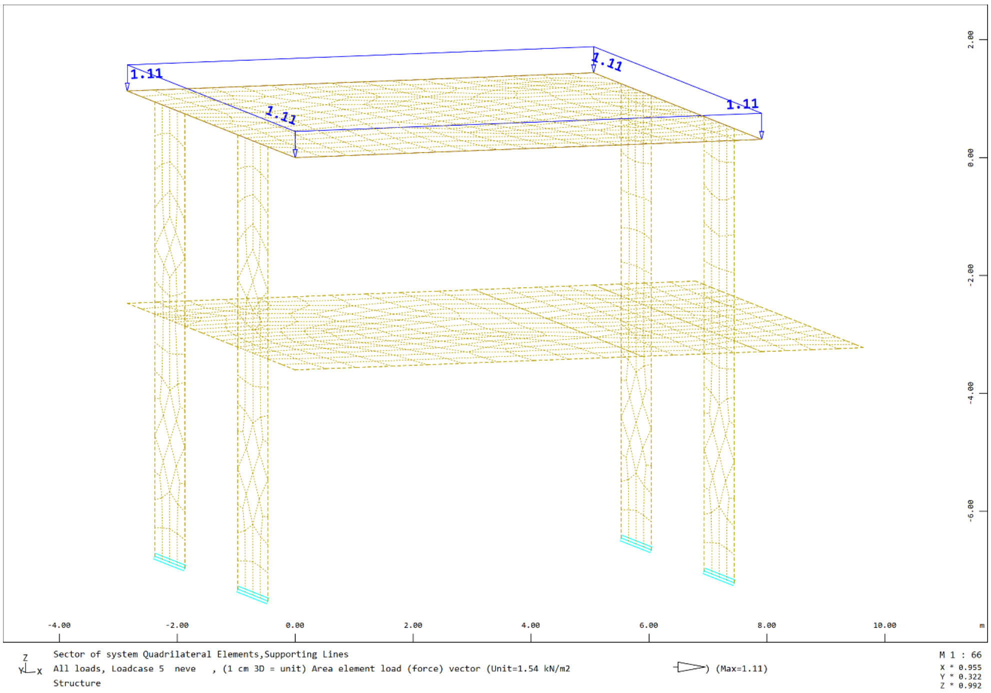


Figure 37: Snow load

3.5.11. Concrete Cover and Reinforcement

According to Eurocode 2 (European Commission, 2006), a minimum and maximum value for concrete covers and concrete reinforcement characteristics apply according to safety requirements.[17]

A minimum concrete cover must be provided to ensure:

- → the safe transmission of bonding forces
- → protection of the steel against corrosion
- → adequate fire resistance

On the other hand, as dictated by Eurocode 2, a reinforced concrete section with reinforcement less than the minimum required reinforcement is to be considered an unreinforced concrete section (European Commission, 2006).

The requirements and verifications for slabs in this chapter were carried out considering the slab as a beam with a width of 1 m.

---

[17] The calculations that follow are based on the European Codes and Italian Annexes. The have been translated into a calculation spread sheet, see Appendix 7 – Specifications of Reinforced Concrete Structures Excel Sheet.

3.5.11.1. Cover

The nominal concrete cover is given by the minimum value of the concrete cover including a design tolerance margin for any deviations of 10 mm. This value will determine the minimum thickness of the concrete cover in the slabs of the structure.

$$c_{nom} = c_{min} + \Delta c_{dev} \tag{42}$$

In which:

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

$c_{min,b} = \Phi_{\text{long}} = 14\ mm$	Minimum cover requirements in relation to steel bond
$c_{min,dur} = 15\ mm$	Values of minimum cover requirements in relation to durability for reinforcing steel in accordance with EN 1008
Class XC1	
Ordinary conditions	
$\Delta c_{dur,\gamma} = 0\ mm$	As recommended by the norms
$\Delta c_{dur,st} = 0\ mm$	As recommended by the norms
$\Delta c_{dur,add} = 0\ mm$	As recommended by the norms

Therefore

$$c_{min} = \max\{14\ mm; 15\ mm; 10\ mm\} = 15\ mm$$

$$c_{nom} = 15 + 10 = 25\ mm$$

3.5.11.2. Reinforcement

It is important that the designed reinforcement is within the limits determined in this paragraph and has a diameter of no less than 12 mm.[18]

The minimum value of the reinforcement area is given by the following expression (43).

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} * b_t * d > 0.0013 * b_t * d \tag{43}$$

In which:

f_{ctm}	is the average tensile strength of concrete
f_{yk}	is the yielding stress of the steel
b_t	is the section width
$d = h - c$	is the effective height, thus the total height without the concrete cover

Therefore

$$A_{s,min} = 0.26 * \frac{2.94}{450} * 1000 * (320 - 30) = 492.95\ mm^2 > 0.0013 * 1000 * 290 = 377\ mm^2 \therefore \boldsymbol{ok}$$

Conversely, the maximum value of the reinforcement area is calculated as follows (formula (44)).

[18] As per the Italian Annexes, NTC 2018 (Redazione DEI, 2019).

This value must not be exceed as safety would be compromised.

$$A_{s,max} = 0.04A_c \tag{44}$$

In which:

$$A_c = b_t * d = 1000 * 320 = 320000\ mm^2$$

Therefore

$$A_{s,max} = 0.04 * 320000 = 12800\ mm^2$$

3.5.11.3. Summary

The design values for the two floors used in the finite element model can be seen in Table 30.

Table 30: Limitations and values for covers and reinforcement

Selection	Cover [mm]	Bar diameter [mm]	Minimum reinforcement [cm²/m]
Group element	**Cover (shear)**	**Shear reinforcement**	**Shear reinforcement**
	Cover (long.)	**Longitudinal reinforcement**	**Longitudinal reinforcement**
Roof slab	35.0 - 45.0	12	4.93
	35.0 - 45.0	12	4.93
First floor slab	35.0 - 45.0	12	4.93
	35.0 - 45.0	12	4.93

3.5.12. Loaded Conditions

3.5.12.1. ULS Load Combination

The load combination at the ultimate limit state is given by the following load combination.

$$E_d = E\left\{\sum_{j\geq1}\gamma_{G,j} * G_{k,j} + \gamma_P * P_k + \gamma_{Q,1} * Q_{k,1} + \sum_{i>1}\gamma_{Q,i} * \psi_{0,i} * Q_{k,i}\right\} \tag{45}$$

Shear Forces

As it can be seen in Figure 38, shear forces are mainly located around the septa. This is already an indication of a zone that should not be lightened. In fact, a lightened slab presents lower shear resistance compared to a full concrete slab. A more detailed explanation of this concept will be provided later in this graduation research paper (see chapter 3.6).

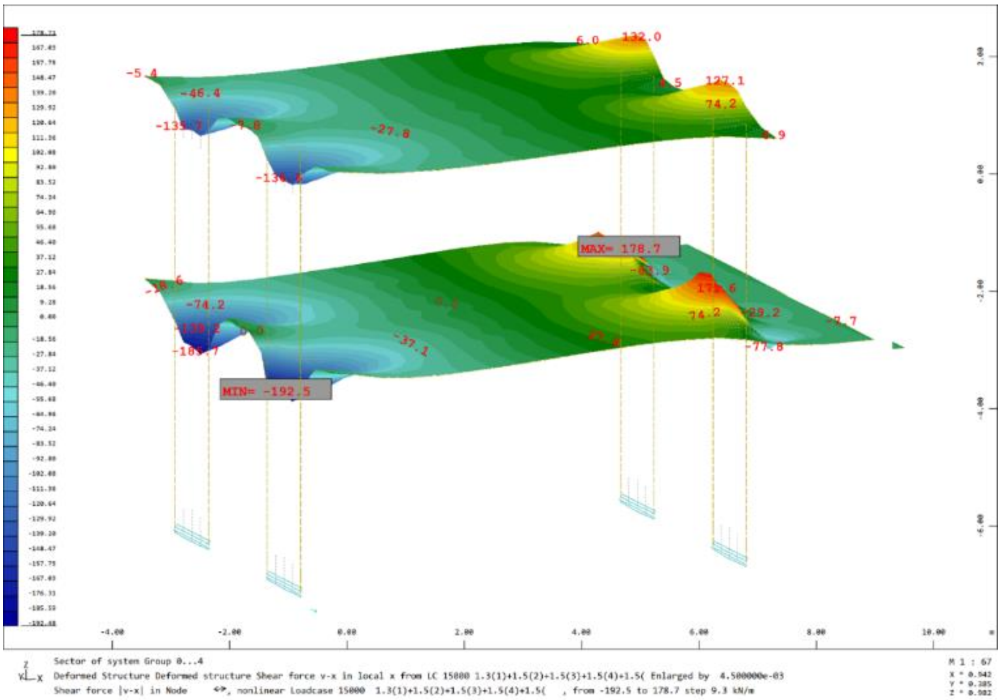


Figure 38: Shear Forces

Bending Moments

From Figure 39 it can be stated that the floors behave as a one-way type of slab; in fact, bending occurs in only one way, namely spanning in the shorter direction between the supported edges.

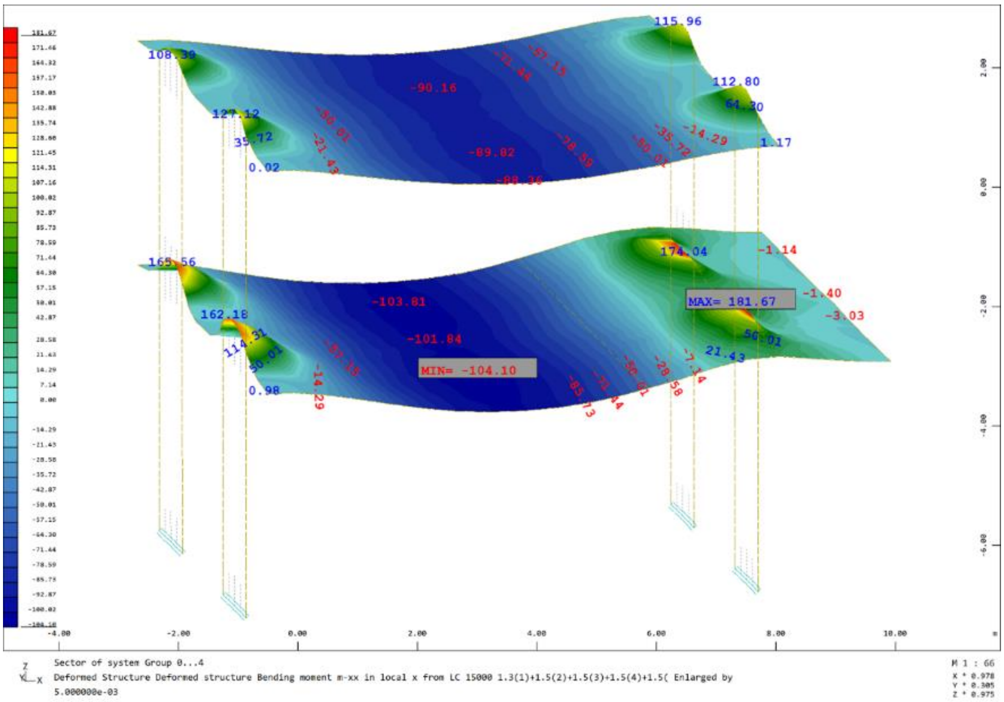


Figure 39: Bending moments along x direction

On the other hand, there is little to no bending moment in the longer span, as shown in Figure 40.

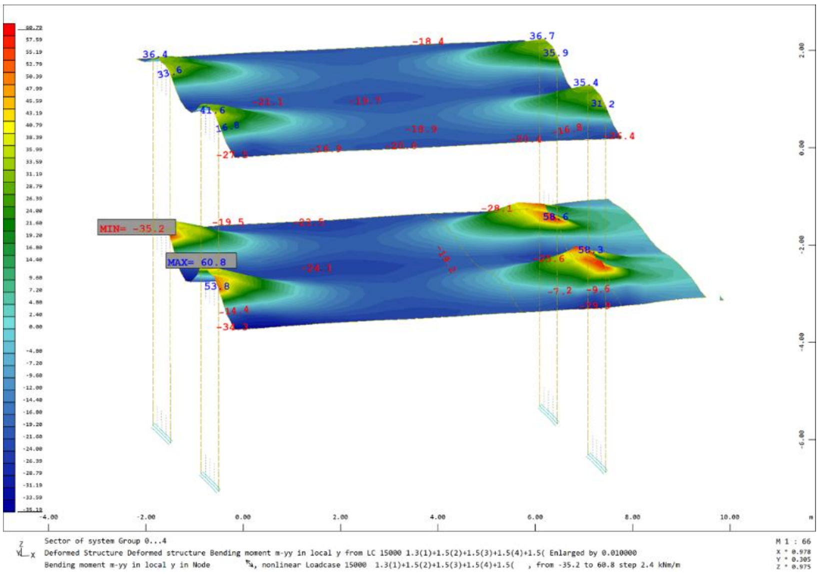


Figure 40: Bending moments along y direction

It is expected that bending reinforcement should be placed in the middle section of both slabs (see the dark blue areas in Figure 40), as the maximum bending moment reaches a value of 104.3 kNm approximately. However, this will be further discussed in the coming sections.

Support Reactions

The support reactions are important to mention, as these determine the bearing capacity of the foundations and it is expected that the value will decrease with a relived slab.

The support reactions are derived from the normal forces of the bearing walls as depicted in Figure 41. As it can be seen the FEM model responded as expected and consequently has been modelled correctly. The reason for this is that the normal forces increase as the proceed towards the ground level; this accounts for the weight of the two slabs being combined.

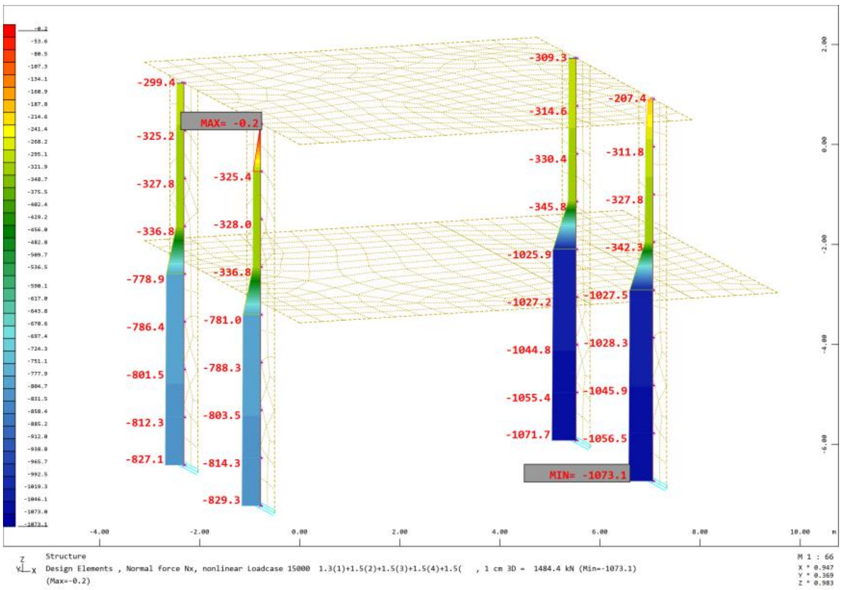


Figure 41: Support Reactions

In conclusion the reaction forces at the middle point of the walls reach a maximum value of 1074 kN approximately.

Reinforcement

The reinforcement analysed in the FEM software relates to four layers, namely two principal reinforcements and two cross reinforcements (see Figure 42).

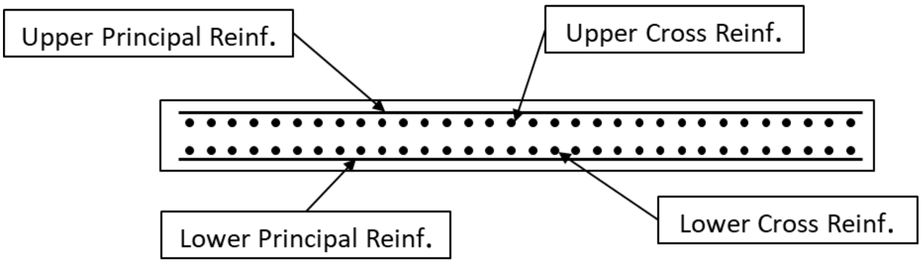


Figure 42: Cross-sectional view of generic slab

The four types of reinforcement serve different purposes as it can be seen in Table 31.

Table 31: Purposes of reinforcement

Type of reinforcement	Purpose	Additional description
Upper principal reinforcement	to aid resisting negative moment in the main direction (direction of bending moment in longest span)	Above the bearing walls negative moment may reach high values, requiring additional reinforcement
Lower principal reinforcement	to aid resisting positive moment in the main direction	In the central zone of the slab positive moment may create excessive vertical deflections, requiring reinforcement to avoid collapse
Upper cross reinforcement	to aid resisting negative moment in the perpendicular direction (shorter span)	In presence of high negative moments in the shorter spans, e.g., above the walls, reinforcement may be needed
Lower cross reinforcement	to aid resisting positive moment in the perpendicular direction	This type of reinforcement is expected to be unnecessary, as it helps resist high positive moments in the perpendicular directions, which, being a one-way slab, should be almost zero

In general, the required reinforcement is calculated as follows.

$$A_s = \frac{M_{Ed}}{0.9 * f_{yd} * d} \tag{46}$$

In which:

M_{Ed} is the absolute maximum value of the bending moment according to situation

f_{yd} is the design tensile strength being for B 450C type of steel equal to 391.3 MPa

d is the effective height being 290 mm

Based on the bending moments derived with Sofitsik, the following steel reinforcement values can be calculated with the aid of formula (46) (see Table 32). To note that moments in the x direction will be considered for the cross reinforcement, whereas moments parallel to the y direction are used to determine the principal reinforcement.

Table 32: Moments in Slabs and respective expected steel area for reinforcement

		Negative (for upper reinf.)	A_s [mm²/m]	Positive (for lower reinf.)	A_s [mm²/m]
Roof	**Mx [kNm]**	115.7	1132.9	89.8	879.3
1st floor		173.2	1695.9	104.1	1019.3
Roof	**My [kNm]**	41.5	406.3	19.3	189
1st floor		60.8	595.3	22.1	216.4

The steel areas calculated above neglect the minimum reinforcement limit (see chapter 3.5.11.2), having a value of 493 mm²/m.

Implementing this adjustment to the previously calculated steel areas, the following values can be expected (refer to Table 33).

Table 33: Expected Steel Area Values

		Upper Reinf. [mm²/m]	Upper Reinf. [cm²/m]	Lower Reinf. [mm²/m]	Lower Reinf. [cm²/m]
Roof	**Longitudinal reinforcement**	1132.9	11.3	879.3	8.8
1st floor		1695.9	17	1019.3	10.2
Roof	**Cross reinforcement**	406.3	–	–	–
1st floor		595.3	6	–	–

As declared in previous paragraphs, the model of this structure will behave substantially as a monodirectional slab. For this reason, low moments have been determined in the y direction, thus a lower amount of reinforcement is required in the upper section and no reinforcement in the lower zone of the slab.

Note that the moments displayed in Table 33 are based on the ultimate limit state combination of actions, whereas SOFiSTiK performs the checks iterating each case and determining the most optimal and efficient one. For this reason, the results that are described in the following paragraphs – being the required reinforcement bars calculated with the FEM software – will vary slightly from the ones calculated above.

Upper Principal Reinforcement

The upper principal reinforcement will span from one septum to the other one, following parallelly the x-axis in the system of the structure in this report and will be positioned as shown in Figure 43.

It is logical to state that, since the reinforcement is determined as a function of the acting moment, armour is only required in the vicinity of the load-bearing walls. The values obtained with SOFiSTiK differ slightly from those calculated previously; however, the difference is minimal as to confirm the correctness of the model.

The necessary reinforcement for the upper slab ranges from 9 cm^2/m to 10.8 cm^2/m.

Recalling that the area of steel reinforcement is calculated following formula (47), the characteristics for the reinforcement can be determined rearranging the equation.

$$A = \pi r^2 n \tag{47}$$

Note that in the company Aig Associati and Partner a spacing of 15 cm is preferred, thus the number of bars per meter (n) will have a value of 6.6, following the equation below (48).

$$n = \frac{1\,m}{spacing} = \frac{1}{0.15} = 6.6\ bars\ per\ meter \tag{48}$$

This aids at easily determining the bar diameter, as follows.

$$\Phi = 2r = 2\sqrt{\frac{A}{\pi n}} \tag{49}$$

Considering a steel area of 10.8 cm^2/m, the reinforcement will be ϕ14-15.

On the other hand, the reinforcement required for the first-floor slab varies between 17.5 cm^2/m and 17.7 cm^2/m. Hence, the reinforcement will be ϕ18-15 or ϕ20-15.

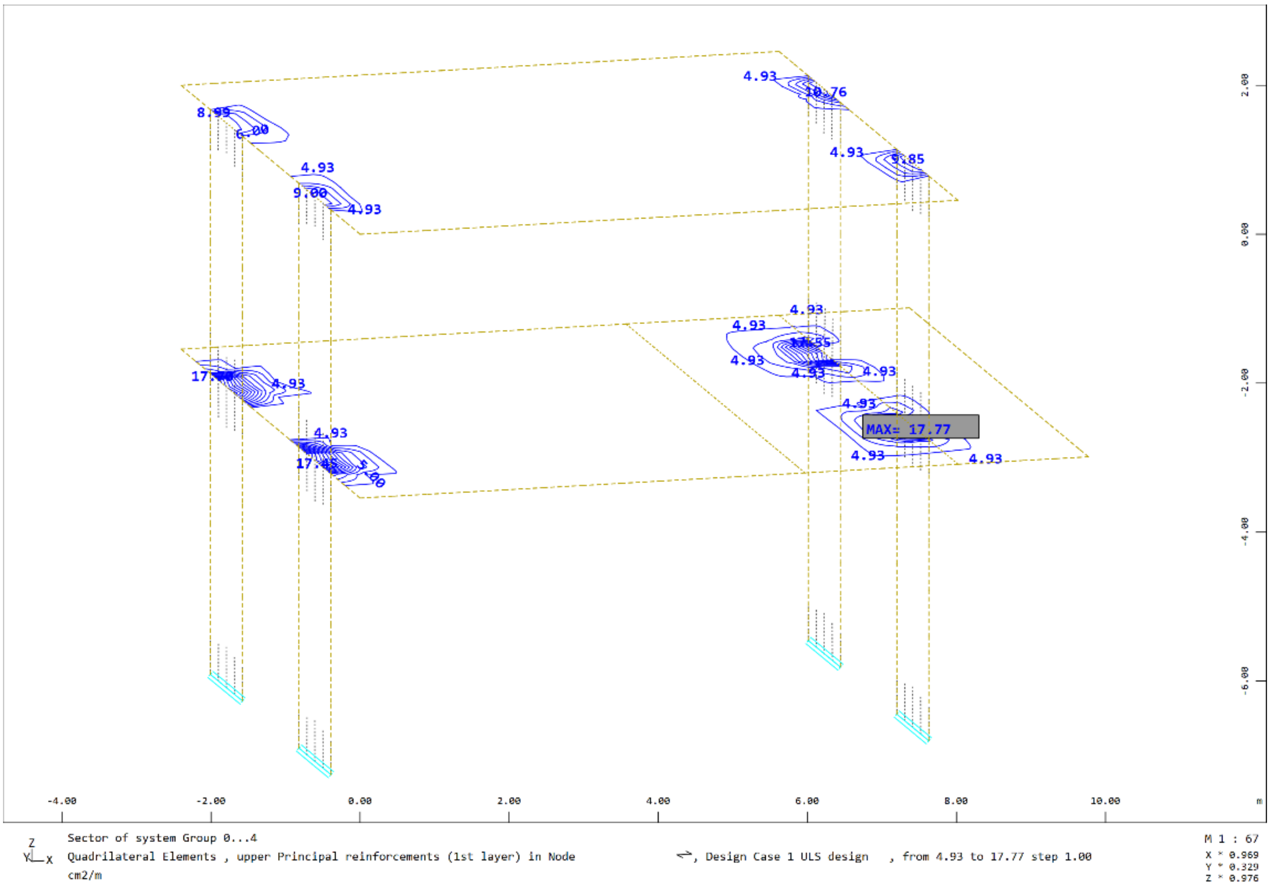


Figure 43: Upper Principal Reinforcement

Lower Principal Reinforcement

Following the same procedure as for the upper principal reinforcement, the required reinforcement bar dimensions can be established referring to SOFiSTiK's outcome (see Figure 44).

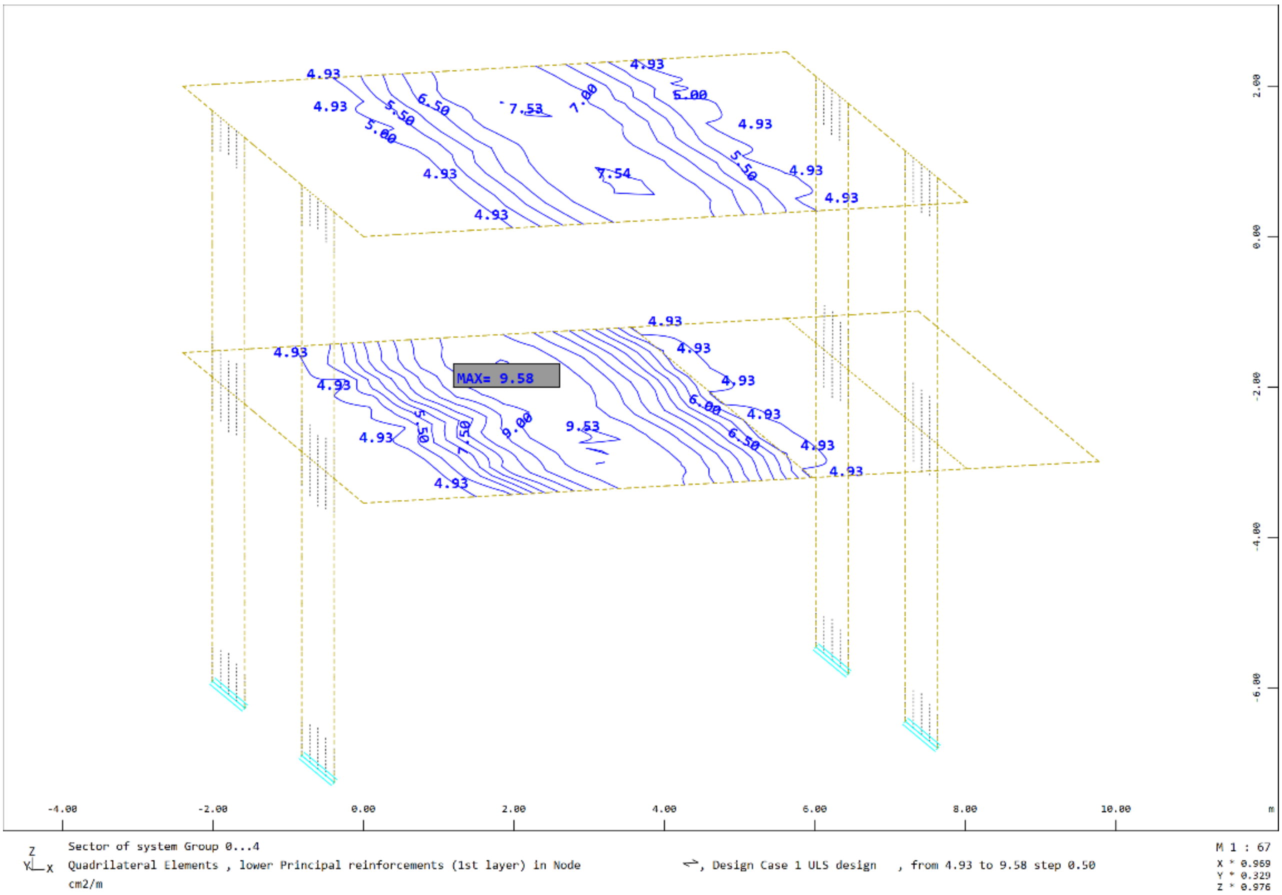


Figure 44: Lower principal reinforcement

As predicted in Table 33, the predicted values are close to the FEM software's outcome.

In fact, the roof slab requires reinforcement ranging from the minimum area of steel – 4.93 cm^2/m – to 7.54 cm^2/m, meaning that the reinforcement will be ϕ12-15.

On the other hand, the first-floor slab experiences a higher bending moment, hence the steel area ranges from 4.93 cm^2/m to 9.6 cm^2/m, resulting in ϕ14-15.

Upper Cross Reinforcement

The upper cross reinforcement is assumed perpendicular to the principal reinforcement, meaning that it spans in the y-direction and serves resisting bending moments along that same axis. Since the model represents a nearly monodirectional slab system, the reinforcement required will be minimum.

As depicted in Figure 45, the area of steel extrapolated ranges from 4.93 cm^2/m to 8 cm^2/m in the lower slab. On the other hand, the value shown on the roof floor, differing very little from the minimum reinforcement required, can be excluded.

This translates – using equation (49) – to a set of reinforcement of:

- → ϕ10-15 for the roof slab
- → ϕ12-15 for the first-floor slab

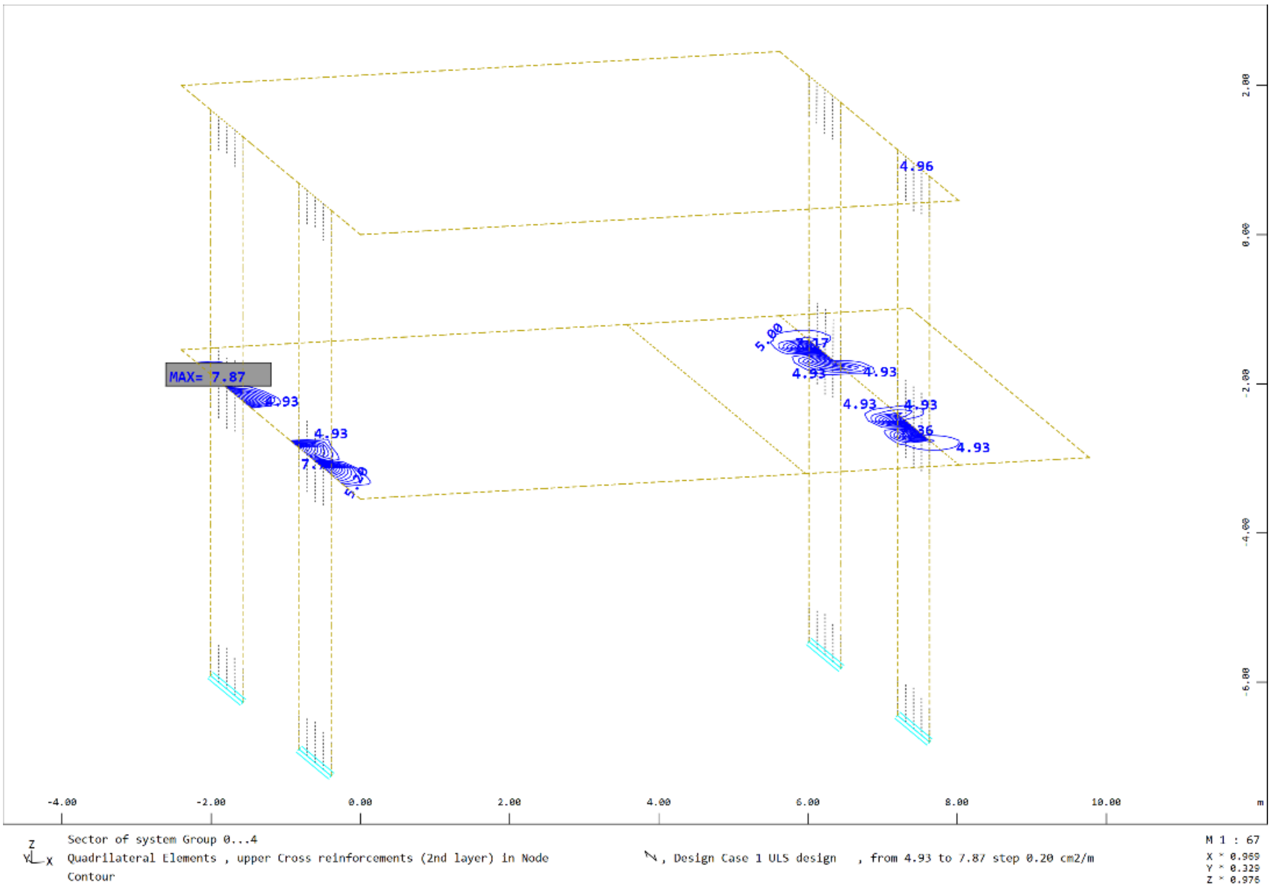


Figure 45: Upper cross reinforcement

The reinforcement dimensions for the roof slab, do not comply with the minimum reinforcement limits stipulated in chapter 3.5.5.2, hence a ϕ12-15 is preferred instead.

Lower Cross Reinforcement

Lastly, as predicted in Table 33 the lower cross reinforcement is not required as the bending moment shows low values, which unreinforced concrete is able to withstand.

3.5.12.2. SLS Load Combination

Three serviceability limit states will be analysed in the following paragraphs.

The combinations will have different load configurations (expressed by the formulae below (50),(51) and (52)), however, the partial and combination coefficients will remain the same for the three situations. the values are shown in Table 34.

Table 34: Partial and combination factors – SLS

Act	Part	γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2			Designation
	LC							Fact	Type	
G_1	G	1.00	1.00	1.00	1.00	1.00	1.00			Structural Permanent
	1							1.00	PERC	Self-weight
G_2	G	1.00	0.80	1.00	1.00	1.00	1.00			Non-structural permanent
	2							1.00	PERC	non-structural permanent
Q_A	Q	1.00	0.00	1.00	0.70	0.50	0.30			Residential environments
	3							1.00	COND	variable residential
	6							1.00	COND	variable balcony
Q_H	Q	1.00	0.00	1.00	0.00	0.00	0.00			Covers, maintenance only
	4							1.00	COND	variable roof
S	Q	1.00	0.00	1.00	0.50	0.20	0.00			Snow actions
	5							1.00	COND	Snow

Frequent Combination

The load combination at the frequent serviceability limit state is given by the following load combination.

$$E_{d,freq.} = E\left\{\sum_{j\geq 1} G_{k,j} + P_k + \psi_{1,1} * Q_{k,1} + \sum_{i>1} \psi_{2,i} * Q_{k,i}\right\} \tag{50}$$

The frequent combination of loads is used to check one of the crack states of the concrete, assuming that the loads occur on a frequent basis.

Long-Term Combination

The load combination in the quasi-permanent or long-term serviceability limit state is given by the following load combination.

$$E_{d,perm.} = E\left\{\sum_{j\geq 1} G_{k,j} + P_k + \sum_{i>1} \psi_{2,i} * Q_{k,i}\right\} \tag{51}$$

The second cracking state of the concrete is checked using the long-term combination of loads, which assumes that the loads are quasi-permanent on the structure.

Additionally, vertical deformations are verified using this combination as well as the tensions and stresses in the concrete.

Characteristic Combination

The load combination at the rare (or characteristic) limit state is given by the following load combination.

$$E_{d,rare} = E\left\{\sum_{j\geq 1} G_{k,j} + P_k + Q_{k,1} + \sum_{i>1} \psi_{0,i} * Q_{k,i}\right\} \tag{52}$$

Lastly, the characteristic or so-called rare combination is used to assess a second state of the concrete's tensions and stresses.

Crack Control

As already mentioned in chapter 3.5.7.2, it is of high importance that the cracks do not exceed the following values in the different load cases:

- → In frequent combination of actions *≤ 0.40 mm*
- → In quasi-permanent combination of actions *≤ 0.30 mm*

In Figure 46, the coloured area represents the area of slabs that are predicted to crack, with their width expressed in millimetres.

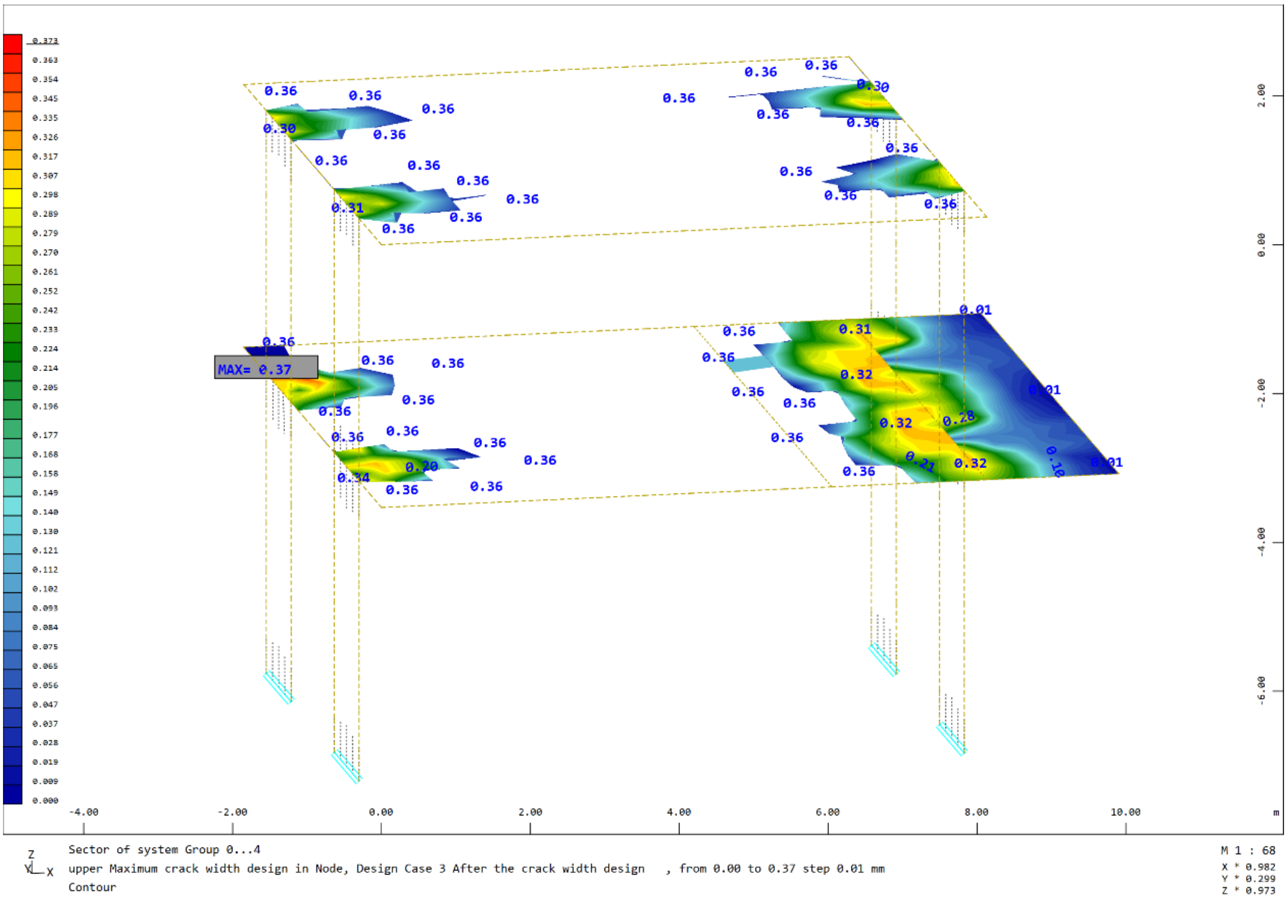


Figure 46: Expected cracks in frequent SLS

As it can be noticed, in frequent combination of the actions the maximum crack width calculated in

SOFiSTiK has a value of 0.37 mm, meaning that safety is reached, as the value if lower than 0.40 mm.

On the other hands, as shown in Figure 47 the cracks occur on a smaller surface area compared to the frequent combination of actions.

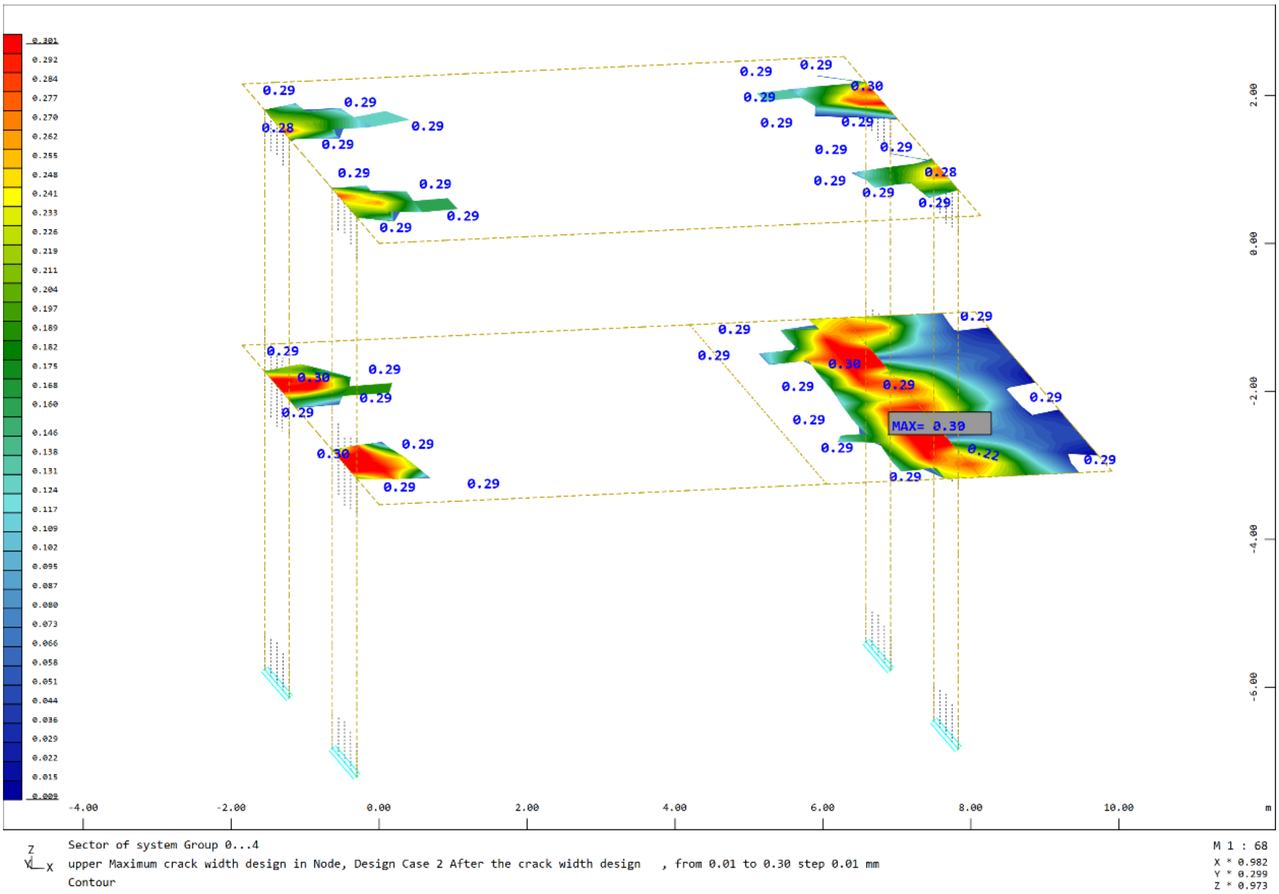


Figure 47: Expected cracks in quasi-permanent SLS

Lastly, their width reaches a maximum of 0.30 mm, which is taken as within limits, hence safety is reached.

Concrete Tension Checks

As previously mentioned in chapter 3.5.7.2, tensions in concrete must be checked and it must be made sure that these do not exceed the following values:

- For the characteristic SLS combination $\sigma_{c,max} \leq 0.60\, f_{ck} = 0.6 * 30.71 = 18.4\, MPa$
- For the long-term SLS combination $\sigma_{c,max} \leq 0.45\, f_{ck} = 0.45 * 30.71 = 13.8\, MPa$

Since compressive strength is taken into consideration it is expected to detect peak values in proximity of the supports. In fact, the slab of concrete will be placed between the upper and lower septa, experiencing their load in compression.

Moreover, many FEM software – including SOFiSTiK – are designed in a way as to portray the most accurate results. This simulation is used to perform a structural analysis on how a particular component or design would react to stresses in the real-world context.

The simulation breaks down the entire model into smaller elements within a mesh, which is used to verify how different elements of a design interact and behave in the presence of simulated load inputs. This translates into results being showed per smaller areas – reaching the size of a point[19] – rather than per area. Thus, result interpretation is of high importance in structural engineering.

In this case the stresses are obtained by looking at the point load (F)[20] per mesh area (A), as shown in formula (53).

$$\sigma = \frac{F}{A} \tag{53}$$

The stresses experienced in the concrete slab in characteristic of rare SLS combination can be seen in Figure 48.

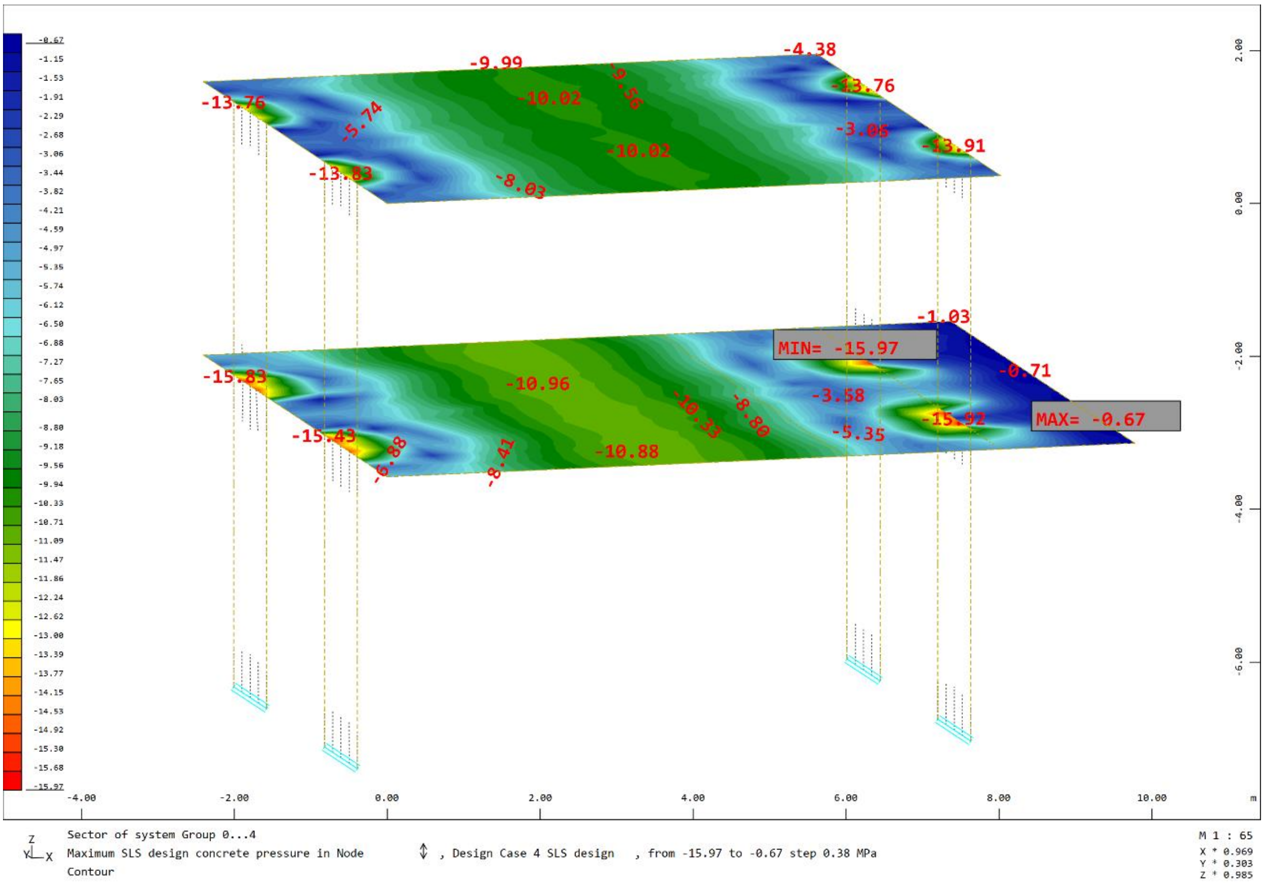


Figure 48: Compressive Stresses in rare SLS

As represented in the graphic above, the prediction resulted correct: the maximum compressive strengths are located in proximity of the septa, with a maximum value of approximately 16 MPa. This respects the limits as the value is less than 60% of the compressive strength of the C30/37 concrete (18.4 MPa).

On the other hand, the maximum compressive strength in quasi-permanent SLS combination of loads (see Figure 49) does not comply with the limits set being of 13.8 MPa. As a matter of fact, the maximum compression stress experienced reaches the value of 14.2 MPa.

[19] Note that the size of the mesh can be altered by the engineer/designer.

[20] According to the equivalent point load per SLS case.

This situation occurs for two reasons:

a) Loads usually never act alone; on the contrary, several loads act simultaneously and load combinations take this event into account.
The quasi-permanent combination of loads considers loads and their effect on the long term. This means that snow is not taken into account and the accidental load in the residential zones account for only 30% of their full load.
On the other hand, the rare combination takes in consideration 70% of the variable loads in residential zones and 50% of snow load.
b) As mentioned previously the maximum compressive tension in concrete is found in one specific point, which does not mean that this stress state will be experienced in the whole section of the slab.

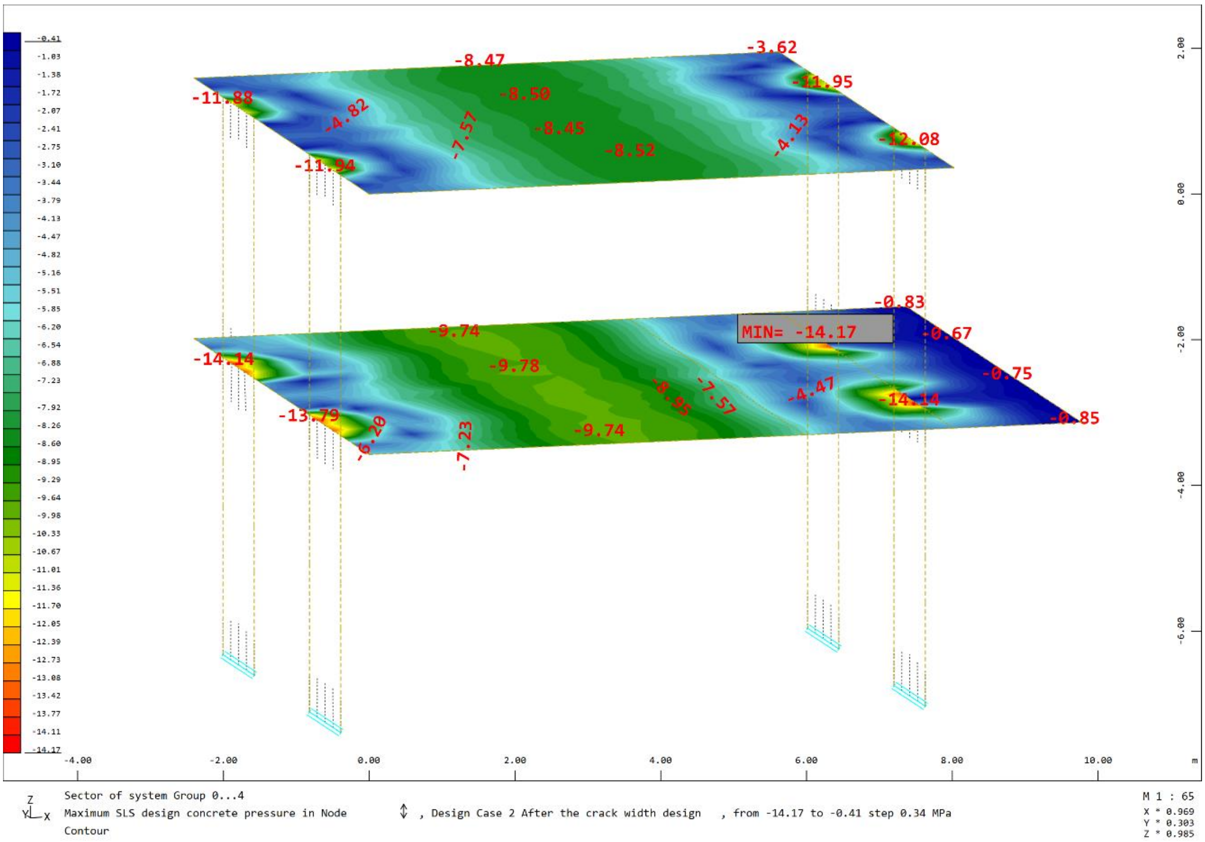


Figure 49: Compressive Stresses in long-term SLS

The failure of the tension check in the quasi-permanent SLS state is given by the heavy load on the structure. Hence by adopting lightening formwork as designed, this issue should be overcome. Nonetheless, checks will be performed for each approach.

Steel Tension Checks

According to chapter 3.5.7.2, the maximum steel tension must not exceed the following value:

$$\sigma_{s,max} \leq 0.80\, f_{yk} = 0.8 * 450 = 360\, MPa \tag{54}$$

Furthermore, the check is performed using the serviceability limit state, rather than the ultimate limit state, since the load is not constant. In fact, the magnitude and direction of the load may vary regularly or irregularly. Such loads are known as fluctuating loads and can cause fatigue failure to occur at load values far below the ultimate load combination.

With the aid of the FEM software, two scenarios have been analysed, one maximizing the load combinations on the roof slab – in this case – and the second one those on the first-floor slab.[21] The outcome is shown respectively on the left and right of Figure 50.

The maximisation of the loads on the roof slab result in a steel tension equal to 307 MPa, whereas the second scenario, in which the combination of loads results in a concentration of the loads on the first-floor slab, shows a maximum steel stress of 274.2 MPa.

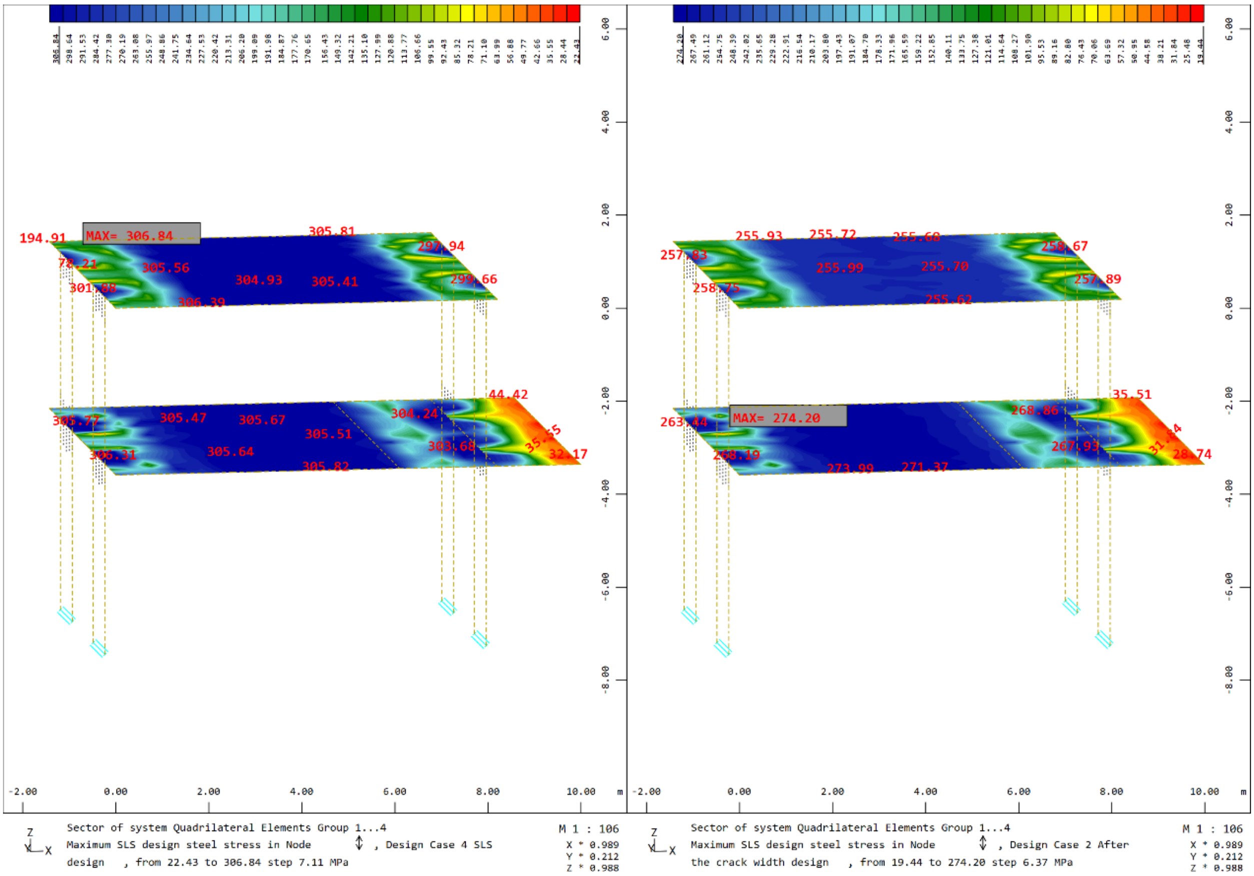


Figure 50: Steel Tensions

In both cases, the maximum allowable stress of 360 MPa is not exceeded, meaning that safety is established.

Deformation Checks

As previously mentioned in chapter 3.5.7.2, the deformation check will be performed considering the quasi-permanent – or long-term – SLS load combination.

As no adjacent part of the structure are present in the structural model considered in this chapter, the deformation will be only checked considering the value of *span/250*.

[21] Note that the optimization in other models might not show the same outcomes, in the sense that some optimizations might show two critical load combinations, both on the same slab.

The span of the balcony reaches 1.8 m, thus the maximum allowable deflection is equal to 7.2 mm.

$$\frac{1800\ mm}{250} = 7.2\ mm$$

Whereas, for the slabs the maximum deflection is equal to 33.2 mm.

$$\frac{8300\ mm}{250} = 33.2\ mm$$

As depicted in Figure 51, a deflection upwards of 1.1 mm approximately can be expected in the balcony section of the first-floor slab; on the other hand, a maximum downwards deflection of 4.6 mm is likely to occur in the first-floor slab’s central zone.

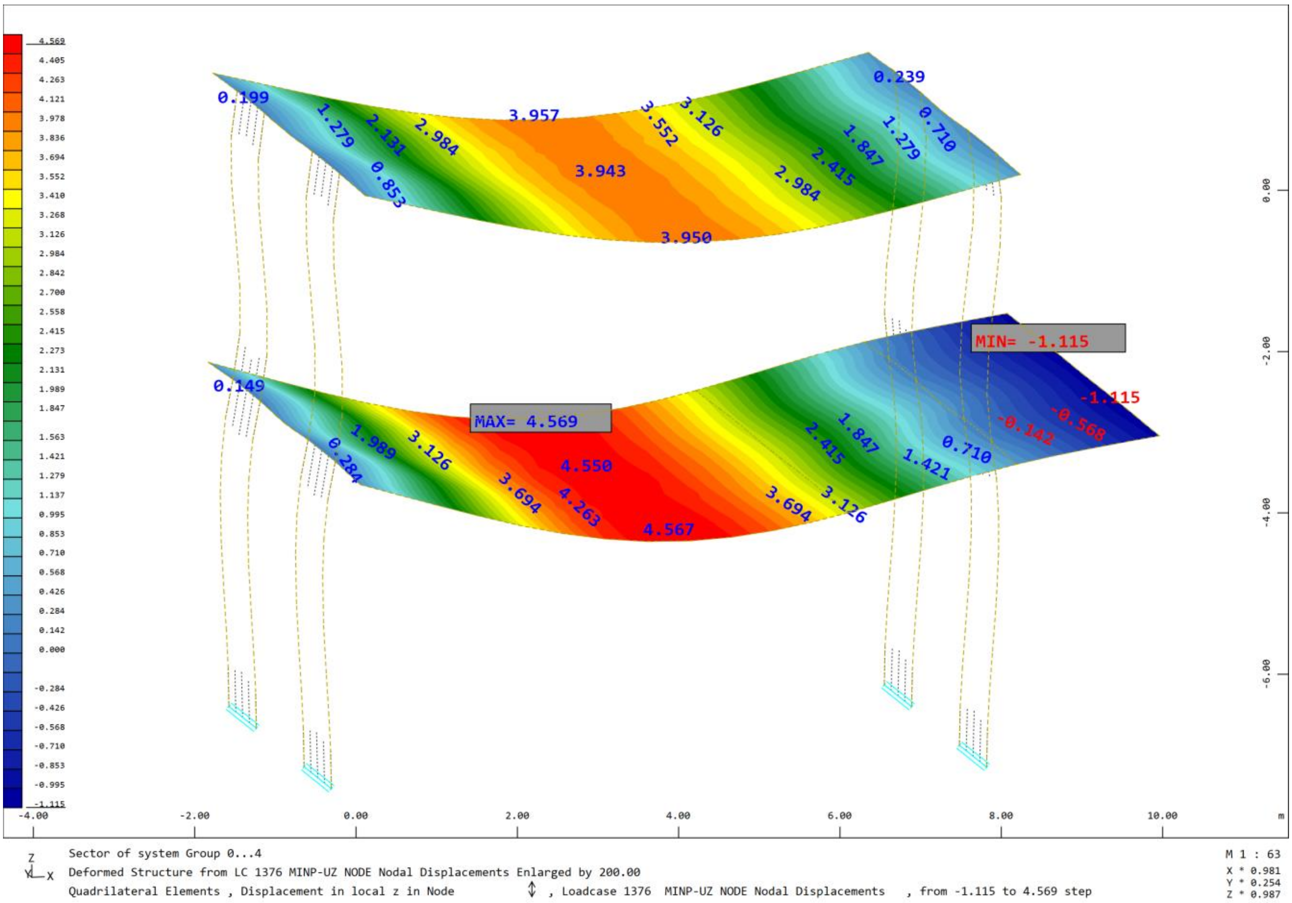


Figure 51: Vertical deflections

Both these values are within the limits imposed; hence, the structure’s predicted deflections are allowable and no further action needs to be taken.

3.5.13. Conclusion

It can be concluded that the portion of the structure analysed in this chapter complies with all the requirements, apart from the concrete’s allowable compression tension. Nonetheless, the analysis of the slabs without lightening formwork has been performed to compare the results with the lightened structure. This will aid at determining the benefits and drawbacks of a relived slab.

3.6. Determination of Sections to be Lightened

In this section, an evaluation of the positioning of the lightweighting systems in the slabs will be presented. This will determine the portions of the slab whose characteristics will be adapted according to the FEM modelling methods provided by Geoplast.

For safety reasons, it is optimal to design full slab sections in the slab support areas, thus in the presence of load-bearing septa or pillars. In these areas, in fact, the shear forces will be greater than in the central sections of the slab. It is therefore advisable to provide the maximum shear resistance understandably given by a solid slab section.

The assessment will be made by establishing the shear resistance without cross reinforcement, given solely by the characteristics of the concrete and longitudinal reinforcement, of a lightened slab. Therefore, the reinforced concrete section to be considered will be the solid area around two formwork elements.

3.6.1. Reinforced Concrete Section

The section to be studied varies according to the lightening system chosen; in this case the pre-set block is the 'New Nautilus Evo H16 Single' produced by Geoplast. The dimensions of this lightening system are 16 cm in height and 52 cm in plan in both directions. In addition, the pre-determined rib, that is, the spacing between blocks, is 140 mm.

These figures establish the basis of the full section to be analysed according to the formula (55).

$$B = N + b = (140 + 520)\, mm = 660\, mm \tag{55}$$

Regarding the thickness of the slab and thus of the two reinforced concrete wings, the predetermination is given by the structural type and the floor span. As the floor slabs in the 'Casa Haus inge' project are on columns with a span of between 8 and 9 metres, the following formula was adopted (56).

$$H = \frac{L}{25} = \frac{8\, m}{25} = 0.32\, m \tag{56}$$

The lower and upper spacings are chosen symmetrically or by placing the larger spacing at the bottom. The choice of placing the largest spacing in the lower part of the section is given by fire safety considerations (Geoplast S.p.A. , 2019).

In this case the spacing will be 72 mm in the lower area and 60 mm in the upper area of the slab.

The above data leads to an H-section of reinforced concrete as shown in Figure 52.

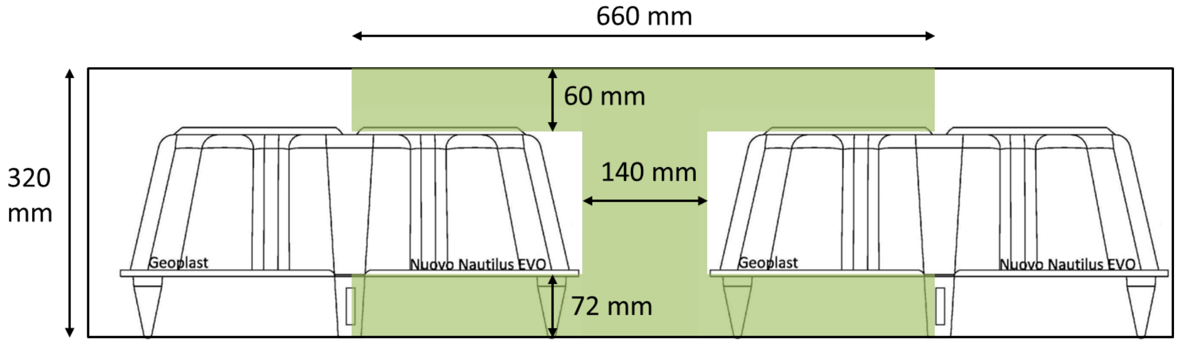


Figure 52: H-section of reinforced concrete

3.6.2. Shear Resistance

3.6.2.1. *Without Shear Reinforcement*

The calculation for the shear strength of the concrete H-section will be calculated according to the Italian standards (NTC 2018) considering the strength without shear reinforcement.

$$V_{Rd} = \max\begin{cases}\left[0{,}18 * k * \dfrac{(100 * \rho_l * f_{ck})^{\frac{1}{3}}}{\gamma_c} + 0{,}15 * \sigma_{cp}\right] * b_w * d \\ \left(v_{min} + 0{,}15 * \sigma_{cp}\right) * b_w * d\end{cases} \tag{57}$$

In which:

$$k = 1 + (200 + d)^{\frac{1}{2}} \leq 2$$

$$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}}$$

$$\rho_l = \frac{A_{sl}}{b_w * d} \leq 0{,}02$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} \leq 0{,}2 f_{cd}$$

d is the effective height of the section

b_w is the minimum width of the section

Note that in favour of safety, the average compressive stress in the section (σ_{cp}) will be zero, since the compressive force applied to the section will be excluded. In fact, the concrete section increases its shear strength when compressed.

The lightened section compared to the solid section has a smaller minimum width; in fact, the solid section has been considered with a width of 1 metre, on the other hand, given the geometry of the lightened section, the minimum width of it will be equal to the rib, therefore 14 cm.

The shear strength was calculated with the aid of a calculation sheet; below are the results.

$$V_{Rd} = \max\begin{cases}25.64\ kN \\ 19.50\ kN\end{cases} = 25.65\ kN$$

This resistance is valid for one H section of reinforced concrete; however, the verification will be carried out per linear metre.

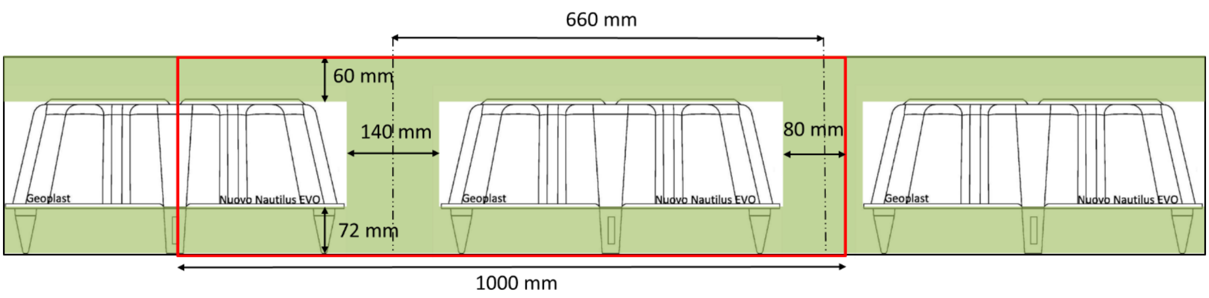


Figure 53: Ribs per 1 metre

To determine the number of beams in a metre, the spacing (S) between ribs is to be considered.

$$\frac{number\ of\ beams}{m} = \frac{1}{S} = \frac{1}{0.66} = 1.52 \tag{58}$$

The shear strength per linear metre is therefore given by 1.5 H-sections of concrete.

$$\frac{V_{Rd}}{m} = 25.65 * 1.5 = 38.5\ kN \tag{59}$$

3.6.2.2. With Shear Reinforcement

In order to optimise the width of the slab sections where the formwork can be placed, the shear strength of the H-section will also be established in the presence of shear reinforcement as shown Figure 54.

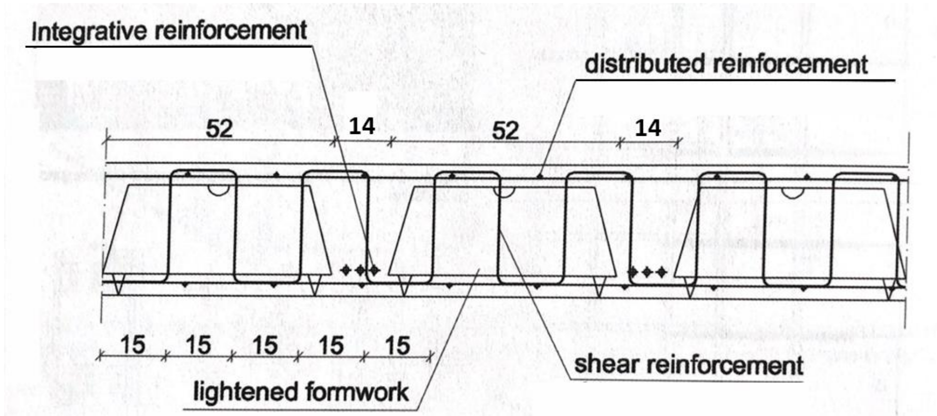


Figure 54: Positioning of shear reinforcement

The calculation for the shear strength of the concrete H-section will be calculated according to the Italian standards (NTC 2018) considering the shear reinforcement strength, as follows.

$$V_{Rd} = \min \begin{cases} 0{,}9 * d * \dfrac{A_{sw}}{s} * f_{yd} * (\cot\alpha + \cot\theta) * \sin\alpha \\ 0{,}9 * d * b_w * \alpha_c * \nu * f_{cd} * \dfrac{(\cot\alpha + \cot\theta)}{1 + \cot^2\theta} \end{cases} \tag{60}$$

In which:

$\alpha_c = 1$

$\nu = 0{,}5$

d is the effective height of the section

b_w is the minimum width of the section

A_{sw} is the area of the cross reinforcement

s is the spacing between shear reinforcement

α is the angle of inclination of the stirrups with respect to the floor axis

θ is the inclination of the concrete rafters with respect to the floor axis

As depicted in Figure 54, the cross reinforcement designed for the lightened slab will have a 'hat' shape with a diameter of 10 mm and a spacing of 10 cm. The choice of such reinforcement is due to their significant increase in strength (specimen 6,7 and 8 in this case) as can be seen in Figure 55 (Coronelli , Martinelli , & Foti , 2015).

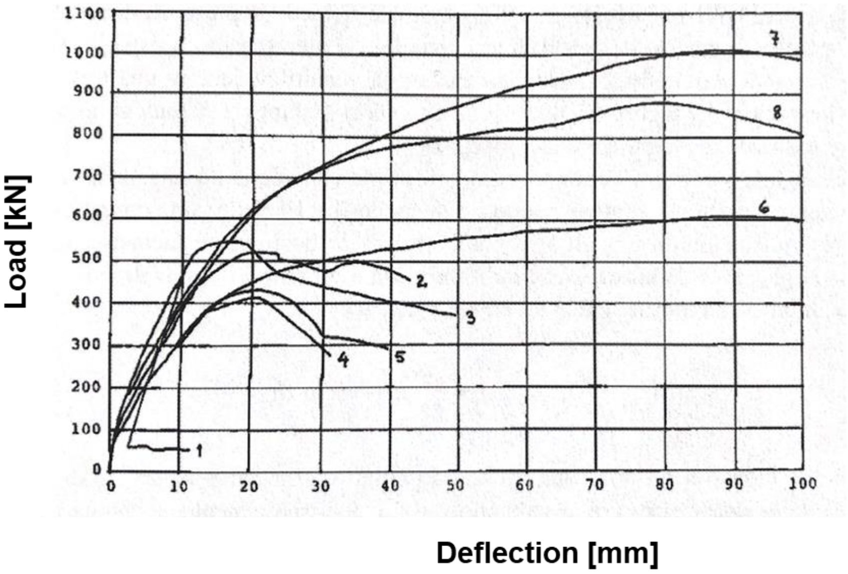


Figure 55: Study with 'hat' reinforcement

The graph in Figure 55 describes the deformation of a slab subjected to loads with 8 shear reinforced concrete specimens:

1. Near-immediate-break section
2. Reinforced 'hat' section with dense spacing – type 1
3. Tightly pitched 'hat'- reinforced section – type 2
4. Unreinforced section
5. Unreinforced section
6. Reinforced 'hat' section – type 1
7. Reinforced 'hat' section – type 2
8. Reinforced 'hat' section – type 3

Therefore, the shear resistance is equal to the following value. This was calculated with the aid of a calculation sheet.[22]

$$V_{Rd} = \min\begin{cases} 53.48\ kN \\ 158.97\ kN \end{cases} = 53.48\ kN$$

As performed for shear strength without cross reinforcement, the value will be expressed per linear metre.

$$\frac{V_{Rd}}{m} = 53.48 * 1.5 = 80.22\ kN$$

Given the two resistant values for shear forces, a study of the shear forces on the solid slab will be set out in the following paragraphs with the aim of determining the portion of the slab that can be lightened.

3.6.3. Shear Forces

The shear forces of the solid-section floor were obtained with the finite-element modelling programme 'SOFiSTiK'.

With reference to the previously calculated resistance values, three sections are identified:

[22] See Appendix 7 – Specifications of Reinforced Concrete Structures Excel Sheet.

1. Full section — from 80.2 kN to maximum shear force
2. Lightened section with shear reinforcement — from 53.48 kN to 80.2 kN
3. Lightened section without shear reinforcement — from minimum shear force to 53.48 kN

3.6.3.1. Full Section

As defined in the previous paragraph, the solid section will be positioned in the area of the floor slab with shear forces greater than 80.2 kN. These correspond to the sections shown below, on the left for the roof slab and on the right for the first-floor slab (see Figure 56).

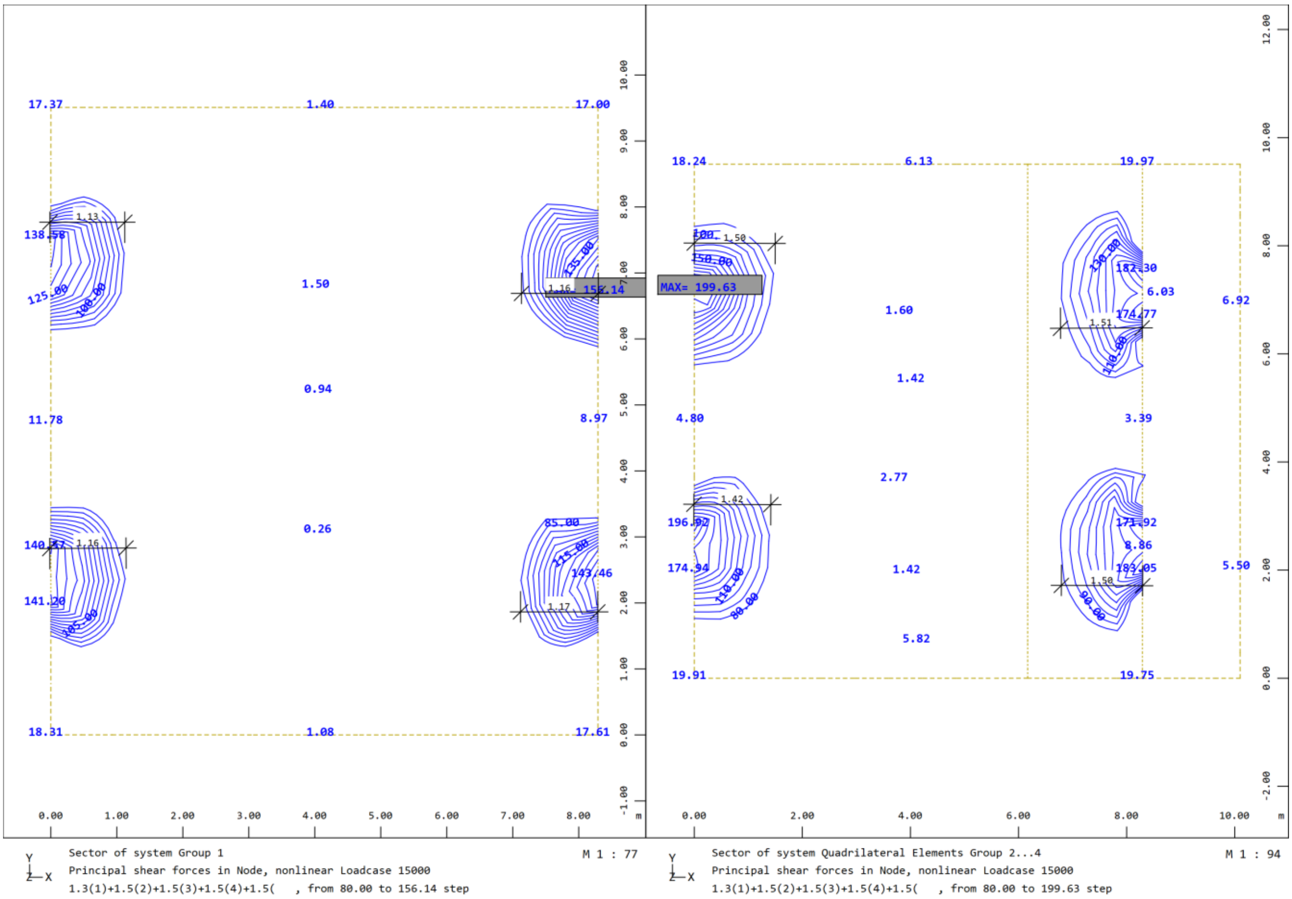


Figure 56: Full Section

For the roof, the solid section is required on both the right and left positioned on an area 1.20 metres wide from the outer edge.

On the other hand, for the first-floor slab, the solid section is required on the left positioned on both the left and right sides over an area 1.50 metres wide from the outer edge.

3.6.3.2. Lightened Section with Shear Reinforcement

The second floor section adjacent to the solid section will be the shear-reinforced lightened section, thus in the portion of the floor with shear forces between 80.2 kN and 53.48 kN (see Figure 57, with the roof on the left side of the picture and the first floor slab on the right).

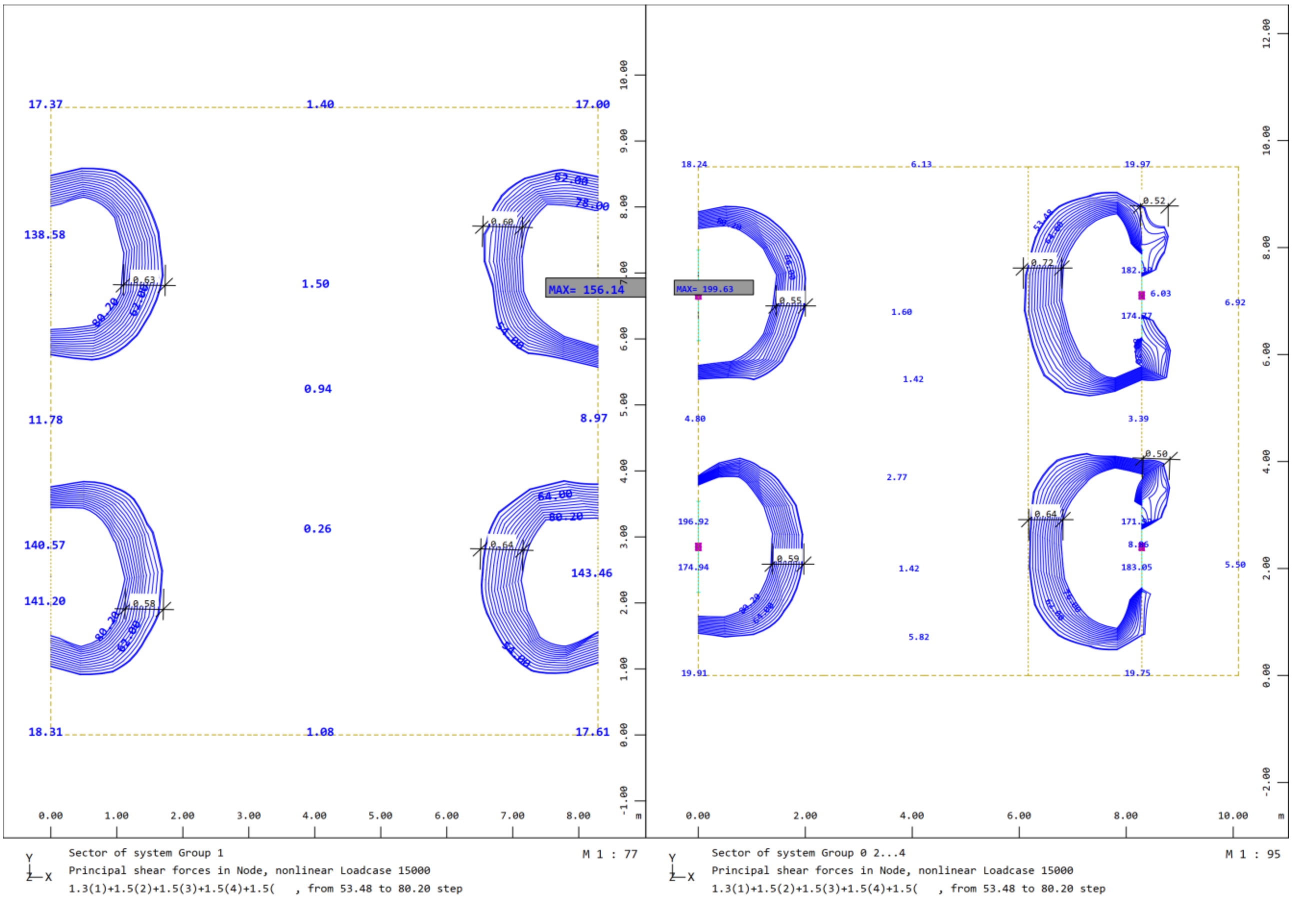


Figure 57: Lightened section with shear reinforcement

For the roof, the shear-reinforced lightened section is required on the left positioned over an area 0.63 metres wide adjacent to the solid section, and on the right over an area 0.64 metres wide from the solid section.

For the first-floor slab, the shear-reinforced lightened section is required on the left positioned on an area 0.60 metres wide from the solid section; on the inside on an area 0.72 metres wide adjacent to the solid section, and on the balcony on an area 0.52 metres wide adjacent to the solid section.

3.6.3.3. Lightened Section without Shear Reinforcement

The rest of the slab portion will be designed with a lightened concrete section without the need for shear reinforcement.

3.6.4. Results

3.6.4.1. Optimization

The outer balcony section of the first-floor slab will be designed entirely solid due to its reduced width (1.80 metres). This decision will result in easier and therefore quicker installation.

Furthermore, in order to facilitate and optimise the design phase, symmetry and standardisation of dimensions is preferred.

Thus, for the roof slab, the solid section will have a width of 1.20 metres from both outer edges, while the lightened shear-reinforced section will have a width of 0.65 metres.

For the first-floor slab, the solid section will have a width of 1.50 metres from both outer edges, while the shear-reinforced lightened section will have a width of 0.65 metres.

In some cases, the optimised dimensions do not respect the shear strength limits calculated and established in the previous paragraphs, however, the choice was made with the awareness of having to ascertain the results of the model of the lightened slab and to guarantee flexibility in the case of subsequent adaptations.

3.6.4.2. Roof Slab

As can be seen from Figure 58, the slab that will be used for the roof of the building will have a total 5.90 metres wide portion that is lightened, this is approximately 70% of the slab.

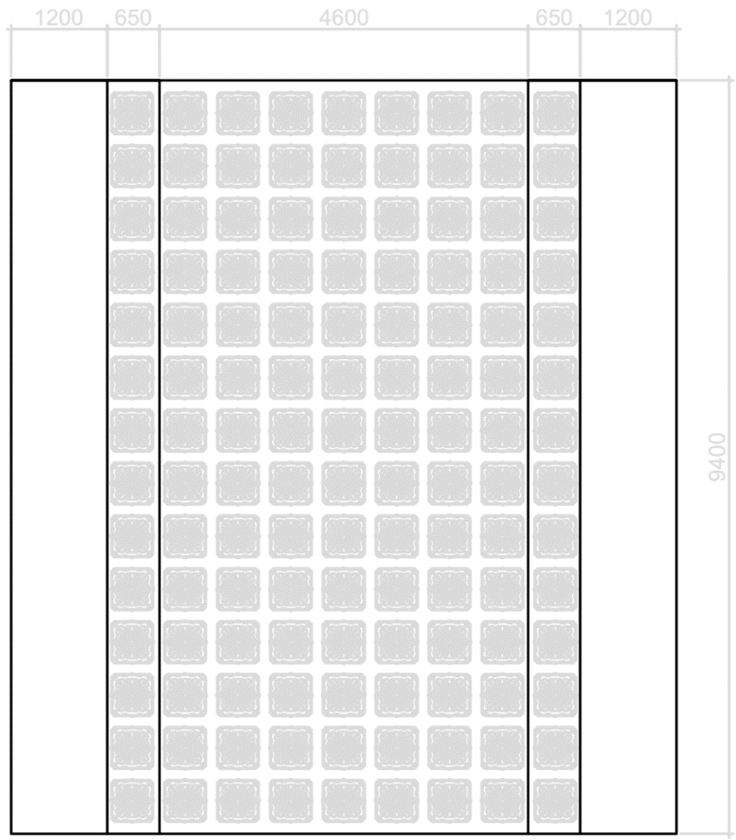


Figure 58: Top view - Roof slab

Figure 59: Longitudinal cross-section - Roof slab

3.6.4.3. First Floor Slab

On the other hand, the floor slab on the first level has higher loads due to the wear of the area, so the section that can be lightened will have a total width of 5.30 metres, therefore approximately 60% of the total slab.

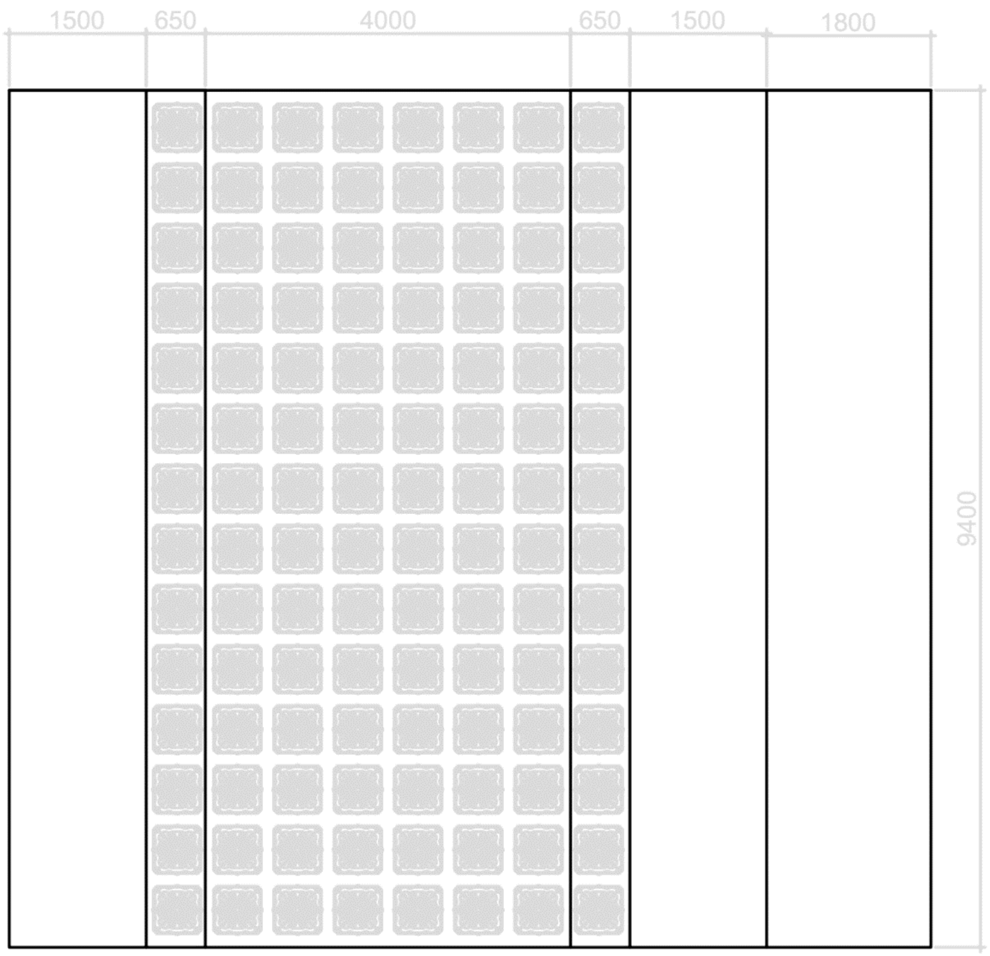


Figure 60: Longitudinal cross-section - First floor slab

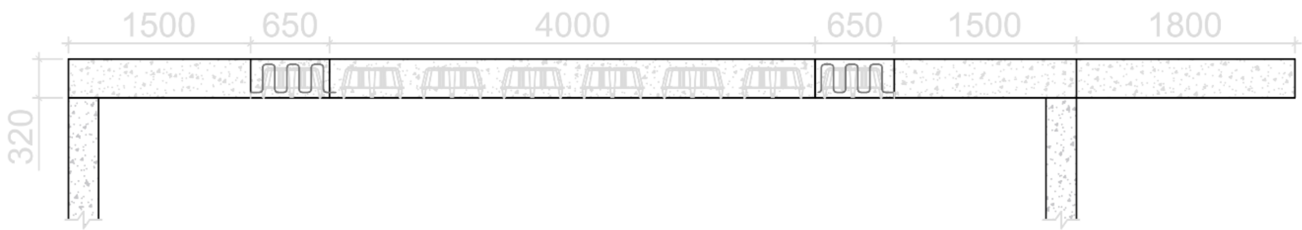


Figure 61: Top view - First floor slab

3.7. Proposed Alternatives

Based on the previous paragraphs describing the finite element method modelling approach (see 3.3), three main approaches have been identified.

The first two describe modelling the relieved slab with a reduced floor height and with a reduced flexural strength of the concrete.

The third approach consists of modelling the relived flat slab with two full upper slabs and a lightened middle section, with altered self-weight and elastic modulus. The material for the lightened slab used in the finite element modelling will therefore be a package of three materials, referred to in this report as 'sandwich' material.

3.8. Preliminary Overview of Multiple Criteria Analysis

The decision aiding method used in this document is a so-called 'Multiple Criteria Analysis' (MCA). This aids at ranking the different proposed solutions, clarifying the decision and recommends or favour a solution that fulfils all the objectives and stakeholder's desire.

3.8.1. MCA Method

The chosen method consists in assigning weights to the criteria, vial the 'Analytical Hierarchy Process' (AHP). The hierarchy is divided in different levels:

1. Level one — the goal of the project
2. Level two — the various criteria
3. Level three — the proposed alternatives

The three levels of the hierarchy aid at a deeper understanding regarding how the alternatives relate to the criteria and how these accomplish or not the goal of the project.

For this document the criteria are manipulated and compared to each other – two-by-two – based on (Saaty, 1988). This involves comparing the criteria against each other using a comparative scale rating.

To the criteria a score from 1 to 9 will be assigned, based on the difference between criteria levels. The following scale will be used.

Table 35: Criteria Scale

Score	Explanation
1	Equal importance
2	Slightly more important
3	More important
4	Moderately important
5	More important than moderately
6	Strongly important
7	More important than strongly
8	Very strongly important
9	Extremely important

3.8.2. Criteria Description

The analysis of a relived flat slab can result challenging when using FEM software. In fact, the option to input the relieving bodies is lacking, hence it is required to model the flat slab with an alternative approach. In this case, as mentioned previously the approaches are the following:

1. Modelling the slab with reduced flexural strength
2. Modelling the slab with a reduced thickness
3. Modelling the slab with a 'Sandwich' material

All the solutions provided decisive outcomes; nonetheless, their outcomes derive from different principles. For this reason, the following criteria have been chosen to compare all the solutions and determine the most feasible one.

3.8.2.1. Designing Time

When considering a new project, the cost of it has a major impact on both the client and the firm. This component of the project comes not only from the final cost of materials and construction but also from the preliminary and design phase. It is therefore clear that time is reflected in money.

For this reason, it is important to find a solution that requires little time as possible to model the lightened slab with the FEM software.

To assess this criterion designing times will be compared with the following assumptions:

a) The preliminary creation of the calculation spread sheet is not included in the count
b) The spread sheet is already open
c) A model in the FEM software is already created as to reduce the designing time
d) The SOFiSTiK interface is already open
e) The SOFiPLUS interface is to be opened
f) The preliminary preparation of the TEDDY text is not included in the count; however, the implementation of the six features (refer to chapter 3.9.3.2) is comprised
g) The TEDDY text interface is already open

The times will be recorded on a stopwatch and will be repeated three times per alternative, in order to increase accuracy and reliability of the results. Moreover, it is understandable that the first time performing the test will take longer compared to the last attempt, due to the fact that over time the approach will become familiar, hence faster. For this reason, the following sequence for the three attempts will be used (see Table 36).

Table 36: Sequence of Attempts

	Attempt 1	**Attempt 2**	**Attempt 3**
Sequence	Alternative 1 Alternative 2 Alternative 3	Alternative 2 Alternative 3 Alternative 1	Alternative 3 Alternative 1 Alternative 2

Note that to maintain reliability and consistency of the results, the scores have been obtained recording one person performing all the attempts. It is logical to state that different people performing the same 'test' might score lower or higher values – based on their knowledge in the matter and related skills – nonethelss, the general outcome is expected to stay unvaried.

3.8.2.2. Modelling and Recreation Ease

Since the structural analysis must be performed on the structure as a whole, the type of modelling approach plays an important role. Indeed, alongside the design time, the ease of both modelling and recreating the approach must be considered. This aspect is strongly related to the reduction of design time but refers to the convenience and ease of duplicating the approach for future references. As a matter of fact, the firm requires a solution that can be produced easily by other engineers.

3.8.2.3. Required Skills

Since the approaches are meant to be implemented in the company's working system, it is important to state if additional skills – excluding a basic understanding of both Autodesk and SOFiSTiK – are required. In fact, not only does this criterion related to modelling and recreation ease but to the amount of designing time as well.

All in all, if additional skills are necessary for the implementation of the approach, both designing time and recreation ease would be negatively affected.

3.8.2.4. Accuracy of the Model

In this case the criterion 'accuracy to the model' refers to what extent the model depicts the real-life structural element. This refers to its geometry (see Figure 62) as well as its behaviour, such as the tensional stress in the concrete.

Figure 62: Voided slab section

Even though it is expected that all three approaches present the same outcome regarding detailing in the construction phase, it is important to reflect as much as possible the real-life situation.

Moreover, a balance between time saving and correctness must be maintained. In fact, when optimising design time, a tendency to decrease precision can be seen. This is to be limited to the extent permitted.

3.8.2.5. Risks

This criterion aims at assessing designing errors related to the model. In fact, when designing it is highly important not to underestimate or overestimate the various factors, such as – respectively – loads and resistance of the materials. For this reason, the loads are generally increased whereas the strength of each material is decreased. The latter will be affected by changing the material's characteristics as planned in each FEM approach.

This results in the requirement to adjust and verify the validity of each approach. A way to measure this criterion in this paper is to assess what further implementations are necessary in the calculations of the reductive factors.

3.8.3. Criteria Comparison

As mentioned before a two-by-two comparison will be performed for each pair of criteria. For a clear understanding the following matrix (see Table 37) will be used, where the criteria in the first column will be compared to the following column, based on which has more importance in this project. Lastly, the sum of each column will be assessed.

Table 37: 1st Criteria Matrix – Comparison between criteria

	Designing Time	Accuracy	Modelling Ease	Required Skills	Risks
Designing Time	1.00	3.00	2.00	4.00	2.00
Accuracy	0.33	1.00	0.50	2.00	2.00
Modelling Ease	0.50	2.00	1.00	3.00	1.00
Required Skills	0.25	0.50	0.33	1.00	0.50
Risk	0.50	0.50	1.00	2.00	1.00
Sum	**2.58**	**7.00**	**4.83**	**12.00**	**6.50**

As it can be seen from Table 37, each criterion situated in the left row has been compared to the criteria in the columns of the table. The scores have been determined upon discussions with the firm's desire and logical reasoning. In fact, the structural engineering team at Aig Associati and Partner strongly advocate for a lower designing time over the accuracy of the model, hence 'designing time' scored 3 compared to 'accuracy of the model'.

The detailed reasoning is described as follows:

- **Designing time** is considered being
  - more important compared to **accuracy of the model**, as agreed with the engineer's team. In fact, as time is reflected in money, it is of utter importance for the company to save costs as much as possible.
  - slightly more important compared to **modelling and recreation ease**, considering that a easier model will result in a quicker design phase.
  - moderately important compared to the **required skills**, in fact it is important for engineers to acquire new skills in order to foster personal and company growth. Once new skills are acquired and mastered, it is logical to say that the modelling duration will decrease with time.
  - slightly more important compared to **risks** in the model for the fact that risks in this case relate to the adjustments needed to determine the relived material's properties, hence this can be easily dealt with proof reading and peer reviewing.
- **Modelling and recreation ease** is considered being
  - slightly more important compared to **accuracy of the model**, as this criterion heavily relates to the amount of time needed to model a structure.
  - more important compared to **required skills** for the same reasoning explained above.
- **Risks** are considered being
  - slightly more important compared to the **required skills**, as it is crucial to provide a correct and reviewed model for the safety of both the structure and the people.

The rest of the scores are all relating to each other and need to be equal; for example, if designing time scores 3 compared to accuracy, then accuracy will score 1/3 compared to designing time. This needs to be carefully applied for all the criteria comparisons.

The next step is to give a weight to each criterion; this is done by simply dividing each score of a criterion by its sum. Finally, the arithmetic mean will be taken to provide a normalized weight. The sum of the normalized weights needs to converge to the unit as to provide a fair multi criteria analysis.

The results are shown in Table 38.

Table 38: 2nd Criteria Matrix – Criteria's weight

	Designing Time	Accuracy	Modelling Ease	Required Skills	Risks	Normalized Weight
Designing Time	0.387	0.429	0.414	0.333	0.308	**0.37**
Accuracy	0.129	0.143	0.103	0.167	0.308	**0.17**
Modelling Ease	0.194	0.286	0.207	0.250	0.154	**0.22**
Required Skills	0.097	0.071	0.069	0.083	0.077	**0.08**
Risks	0.194	0.071	0.207	0.167	0.154	**0.16**

In this case designing time is considered the most important criteria with a score of 37%, followed by modelling ease with 22%, then accuracy with 17%, risks with 16% and finally required skills with 8%.

3.8.4. Consistency Ratio

It is highly important to make sure that there is consistency between the values assessed above. This is a factor specific to the APH method – used in this document – and it delivers sensible and consistent choices.

The consistency value (λ) is found as follows:

1. By taking each value from Table 38 and multiplying it by its normalized weight
2. Then summing each row and dividing it by the weight

The results of such steps are indicated in Table 39.

Table 39: 3rd Criteria Matrix – Consistency Check

	Designing Time	Accuracy	Modelling Ease	Required Skills	Risks	λ
Designing Time	0.374	1.122	0.748	1.496	0.748	**12.00**
Accuracy	0.057	0.170	0.085	0.340	0.340	**5.83**
Modelling Ease	0.109	0.436	0.218	0.654	0.218	**7.50**
Required Skills	0.020	0.040	0.026	0.079	0.040	**2.58**
Risks	0.079	0.079	0.158	0.317	0.158	**5.00**

Once the consistency value is found, the consistency ration (CR) can be finally calculated.

This determines whether the matrix is consistent and can be calculated as follows.

$$CI = \frac{\lambda_{max} - n}{n - 1} \tag{61}$$

$$CR = \frac{CI}{RI} \tag{62}$$

$$Aw = \lambda_{max} w \tag{63}$$

In which:

n is the number of dimensions of the matrix
λ_{max} is the average of the consistency value
RI is the random index
w is the criterion normalized weight
A is the comparison matrix

Note that the random index (*RI*) is obtained from the following table (Wajeeha A Qazi, 2018).

Table 40: Random Index for Different Matrix sizes

N	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
RI	0	0	0.58	0.90	**1.12**	1.24	1.32	1.41	1.45	1.49

The average value of the consistency value (λ_{max}) is established with equation (63) in the following way.

$$Aw = \lambda_{max} w$$

$$\begin{bmatrix} 1 & 3 & 2 & 4 & 2 \\ \frac{1}{3} & 1 & \frac{1}{2} & 2 & 2 \\ \frac{1}{2} & 2 & 1 & 3 & 1 \\ \frac{1}{4} & \frac{1}{2} & \frac{1}{3} & 1 & \frac{1}{2} \\ \frac{1}{2} & \frac{1}{2} & 1 & 2 & 1 \end{bmatrix} * \begin{bmatrix} 0.374 \\ 0.170 \\ 0.218 \\ 0.079 \\ 0.158 \end{bmatrix} = \lambda_{max} * \begin{bmatrix} 1.955 \\ 0.880 \\ 1.142 \\ 0.410 \\ 0.807 \end{bmatrix}$$

$$\begin{bmatrix} 1.95 \\ 0.88 \\ 1.14 \\ 0.41 \\ 0.81 \end{bmatrix} = \lambda_{max} * \begin{bmatrix} 0.374 \\ 0.170 \\ 0.218 \\ 0.079 \\ 0.158 \end{bmatrix}$$

$$\lambda_{max} = \begin{bmatrix} 5.225 \\ 5.175 \\ 5.238 \\ 5.157 \\ 5.095 \end{bmatrix}$$

$$\therefore \lambda_{max} = \frac{5.225 + 5.175 + 5.238 + 5.157 + 5.095}{5} = 5.18$$

The consistency ratio should never exceed the value of 0.1, if it does, changes are required in the matrix. In this case the consistency ratio has a value of 0.04, hence the MCA approach is defined fair and consistent.

$$CI = \frac{\lambda_{max} - n}{n - 1} = \frac{5.18 - 5}{5 - 1} = 0.045$$

$$CR = \frac{CI}{RI} = \frac{0.04}{1.12} = 0.04$$

Following the steps, a sensible MCA has been created and can therefore assess justly the different proposed alternatives.

Finally, the alternatives will be scored out of 9 and their score per criterion will be multiplied by its weight. The different weighted scores are summed, and these will provide the final overall score.

3.9. Analysis of the Alternatives

Even though three alternatives will consist in modelling the slab in different ways, the results should all converge; in fact, the three FEM approaches aim at examining one real life situation.

To determine the correctness of the three models it must be ensured that self-weights and deflections result equal. For the ease of reaching this condition, the same section utilized for the full slab and described in chapter 3.5.2 will be adopted for all three approaches proposed, with the difference of the material's characteristics.

In fact, as determined in chapter 3.6, the section to be lightened will present altered material properties as according to the following paragraphs.

Moreover, the calculation method, the analysis type and the loads applied will refer to the full slab's previous description (see respectively chapters 3.5.3, 3.5.4 and 3.5.8), with the difference of the materials used, hence the results.

3.9.1. Alternative 1 – Reduction of Flexural Strength

Since the addition of voids in the concrete slab understandably results in less material being included, it is clear that one of the ways in which the lightened slab may behave is to exhibit lower bending strength. This can be easily modified in the FEM program by manually inputting an adjusted elastic modulus for the concrete.

As described in chapter 3.3.1, the reduced young's modulus for this alternative is equal to 89% the modulus of the concrete chosen. In this development concrete type C30/37 will be adopted. This presents an elastic value of 33019 MPa, therefore the elastic modulus for the relived concrete section will have a value of 29377.6 MPa.[23]

$$E_{lightened} = E * 0.89 = 33019 * 0.89 = 29377.6\ MPa \tag{64}$$

[23] The approximations are based on the spread sheets created for this project. See Appendix 2 – Lightened Slab Specifications Excel Sheet.

Moreover, the self-weight of the lightened slab must be lowered too, as a voided slab weights less than a full one. Based on the calculations performed in chapter 3.3.3.1, the reductive coefficient to be applied to the concrete's density is equal to 77%, hence the reduced concrete section will present a density of 19.3 kN/m^3.

$$\gamma_{lightened} = \gamma * 0.77 = 25 * 0.77 = 19.3\frac{kN}{m^3} \quad (65)$$

3.9.1.1. Characteristics of the Relieved Material

The structure presents the same materials as described in chapter 3.5.5; nonethelss, the lightened section (see purple zone in Figure 63), based on the previous calculations, will present altered characteristics (refer to Table 41).

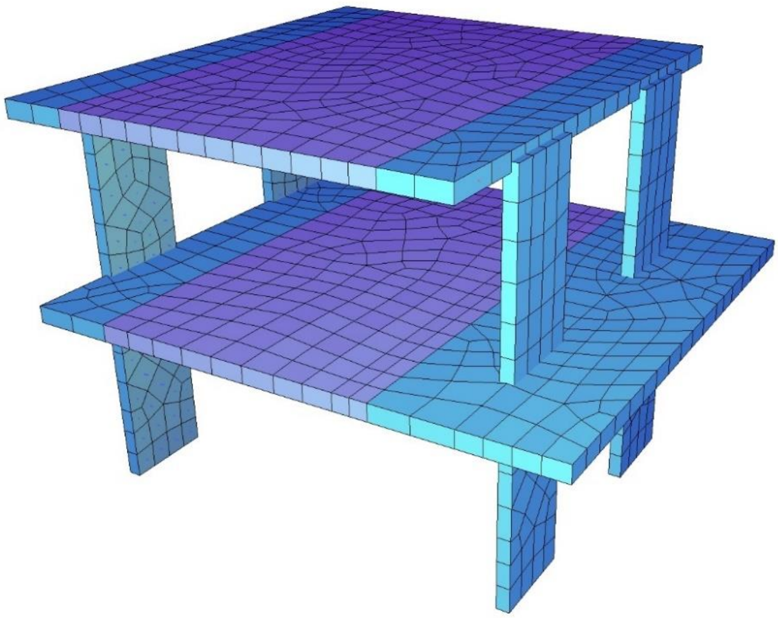


Figure 63: finite element model - Alternative 1

Table 41: Lightened C30/37 Characteristics

Young's modulus E	29378	[N/mm2]	Safetyfactor	1.50	[-]
Poisson's ratio μ	0.20	[-]	Strength fc	26.10	[MPa]
Shear modulus G	12241	[N/mm2]	Nominal strength fck	30.71	[MPa]
Compression modulus K	16321	[N/mm2]	Tensile strength fctm	2.94	[MPa]
Nominal Weight γ	19.3	[kN/m3]	Tensile strength fctk,05	2.06	[MPa]
Mean density ρ	2400.0	[kg/m3]	Tensile strength fctk,95	3.82	[MPa]
Elongation coefficient α	1.00E-05	[1/K]	Bond strength fbd	3.09	[MPa]
			Service strength fcm	38.71	[MPa]
			Fatigue strength fcd,fat	15.26	[MPa]
			Tensile strength fctd	1.37	[MPa]
			Tensile failure energy Gf	0.14	[N/mm]

In comparison with a standard C30/37 type of concrete, the relived concrete in this chapter presents alterations in the elastic, shear and compression modulus, as well as the nominal weight. In fact, the values differ as follows.

Table 42: Differences between standard and altered C30/37 type of concrete

		C30/37	Altered C30/37
Young's modulus	**E**	33019 MPa	29378 MPa
Shear modulus	**G**	13758 MPa	12241 MPa
Compression modulus	**K**	18244 MPa	16321 MPa
Nominal Weight	**γ**	25 kN/m^3	19.3 kN/m^3

The elastic modulus for the altered concrete type has been reduced by 11% - as determined previously – which consequently reduced the shear and compression modulus by the same amount. On the other hand, the nominal weight has been decreased of 23% compared to the standard material.

Regarding the stress-strain deformations accounting for both limit states (ULS and SLS), these remain unchanged compared to the values indicated in Table 19, meaning that the stress-strain curve follows the same trend as the unchanged material.

In the matter of the type of reinforcement steel, B450 C will also be utilized (see chapter 3.5.5.2 for further details).

3.9.1.2. Application

In order to alter the materials properties in the FEM software, the general properties must be altered. In SOFiSTiK 2023, these can be applied in the design code for the materials (see Figure 64).

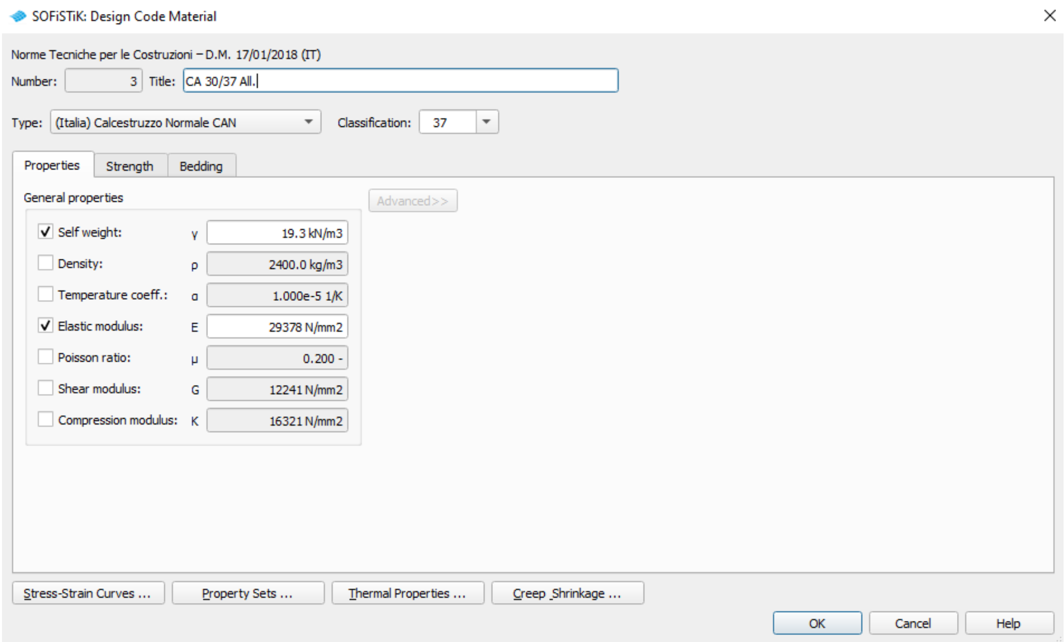


Figure 64: Design code material - Alternative 1

3.9.1.3. Designing Time

Following the approach described in chapter 3.8.2.1, the following times where recorder (see Table 43).[24]

Table 43: Recorded times - Alternative 1

Attempt 1	Attempt 2	Attempt 3	Average time
1 min 09 sec	1 min 02 sec	1 min 04 sec	1 min 05 sec

As it can be deduced by the steps to undertake – being two, changing self-weights and elastic modulus – it was expected to record low time scores on the stopwatch.

3.9.1.4. Modelling and Recreation Ease

Based on the low amount of time needs as well as the steps needed to adjust the model in the finite element application, the modelling and recreation of this alternative is considered straightforward and simple.

3.9.1.5. Required Skills

Similarly to the criterion 'recreation and modelling ease', the required skills for this approach are considered very low. In fact, only a basic knowledge of the FEM program and its AutoCAD extension is required. This is assumed to be within the expertise of a structural engineer or designer in general.

3.9.1.6. Self-Weight

Since the models as well as the external loads considered in the structure's section do not differ from the full slab's conditions (refer to chapter 3.5) only the differences will be discussed in these paragraphs.

A distinction of each modelling approach is the reduced nominal weight of the concrete used, in this case of 6.2 kN/m^2; this can be identified in Figure 65.

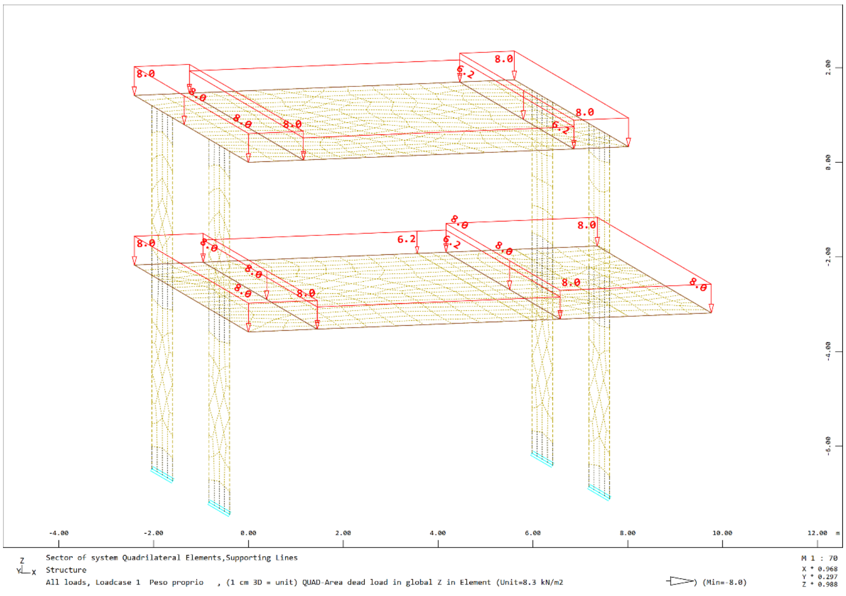


Figure 65: Self-weight of slabs - Alternative 1

[24] Refer to Appendix 9 – Designing Times Recorded for screenshots of stopwatch.

As illustrated above, the septa's self-weights are not depicted, for the reason that these have been left untouched in the model. On the contrary, the middle section of both the slabs have been reduced resulting in a weight of 6.2 kN/m^2.

Since the calculation of the modified material's density has been determined using the manual provided by Geoplast (Geoplast S.p.A. , 2019), it is reasonable to state that this prototype depicts the correct alteration. It is, therefore, of utter importance that the other two approaches show the same self-weight as determined in Figure 65.

3.9.1.7. Deformations

The deformations are examined considering only the self-weight of the structure, as to assess its validity.

It is of utter importance that all three alternatives present the same deflections – considering a small margin of error.

As shown in Figure 66, the deflection due to the self-weight of the structure accounts for 3 mm in the upper slab and 2.1 mm in the lower slab.

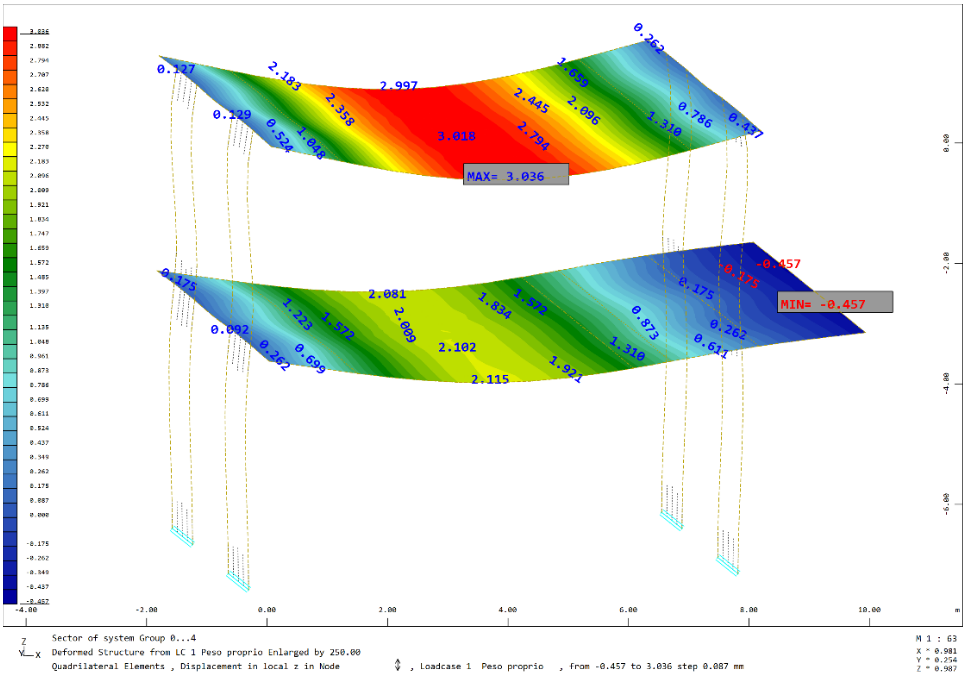


Figure 66: Deformations - Alternative 1

3.9.1.8. Flexural Strength and Concrete Stress

This characteristic of the concrete is heavily influenced considering an altered type of material, such as in this case. In fact, flexural strength in a full concrete section determines the stress prior its rupture or yielding point.

In this case, a middle section of the lightened slab has been considered; hence, it experiences compression in the top section and tension in the bottom one. This is clearly shown in Figure 67, where the stress distributions thought section are represented at the ultimate limit state.

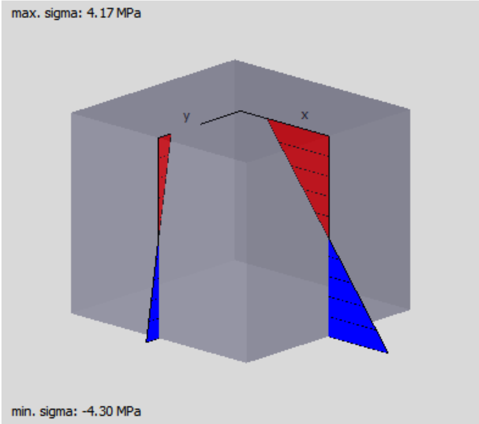


Figure 67: Concrete's stress-diagram

As it can be noticed, the stress diagram is the same as in a full block (see Figure 67); the stresses are symmetrically distributed due to the shape of the slab's cross section and the stresses follow a linear trend. On the other hand, the expected behaviour of a lightened slab section, follows the stress diagram shown in Figure 68. The diagram shows the stress distribution along the section A-A, where the middle section is completely hollow, hence no stresses will be experienced.

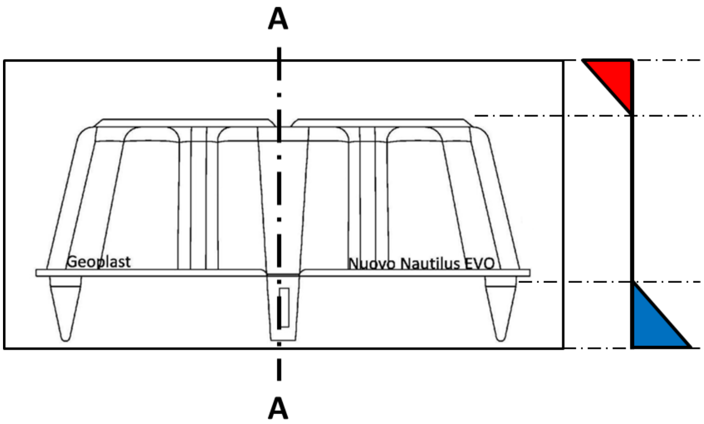


Figure 68: Expected stress-diagram in lightened slab section

It can be therefore, stated that the stress diagram extrapolated from the lightweight slab modelled with finite elements – according to this alternative – is not accurate with respect to the physical element.

Concrete Tension Checks

As mentioned in chapter 3.5.7.2, concrete cannot exceed the following stress values:

- → For the characteristic SLS combination $\sigma_{c,max} \leq 0.60\ f_{ck} = 0.6 * 30.71 = 18.4\ MPa$
- → For the long-term SLS combination $\sigma_{c,max} \leq 0.45\ f_{ck} = 0.45 * 30.71 = 13.8\ MPa$

It has been noticed, that by modelling the slab as an ordinary concrete element, the limits imposed by the Eurocodes were exceeded, failing the safety verifications (European Commission, 2006). Nonethelss, if the slab is deprived of part of its weight, it is expected that it will not exceed the compressive stresses indicated above.

This has been confirmed according to the results reached with the finite element model, as seen in Figure 69. In fact, on the left side of the picture the characteristic or rare combination is shown and the maximum stress value amounts to 15.51 MPa, hence not exceeding the limit of 18.4 MPa.

On the other hand, the right side of the figure depicts the stress results in quasi-permanent serviceability combination of the loads. The maximum stress encountered amounts to 13.64 MPa, being within the allowable stress (13.8 MPa).

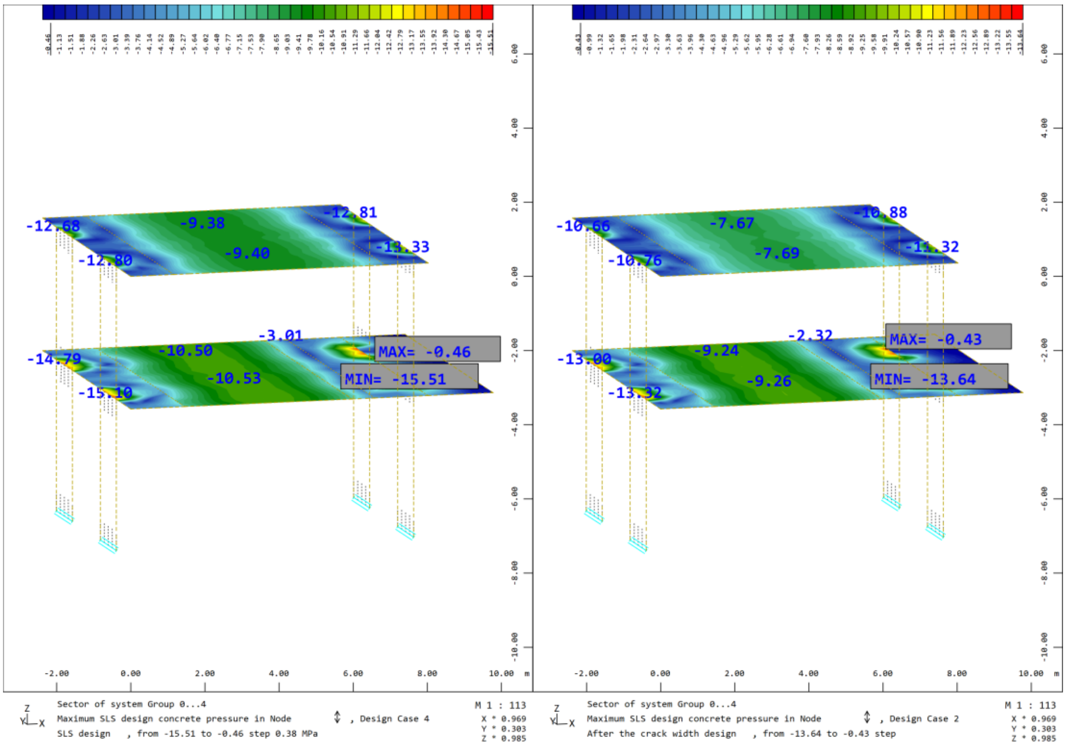


Figure 69: Compressive stresses

This reflects the effectiveness of adopting the expedient of lightened concrete flat slabs.

3.9.1.9. Accuracy of the Model

Regarding the accuracy of the model with a reduced flexural strength, it is believed that the behaviour is replicated to a hight extent compared to the real-life element. Nonethelss, the section is described as a solid element lacking the strength given by the zones distinguished by the standard concrete's properties. This is, however, also given by the lack of the voided sections as a general limitation for all the solutions proposed.

Moreover, as explicated in chapter 3.9.1.8, the behaviour of the relived concrete does not follow the trend of the physical element, making this approach reach low levels of accuracy.

3.9.1.10. Risks

As described in chapter 3.8.2.5, this criterion refers to the possible designing error and number of further implementations for the calculations of the reducing factors.

For the reduction of the slab's flexural strength no additional calculations to the given ones[25] had to be performed. Moreover, the results extrapolated from this approach have been utilised as reference figures to assess the correctness of the other approaches.

Nevertheless, the low accuracy of the model, accounts for possible risks in interpreting the results such as reinforcement required and stresses the concrete element undergoes, as seen in chapter 3.9.1.8.

3.9.2. Alternative 2 – Reduction of Slab Thickness

As identified in this paper, another way to reflect the lightened slab's behaviour in a finite element software is to reduce the thickness of the slab as to reduce its shear resistance and torsional resistance. The reduced thickness, as determined in chapter 3.3.2, has a value of 30.8 cm.

Additionally, to fully replicate the real-life lightened element, the density of the material must be reduced too. This has been determined using the same value as for the reduced flexural strength alternative, with the difference of the addition of an adjustment value.

The adjustment value is necessary to reach the same self-weight when considering a slab with unaltered thickness. As a matter of fact, the self-weight of a slab relates to the density of the material and the thickness of the element. Consequently, the reduction of the thickness further reduces the own weight of the element. This is to be avoided; hence, an adjustment factor is applied as follows.

$$\gamma_{lightened} = \gamma * (0.77 + a) = 25 * (0.77 + 0.039) = 20.23 \frac{kN}{m^3} \tag{66}$$

In which:

$$a = 1 - \left(\frac{W_{lightened,b}}{W_{lightened,a}}\right) = 1 - \left(\frac{5.94}{6.18}\right) = 1 - 0.96 = 0.039$$

$$W_{lightened,a} = 19.3 \frac{kN}{m^3} * 0.32m = 6.18 \frac{kN}{m^2}$$

$$W_{lightened,b} = 19.3 \frac{kN}{m^3} * 0.308m = 5.94 \frac{kN}{m^2}$$

3.9.2.1. Characteristics of the Relieved Material

As described previously, the only difference in the relived material's characteristics is its nominal weight being reduced of 19% approximately, thus the rest of the characteristics will remain unaffected (refer to chapter 3.5.5.1 for further information).

3.9.2.2. Application

Two steps are required to alter the lightened section's properties. The first step is to access the design code of the material to change its density as required (see Figure 70).

[25] (Geoplast S.p.A. , 2019)

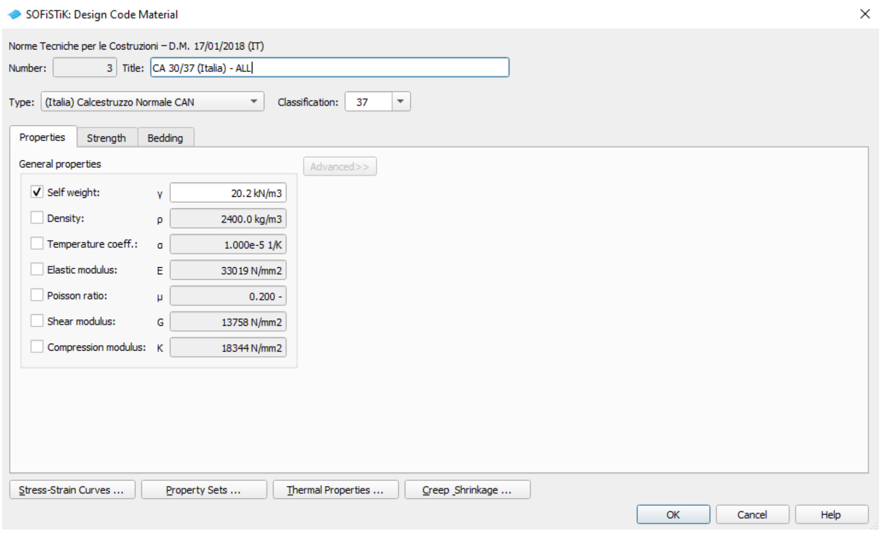


Figure 70: Design code material - Alternative 2

The second step refers to the changed thickness of the slab, which needs to be inputted when designing the structure on the AutoCAD extension of SOFiSTiK. In fact, to ease the designers work, the FEM software presents an extension called 'SOFiPLUS (-X) Modelling' which is an AutoCAD extension that easily links with the finite element software.[26]

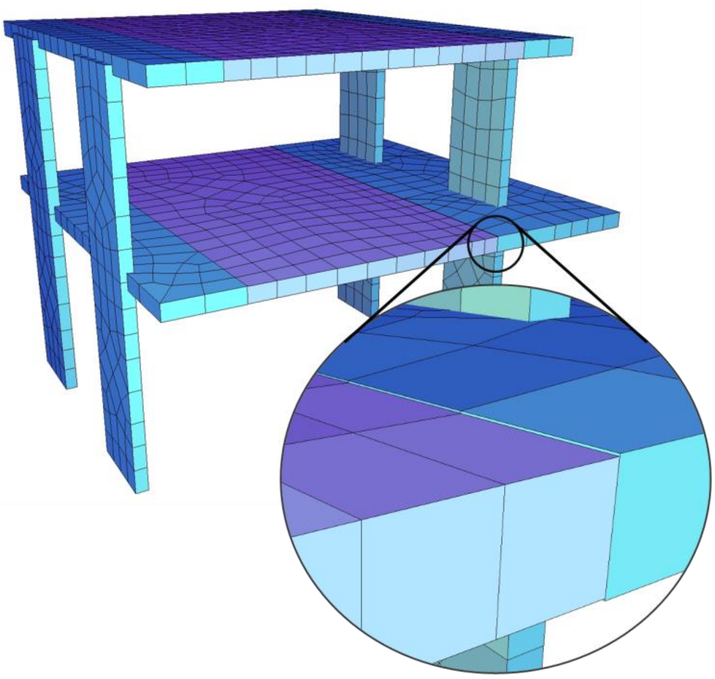


Figure 71: finite element model – Alternative 2

3.9.2.3. Designing Time

Following the approach described in chapter 3.8.2.1, the following times where recorder (see Table 44).[27]

[26] Both SOFiSTiK and SOFiPLUS (-X) Modelling are provided when downloading the software. With SOFiPLUS, customers can use AutoDesk AutoCAD, Autodesk Revit or McNeel Rhinoceros to create the structural model.

[27] Refer to Appendix 9 – Designing Times Recorded for screenshots of stopwatch.

Table 44: Recorded times - Alternative 2

Attempt 1	Attempt 2	Attempt 3	Average time
1 min 23 sec	1 min 37 sec	1 min 17 sec	1 min 26 sec

Compared to the first alternative, since two steps had to be performed, the recorded times are slightly higher than those quantified in the first alternative.

3.9.2.4. Modelling and Recreation Ease

Compared to the first approach proposed the recreation ease of the reduced thickness is considered lower, as two steps need to be undertaken (see description in chapter 3.9.2.2). Nonethelss, since application phase requires little amount of time the solution is judged to be efficient.

Moreover, it must be noted that the reduction of the thickness and the self-weight adjustments have been used extensively in different companies, as this was one of the first developed solution to model a lightened concrete slab.

3.9.2.5. Required Skills

As far as additional skills to the essential software knowledge of the finite element method and its extensions are concerned, no other advanced capabilities are required. In fact, to apply the alterations to the standard material a 'user-friendly' interface – shown in Figure 70 – can be utilised.

However, regarding the calculation of the reduced density and its implementation to reach the correct model, logical and critical thinking are essential.

3.9.2.6. Self-Weight

As seen previously, the altered nominal weight (20.2 kN/m^3) differs in value compared to the figure adopted for the first alternative, being 19.3 kN/m^3. This is given in order to achieve the same results, hence a valid model.

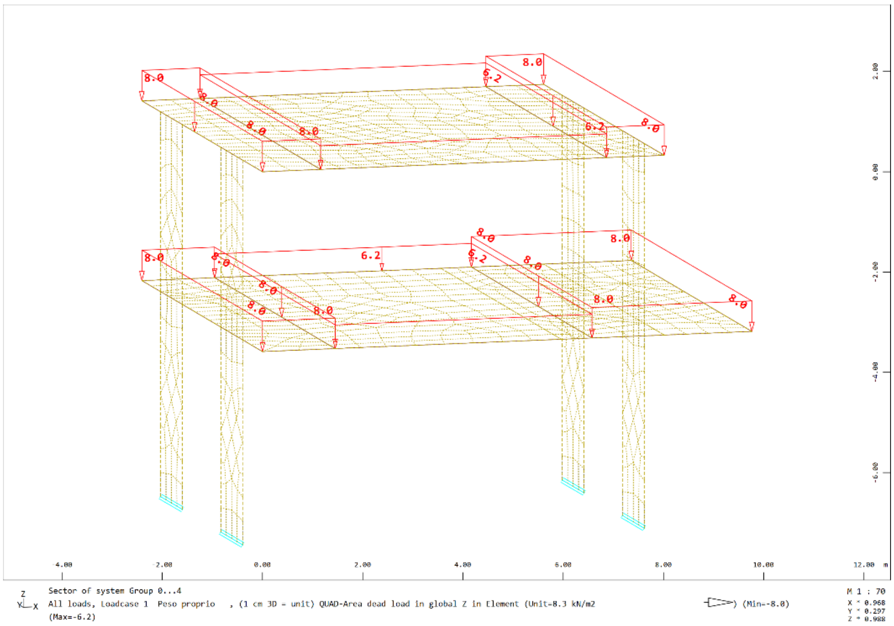


Figure 72: Self-weight of slabs - Alternative 2

The accuracy and correctness of the model is established by the convergence of the results nominal weight's – being of 6.2 kN/m^2 spread over the lightened section of the slab – of both alternative one and two.

3.9.2.7. Deformations

According to what stated in chapter 3.9, the deflection values should converge to around 3 mm for the upper slab and 2.1 mm for the lower slab. These values are given by the results given by the first approach.

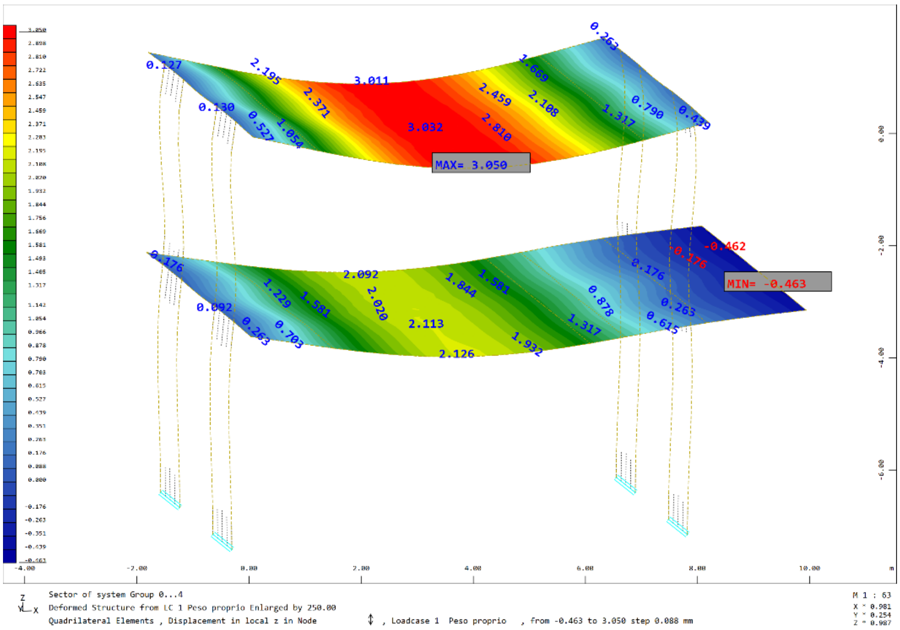


Figure 73: Deformations - Alternative 2

The outcomes reached with the reduction of the thickness (see Figure 73) converge with the required figures; in fact, the upper slab deflects downwards of 3 mm and the lower slab of 2.1 mm. Thus, the calculations have been performed as necessary and the model is considered valid.

3.9.2.8. Flexural Strength and Concrete Stress

As seen in the first alternative, the stress distribution thought the slab's section can be an indication of the verisimilitude of the approach used.

The stress diagram at ultimate limit state for the approach utilizing a fictitious thickness, can be visualise in Figure 74.

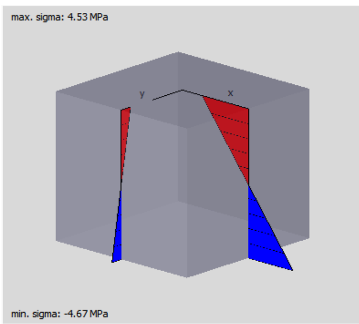


Figure 74: Concrete's stress-diagram

Since the section has been modelled as a full concrete slab with reduced thickness, it is understandable to expect the same behaviour of a standard concrete slab. This refers to the symmetricity and linearity of the stresses, lacking to identify the hollow sections.

Concrete Tension Checks

The effectiveness on relived flat slabs, regarding the compression stresses of concrete, has already been proven with the first alternative (chapter 3.9.1.8). Nonetheless, it is interesting to remark the fact that even though the lightened slabs have been modelled with two different approaches, the results shown in Figure 75 converge with those found in the structure modelled with a reduced flexural strength.

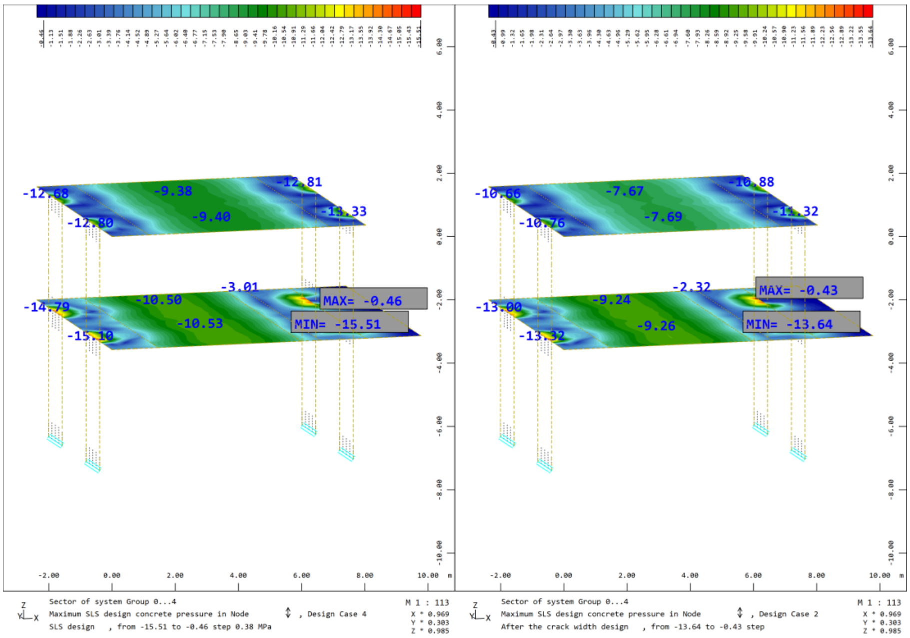


Figure 75: Compressive stresses

The convergence of results accentuates the fact that both models have been processed correctly; however, it underlines the low accuracy to the real model.

3.9.2.9. Accuracy of the Model

Regardless the reduced dead-weight and thickness of the lightened slab, the solution described in this chapter, is believed to less represent the real-life element. In fact, the latter does not present a reduced thickness and exhibits sections with the properties of the standard concrete (C30/37).

Moreover, as seen in the first approach, the stress distribution is not in accordance with reality, resulting in a lower degree of representation of reality.

It must be noted that this solution has been the first approach utilized in the company Aig Associati and Partner, as other approaches were not yet developed. Thus, its effectiveness is assured despite its low representation of reality.

3.9.2.10. Risks

Risks in this solution may derive from the determination of the correction factor regarding the calculation of the adjusting factor for the self-weight. In fact, by assigning the slab's section a fictitious thickness its calculated self-weight required to differ from the one calculated in chapter 3.3.3.1. This is considered a heightened risk of error in the calculation phase.

Additionally, its low degree of accuracy of the model heightens the probability of risks, regarding the interpretation of the data.

3.9.3. Alternative 3 – 'Sandwich' Material

The last approach, aims at altering only the inner section of the slab, as shown in orange in Figure 76 below.

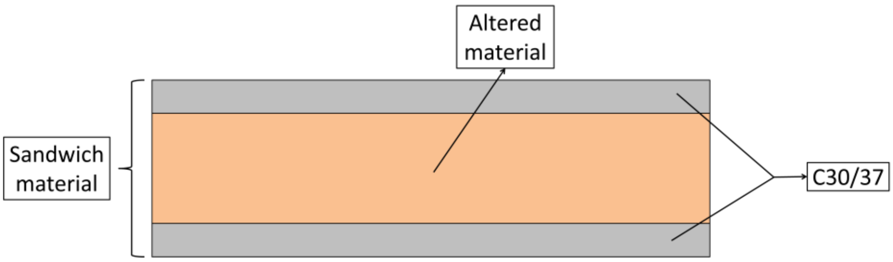


Figure 76: Sandwich material

The two outer layers will be modelled as 8 cm thick slabs with a standard concrete type (C30/37). On the other hand, the thickness of the altered material will be of 16 cm and will present a reduced elastic modulus equal to 27% of the standard material (see chapter 3.3.1 for further explanation).

$$E_{lightened} = E * 0.27 = 33019 * 0.27 = 8788.5\ MPa \tag{67}$$

Moreover, as calculated in chapter 3.3.3.2, the density will be 54% of the density of concrete.

$$\gamma_{lightened} = \gamma * 0.77 = 25 * 0.54 = 13.52\ \frac{kN}{m^3} \tag{68}$$

Lastly, to exhibit the reduced shear strength a factor of 62% (see calculations in chapter 3.3.2.2) will be applied to the altered material.

3.9.3.1. Characteristics of the Relieved Material

In this alternative the relived slab will present two materials: a standard C30/37 type of concrete for the outer slabs and an altered C30/37 type of concrete for the inner section. The latter will be further detailed in this paragraph.

The alterations applied on the material are based in the previous calculations and exhibit the following characteristics (see Table 45).

Table 45: Altered C30/37 Characteristics

Young's modulus	**E**	8798	[N/mm2]	Safetyfactor		1.50	[-]
Poisson's ratio	**μ**	0.20	[-]	Strength	fc	25.5	[MPa]
Shear modulus	**G**	3662	[N/mm2]	Nominal strength	fck	30.00	[MPa]
Compression modulus	**K**	4883	[N/mm2]	Tensile strength	fctm	2.90	[MPa]
Nominal Weight	**γ**	13.5	[kN/m3]	Tensile strength	fctk,05	2.03	[MPa]
Mean density	**ρ**	1252.5	[kg/m3]	Tensile strength	fctk,95	3.77	[MPa]
Elongation coefficient	**α**	1.00E-05	[1/K]	Bond strength	fbd	2.59	[MPa]

			Service strength fcm	38.00	[MPa]
			Fatigue strength fcd,fat	14.96	[MPa]
			Tensile strength fctd	1.15	[MPa]
			Tensile failure energy Gf	0.14	[N/mm]

As determined previously, a reduction on the young's modulus – consequently on the shear and compression modulus too – has been applied, decreasing it of 73%. Moreover, the nominal weight values nearly half of the respective standard material.

The combination of the two layers – the standard for the outer slabs and the altered material for the middle portion – constitute the so-called 'sandwich' material, which presents the following characteristics.

Table 46: Sandwich material characteristics

Young's modulus E	20904	[N/mm2]	Safetyfactor	1.50	[-]
Poisson's ratio μ	0.20	[-]	Calc Strength fy	25.80	[MPa]
Shear modulus G	8710	[N/mm2]	Ultimate strength ft	30.36	[MPa]
Compression modulus K	11613	[N/mm2]			
Nominal Weight γ	19.3	[kN/m3]			
Weight buoyancy γa	18.3	[kg/m3]			

Compared to the standard C30/37 type of concrete, the 'sandwich' material presents an elastic modulus equal to 63% of the standard modulus (33019 MPa). Consequently, both the shear and the compression modulus are reduced by the same amount.

The sandwich material has been applied to the structural model in accordance with the findings of chapter 3.6 (see Figure 77).

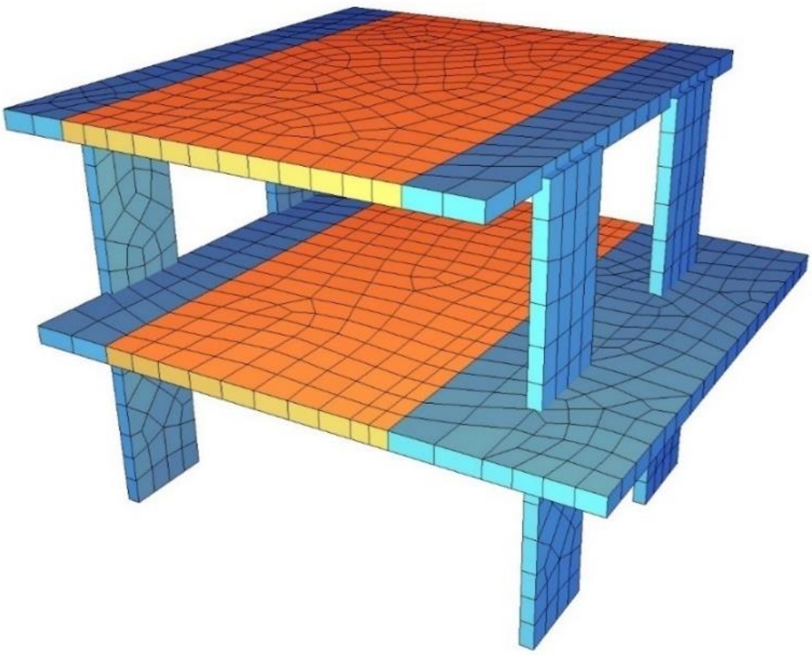


Figure 77: finite element model - Alternative 3

3.9.3.2. Application

Conversely to the first two approaches, the sandwich material requires text input instead of graphical input. In fact, as a feature of SOFiSTiK, users can decide whether to input data graphically – using the window and tasks provided by the software by default – or using the text editior 'TEDDY'.

SOFiSTiK comes with a text interface integrated in all applications. The text menu enables the full capabilities of SOFiSTiK to be exploited for optimised workflows. However, this requires the knowledge of programming syntax, which in this case is CADiNP.[28]

Using the TEDDY interface, which fully supports and utilises CADiNP syntax, the so defined 'sandwich' material can be created. In fact, two materials are defined – being the outer slabs and the altered material – and with the use of CADiNP syntax the combination of the three outputs the desired layered material.

With the help of both engineer Leonardo Mattei and engineer Emanuele Agostini the TEDDY text has been created and implemented in the finite element application.[29]

The important features to add are the following:

I.	The desired concrete type for the outer materials	C30/37
II.	The desired reinforcement type	B450 C
III.	The altered elastic modulus	8789 MPa
IV.	The altered density	13.52 kN/m^3
V.	The various thicknesses of the layers	8 cm – 16 cm – 8 cm
VI.	The reduction coefficient for shear reinforcement	0.62

Once the six above mentioned features are implemented and the results are achieved, the outcome results in the creation of the 'sandwich' material, which can be imported into SOFiSTiK, as seen in Figure 78.

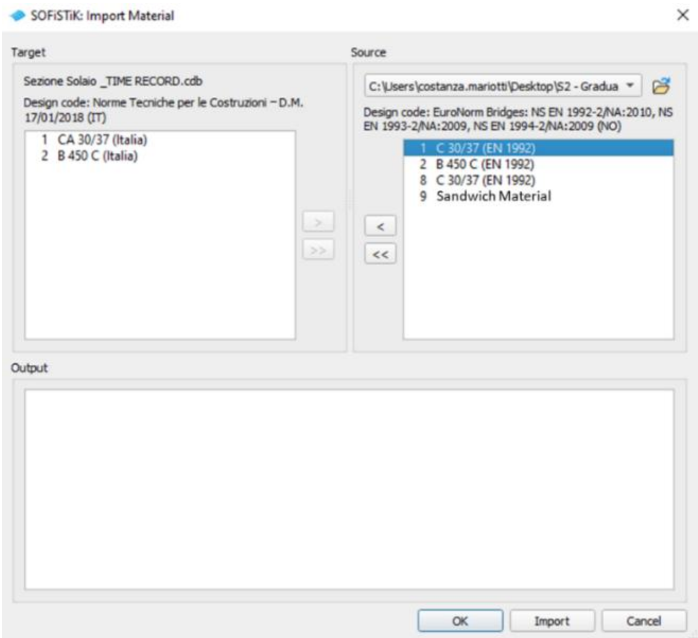


Figure 78: Standard Interface for importing new materials

[28] CADiNP is a computer language based on the syntax developed in 1976 in Germany, now known as CADINT.

[29] See Appendix 10 – Extrapolation of CADiNP Text.

3.9.3.3. Designing Time

Following the approach described in chapter 3.8.2.1, the following times where recorder (see Table 47).[30]

Table 47: Recorded times - Alternative 3

Attempt 1	Attempt 2	Attempt 3	Average time
2 min 30 sec	1 min 52 sec	1 min 35 sec	1 min 59 sec

Since this approach, requires the most steps – remembering the use of the CADiNP syntax – it is clear to state that this alternative would have scored the highest design times, as it did.

3.9.3.4. Modelling and Recreation Ease

Considering the high number of steps to undertake to implement the adaptations of the slab's model compared to the previous proposed expedients, this solution is considered more demanding. This is a resultant of the CADiNP text execution.

On the other hand, once the programming syntax has been prepared, its implementation into the FEM software consists of importing all the four materials – standard concrete for outer slabs, altered material for inner section, reinforcement and finally the layered material – utilizing the standard interface.

3.9.3.5. Required Skills

The layered material alternative, despite its effectiveness and accuracy to the realistic element, requires the additional knowledge of the programming syntax called CADiNP.[31]

Despite the complexity of the language at first glance, the textual interface is considered to unleash the full capabilities of SOFiSTiK and to optimise workflows enormously (SOFiSTIK for you, 2020). Furthermore, the textual inputs required in this alternative amount to a total of six and have been clearly described and indicated in the TEDDY interface.[32] This helps to broaden the target group of users and reduce the level of knowledge to be acquired.

3.9.3.6. Self-Weight

Given the fact that the nominal weight of the sandwich material converges with the value from the first alternative, being of 19.3 kN/m^3, it is expected that the self-weight of the lightened slabs converges to in this model.

[30] Refer to Appendix 9 – Designing Times Recorded for screenshots of stopwatch.

[31] This syntax is applied in SOFiSTiK 2023, other FEM software might make use of other computer languages.

[32] Refer to Appendix 10 – Extrapolation of CADiNP TextAppendix 10 – Extrapolation of CADiNP Text, where the indication and description of the text to be implemented and/or modified, is the text in green.

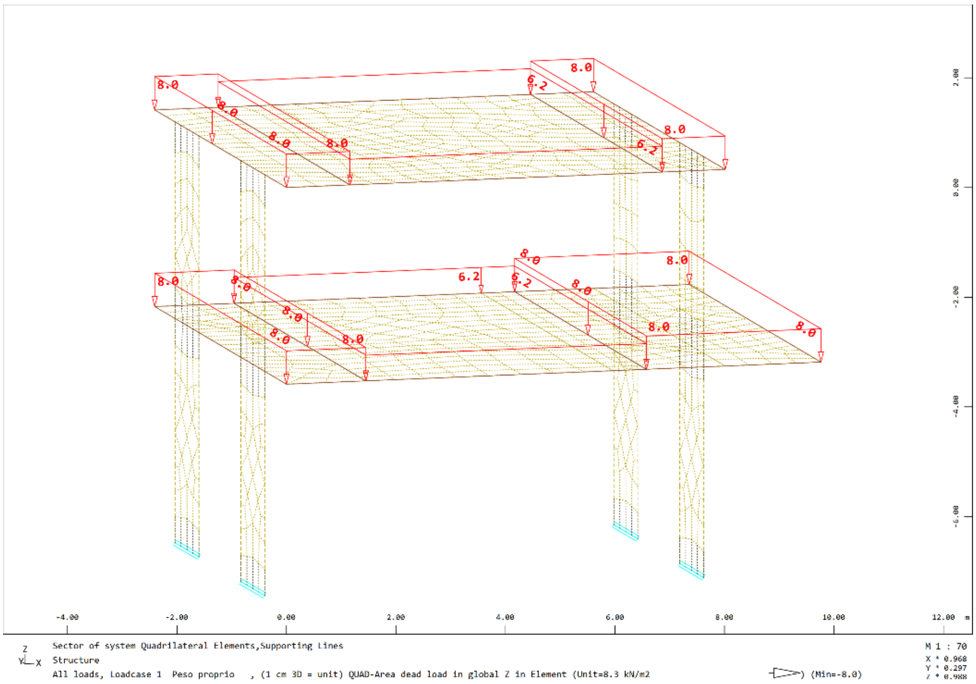


Figure 79: Self-weight of slabs - Alternative 3

As predicted, the values for the self-weight reach 6.2 kN/m^2 spread over the lightened section.

3.9.3.7. Deformations

To fully verify the accuracy of the model, deflection due to its self-weight will be assessed in this paragraph.

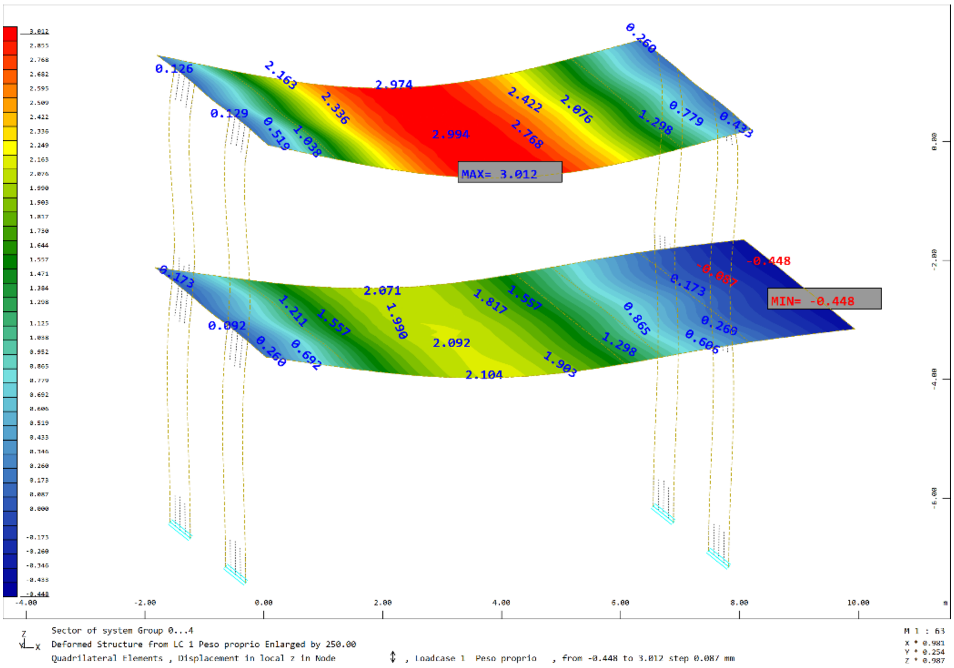


Figure 80: Deformations - Alternative 3

Since the deflections show 3 mm in the upper slab and 2.1 mm in the lower slab, the effectiveness of the model is confirmed in all three solutions proposed.

3.9.3.8. Flexural Strength and Concrete Stress

Conversely to the other alternative described above, modelling the slab as a package of three materials is the most accurate approach. In fact, as seen in Figure 81, the stresses distribute in a way as to better reflect the hollow sections in the slab.

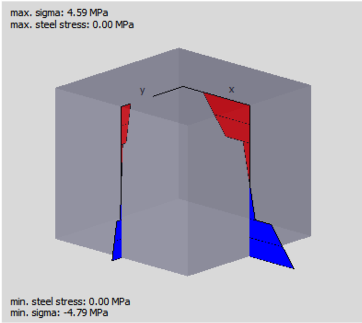


Figure 81: Concrete's stress-diagram

Since the reliving formwork could have not been modelled in the finite element software, it is obvious that the interpretation of such model would create difficulties. Nonethelss, this approach shows results that nearly converge with the expected behaviour of a lightened slab (refer to Figure 68).

Concrete Tension Checks

Conversely to the first two proposed solutions, modelling the flat slab as a package of materials reproduces results slightly different to the other.

In fact, modelling the slab with a reduced thickness or reduced elastic modulus yields the same results. This is given by the fact of their low accuracy degree. On the other hand, this approach, as seen in chapter 3.9.3.8, shows a more realistic behaviour, in terms of stress distribution.

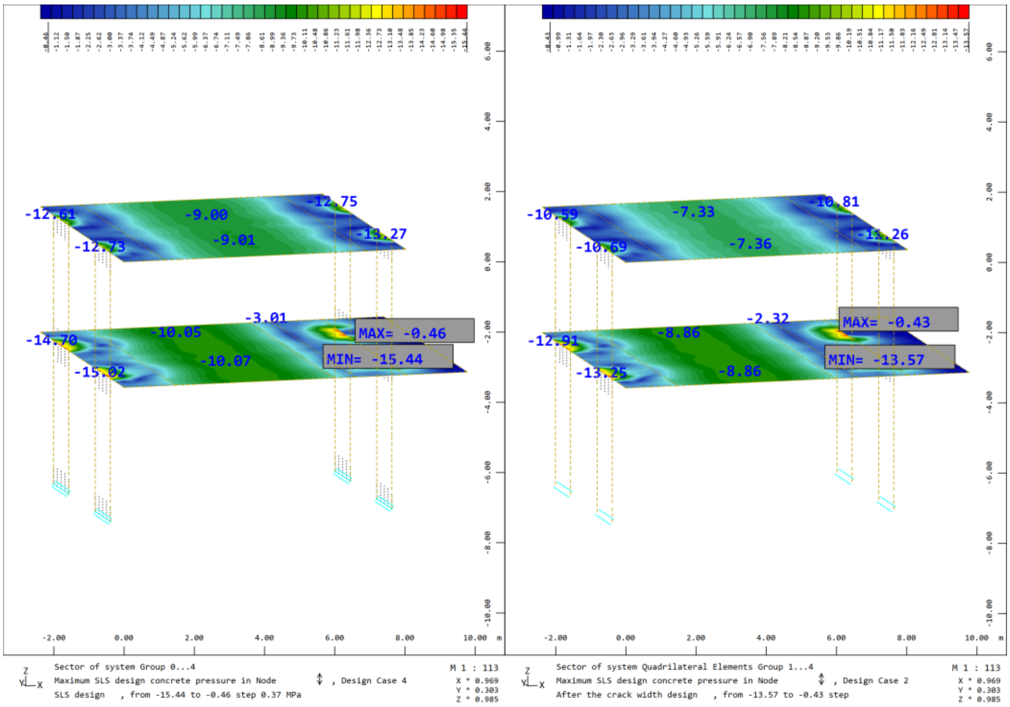


Figure 82: Compressive stresses

As it can be seen from Figure 82, the maximum compressive value experienced in rare serviceability limit state reaches 15.44 MPa and that achieved in long-term limit state reaches 13.57 MPa. Both the values are within the imposed limits of 18.4 MPa and 13.8 MPa respectively.

3.9.3.9. Accuracy of the Model

The solution described in these paragraphs is considered the most true-to-reality model compared to all the alternatives proposed. This is given by the fact that its section presents the properties of the standard C30/37 concrete type as well as the properties of the equivalent lightened slab.

Moreover, the outer slabs can be modelled with great precision to the real element, in the sense that its thicknesses are variable and can be determined from the existing section and properties of the lightening formwork.

3.9.3.10. Risks

Compared to the previous two alternatives, the use of a layered material is considered to present a low risk of possible mistakes. In fact, the steps to calculate the reduction factor (refer to chapter 3.3.3.2) do not require any implementation. Moreover, the application of the results into the TEDDY text is to be assessed on the basis of the designer's skills but will not be considered source of errors in this report. This is given by the fact, that the mistakes do not require peer-reviewing as these will be visible in the FEM programme's graphical output.

Lastly, as risks may arise from the inaccuracies in the model, it can be stated that since this model is the most accurate compared to the previous two, risks of errors is lowered.

4. Results

4.1. MCA Results

With the aid of a spread sheet,[33] the three variants have been compared using the method described in chapter 3.8.1 and the following results were reached (see Table 48).

Table 48: MCA Results

	Designing Time	Accuracy of the Model	Modelling Ease	Required Skills	Risks	Overall score
	37 %	17 %	22 %	8 %	16 %	
Reduced flexural strength	0.231	0.024	0.119	0.042	0.032	0.45
Reduced thickness	0.089	0.013	0.065	0.027	0.019	0.21
Sandwich material	0.051	0.132	0.036	0.011	0.109	0.34

The results scale the variants in the following way:

Choice 1. Reduced Flexural Strength
Choice 2. Sandwich Material
Choice 3. Reduced Thickness

This is given due to the fact that modelling the lightened slab with a reduced flexural strength recorded the lowest amount of time, followed by the reduced thickness approach.

On the other hand, as described in chapter 3.9.3.9, the sandwich material approach has been defined as the most accurate model opposed to reality, whereas the reduced thickness strategy has been considered the least realistic one.

Regarding the third criterion, the first alternative – being the reduced flexural strength – has been assessed as the easiest model to recreate and model on the contrary of the layered material strategy, which scored last.

Due to the fact, that the third approach – also referred to as the sandwich approach in this paper – required a basic knowledge of computer syntax conversely to the other solutions, this scored the lowest in the MCA. In fact, modelling the lightened slab with altered elastic modulus and nominal weight required no additional skills to the basic knowledge of a designer or engineer. On the other hand, the strategy referring to the reduced thickness required logical thinking and attention to detail, which has been considered to be a skill easier to acquire than the computer language required for the sandwich material approach.

Lastly, the risk of errors has been mainly encountered in the second variant – the reduced thickness – as an adjustment factor had to be taken into consideration for the creation of a valid model. Hence, this alternative scored the lowest, oppositely to the reduced sandwich material approach, which scored first.

All the above-mentioned reasons and scores combined, provided the final choice for the most optimum and efficient way of F.EM. modelling a lightened concrete slab: reducing the material's flexural strength of the section to be lightened.

[33] Refer to Appendix 8 – Multiple Criteria Analysis Excel Sheet.

4.2. Winning Solution’s Detailed Design

4.2.1. Foreword

Conversely to the analysis of a portion of the nursing home in this development (see chapter 3.5), this chapter aims at analysis the whole structure as to derive the structural requirements and the benefits of a relived slab system. Nonethelss, the focus will be put on the slabs and their results, neglecting the results and behaviour of the columns and septa, as these lay outside of this graduation’s project scopes.

The structural model (see Figure 83 and Figure 84) has been determined on the basis of the architectural plan,[34] designed accordingly to the client’s desires and wishes.

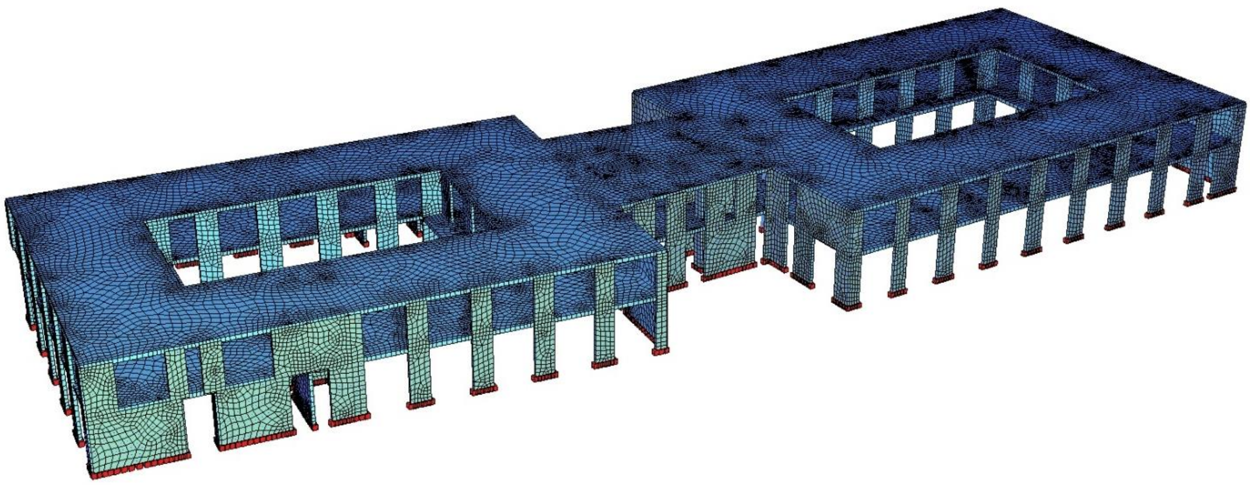


Figure 83: FEM Model of Casa Haus inge - view 1

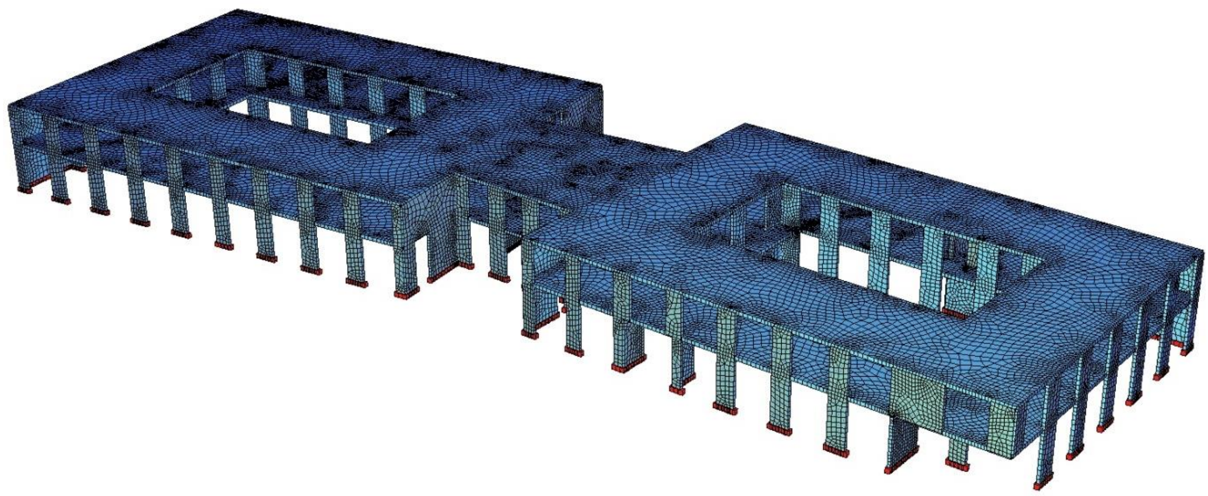


Figure 84: FEM Model of Casa Haus inge - view 2

[34] Note that the architectural plan does not only determine the shape and design of the structure and its bearing elements, but it also identifies the future usage of the zones. As a matter of fact, the zones indicated as individual/double rooms will show a different load compared to the more crowded zones such as the balconies and the corridors.

For further details on the architectural plan, refer to Appendix 11 – Architectural Plan for Casa Haus inge.

Moreover, the seismic load will be added to the external loads for the reason that the bearing walls are positioned in a way as to brace the building for such occurrence. This will also serve at determining the advantages and differences that are predicted to arise, in comparison with the results calculated in chapter 3.4.2. In fact, the results reached are based on the whole structure with standard slabs, hence an elevated self-weight.

The structural analysis in this chapter is based on the previously performed study of the portion (see chapter 3.5), thus specification might stay unvaried. These refer to the following characteristics:[35]

- → Calculation methods — refer to chapter 3.5.3
- → Analysis type — refer to chapter 3.5.4
- → Exposure class — refer to chapter 3.5.6
- → Combination of loads — refer to chapter 3.5.7
- → Load analysis, specifically
    - o Non-structural permanent weight — refer to chapter 3.5.8.2
    - o Variable loads — refer to chapter 3.5.8.3
    - o Snow load — refer to chapter 3.5.8.4
- → Concrete cover and reinforcement — refer to chapter 3.5.11

Lastly, the differences will be reported in the sections below.

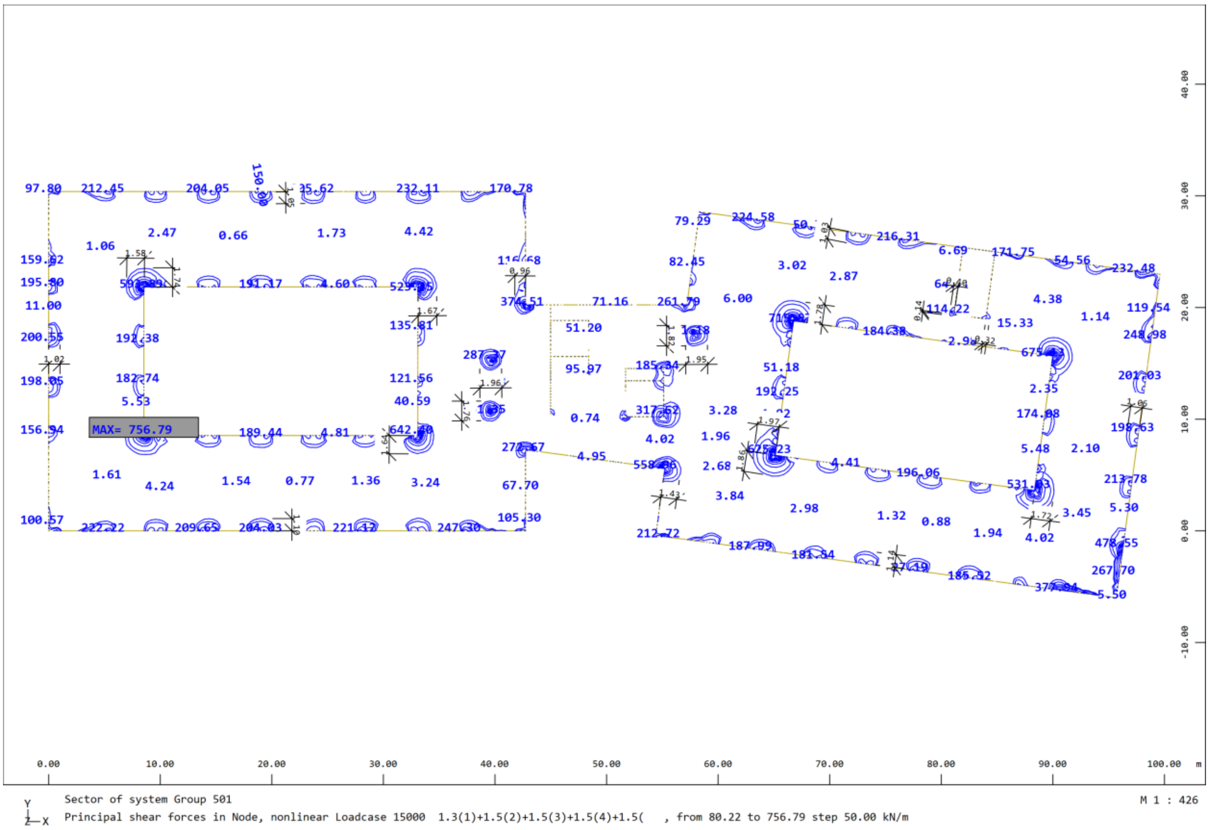
4.2.1. Determination of Section to the Lightened

In the same way as performed with the portion of the structure (see chapter 3.6), the sections that are structurally safe to be lightened are determined on the basis of the shear loads on the slabs.

From the analysis of the structure considering two standard full flat slabs, the coming results have been determined.

Based on the shear resistance calculated in section 3.6.2.2 of this report – reaching a value of 80.22 kN – the following ranges have been identified (see Figure 85, top figure for roof slab and bottom figure for first-floor slab).

[35] For the reader's convenience, the listed items will not be repeated in this section of the research document. Nonethelss, it is recommended to refer to the sections as indicated in the list above.



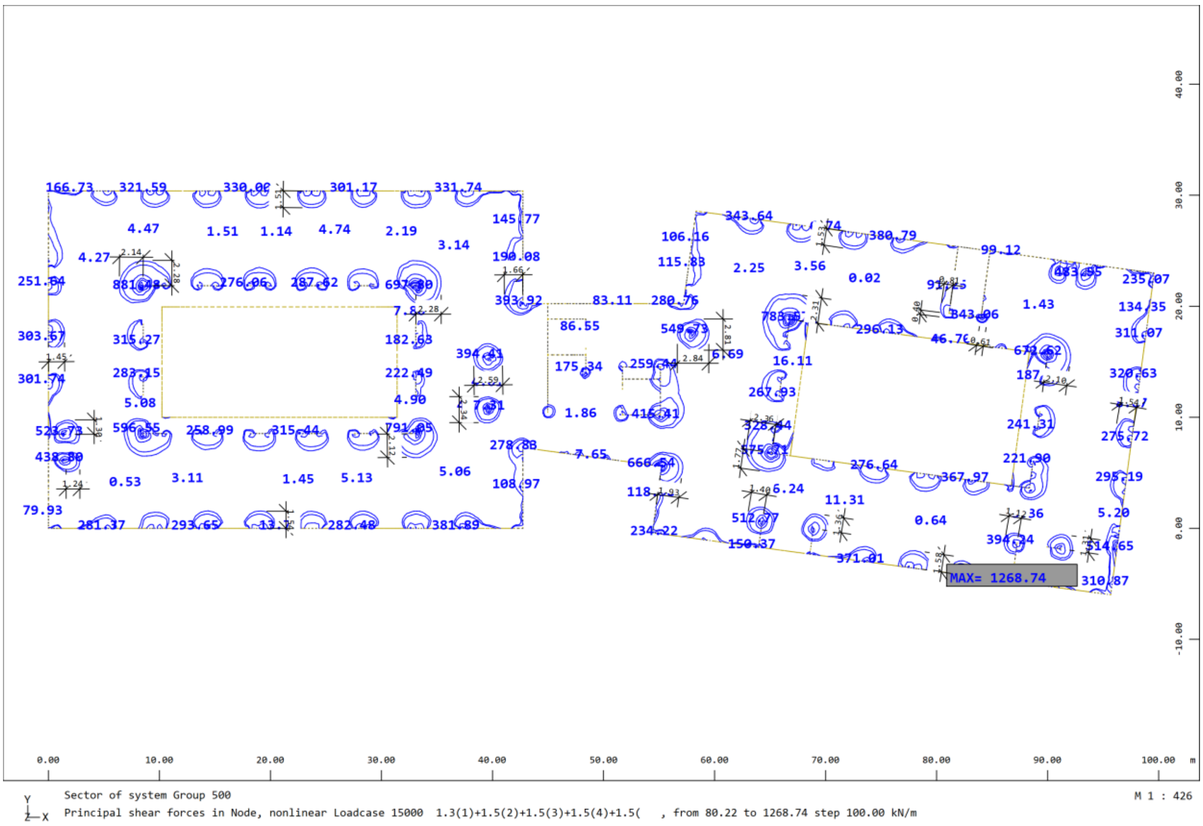


Figure 85: Shear Forces ranging from 80.22 kN to the max

As it can be noticed, the two units – being the two outer sections of the buildings with hollow centre – can be easily relived of their self-weight by placing the formwork in a way to avoid the outer perimeters with a distance describe in the picture above (Figure 85).

On the other hand, the inner section – being the one connecting the two units – presents many septa, hence it would result more challenging and time consuming placing the hollow plastic volumes in that section compared to leaving the slabs unrelieved. For this reason, it has been decided to only lighten the two outer units of the building.

The relived sections for the upper flat slab can be visualised in Figure 86.

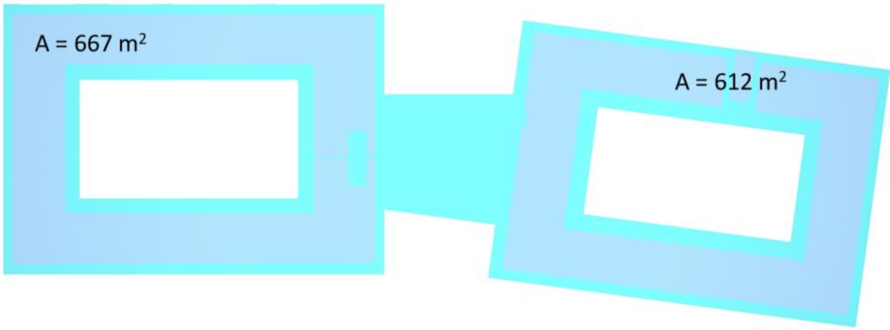


Figure 86: Relieved portions - Roof Slab

As it can be noticed, the lightened portions of the slab are not placed in vicinity of the three columns designed in the development and are positioned according to the measures found in Figure 85.

Lastly, the lightened portions are 62% of the total surface are of the roof slab, as indicated in equation (69).

$$\% = \frac{A_1 + A_2}{A_{tot}} * 100 = \frac{667 + 612}{2051.27} * 100 = 62\% \tag{69}$$

On the other hand, it is understandable that the first-floor slab experiences a heavier load, due to the transfer of all the upper loads thought the septa and columns onto the slab. This influences the area and portions of the slab that is safe to be lightened, by decreasing its dimensions, as seen in Figure 87.

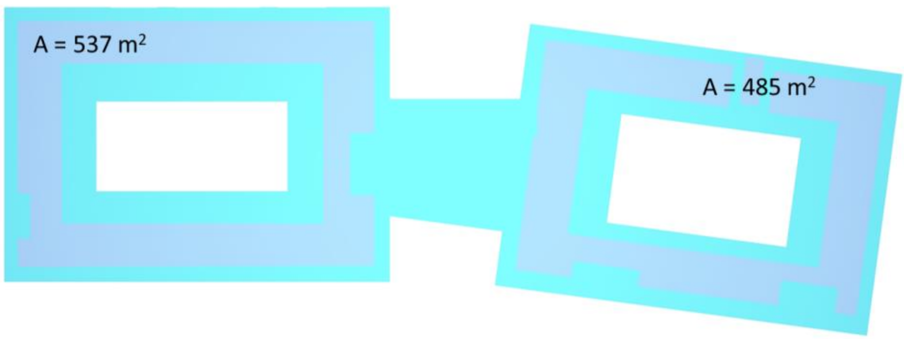


Figure 87: Relieved portions - First-Floor slab

The lightened portions in the first-floor slab, contribute to 50% of the total surface area of the floor, refer to equation (70).

$$\% = \frac{A'_1 + A'_2}{A'_{tot}} * 100 = \frac{537 + 485}{2051.27} * 100 = 50\% \tag{70}$$

The results shown above can be translated into the determination of the number of relieved sections in the structure as a whole, being 56%.

$$\% = \frac{A_1 + A_2 + A'_1 + A'_2}{A_{tot} + A'_{tot}} * 100 = \frac{2301}{2051.27 * 2} * 100 = 56\% \tag{71}$$

4.2.2. Characteristics of the Materials

The materials used in the FEM model of the nursing home are:

- → Concrete type C30/37 for the slabs as well as the bearing walls and columns
- → Concrete type C30/37 with reduced weight and reduced flexural strength, according to the specifications found in chapter 3.9.1.1
- → Steel type B 450 C used as reinforcement steel for all the elements.

4.2.3. Load Analysis

4.2.3.1. Self-Weight

The structure's own weight is given by the two floors with a thickness of 32 cm and density of 25 kN/m^3 in the full sections and with a density of 19.3 kN/m^3 in the relived sections, as shown in purple in pictures Figure 86 and Figure 87.

4.2.3.2. Seismic Load

As determined in chapter 3.4.2, the design values (see Table 49) that describe the spectrum unique to this building have been added to the FEM model.

Table 49: Spectrum Design values

ULTIMATE LIMIT STATE	a_g [m/s^2]	F_0	T^*_c [s]	Behaviour factor
SLV	0.55	2.628	0.349	1.5

As it can be noticed, the decisive limit state in the ultimate state, is the life preservation one (SLV). This is given due to the importance to preserve human lives in case of an earthquake.

The values shown in Table 49 plot the following seismic spectrum depicted in Figure 88.

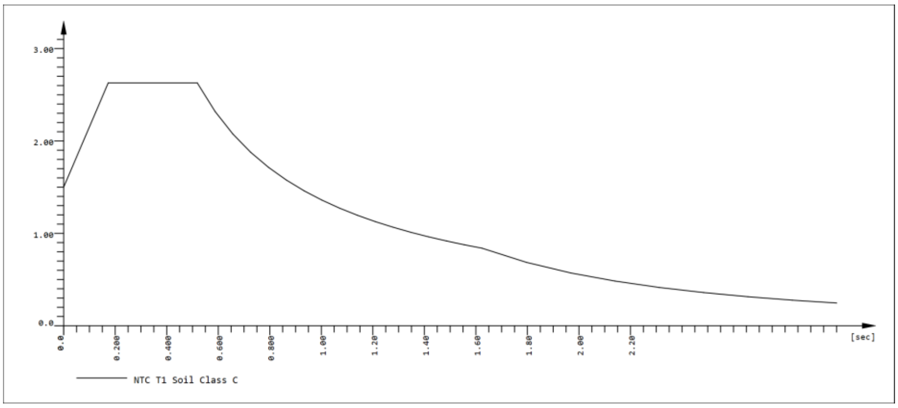


Figure 88: Design Spectrum - Casa Haus inge

Since the building is designed in a low seismic zone as well as according to the Italian construction codes, the resistance checks for the live preservation limit state (SLV) refer to the ultimate limit state and serviceability limit state verifications (Redazione DEI, 2019). Hence the structure is considered safe if the limitations set in ULS and SLS are met.

For the purpose of this graduation paper, a comparison of the equivalent horizontal force of the relived slabs and the standard slabs will be presented.

Comparison Between Full and Relived Slabs in Terms of Seismic Reaction

As it is logical to state, a smaller mass will be less effected by a seismic event. This is confirmed by consulting equation (32) (also reported below).

$$F_h = S_d(T)\frac{W\lambda}{g}$$

As it can be deduced, the mass of a building (W) is proportionally related to the horizontal action of the local earthquake.

It must be further notated that accidental loads, such as those of the furniture in the rooms and the people in the building, contribute to the excited mass in the occurrence of an earthquake. The mass in the case of standard concrete flat slabs contributing reaches a value of 46769 kN (for a detailed explanation refer to chapter 3.4.2.12).

On the other hand, considering the amount of relived surface being 56% in total and the reduced self-weight of the lightened portions (19.3 kN/m^3), the total mass of the building can be determined as follows.

$$G_{relived,1} = \left(\left(2051.3\,m^2 * 0.32\,m * 25\frac{kN}{m^3}\right) * 2\right) * 44\% + \left(2301m^2 * 0.32 * 19.3\frac{kN}{m^3}\right) = 21431.5\,kN \quad (72)$$

As stated by the Italian codes, the combination of the loads according to the seismic combination needs to be included in the following way (Redazione DEI, 2019).

$$W_{relived} = G_1 + G_n + Q_i * \psi_{2,i} \quad (73)$$

The same values estimated in chapter 3.4.2.12 are going to be applied in this section too. These refer to:

- → a total additional load of 520 kg/m^2 on the first floor
- → an additional weight of 70 kg/m^2 on the roof
- → a variable load of 300 kg/m^2

Hence, a value of approximately 35380 kN is calculated.

$$W = G_1 + G_2 + G_3 + Q_1 * \psi_{2,1} = 21431.5 + 5.2 * 2051.3 + 0.7 * 2051.3 + 3 * 2051.3 * 0.3 = 35380\,kN$$

Referring to equation (32), the horizontal seismic action can be derived.

$$F_h = S_d(T)\frac{W\lambda}{g} = 1.48\frac{m}{s^2} * \frac{35380.1kN * 1}{9.81\frac{m}{s^2}} = 5337.7\,kN$$

A reduction of around 24% of the initial force found (7039.5 kN) can be extrapolated. This means that by designing and constructing the building with relieving formwork, the earthquake action is reduced of 24% compared to using standard concrete flat slabs.

4.2.3.3. Sum of Loads

The total loads predicted and acting on project's building can be identified in Table 50.

Table 50: Predicted and Acting Loads on Building

Load Type	Description	Vaule
G	Own weight due to reinforced concrete elements	**42555.4 kN**
G_2	Permanent non-structural load	
	Green Bundle	0.7 kN/m^2
	Photovoltaic panels	0.5 kN/m^2
	Partitions	2 kN/m^2
	Windows and balcony doors	3.2 kN/m
	Parapet	2 kN/m
	Pavement	3.2 kN/m^2
	Sum	**13429.2 kN**
Q_{A1}	Accidental load	
	Residential areas (in rooms)	2 kN/m^2
	Crowded areas (in corridors)	4 kN/m^2
	Sum	**5953.4 kN**
Q_{A2}	Accidental load in balcony	4 kN/m^2
		627 kN
Q_H	Accidental load due to roof maintenance	1 kN/m^2
		2091.6 kN
S	Distributed load due to snow	1.11 kN/m^2
		2321.7 kN

4.2.4. Loaded Conditions

Note that the partial and combination factors remain unvaried compared to those shown in chapter 3.5.

4.2.4.1. ULS Load Combination

The load combination at the ultimate limit state is given by the following load combination.

$$E_d = E\left\{\sum_{j\geq1}\gamma_{G,j} * G_{k,j} + \gamma_P * P_k + \gamma_{Q,1} * Q_{k,1} + \sum_{i>1}\gamma_{Q,i} * \psi_{0,i} * Q_{k,i}\right\} \tag{74}$$

Shear Forces

The shear forces will be identified for both the slabs; nonethelss, a general outcome of higher shear forces around the bearing element is expected. This is due to the punching effect of both the columns and the bearing walls.

Shear stress may occur when positioning a floor on a pile support or a bearing wall, as in the instance of 'Casa Haus inge'. In the case that the punching shear results greater than the punching shear resistance, reinforcement is required. Moreover, since punching shear forces are localized in a reduced area around the column head or the septum head, a study of the effective diameter has been performed.[36]

Regarding the analysis of the septum's effective diameter, since the procedure is standardised for round and square columns, the wider element is analysed as a square column. This can be seen in Figure 89.

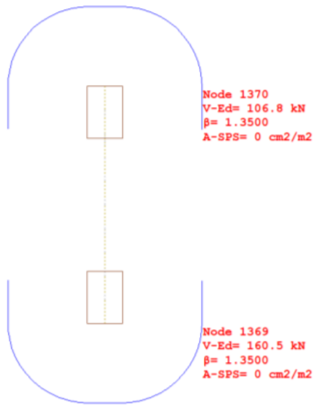


Figure 89: Punching Shear Check for bearing walls

The bearing wall's equivalent columns have been interpolated as two outer columns with square base (see Figure 89). Moreover, the FEM software shows the shear force experienced (V_{Ed}), the eccentricity factor (β) and the required punching shear reinforcement (A_{sps}). As seen in this case, the shear resistance is sufficient as to avoid the placement of shear reinforcement above the bearing wall.

Inevitably, the higher shear forces are shown in the zones where supports are designed; this can be seen in both Figure 90 and Figure 91.

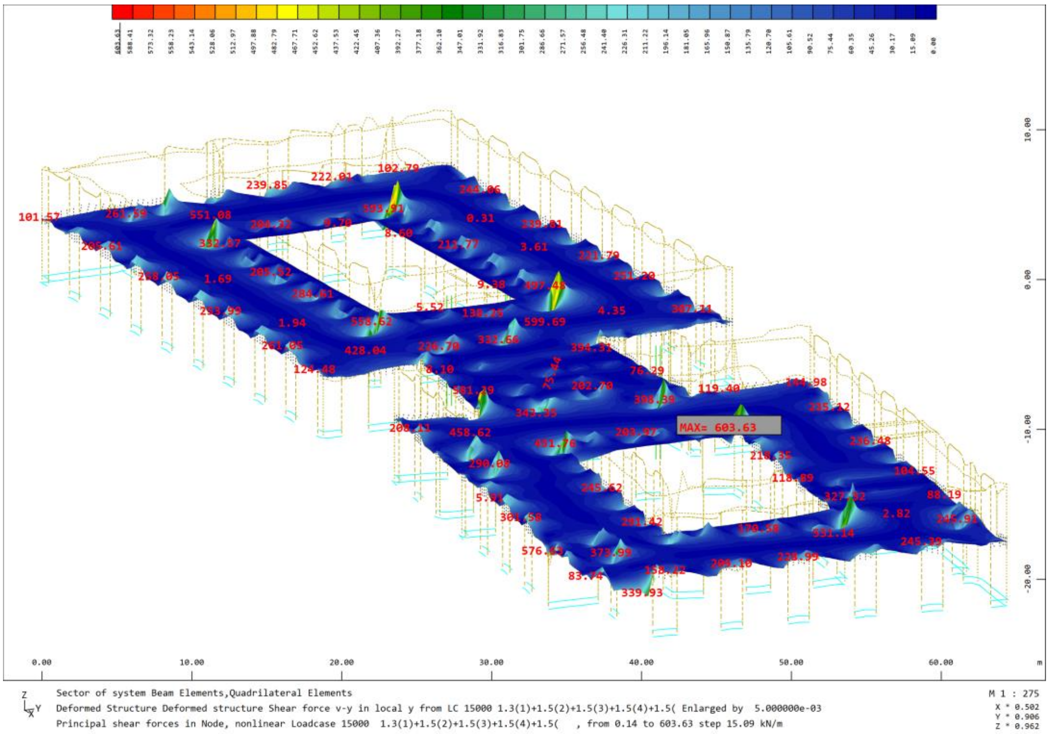


Figure 90: Shear Forces - First-floor slab

[36] Refer to Appendix 12 – Results of the Lightened Nursing Home FEM Model.

Regarding the first-floor slab 8 zones above bearing walls have been identified with high shear forces, requiring punching shear reinforcement.[37] These shear forces are identified by values above 500 kN in Figure 90.

Similarly, in the roof slab high values of shear forces have been identified along the bearing walls.

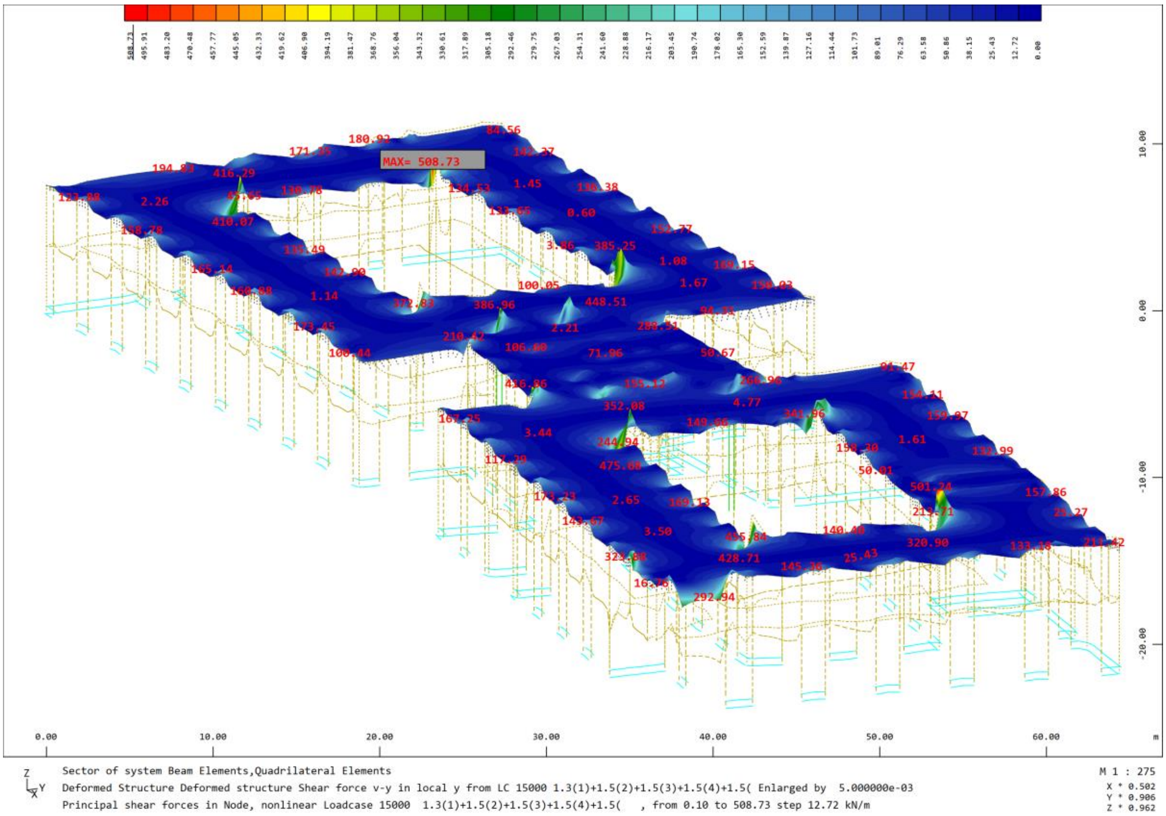


Figure 91: Shear Forces - Roof Slab

Of all the zones above supports, eight have been analysed to required punching reinforcement.[38] These are shown in Figure 91 as all the zones with shear forces exceeding the value of 390 kN.

Bending Moments

Conversely to the model analysed previously (refer to chapter 3.5) the slabs in this structure are characterised by a bidirectional nature. This means that bending moments occur in the two directions.

As it can be seen in Figure 92, the bending moments in the first-floor slab behave dually (see upper part for moments around the y axis and lower part for the moments around the x axis of the system).

Regarding the values around the y axis, a maximum positive moment above a supporting wall of 334.7 kNm is reached. Whereas a minimum value of 118 kNm can be seen in one of the building's outer corners. This translates in a high vertical deformation value as well as in a high reinforcement value required in the zone.

[37] Refer to Appendix 12 – Results of the Lightened Nursing Home FEM Model.

[38] See above.

Similarly, the moments around the x axis reach a maximum positive value of 354.5 kNm above the same support as mentioned previously. Conversely, the minimum moment is experienced mid-span of the slab, with a moment of 101.8 kNm. It is expected to require bending reinforcement in this area.

For the floor slab a similar trend compared to the first floor is expected; nonethelss, since the loads are lower and no other load is transferred from above, the values may be lower.

As it can be seen in Figure 93, the bending moments around the y axis reach a maximum value of 238 kNm and a minimum value of 115.7 kNm. It is interesting to note that these moments occur in the same locations as in the firs-floor slab. Hence the same requirements and results are respectively expected.

In a similar manner, the moments around the x axis follow the same trend as the moments in the y direction. This means that the location of their peak moments is the same compared to the first-floor slab, but their values are lower (see Figure 93). In fact, the maximum value reaches 243.8 kNm whereas the minimum moment is of 78.2 kNm.

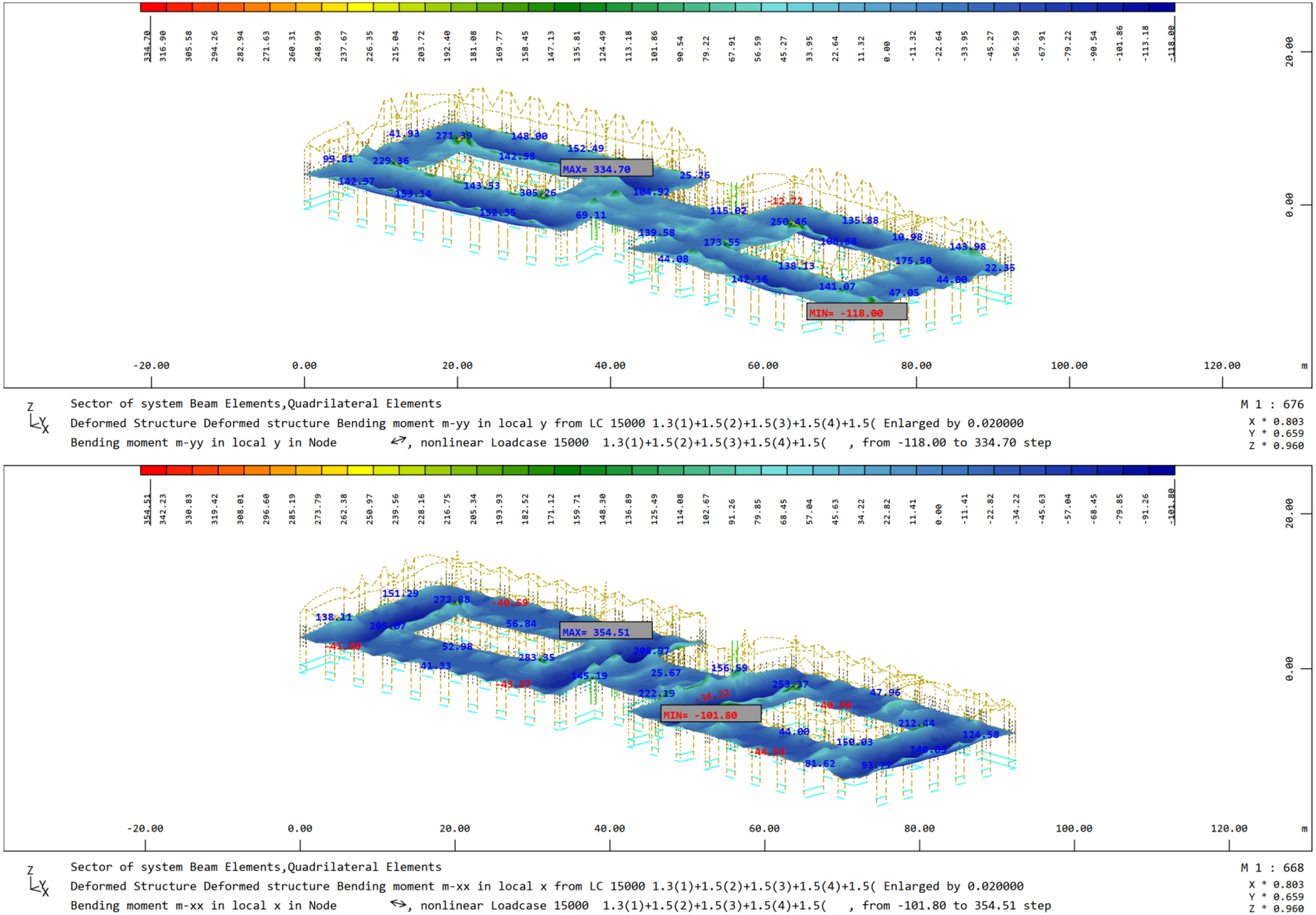


Figure 92: Bending moments - first-floor slab

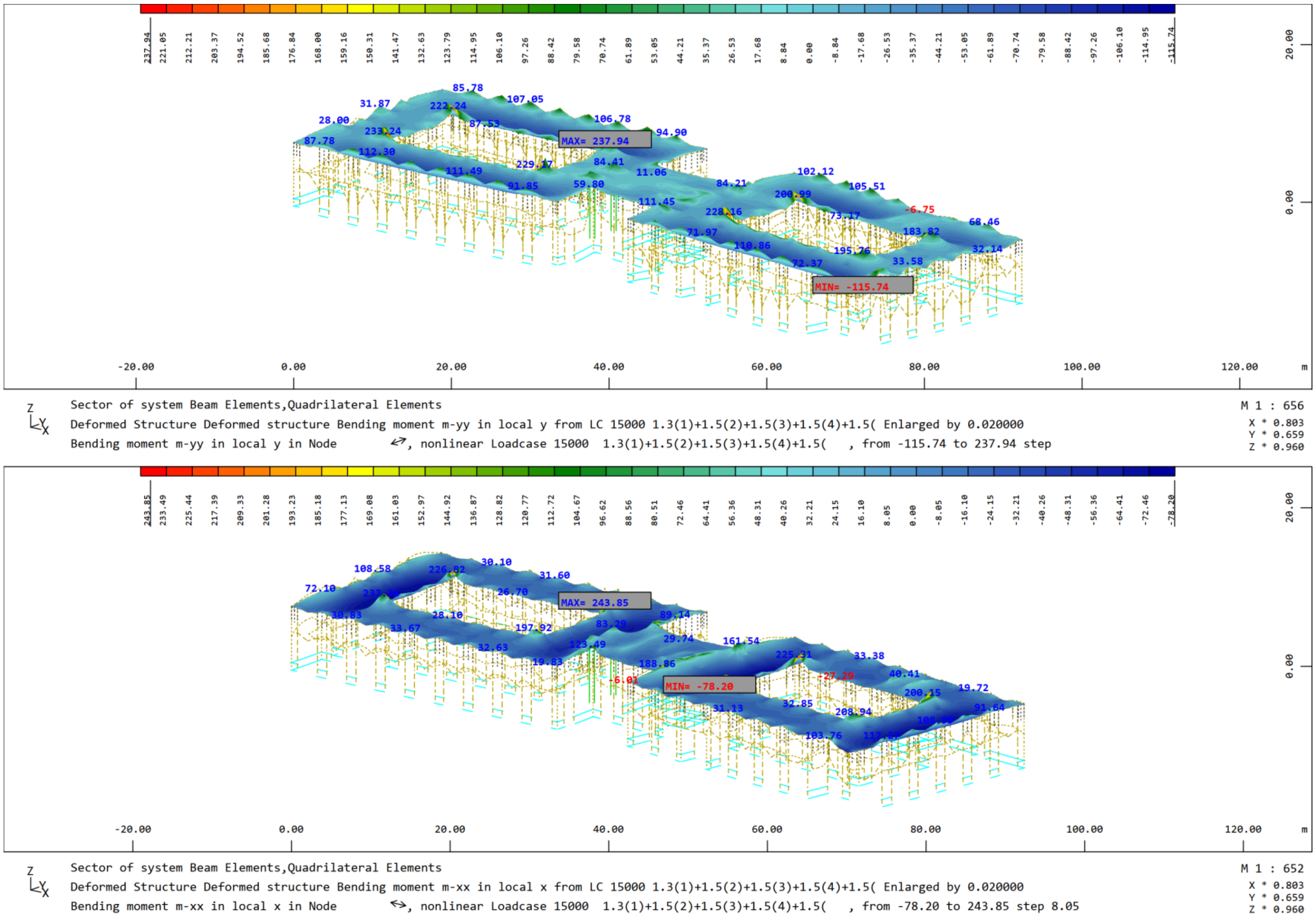


Figure 93: Bending moments - roof slab

Support Reactions

The support reactions are important to estimate and analyse for two reasons:

- → These determine the bearing capacity necessary for designing the foundation
- → These are affected as the overall weight has been lightened

For this reason, a comparison between the reaction forces due to the full slabs and the lightened model will be presented in this chapter.

In Figure 94, the support reaction of the building, generated from all the structural element – including slabs designed as full concrete flat slabs – as well as the external loads applied, can be visualised.

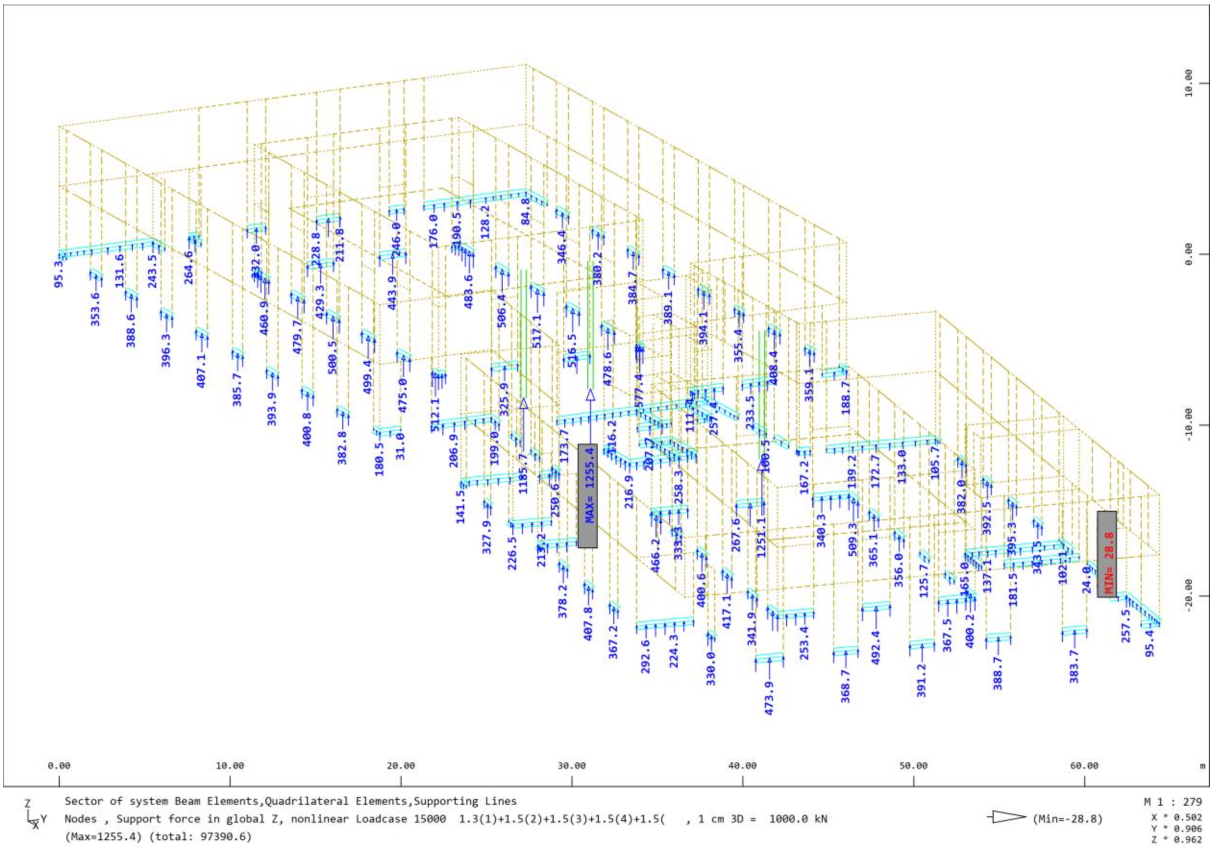


Figure 94: Support reactions - standard model

As it is noticeable, the support reactions are determined along the width of each septum, for this reason the comparison will be performed focusing on the maximum reaction force produced. This is located on of the columns of the structure and reaches a value of 1255.4 kN.

This force, as mentioned previously, together will the rest of the support reactions determined in the structure, will determine the type of foundation as well as the bearing capacity these should be designed to support. Reasonably, by lightening some elements of the structure, the overall loads generated would decrease to an extent, resulting in an inferior bearing capacity required for the foundations. Lastly, a smaller bearing capacity would allow the foundation structure to be reduced, decreasing the overall costs of the construction.

As predicted, the support reactions, produced by the same structural system with the difference of relived slabs (see Figure 95), result lower than those seen in Figure 94.

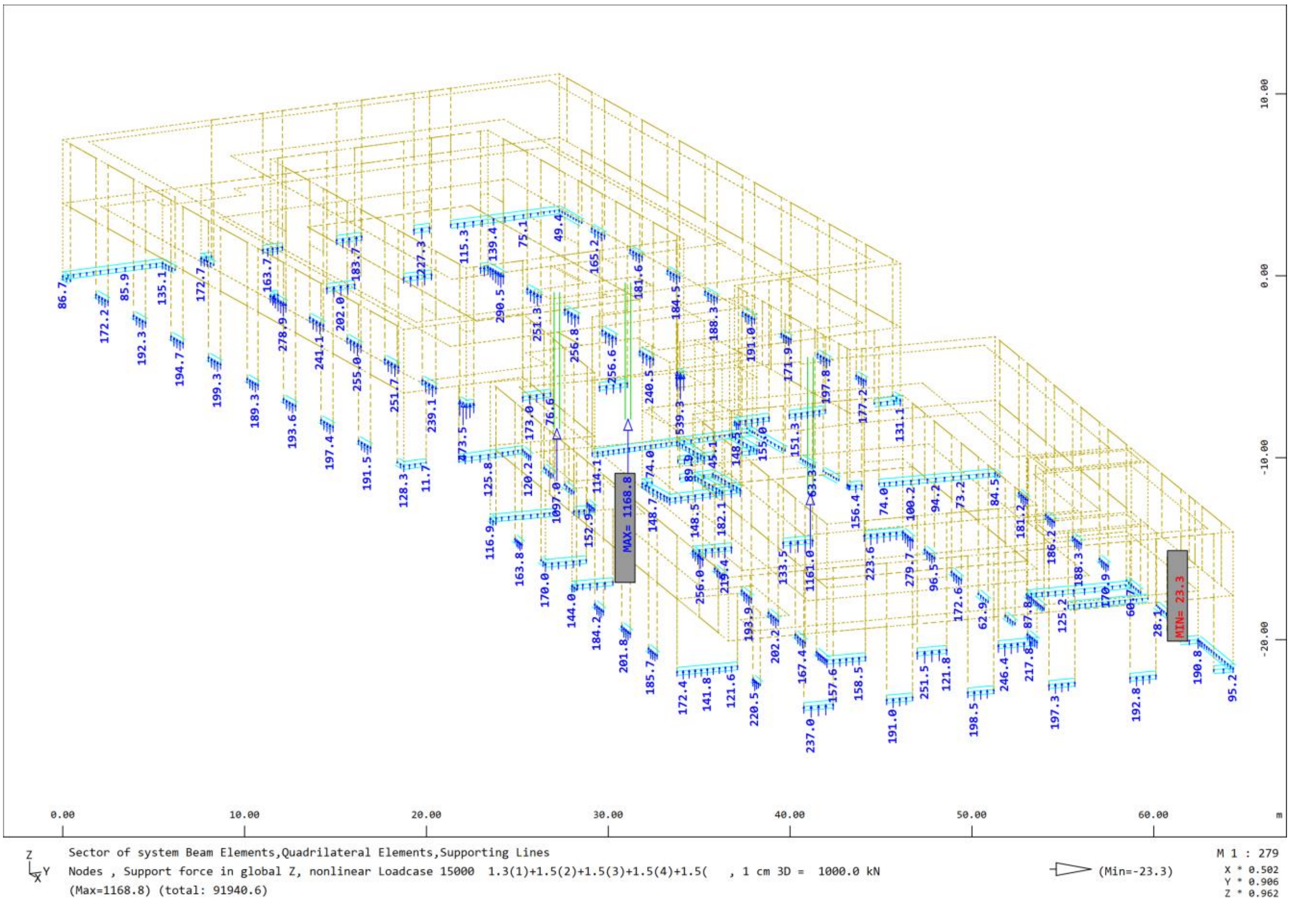


Figure 95: Support reactions - lightened model

Comparing the maximum reaction forces found in the central column of the structure, it can be seen that the force decreased of approximately 87 kN, reaching to a value of 1168.8 kN. Decreasing the maximum value of 7% compared to the standard non lightened model is considered beneficial to the foundation design.

Reinforcement

Given the bidirectionality of the slab's reinforcement is expected both in the principal and cross direction. Moreover, as punching shear occurs at an elevated rate around the supporting elements, upper reinforcement is predicted to be placed on the zones where columns or bearing walls are placed. Nonetheless, punching reinforcement might be additionally required in those sections as well.

Lastly, it must be noted that the company Aig Associati and Partner follows a general approach regarding reinforcement for flat slab. This is described by the positioning of a general reinforcement mesh in both directions of ϕ12-15. For this reason, it is critical to subtract the already assumed reinforcement area of 7.46 cm^2/m to the value determined with the FEM software.

Upper Principal Reinforcement

The upper principal reinforcement will span following the x-axis in the system of the structure in this report and will be positioned according to Figure 96 (in the top section of the picture, the roof slab is depicted, whereas in the bottom section, the first-floor slab is shown).

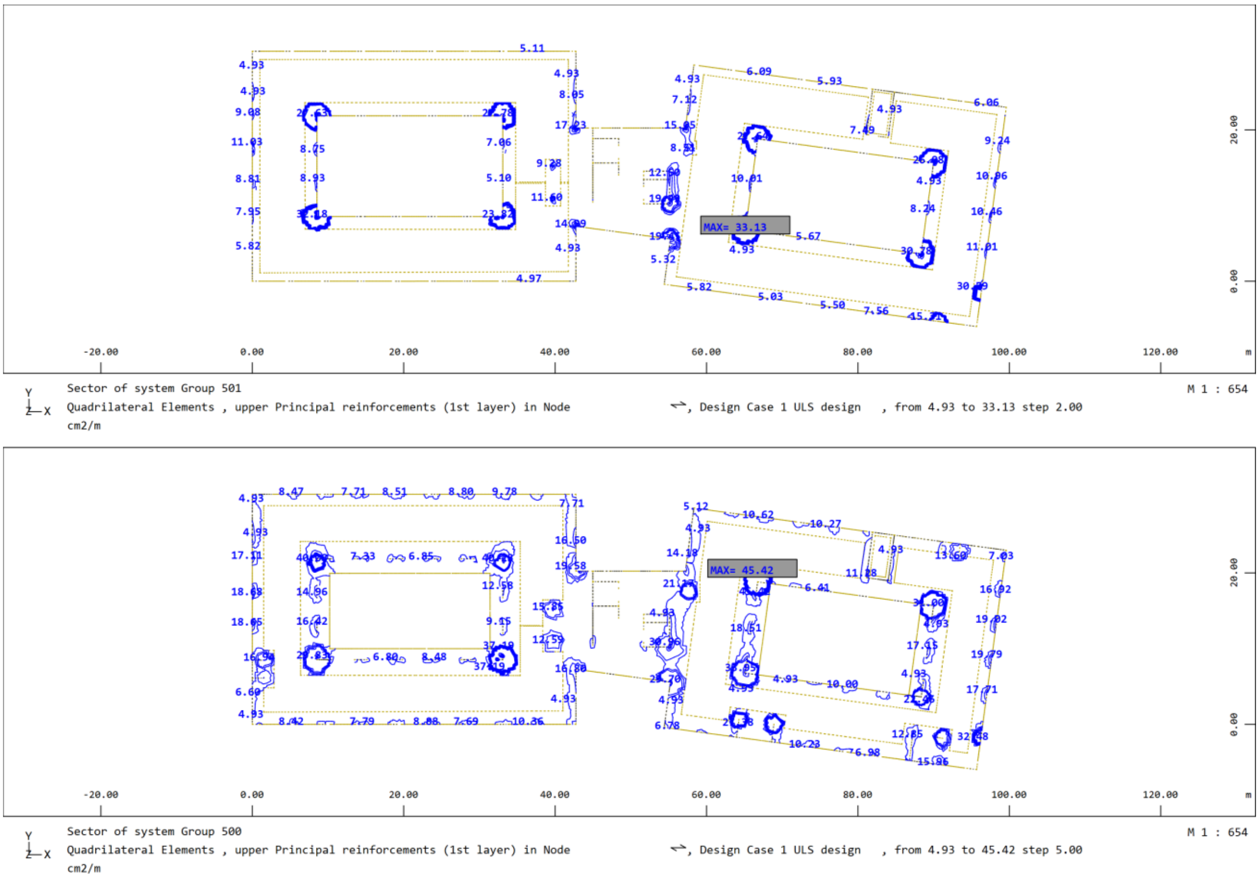


Figure 96: Upper principal Reinforcement

The necessary reinforcement for the upper slab ranges from 4.9 cm^2/m to a maximum of 33.13 cm^2/m and 45.42 cm^2/m for the lower slab. Nonethelss, as mentioned previously, the base mesh of 7.46 cm^2/m must be deducted from the values found. Thus, the required reinforcement is reduced to 25.7 cm^2/m for the roof slab and 37.9 cm^2/m for the first-floor slab.

Recalling the procedure for determining the steel reinforcement's details (equations (47),(48) and (49)), the characteristics for the reinforcement will be type ϕ22-15 for the roof slab and ϕ28-15 for the first-floor slab.

Lower Principal Reinforcement

The lower main reinforcement extends following the x-axis in the system of the structure in this report and will be positioned according to Figure 97 (the upper section of the image shows the roof slab, the lower section shows the first-floor slab.

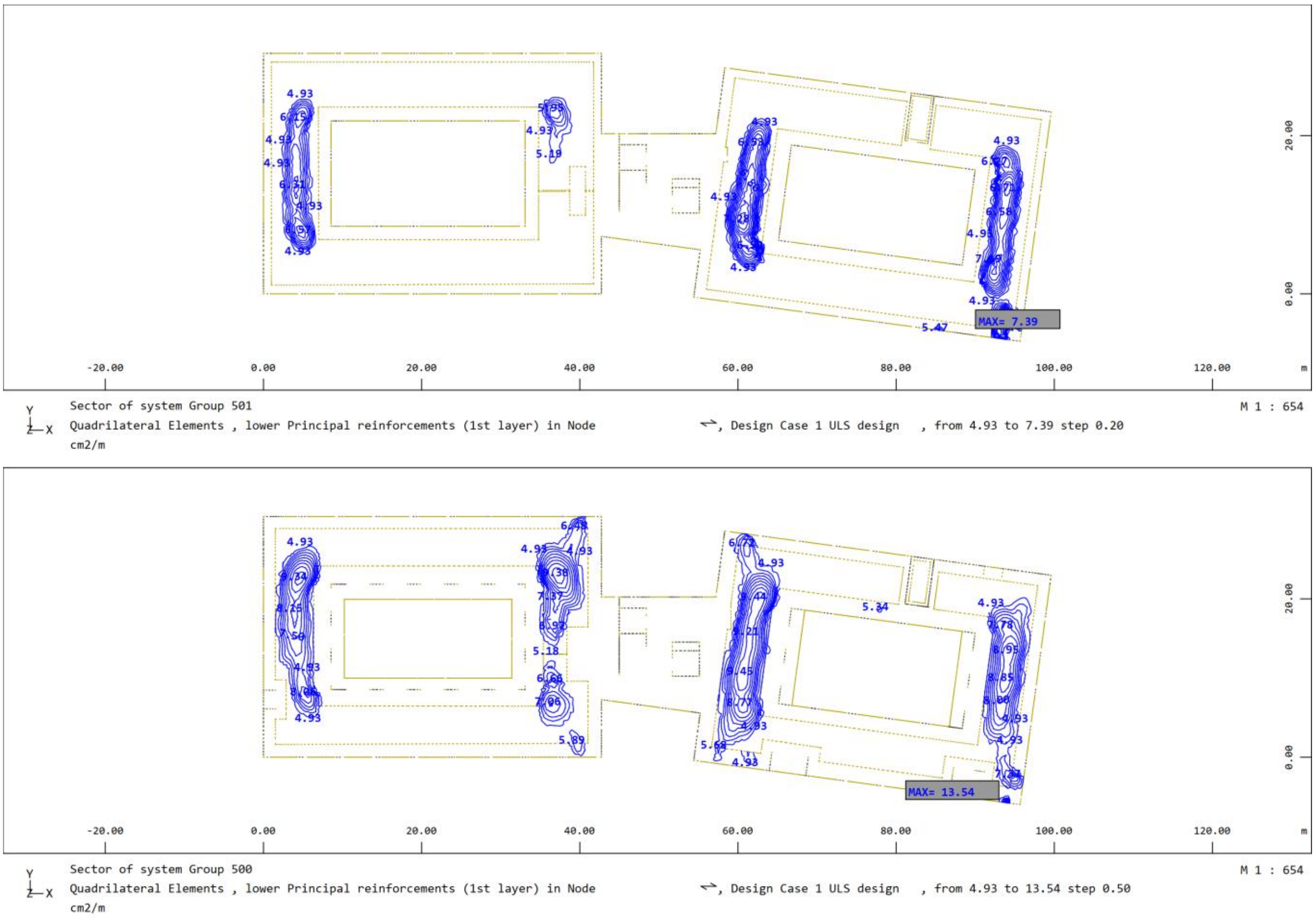


Figure 97: Lower principal reinforcement

The required reinforcement ranges from 4.93 cm^2/m to a maximum of 7.39 cm^2/m for the upper slab and 13.54 cm^2/m for the firt-floor slab. Nonethelss, the base mesh of 7.46 cm^2/m must be deducted from the values found. Thus, the required reinforcement is not necessary for the roof slab and is reduced to 6.08 cm^2/m for the first-floor slab.

Recalling the procedure for determining the steel reinforcement's details (equations (47),(48) and (49)), the characteristics for the reinforcement will be type ϕ12-15 for the first-floor slab.

Upper Cross Reinforcement

The upper transverse reinforcement runs along the y-axis in the system of the structure in this report and will be positioned according to Figure 98 (the upper section of the diagram shows the roof slab, the lower section the first-floor slab).

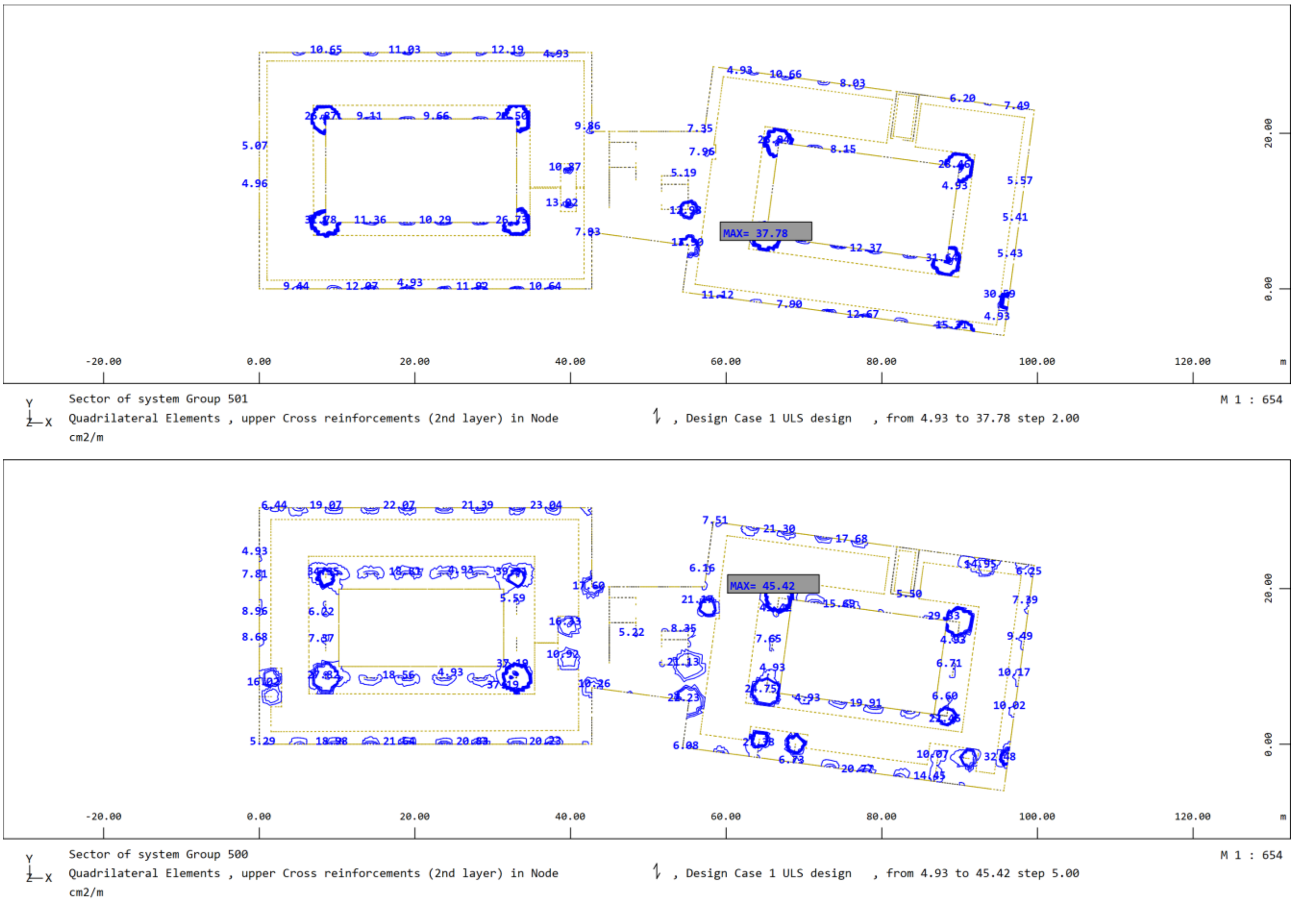


Figure 98: Upper Cross reinforcement

The necessary reinforcement ranges from 4.9 cm^2/m to a maximum of 37.78 cm^2/m for the upper slab and 45.42 cm^2/m for the lower slab. However, the base mesh of 7.46 cm^2/m must be deducted from the values found. Hence, the required reinforcement is reduced to 30.32 cm^2/m for the roof slab and 37.9 cm^2/m for the first-floor slab.

Recalling the procedure for determining the steel reinforcement's details (equations (47),(48) and (49)), the characteristics for the reinforcement will be type ϕ24-15 for the roof slab and ϕ28-15 for the first-floor slab.

Lower Cross Reinforcement

The lower cross reinforcement courses alongside the y-axis in the structure system of this report and will be positioned according to Figure 99 (the upper section of the diagram shows the roof slab, the lower section the first-floor slab).

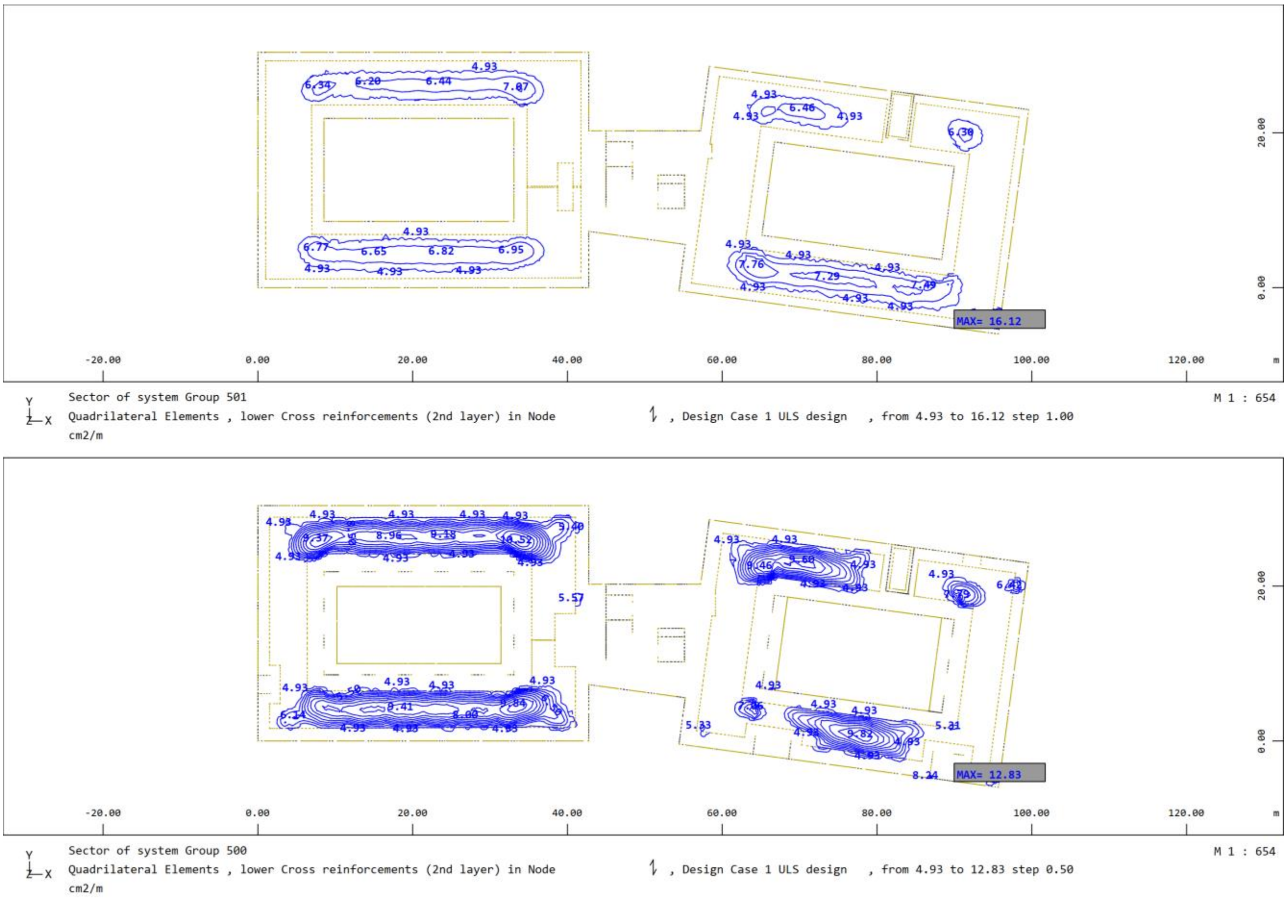


Figure 99: lower cross reinforcement

The necessary reinforcement for the roof slab ranges from 4.93 cm^2/m to a maximum of 16.12 cm^2/m and 12.83 cm^2/m for the firs-floor slab. Nevertheless, the base mesh of 7.46 cm^2/m must be deducted from the values found. Therefore, the required reinforcement is reduced to 8.66 cm^2/m for the roof slab and 5.37 cm^2/m for the first-floor slab.

Recalling the procedure for determining the steel reinforcement's details (equations (47),(48) and (49)), the characteristics for the reinforcement will be type ϕ14-15 for the roof slab and ϕ12-15 for the first-floor slab.

It is interesting to note that the maximum reinforcement in the cross direction is required on both slabs on the corner which is not supported by bearing walls from the foundation level.

As a rule, in structural engineering, whatever structural element is found on the upper storeys must be supported by an element of minimum equal dimensions in the lower storeys. This is not the case in this specific zone, as per the architect's desire to keep an outdoor area, as shown in Figure 100.

Figure 100: Zone not supported by bearing elements from the foundation level

Since the bearing elements starting from the foundation level are lacking in this zone, it was expected to require additional reinforcement. Additionally, lightening the slabs in the structure enabled no further measures for safety. In fact, without the hollow formwork installations, supporting beams had to be designed to carry the load and spread it to the bearing elements in a safe manner.

4.2.4.2. SLS Load Combination

Frequent Combination

The load combination at the frequent serviceability limit state is given by the following load combination.

$$E_{d,freq.} = E\left\{\sum_{j\geq 1} G_{k,j} + P_k + \psi_{1,1} * Q_{k,1} + \sum_{i>1} \psi_{2,i} * Q_{k,i}\right\} \tag{75}$$

Quasi-Permanent Combination

The load combination in the quasi-permanent or long-term serviceability limit state is given by the following load combination.

$$E_{d,perm.} = E\left\{\sum_{j\geq 1} G_{k,j} + P_k + \sum_{i>1} \psi_{2,i} * Q_{k,i}\right\} \tag{76}$$

Rare Combination

The load combination at the rare (or characteristic) limit state is given by the following load combination.

$$E_{d,rare} = E\{\textstyle\sum_{j\geq 1} G_{k,j} + P_k + Q_{k,1} + \sum_{i>1} \psi_{0,i} * Q_{k,i}\} \tag{77}$$

Crack Control

It is important not exceed the following values in the different load cases:

- → In frequent combination of actions *≤ 0.40 mm*
- → In quasi-permanent combination of actions *≤ 0.30 mm*

The analysis for the first-floor slab shows that both load conditions – frequent and long-term – are verified and safe. In fact, as seen in Figure 101, the cracks in frequent combination (see upper part of picture) have a maximum value of 0.40 mm. On the other hand, the crack width in quasi-permanent combination (see lower part of picture) do not exceed a maximum value of 0.30 mm. Hence, safety is reached.

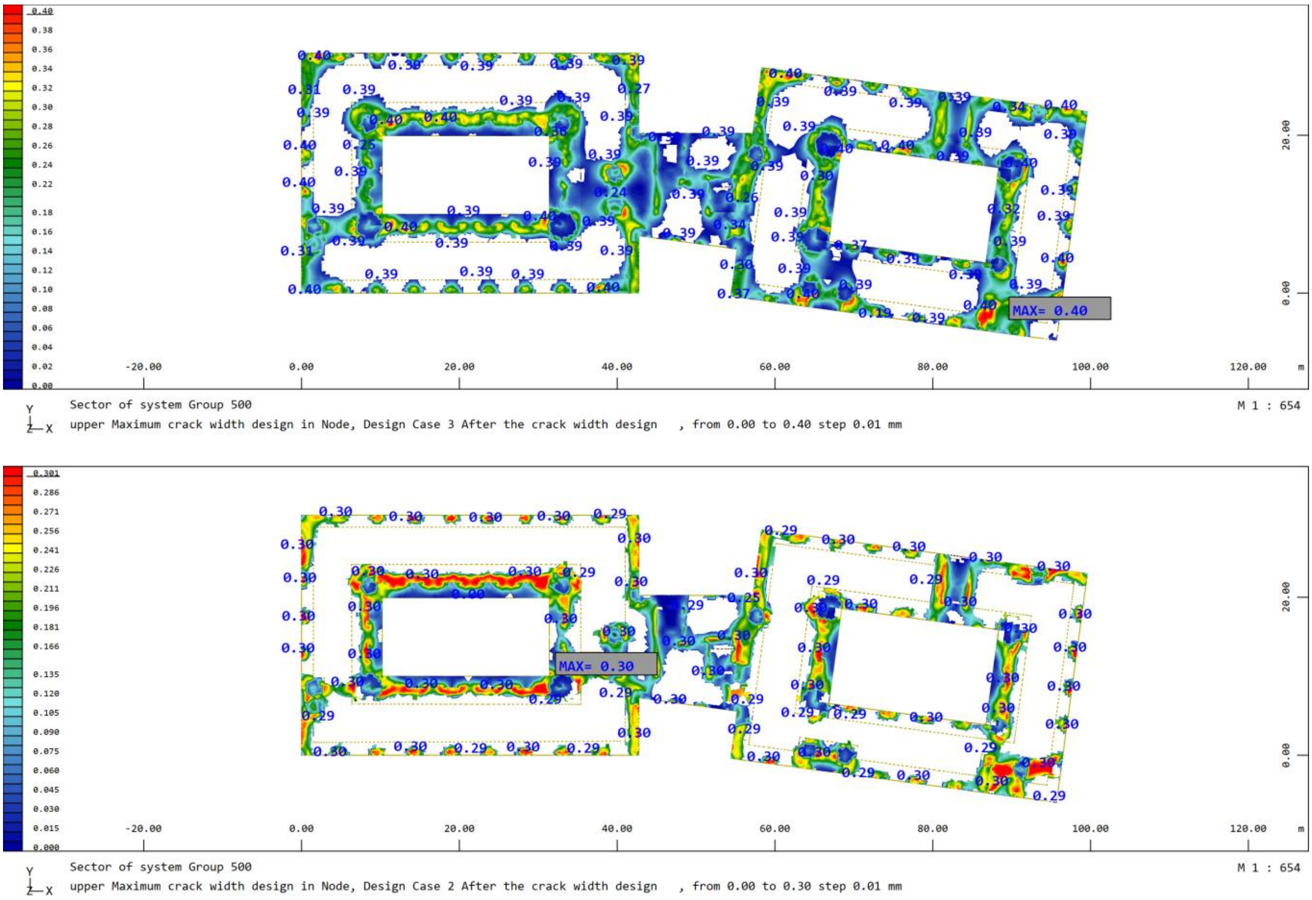


Figure 101: Cracks Expected in first floor slab

Regarding the roof slab, similar patterns but in a less distributed matter can be expected for cracks in the concrete, as shown in Figure 102.

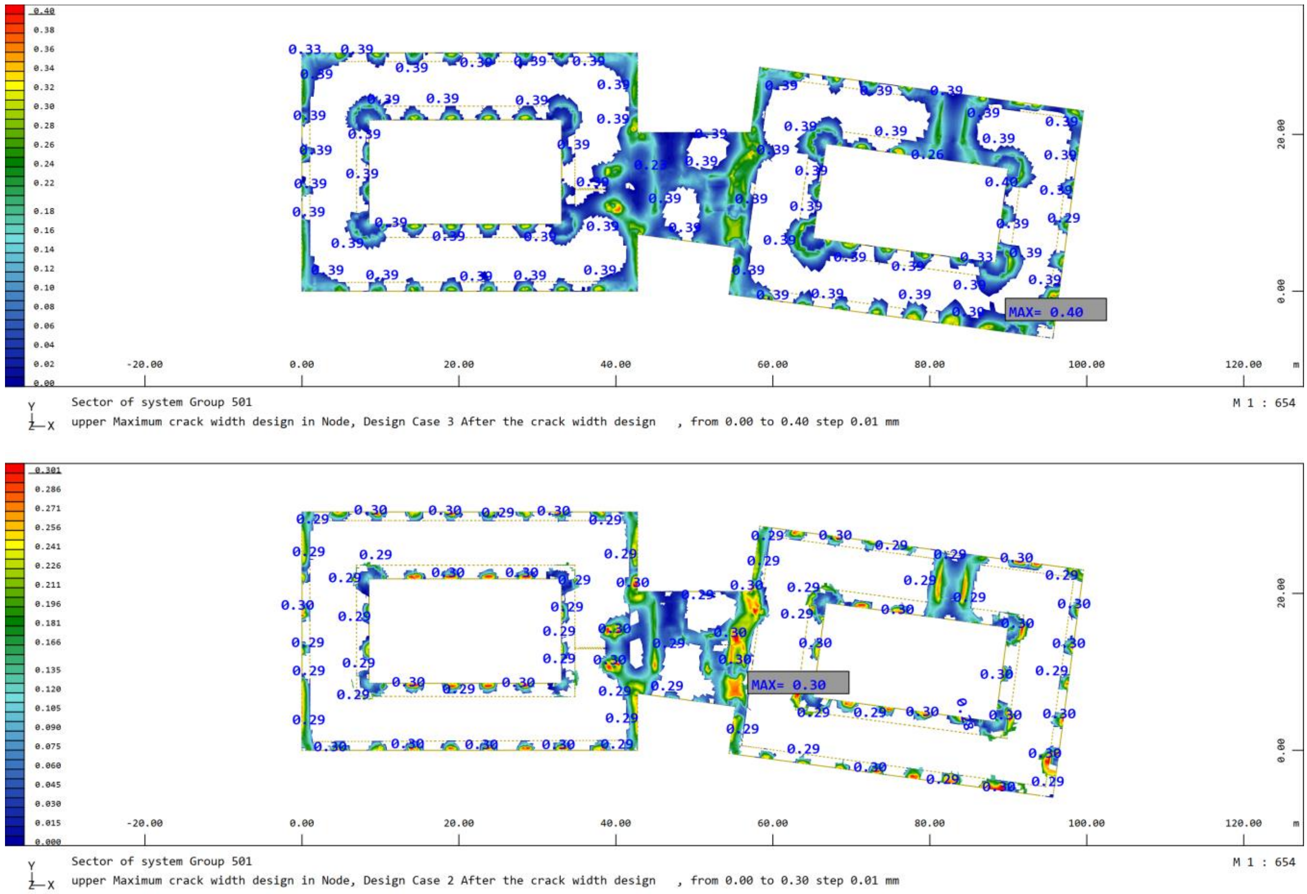


Figure 102: Cracks expected in roof slab

Even though cracks may occur, their width is within safety limits; in fact, in frequent combination (upper section of Figure 102) cracks reach a value of 0.40 mm, whereas in quasi-permanent combination (lower section of picture) these do not exceed a width of 0.30 mm.

Concrete Tension Checks

According to the Eurocodes the maximum tension values in the concrete elements should not exceed (European Commission, 2006):

- → For the characteristic SLS combination $\sigma_{c,max} \leq 18.4\, MPa$
- → For the long-term SLS combination $\sigma_{c,max} \leq 13.8\, MPa$

As seen in Figure 103, the values reached in the first-floor slab amount to a maximum compression stress of 15.52 MPa in rare combination and of 13.27 MPa in quasi-permanent combination. Both the values are within the safety limits.

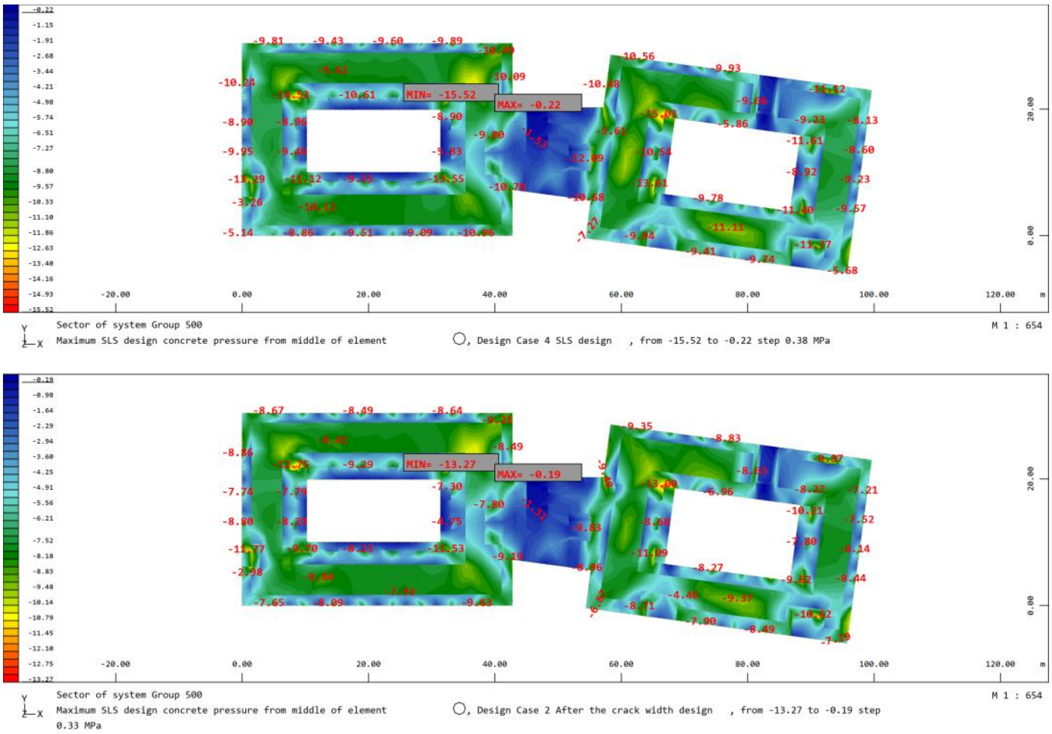


Figure 103: Concrete tension - first-floor slab

On the other hand, the values in the roof slab reach higher stresses; nonethelss, still meeting the safety limits. In fact, as depicted in Figure 104, the stress values in characteristic load combination amount to 17.1 MPa and those in long-term combination reach the limit of 13.8 MPa.

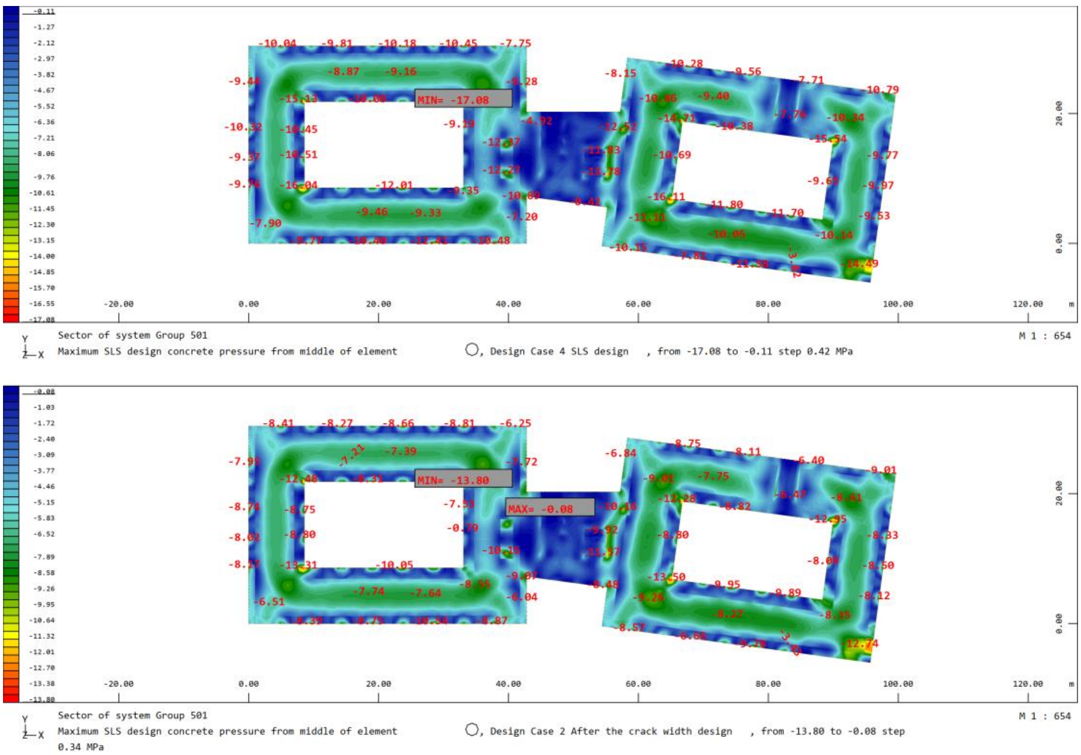


Figure 104: Concrete tension - Roof slab

Noticing that both slabs are within safety limits, but some values reach the maximum allowable limit, it can be stated that lightening the slabs brough benefits to the structure regarding safety. In fact, it is logical to state that the limits would have been surpassed if the hollow plastic bodies were not to be included.

Steel Tension Checks

To fulfil the safety requirements the following steel tension value should not be surpassed:

$$\sigma_{s,max} \leq 360\ MPa$$

As it can be noticed in Figure 105, the first-floor's slab reinforcement bars do not exceed the maximum tension allowed, in neither of the cases. In fact, the maximum tension experienced is 184.7 MPa.

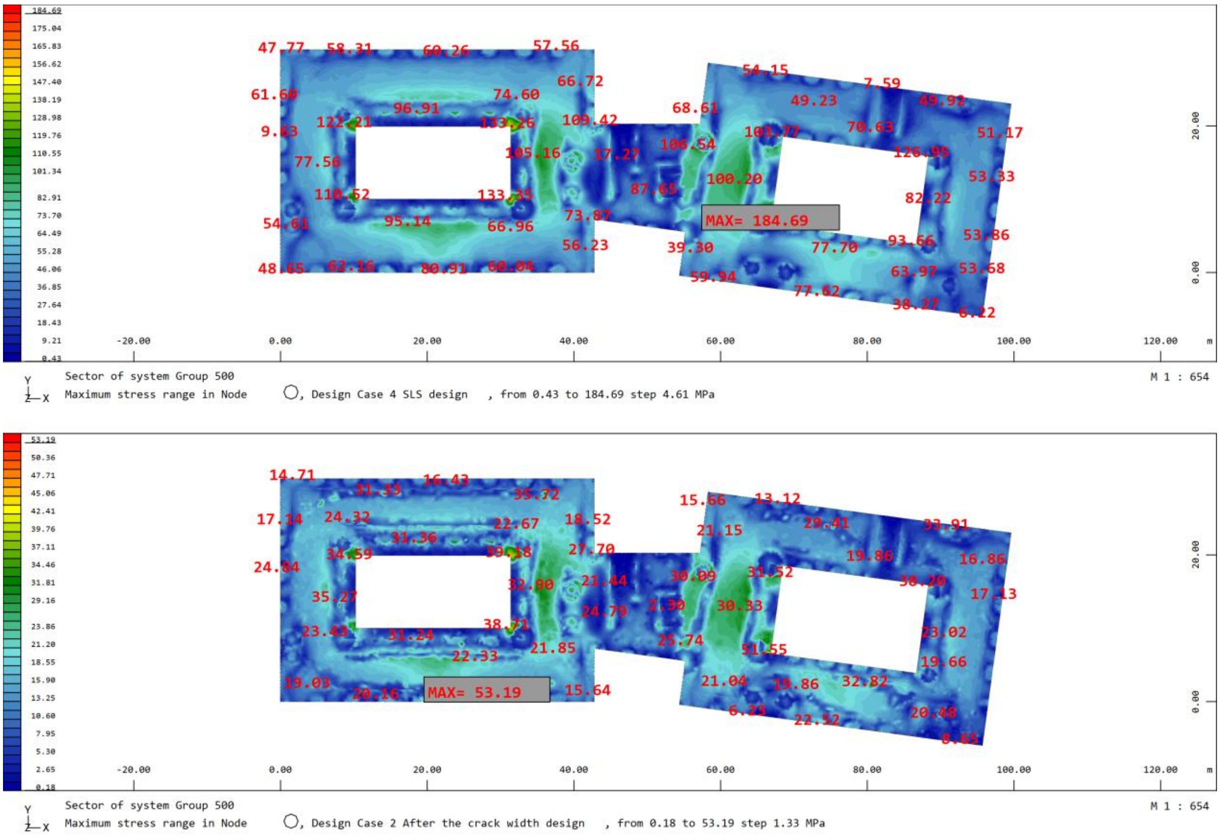


Figure 105: Steel tensions in first-floor slab

Equally, in the roof's slab reinforcement, safety is met as the maximum tension reaches a value of 118.3 MPa.

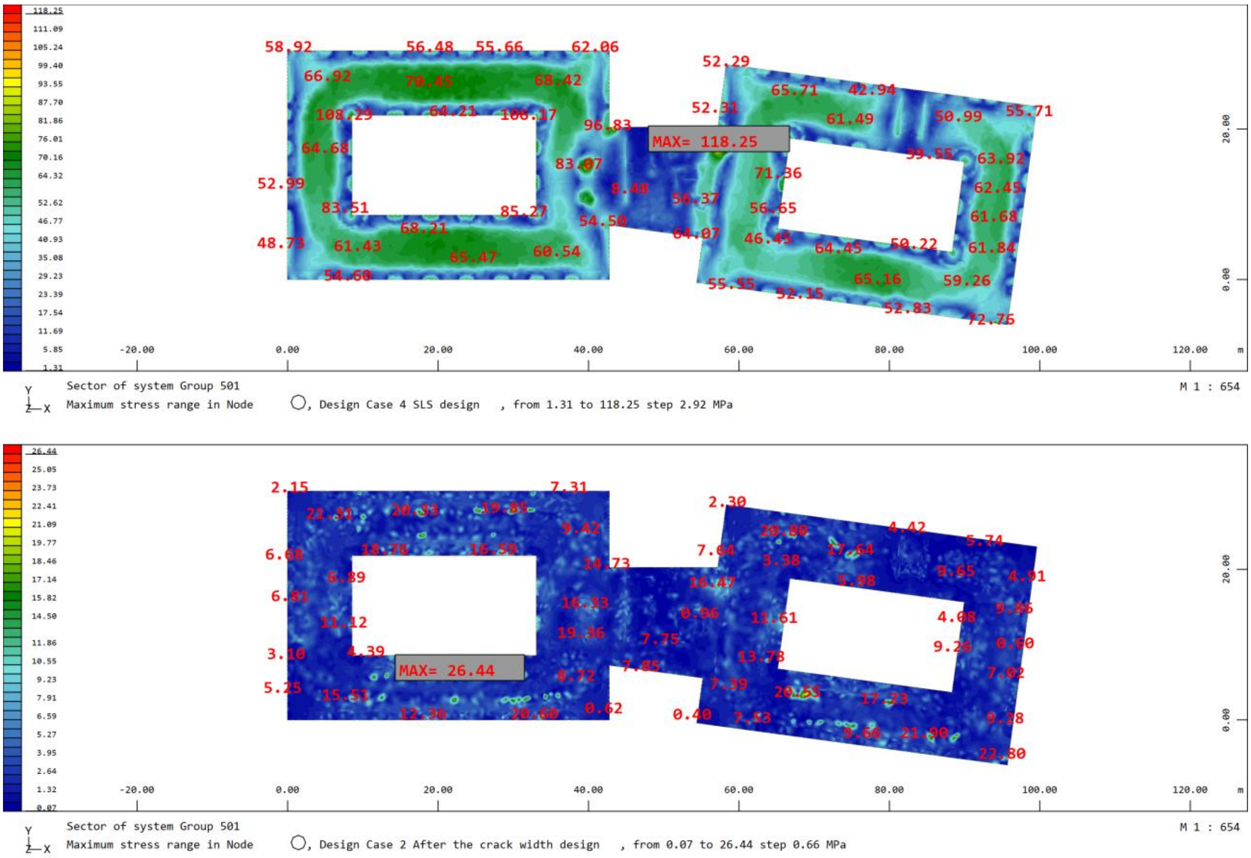


Figure 106: Steel tensions in Roof slab

Deformation Checks

Conversely to the deformation check performed on the section of the structure in chapter 3.5.12.2, the building considered in this case presents many elements. Therefore, the overall aesthetic of the building could be compromised by the deformations and damages to adjacent parts may occur.

In the case of 'Casa Haus inge' on the first floor as well as on the ground floor partition walls have been designed as well as glass doors and windows spanning through the whole height. This means that if both the slabs deform excessively, these elements could be negatively affected.

For this reason, a limit of *span/500* is considered in quasi-permanent loads and should not be exceeded.

As depicted in Figure 107, the deformations in the building reach a maximum of 4.76 mm in the roof slab and of approximately 6 mm in the first-floor slab. The respective spans of the floors are 10.7 m and 9.6 m, hence the maximum allowable vertical deformations are 21.4 mm in the section considered in the roof slab and 19.2 mm for the portion of first-floor slab analysed. This results in safety being reached in both cases.

Moreover, a critical point in this development must be additionally analysed: the unsupported corner (see Figure 100). In fact, this section of the slab deforms majorly and presents high bending moments in the y-direction, as seen in chapter 4.2.4.1.

Its span has a value of 5.4 m meaning that the maximum possible deformation is of 10.8 mm.[39] Since the deformation amounts to 4.2 mm in the roof slab (see upper part of Figure 107) and it reaches a value of 3.6 mm in the first-floor slab (see lower part of the picture), safety is met in all cases.

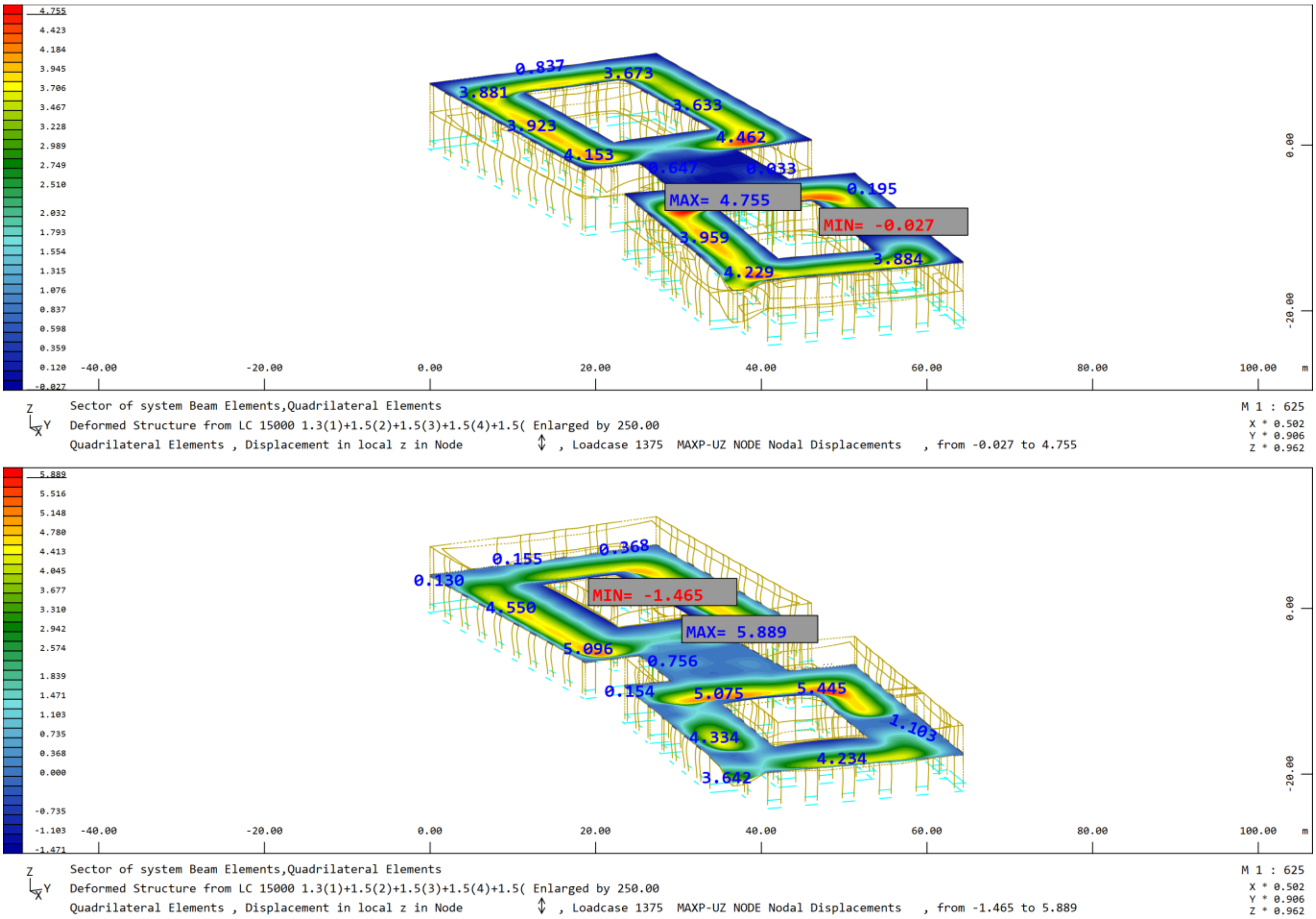


Figure 107: Vertical deformations in the slabs

[39] It must be noted, that since no adjacent element are designed in the zone of the unsupported corner, the limit value of span/250 could have been considered. Nonetheless, since the zone is considered a critical zone, the more conservative value of span/500 is used.

5. Discussion and Recommendations

5.1. Sandwich Material

It is of utter importance to state that the sandwich material approach, despite being considered the second choice due to its elevated designing times recorded, is considered the most optimum and accurate one. This can be stated owning to the fact that it is possible to bypass the CADiNP text input by converting this into a 'user-friendly' interface on SOFiSTiK directly.

Doing so, the designing times would drastically decrease and would therefore be considered the winning variant.

This implementation has not been proposed in the research paper because of the limited amount of time of the graduation phase and the various deadlines. It is, therefore, highly recommended to research further into this matter to determine the effectiveness of the sandwich material approach.

5.2. Foundation Design

Since the analysis and design of the underground elements were out of scope in this graduation paper, these have not been included. Nonethelss, as mentioned briefly in in chapter 4.2.4.1, the foundations are heavily influenced by the choice of lightening a slab or not. In fact, it is expected to require less concrete and be able to design different types of foundations compared to if the structure excluded reliving bodies. For this reason, it could be interesting to determine the foundation type and requirements as well as analyse the definite benefits of reliving structural elements of part of their weight. It is therefore recommended to further expand the research.

6. Conclusion

As seen in the various chapters of this final thesis report, lightweight slabs are a widely used technology in construction due to their numerous advantages.

Despite the common misconception that strength is lost by extracting part of the volume of the materials from their elements, it has been proven that safety is met in every situation. In some cases, it is necessary to take heavier measures than a standard unreinforced slab, such as reinforcement requirements. In other cases, for instance with regard to concrete compressive stresses, lightweight slabs have made it possible to achieve safer situations, which otherwise would not have been achieved. In fact, overall, the main advantage of the inclusion of hollow plastic bodies in the slab is the reduction of self-weight, which decreases the seismic action on the building, reduces the amount of concrete required for both above- and underground structures, and contributes to meeting safety requirements. However, modelling a raised flat slab using the finite element method can be challenging.

Regarding the problem just described, many solutions can be found, and this research paper aims to establish the most efficient and least time-consuming approach.

Based on five criteria that were important to the client and the project itself, a solution was identified. Of the three alternatives, describing the modelling of the lightened flat plate with reduced bending strength, with a fictitious thickness and with a layered material, the first - the modelling of the lightened flat plate with reduced bending strength and self-weight - proved to be the most efficient and quickest solution.

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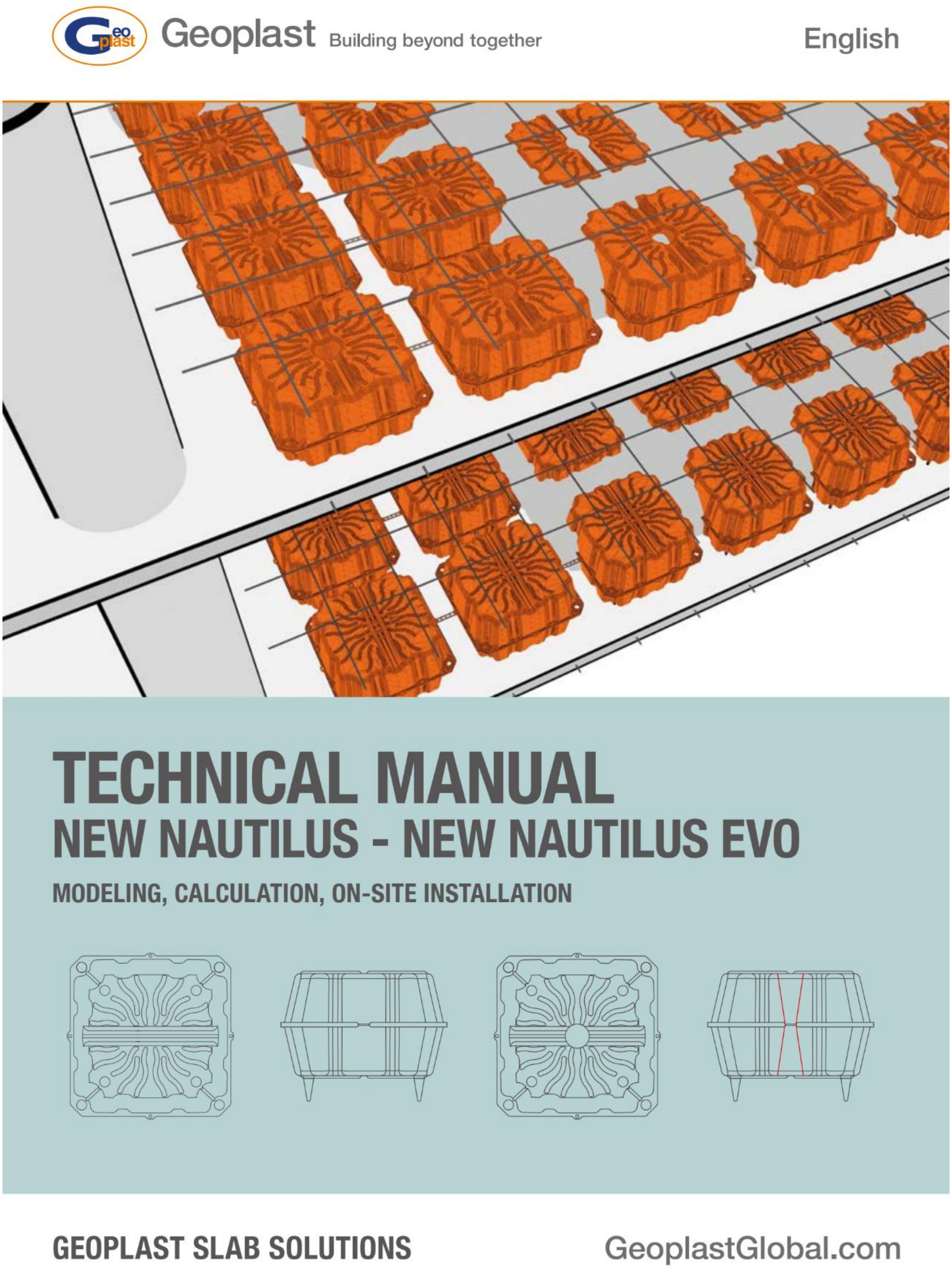
8. Appendices

8.1. Appendix 1 – Geoplast Technical Data Sheet

Note that due to the extensiveness of the manual (total of 68 pages) only the sections applied in the graduation project have been attached.



Geoplast Building beyond together

English

TECHNICAL MANUAL
NEW NAUTILUS - NEW NAUTILUS EVO
MODELING, CALCULATION, ON-SITE INSTALLATION

GEOPLAST SLAB SOLUTIONS

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INDEX

NEW NAUTILUS EVO TECHNICAL MANUAL

1. INTRODUCTION

1.1 SLABS

Nowadays, reinforced concrete slabs are very common structures in the construction market. Although the traditional beam slabs and hollow block slabs are still very used, other technologies have certainly gained the upper hand.
If we get a closer look at what happens around the world, beam slabs remain in use in the countries where the cost of materials are higher than labor costs.
On the other hand, the richest and the most industrialized countries, mostly use deck plate systems.
Reinforced concrete slabs:

1.1.1 STRENGTHS

The reasons are easily identifiable, as the reinforced concrete plates:

1. they are very solid thanks to the side deformation prevention, which allows them to deform less and reduce thickness:
   - **a** thickness reduction allows the economization of materials;
   - **b** massing reduction, allows the maximization of the ground surface exploitation, which represents an important cost;
2. there is no need of beams:
   - **a** once again they allow the reduction of the volumetric footprint of the deck;
   - **b** evitano tempi e costi di casseratura delle travi;
   - **c** facilitano il passaggio degli impianti, riducendone notevolmente i tempi di posa;
3. avoid the scaffolding of the beams:
   - **a** facilitate the passage of the installations, significantly reducing installation times;
   - **b** meshes and straight bars are easier and faster to install;
   - **c** it is possible to use pre-fabricated reinforced systems, in order to make the work faster (like BAMTEC layers of reinforcements);
4. they have excellent fire and acoustic behavior, thanks to their mass.

If we read the points above, it would seem that there are no reasons why we should not make our floors with slabs, however these structures also have some weak points, which in fact limit their use, in respect to other more efficient methods:

1.1.2 WEAK POINTS

1. They are massive structures:
   - **a** they consume high quantity of concrete;
   - **b** they are very heavy: there are large spans between the pillars, but the self-weight prevails and the result is very expensive.
2. They are non-ductile structures to which the principles of the hierarchy of resistances are not applicable:
   - **a** cannot work on a loom;
   - **b** they need bracing with septa or similar;
   - **c** they need low structures.

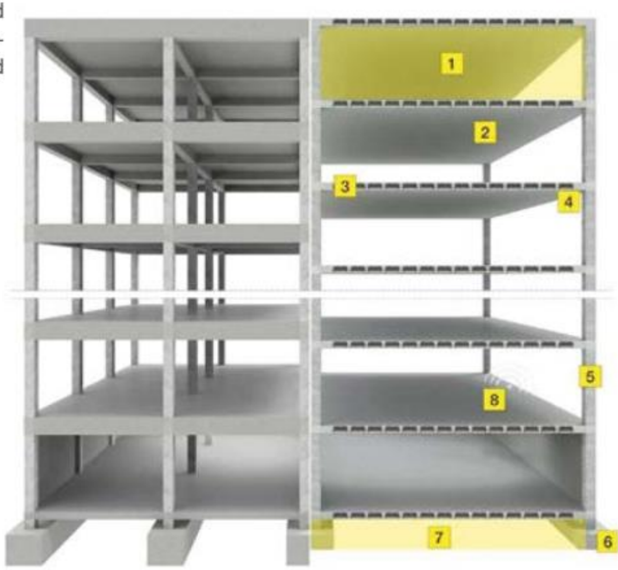
Figura 1 - lightened slab with New Nautilus Evo of Geoplast.

1.2 LIGHTENED SLABS

1.2.1 ADVANTAGES

To create a structure with all the characteristics and strength points of standard slabs, but without their self-weight would look like an ideal solution, but lightened slabs:

1. limit volumes;
2. eliminate beams/straight soffit;
3. concrete is less expensive;
4. allow larger spans;
5. allow optimization of vertical structures;
6. reduce foundation load;
7. reduce excavation load.



1.3 RETICULAR SLABS

The slabs maintain their bidirectional structure and create an orthogonal grid through the installation of disposable blocks (in concrete or terracotta) or reusable (in plastic or fiberglass). Leaving massive capitals in the pillars for the punching.

1.3.1 ADVANTAGES

The advantages offered by this type of solution, are multiple:

1. these are slabs without beams;
2. the quantity of concrete needed is reduced;
3. they are very light;
4. less steel is used;
5. the blocks to make the grids are very cheap.

1.3.2 WEAK POINTS

These structures also have some disadvantages:

1. they lose a lot of inertia compared to the massive slabs of equal thickness, so they must compensate for the greater deformability with higher thicknesses;
2. if they do not comply with certain geometric parameters, they do not have sufficient torsional stiffness to get a SLAB, therefore they have lower performance than the equivalent FULL slab;
3. they must be reinforced grid by grid similar to the beams, with consequent slowing of the laying process;
4. they do not have good acoustic behavior;
5. they have a mediocre fire behavior (no more than REI 90');
6. due to the considerable deformability they have a limited range of use of lights and loads;
7. in the case of recoverable blocks, except for some particular applications, they need a false ceiling.

1.3.3 CONCLUSIONS

This type of solution is ideal alternative to the slab and maintains its characteristics and advantages but in a well-defined field of application, apart from these applications they become less competitive.

Figure 2 - example of reticular slab

1.4 LIGHTENED SLABS WITH HOLLOW ARTICLES

Hollow articles are embedded in the pour, they are usually made of cubic shaped low-density polystyrene or plastic. Blocks remain embedded in the pour and create a grid of ribbings, which are enclosed between two massive upper and lower slabs.

1.4.1 ADVANTAGES

This solution is more efficient than most reticular slabs:

1. presence of the lower slab makes it perfect for all the purposes;
2. the same thickness or even lower thickness of a full slab can be maintained;
3. lightness and concrete savings are guaranteed;
4. they can be reinforced in the same way as massive slabs;
5. quantity of steel is reduced;
6. they have good acoustic behavior;
7. great fire behavior (up to REI 240');
8. there is no need of a false ceiling.

1.4.2 WEAK POINTS

These structures also have some disadvantages:

1. in comparison to reticular slabs they consume more concrete and weight more;
2. they consume more steel with the same concrete consumption and inertia compared to the reticular slabs, due to the lower lever.

1.4.3 CONCLUSIONS

This type of solution is ideal for narrow spans and reduced loads can be economically less interesting than the reticular slab, despite having clearly superior performances.
On the contrary, it is absolutely competitive if compared to the full slab solution, especially in the range of thicknesses from 28 to 60 cm, and spans between 8 and 14 m.

1.5 LIGHTENED SLABS WITH HOLLOW ARTICLES IN PLASTIC

1.5.1 GENERAL CHARACTERISTICS

In the last 10-15 years ago, lightened slabs were created through the use of cubic blocks in low density polystyrene.

This construction technology had some disadvantages:

1. blocks were fragile and suffer from weathering (water imbibition);
2. blocks were occupying a lot of space and did not facilitate worksite logistics;
3. It was difficult to block them and keep them lifted by inferior reinforcements.
   This made it necessary to cast the lower slab, lay and block the lightened slabs, complete the installation of the reinforcements and complete the casting.
   This practice made work time-consuming;
4. in case of fire it has been shown that the polystyrene sublimates creating overpressure inside the hollow that can cause explosions of the slabs, furthermore these gases are toxic.

During recent years a new technical method has arrived in the market. This solution finally allows the overrun of these limitations. Those are recycled polypropylene formworks, 52 x 52 cm with variable height.
They can be “single”, or “double”, by putting together two “single”.

Figura 3 - layout of lightening New Nautilus Evo double

Figura 4 - plastic lightening “single” type

Figura 5 - plastic lightening “double” type

2. TECHNICAL DATA

THE UPPER SPACER

8 mm · EC2

In the upper section of the formwork there are uniformly distributed spacers 8 mm thick.
These elements allow the upper reinforcement to be placed directly over the formwork in order to guarantee a suitable concrete covering.

THE SIDE TAB

2 x pz. · 100 - 200 mm · PP polipropilene

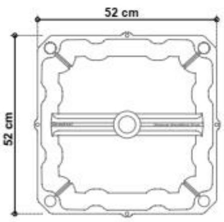
Every formwork is provided with side spacers that allow the correct installation of the elements according to the width of the beams, which is to be calculated during the design stage. The elements are marked from 100 to 200 mm and can be hooked to the side loops.

THE LOWER FOOT

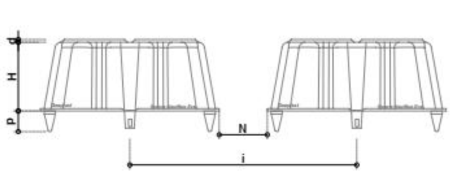
50 - 100 mm · 4 x pz. · R.E.I. 180

The lower spacer feet are integral elements of the formwork: they are molded with the rest of the formwork and allow the creation of the lower slab with a thickness evaluated during the design stage.
The feet have a variable height from 50 to 100 mm.

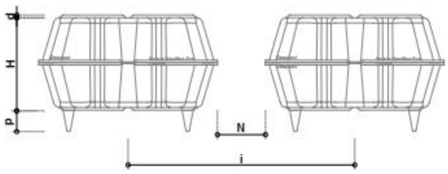
DIMENSIONAL TABLES



NEW NAUTILUS EVO SINGLE



NEW NAUTILUS EVO DOUBLE



HEIGHT	Foot p (mm)	Spacer d (mm)	Actual size (mm)	Weight (kg) ± 10%	Beam width N (mm)	Formwork bearing (pz./m²)	Concrete consumption CLS (m³/m²)	Formwork volume (m³/pz.)
H10 SINGLE	0-50-60-70-80-90-100	8	520 x 520 x H100	1.12	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.038 0.041 0.045 0.048 0.051 0.054	0.024
H13 SINGLE	0-50-60-70-80-90-100	8	520 x 520 x H130	1.18	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.057 0.062 0.066 0.069 0.073 0.076	0.028
H16 SINGLE	0-50-60-70-80-90-100	8	520 x 520 x H160	1.25	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.077 0.082 0.087 0.091 0.095 0.098	0.032
H20 SINGLE	0-50-60-70-80-90-100	8	520 x 520 x H200	1.35	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.099 0.105 0.110 0.116 0.120 0.125	0.039
H24 SINGLE	0-50-60-70-80-90-100	8	520 x 520 x H240	1.45	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.120 0.128 0.134 0.141 0.146 0.151	0.046
H28 SINGLE	0-50-60-70-80-90-100	8	520 x 520 x H280	1.55	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.142 0.151 0.158 0.165 0.172 0.178	0.053

HEIGHT	Foot p (mm)	Spacer d (mm)	Actual size (mm)	Weight (kg) ± 10%	Beam width N (mm)	Formwork bearing (pz./m²)	Concrete consumption CLS (m^3/m^2)	Formwork volume (m^3/pz.)
H20 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H100 + H100	2.24	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.099 0.105 0.110 0.116 0.120 0.125	0.048
H23 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H100 + H130	2.30	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.095 0.103 0.111 0.118 0.124 0.130	0.052
H26 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H130 + H130	2.36	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.114 0.123 0.131 0.139 0.146 0.152	0.056
H29 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H130 + H160	2.43	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.134 0.144 0.152 0.160 0.168 0.174	0.060
H30 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H100 + H200	2.47	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.136 0.146 0.155 0.164 0.171 0.178	0.063
H32 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H160 + H160	2.50	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.154 0.164 0.173 0.182 0.189 0.197	0.064
H33 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H130 + H200	2.53	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.156 0.166 0.176 0.185 0.193 0.201	0.067
H34 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H100 + H240	2.57	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.158 0.169 0.179 0.189 0.197 0.205	0.070
H36 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H160 + H200	2.60	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.175 0.187 0.197 0.206 0.215 0.223	0.071
H37 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H130 + H240	2.63	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.177 0.189 0.200 0.210 0.219 0.227	0.074
H38 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H100 + H280	2.67	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.180 0.192 0.203 0.213 0.223 0.231	0.077
H40 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H200 + H200	2.70	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.197 0.210 0.221 0.231 0.241 0.250	0.078
H41 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H130 + H280	2.73	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.199 0.212 0.224 0.235 0.245 0.254	0.081
H44 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H200 + H240	2.80	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.219 0.232 0.245 0.256 0.267 0.276	0.085
H48 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H240 + H240	2.90	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.241 0.255 0.269 0.281 0.292 0.303	0.092
H52 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H240 + H280	3.00	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.262 0.276 0.293 0.306 0.318 0.329	0.099
H56 DOUBLE	0-50-60-70-80-90-100	8	520 x 520 x H280 + H280	3.10	100 120 140 160 180 200	2.60 2.44 2.30 2.16 2.04 1.93	0.284 0.301 0.317 0.331 0.344 0.356	0.106

3. CALCULATION MANUAL

3.1 SLAB THEORY

The slabs are a type of two-dimensional structural elements which, preserving their thicknesses, combine excellent structural performance, fast installation as well as economic and functional advantages.
The structural behavior of the slab elements is characterized by the prevailing flexural behavior (flexion, shear, torsion), which ensures a transfer of loads along orthogonal paths according to a single preferential direction or two or more preferential directions, therefore dividing into "mono-directional slabs", in the first case, or, "bidirectional slabs" in the second case.
The loads that weigh on the slab can be transmitted in various ways, both point-wise and continuously, therefore the following categories of slabs can be identified:

- slab with constant thickness on columns with or without capital (flat slab);
- slab with variable thickness, with local thickening in correspondence of the columns (mushroom slab);
- slabs on edge beams, placed on two or four sides;
- slabs on load-bearing walls.

As far as the calculation phase is concerned, the codes are generally related to the elastic analysis, therefore in the absence of cracking, and ignoring the reinforcement, acceptable hypotheses in the verification phase at the operating limit states.
On the contrary, when the ultimate limit states are taken into consideration, the behavioral non-linearity, due to the concrete cracking and the plasticizing of the steel, is necessarily taken into consideration.
Certainly, the key to understanding the many aspects of the mechanical behavior of the slab is still provided by the theory of elasticity, in the hypothesis of small displacements. The elastic model, based on the hypothesis of displacement continuity, tensions and deformations, makes it possible to preserve the concept of internal action as the local resultant force of the tensions that are having effect on a unitary section, whatever the nature of the stress.
A reference to the elastic method is obliged.

As for the lightened slabs, the shape of the box configures the slab as a series of crossed ribs closed above and below by two slabs of variable thickness by choice.

Experimental results found in the appendix certify that this structure maintains the same nature and behavior as an orthotropic slab according to EC2.
According to Eurocode 2, in fact, in the structural analysis may not be necessary to decompose the ribbed or lightened slabs into discrete elements, as long as the wing or the upper structural part and the transverse ribs have adequate torsional stiffness.
This assumption is valid if:

- the pitch of the ribs does not exceed 1500 mm;
- the height of the rib, below the wing, is not more than 4 times its width;
- the thickness of the wing has equal or greater value than the highest value between 1/10 of the net span between the ribs and 50 mm;
- there are transverse ribs distant from each other no more than 10 times the total thickness of the slab.

A different pitch of the ribs leads to a different bending-resistant mechanism; in fact, in a mono-directional slab, the zones of slab that are more distant from the rib itself do not appear to be fully collaborating.

Following the same logic, the distribution of the normal stresses in slab and counter-slab that are created due to the flexing of the deck are concentrated near the ribs and are decreasing while moving further from them (a phenomenon called shear moment).
This kind of behavior is less important for the structures in question, since the ribs on two sides prevent the behavior from being purely one-way.

3.2 PREDIMENSIONING

3.2.1 DETERMINATION OF THICKNESS

The constructive configuration in which the technology **New Nautilus EVO** expresses its potential to the maximum, that of a bidirectional plate, in the situation such as to have a relation between the spans in the two orthogonal directions expressed as:

$$\frac{L_x}{L_y} \in \left\{1 < \frac{L_x}{L_y} < 1.7\right\}$$

Outside this range the behavior becomes purely mono-directional.
A first way to obtain an indicative size of the slab thickness is with simple proportions, based on structural types in use and spans to be covered:

- full plate on pillars $d = \frac{L}{25}$
- lightened plate on pillars $d = \frac{L}{28}$
- full plate on beams $d = \frac{L}{30}$
- lightened plate on beams $d = \frac{L}{32}$

The minimum size of the upper and lower slabs is constrained by minimum covering requirements to be secured to the bars. Inside the slab the minimum reinforcement covering required by the regulations must be ensured for the category of exposure relating to the calculation hypotheses made, plus the two diameters of the basic reinforcement in the two directions.
The thickness of the slab is considerably reduced, compared to a basic reinforcement composed of elements with electro-welded meshes, if a loose bar solution are used.

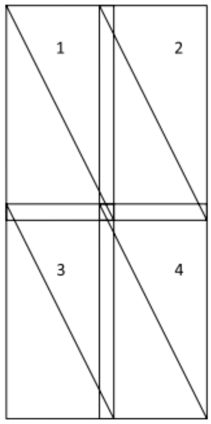


Figura 6 - arrangement of electro- welded meshes.

As you can see from the drawing, the position of armatures containing electro-welded meshes gives the necessary overlapping, at least partial, in the border areas between two adjacent meshes. In the small section belonging to all four of the designed meshes it will be necessary to have the space to place eight bars in the slab, plus the space for necessary distances between the lower bar and the upper side of the slab, and between the upper bar and the formwork, in a way that allows a correct flow of the concrete pour.

To these geometric estimations we must add fire safety considerations; in fact, greater thicknesses fit with better fire load resistance.

The size of the ribs is also related to the type of additional reinforcements required; in fact, if it is required and it is decided to reinforce the ribs, it must be done in a way that the bars inside them can be placed, ensuring at the same time a correct distance from the formworks, as well as reinforcement gap.
The minimum spacing must be such that:

$$C_{barre} > max\left\{\varphi_x^{max}; \varphi_y^{max}; 20\ mm\right\}$$

Furthermore, more massive ribs, as it will be shown later, correspond to greater shear resistance, therefore a larger size may be required to handle high stresses.

3.3.3 F.E.M. MODELING

È possibile, poi, modellare agli elementi finiti le piastre in calcestruzzo armato alleggerite con elementi **Nuovo Nautilus EVO**, elements, as full plates with reduced values of stiffness and mass. You can adopt one of the following solutions:

a. shape the plate with **New Nautilus EVO** as a slab of the construction thickness, but with multiplicative coefficients for mass and inertia reduction;

b. shape the plate with **New Nautilus EVO** as a full slab of reduced thickness, in order to obtain equivalent stiffness and weight;

c. shape the plate with **New Nautilus EVO** as a slab of the construction thickness but introduce coefficients on the reinforced concrete material of the portion of the plate occupied by the lightening, in order to reduce the Young's modulus and self-weight.

The solution **a** is obtained as follows:
remembering that the flexural stiffness value D is:

$$D = \frac{E \cdot I}{\left(1 - v^2\right)}$$

Where:

- E is Young's Module;
- I is the Moment of Inertia;
- v is the Poisson's Module.

Our goal is to shape an isotropic plate to the finite elements, with the same stiffness as a light plate:

$$D_{full} = D_{void}$$

$$\frac{E_{full} \cdot I_{full}}{\left(1 - v^2\right)} = \frac{E_{void} \cdot I_{void}}{\left(1 - v^2\right)}$$

$$R_f = \frac{E_{void} \cdot I_{void}}{E_{full} \cdot I_{full}}$$

The coefficient R obtained is the reductive coefficient to be applied to a full plate in the program with finite elements in order to obtain the equivalent flexural stiffness of the plate with **New Nautilus EVO**, and, if the material is the same, it becomes:

$$R_f = \frac{I_{void}}{I_{full}} < 1$$

CALCULATE THE INERTIA OF THE LIGHTENED SLAB

ELEMENT	INERTIA [mm⁴]	BARYCENTER [mm]	AREA [mm²]	VOLUME [m³]
H10	38577300	48.3	46361	0.024
H13	84834500	62.9	60350	0.028
H16	158272000	77.6	74339	0.032
H20	309335100	97.1	92909	0.039
H23	448121900	115.3	106712	0.052
H24	534784000	116.6	111643	0.046
H26	648180500	130.0	120701	0.056
H28	849526400	136.2	130295	0.053
H29	901349400	145.0	134690	0.060
H30	1002580000	151.2	139353	0.063
H32	1212130200	160.0	148600	0.064
H33	1332396800	165.8	153342	0.067
H34	1465266900	171.8	158004	0.070
H36	1729333800	180.5	167331	0.071
H37	1883915700	186.3	171994	0.074
H38	2052377400	192.2	176649	0.077
H40	2373891500	200.0	185982	0.078
H41	2570833000	206.8	190645	0.081
H44	3163889700	220.0	204635	0.085
H48	4109701700	240.0	223200	0.092
H52	5230090000	260.5	241938	0.099
H56	6534840000	280.0	280590	0.106

Therefore, the inertia of the lightened rib can be found, using Huygens-Steiner, as:

$$I^{void}_{x/y} = \frac{1}{12} B_t \cdot H_t^3 + B_t \cdot H_t \left(\frac{H_t}{2} - y_G^{void}\right)^2$$

$$-I^{nau}_{x/y} - A_{naut} \cdot \left[\left(y_G^{naut} + S_i\right) - y_G^{void}\right]^2$$

- H_t : total thickness of the slab;
- B_t : interaxial spacing between the ribs;
- y_G^{void}: barycenter of the final lightened section;
- y_G^{naut}: barycenter of the empty zone;
- $I^{void}_{x/y}$: moment of inertia in the x or y direction of the final lightened section;
- $I^{nau}_{x/y}$: moment of inertia in the x or y direction of the empty zone;
- A_{naut}: surface of the empty zone.

Dividing the obtained quantity by the length of the interaxial spacing between the formworks, we find the inertia value per linear meter. While the inertia (per linear meter) of the solid section can be calculated as that of a rectangle of height equal to the height of the concrete section, and of width equal to the width of an element **New Nautilus EVO** plus the size of the rib, the equivalent quantity for a lightened slab can be calculated as that of an I, profile, with lower and upper wings given by the slabs, and a core with dimensions of the rib.

The solution **b** is obtained by finding a thickness of a full plate, that has the same flexural stiffens of the lightened plate:

$$S_t = \frac{E \cdot I_{void}}{\left(1 - v^2\right)} = \frac{E \cdot H_t^3}{12 \cdot \left(1 - v^2\right)}$$

Resolving it, gives the expression of the fictitious thickness:

$$H_f = \sqrt[3]{12 \cdot I_{void}}$$

In a completely similar way to the one done for flexural stiffnesses, torsional and shear stiffnesses also must be reduced in order to shape the behavior of the lightened plate correctly

$$R_t = \frac{S_{t,void}}{S_{t,full}} < 1$$

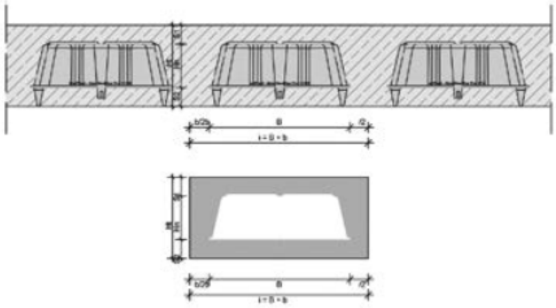


Figura 18 - section type - calculation of the reduction coefficients

The torsional stiffness of the full slab is determined according to the formula:

$$I_t = \alpha \cdot H_{tot}^3 \cdot i$$

Where the factor α is a function of the relation i/H_{tot}:

i/H_{tot}	1.5	2.0	3.0	4.0
α	0.196	0.229	0.263	0.281

i/H_{tot}	6.0	8.0	10.0	∞
α	0.299	0.307	0.313	0.333

Similarly, it can be obtained for the lightened slab, using the Bredt's formula:

$$t_1 = \frac{N_{x/y}}{2}$$

$$t_2 = S_s$$

$$t_3 = S_i$$

$$b_k = i - T_1$$

$$d_k = H_t - \frac{S_s}{2} - \frac{S_i}{2}$$

$$I_t^{void,x/y} = \frac{4 \cdot b_k \cdot d_k}{\frac{2}{b_k \cdot t_1} + \frac{1}{d_k \cdot t_2} + \frac{1}{d_k \cdot t_3}}$$

So, the reduction factor turns out to be:

$$R_t = min\left\{\frac{I_t^{void,x}}{I_t^{full}}; \frac{I_t^{void,y}}{I_t^{full}}\right\} < 1$$

The multiplicative factor that takes into account the reduction of the shear strength is obtained by comparing the areas resistant to the shear stress of the full slab and the rib of the lightened slab:

$$R_s = \frac{A_{s,void}}{A_{s,full}} < 1$$

While a portion of unitary width of full slab reacts entirely to the shear, the shear area of the lightened slab is given only by the full area of the section.

Regarding the weight of the lightened slab itself, this can be calculated by subtracting the volume of the **New Nautilus EVO** formwork per square meter, from the weight of the corresponding full slab:

$$W_{void} = \left[H_{tot} - \frac{1}{\left(52\,cm + B\right)^2} \cdot Vol_{Naut}\right] \cdot \gamma_{cls}$$

This value allows you to find the reduction factor of the self- weight:

$$R_w = \frac{W_{void}}{W_{full}} < 1$$

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MIX
Paper from responsible sources
FSC® C147178

8.2. Appendix 2 – Lightened Slab Specifications Excel Sheet

Lightened Slab Specifications

8.3. Appendix 3 – Automated Excel Sheet for Seismic Spectrum (NTC 2018)

Seismic Spectrum (NTC 2018)

8.4. Appendix 4 – Seismic Action Excel Sheet

Seismic Action

8.5. Appendix 5 – Wind Load Excel Sheet

Wind Load

8.6. Appendix 6 – Snow Load Excel Sheet

Snow Load

8.7. Appendix 7 – Specifications of Reinforced Concrete Structures Excel Sheet

Reinforced Concrete Specifications

8.8. Appendix 8 – Multiple Criteria Analysis Excel Sheet

Multiple Criteria Analysis

8.9. Appendix 9 – Designing Times Recorded

8.9.1. Alternative 1

Attempt 1	Attempt 2	Attempt 3
		

8.9.2. Alternative 2

Attempt 1	Attempt 2	Attempt 3
		

8.9.3. Alternative 3

Attempt 1	Attempt 2	Attempt 3
		

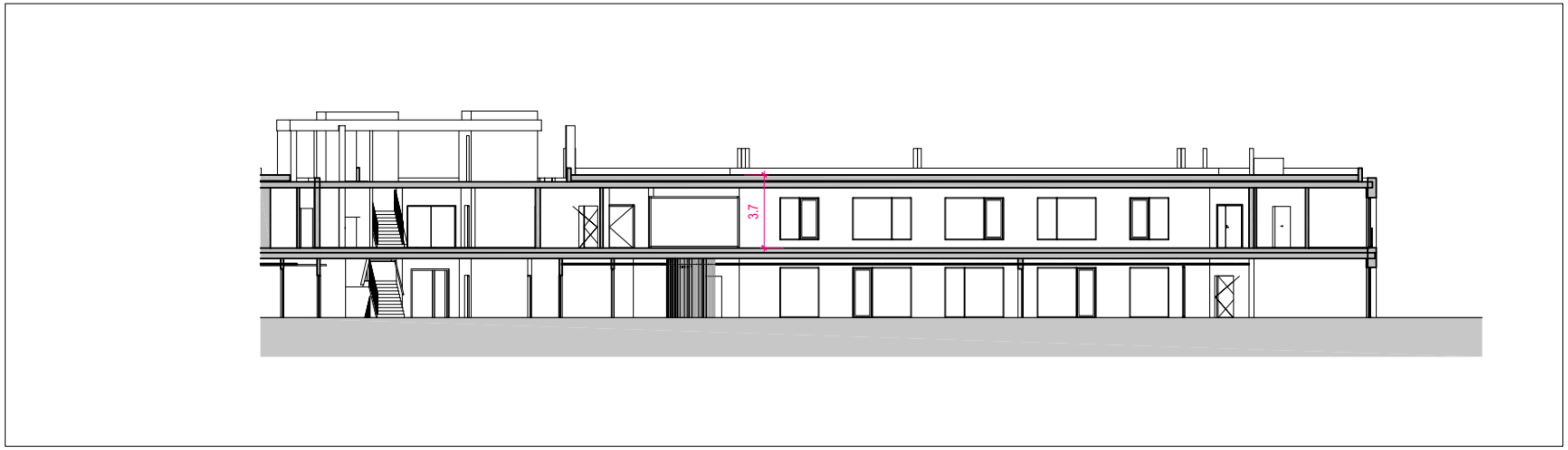
8.10. Appendix 10 – Extrapolation of CADiNP Text

```

!#!Info Example:    Static of a Bubble Deck
!#!Info Program:    BEMESS
!#!Info Keyword:    bubble deck; layer
!#!Info Date:       05/24/2007
$ Design of bubble decks:
$      In bending design, the compression zone is limited to the thickness
$      of the outer material layers. In shear design, the shear capacity
$      is reduced to CTRL BUBB of the shear capacity of a full section.
$      Default CTRL BUBB 0.55.
$      The longitudinal reinforcement is taken into account in shear design
$      [please refer to ro_v]. Shear links are not allowed. Aditionally, within
$      punching perimeters, also the normal shear check will be performed for
$      bubble elements. In WINGRAF the utilisation of shear capacity can be
$      checked with VED/VRDmax [VRDmax of a full section].
$      Values VED/VRDmax > CTRL BUBB simultaniously mark a shear design failure.
$  Please use animator and select loadcase 1, find element info and
$  klick an element in midspan to check layered element behaviour.
$
$ For prestressed slab see ase.dat\english\bridge\voided_slab.dat

$  Comment:
$  If a slab is explicit devided with e.g. 50 cm in between it behaves
$  like two separate slabs and has a higher deflection.
$  The SOFiSTiK layered material is mainly used for timber glued plates
$  or plates with shear connection (no air).
$  The shear stiffness is reduced but it does not behave like two separated plates.
$  In case of bubble deck we assume smeared holes (bubbles) which do not
$  destroy the shear capacity completely. A shear check must be done separately anyway.
$  If you really want to analyze a box girder section with a concrete slab
$  on top and on bottom, you must define two slabs (shell system).

!#!KAPITEL Material and System generation
+PROG AQUA urs:1
HEAD Bubble deck
NORM 'NS' 'en199X-200X-BRIDGE'      CAT 'B'   $ road bridges
echo mat
CONC 1 C    30
STEE 2 S 450C
CONC 8 C    30     EC 8789 GAM 0.541*25   $ Bubble deck inner layer - smeared holes -> average stiffness EC 10000
SSLA SERV 1.00 ; let#fcr -10000/1000 $definiton of work law so that it fits to EC for nonlinar analysis
SSLA     EPS     SIG        TYPE
          0       0         POL     $-------------
        -1.0     #fcr       POL     $
        -3.5     #fcr       POL     $ compressive zone
        -9.0      0
SSLA ULTI 1.50
SSLA     EPS     SIG        TYPE
          0       0         POL     $-------------
        -1.0     #fcr       POL     $
        -3.5     #fcr       POL     $ compressive zone
        -9.0      0
       $ see also ase.dat\english\bridge\voided_slab.dat

MLAY NO 9 T0 0.080 1  $$
          T1 0.160 8  $$
          T2 0.080 1                 $ Material 9 = layered material ! Please
$           -------                  $ notice that the thickness of the outer layers
$ total h =  0.300 m  = 0.06+0.18+0.06  $ is used to limit the compression heigth !
$        Material number  1    8    1
                                     $ Material 9 = Schichtmaterial ! Bitte beachten
                                     $ Sie, dass die Dicke der äußeren Schichten verwendet
                                     $ wird, um die Druckzone zu begrenzen!
END

PROG SOFIMSHA urs:2
HEAD
UNIT 0
SYST GIRD GDIV 10000 POSZ
```

```
+PROG SOFILOAD urs:5
HEAD
UNIT 5 $ units: sections in mm, geometry+loads in m
LC 901 TYPE NONE
  QUAD grp 1 TYPE PZZ 2.50*1.40+3.00*1.60 $ additional g1 + L
LC 902 TYPE NONE
  QUAD grp 1 TYPE PZZ 2.50*1.00+3.00*1.00 $ SLS
END

!#!KAPITEL Analysis
+PROG ASE urs:3
HEAD
LC 1     FACD 1.0   TITL    'g1'
LC 1001  FACD 1.40  TITL    '1.40 g1 + 1.60 L'
  LCC 901
END

+PROG BEMESS urs:4
HEAD Definition of reinforcement parameters and minimum reinforcement
$
$ normal rooms inside
GEOM - HA 25[mm] DHA 10[mm] HB 25[mm] DHB 10[mm]
DIRE  0 0
PARA NOG - DU 7[mm] ASU 0 0 ASL 3.77[cm2/m] 3.77[cm2/m]
              $ ^ minimum reinforcement for nonlinear analysis
END

+PROG BEMESS urs:7
HEAD
CTRL ULTI      ! as loadcase 1001 already on ULS level!
CTRL BUBB 0.62 ! ratio of bubble deck shear strength compared to full section
LC 1001
END

+PROG ASE urs:6
HEAD nonlinear analysis
SYST PROB NONL NMAT YES
REIQ LCR 1
LC 1002  FACD 1.00  TITL    '1.00 g1 + 1.00 L'
  LCC 902
END

$ Clean file folder:
+sys del $(project).$d?
```

8.11. Appendix 11 – Architectural Plan for Casa Haus inge

8.11.1. Longitudinal Cross-Section



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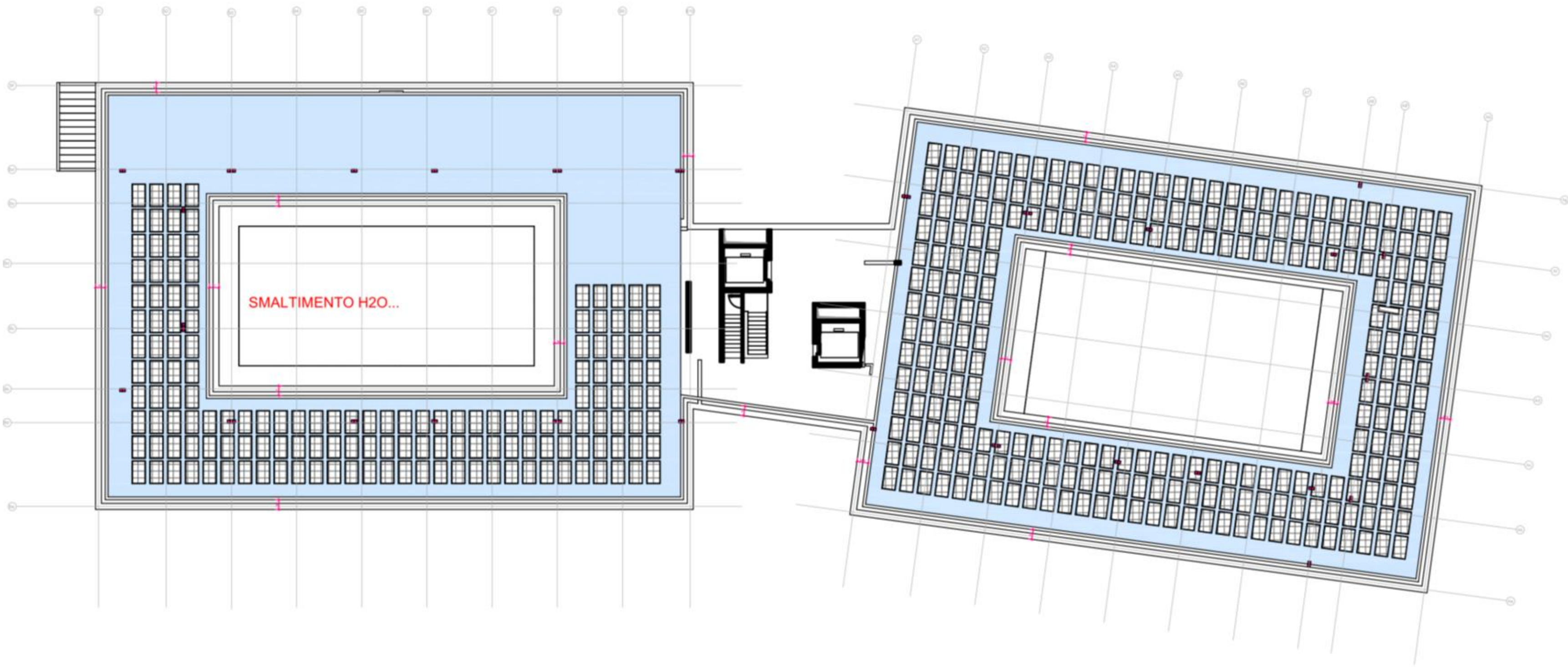
8.11.2. Ground Floor's Plan

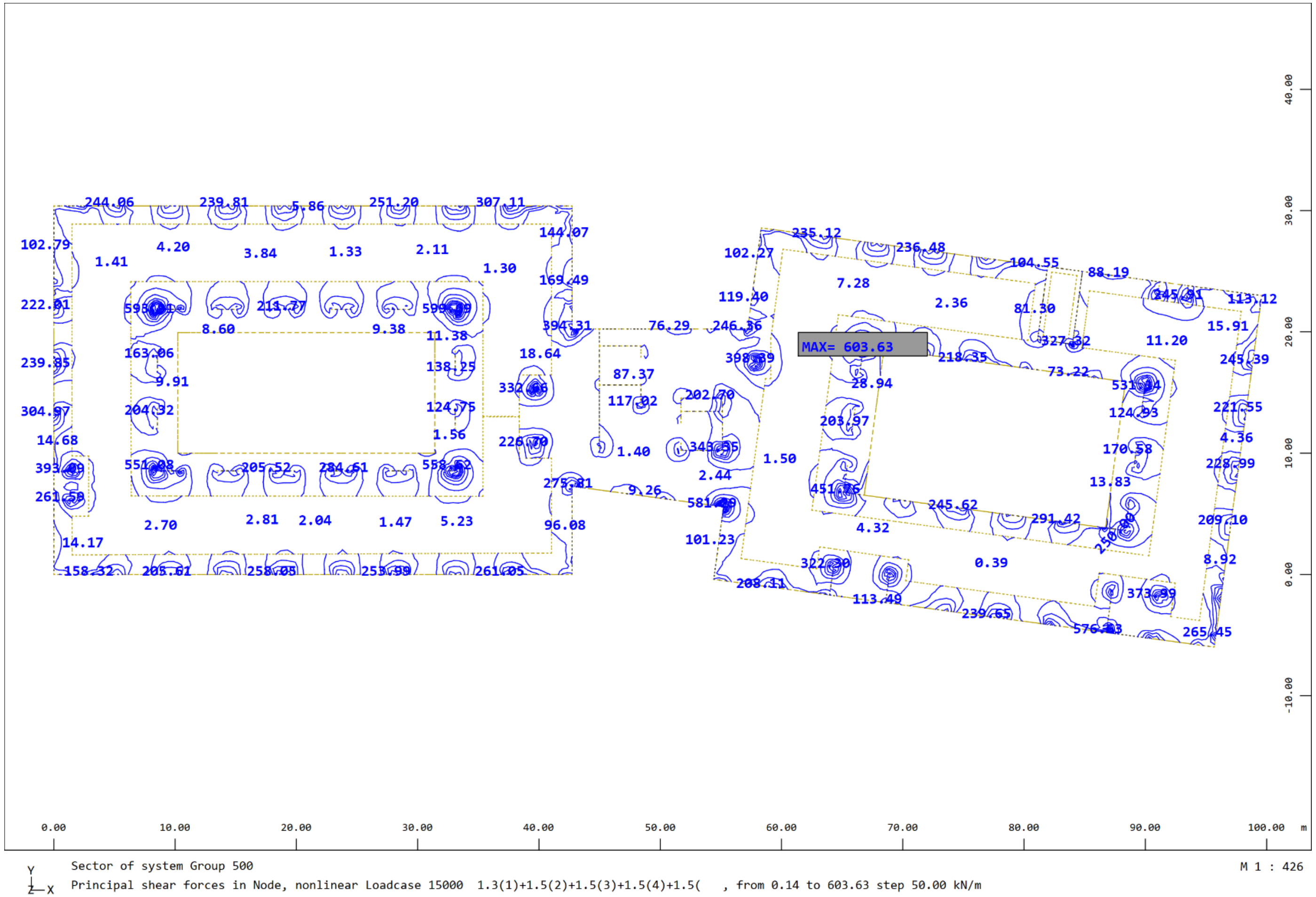
8.11.3. First Floor's Plan

8.11.4. Roof's Plan



8.12. Appendix 12 – Results of the Lightened Nursing Home FEM Model

8.12.1. Shear Forces – First-Floor Slab



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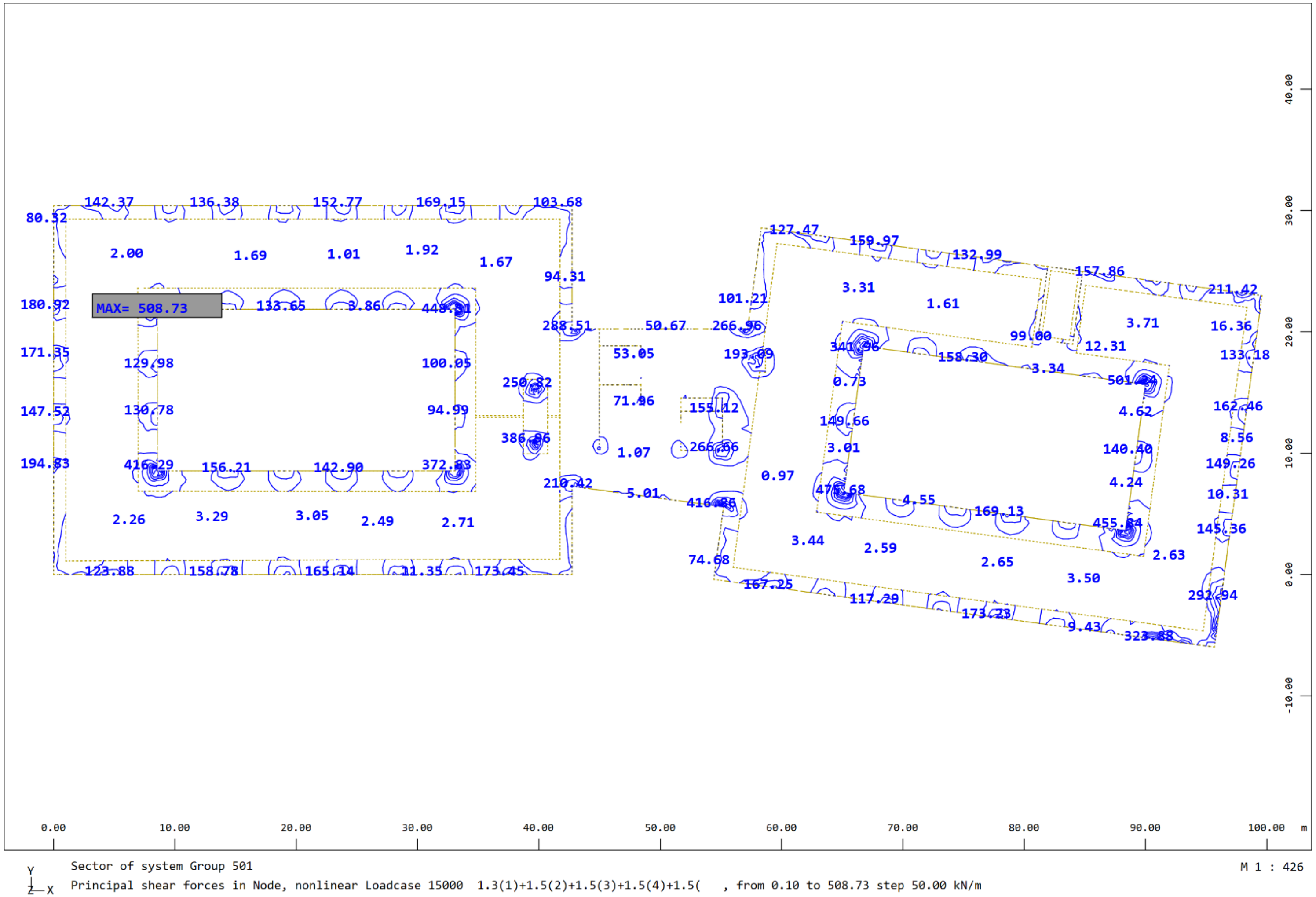
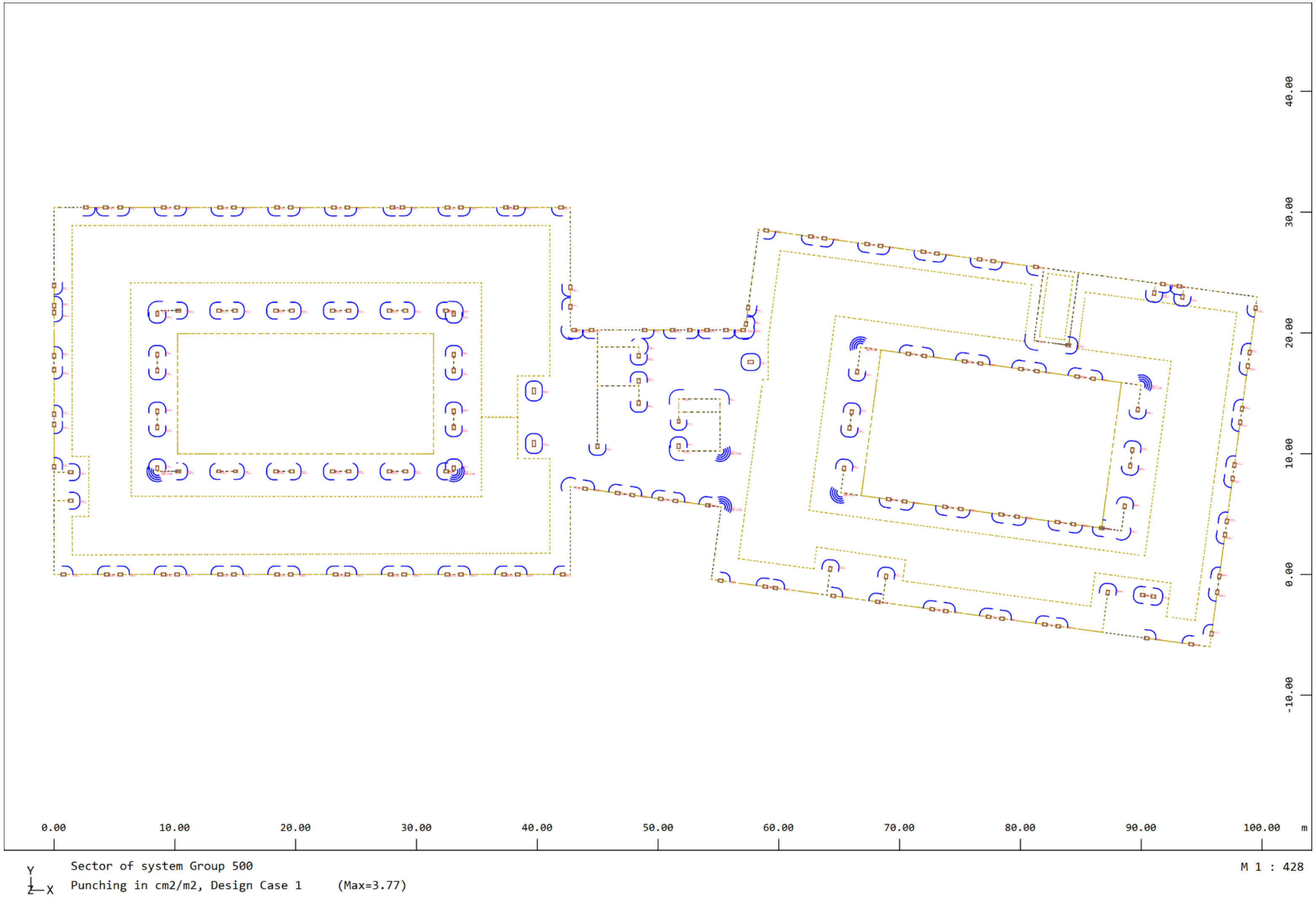
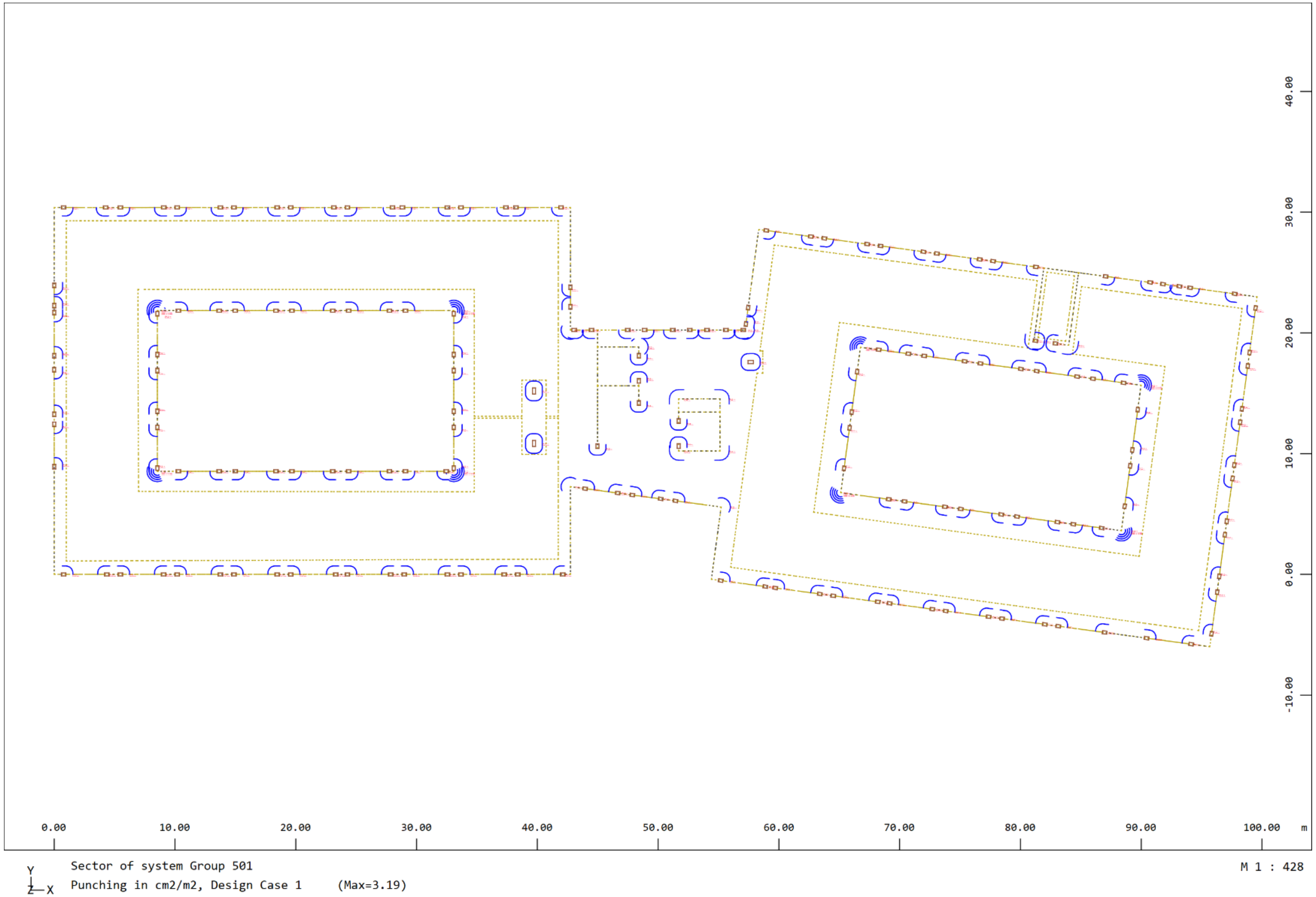
8.12.2. Shear Forces – Roof Slab



8.12.3. Punching Shear – First-Floor Slab



8.12.4. Punching Shear – Roof Slab

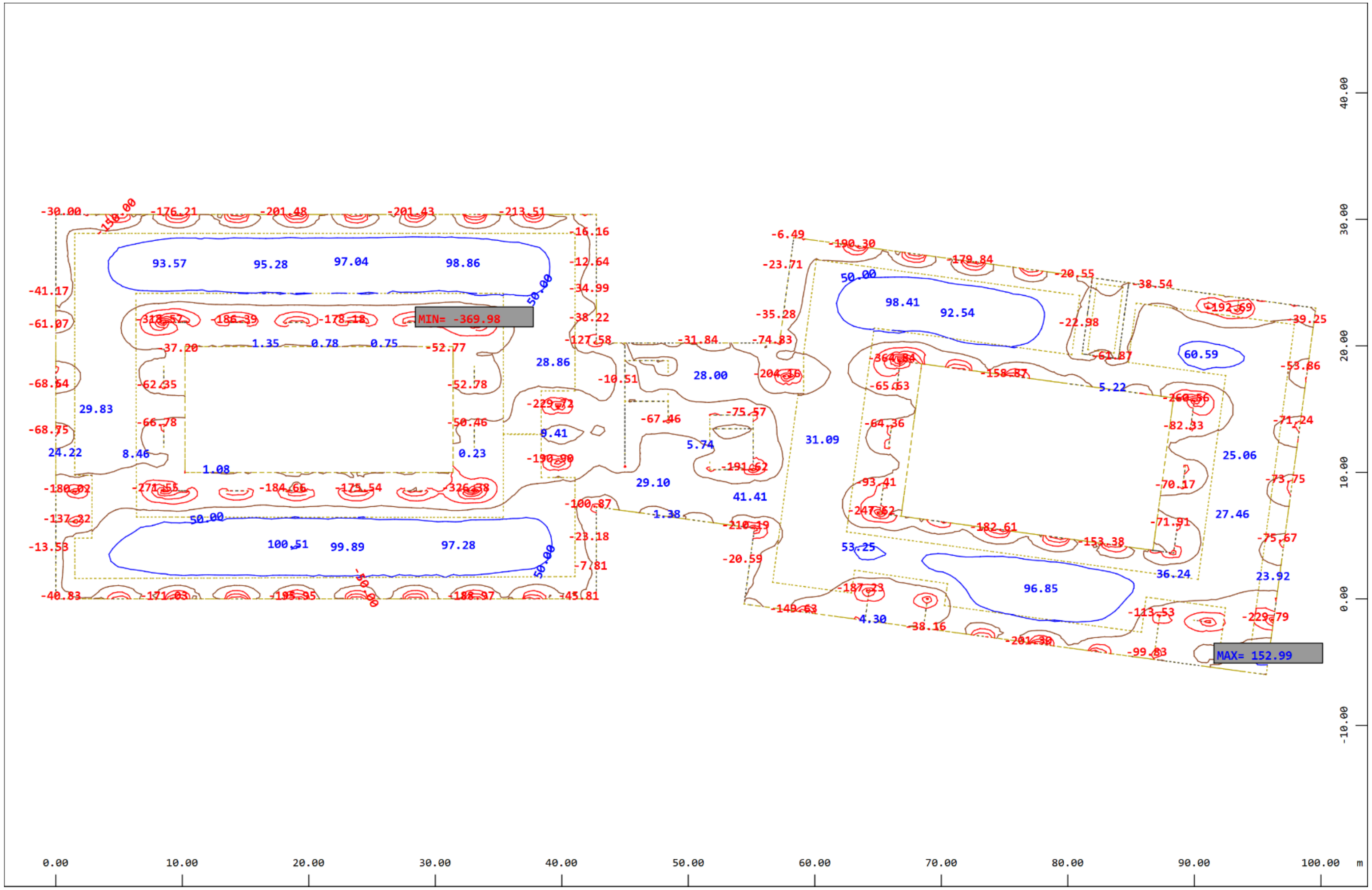


8.12.5. Punching Shear – Summary of Nodes Requiring Punching Reinforcement

Nr	Node	Typ	X	Y	V_{ULS}	ucrit	%u0	beta	V_{max}	AssSum	ast	nperi
1	No		[m]	[m]	[kN]	[m]	[o/o]	[-]	[MPa]	[cm2]	[cm2/m]	
2	1205	L	55.223	5.580	365.4	1.629	36	1.20	0.89	11.01	17.68	4
3	1305	L	55.123	10.220	336.0	1.629	36	1.20	0.81	10.13	13.75	4
4	1339	L	33.085	21.850	380.2	1.629	36	1.20	0.92	11.46	19.92	4
5	1342	L	8.540	21.850	525.6	1.629	36	1.20	1.27	15.84	52.64	4
6	1343	L	8.540	21.850	382.0	1.629	36	1.20	0.93	11.51	20.20	4
7	1377	L	33.085	8.540	469.3	1.629	36	1.20	1.14	14.14	37.45	4
8	1380	L	33.085	8.540	347.3	1.629	36	1.20	0.84	10.47	15.18	4
9	1401	L	8.540	8.540	417.7	1.629	36	1.20	1.01	12.59	26.41	4
10	1404	L	8.540	8.540	379.4	1.629	36	1.20	0.92	11.43	19.80	4
11	1420	L	66.756	18.814	399.9	1.629	36	1.20	0.97	12.05	23.18	4
12	1441	L	89.990	15.671	447.6	1.629	36	1.20	1.08	13.49	32.50	4
13	1444	L	89.990	15.671	362.4	1.629	36	1.20	0.88	10.92	17.25	4
14	1456	L	88.359	3.620	364.2	1.629	36	1.20	0.88	10.98	17.51	4
15	1477	L	65.126	6.763	448.6	1.629	36	1.20	1.09	13.52	32.71	4
16	1480	L	65.126	6.763	425.8	1.629	36	1.20	1.03	12.83	27.98	4

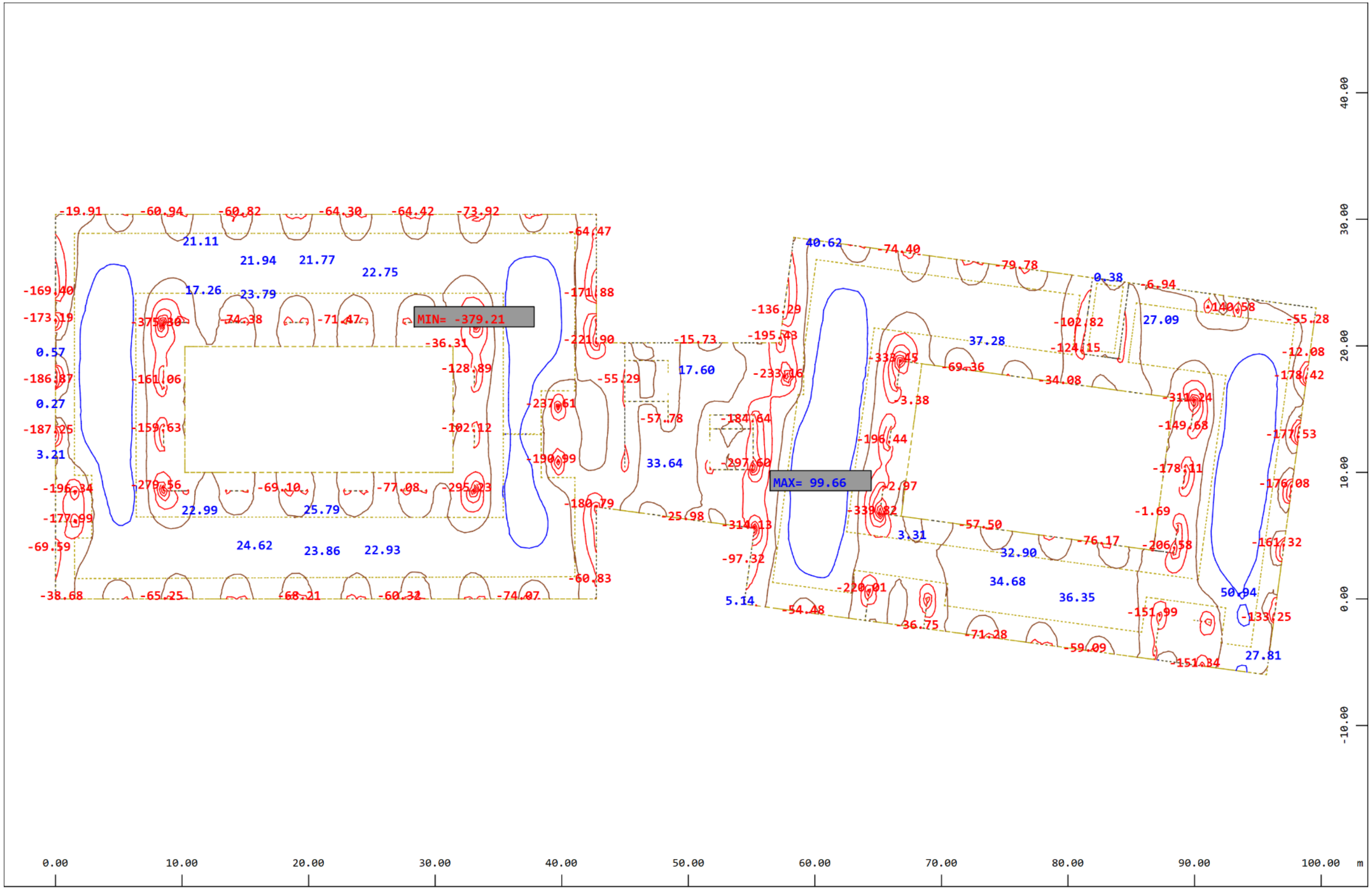
Typ	I=inner column, E=edge column, C=corner column, F=foundation, W=end of wall, L=wall corner, G=end_of_girder	V-ULS	design shear force (reduced by bedding pressure)
column	dimension of column or wall thickness at end of walls	ucrit	effective length of 1. perimeter, reduced due to openings and edges
%u0	ucrit = ... % of a full circle (ucrit/u0-tot)	beta	excentricity factor
v-max	shear stress at reduced critical 1. perimeter	AssSum	shear reinforcemend - total sum of all nperi perimeters
ast	min. required tension reinforcement in the punching zone	nperi	up to this perimeter, shear reinforcement is required

8.12.6. Bending Moments – First-Floor Slab



Y
Z X
Sector of system Group 500
Bending moment m-yy in local y in Node , nonlinear Loadcase 15000 1.3(1)+1.5(2)+1.5(3)+1.5(4)+1.5(, from -369.98 to 152.99 step 50.00 kNm/m

M 1 : 426



Sector of system Group 500
Bending moment m-xx in local x in Node ↔, nonlinear Loadcase 15000 1.3(1)+1.5(2)+1.5(3)+1.5(4)+1.5(, from -379.21 to 99.66 step 50.00 kNm/m

M 1 : 426

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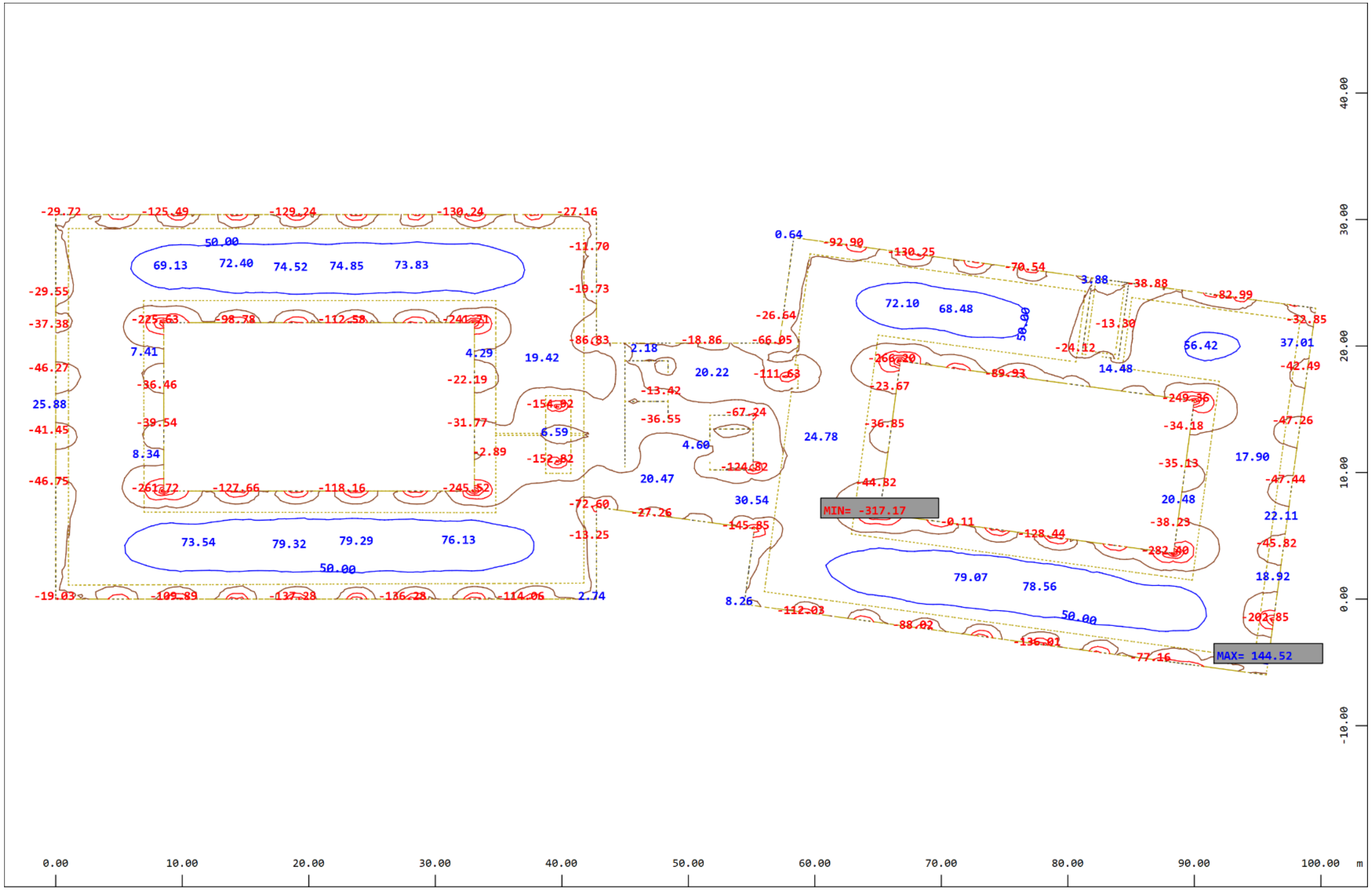
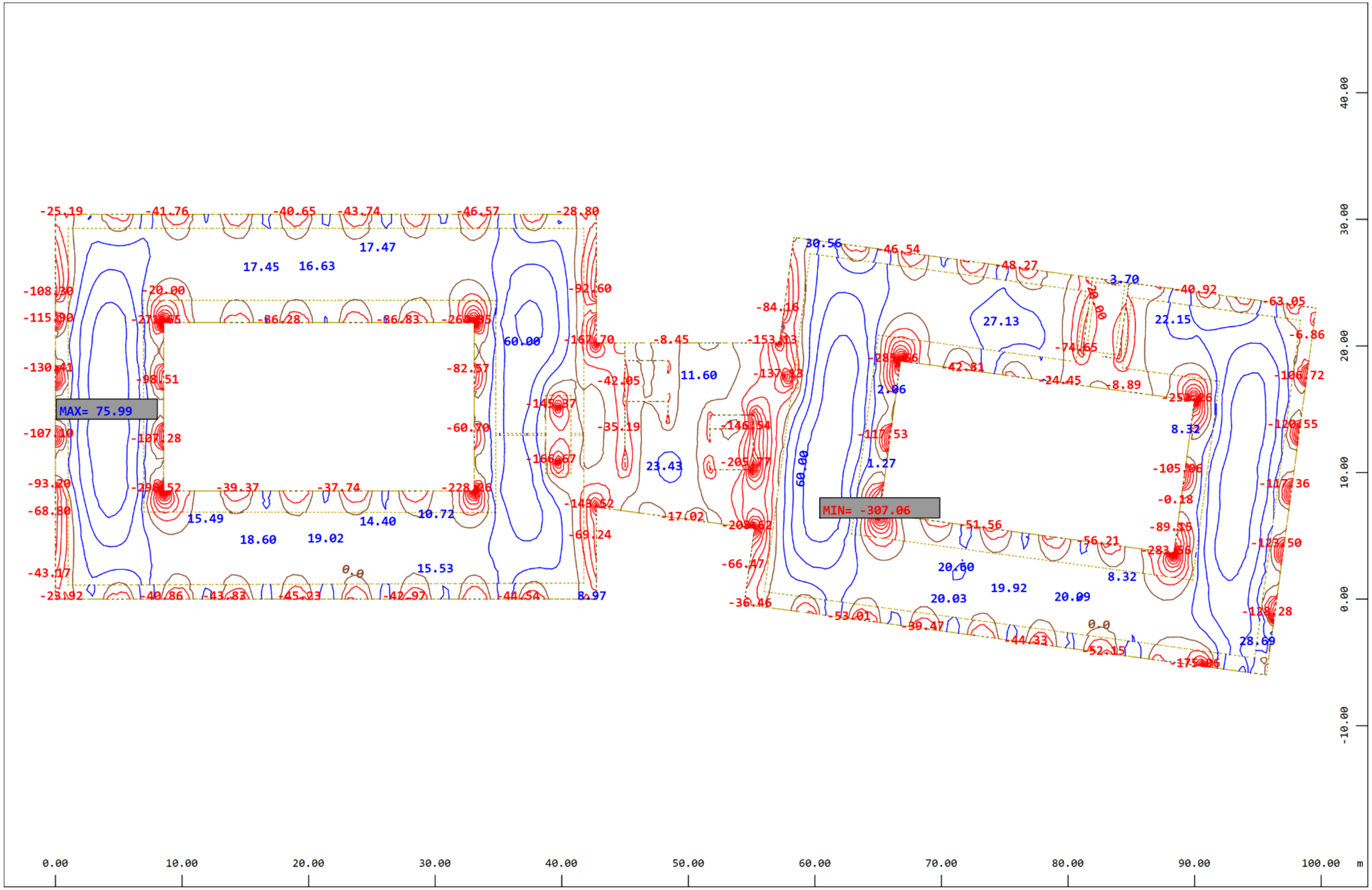
8.12.7. Bending Moments – Roof Slab



Sector of system Group 501
Bending moment m-yy in local y in Node ↕, nonlinear Loadcase 15000 1.3(1)+1.5(2)+1.5(3)+1.5(4)+1.5(, from -317.17 to 144.52 step
50.00 kNm/m

M 1 : 426



Y
Z X

Sector of system Group 501

M 1 : 426

Bending moment m-xx in local x in Node <->, nonlinear Loadcase 15000 1.3(1)+1.5(2)+1.5(3)+1.5(4)+1.5(, from -307.06 to 75.99 step 20.00 kNm/m

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