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Seismic retrofitting within sustainable building assessment schemes

Proposal for a seismic resistance related criterion in green building rating systems

Adeguamento sismico nelle certificazioni di sostenibilità degli edifici

Proposta di un criterio riguardante la resistenza al sisma nei sistemi di valutazione degli edifici green

Supervisors:
M. Sc. Lukas Rauber
Dipl.-Ing. Georgios Balaskas

Examiners:
Full Professor Gian Paolo Cimellaro
Professor Dr.-Ing. Benno Hoffmeister

Candidate:
Eros Tanzi
300786
448930

Abstract

In a historic period in which the climatic crisis is becoming more threatening than ever, actions need to be taken to reduce the CO_2 emissions in every possible human activity. One of the measures in the construction sector is the adoption of sustainable building certifications on a voluntary basis, whose results are expressed with a comprehensive grading that clarifies the environmental impact of the building. However, such protocols currently do not consider sufficiently the seismic vulnerability especially of existing buildings, built with lower or no seismic provisions.

Even for modern buildings, current codes are providing buildings that are safe against seismic actions with the aim of avoiding human life losses, but they do not consider many attempts for reducing the damages inflicted to the structure. Less or easily repairable damages mean also more sustainable buildings, implying lower environmental, social, and economic impacts. That is why a seismic resistance related criterion should be introduced in the assessment of green buildings' certifications.

Moreover, the European Union is pushing towards their renovation of its building stock both on a seismic and energetical point of view, since it is prevalently constituted by buildings constructed some decades ago. Within this master thesis, the necessity of such a criterion is emphasized through the investigation of an idealized case study constituted by a reinforced concrete building erected in the 1970s, adopting alternative seismic analysis methods. Two different retrofit concepts are developed and compared both regarding the improvement on the structural behavior of the building, as well as considering their environmental impact assessed by using different parameters. The reference structure and its strengthening approach is examined for a high seismicity region (Zambrone, south of Italy) and a low-to-moderate seismic area (Aachen, Germany) representing the variation of seismicity through Europe.

Contents

1 Introduction

The purpose of this master thesis is to highlight the connection between seismic design and sustainability in the construction sector, underlining how the structural behavior under seismic loads could affect green building rating systems.

Such goal is pursued through the study of an idealized building, constituted by a moment resisting frame in reinforced concrete. The building will be placed in two different parts of Europe: Germany and Italy. The same plan and elevations will be maintained but the main structural members (beams and columns) will be dimensioned taking into account the selected site. In both the cases the location is one of the highest seismicity regions in the corresponding country, but the design actions will be quite different, considering the higher seismic hazard present in Mediterranean countries than in central Europe.

Firstly, the structural elements are going to be dimensioned through the use of a numerical model created with the software SAP2000, with reference to the fundamental load combination for ultimate limit state according to Eurocodes and to the seismic loads that were used in design during the 1970s, period in which the case study is considered to be built. Then, the verifications will be carried out considering the current seismic actions. The structure is expected not to satisfy the requirements and consequently refurbishments will be designed, involving different retrofitting technologies. In particular, shear concrete walls and timber light frame walls will be introduced as strengthening measures to improve the seismic behavior of the investigated structure.

Another aspect that the study aims to investigate is the use of simplified or advanced seismic analysis methods, such as the modal response spectrum analysis and the nonlinear pushover analysis. The differences related to the outputs provided by the application of each method on the existing structure, assessing the possible need for retrofit, will be hopefully a solid base for a new criterion in the certification schemes for sustainable buildings.

For what concerns the evaluation of environmental loads associated to each possible renovation, it is going to be performed through the exploitation of Environmental Product Declarations (EPDs) of the various components involved in the construction of the different shear walls. These documents are reporting the impacts of the interested product in several life-cycle stages and according to many environmental indicators, which makes them suitable for evaluations embodied into green building rating systems, as it already happens in some of their criteria nowadays.

1.1 MotivationsError! Bookmark not defined.

Some of the most relevant arguments backing up this study are hereby described.

1.1.1 Environmental impact of the construction sector

The whole world is nowadays facing enormous challenges to face the climate crisis and the gears of different countries are moving with the aim of reducing the carbon emissions. For instance, the well-known policy of the European Union is to fulfil the European Green Deal, by reaching climate neutrality by 2050 [1]. In the delineated context, the AEC world (Architecture, Engineering and Construction), plays a very decisive role, considering that in the last years the construction sector has accounted for 36% of the total end energy use and for almost the 39% of the total energy and process related global CO_2 emissions [2].

1.1.2 Green Building Rating Systems: leading by example

The construction industry is typically a sector that develops and evolves with its own slower rhythm, especially when compared to manufacturing in general and other production chains. This is valid also for the adoption of new practices oriented to sustainability, all over the life of a building. In this regard one of the most relevant actions implemented in the AEC sector, is the application of sustainable building assessment schemes, in fact "the role of certification schemes is an important example and guideline which is already being recognized by the industry" [3].

Each of these protocols establishes a way to evaluate the sustainability of a building under study, throughout the different phases of its lifecycle, taking into consideration many possible fields and aspects. The final output of the adoption of these methodologies is a certificate issuing the grade of sustainability, always very intuitive and easy to be understood also by common citizens (e.g. platinum, gold, silver).

These Green Building Rating Systems (GBRSs) are in most of the cases available both for new and existing buildings, with different evaluation methods and application rules. Moreover, they can be often used for coupling the image of the building's owner or the construction's customer to a certain label of responsibility and sustainable activity, which nowadays is very appealing for brands seeking a respectable social reputation. But most importantly, they are extremely effective in setting new benchmarks for sustainable constructions. Setting a benchmark means also creating a new perspective, showing a concrete example that can be followed to obtain the same results or even better accomplishments, enabling a comparison between similar projects. This helps to share the best practices and to spread awareness, not only among the professionals but also giving a reference to everybody, hopefully boosting sustainability in one of the most impactful industries.

Accordingly, GBRSs can be useful in leading by example the future designs and opening the path for expanding the challenge of a more sustainable construction world to all the involved stakeholders.

1.1.3 Renovation of the European building stock

Considering the European landscape, 53% of the residential building stock was built before 1971 [4], when the first thermal regulations had not yet been released. In the same period there were very few laws about seismic design and the ones that existed were fragmented in the various countries, mostly related to the damages and experiences due to earthquakes happened during the first half of the 20th century. As a consequence, the current building stock strives for retrofitting in both regards of seismic capacity and energetic efficiency.

For instance, in the following image is reported the situation of the Italian building stock [5]:

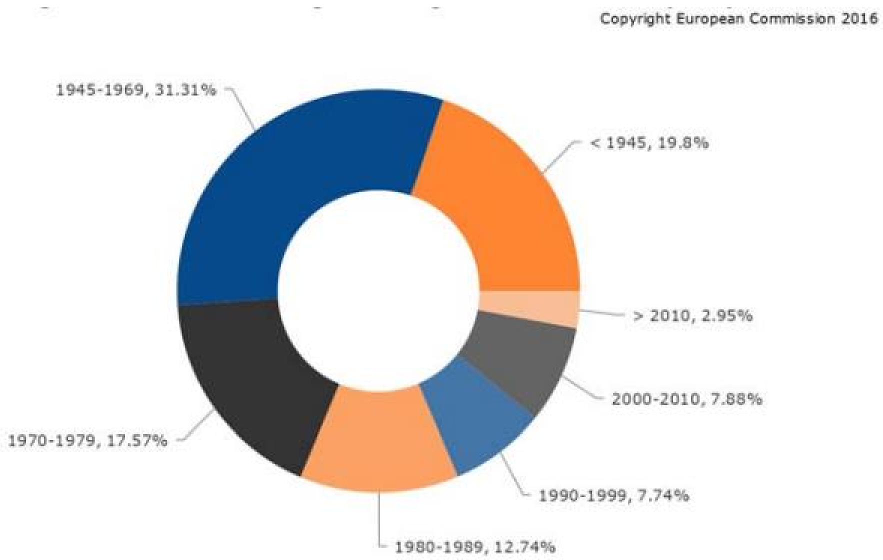


Figure 1 Share of buildings constructed in different periods in Italy, as it was surveyed in 2014 [5]

Indeed, the European Green Deal [6] pushes the different nations of the old continent to implement some measures useful to achieve a better condition, as it happened for instance with the Italian application of fiscal detractions for refurbishments, either in the energetic or seismic behavior of the buildings [7]. The renovation wave should also enhance the resilience of the buildings against climate change and the extreme weather events that are influenced by it, between which according to recent research it is possible to also list earthquakes [8].

Therefore, the development of this study refers to a seismic retrofitting case study.

1.1.4 Structural damages and environmental impact

A possible relation between sustainability and seismic design, concerns the damages to which a building can be subjected. The more damaged is the structure, the higher will be the cost of reconstruction or refurbishment. This is valid in financial terms but also concerning working hours, materials exploitation, disposal of the wastes, and many other fields related to the reparation after the event. All these aspects can be linked to the environmental sphere, since most of the operations involve a certain amount of equivalent CO_2 emissions, therefore affecting the sustainability of the building during its lifecycle.

All in all, a robust and safe structure, able to undergo earthquake solicitations and providing the required performances, may be considered as one of the bases for the achievement of sustainable constructions.

1.2 Issue definition

The following paragraphs explain what are the main points that are going to be addressed in the thesis.

1.2.1 How to address sustainability

Firstly, it is fundamental to define precisely what is sustainability. It is a concept that has been developed in the last few decades and it is based on various dimensions, the so called three pillars: environment, social, and economy. One of the first world-wide accredited definition of the concept was given in 1987 in the Brundtland Report, sponsored by the United Nations:

"Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs." [9]

In the current climate crisis, it is therefore necessary to highlight every possible way to reduce emissions that are harmful for the environment, always respecting the other two fields touched by sustainability, namely the economic and the social spheres. Hence, an evaluation of how much the sustainability of a building can increase if an adequate seismic reinforcement intervention looks to be on point.

This must be done also taking a close look to sustainable building assessment schemes and focusing on them as references for really understanding how the sustainability can be evaluated and correlated to the different aspects of the construction's sector.

1.2.2 Influence of seismicity

Keeping a focus on the behavior under seismic solicitations, such loads can widely vary around Europe depending on the position of the building. European projects in the recent period have focused on this theme producing maps [10] displaying the hazard and risk related to earthquakes in the old continent, where "earthquake hazard describes what level of ground shaking at the earth's surface is expected due to future earthquakes, earthquake risk comprises information about the potential damage of such strong ground shaking". [11]

As it can be seen in the next page, the maps intuitively show a huge gap between the effects that could be encountered in Mediterranean countries such as Italy and Greece, and in Central and especially northern Europe. This issue is going to be developed in the case study, placing the building in Aachen (North-Rhein Westphalia, west of Germany) and in Zambrone (Calabria, south of Italy).

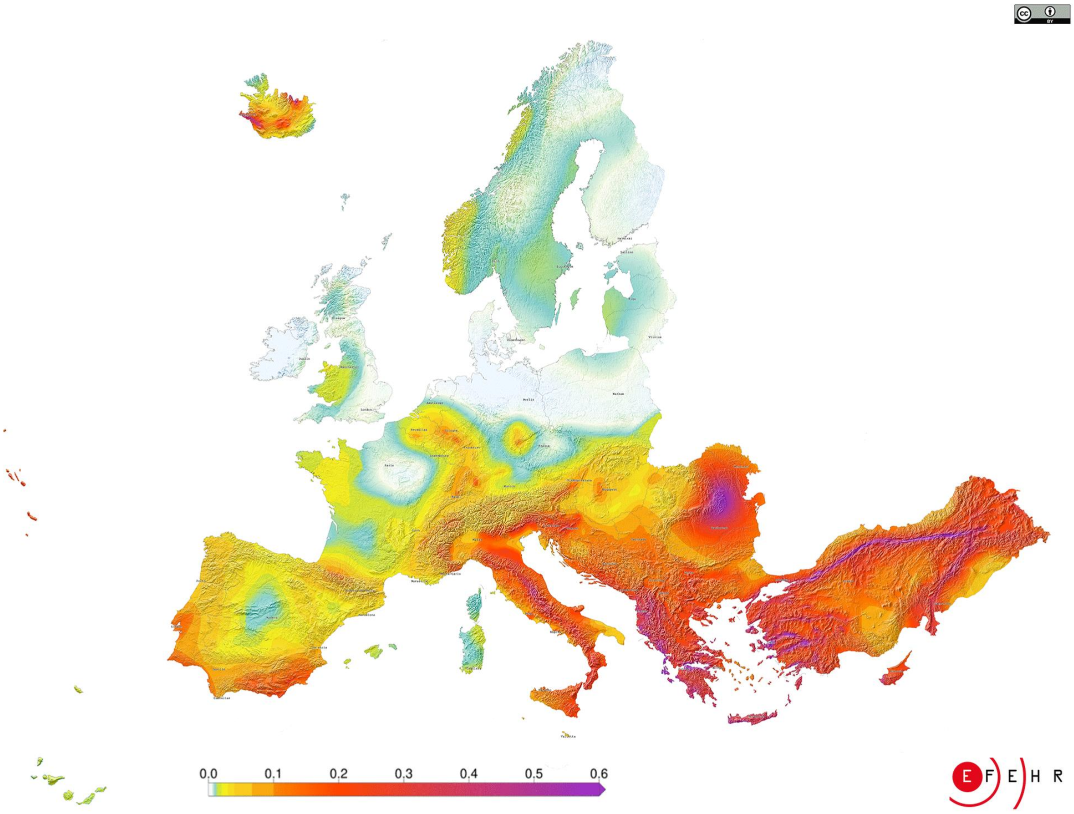


Figure 2 The 2020 European Seismic Hazard Model (ESHM20) [10]

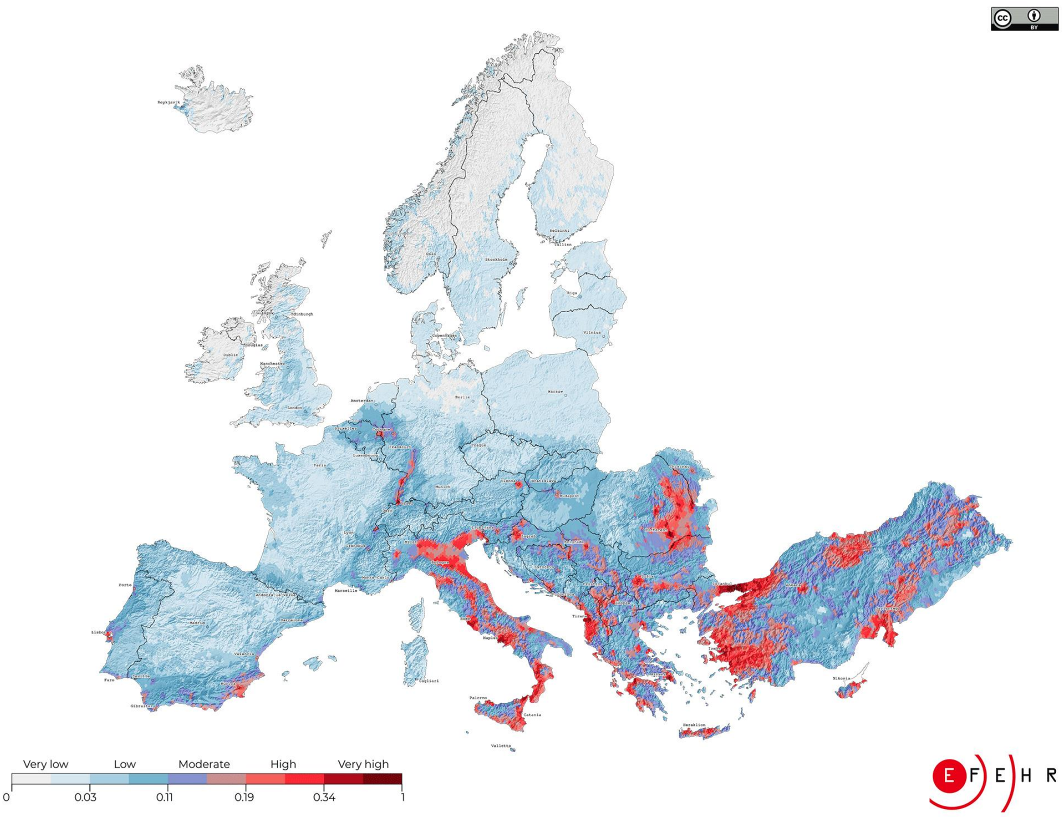


Figure 3 The 2020 European Seismic Risk Model (ESRM20) [10]

1.2.3 Generalization of the study for criterion proposal

Given that the main problems in terms of seismic resistance can be encountered in existing buildings, especially if approaching the end of their nominal life (usually taken as 50 or 60 years depending on the countries and the context), the investigation focuses on the refurbishment of old structures, carried out with different retrofitting techniques. With the goal of obtaining some representative environmental results, a very generic case study of existing building was developed.

An important point that is faced in the study is the use of alternative seismic analysis methods: modal response spectrum analysis method and pushover analysis applied with the N2 method. This is expected to provoke differences in the verifications, since one method is more sophisticated than the other.

Furthermore, the retrofitting technique is always studied as addition of shear walls to the existing moment resisting frame, but what differs is the material used for the creation of the wall: timber or reinforced concrete. Consequently, the environmental impact of the intervention could change depending on the technology adopted.

Lastly, the comparison of the results obtained with variations of the mentioned parameters, is supposed to lead to the final aim of the study: the proposal of an earthquake related criterion to be included in the sustainability certification schemes.

2 State of the art

This chapter aims to provide an overall view on the inclusion of seismic resistance and resilience into the Green Building Rating Systems most diffused in Europe, setting the base for finally proposing some critical suggestions regarding the criteria included in the schemes, with particular reference to the structural behavior under seismic actions and its impact on overall sustainability.

Firstly, here is an idealized example to shortly understand how these protocols work and how the goal of a new criterion proposal can be pursued. During the sustainable certification of a building, points are awarded if certain established criteria (dealing with environmental, social, or economical performances) are satisfied. In the same way as the investigated building can gain points thanks to the efficiency of its heating systems, allowing it to spend less energy compared to an average reference building, the certifications could involve criteria regarding the structural capacity. In particular, some points could eventually be awarded for an excellent response of the structure to earthquakes, compared to the average reference building, affecting the assessment of the final sustainability score and certification.

By consequence, a selection of GBRSs is performed, to analyze them in depth with the research on the state of the art. The most popular certifications around Europe, and also very famous globally, are chosen: the American LEED (Leadership in Energy and Environmental Design) and the British protocol BREEAM (Building Research Establishment Environmental Assessment Method). The German DGNB (Deutsche Gesellschaft für Nachhaltiges Bauen) and the Italian scheme ITACA are selected too. All of them are going to be investigated principally concerning whether they present relationships between seismicity and awarded points for the certification. As a comparison to this, the share of energy-related points in each sustainability certification will be calculated too, in addition to the gathering of general information as an overview of the selected GBRSs.

The results of this chapter will hopefully define a clearer frame for acting with the final aim of a criterion proposal regarding the seismic dimension.

2.1 Introduction to Green Building Rating Systems

The field of building certification is very broad, and it is developed in different subjects and objectives. Moreover, it is related both to legislative requirements and to voluntary certifications, with which the owner of the building pursues a certain goal following the criteria demanded by the selected protocol. The certifications space in many different applications, from the smartness grade of a building, passing through its resilience, to the assessment of how green the building is and many other options.

For sure, with deeper research, it would be possible to discover many other fields in which a construction can be certified, but the most popular kind of recognition of the recent period is the one concerning the sustainability of a building. These assessments collect many different sectors that affect the consumption of raw materials and emissions due to constructions, other than several aspects of sustainability, being it environmental, social or even economical. In a situation where the world is getting more active in facing the climate crisis, this type of assessment is largely demanded, and it is becoming a powerful tool in leading by example in the world of sustainable constructions.

This kind of certification is often applied to projects from medium to large extensions, requested by important clients also for achieving a sort of green labeling for their brand. Even if this behavior could be in some cases identified as green washing (i.e. "the creation or propagation of an unfounded or misleading environmentalist image" [12]), the incisiveness of such a procedure is hardly deniable: the final result of a building constructed or refurbished with innovative and green practices is a good starting point for driving the renovation wave of the old existing building stock. Indeed, the implementation of good practices is the first step to obtain a spreading of sustainability all over the society, also acting on the sensibilization of communities.

Each of these methodologies is based on the evaluation of building sustainability considering different aspects and life-cycle phases, delivering a quantitative output to represent what is the overall performance of the selected construction, evaluated in the most complete and detailed way possible. The sections and criteria are therefore covering practically every portion of the building: from the technical plants, with their efficiency and the use of renewable energies, other than a responsible use of water resources, to the choice of recycled and local materials; furthermore, different construction phases are taken into consideration, from the construction to the decommissioning and deconstruction or the demolition of the building. Despite the great complexity and variety of the topics undertaken, all these protocols scarcely examine (if not at all in some cases) the hazard of facing natural calamities, which are getting more frequent and powerful, as it has been demonstrated by the first visible manifestations of the climate change phenomenon. Apart from floods, storms, droughts, and other climate related hazards, not even earthquakes have a significant impact on the building sustainability assessment schemes.

The next paragraphs are a description of the considered certification systems, with a particular focus on what is their position with respect to resilience and seismic threats. In addition is described an overall view of the various GBRSs characteristics and a quantification of the weight of energy-related criteria in each of the analyzed schemes. Before concluding with some observations about the certifications, is also described the main reference given by the European Union for sustainable building assessment, which is the Level(s) framework.

2.2 Common background of Green Building Rating Systems

Before approaching each of the selected sustainability certification of buildings more in detail, it seems necessary to highlight what are the main points shared in these assessments and what are the characteristics that are common to all of them.

- Firstly, sustainability certifications are in any case pursued on a **voluntary basis**. These instruments are setting more advanced and more detailed standards for the design, execution and use of different constructions, with respect to what are the requirements of the laws and design codes.
- The rating systems’ goal is usually to enhance sustainability in the construction sector, by making the buildings’ owners and operators more environmentally responsible. The temporary window in which this awareness gain is expected, covers the **whole life cycle** of the certified buildings, “from cradle to grave”.
- The implementation of a particular certification is differentiated for the different possible projects to be certified. Different cases for what concern the **typology of intervention** are specified, such as neighborhood’s development, new buildings, operation and maintenance of existing buildings, and so on.
- In addition, each of the branches of the protocols can be further subdivided according to the **use category** of the selected building. E.g.: offices, residential, commercial, etc.
- Being developed in different countries and contexts, the various certifications can refer to **different standards and codes**. For instance, LEED makes reference to American standards such as ANSI, ASHRAE, ASTM; while BREEAM, being developed in Great Britain, refers to Eurocodes.
- Every certification has a system to **award points** and evaluate how green a building is. Therefore, each protocol has a particular structure, delineating what are the fields of study and the criteria used within the different categories, for the determination of sustainability. Finally, summing up all the points gained by the investigated building, the certification awards a final grade. Each certification has its own **grading scale** and combines the points obtained in the several categories with a specific method.
- In all the certifications, the **main category** is the one related to **energy**, which is considered as the most influential subject on sustainability within a construction. On the other hand, resilience in general and in particular the resistance of buildings to seismic actions are very scarcely considered.
- The certification guidelines are usually **open source**.

Considering the previous observations, it seems reasonable to divide the analysis of the protocols into the following paragraphs:

1) Introduction and GBRS development: identifying the context and timing in which the selected certification was created.
2) Intervention typology and building use destination: describing which are the possible fields of application for the analyzed protocol.
3) Certification's structure: depicting how the certification assesses points to the investigated construction, including what are the categories under study.
4) Evaluation of results and corresponding outputs: how the points are awarded throughout the certification and in which way the result is given out.
5) Focus on the fields investigated by the protocol: quantification of the weight of the energy-related criteria and the resilience-related criteria in the certification's grading, with a deeper analysis on what are the requirements related to earthquakes.

2.3 LEED

2.3.1 LEED – Introduction and GBRS development

Leadership in Energy and Environmental Design (LEED) is a certification whose developing started during the 90s, by the United States Green Building Council (USGBC) and it is one of the most diffused sustainable building assessment schemes all over the world. The first version was released in the year 1999. Since then, the protocol has spread all over the globe, in the same modality as the Green Building Council (GBC) connected with the institution of national councils in many countries, becoming a reference in the sector for various nations. The different conditions in which it may be applied are faced by the local GBCs with some adjustments to the methodologies.

Figure 4 LEED logo

For instance, the Italian Green Building Council, has developed some certifications along the lines of LEED protocols, which are then tailored to the cultural and architectonical characteristics of the nation. Namely, the Italian methodologies aim to assess respectively the sustainability of homes, historic buildings, multifamily dwellings, and neighborhoods. LEED's goal is to provide "a framework for healthy, efficient, carbon and cost-saving green buildings" [13].

2.3.2 LEED – Intervention typology and building use destination

The latest version of the protocol is LEED v4.1 which, as usual, focuses its study on different fields and applications in the construction sector. The following list represents all the possible certifications handed out by the USGBC [14]:

- Building Design and Construction (BD+C)
- Building Operations and Maintenance (O+M)
- Residential BD+C
- Interior Design and Construction (ID+C)
- Cities and communities

For the aim of this study, the last two categories are irrelevant since they do not deal directly with building engineering and structural aspects. Instead, the first three fields could be interesting and are consequently investigated in a deeper way.

Unfortunately, the handbooks of the new versions are available only the BD+C case. In the following table, the destination of use that can be certified for each intervention typology are listed:

Intervention typology	Version	Destination of use
Building Design and Construction (BD+C)	v4.1	• Healthcare • Hospitality • Warehouse and Distribution Centers • Data Centers • Retail • Schools • New construction and Major Renovation* • Core and Shell Development**
Building Operation and Maintenance (O+M)	v4 (January 2018)	• Existing buildings • Schools • Retail • Data Centers • Hospitality • Warehouses and distribution centers • Multifamily
Residential BD+C	v4.1	• Single family homes • Multifamily homes • Multifamily homes core and shell

* for all those buildings not considered in the other categories;

** for those cases in which at least 40% of the investigated building gross floor area is not yet completed at the time of certification.

2.3.3 LEED – Certification's structure

LEED certification is based on credits, awarded when the case study complies with specific criteria. Criteria corresponding to the same field of study are grouped into categories. The following are all the categories adopted into every LEED certification:

- Location and Transportation (LT)
- Sustainable Sites (SS)
- Water Efficiency (WE)
- Energy and Atmosphere (EA)
- Materials and Resources (MR)
- Indoor Environmental Quality (EQ)
- Innovation (IN)
- Regional Priorities (RP)

In addition, the version v4.1 for Building Design and Construction adds a new category, which was already used as an isolated criteria in the residential version v4: Integrative Process (IP).

Moreover, the requirements to gain points can be expressed either as “prerequisites” and as “credits”. As the own words express, all the prerequisites must be accomplished for being able to obtain the certification, while the credits are the real form of evaluation for the building’s sustainability, enabling to obtain points if the criterion is satisfied.

Furthermore, LEED establishes some types of credits, identified as “pilot credits”, which are slightly different. They cannot be used to get additional points, if not in the innovation category, but they could become actual credits in an upcoming version of the rating system, based on the feedback received by USGBC from the projects that pursue them. Therefore, these achievements are optional, but they could earn the project some points in the Innovation category, which will be further investigated in another paragraph.

2.3.4 LEED – Evaluation of results and corresponding outputs

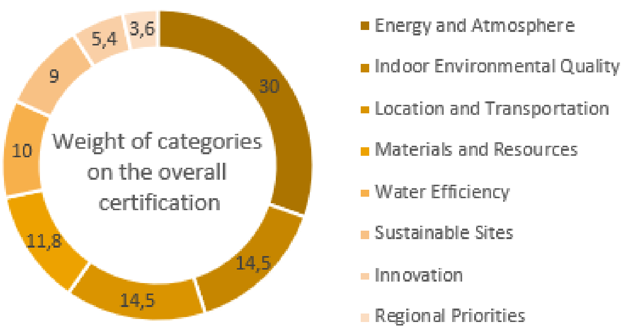
The efforts during the design and execution of the project can be focused on the strengths associated to the specific circumstances and characteristics of the case study, in order to gain points and achieve the expected outcome. Indeed, for each credit that is accomplished a certain amount of point is awarded in the certification. The number of points linked to the various criteria depends on the requirements themselves, being related on their incisiveness on sustainability, in fact the majority of LEED credits in the latest protocol version (v4.1) are related to operational and embodied carbon. Once the protocol’s criteria are checked, the final score is obtained simply by summing all the points gained in the various credits.

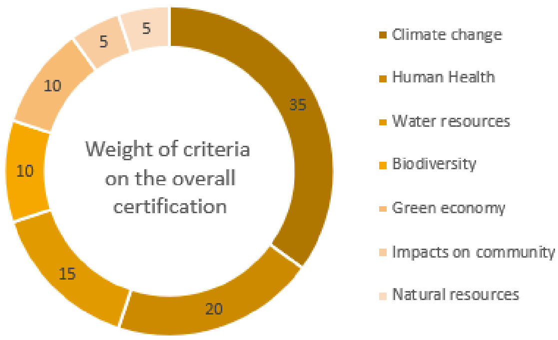
The maximum score is 110 points, but to obtain a certification the building must achieve a minimum score equal to 40 points. The assessment of the various sustainability levels is illustrated by the next graph.

2.3.5 LEED – Focus on the fields investigated by the protocol

The scope of this paragraph is to determine the weight of energy related criteria and to analyze the possible reference of any criteria in the certification to the resilience of the building, in particular regarding seismic capacity, assessing the corresponding weight in the certification.

Even if the points associated to the criteria may change from a field of study to the other (e.g. new construction and operation & maintenance), the total incidence of each category on the score has been roughly estimated [15] (displayed on the pie chart on the left). Instead, according to the USGBC website [13], the weight of the criteria grouped into different field, which do not correspond directly to the categories, can be considered (graph on the right):





The guidelines for the certification that were available are the following:

Manual of intervention typology certification	Version (year)
Building Design and Construction	v4.1 (2020)
Operation and Maintenance	v4 (2018)
Homes	v4 (2019)

Starting from the less recent guideline, it might be reasonable to proceed in chronological order.

- **Operation and Maintenance (O+M) (v4, 2018)** [16]
  Most of the credits from the category Energy and Atmosphere can be considered in the assessment of **energetic issues** weight in the certification. With a total of 37 points available on the 110 maximum points of the certification, the energetical field is considered to have an impact of **33,6%** on the sustainability of the operation and maintenance of existing buildings.
  With a deep study of its prerequisites and credits, it was possible to conclude that resilience and seismic actions are not included at all in the evaluation of sustainability according to LEED's protocol Operation and Maintenance. In fact, **no credits** involving resilience, especially of **seismic-related** type, are involved in this rating system.
- **Homes (v4, 2019)** [17]
  The certification for new buildings in the residential sector is subdivided into two options: multifamily midrise buildings and homes/lowrise multifamily buildings. It differs from the one of existing buildings for some criteria, for example "LT prerequisite: floodplain avoidance" is somehow considering resilience against flood, or more precisely, it involves an adequate design choice to avoid the risk of such a climatic event.
  For the multifamily midrise buildings, the total score for **energy** issues equal to 40 points over 110. The corresponding percentage is therefore **36,4%.**
  In the case of homes and lowrise multifamily buildings, the maximum amount of points concerning **energy** is 41, which accounts for **37,3%** of the total 110 score of the sustainability evaluation.
  Despite not having any actual credit concerning resilience, with additional research in the online LEED credits library [18] it was possible to find out the existence of two pilot credits focused on building resilience and spendable in the Innovation category. The interested criteria are "Assessment and Planning for Resilience" [19] and "Design for enhanced resilience", respectively accounting for 1 and 2 points (but for the latter only one point can be awarded for seismic-related merits) and rewarding the importance of a proper long-term planning and the actual implementation of design procedures with the goal of achieving a higher structural resilience.
  Hence the **resilience to earthquakes** might gain the project up to 2 points in the overall score for the LEED certification of residential buildings according to the version v4, thanks to the possibility to exploit Pilot credits in the category Innovation. This amounts to the **1,8%** weight of the sustainability certification.
- **Building Design and Construction (BD+C) (v4.1, 2020)** [20]
  Proceeding with the most updated certificate, the LEED BD+C covers several different building uses. Therefore, the evaluation and consideration of the credits could vary

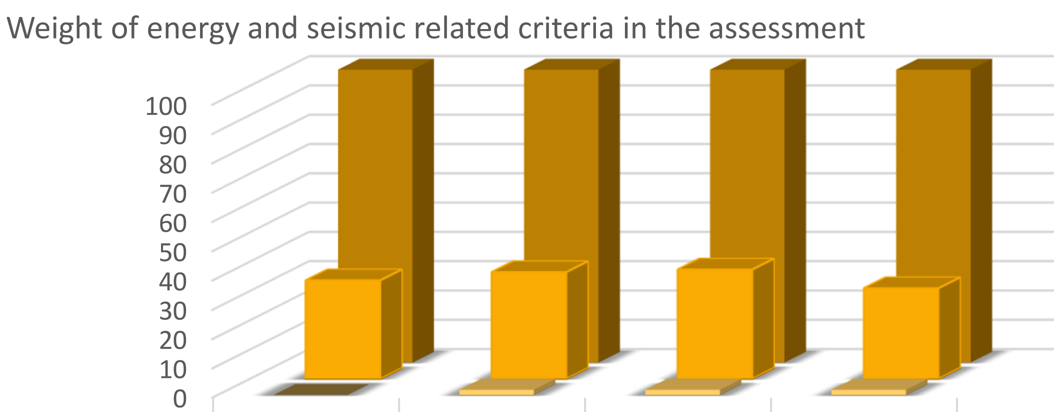
between different the various typologies of the certificate. Some differences are also present with respect to the other two assessments previously investigated.

Starting from energy issues, a new credit is regarding the Integrative process, whose aims also comprehend the definition of a target energy performance, no later than in the schematic design phase. This credit's value on the certification is one point.

The maximum amount of point due to **energy** issues for new constructions is available in the healthcare sector and it is 34 points, which is the **30,9%** of the total.

For what concerns the construction resilience, the same principle applied in the case of the residential buildings' certification is valid, with the exact same criteria and for all the destination of use considered in the BD+C protocol. Being always 2 points, the **seismic resilience** accounts for **1,8%** of the certification's score.

For wrapping up this paragraph, here is a diagram explaining the weightings of the energy- and seismic-related criteria in the various LEED certifications.



	O&M	Homes (midrise)	Homes (lowrise)	BD+C
Seismic-related criteria	0	1,8	1,8	1,8
Energy-related criteria	33,6	36,4	37,3	30,9
Maximum points percentage	100	100	100	100

Figure 5 Weight of energy and seismic related criteria in assessment of LEED certifications

2.4 BREEAM

2.4.1 BREEAM – Introduction and GBRS development

Building Research Establishment Environmental Assessment Method (BREEAM) is the first ever developed third party certification for the sustainability of buildings. It was first published in 1990 by the British association Building Research Establishment (BRE).

Figure 6 BREEAM logo

The association purpose is to act on multiple fields related to constructions, individuated as net zero carbon, whole life performance, health and social impacts, circularity and resilience, biodiversity, disclosure, and reporting. To achieve the best sustainability practices and to spread them, BRE has implemented several services, research centers, products and tools. Such tools range from the advisory, to the application of mere research in various fields concerning the construction sector, through the use of assessment methods in different cases, also with certifications that can be addressed to a variety of scenarios, such as buildings different life-phases, infrastructures construction, communities and home quality (with the specific assessment of the “Home quality mark” [21]).

An important characteristic of the BREEAM way of operating, is that for each building sustainability assessment, the protocol can be applied anywhere in the world except for some countries where the BREEAM National Scheme Operators (NSOs; local branches that apply the protocol in the relative country) worked to adjust the assessment scheme to the peculiarities of the local framework, as it happened for instance in Germany.

Further discussion is provided considering the sustainability evaluation methodologies dealing with buildings.

2.4.2 BREEAM – Intervention typology and building use destination

As far as the BREEAM certification is applied to buildings, there are three different typologies in which it could be used, where the feature that sets the difference between them is the stage of life considered. Thus, the are three possibilities in the sense of intervention typologies [22]:

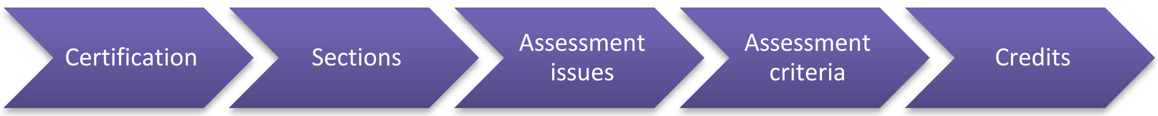
- New constructions:
  a framework for delivering high performing and sustainable newly built assets.
- In-use:
  enabling property investors, owners, managers, and occupiers to drive sustainable improvement in the operational phase of the buildings considered, leading to benchmarking.
- Refurbishment and fit-out:
  for the delivery of projects to a high performing and sustainable standard. Possible refurbishments are of the external envelope, structure, core services, local services, and interior design of existing.

Moreover, the destination of use for the buildings considered is very wide. As claimed by BRE [22], the following asset types can be certified, making no difference based on intervention typology, allowing the certification scheme to have a very broad application field.

Intervention typology	Version	Destination of use
New constructions	V6.0	• Houses and apartments • Education • Transportation hub • Hospitality and residential institutions • Community • Sports and leisure facilities • Government services • Healthcare • Offices • Retail • Industrial • Data centres Other
In-use	V6.0	
Refurbishment and fit-out	V2.2	

2.4.3 BREEAM – Certification's structure

The structure of the assessment is divided into **sections**, which are the same for each typology, while each section is composed by one or more **assessment issues**, which are variable from one certification to the other. For example, the new construction methodology presents different assessment issues in the international version (released in 2021) and the United Kingdom version (released in 2022); they can also vary between different building uses. Then, within each assessment issue a variable number of **assessment criteria** are allocated, with a corresponding number of points (also called BREEAM credits). The credits for each issue are assigned according to an explicit weighting system derived from a combination of consensus-based weightings and ranking by a panel of experts [23].



The sectors considered in the various protocols, with some slight differences in some cases, are:

- Management
- Health and wellbeing
- Energy
- Transport
- Material
- Water
- Waste
- Land use and ecology
- Pollution
- Innovation

For the in-use certifications, an additional category is implemented: “resilience”, while the section “material” is replaced by “resources”.

Another peculiarity, present in the case of Refurbishment certification for international non-residential projects, is that the scheme provides a modular framework divided in four parts: fabric and structure, core services, local services, interior design. To each of them some specific criteria are associated and can be used according to the type of refurbishment selected in the specific case.

2.4.4 BREEAM – Evaluation of results and corresponding outputs

Each criterion can be assessed following the guidelines and if the building demonstrates compliance to it, the associated number of points are gained in the certification process. Once the total credits obtained in a section are established, they are weighted considering the maximum points available on the certification score for the selected category: in this way it is obtained the overall environmental section score. Finally, all the sections contribute with their predefined weight, to the certification grading. Namely, the section scores are summed to evaluate the overall BREEAM score, which is associated to the rating of the building.

Additionally, for each Innovation credit achieved, 1% is added to the score, and provided that the several issues having minimum standards are satisfied, the certification is handed out.

The final output of BREEAM certification is a rating, that classifies the building as (in ascending order): unclassified (so the certification is not rewarded), pass, good, very good, excellent and outstanding. Each of them is coupled with a number of stars that varies between 0 and 5.

Here is the score required to achieve each results:

Considering that the evaluation is carried out with reference to a benchmark, BRE has also defined how good the intervention is compared to other projects. Based on the rating obtained, it declares to which percentage of best projects the case study is fitting in.

For example, in the case of refurbishment's certification:

Rating	Percentage of projects achieving the rating
Outstanding	Less than top 1% of refurbishment or fit-out projects (innovator)
Excellent	Top 10% of refurbishment or fit-out projects (best practice)
Very good	Top 25% of refurbishment or fit-out projects (advanced good practice)
Good	Top 50% of refurbishment or fit-out projects (intermediate good practice)
Pass	Top 75% of refurbishment or fit-out projects (standard good practice)

2.4.5 BREEAM – Focus on the fields investigated by the protocol

It is possible to further study the manuals of the various choices of BREEAM certification, with particular attention to the fields of energy-related criteria, which is usually the most influential subject in the sustainability assessments, and resilience with a focus on the aspects concerning the seismic behavior of the building.

The newest available handbooks for the three certification typologies are showed below:

Intervention typology	Certification manuals	Version (year)
In-use	BREEAM In-use International. Technical manual: residential	V6.0.0 (2020)
	BREEAM In-use International. Technical manual: commercial	V6.0.0 (2020)
Refurbishment and fit-out	BREEAM International Non-Domestic Refurbishment	V1.4 (2015, updated in 2017)
	BREEAM UK Refurbishment and fit-out. Non-Domestic buildings	V2.2 (2014, updated in 2020)
	BREEAM Refurbishment Domestic Buildings	V2.0 (2014, updated in 2016)
New constructions	BREEAM International New Construction	V6.0 (2021)
	BREEAM UK New Construction	V6.0 (2022)

For the developed research, the international guidelines are the ones of main interest, hence the investigation is furtherly developed for them only, starting from the oldest and proceeding in chronological order.

- **BREEAM Refurbishment Domestic Buildings** [23]
  The domestic refurbishment in this scheme is classified two categories: "Alterations to existing dwellings and extensions" and "Domestic conversions and change of use projects". Additionally, there are two options for the life cycle stages from which the environmental impacts are arising: "Design stage (DS)", leading to an interim BREEAM certified rating, and "Post refurbishment stage (PRS)" which enables a final BREEAM certified rating. The latter can also be approached by making a review of the interim DS assessment.
  The manual provides a table in which the weight of each environmental section on the overall certification grading is given. In the case of **energetical issues**, the total weight on the final score is **43%**. While for resilience a deeper investigation through the whole assessment guide was performed, identifying a relation to it only in the assessment issue "Pol 03 Flooding". It rewards the dwellings located in low flood risk areas, or the ones in which the refurbishment is carried out with a flood resistance particular attention. It is worth 2 credits over a total of 8 for the environmental section "Pollution", which weights 8% on the final assessment. This means that resilience issues are considered for a maximum of 2% of the rating system, but for **seismic-related** capacity there are no points awarded (thus **0%**).

- **BREEAM Refurbishment Non-Domestic Buildings** [24]
  In the case of non-domestic building complete retrofitting the environmental section "**Energy**" accounts for just the **19%** of the final score. Despite this result could appear quite low with respect to other certifications, it is decisive to notice how the BREEAM protocol considers a wider range of application fields than many other possible assessments. For instance, BREEAM collects 10 sections while LEED has only 8 categories. Then, there are other credits related to the building fabric that can be considered consequently related to energetical issues, which account for 0,6% of the final score.
  Regarding the building's resilience, it is taken into account slightly in the overall Refurbishment certification. The issues concerning this theme are "Pol 03 Flood risk management and reducing surface water run-off", "Mat 05 Design for durability and resilience" and "Wst 05 Adaptation to climate change". Considering also the chance of achieving credits from the Innovation section and exemplary credits, all together they amount to 8,1% of the global score.
  Continuing then with a more detailed inspection of the criteria related to **seismic** sphere of influence, the only assessment issue linked to it is "Hea 07 Hazards". This gives out the **0,7%** of the final grading, if a risk assessment is carried out (and if

seismic hazard is considered to be very influential for the case study's location) and mitigation measures are implemented.

- **BREEAM In-Use International (residential)** [25]
  From the beginning of this guideline, it is clearly stated that "the primary aim of BREEAM In-Use is to mitigate the operational impacts of existing assets on the environment."
  To do so, two asset types are distinguished for a better evaluation: apartment buildings and individual homes (each with further sub-types). Moreover, the assessment process is broken down into two parts:
  1. Asset Performance: benchmarking the performance of the asset, outlining areas of best practice, as well as potential scope for improvement.
  2. Management Performance: benchmarking the building management processes used within an asset, outlining areas of best practice, as well as potential to reach optimal asset performance.
  By consequence, the investigation on assessment issues typologies is split into the two parts. From tables available in the manual, it is easy to find out that the issues allocated into the **energy** environmental section weight **28,5% and 29,5%** respectively for asset and management performance. The same can be done for the section called "Resilience", which accounts for 14,5% of the overall score in the asset performance and 11% in the management part.
  Diving into the last section mentioned, a focus on the earthquake related weight can be performed. Regarding the asset performance part, the issue "Rsl 03 Natural hazard risk assessment" in which the absence of risks or the preparation of an emergency plan (also for **seismic** related hazards) is awarded with the **3,2%** of the final certification grading. Instead, in the case of management performance, the earthquake related issues are "Rsl 06 Emergency plans and climate-related physical risks", which could provide 2,4% weight of the overall rating, and "Man 04 Environmental policies and procedures" with an optional 1% thanks to its state of exemplary credit in case resilience and climate risks are addressed. Therefore, the maximum weight of **3,4%** is associated to seismic resilience in the management performance part.

- **BREEAM In-Use International (commercial)** [26]
  Also for the commercial version of the in-use certification there are multiple asset types that can be treated. The fields covered are very broad and they space from education to healthcare, involving also offices, retails and many other use destinations. Each asset is then ramified into different sub-assets. Again, the assessment is performed separately for asset performance and management performance. The weighting of "Energy" and "Resilience" environmental sections is then considered for each of the two parts.

The **energy** related criteria account for the **25%** and **27%** of the total certification, respectively in asset and management parts, whereas the resilience influences 13% and 11% of the scores for them.

About building resistance to **seismic** events, the issues addressing it are the same as in the case of residential certification, but with a different weight on the overall rating system. Indeed, the criteria are "Rsl 03 Natural hazard risk assessment" has an importance of **2,9%** in the asset performance, while the sum of the credits "Rsl 06 Emergency plans and climate-related physical risks" and "Man 04 Environmental policies and procedures" provides a weight of **3,2%** on the maximum score for management performance.

- **BREEAM International New Construction** [27]

  The latest international version for new constructions has been released in december 2021. According to the manual, "it is important to recognise that BREEAM primarily reflects the overall performance of the building rather than just the opportunities or limitations placed on specific stakeholders involved in the procurement process". In the same way as it happens for the refurbishment interventions, there is the possibility of achieving an interim certification during the design stage and a final certification, once the construction is completed (post-construction stage). The post-construction stage certification can be based on the interim assessment or on a completely new assessment.

  New construction is defined in the scheme as "'development that results in a new standalone structure, or a new extension to an existing structure, which will come into operation or use for the first time upon completion of the works".

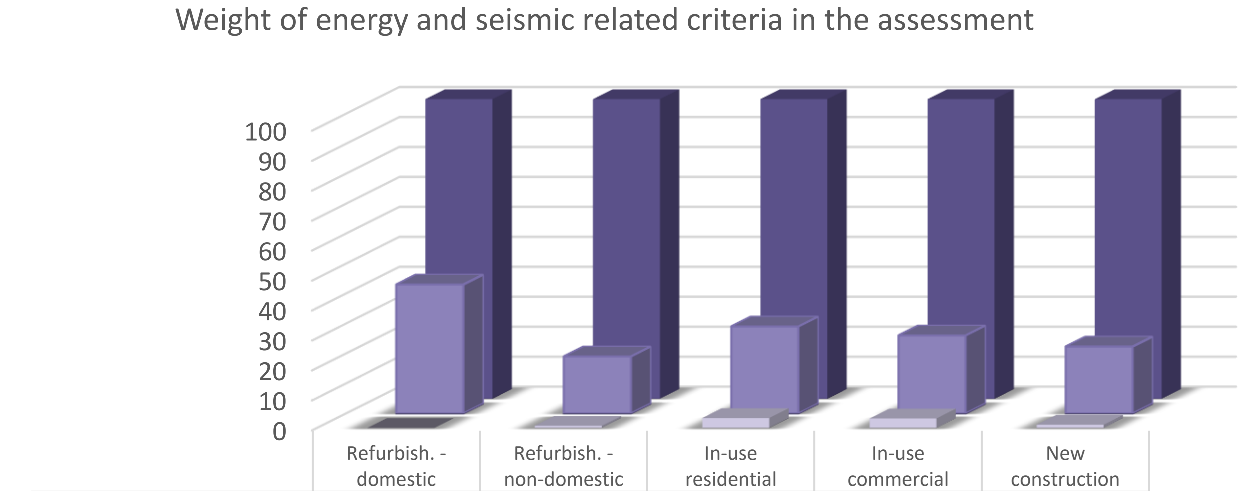
  Regarding the building use destination that can be addressed by the certification, the applications are covering practically everything, as it happened in the in-use certification, but collecting some particular cases under the category "non-standard building types".

  In addition, for different building categories the evaluation can be conducted with reference to particular frameworks, such as "fully fitted", "shell only" and "shell and core" for the non-residential buildings, "partially fitted" or "fully fitted" for dwellings.

  The weighting of the sections changes from one framework to the others, hence a simplification is made and the reported percentages for describing the importance of the topics are the highest possible to find between the various cases.

  Consequently, it is obtained that the **energetic field** is responsible for **22,29%** of the rating, while the resilience covers the 5,24%. Such value is affected in the share of **1,08%** by the **seismic** related criterion "Hea 07 Hazards", same issue addressed into the refurbishment scheme for non-residential buildings.

Finally, the following graph represents the percentage weight of criteria regarding energy and seismic events on the overall ratings of BREEAM certifications:



	Refurbish. - domestic	Refurbish. - non-domestic	In-use residential	In-use commercial	New construction
Seismic-related criteria	0	0,7	3,3	3,15	1,08
Energy-related criteria	43	19	29	26	22,29
Maximum points percentage	100	100	100	100	100

Figure 7 Weight of energy and seismic related criteria in assessment of BREEAM certifications

2.5 DGNB

2.5.1 DGNB – Introduction and GBRS development

DGNB stands for Deutsche Gesellschaft für Nachhaltiges Bauen, also referred in English as the German Sustainable Building Council [28]. It is a non-profit organization founded in 2007, that is part of the World Green Building Council [29], committed to the promotion of demonstrably good buildings.

Figure 8 DGNB logo

The association have various ways to promote sustainable buildings, within which it is possible to find the green building rating system, the so called DGNB academy and DGNB navigator, other than activities in the field of research and development. DNGB's final aim is to enhance sustainability, meant as quality and future viability, encompassing environmental, economic, and sociocultural factors.

In addition, DGNB has established the "DGNB Climate Positive award" for those buildings which demonstrably operate in a carbon neutral manner, based on their real consumption data. The association is also making efforts to increment its influence internationally and to share its expertise beyond German borders, in fact international versions (in English) of the protocols for new construction and in-use buildings were published in 2020, and for the renovation rating system in 2022. Besides, the schemes are referring in the most cases to international standards (ISO) for the setting of criteria and their evaluation. It is possible to adapt the system to the specific characteristics of the countries in which it is applied, after consultation with the DGNB certification body.

2.5.2 DGNB – intervention typology and building use destination

Going forward to DGNB's sustainability certification system, it consists of different possible assessments as regards the building scale, other than a certification tool for sustainable districts. Focusing on the study of single buildings, here are the options in terms of intervention typology.

- New construction buildings [30]:
  applicable to those projects completed within the last three years.
- Renovation of buildings [31]:
  this scheme is supposed to close the gap between the certifications of new and existing buildings, providing a protocol to certify buildings which are strongly refurbished and renewed. The methodology presents itself as very similar to a new building case, but with a deeper focus on the core renovation and considering also the user comfort. It can be adopted only if in the last three years were applied an energetically effective envelope refurbishment (façade and roof surfaces) for 50% or 25% of the total area and respectively one or two major changes in generation and distribution of building systems such as heating, cooling or ventilation.
- Existing buildings [31]:
  based on the actual use of the property being evaluated, certifying the sustainability and the good practices adopted in the operational phase of the building's lifecycle. It can be applied for buildings completed at least by three years, using the previous year's consumptions as an evaluation basis. This kind of certificate has a three-year validity, with the possibility of a simple recertification process. Unfortunately, the handbooks for this evaluation scheme are available only in German language.

- In-use buildings [32]:
  aimed at all the stakeholders involved in the active life of the case study, from the owners to the building users. It can be applied to a single building or to a portfolio, case in which the protocol shows all its potentialities. The certified building has to be in operation at least by 1 year. Obviously, all legal requirements must be fulfilled [33].

Other than the reported systems, DGNB has ideated some other protocols to certify the sustainability of different operations related to the construction sector. In particular, the systems available regard the deconstructions and the construction sites.

As far as regards the destination of use of the certified buildings, the options are the same throughout all of the possible intervention typologies. They are listed in the following table:

<table>
<tr><th>Intervention typology</th><th>Version</th><th>Destination of use</th></tr>
<tr><td>New construction buildings</td><td>2020</td><td rowspan="4">• office and administration
• educational
• residential
• hotel
• consumer market
• shopping centre
• department stores
• logistic
• production
• mixed-use</td></tr>
<tr><td>Renovation of buildings</td><td>2022</td></tr>
<tr><td>Existing buildings</td><td>-</td></tr>
<tr><td>In use buildings</td><td>2020</td></tr>
</table>

2.5.3 DGNB – Certification's structure

To analyze more precisely the DGNB certification, the latest international versions for both new and in use buildings were collected. Depending on the use destination, the evaluation can involve up to 40 criteria related to sustainability, which are meant to provide an incentive to establish sustainable buildings and processes in a long-term conception of the construction lifecycle [34].

Starting from the **DNGB System New Construction**, international version released in 2020, it is composed by six sections that collect the various criteria, from now on referred as **topics**:

- Environmental quality (ENV)
- Economic quality (ECO)
- Sociocultural and functional quality (SOC)
- Technical quality (TEC)
- Process quality (PRO)
- Site quality (SITE)

The first three categories are developed as vertical fields, each focused on its main goal, and they are all worth the 22,5% of the final rating. Instead, the last three topics are ideated as horizontal subjects, with the interdisciplinary function of touching different aspects of the three pillars of sustainability, and they account respectively for the 15, 12.5 and 5 % of the assessment value.

Then, each topic can be subdivided in one or more **criteria groups**, indicated by the topic code followed by the group's number (e.g. ENV1: effects on the global and local environment). Finally, to each group correspond various **criteria**, which are the real object of the evaluation. For instance, ENV1.1: building life cycle assessment. In each criteria many **indicators** are listed and can be fulfilled, with the consequential awarding of **evaluation points**.

All criteria must be considered in the certification process, otherwise the certification can't be awarded.

Moreover, the new construction certification collects many different schemes available under the DGNB Certification System, each one related to a different building use destination, from the ones listed in the previous paragraph. Consequently, for each of these schemes was implemented an own way to weight the criteria.

The evaluation scheme to be taken in consideration is selected according to the primary use of the building, identified as the function exploited in the higher percentage of the building's area. Furthermore, some minimum requirements are investigated and need to be respected to achieve the certification.

Another important feature of the system is the presence of a pre-certificate, which has the objective to act at an early stage of the design process, where the ability of influencing the final output of the project is notoriously higher than in more advanced phases.

All the certification's structure discussed so far is valid also for the Renovation assessment, whose guidelines are actually based on the ones for new constructions.

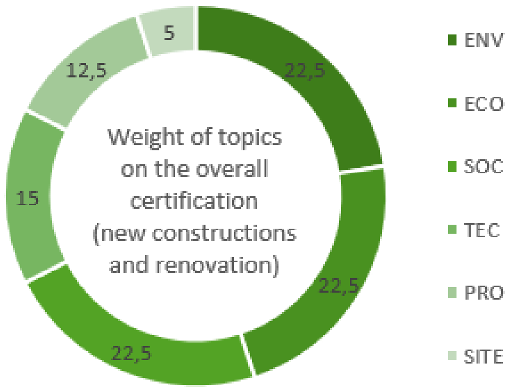
Going on with the **DGNB System Buildings in Use** (2020 version), it was created on a similar basis to the new construction case. The main difference is that, in spite of comprehending also the transversal criteria, it takes into account just the so called three pillars of sustainability:

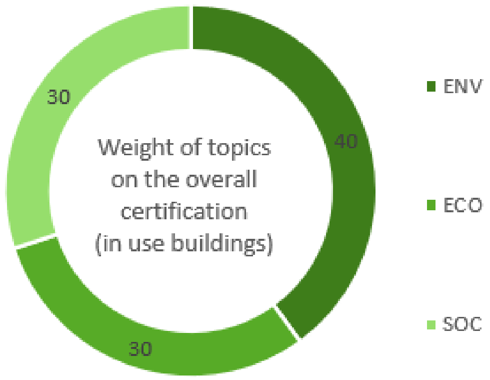
- Environmental quality (ENV), with a 40% of the rating's share;
- Economic quality (ECO), for the 30%;
- Sociocultural and Functionality quality (SOC), accounting for the 30%.

Therefore, the system rotates around three main fields of action, which correspond to the pillars of sustainability.

The other difference from the previously analyzed protocol is the lack of criteria groups. Indeed, the certification's structure ramifies directly from the topics to the criteria, which are a total of 9 elements (three for each field of study). Furthermore, those criteria are applied with an approach following a continual improvement philosophy, with a cyclic application of each criterion subdivided the indicators within each criterion in four main steps: plan, do, check, act.

In this case, the building use is not decisive for the aim of the certification, involving no change in the criteria to be used or in their weight.





To conclude this paragraph, also the **Existing buildings** scheme has an own structure, considering five different aspects: ecology, economy, social issues, technology and process. All these subjects are combined in a holistic and performance-oriented approach. This type of evaluation is divergent on some points from a new building certification, among which there is the focus on sustainable building operations, a reduced number of criteria (from 38 for new constructions to 22 in this case) and the fact that it is simpler to start the process, making it feasible to certify older buildings too.

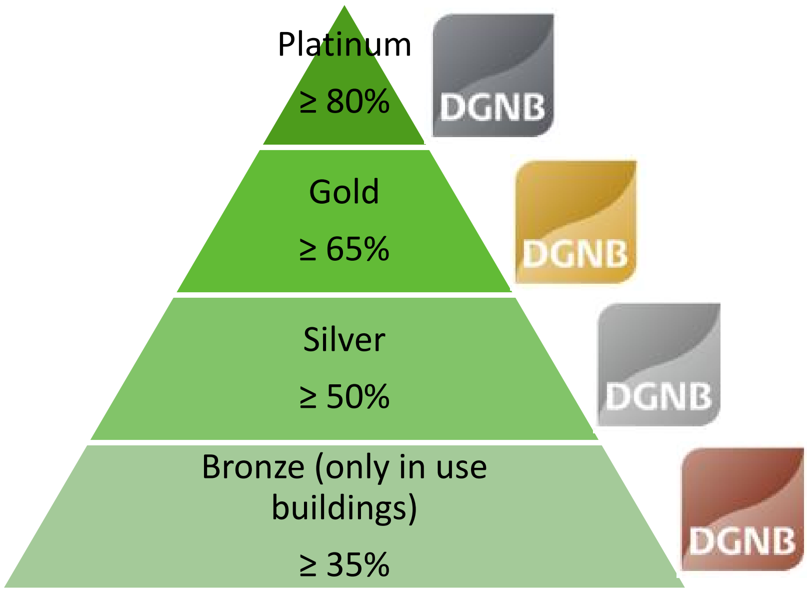
2.5.4 DGNB – Evaluation of results and corresponding outputs

In all the cases of new, retrofitted and in use buildings, the final quantitative output of the assessment procedure is denominated **performance index** and it is expressed as a percentage of the reference benchmark performance. In order to evaluate the case study, DGNB has set target values for each criterion and depending on the actual performance of the building, a certain number of **evaluation points** are awarded, representing the comparison with the target. Each criterion can be characterized by different **indicators**, to which the evaluation points are connected. Some of them can be subject to rewarding of some **bonuses for overfulfillment**, linked to the respect of UN Sustainable Development Goals. This means that by respecting additional requirements for certain criteria some more points are headed out, with the possibility of exceeding the maximum score of the considered criterion.

For new and retrofitted building assessment schemes, the results of the certification can be of three different types depending on the total performance index: Platinum, Gold and Silver. They also require the achievement of a minimum performance index in all the relevant topics (except for site quality), which is respectively 65%, 50% and 35%.

The same types of achievements are adopted also in the assessment of the renovation scheme.

For what concerns the scheme for existing buildings and the Building In Use certification, there is an additional level for the awards (Bronze) and no minimum performances are applied.



The Bronze level was introduced to comply with the eventuality of a limited optimization potentialities, acting on older buildings whose facilities and structures could be less performing. Instead, the building age classes are not considered since the evaluation in this way is easier and the comparison of buildings seems to be more meaningful.

2.5.5 DGNB – Focus on the fields investigated by the protocol

With a detailed inspection of the guidelines, it is possible to analyze what are the main fields of application and their overall weight on the certification. All the topics of evaluation can be applied to different building uses, corresponding to different weights for the criteria considered, but the manuals published by DGNB are just related to the intervention typologies and are the one that have already been listed.

In the following paragraph is discussed the percentage of the final score that is dependent on energy-related criteria and the same for those evaluations concerning resilience, with a particular focus on seismicity and the criteria connected to it. The three international protocols are going to be considered.

- **New construction buildings (international version 2020)** [35]
  Regarding the **energetic** issues the average weight on the certification between the many possible building uses is **27,04%**.
  Focusing now on the resilience related criteria, a very significant statement reported in the New Construction criteria manual is: “the scores attained in the assessment are always evaluated on the building’s entire life cycle”. This means also that the DGNB association’s vision is that the building must achieve a certain grade of

resilience, considering all the possible events happening during the whole life of the building. But in spite of the hopeful intentions, the certification accounts for resilience only with the 0,55% of the grading. It is done through the criterion "SITE1.1 Local Environment", evaluating the choice of the building site and its geographical peculiarities, with the relative hazards due to extreme events. Inside the mentioned criterion, **seismic** issues account for 20% of the evaluation points, assessing only the **0,22%** of the overall certificate score.

- **Retrofitted buildings (international version 2022)** [36]
  In this case the criteria are exactly the same as discussed for the new construction protocol, but the average of the **energy-related** weighting of those criteria is **27,19%**, while for the **seismic** resilience it is exactly equal as previously: **0,22%** due to the "SITE1.1 Local environment" criterion.

- **In use buildings (international version 2020)** [37]
  Adding the contributions of three criteria (ENV1-B, ECO3-B and SOC2-B) the final importance of the **energy** related issues sums up to **32,9%** of the total certification score. Whereas the resilience-linked criteria considering the **seismic** field are amounting to **1,5%** of the overall rating, considering the influence of geographical positioning of the case study with respect to probable calamities, pointed out in the criterion "ECO2-B Risk management and long-term asset value".

The next graphic representation is intended to show visually the amount of credits related to energetic and seismic fields, over the complete certification.

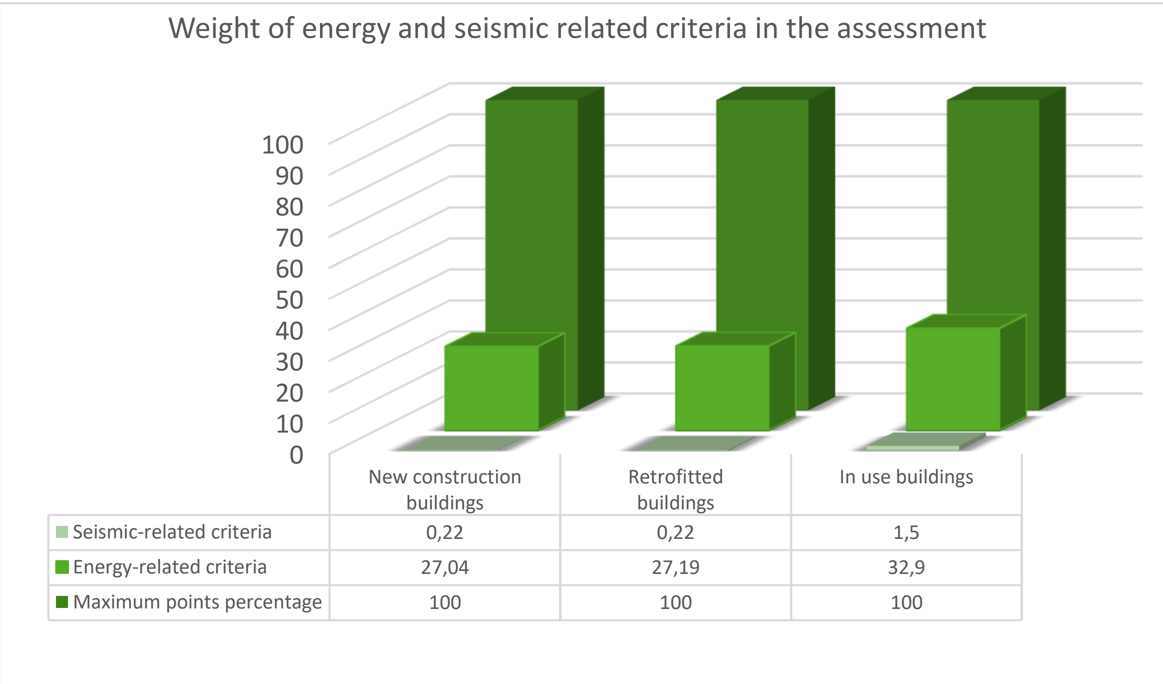


Figure 9 Weight of energy and seismic related criteria in assessment of DGNB certifications

2.6 Protocollo ITACA

2.6.1 ITACA – Introduction and GBRS development

ITACA (Istituto per l'innovazione e la Trasparenza degli Appalti e la Compatibilità Ambientale) [38] is an italian body managed by the Conference of Regions and autonomous provinces. It was instituted in 1996 to activate actions shared in different regions, to promote and guarantee an efficient coordination between them and a better connection to the statal organizations. Its main objectives are the transparency of construction tenders and their environmental compatibility.

Between ITACA's many activities, *Protocollo ITACA* is a Green Building Rating System thought as a guide to suggest buildings' environmental sustainability, providing instruments to measure the project's performance compared to the current common uses [39].

Figure 10 ITACA logo

2.6.2 ITACA – Intervention typology and building use destination

The protocol as it was conceived is more focused on new designs, but it was also developed in specific formats for the refurbishments of existing buildings. One difference from the certification schemes analyzed up to this point, is that ITACA developed different manuals based on the building typology (offices, commercial, and so on) instead of creating guidelines for the various interventions (new construction, retrofitting, etc.). Then for each criterion it is specified if it's included both in new and refurbished building assessment schemes or if it is just in one of them. Hence, different evaluation manuals were published considering various types and uses of the case study.

A negative note is that the latest versions of some protocols date back to 2011 and they could be nowadays outdated. They are available for the following use destinations: offices, commercial, industrial, and educational buildings.

Meanwhile, the residential building protocols (new construction and renovation) where updated in 2015 and 2019, following a collaboration with UNI (Ente Nazionale di Unificazione, which is the main provider for technical standards in Italy, that are often included in legislations and become in this way mandatory), giving birth to the reference practice PdR UNI 13:2015 "Sostenibilità ambientale nelle costruzioni – Strumenti operativi per la valutazione della sostenibilità" (constructions environmental sustainability – operative tools for sustainability evaluation), that accomplishes also the European normative of art. 2.2 of the EU regulation 1025/2012 [40]. The latest version of the same instrument is the PdR UNI 13:2019, that fixes the requirements inspected by the Italian accreditation body ACCREDI, but the only open-source scheme is the one from 2015.

The following table shows a little recap of the various protocols with the corresponding destination of use. As mentioned, the intervention typologies are the same for all the schemes and the difference is applied directly in the criteria.

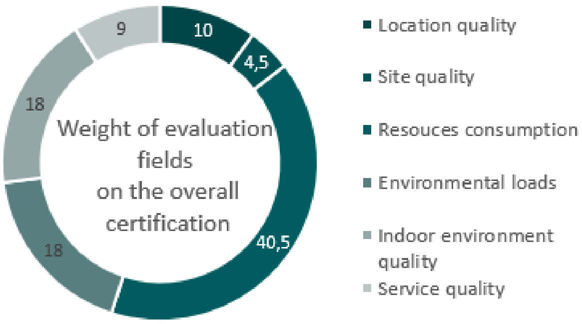
Destination of use	Version	Intervention typologies
Offices	2011 (published in 2012) [41]	• New design • Construction
Commercial	2011 (published in 2012) [42]	
Industrial	2011 (published in 2012) [43]	
Educational	2011 (published in 2012) [44]	
Residential	PdR UNI 13:2015 [45] or PdR UNI 13:2019 (not available)	

2.6.3 ITACA – Certification's structure

The ITACA protocol certification assumes the following structure, organized according to a hierarchical model:

1) The fundamental unit for the evaluation of the sustainability is the **criterion.** The criteria describe all the productive process, from the building site to the maintenance program. They are the main basis to evaluate the performances of single components of the construction.
2) Each criterion is part of a wider subdivision: the **category.**
3) The categories that act on the same subjects are grouped in **evaluation fields.**
   The list of evaluation fields used by ITACA is the following:
   - Site quality
   - Resources consumption
   - Environmental loads
   - Indoor environmental quality
   - Service quality
4) At last, more than one field could constitute a **tool**, which is the most general categorization utilized in the protocol.

Additionally, the protocol makes also reference to two main topics for the weighting of the criteria. In fact, a part of the evaluation field "Site quality", referred to as "Location quality", is comprehended in the topic "Site selection", which accounts for the 10% of the total evaluation, while all the other categories are included under the name of "Building quality" for 90% of the final score. Then each evaluation field has a certain weight, in percentage, in each of the two topics, which can be then converted into the total weight in the certification:



These weightings of the evaluation fields are always the same for the various building uses, but the change in weight is going to affect the criteria one by one, since the active criteria may change between the protocols.

2.6.4 ITACA – Evaluation of results and corresponding outputs

Depending on the intervention typology, on the context in which it is operated and on the final use destination, some criteria can be excluded by the overall evaluation of the building. In some cases, whole categories are disactivated. Once the framework of the assessment is set, for each criterion a **performance indicator** is calculated and then is compared to the relative **benchmark scale**, in order to assign a **rating** (that could vary between -1 and 5) for each criterion of the case study. Based on the evaluation of all the ratings interested in the framework, it is possible to determine the final **score**, aggregating all the single ratings. Since not all the criteria and categories have the same environmental impact, the final score is assessed as a weighted average of the ratings to their environmental impact. In this way, it is also possible to compare the sustainability of the building in different fields, obtaining results from the criteria to the tools and finally the overall score, following the hierarchical sequence that constitutes the certification framework (criterion, category, evaluation field, tool, overall score).

The overall score, exactly like the ratings, is included in a range between -1, meaning that the building has a performance below the standard and the current common practices, and +5, indicating a way more advanced performance. The grade 0 corresponds to those buildings which are just sufficiently complying with standards and represents the current construction practices. Since the association is strictly connected to the Italian regions, sometimes the regional bodies may institute tax or economic incentives in case a fixed threshold in terms of ITACA's overall score is surpassed.

The complete evaluation can be carried out by means of an online software, with free accessibility for all the users registered on the website of the protocol.

The following pyramid summarizes the overall scores that is possible to achieve in the certification:

2.6.5 ITACA – Focus on the fields investigated by the protocol

Having access to the many different manual versions, it is possible to investigate what are the criteria adopted for each type of building that can be certified, also distinguishing between new constructions and existing buildings that undergo a refurbishment. All the guidelines mentioned earlier were analyzed.

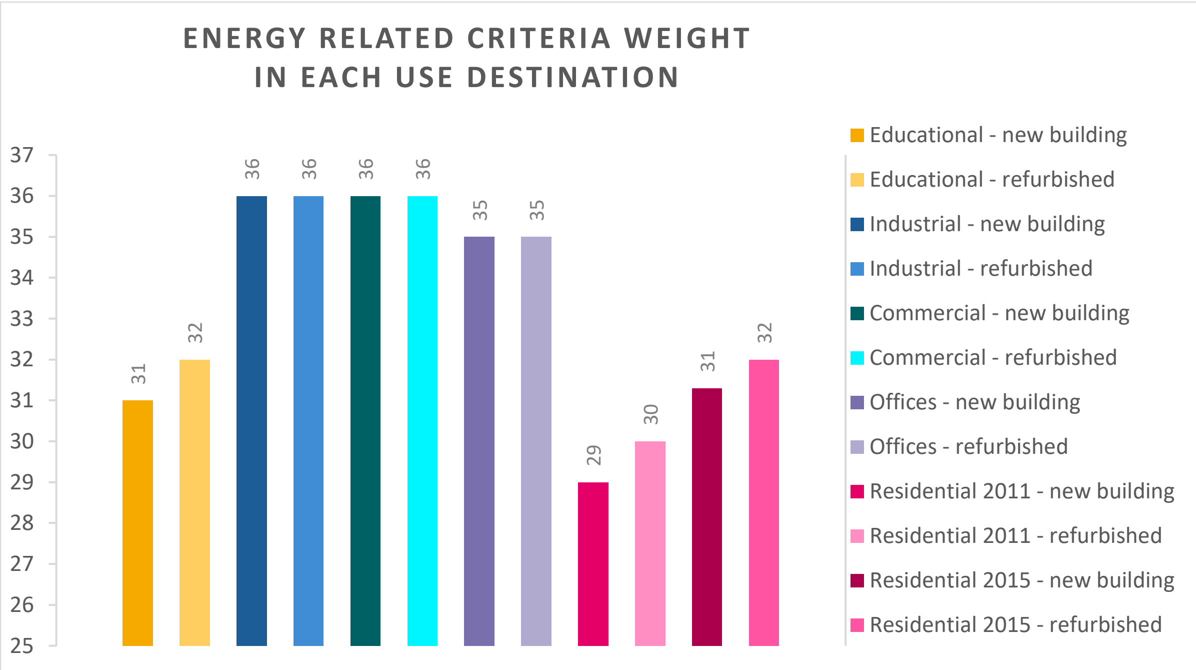
Following a thorough inspection of the guidelines, some observations arise:

i) What leaps off the page is that the from the 2011 versions are always the same criteria in the several building uses, but with some disactivated ones depending on the scheme.
ii) In the 2015 version for residential buildings were introduced just 3 new criteria: B.4.8 (local materials), B.4.11 (certified materials), and E.3.6 (home automation plants). The other criteria are the same used in the residential protocol of 2011.
iii) The criteria related to the site selection have the most variety in the industrial destination of use, but none of them is anyway related to seismicity or resilience. In fact, the choice of the location is evaluated most of all according to the position with respect to infrastructures and transports and then (just for the industries) it considers their possible influence on water resources and how much the plant is dispersed over the territory.
iv) The activation or deactivation of different criteria follows a certain logic depending on the conditions in which the assessment is operated: the destination of use and the type of intervention (on existing or new buildings).

The standard is lacking any reference to resilience, and in particular from a seismic-wise point of view. In any case, some criteria were considered worth of a supplementary investigation ("B.4.1: reuse of existing structures", "C.3.2: Solid wastes produced in operative phase", "C.1.2: Expected emissions in operative phase"). Even considering them, no inclusion of seismic-related or even resilience related subjects was made.

Thus, it is possible to conclude that ITACA protocols take in consideration the resilience of the buildings' structures for a total of 0% of its criteria and thereby the **seismic** resistance weight over the complete assessment of sustainability in the Italian protocol is **0%**.

Going on with to the total weight of the **energy**-related criteria on the sustainability assessment, their total number (for a mere quantification of the amount of such criteria, without considering the relative weights in the assessment), is 13 over a total of 49 issues, considering all the different us destinations together, consisting of the **26,5%**. But even more interesting is the precise evaluation of its weight in the different building use cases, and the results are summarized in the following graph:



Then, to enable a comparison with the other GBRSs, the results were subdivided into residential (only for the latest version) and non-residential buildings, always keeping the difference between refurbishment and new construction. The results reported above for the building uses that were grouped under "non-residential buildings" were unified with an average.

The final results of the investigation on protocollo ITACA are reported in the next page.

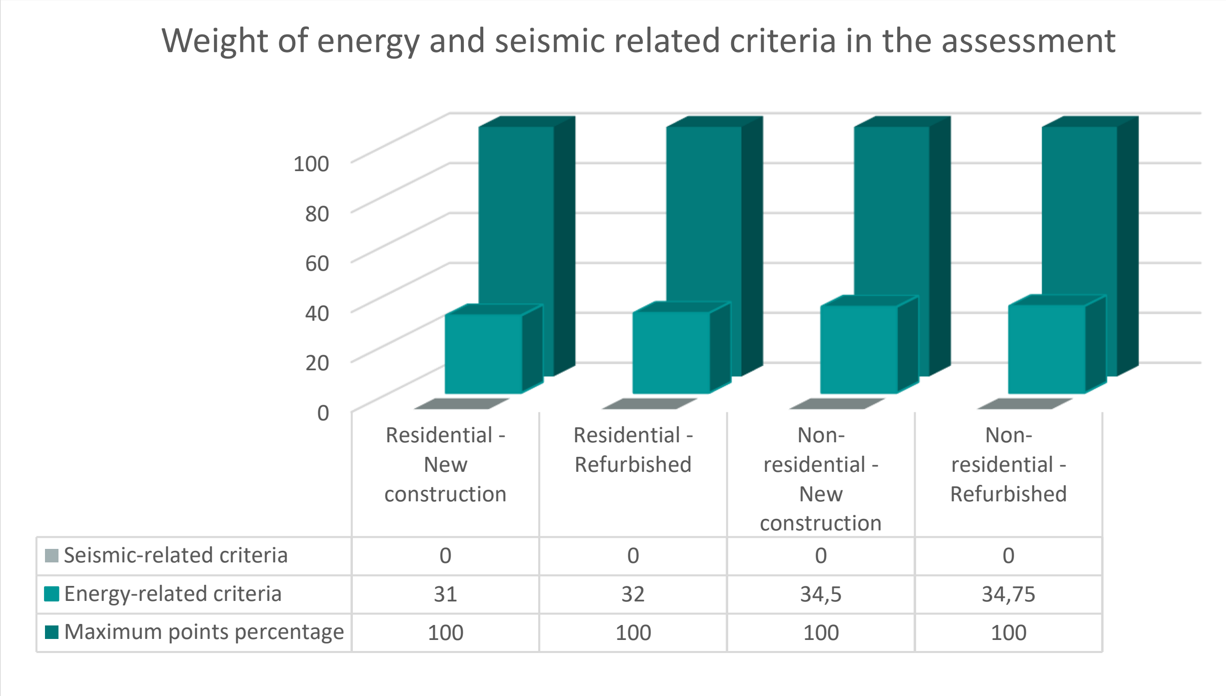


Figure 11 Weight of energy and seismic related criteria in assessment of ITACA certifications

2.7 Level(s) European Framework

2.7.1 Level(s) – Introduction

Level(s) is a voluntary reporting framework, which can be used by all stakeholders in the building and construction value chain. It is the first-ever European Commission framework for improving the sustainability of buildings, living by the values of flexibility, resource efficiency, and circularity. Moreover, Level(s) is the EU Green Deal's preferred approach to long-term sustainability in the built environment. [46]

Figure 12 Level(s) logo as reported in some documents

A good definition of the framework is provided by the Irish Green Building Council: "Level(s) is the EU initiative that joins up sustainable building thinking across the EU by offering guidance on the key areas of sustainability in the built environment and how to measure them during design and after completion" [47].

The main objective of the project is to set a common reference for dealing with sustainability of buildings in Europe. This also involves the idea of becoming a sort of guideline and inspiration for the updating of Green Building Rating Systems, which are constantly evolving and should reflect what are the newest challenges and innovations for turning the

construction industry into a more sustainable sector. Namely, International sustainability certification tools are aligning their schemes to Level(s), ensuring common EU policy objectives are integrated [48].
An important point for the ideation of Level(s) is that it is based on circularity. In fact, one of its aims is "to promote the use of life cycle assessment (LCA) and life cycle costing (LCC)" [46] which are valuable instruments for analyzing sustainability with a neutral and standardized methodology, like it has already been done in some assessment protocols (e.g. in DGNB guidelines).

The framework is based on research by the JRC (Joint Research Center of the European Union) conducted between 2015 and 2017 [49], developing indicators for the performance of the buildings in terms of sustainability. Then the Beta version of Level(s) was released, which was used to test more than 130 building projects in 21 countries [50]. After this test-phase lasting 2 years, feedbacks from professionals were gathered and used for a final calibration of the framework, which was released in October 2020.

Another goal of the initiative is to guide the designers along the process towards sustainability, which can be aided with the mapping tool and reporting template provided by Level(s) [51].

2.7.2 Level(s) – Intervention typology and building use destination

The Level(s) indicator can be applied to designed, built, and occupied buildings, in the various phases of a building life cycle [52]. Therefore, all the indicators developed in the programme are applicable to the embraced study, not depending either on the intervention typology or on the building use. By consequence, the comparison between different cases and buildings is much easier.
On the other hand, is it important to notice that the only two building uses taken in consideration by the framework are [52]:

- Offices
- Residential buildings

For what concerns particular intervention typologies, they're not precisely identified in the protocol, since it can be applied to any stage of the building's life, as mentioned previously.
Regardless of the type of project, the framework seeks the principles of circularity and considering the products' entire lifecycle.

2.7.3 Level(s) – Structure of the framework

Level(s) is delivered in the form of User Manuals and reporting templates. Each manual explains a sustainability concept, how to implement it and how to record and measure the results (using the templates). They can be used individually as standalone concepts but work better when approached as a suite (as there is some overlap) [47].

There are three different user manuals: the first is an introduction to Level(s), the second shows how to set up a project according to the framework's methodology and the User Manual 3 is a detailed description of how to apply each of the performance indicators. [53] With the use of these documents, the designers are able to be guided in applying sustainable principles and the final results of the assessment should not change depending on who is delivering them.

Then, the framework is composed by six **macro-objectives**, which are:

- Greenhouse gas emissions along a building's life cycle
- Resource efficient and circular material lifecycles
- Efficient use of water resources
- Healthy and comfortable spaces
- Adaptation and resilience
- Optimized life cycle cost and value

In each of them a certain number of **performance indicators** are grouped, to which a certain unit or parameter is coupled. They range from usual measurements like the Global Warming Potential to other more particular tools evaluating for instance the adaptability of the building. Each indicator is described by one to three **levels**, and then by a part of the **User Manual 3**, which is split into all of the indicators.

The whole protocol is subdivided into three different levels, from which the name was made up, that synchronize with the workflow of a design and build project [47]:

1) The first level doesn't involve any quantity and is thought to be a sort of introduction to the sustainability problems straight from the start of the design. The main fields of application for sustainability in the case study are defined and the main goals are selected, together with the client, since the early stages of design.
2) The second level has the goal of helping the designers throughout the decisions to be made, in order to achieve the best solution possible. Based on the choices made during level one, the professionals can apply the principles related to the main sustainability objectives, following the guidelines of the framework and referring to the indicated tools and standards.
3) The last level compels with the stages during the use of the building. It concerns the feedback given by the operational stages and by the monitoring what are the real results in comparison to what was the output expected, according to the design developed at level two. The goal is to understand in which fields the models and design strategies are more or less precise and where they need some corrections.

2.7.4 Level(s) – Evaluation of results and corresponding outputs

Being a framework from the European Union, Level(s) does not want to assess a final grade for the project, but it needs to be interpreted as a set of instruments to enhance sustainable buildings. It is not a Green Building Rating System, but a sort of guide for the various schemes that can be found around Europe, trying to underline which are the paths to be undertaken for improving the performance of buildings.

A very relevant feature of Level(s) is that it doesn't involve any benchmark, since the conditions in which the buildings are inserted are vary variable around the continent. On the contrary, the framework aims also to encourage the EU member states to develop their own set of benchmarks [47].

Moreover, the protocol provides the base for setting targets depending on the case study, during the early design stages with the application of the first level. In this way, despite the use of the same indicators, which still allows comparability, the application of Level(s) develops its own characteristics in each project depending on the case study itself and on the definition of the objectives with the client.

Another consideration to be highlighted is the presence of an online tool which guides the designers and other stakeholders in the application of the framework's guidelines [54], whose name is CAT (Calculation and Assessment Tool).

2.7.5 Level(s) – Focus on the fields investigated by the framework

"The sustainability indicators within each macro-objective describe how the building performance can be aligned with the strategic EU policy objectives in areas such as energy, material use and waste, water, indoor air quality and resilience to climate change."[55] Therefore, as it has been done for the Green Building Rating Systems analyzed so far, a further inspection of the issues addressed in the protocol is carried out, with particular focus on potential seismic-related criteria and on energy-related criteria. By consequence it will be clearer whether the European strategies for sustainable buildings are involving safety against earthquakes.
As expected, the energetical field is considered into the framework, by means of the indicator "1.2. Life cycle Global Warming Potential (CO_2 eq/m^2/year)", in which the level 2 corresponds to the calculation of the life cycle GWP emissions of the project using a software complying with the European standard EN 15978. In addition, an "hotspot analysis" should be performed from the results, which is basically a critical review of the most advantageous and feasible interventions that can be applied to obtain better performances.

Going on to resilience it is possible to take a deeper look into the macro-objective number 5 "Adaptation and resilience to climate change", and to notice that it contains three different indicators. Excluding the last one (5.3. Sustainable drainage), which is obviously not related to seismic events and not even to energetical issues, here is a description of the other two criteria:

- 5.1. Protection of occupier health and thermal comfort
  In the case of level 2, the indicator is addressed to designers that "are at the stage of having to assess the energy requirements of a building and wish to make a quantitative assessment of the indoor thermal conditions under projected future climate conditions" [56]. Thereby, also this indicator can be classified as energy-related for the purpose of this study.
- 5.2. Increased risk of extreme weather
  To the current state of the art, this indicator is applicable only at level 1, in which its purpose is to let all the stakeholders be aware of possible extreme weather events in the building location. Moreover, the design should be optimized for adaptation to such events.

The indicator 5.2. is in reality referring to extreme weather, such as "pluvial flooding, fluvial flooding, windstorms, coastal flooding, droughts, heatwaves, hail and snow" [57]. Consequently, it is not referring to extreme loading conditions due to earthquakes.

In conclusion, the Level(s) framework does not actually account for earthquake hazards through its indicators. But remembering the life cycle perspective in which the protocol wants to act, a consideration of risks and possible consequences of seismic actions might be a relevant topic to be developed in the framework. This is valid especially for the high-seismicity context in which the south of Europe its immerged.

2.8 Conclusions on current state of the art of GBRSs

The following diagram shows a comparison of the weighting for energy-related and seismic-related criteria in the four assessment schemes investigated during the research, considering various protocols available in each of them.

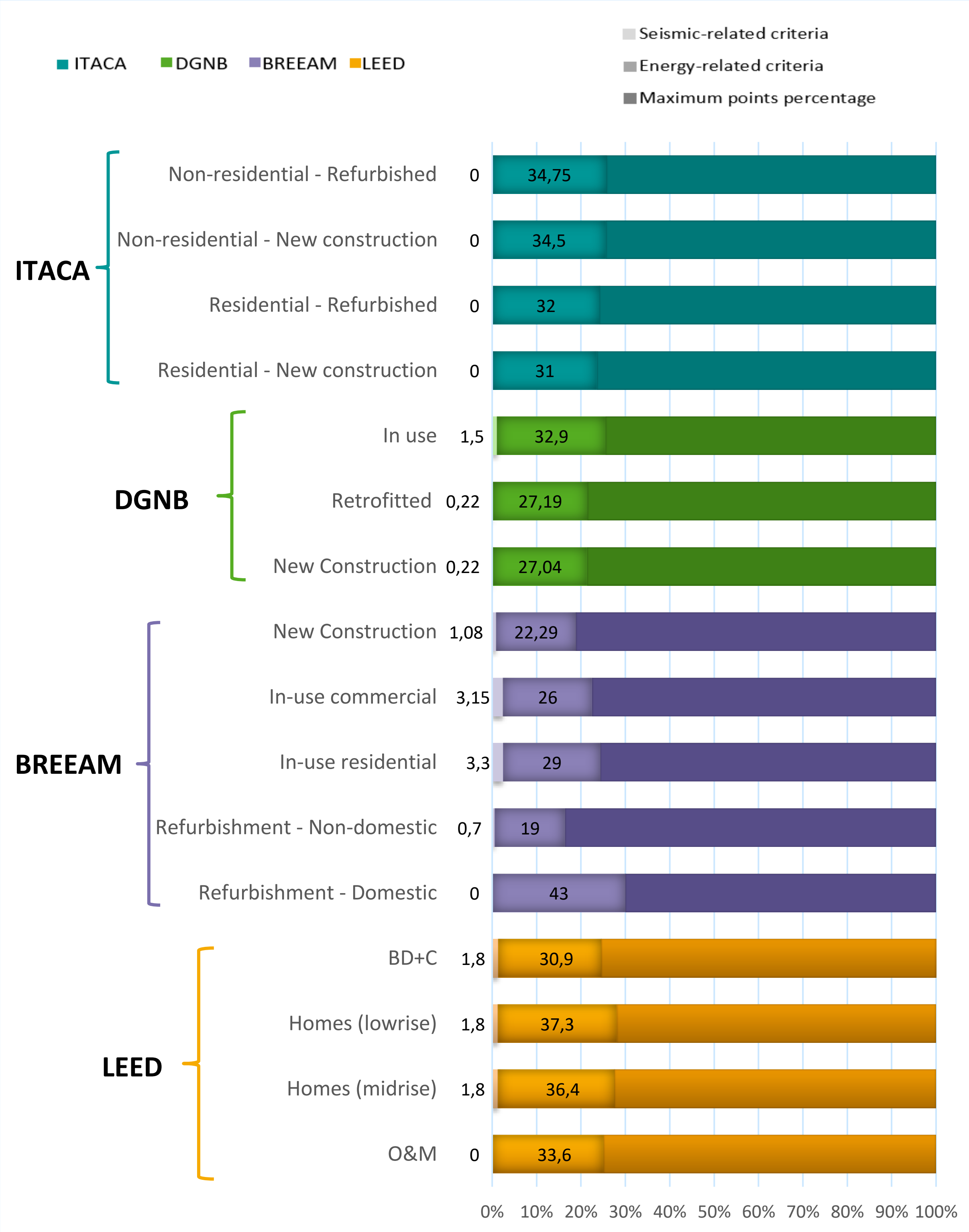


Figure 13 Weight of energy and seismic related criteria in the analyzed Green Building Rating Systems.

With respect to the results, some observations can be drawn:

i) the maximum weight given to seismic-related issues is 3,3 of the in-use residential protocol provided by BREEAM.

ii) instead, for the new construction and refurbishment schemes from the same operator, the rating percentage influenced by seismicity is lower. The same observation is valid for the DGNB certification, with a maximum of 1,5% due to seismic concerns in the "In use" certification and lower values for the other intervention types. On one hand, this looks almost like a sort of path where the earthquakes are most considered for existing buildings, but on the other hand, LEED certification for operation and maintenance does not involve any seismic connected criterion, while 1,8% of the total rating can be affected in case of new buildings, both residential and non-residential.
Consequently, it seems like the intervention type doesn't necessarily make a difference for the influence of seismic issues in sustainability protocols. The same can be extracted for differences in building use.

iii) Surprisingly ITACA doesn't introduce earthquakes at all, despite being an Italian protocol. This could also be influenced by the fact that the schemes were published a bit back in the years with respect to most of the other assessment analyzed. Also the other two schemes with no points at all regarding seismic issues, LEED O&M and BREEAM domestic refurbishment, were published respectively in 2018 and 2014 (updated in 2016).
Hence, there might be a very little tendency of including more consideration for seismic risks into the newest certification schemes.

iv) Another question was the possible correlation between the weight of energetic criteria and the one of seismic-related issues, which does not seem to appear in general. As a matter of fact, the highest rating for energy-related topics is provided in BREEAM domestic refurbishment, with 43%. The same protocol doesn't account at all for earthquakes. On the contrary, in the DGNB case the highest value is 32,9 for in use buildings, which has also the highest share of rating for seismic-related criteria (1,5%) among all the DGNB's assessment schemes.

v) Then, it looks like the certifications for the residential sector are giving higher weight to the energetical parts, compared to the protocols for other building types. There is an exception in this case too, with ITACA protocols having higher energy-related criteria weightings for non-residential buildings.

In general, the weighting of the criteria is very complex and depend on many factors, primarily also on the other criteria and topics that are covered by the assessment, which change from certification to certification, influenced by the use of the building too. Thereby, an actual evaluation of which should be the right percentage of weight for both energy- and seismic-related topics would be very complicated to implement. Anyway, one hypothesis could be setting an ideal weight of energy-related issues around one third of the overall evaluation, with an importance around 5% for resilience of the building, including aspects regarding safety against earthquakes and, generally, an appropriate structural resistance.

Besides, it is important to take a closer look to what aspects of seismic safety are considered in the assessments. Indeed, in all cases of the most common Green Building Rating Systems in Europe there are **no references to the seismic design** of buildings, but **only to their location** and the relative earthquake hazard.

Substantially, when resistance to earthquakes is taken into consideration, it is done only by evaluating the hazard of seismic actions in the construction’s location or region, whereas the importance of a safe and robust structure is not included. This appears as a bit of a missing issue in the certifications, since the high efficiency of materials and building system could become useless in case of a devastating event such as earthquakes, especially in the case of seismic activity with a very long period of return.

That implies that a sustainable structure must also be a safe structure, otherwise damages in the elements (with the corresponding need for refurbishment and consequential environmental impacts for waste produced, new materials, application) and risks for people could jeopardize the actual sustainability of the building.

The consideration assumes high relevance especially in those countries where the seismic activity is particularly intense, such as are the Mediterranean countries for the European continent, like Italy or Greece.

The following questions are derived from this state of the art, to be addressed in this study:

- In which way might seismic design of a retrofitting be included in sustainable building assessment schemes?
- When does resistance to seismic actions improve the sustainability of a building?

3 Methodology

Once a clear picture of the GBRSs has been given, it is possible to proceed with the next steps of the thesis. Since the goal is to find a correlation between seismic retrofitting design and environmental impacts, the procedure will involve both these fields, through the investigation of a case study.

Here is a schematic list of the main parts in which the work is subdivided:

1) Ideation of a case study.
2) Dimensioning of the existing structure, considering different actions linked to the two possible locations in which the building is placed: Germany and Italy.
3) Verification of the structure considering the current seismic loads according to the Eurocodes, using the modal response spectrum analysis, to define whether a retrofit intervention is necessary.
4) Design of different retrofit possibilities: timber frame shear walls and concrete shear walls, in both the locations.
5) Evaluation of results in terms of structural behavior, using the pushover analysis coupled with the N2 method, for the existing building and the retrofitted configurations.
6) Evaluation of results in terms of environmental impact, with the use of EPDs.

The next paragraphs aim to describe exhaustively all the procedures actuated in the mentioned steps, while the disclosure of the software-based numerical analysis and the corresponding outcomes will be treated in the next chapter.

3.1 Ideation of a case study

In this part of the study, it is necessary to select a building that could be placed both in the south of Italy and in western Germany in a realistic way. Precisely, the two locations are:

1) **Zambrone** (Calabria region, south of Italy, coordinates: (38.70, 15.99));
2) **Aachen** (Nordrhein-Westfalen region, west of Germany, coordinates: (50.78, 6.08)).

Such a choice for the positioning of the structure was made in order to compare the seismic and environmental study in two areas in which the seismic hazard is quite different.

As it is known from past events, Italy is a country with a quite high seismic hazard, especially in the center and the south, while Germany is relatively safer in comparison. In both the cases were selected municipalities in which the risk is among the highest in the respective nation.
In particular, the selection of Aachen (whose name in Italian language is Aquisgrana) seemed a natural choice since it is also where RWTH university is based. Instead, in the Italian

counterpart, choosing a town placed in Calabria appeared very interesting for the historical record of several disruptive earthquakes that took place in this region, as it happened for example in the end of 1908 with an earthquake that caused around 125000 fatalities between Messina (Sicily) and Reggio Calabria [58].

The following image is a representation of the municipalities object of this study. The color ranges are taken from the seismic maps of the two countries, from which it is possible to extract the peak ground acceleration (PGA) associated to a return period of 475 years. Such value of PGA is used in seismic design for ultimate limit state (ULS) (it is fundamental to notice that the colors on each seismic map are referred to the relative country only and are not using a common color range for showing the entity of the PGA):

Figure 14 Locations of the case study and superposition of the PGA value maps used for seismic design for Germany [59] and Italy [60]

Then, instead of looking for two similar buildings, each located in one of the two contexts, it was preferred to compose a case study from scratch. By doing so, eventual lack of information on the existing buildings were avoided, substituting them with assumptions considering the background both in terms of geography and historic period.

3.1.1 Construction technology and building use

Starting from the construction material, the choice made reflects the fact that **reinforced concrete** is the most widespread all over the world. Such technology was considered one of the best fits in years in which the construction sector was expanding, thanks to its availability, cheapness, and various application opportunities. Therefore, it seems reasonable to use it for both the locations, giving the possibility to make some comparisons eventually.

Consequently, this choice enabled to study a structure quite generic and with a rather simple scheme, that could be used for both residential and office buildings, which are the only two building use destinations so far considered by Level(s).

3.1.2 Reinforced concrete structure typology

Moreover, a reinforced concrete structure could be considered of different types depending on the way in which it resists to horizontal forces. A core could be inserted, for example in a position hosting the vertical distribution elements (stairs and lifts), or some shear walls could be placed in different sections, or even a structural frame could be designed to withstand the horizontal loads without any reinforced concrete wall. The latter is usually referred to as a Moment Resisting Frame (**MRF**) and it was chosen for the case study. This is also due to the idea of introducing a **retrofit exploiting shear walls**, which might be particularly feasible in the case of a structure composed only by beams and columns.

3.1.3 Geometrical scheme of the building

Once both the structural material and typology were established, the next development was about the geometrical features of the building. Considering the mentioned possible use destinations, a simple grid concerning three rows of five pillars looked like a good solution, with a span of 6 meters between every column. Then, in a plan view, the grid is composed by eight identic squares, each with a side length of 6 meters. Instead, considering the elevation, the space between the beams' axis of two consecutive floors is of 3 meters, for all the five floors.

Here are some representations of the structure's geometry, extracted from the numerical model implemented:

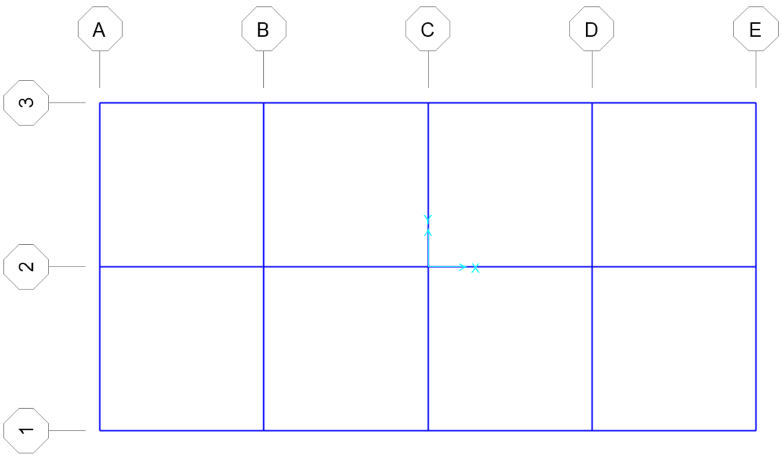


Figure 15 Plan view of the model

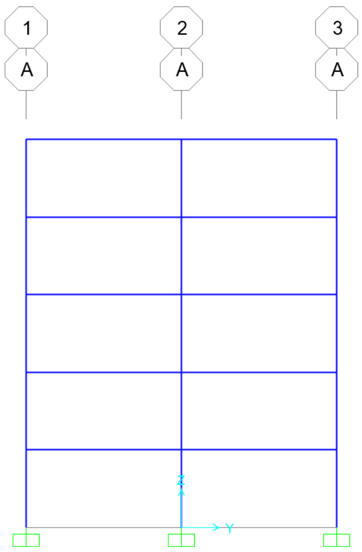


Figure 16 Transversal elevation of the model

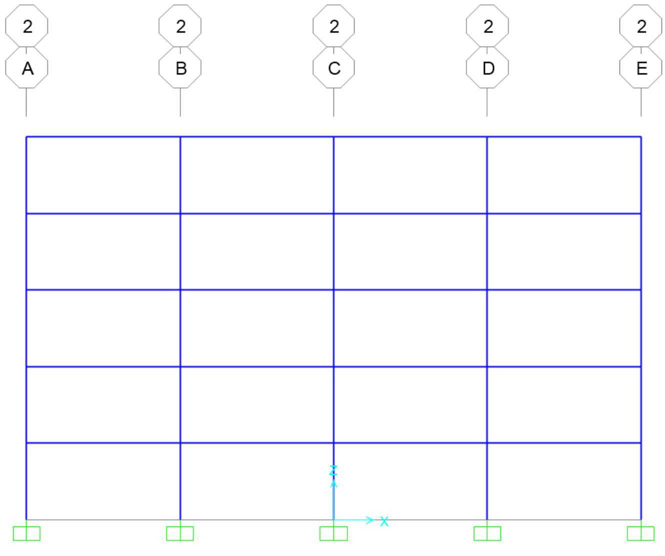


Figure 17 Longitudinal elevation of the model

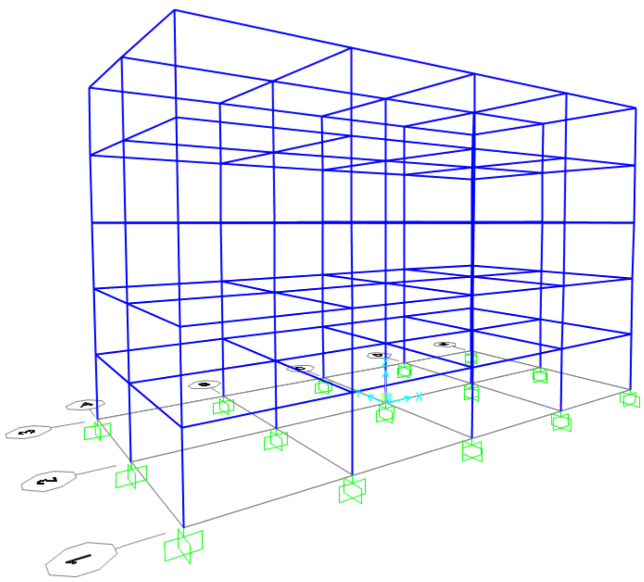


Figure 18 3D view of the model

A **simple geometry** like the one selected scheme is one of the prerequisites indicated by the Eurocode 8 [61] for the utilization of simplified methods, for instance equivalent lateral force method. But also modal response spectrum analysis or nonlinear methods might be used.

The model and the case study in general are referred only to the part of the structure above the **foundations**, to avoid excessive complications due to the interaction of such elements with the ground. As it can be seen in the previous figures, the model does not represent the foundations but their connection to the upper structure was assumed as a **fixed constraint**, in all the columns, involving that the frame is highly hyperstatic.

3.2 Dimensioning of the existing structure

For this procedure various assumptions were made, trying to reproduce a model as realistic as possible in terms of dimensions and characteristics of the existing structure, ranging from the type of loads applied for the design to the materials used.

3.2.1 Load combination

After having set the geometry of the case study, it was possible to start with the dimensioning of the structure with a rather simple method, using for both the locations the so called "**fundamental load combination**" as reported in the current Eurocode 0: [62]

$$F_{Ed} = \sum_{j\geq 1} \gamma_{G,j} G_{k,j} \quad "+" \; \gamma_P P \; "+" \; \gamma_{Q,1} Q_{k,1} "+" \; \sum_{i\geq 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \qquad (1)$$

Where:

"+ " implies "to be combined with";

∑ implies "the combined effect of";

G are the permanent actions on the structure;

Q are the variable actions on the structure;

P are the actions due to prestressing;

γ are partial factors (found in annex A of the same Eurocode);

Ψ are factors reported in the annex A of the same Eurocode, accounting for the non-contemporaneity expected for variable loads.

The loads considered for the computation were:

- G_1: permanent loads due to the weight of structural components, in this part of the study calculated as a function of their volume and density;
- G_2: permanent loads carried by the structure, due to non-structural components (infill walls, flooring, etc.);
- Q_i: variable loads. The considered variable loads were: occupancy load, snow load and wind load.

Some of the involved variable loads, such as the wind and snow effects, differ based on the geographic context in which the design is carried out. Therefore, the dimensioning of the structure varies between the two selected locations. The calculation of these loads was performed with reference national annexes of Germany and Italy.

For what concerns occupancy loads, in the first place both loads for offices and residential buildings were considered, also with the corresponding reductive coefficients accounting for the non-contemporaneity of loads on different floors (as suggested by Eurocode 1 [63]) and differencing the loads on the occupied floors from the loads on the roof.

The non-contemporaneity factor can be calculated as:

$$\alpha_n = \frac{2 + (n-2)\Psi_0}{n} \qquad (2)$$

Where n is the number of storeys (>2) above the loaded structural elements from the same category, while Ψ_0 is the same factor that multiplies the variable loads in the fundamental load combination.

After a first analysis it was decided to go on only with the case of offices (category B), since it involves higher variable loads due to occupancy (on floors). The same values for these actions were introduced for both Italy and Germany, using the specifications reported in Eurocode 1 [63]:

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to 2,0	2,0 to 3,0
- Stairs	2,0 to 4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5

Figure 19 Distributed loads applied depending on the use category of buildings, according to Eurocode 1 [63]

No thermal effects were considered in the study and, given the type of structure, no prestressing actions needed to be introduced.

The application of the loads on the structure and the consequential verifications were carried out by means of a numerical model implemented in the software SAP2000 by CSI (Computers & Structures, Inc.).

In addition, some thought to the consideration of seismic action was given. In the German case study, no seismic load was applied for the dimensioning of the existing structure. Indeed, the seismic loads in the country are usually low and the structure was fictitiously designed in the 1970s, period in which seismic design was not much diffused in Germany and probably the regulations did not even account for it. On the other hand, in Italy some standard with this regard had already been published, thus a deeper investigation on that period legislation is worth to be undertaken.

3.2.2 Seismic design in Italy in the 1970s

The development of standards and laws for design against seismic loads in Italy has always followed a path of event-response. This means that after each disastrous earthquake some new measures were adopted to try to reduce the damages and fatalities for similar calamities, but the problem was never really tackled with a long-perspective view. The same situation probably happened also in other countries all over the world.

In addition, the legislation throughout the years had to introduce new methods for seismic analysis, as they were developing and new tools were available.

This introduction leads to the 1970s. In that decade, after a long story of seismic activity in the Italian peninsula, the first definition of different seismic zones in the country was officially documented in 1974 [64]. Then, the ministerial decree of the 3rd of March 1975 [65] introduced for the first time the possibility of exploiting dynamic methods of analysis, also defining a response spectrum curve influenced by the recently established seismic zones.

The following graph shows a comparison of the spectrum according to that standard and the spectra obtained from the Eurocode, considering the municipality of Zambrone:

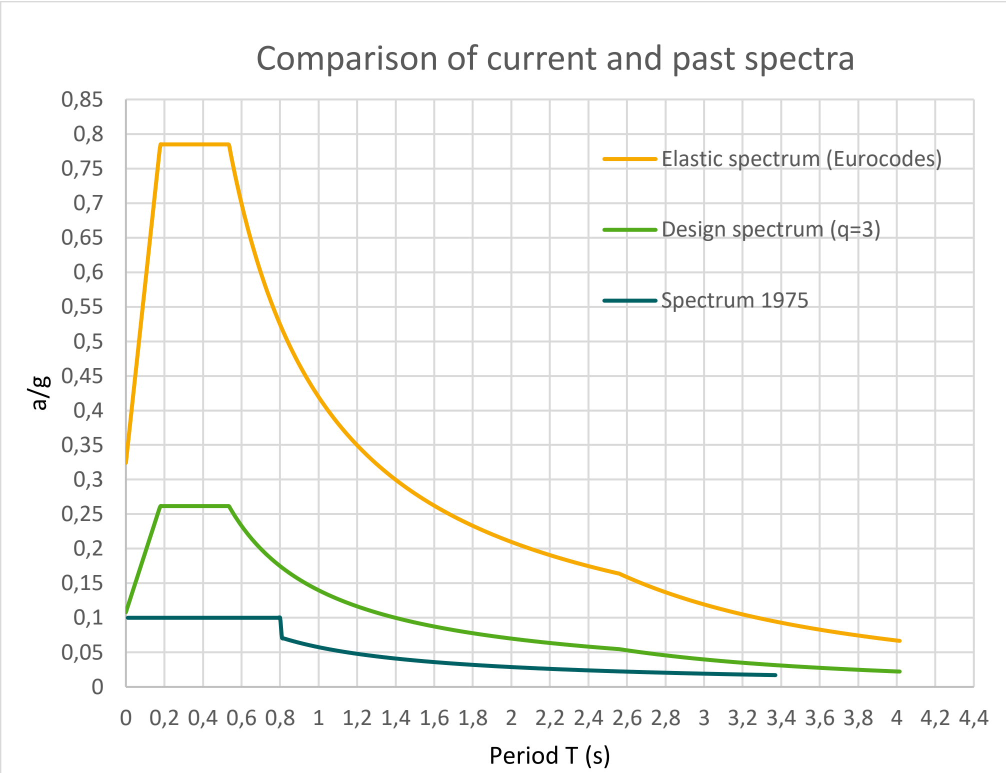


Figure 20 Representation of the spectrum according to the law from 1975, compared to the elastic spectrum from Eurocode and a plausible design spectrum with a behavior factor q=3

It is easy to notice how the codes evolved in the last 40 years: the elastic spectrum from the current Eurocode presents a maximum acceleration that is almost 8 times higher than the one provided in 1975. Even accounting for a rather high behavior factor (q) equal to 3, which decreases the accelerations applied by the elastic spectrum, the design spectrum is still much greater than the curve from the past standard.

A precise definition of the spectra with all the related parameters from the Eurocodes is going to be provided in next paragraphs, while the calculation of the past spectrum will be shown in detail in the next chapter.

As a consequence, even applying seismic actions in the dimensioning of the existing structure located in Zambrone, the necessity of seismic retrofitting is widely foreseen.

The mentioned seismic actions, to be applied in the Italian case, are provided by a modal response spectrum analysis that was performed with SAP2000 (after defining the vibration modes of the structure using the same software), applying the response spectrum curve obtained from the mid-70s code.

The load combination used to account for seismic actions is given by the Eurocodes [62] and it is the following:

$$F_{Ed} = \sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } A_{Ed} \text{ "+" } \sum_{i\geq 1} \Psi_{2,i} Q_{k,i} \ (\text{with } j \geq 1;\ i \geq 1) \qquad (3)$$

Where:

P is the prestressing action (not present in the case study);

A_{Ed} is the accidental action (seismic action in this case, which will account for the two main horizontal directions in which the seismic load could act);

$G_{k,j}$ are the permanent actions (structural and carried weight loads);

$Q_{k,j}$ are the variable loads, associated to the partial factor $\Psi_{2,I}$

It is important to notice that in this case the permanent actions are referring to the inertial mass of the building, which will be presented in the disclosure about modal response spectrum analysis, provided in detail in the next paragraphs.

Finally, this assumption for the type of seismic analysis implies setting the **construction year**, for instance, as **1978** (for both the German and Italian structure), with a consequential building life of 45 years up to nowadays. Such an age is reasonable in terms of possible inspections for ensuring the safety and proper health of the structure, also querying an eventual need for seismic retrofit.

3.2.3 Materials adopted for the existing structure

Being the case study ideally built in the end of 1970s, research on the most common materials used in that period was carried out. Starting from the Italian context, according to previous studies on the materials used for reinforced concrete over the last century, the steel adopted for the rebars was changing rapidly in the second half of the 20th century. In the period to which this study refers, one of the most used types was FeB32. [66]

For what concerns concrete, the Italian standards at the time involved a minimum cubic resistance of 15 MPa, while its maximum value was 30 MPa [67]. This means that the characterization of the material in the best case was equivalent to the current concrete resistance class C25/30. Looking at experimental results on various buildings in the center of Italy, the actual resistance reached is often slightly below the maximum value according to the code [67]. Taking for comparison the British standards from the same period, the minimum required compression resistance of concrete was 20 MPa [68].

The same properties were adopted for both the locations, also accounting for the fact that reinforced concrete's technology evolved more or less simultaneously all-around Europe.

Following these considerations and taking into consideration that the in the study also the mean mechanical properties of the material are going to be used, here are the properties of the materials that were chosen for the modeling of the existing building:

Steel FeB32	**Concrete C25/30**
Elastic modulus: E = 200 GPa Density: ρ = 7700 kg/m^3 Characteristic yield stress: f_y = 320 MPa Characteristic tensile stress: f_u = 500 MPa Average yield stress: $f_{y,m}$ = 430 MPa Average tensile stress: $f_{u,m}$ = 645 MPa	Elastic modulus: E = 31 GPa Density: ρ = 2500 kg/m^3 Characteristic concrete cylinder strength: f_{ck} = 25 MPa Poisson's ration ν = 0,2 Average concrete cylinder strength: f_{cm} = 33 MPa (taken from table 3.1 of Eurocode 2) [69] Shear modulus: G = 12,92 GPa

Figure 21 Main properties of the materials adopted in the existing building

3.2.4 Structural verifications and dimensioning of reinforced concrete elements

For the dimensioning of the existing structures a first hypothesis for the dimensions of the elements was made for the two locations.

In the **German case study**, the cross sections of the elements were taken as:

- **Columns: 40x40 cm²**;
- **Beams: 50x30 cm²**, where 50 cm is the depth of the beam, while 30 cm is the width.

For the **Italian location** the cross-section of the beam was maintained the same as in the German one, but the columns' dimensions were increased, considering the presence of a seismic load in the dimensioning procedure:

- **Columns: 50x50 cm²**;
- **Beams: 50x30 cm²**.

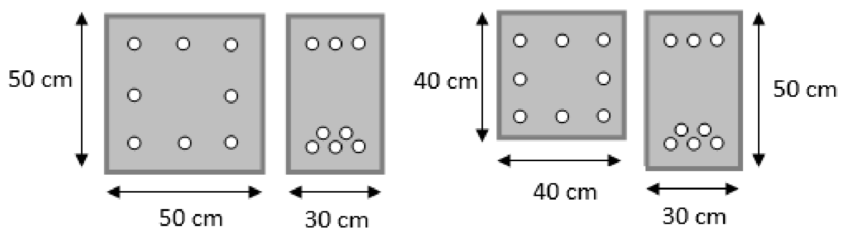


Figure 22 First hypothesis cross-sections adopted for the dimensioning of the structure in location 1 and 2 respectively on the left and the right, with a possible configuration of longitudinal reinforcement

A relevant observation regarding the complete dimensioning of the existing structure is that the dimensions of the elements were kept **constant all over the height of the building**. This assumption was made considering that reducing the cross-section of the structural members could be coupled with a higher executive, and consequently economical, effort. Moreover, accounting for the construction age assumed, the design of elements whose dimensions varies depending on the actual solicitations could have involved a huge computational effort, especially compared to the use of calculation tools that are available nowadays. Hence, taking the most critical elements as a reference and use those cross-sections for the whole building seems to be realistic.

As previously mentioned, the structural verifications were carried out by means of the software SAP2000, enabling to define the amount of longitudinal and transversal reinforcement used in the structural members.

Concerning the steel reinforcement, the design possibilities differ based on the type of element that is evaluated:

- For **columns** SAP2000 enables to use two different configurations: “reinforcement to be designed” and “reinforcement to be checked”. The former gives results in terms of required area of longitudinal and transversal reinforcements. Then, fixing in the section properties the amount of rebars that are necessary, the following results can be obtained through the software, with the relative checks:
  - Slenderness check
  - Axial force and biaxial moment check for N_{Ed}, M_{Ed2}, M_{Ed3}
  - **Demand/Capacity (D/C) PPM ratio**
  - Shear design for V_2, V_3
  - Axial compression ratio
  - Joint shear design
  - (1,3) Beam/Column capacity ratios

- For **beams** SAP2000 does not allow to fix a quantitative of reinforcement to be checked, but in every analysis it gives as an output the amount of longitudinal reinforcement (at top and bottom of the section) and the amount of shear reinforcement. These evaluations are derived from the following analysis’ outputs:
  - Design moments M_3, with the corresponding flexural reinforcement for moment M_3
  - Shear reinforcement for Shear V_2
  - Torsional reinforcement for Torsion T

  Not being able to set a fixed amount of reinforcement in beams was a critical point in the non-linear analysis procedure that will be discussed later, since it added an uncertainty in the development of automatic plastic hinges in the nonlinear analysis. But apart from the definition of plastic hinges, this meant that no checks on the designed beams can effectively be performed directly in the model. Therefore, the verification carried out autonomously was based on the ratio between the amount of steel reinforcements necessary according to the analysis with current loads against the reinforcements fixed during the dimensioning of the structure fictitiously performed in the year of construction 1978.

$$check = \frac{Area\ of\ reinforcement\ required}{Area\ of\ reinforcement\ defined\ in\ the\ element} = \frac{A_{s,required}}{A_{s,element}} \tag{4}$$

If this ratio exceeded 1, then the design was not safe enough against bending moment (longitudinal reinforcement) or shear (transversal reinforcement).

For additional details on the computation of reinforcements made by the software, reference can be made to the user manuals of SAP2000 for concrete frame design according to Eurocode 2 [70].

Then, the composition of the elements' cross-sections was determined, in terms of materials, geometry, and amount of reinforcement, and it could be used to assess whether the structure needs to a seismic retrofit considering the currents codes.

3.3 Structural verification against current seismic loads

This paragraph aims to illustrate how the analyses will account for seismic loads in the evaluation of the structural behavior of the existing building, following the guidelines provided by the Eurocodes. Such study is performed to assess the potential necessity of seismic retrofit.

3.3.1 Introduction to seismic design

The first part of Eurocode 8 [61] introduces two requirements that must be satisfied when designing a new structure considering seismic loads:

- No-collapse requirement: the structure shall be able to withstand design seismic actions with an established probability of exceedance P_{NCR} in 50 years or with reference return period T_{NCR}. Their recommended values are respectively 10% and 475 years.
- Damage limitation requirement: for actions coupled to a recommended value of P_{DLR} = 10% in 10 years, or a return period T_{DLR} = 95 years, the structure shall not encounter damages and associated limitations of use.

An importance factor should be considered in the no-collapse requirement to enlarge the considered seismic actions. It depends on the relative importance class of the building, based the function of the considered structure.
In the case study the class II can be chosen, with a corresponding importance factor $\gamma_1 = 1$.
In order to satisfy the standard's requirements, checks at the Ultimate Limit States (ULS) and Damage Limitation States (DLS) should be performed.

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Figure 23 Table 4.3 from Eurocode 8 [61], defining the importance classes

Regarding the **assessment and retrofitting of existing buildings**, slightly different requirements are reported in the part 3 of the same Eurocode [71]. In fact, there are three Limit States (LS):

- **Near Collapse (NC)**: heavily damaged structure, with low residual vertical strength and stiffness. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake. The recommended probability of exceedance in 50 years for this LS is 2%, which corresponds to a return period of **2475 years**.
- **Significant Damage (DG)**: the structure has some residual lateral strength and stiffness, even though it is significantly damaged, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged and moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity and it is likely to be uneconomic to repair. The return period is the same used in the no-collapse requirement for new designs: **475 years**, associated with 10% probability of exceedance in 50 years.
- **Damage limitation (DL)**: the structure is only lightly damaged. Non-structural components may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible and no repair measures are needed for the structure. The corresponding return period is **225 years**, considering a 20% probability of exceedance in 50 years.

Limit state	Return period [years]	Probability of exceedance in 50 years [%]
Damage Limitation (DL)	225	20
Significant Damage (SD)	475	10
Near Collapse (NC)	2475	2

Figure 24 Table summarizing the limit states for the analysis of existing structures

But the most meaningful difference between the new structures approach and the one for the existing buildings, is that in the first case the material properties are considered as the design values obtained starting from the characteristic ones, while in the latter case they are identified as average properties. The values of the mean properties should also be decreased by a confidence factor (CF) depending on the knowledge level of the building. This level depends on the on-site inspections' tests carried out and the documents that provide information about the structure. Since the fictitious structures are dimensioned during the process, the maximum knowledge level was adopted, to which corresponds a confidence factor equal to 1.

Therefore, the evaluation of the existing building will be carried out using mean properties of the materials.
Successively, the methodology indicated by the Eurocode for the **study of new elements** added into an existing structure will be disregarded in the case study, avoiding the consideration of characteristic properties of the materials (same approach to be applied to the design of new structures) and **still using average values**, also for the new components. The reason of this choice is to enable a comparison between the two retrofitting technologies (based on concrete and timber respectively), since the light timber frame shear walls were studied in RWTH Aachen laboratories only with a restricted number of tests. Hence, it was not possible to gather results about characteristic properties, but only on average values.

In any case, the objective of seismic design is to "ensure an overall dissipative and ductile behavior" [61], which is why it is important to avoid brittle failures of premature unstable mechanisms. By consequence the **capacity design** is exploited to obtain the hierarchy of resistance of different structural components.

Another observation related to seismic design principles regards the computation of the masses that must be involved in the analysis. Being earthquakes dynamic phenomena, it is necessary to consider the inertial mass of the structure, evaluated through the gravity loads applied to the structure:

$$m = \sum G_{k,j} \text{"+"} \sum \Psi_{E,i} Q_{k,i} \qquad (5)$$

Where $\Psi_{E,i}$ is the combination coefficient that takes into account the likelihood of variable loads not being present over the entire structure during the earthquake. It also accounts for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

$$\Psi_{E,i} = \varphi \Psi_{2,i} \qquad (6)$$

With φ taken from a table depending on the storey and the type of variable action considered. In the case of storeys with correlated occupancies, as in the case study, $\varphi = 0{,}8$.

3.3.2 Seismic analysis methods

To assess the performance of a building at different limit states, it is then necessary to perform a seismic analysis, which can be of different types.

The classification of such analysis depends on two factors: whether the loads applied are static or dynamic, and the linearity or nonlinearity materials' behavior (elastic or plastic constitutive laws):

	Static analysis	**Dynamic analysis**
Linear analysis	Equivalent lateral force (simplified response spectrum analysis)	Modal response spectrum analysis
Nonlinear analysis	Pushover analysis	Time-history analysis

Figure 25 Classification of different seismic analysis methods

The response of the case study against the current standard's seismic loads will be analyzed through the use of modal response spectrum analysis first, for what concerns the structural verifications to assess the possible need for seismic retrofit. Then, the pushover analysis will be performed for comparing the results of the retrofit interventions, also with the initial configuration of the existing building. Afterwards, the results on the existing structure with both the seismic analysis methods will be considered together, enabling a comparison between linear and nonlinear methodologies.

It is important to highlight that the verifications performed with the modal response spectrum analysis on the case study will only cover the LS of significant damage requirements, considering seismic loads related to a return period of 475 years. The same is valid for the assessing of the retrofits with the pushover analysis.

According to the part 1 of Eurocode 8, the design might satisfy some conditions for which the analysis method that is required to be used could involve a more or less simplified approach. The same principles should govern the design of buildings in seismic conditions:

- Structural simplicity
- Uniformity, symmetry and redundancy
- Bi-directional resistance and stiffness
- Torsional resistance and stiffness
- Diaphragmatic behavior at story level
- Adequate foundation

In the case study analyzed, all of these properties are present or assumed to be satisfied (like in the case of diaphragmatic behavior at story level that is specifically adopted in the model, or for the adequate foundation, which are assumed to be proper). Therefore, the case study could be also approached with the simplified method of equivalent lateral force.

However, given the simplicity of characterizing the modes of vibration in the 3D model adopted, the structure will be studied through a modal response spectrum analysis. To adopt such method, it is necessary to define the response spectra involved.

3.3.3 Elastic and design response spectra

An elastic response spectrum represents the expected values of acceleration that could be applied on a structure in case of earthquake with a defined **return period**, in a selected location. Apart from the already mentioned return period that is due to the limit state of interest, and the **geographical position** which influences the reference peak ground acceleration, a spectrum depends on many variables:

- **Ground type**: depending on where the structure is located, it could come in contact with different types of ground. The ground types are differenced by the value of the average shear wave velocity, which is important because it represents the velocity with which an earthquake could approach a building. The higher the velocity, the higher the loads applied on the structure. Generally, Eurocode 8 refers to seven different classes, but the National Annexes could apply more specific values or classifications depending on the geographical context in which they operate (as it happens for instance in the case of Germany). The ground type of a certain position can be usually assessed through specific maps.
- **Peak ground acceleration a_g**: it depends on the reference peak ground acceleration a_{gR}, which is the value of acceleration of a certain an earthquake expected to happen in a location with a defined return period (for instance the 475 years normally assigned to the Significant Damage requirement), considering the ground type A (highest shear wave velocity). The importance factor related to the investigated structure γ_1 is also used to compute the PGA:

$$a_g = \gamma_1 a_{gR} \qquad (7)$$

- **Damping correction factor η**: factor that accounts for the damping properties of the structure. It is given by the formula:

$$\eta = \sqrt{10/(5+\xi)} \geq 0{,}55 \qquad (8)$$

  Where ξ = viscous damping ratio of the structure expressed as a percentage, usually 5%.
- **Soil factor S and periods T_b, T_c, T_d:** values used in the formulations of the elastic response spectrum, together with the PGA and the damping correction factor. They are suggested by the Eurocodes in tables, depending on the ground type and the recommended type of response spectra (type 1 for countries with higher seismicity, type 2 for other countries). In any case, national annexes can make different specifications about them.

With all these parameters it is possible to define the horizontal elastic response spectrum $S_e(T)$ (same for both horizontal directions) with the following equations, as reported in the Eurocode:

$$0 \le T \le T_B \ : \quad S_e(T) = a_g S[1 + T/T_B(2{,}5\eta - 1)] \qquad (9)$$

$$T_B \le T \le T_C \ : \quad S_e(T) = a_g S\eta 2{,}5 \qquad (10)$$

$$T_C \le T \le T_D \ : \quad S_e(T) = a_g S\eta 2{,}5[T_C/T] \qquad (11)$$

$$T_D \le T \ : \quad S_e(T) = a_g S\eta 2{,}5[T_C T_D/T^2] \qquad (12)$$

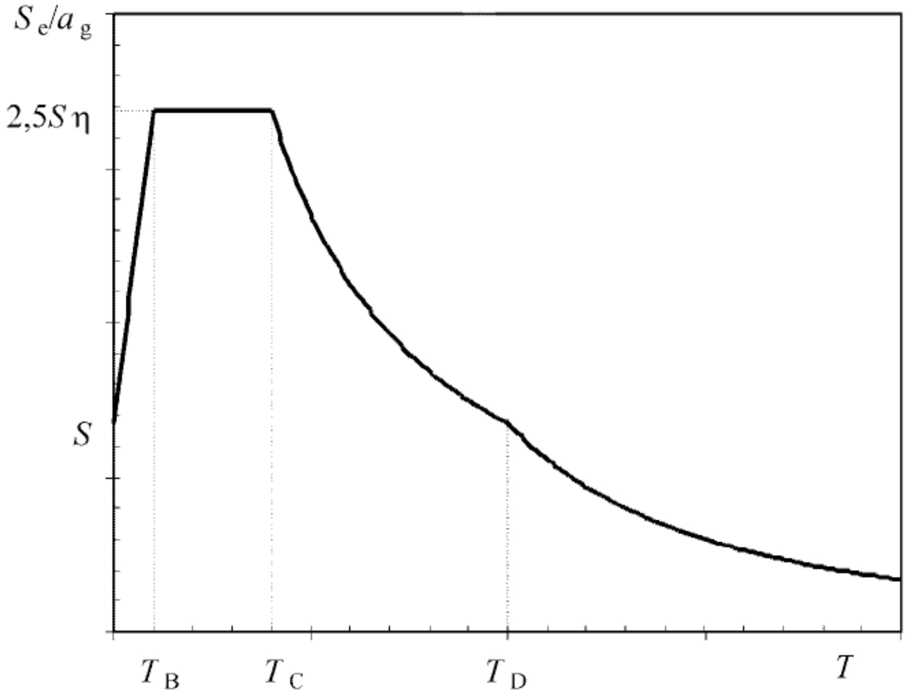


Figure 26 Typical shape of the elastic response spectrum

The actions due to the vertical elastic response spectrum are going to be neglected. Indeed, even if the vertical acceleration is likely to be higher than 0,25g, the case study does not involve any element that needs verifications involving also the vertical seismic actions according to Eurocode 8 (horizontal or nearly horizontal: structural members spanning 20 m or more, cantilever components longer than 5 m, pre-stressed components; beams supporting columns; base-isolated systems). So, **vertical seismic action** can be **neglected**.

Then, since this method acts only in the linear range of the material properties, it is taken into account the further resistance due to plasticization of structural elements by means of an adjustment of the elastic spectrum.

In fact, the design involves an elastic analysis based on a “**design spectrum**”, which is obtained reducing the elastic spectrum by means of the behavior factor q.

The **behavior factor** approximates the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. It also accounts for the influence of the viscous damping being different from 5%. So, q depends on the materials and structural systems involved in the project.
The Eurocode itself provides some values of q, but the final choice is actually taken by the designer, considering the expected ductility of the system. Such decision can be confirmed or supported by some computations done after a pushover analysis of the same structure.

Therefore, here are the formulas that define the design response spectrum:

$$0 \leq T \leq T_B \; : \; S_d(T) = a_g S[2/3 + T/T_B(2,5/q - 2/3)] \qquad (13)$$

$$T_B \leq T \leq T_C \; : \; S_d(T) = a_g S 2,5/q \qquad (14)$$

$$T_C \leq T \leq T_D \; : \; S_d(T) = a_g S 2,5/q[T_C/T] \qquad (15)$$

$$T_D \leq T \; : \; S_d(T) = a_g S \eta 2,5/q[T_C T_D/T^2] \qquad (16)$$

In for the formulas *(15)* and *(16)* the acceleration value must be $S_d(T) \geq \beta a_g$
where β is the lower bound factor for the horizontal design spectrum, recommended as 0,2 in Eurocode. It might be found in National Annexes.

3.3.4 Modal response spectrum analysis

This method considers the response of all modes of vibration contributing significantly to the global response of the building [61]. With “contributing significantly”, reference is made to those modes of vibration that have at least an effective modal mass greater than 5% of the total mass and whose sum of effective total masses is at least 90% of the structure’s total mass.
Such vibration modes represent the response of the structure to horizontal solicitations, described in terms of modal coordinates, and each of them is characterized by a certain natural frequency (and consequently by a natural period). The important particularity of the vibration modes is that they can be used to represent every possible dynamic of the system idealized through a finite number of degrees of freedom. In order to know the vibration modes, a modal analysis can be performed on the 3D model created in SAP2000.

Then, if those relevant modes can be regarded as independent from each other, which means that two vibration modes have periods that satisfy the condition $T_j \leq 0,9T_i$, the maximum value of seismic action effect may be obtained according to the “square root of the sum of the squares” (SRSS) rule:

$$E_E = \sqrt{\sum E_{Ei}^2} \qquad (17)$$

With E_{Ei} = value of the seismic action effect due to the vibration mode i.

If the vibrations mode are not independent, other procedures like the "Complete Quadratic Combination" shall be adopted. In the case study, the CQC modal combination is going to be adopted, since it is more precise and the modes of vibration could happen to not be independent from one another.

Thereby, the modal response spectrum analysis combines the information about the vibration modes with the design response spectrum, obtaining the results of acting stresses in the structure under seismic loads in terms of internal stresses and deformations. This enables the designer to investigate the various checks that are necessary.

For what concerns the consideration of both the acting directions of seismic load simultaneously, the seismic forces obtained through the application of the horizontal response spectrum respectively in the longitudinal and transversal direction, E_{Edx} and E_{Edy}, can be combined with the so-called "100+30 method":

$$A_{Ed} = E_{Edx} \text{ "+" } 0{,}3E_{Edy} \qquad (18)$$

$$A_{Ed} = 0{,}3E_{Edx} \text{ "+" } E_{Edy} \qquad (19)$$

In this way it is possible to obtain the accidental actions A_{Ed} to be used in the load combinations for both dissipative and non-dissipative members.
An eccentricity ratio for the loads equal to 0,05 is taken into account in the application of the modal response spectrum analysis.

Here is the load combination that needs to be applied for checking dissipative members of the structure (beams and columns at the base), which is the same illustrated for the consideration of seismic loads in the dimensioning of the existing structure:

$$F_{Ed} = \sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } A_{Ed} \text{ "+" } \sum_{i\geq 1} \Psi_{2,i} Q_{k,i} \ \text{(with } j \geq 1;\ i \geq 1) \qquad (20)$$

Where the partial factors $\Psi_{2,i}$ in the case study are taken from the annex A of Eurocode 0:

Action	$\boldsymbol{\Psi_0}$	$\boldsymbol{\Psi_1}$	$\boldsymbol{\Psi_2}$
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category H: roofs	0	0	0
Snow loads on buildings, for sites at altitude H≤1000 m a.s.l.	0,5	0,2	0
Wind loads on buildings	0,6	0,2	0

Figure 27 Partial factors for the variable loads involved in the case study [62]

Instead, for the check at Ultimate Limit States for non-dissipative members, the reported load combination requires some adjustments, considering that the analysis performed involves the behavior factor q.
In fact, reducing the elastic response spectrum by means of q, to obtain the design response spectrum, means that the dissipative components of the structure are exploited with their plastic behavior.
That is the reason why the forces applied on dissipative components can be considered lower, checking only the elastic response of the materials.

On the counterpart, not all the building's structural element are designed to be dissipative. Indeed, according to the **hierarchy of resistances**, it is preferable to have a development of plastic hinges in the beams and at the very base of the lowest floor columns. So, all the other elements are going to be considered as non-dissipative, and tested with a different load combination:

$$F_{Ed} = \sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \Omega_T A_{Ed} \text{ "+" } \sum_{i\geq 1} \Psi_{2,i} Q_{k,i} \text{ (with } j \geq 1;\ i \geq 1) \qquad (21)$$

In which the new coefficient $\boldsymbol{\Omega_T}$ is the **overstrength factor** corresponding to the adopted structural system. Such factor is higher than one, hence the actions that must be resisted by the non-dissipative members are higher than in the dissipative case.

As a matter of fact, the application of this load combination was not necessary in the case study, since some elements in the structures were not verified even with load combination used for the dissipative members. Consequently, the need for a retrofitting intervention was already assessed and no analysis exploiting the non-dissipative load combination needed to be implemented.

At that point, study went on with the design of the structure's strengthening.

3.4 Definition and design of retrofit interventions

The retrofit of the existing structure was studied in two possible configurations, exploiting different construction materials but implementing a similar technical solution: the introduction of shear walls into the moment resisting frame.

After setting the characteristics of the shear walls composed by concrete and gathering the information about the light timber frame shear walls that were tested in RWTH laboratories, it was possible to make a first assumption about the position and the amount of elements needed to get a good structural behavior of the retrofitted building, even against the current seismic loads.

3.4.1 Materials properties for retrofit

The two construction materials adopted for the construction of shear walls in the existing frame building are reinforced concrete and timber.

The former has better characteristics compared to the reinforced concrete of the existing structure, in particular its components are:

Steel reinforcement B500C	**Concrete class C30/37**
Elastic modulus: E = 196,6 GPa Density: ρ = 7850 kg/m^3 Characteristic yield stress: $f_{y,k}$ = 500 MPa Characteristic tensile stress: $f_{u,k}$ ≥ 540 MPa Average yield stress: $f_{y,m}$ = 554 MPa Average tensile stress: $f_{u,m}$ = 645 MPa	Elastic modulus: E = 33 GPa Density: ρ = 2500 kg/m^3 Characteristic concrete cylinder strength: f_{ck} = 30 MPa Poisson's ration ν = 0,2 Average concrete cylinder strength: f_{cm} = 38 MPa (taken from table 3.1 of Eurocode 2) [69] Shear modulus G = 13,75 GPa

For what concerns the light timber frame walls, the information about the selected technology was collected from previous research on this kind of shear walls, carried out at RWTH Aachen. The configuration of the walls and the material properties are the one recreated in laboratory tests, whose results also include the force-displacement curve applied in the numerical model.

The characteristics of timber shear wall wooden materials are:

Softwood C24 [72]	**OSB (Oriented Strand Board) panels** [73]
Elastic modulus mean: E = 11 GPa Average density: ρ =420 kg/m^3 Characteristic bending // to grain: 24 MPa Characteristic tension // to grain: 14 MPa Characteristic compression // to grain: 21 MPa Poisson's ratio: ν = 0,3 Shear modulus: G = 0,69 GPa	Dimensions: 1250x2500x15 mm^3 Elastic modulus mean: E = 4,9 MPa Average density: ρ ≥ 600 kg/m^3 Characteristic bending // to grain: 16,4 MPa Characteristic tension // to grain: 9,4 MPa Characteristic compression // to grain: 15,4 MPa

The tensile strength of all the staples used to fix the OSB panels is at least 900 MPa.

The properties of softwood were replicated from models previously developed at RWTH and adopted to define the wood material used for the cross-sections composing the light timber shear walls. Instead, the values for OSB were indicated with only an informative aim, since the modeling of these elements is going to be carried out through links for which the most important information is the force-displacement curve.

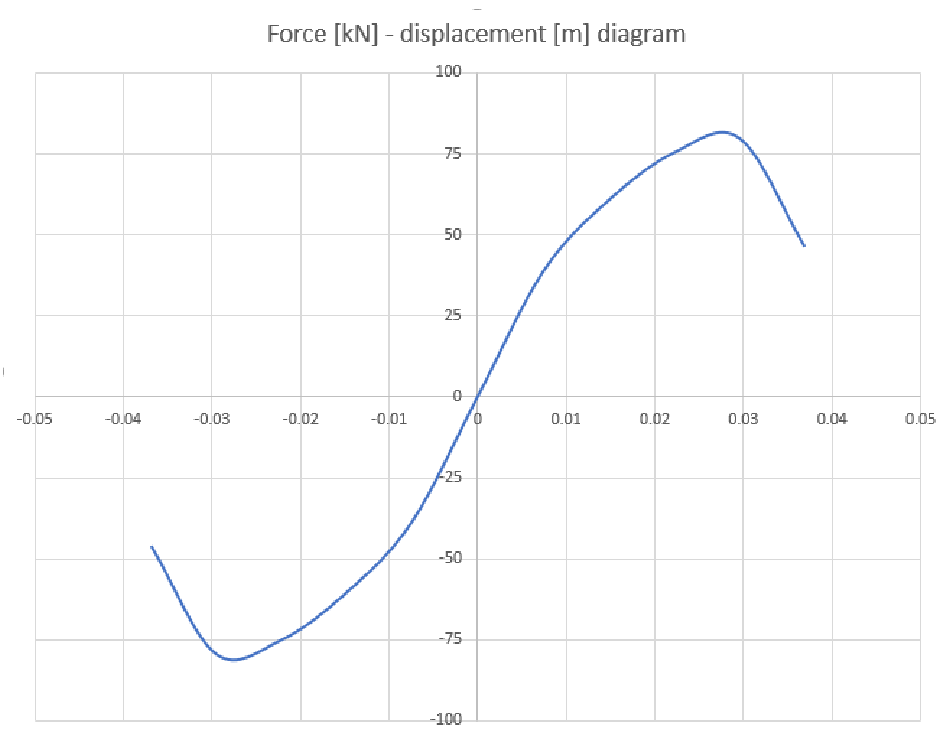


Figure 28 Force-displacement curve characterizing the nonlinear behavior of a single OSB panel against horizontal loads

3.4.2 Design procedure for reinforced concrete shear walls

First of all, the dimensions of one wall modulus were set as hypothesis to 3 of height, incorporating the existing beams, 2 m of width and 20 cm of thickness. The idea is to place these walls in the center of the beams spanning between two columns in the frame, in various locations depending on the results of the simulations, as shown for instance in *figure 20.*

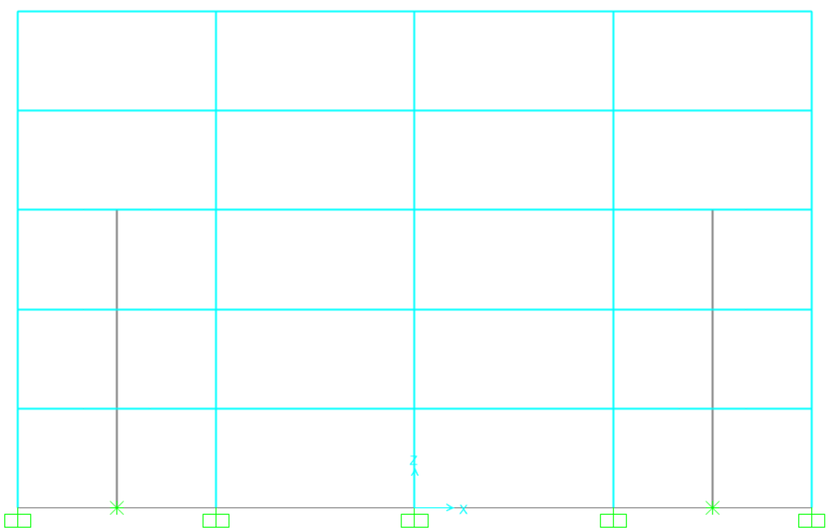


Figure 29 Example of distribution of reinforced concrete shear walls (in grey) into the existing moment resisting frame (in light blue)

The dimensioning of the walls is going to be carried out for the lowest level, in which the horizontal actions and bending moments into the elements reach their maximum. The same wall will be placed in the floors above, selecting in the design how many floors to strengthen with this solution.

Then, the first check that must be performed on the wall is related to its slenderness, as reported in Eurocode 2 [69]. It is the same type of verification adopted for isolated members such as columns.

If the slenderness of the element λ is below a fixed value λ_{lim}, the calculations can neglect the presence of second order effects. The following formula provides the limit slenderness:

$$\lambda_{lim} = 20ABC/\sqrt{n} \qquad (22)$$

In which:

$A = 1/(1 + 0{,}2\varphi_{ef})$ (used A = 0,7 if φ_{ef} = effective creep ratio is unknown);

$B = \sqrt{1 + 2\omega}$ (used B = 1,1 if ω=mechanical reinforcement ratio is unknown);

$C = 1{,}7 - r_m$ (used C = 0,7 if $r_m = M_{01}/M_{02}$ = moment ratio is unknown);

Where M_{01} and M_{02} are the first order end moments, $|M_{02}| \geq |M_{01}|$ and r_m is positive if the end moments give tension on the same side, otherwise it is negative;

$n = N_{Ed}/A_c f_{cd}$ = relative nominal force;

It is important to notice that the shear wall is assumed to resist lateral loads only along its longitudinal direction, while for out-of-plane direction it does not involve any resistance. So, the considered bending actions are in-plane.

Instead, the slenderness of the structure can be calculated as:

$$\lambda = l_0/i \qquad (23)$$

With l_0 = effective length, depending on the constraint scheme of the considered element;

i = radius of gyration of the uncracked concrete section.

Proceeding with the dimensioning, once the bending and axial force acting on the element are determined with the analysis in the numerical model, it is possible to use the design charts from the code to identify which is the required area of steel reinforcement, depending on the geometry of the wall and on the characteristics of both steel and concrete.

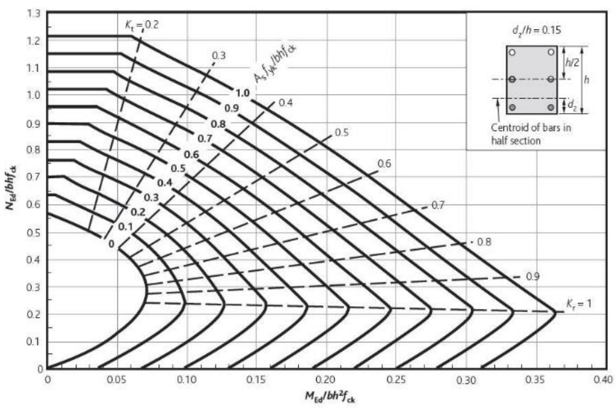


Figure 30 Example of design chart to define the necessary steel reinforcement area [69]

The next step regards the detailing of reinforcement for the shear wall element, for which some conditions must be verified.
Firstly, the vertical reinforcement must be higher than a minimum value:

$$A_s \geq 0{,}002A_c \qquad (24)$$

With A_c = area of concrete in the wall's section. Half of the vertical reinforcement A_s is placed towards each face of the wall.
The same vertical reinforcement has a maximum value too, which is $0{,}04A_c$. In addition, if $A_s \geq 0{,}02A_c$ no links are required between the steel rebars.
Also, the bars must have a diameter at least equal to 12 mm.

The horizontal reinforcement is dimensioned as the 25% of the vertical reinforcement on each face, or as $0{,}001A_c$ choosing the greatest of the two options. The diameter of the bars must be higher than the diameter of the vertical reinforcements divided by 4.
Moreover, there are some limitations about the spacing between the rebars. The spacing for vertical bars must be lower than the minimum between 400 mm and 3 times the wall thickness. At the same time, it must be higher than 100 mm. For the horizontal reinforcement, the maximum value is 2 times the wall thickness.
In the end, the nominal cover must be higher than 20 mm or the bar diameter size.

All the reported conditions should be enough to exhaustively define the reinforced concrete shear walls, after having set the external geometry of the elements and the materials' properties.

3.4.3 Modeling of reinforced concrete shear walls

The walls defined with the shown procedure need to be inserted in the numerical model for the analysis of the retrofitted structure behavior. This operation could be done in two different ways: by inserting some mesh elements representing the wall in the model or by means of the equivalent frame method.

Since the use of mesh requires a very high computational effort, it often preferred in practice to use the **equivalent frame method**, or other similar modeling techniques. It is "one of the most common planar shear models" and it replaces the wall with "an idealized frame structure consisting of a column and rigid beams at the floor levels" [74]. To do so, the wall is represented with a column that has the same geometry of the wall, and it is constituted by the same materials used effectively for the wall. Such element is superimposed in the model to the existing beams, which are modified in the portion that is occupied by the wall in order to attribute to the walls the stiffness that characterizes them in the reality. Indeed, to those beams (commonly referred to as "dummy beams") were applied some property factors to increase their stiffness against bending in both directions and also to compute a larger cross-sectional area.

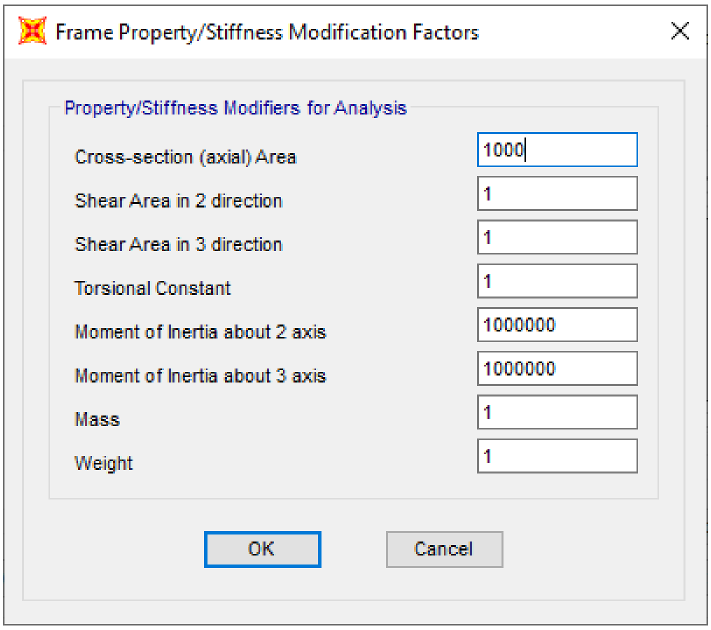
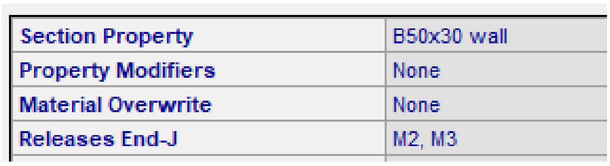


Figure 31 Property modification factors assigned to the frame sections used for the "dummy beams" of the shear walls

Then, hinges were placed at the ends of the **rigid beams** facing the actual existing beams.

In such a way, the bending behavior of the extremely stiff walls are decoupled from the existing beams in contact with them. Otherwise, the original beams would encounter extremely high bending moments due to the continuity with the walls, which would get most of the internal forces since they have a much higher stiffness compared to the other components of the frame.



Section Property	B50x30 wall
Property Modifiers	None
Material Overwrite	None
Releases End-J	M2, M3

Figure 32 Example of rigid beam properties with the assignation of end releases adjacent to the existing beam.

It is necessary to notice that in each wall two different dummy beams are developing from the center axis of the wall itself, which is the reason why in each beam the release of moments was assigned only to one end.

The last property attributed to the modeled reinforced concrete shear walls is the absence of out-of-plane resistance to lateral loads. It was simulated through the release of their base constraint against out-of-plane bending moments, maintaining a fixed constraint in the longitudinal direction of the wall, but obtaining hinges for their transversal relative direction. Consequently, no **out-of-plane bending moments** are induced. Instead, at the intersections between different levels, continuity of the elements is assumed with transmission of all the internal forces.

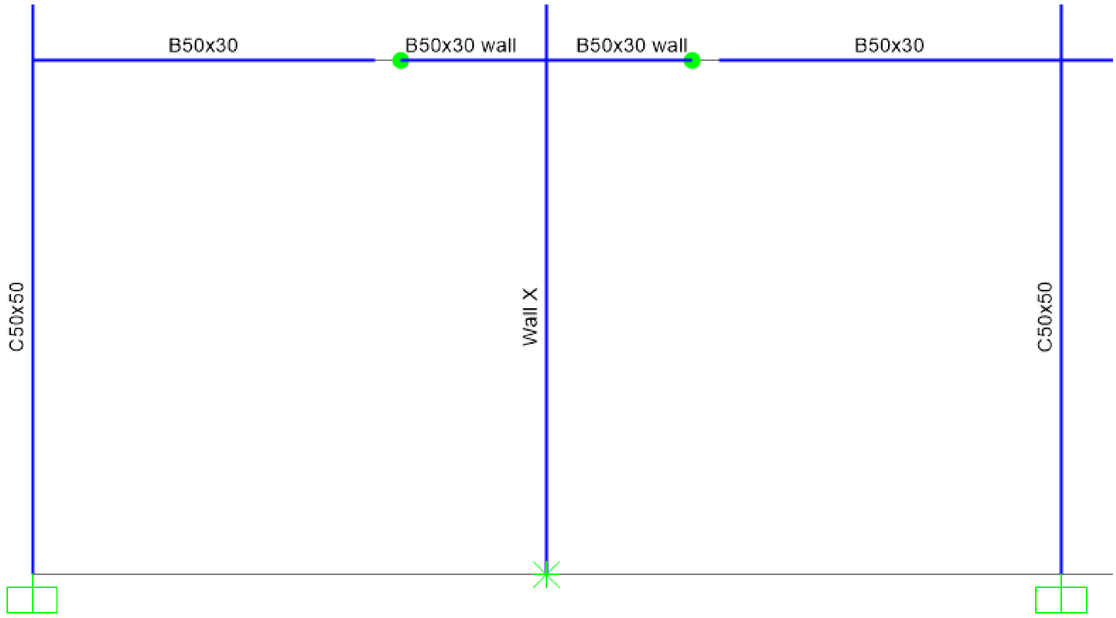


Figure 33 Example of final configuration of a wall, positioned at midspan of the existing beam, with the column "Wall X" in correspondence of the wall's center axis. The release of out-of-plane bending moment resistance is applied at the base, while the two rigid "B50x30 wall" beams show the application of end-release for bending moments (green dots) with respect to the existing beam, which is not directly stiffened by the shear wall.

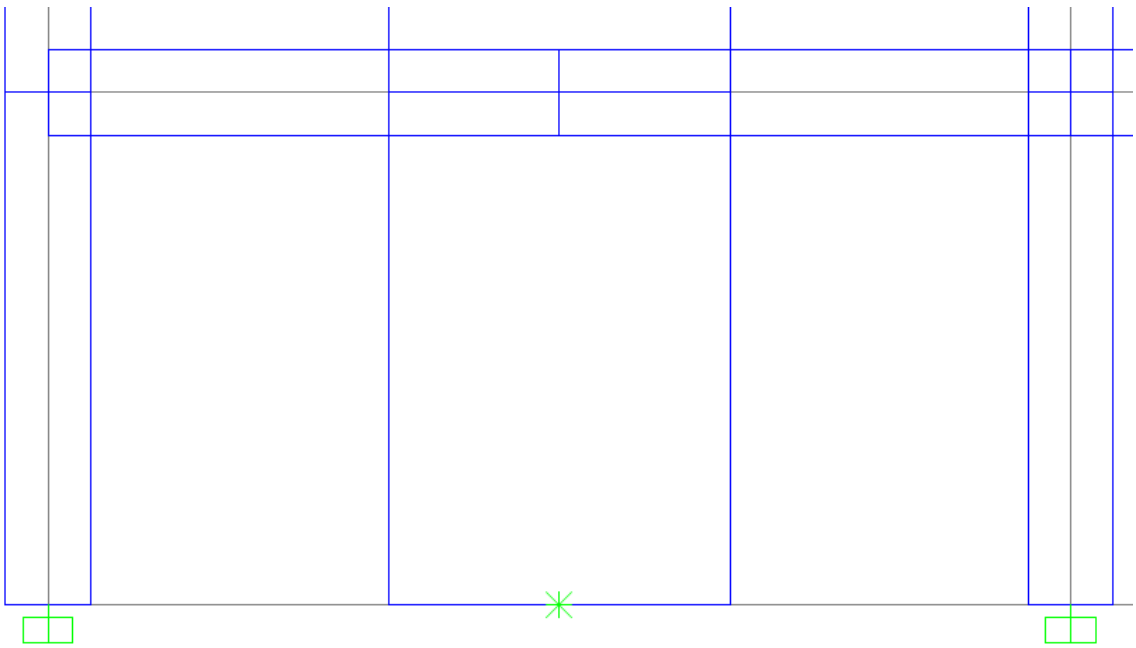


Figure 34 Visualization of the model for a reinforced concrete shear wall in extruded mode, showing the superposition of the walls with the existing beams, then modified to account for the wall's stiffness ("B50x30 wall" sections).

3.4.4 Modeling of Light Timber Frame shear walls

The light timber frame shear walls adopted in the case study were manufactured and experimentally tested in the laboratories of RWTH Aachen.

Such elements are produced exploiting different components, linked together with fasteners. The materials previously introduced, class C24 softwood and OSB panels, are respectively used for the framing structure and the sheathing that confers the resistance to horizontal actions to the shear walls.

The elements of the framing are columns of 120x200 mm^2 cross section at the sides and 60x200 mm^2 in the middle, with a height of 2320 mm, and beams long 1450 mm at the top and 1250 at the base of the wall, with a cross section of 140x200 mm^2. Each module of is composed by three columns with a net spacing equal to 475 mm and two beams (one at the base and one at the top). The elements are screwed together with partially threaded countersunk screws whose length and diameter are respectively 220 mm and 6 mm. Instead, for fixing the wall to the existing structure, tie rods are placed on the sides of the framing. For each anchor, 41 comb nails CNA 4,0x60 and an additional 4 connecting screws CSA 5,0x80 are used as fasteners.

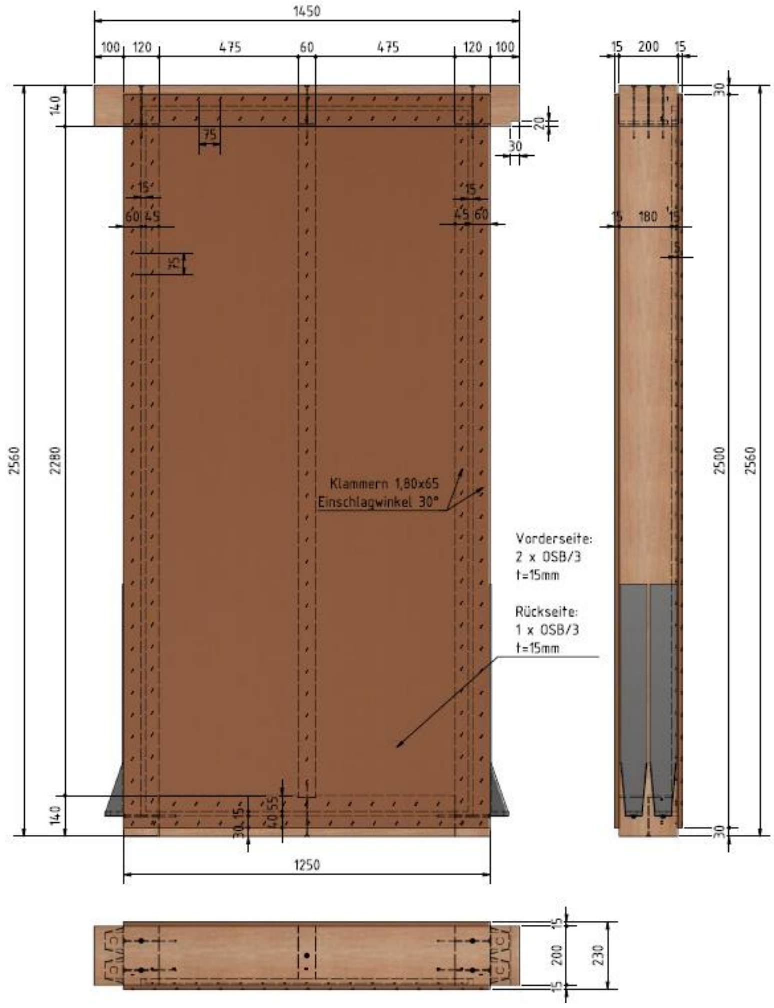


Figure 35 Light frame timber frame shear wall module, with the corresponding dimensions [75]

The timber walls adopted in the case study were named “**Power walls**” and present an innovation with respect to the usual light timber frame shear walls: instead of using a thin panel on each side of the framing, they exploit a total of **three OSB sheathings**, one on one side and two on the other. In this way, it is possible to reach higher structural performances against horizontal loads.

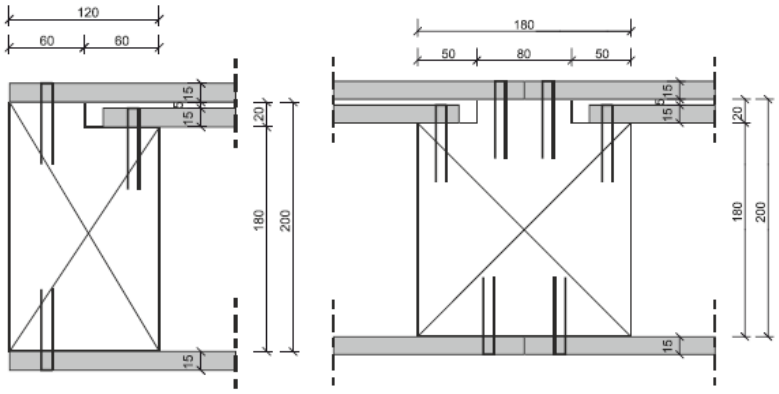


Figure 36 Link between OSB panels of the Power wall and softwood studs.
On the left: side column. On the right: central column in the case of two consecutive moduli. [75]

Moreover, more than one module can be placed to create a wall that is longer in its longitudinal direction. For example, in the case of two adjacent moduli, the number of studs is five since the last column of the first module corresponds with the first of the second module. Consequently, the dimensions of the wall’s components are slightly different.

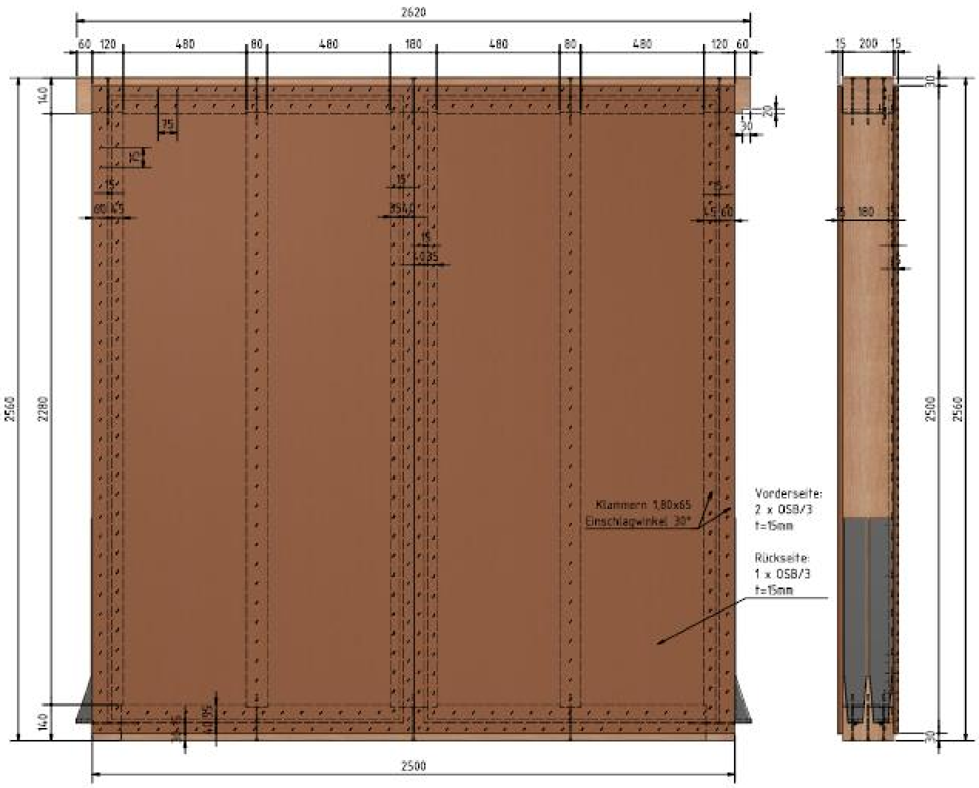


Figure 37 Representation of two consecutive moduli constituting the shear wall with its dimensions.

All the information about the walls composition will result particularly helpful for the evaluation of environmental impact.

For what concerns the **modeling of the light timber frame shear walls**, it was performed respecting the results obtained in the characterization of a single module of wall. Such values need to be intended as mean properties. In fact, the experiments were carried out on a restricted number of specimens, which consequently did not allow to define characteristic properties for the newly manufactured technology of Power walls.
In particular, here is the force-displacement curve that is valid for a module of shear wall, given by the sum of three OSB panels:

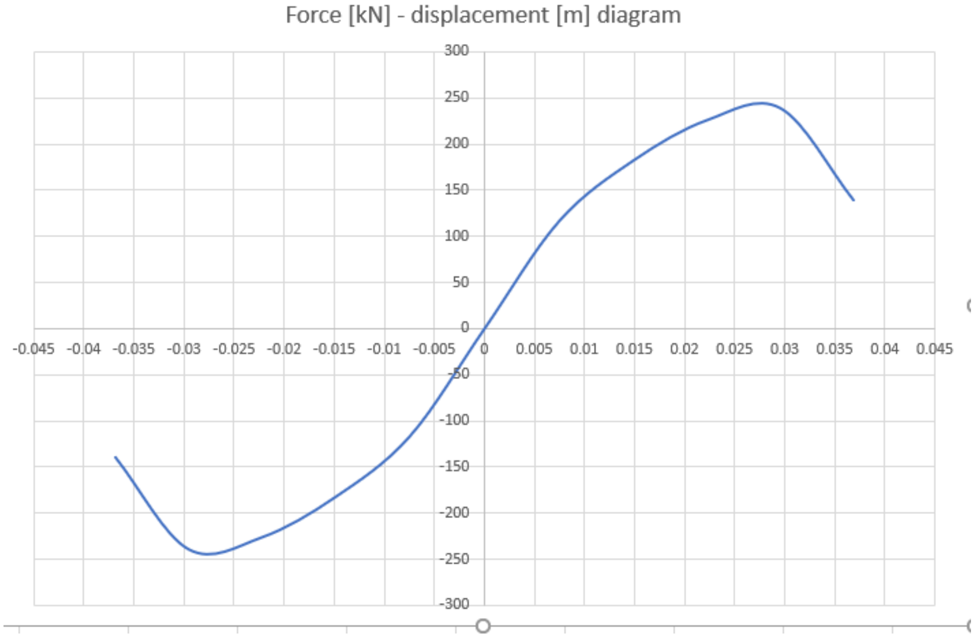


Figure 38 Force-displacement curve associated to the behavior of an entire wall (three sheathings)

This non-linear behavior is modelled in such a way to be distributed inside **two link elements**, connecting opposite corners of the shear wall. Hence, one of the two multi-linear plastic links is characterized by a force-displacement curve due to the presence of one OSB panel, while the other presents twice the force for the same displacement, being associated with two OSB panels.

A relevant aspect is the discrepancy between the height of the module and the actual net height between the floor and the intrados of the upper beam, delimiting the space that the wall needs to fill. Since the **height of the shear resisting part of the wall** (i.e. the OSB panels) is **fixed**, it will be necessary to create a connection between the beam and the shear wall, by means of some wooden elements. The mentioned connection is composed by short columns having the same section used for the wall's columns and by horizontal and diagonal elements having a unique cross section of 400x250 mm^2. These elements are considered to be fixed to the existing reinforced concrete beam, but hinged to the columns of the timber shear walls.

For simplicity's sake, all the wood studs are modeled with a 200x200 mm^2 cross section. The columns in correspondence of the OSB panel's height are considered hinged at both ends and are consequently equipped with M2 and M3 end-releases in the model. This assumption allows the link elements to work against horizontal forces without the transmission of bending moments along the height of a wall. Also at the very base of the wall, the applied constraint is an hinge in both directions.

Lastly, during the addition of timber shear walls to the existing structure, the first attempt was to introduce just one module in the selected spans. Later, it turned out to be necessary to insert more than one in each position, recreating the double-module light timber frame shear wall previously presented as a possible solution.

In the model, it was done without moving the first wall module already placed but adding another one next to it, paying attention to keep a small spacing between them, in order to avoid numerical problems that were initially faced in the analysis of this kind of element. The use of the double-module enabled to operate on a limited number of spans, even without filling the space between the columns completely, which is seen as an advantage for the possibility of positioning some openings in the non-structural spaces, like doors or windows.

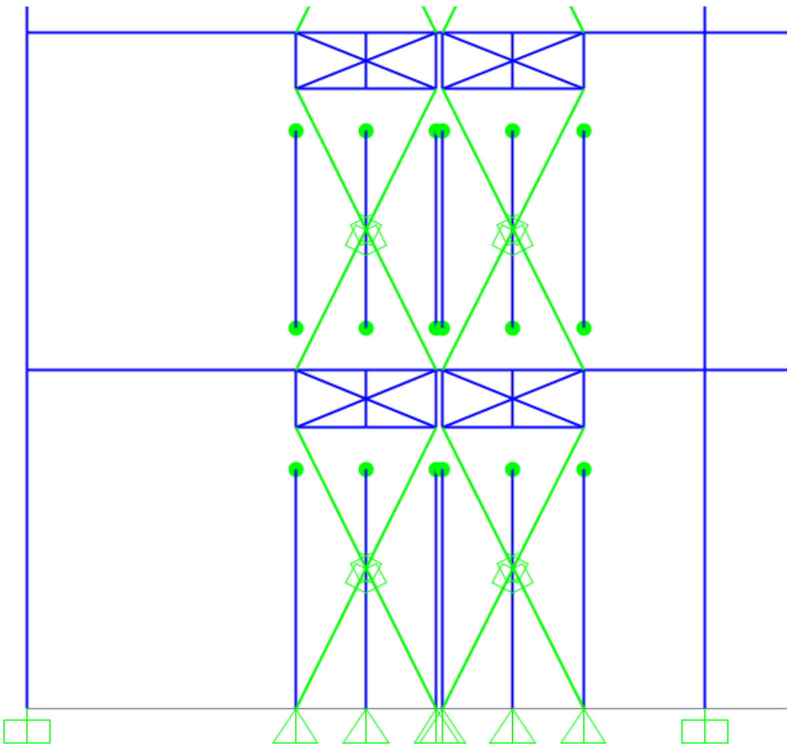


Figure 39 Representation of the modeling of a double-module light timber shear wall, in which it is possible to notice the hinges at the base of the wood columns, the end-releases at the ends of each stud, the use of a connection between the existing frame's beam and the shear wall (to fill the height gap), and the link elements connected to the corners of the shear walls.

To conclude, also in the case of light timber frame shear walls the analysis will be carried out in different configurations, with the aim of obtaining good structural results. The performances against seismic loads, with an adequate amount of timber walls moduli inserted into the existing structure, are expected to be comparable with the results obtained through the introduction of reinforced concrete shear walls.

3.4.5 Criteria for the selection of retrofit interventions

Before continuing with the evaluation of structural results, this short paragraph has the purpose of better clarifying the reasons why the described intervention typologies were selected.

Looking at the technical criteria for the selection of “type, technique, extent and urgency of the intervention” reported in part 3 of Eurocode 8 [71] the following observations are drawn:

i) Being an ideal case study, no local gross errors are considered to be present in the existing structure.

ii) The regularity of the building does not need to be improved. At the same time, the planning of the intervention needs to maintain it.

iii) For what concerns resistance, it could be improved either with new structural elements, or increasing the strength and/or stiffness of the existing components.

iv) The ductility of the building should increase both locally and globally, or at least it should not be compromised as the strength is increased.

Many possibilities were open for the choice of the intervention’s type, considering a retrofit oriented to a better seismic behavior: from the local modification of elements, to mass reduction, passing through the addition of new structural elements and many other options.

For various reasons, the addition of new structural elements was selected:

1) Choosing a type of intervention that can be applied with different materials and technologies enables a solid base for comparison of structural and environmental performances in the two inspected cases.

2) The use of timber as a construction material is often associated with better environmental performances, especially in comparison with traditional construction technologies like reinforced concrete. Such use finds its best application in the utilization of new elements rather than as a material for local retrofit, above all dealing with an existing structure in reinforced concrete.

3) Exploiting the opportunity to implement further study on recently developed and analyzed elements like timber power walls.

4) Considering that the case study was idealized and not much about the internal distributions was specified, it could be quite reasonable that the infill walls between the columns might be substituted by structural elements.

3.5 Evaluation of structural results

Once a first design of retrofit for both the technologies was carried out, it was possible to evaluate the corresponding results in terms of structural resistance to seismic actions. The assessment was carried out by means of the **nonlinear static pushover analysis**, coupled with the **N2 method** for the definition of the performance point.

3.5.1 Pushover analysis

As reported in Eurocode 8 part 3 [71], the nonlinear static (pushover) analysis is a nonlinear static analysis under constant gravity loads and monotonically increasing horizontal loads.

For buildings that satisfy the regularity criteria the analysis may be performed using two planar models, one for each main horizontal direction of the building. To overcome some issues in the computations of the pushover analysis by the software, the method was eventually applied one direction at a time, using two planar models.

In fact, the case study was eligible for the regularity criteria identified in Eurocode 8 part 1 [61], among which it is possible to find:

- Approximately symmetrical plan with respect to two orthogonal axes;
- Compact plan configuration (each floor shall be limited by a polygonal convex line;
- In-plan stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements (rigid floor constraint);
- Slenderness $\lambda = L_{max}/L_{min}$ of the building in plan shall not be higher than 4 (L_{max} and L_{min} taken in orthogonal directions)

In the analysis, the pattern of lateral loads represents the inertial forces due to earthquakes on the structure.

The shape of the load can be determined in different ways, for instance it can increase linearly with height, or it can assume the shape depending on vibration modes, as it was done in the case study.

The increase of the lateral load is performed in steps, which are delimited by the activation of plastic hinges or changes in their behavior coupled with losses of stiffness. Corresponding to each step, the force applied and the corresponding displacements are registered.

Once a certain (target) displacement specified by the designer is reached, or a global instability mechanism is manifested, the analysis ends. The data acquired during the analysis are then used to develop the capacity curve of the structure, which is represented on the force-displacement plane. Precisely, the data considered are usually concerning the displacement on the top floor of the building and the base shear force that is applied on the structure.

3.5.2 Description and application of N2 method

At this point, the N2 method developed at the University of Ljubljana by Fajfar [76] comes into action. “It combines the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system.” [76]

Starting from the elastic response spectrum previously defined, in the case of elastic SDOF systems is valid the relation from which it is possible to obtain the elastic displacement spectrum:

$$S_{de} = S_{ae}T^2/(4\pi^2) \qquad (25)$$

In such a way, instead of using the spectra in the period domain, they can be referred to the displacement domain. This is going to be useful for visualizing the results of pushover analysis and the response spectrum on the same plane.

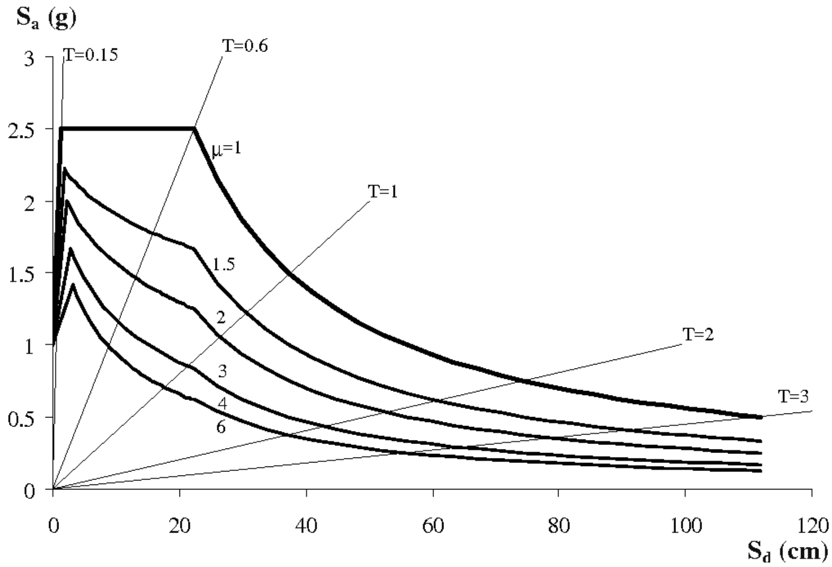


Figure 40 Example of Acceleration Displacement Response Spectrum (ADRS) with the corresponding spectra at the varying of ductility μ [76]

As mentioned, the N2 method exploits an equivalent SDOF system which will be related to the starting planar MDOF model. The equation of motion of MDOF, including only lateral translational degrees of freedom, is the following:

$$\boldsymbol{M\ddot{U}} + \boldsymbol{R} = \boldsymbol{M1}\, a \qquad (26)$$

Where M is the matrix of masses, U and R are the vectors respectively representing displacements and internal forces, 1 is a unit vector and a is the ground acceleration dependent on time. Assuming that the displacement shape ϕ is constant and normalized to its component related to the top floor, the displacement vector is equal to:

$$\boldsymbol{U} = \boldsymbol{\Phi D_t} \qquad (27)$$

With D_t = time-dependent top displacement.

From statics, the internal forces R are equal to the external loads P, which were defined in the pushover analysis as:

$$\boldsymbol{P} = p\boldsymbol{M\Phi} \qquad (28)$$

Then, multiplying from the left for $\boldsymbol{\Phi^T}$ the equation of motion becomes:

$$\boldsymbol{\Phi^T M \Phi \ddot{D}_t} + \boldsymbol{\Phi^T M \Phi p} = -\boldsymbol{\Phi^T M 1}\, a \qquad (29)$$

That is the equation of motion of the equivalent SDOF system and can be rewritten as:

$$\boldsymbol{m^* \ddot{D}^*} + \boldsymbol{F^*} = -m^* a \qquad (30)$$

In which m* is the equivalent mass of the SDOF system:

$$m^* = \boldsymbol{\Phi^T M 1} = \sum m_i \Phi_i \qquad (31)$$

and D* and F* are the displacement and force of the equivalent SDOF system:

$$D^* = D_t / \Gamma \qquad (32)$$

$$F^* = V / \Gamma \qquad (33)$$

Where $V = \sum P_i = pm^*$ is the base shear of the MDOF model and the constant Γ, usually called **modal participation factor** controls the transformation from the MDOF to the SDOF model and vice versa:

$$\Gamma = \frac{\boldsymbol{\Phi^T M 1}}{\boldsymbol{\Phi^T M \Phi}} = \frac{\boldsymbol{\Phi^T M 1}}{\sum \boldsymbol{m_i \Phi_i^2}} \qquad (34)$$

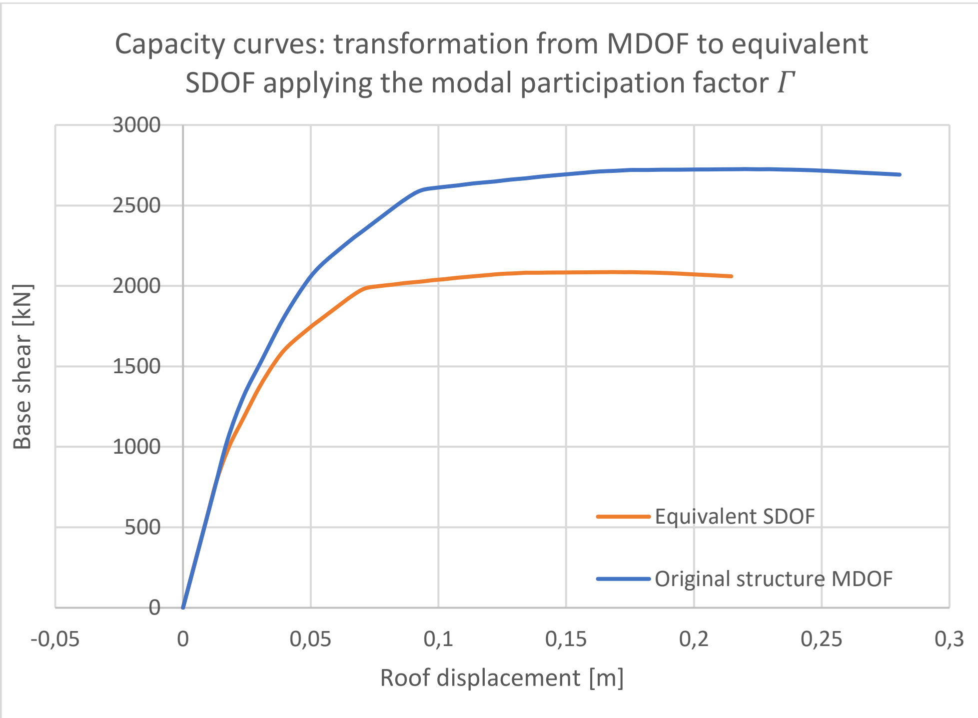


Figure 41 Example of transformation from the original MDOF representing the structure to the equivalent SDOF to be used in the N2 method, through the factor Γ

The graphical procedure of N2 method requires a post yield stiffness equal to zero since the reduction factor R_μ is defined as the ratio of the required elastic strength $S_{ae}(T^*)$ to the yield strength S_{ay}, while the influence of the moderate strain hardening is incorporated in the demand spectra.

$$R_\mu = S_{ae}(T^*)/S_{ay} \qquad (35)$$

Therefore, the capacity curve of the structure, obtained with the pushover analysis, can be approximated by a bilinear curve, in which the yielding force is assumed equal to the maximum force reached. Instead, the final displacement is the same as it was in the original capacity curve. This approach is adopted in the annex B of Eurocode 8, determining the yielding displacement as:

$$d_y^* = 2\left(d_y^* - \frac{F_m^*}{F_y^*}\right) \qquad (36)$$

Where:

E_m^* is the energy dissipated during the displacement (area under the capacity curve);

d_m^* is the ultimate displacement capacity obtained with the pushover analysis;

F_y^* is the maximum force registered during the analysis.

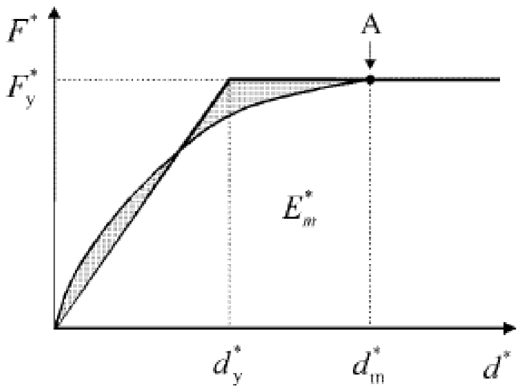


Figure 42 Example of approximation of the capacity curve obtained from pushover analysis to a bilinear curve, maintaining the same energy dissipation [76]

In the end, the bilinear capacity curve can be transformed to the Acceleration-Displacement format, by converting the force into an acceleration, dividing it by the mass of the equivalent SDOF system.

$$S_a = \frac{F^*}{m^*} \qquad (37)$$

In the S_a – S_d curve, an important role is played by the elastic period T* of the idealized bilinear system, which is going to be used for the determination of the intersection with the ADRS and consequently for obtaining the performance point. T* is determined using the yield strength and displacement:

$$T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}} \qquad (38)$$

Now that both the elastic response spectrum and the bilinear capacity curve can be plotted on the same graph, it possible to assess the acceleration demand. It is done by intersecting the elastic branch of the capacity curve (or its extension) with the elastic demand spectra related to the considered limit states. To that point correspond an acceleration demand S_{ae} required for having elastic behavior and the related elastic displacement demand.

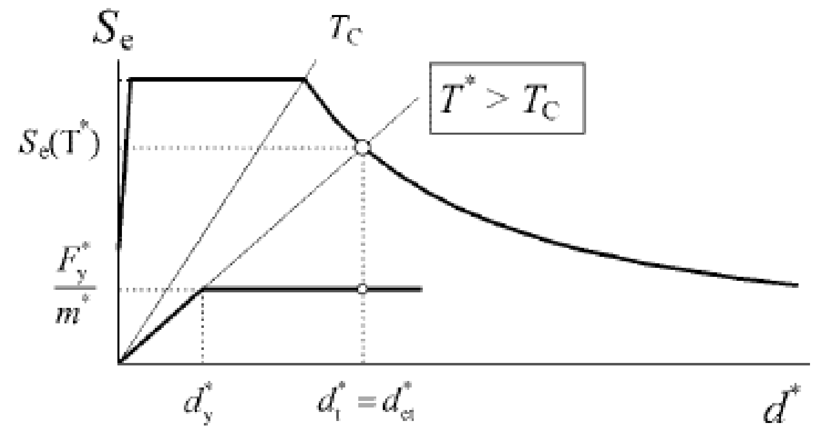


Figure 43 Example of graphical procedure to identify the acceleration demand and the related elastic displacement demand of the equivalent SDOF system, as shown in annex B of EC8-1 [61]

- In the *figure 43* it is also possible to identify the performance point, which often corresponds to the elastic displacement $d_e^* = S_{de}(T^*)$ (if $T^* \geq T_C$) and the yield acceleration S_{ay}.

The yield acceleration S_{ay} represents both the acceleration demand and the capacity of the inelastic system.

The reduction factor R_μ defined by the *formula (35)* is useful to determine the ductility demand μ caused by the selected ADRS, which will correspond to a certain return period.

μ and the inelastic displacement S_d can be determined in two different ways, whether T* is bigger or smaller than T_C:

- If $T^* \geq T_C$
  then $\quad \mu = R_\mu \quad (39)$
  and $\quad S_d = S_{de}(T^*) \quad (40)$

- If $T^* < T_C$
  then $\quad \mu = (R_\mu - 1)T_C/T^* + 1 \quad (41)$
  and $\quad S_d = \mu D^*{}_y \quad (42)$

To conclude, it is necessary to get the value of displacement for the equivalent SDOF and obtain the top displacement for the initial MDOF system with the inverse formula.

$$D_t = D^* \Gamma \qquad (43)$$

The application of the method on the existing structure and the retrofitted structure will enable us to quantify how much the intervention the structural behavior, both terms of base shear force and allowed displacement that can be sustained before the failure is reached. Such failure can be traced to two possible causes, as it was already defined in former studies on seismic assessment of existing buildings [77]:

- the most solicited plastic hinge overcomes the rotation limit assigned to Significant Damage limit state;
- a drop of the base shear force below 80% of the peak resistance is observed on the degrading branch of the pushover curve.

Thus, to obtain the results of the analysis, it is necessary to set the hinges' properties and place them on the various elements. This action will not affect the behavior of the model in elastic analyses, but it will enable to assess nonlinear analyses.

3.5.3 Definition of plastic hinges

The definition of the plastic hinges was a critical part for obtaining the required outputs from the pushover analysis.

SAP2000 gives the possibility of inserting plastic hinges into the selected elements, by specifying their relative position with respect to the starting coordinate of the component. For example, applying hinges at locations 0,05 and 0,95 of a beam whose length is 6 meters, means introducing them respectively at 0,3 m and 5,7 m from the beam starting point.

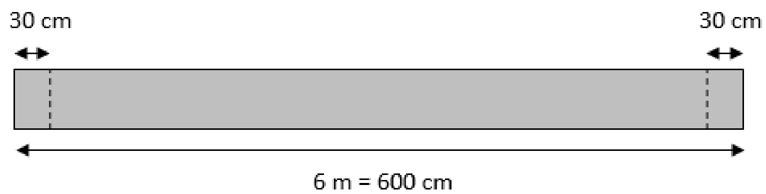


Figure 44 Positioning of plastic hinges in a beam, at relative distances 0,05 and 0,95 (dashed lines)

Apart from the position, the hinge properties can be established automatically, with reference to some American standards already implemented into the software, or defined manually by the user. The former option was adopted as a first approach to define the hinges at the start and end of all the elements in the model (0,05 and 0,95 relative positions, for both beams and columns) but with rather bad results. In fact, the computation of the pushover analysis stopped in the very first steps using this kind of hinges. In addition, since SAP2000 does not allow to set the reinforcement present in the beams but calculates the necessary reinforcement and then uses it for other computations, it was not possible to define plastic hinges relative to the defined existing structure in an automatic way.

Consequently, it was implemented a manual definition of the hinges in the horizontal elements, while the vertical elements were defined automatically.
Adequate references on hinges definition, considering also the interface of the software, were not found in the Eurocodes. Even if part 3 of Eurocode 8 [71], which is related to the seismic assessment and design for existing buildings, mentioned some criterion for the definition of chord rotation capacity at different limit states, a better definition of the properties could be found in the American standard FEMA 273 [78]. Such standard provides data that are strictly compatible with the hinge properties that can be defined and modified on SAP2000.

In fact, the plastic hinges can be defined through a load-displacement curve which is usually selected as the moment-rotation curve assuming a failure due to bending in the case of beams, or considering the interaction of biaxial bending and compression when it regards columns.

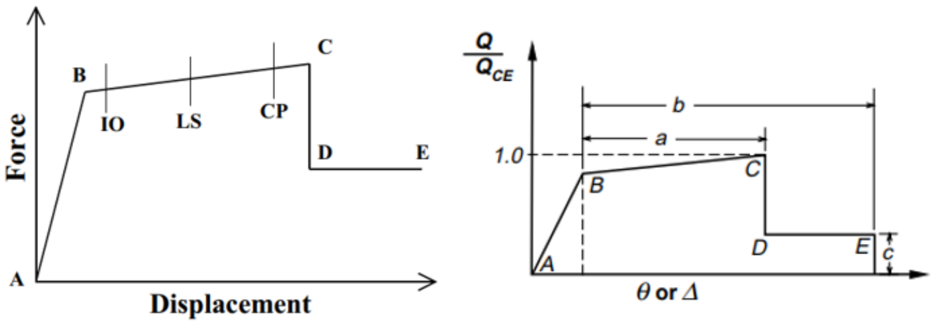


Figure 45 Example of curves characterizing plastic hinges, on the left as reported in the user manual of SAP2000 [79] and on the right as defined in FEMA 273 [78]

Here is an example of how the standard FEMA 273 hands out the values to be used for the definition of the plastic hinges:

Table 6-6 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.005	0.005	0.005	0.01

Figure 46 Parameters for the definition of plastic hinges in the case of beams [78]

After establishing the conditions to be used, which depends on the element's reinforcement and also on their strength against actions, it is easy to enter the table and define the parameters for the definition of the curve and also for estimating where the performance level changes (allowed values for the various limit states).
The possible levels are IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse prevention). Each of these states is represented by a different color for the hinges in the model. Therefore, the different conditions of the elements during the steps of the pushover analysis can be identified very easily. The condition associated to a failure of the structure in the case study is reaching the LS level in one hinge of the structure, which corresponds in the American code to the European limit state of Significant Damage (SD).

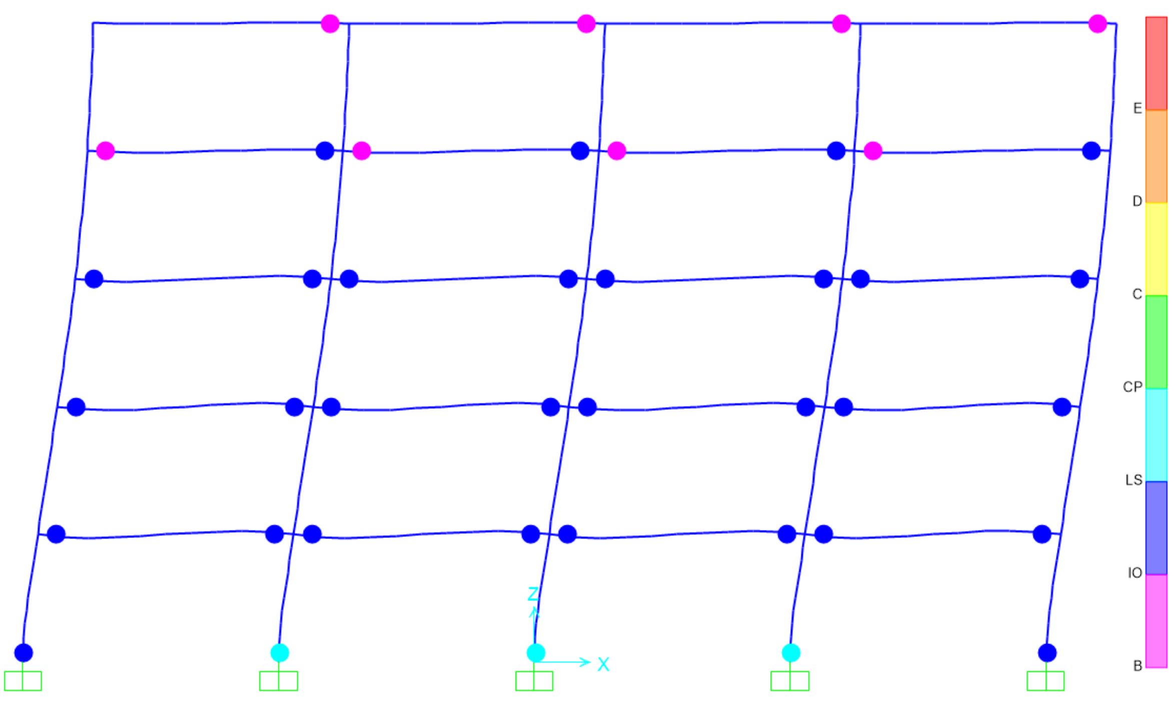


Figure 47 Example of activation of plastic hinges during a pushover analysis. The colors indicate the state of the hinges. For example, the light blue hinges at the base of the columns indicate that the rotation allowed for SD limit state (equivalent to the American Life Safety, LS) has been surpassed.

The plastic hinges can be characterized for the columns and the concrete shear walls in a very similar way. However, in all the vertical elements the plastic hinges are set to be determined automatically by SAP2000.

Instead, the timber frame shear walls do not require the introduction of any plastic hinge, since they are modeled as plastic links that can directly exploit the nonlinear behavior of such structural members.

3.5.4 Structural assessment

Once the respective hinges have been placed on both ends all the elements in the model, the various pushover analyses are run for the different configurations in both directions.

Applying the N2 method as previously depicted, it will be possible to assess the results of different retrofit in terms of structural capacity, characterizing which are the maximum base shear force and the displacement capacity of the structure. Thus, it will be possible to compare the displacement associated to failure (defined by the activation of SD state of a hinge or by the decreasing branch of the capacity curve) and the displacement capacity of the structure, to determine whether the building can sustain the defined seismic loads.

The most relevant evaluation is the one characterized by the Significant Damage limit state elastic response spectrum, associated to a return period of 475 years. Indeed, this kind of action was the only one taken into consideration for the assessment of the existing building.

But also the other two limit states, Near Collapse (NC) and Damage Limitation (DL), could be investigated enabling additional considerations on the results.

Therefore, two main kinds of comparisons were possible:

1) Comparison of the assessment of the existing structure with the pushover analysis coupled with N2 method or with the modal response spectrum analysis using the design spectrum. The consequent observations are going to highlight the difference that is involved in evaluations of structural capacity using linear or nonlinear seismic analysis methods.
2) Differences in terms of results for the two possible retrofit measures under study, also referring to the existing building behavior. In such a way it will be feasible to compare the effectiveness of the two technologies for increasing the resistance to horizontal loads and the ductility of the existing structure.
   The study tries to reach similar structural results with both the interventions and consequently using more timber walls than concrete walls, due to the higher capacity of the designed concrete walls compared to the light frame timber solution. Hence, it will be possible to evaluate whether using more timber walls could imply anyway less environmental impacts than adopting concrete shear walls.

Therefore, the next step of the research regards the evaluation of environmental impacts.

3.6 Evaluation of environmental impact

In order to obtain an evaluation as general as possible of the impacts related to the different retrofitting techniques, it was not chosen a specific product to be used in the evaluation (e.g. a particular steel rebar produced by a certain company). Instead, research of various **Environmental Product Declarations** (EPDs) of the same components was performed, with the aim of assessing an average value of **kg of CO_2 equivalent** emitted respectively in the implementation of concrete and frame timber shear walls. Other parameters such as the **consumption of primary energy sources** and the **net fresh water** usage were taken into account too.

But first, it is necessary to define precisely what an EPD is.

Environmental Product Declarations are a particular type of ecolabelling, grouped in the so-called type III, regulated by the international standard ISO 14025:2011. In the case of construction products, EPDs should also compel with the standard EN 15804.
EPDs can be defined as voluntary programs that provide quantified environmental data about a product, under pre-set categories or parameters. They are certified by third parties and are based on Life Cycle Assessment (LCA). Therefore, the indications provided by EPD is related to the various phases of the product’s life cycle.

EPDs are an environmental labelling that’s not suitable for a **communication** from business to consumer, but works in a good way **between businesses**, since they enable comparison between similar products and are useful when fulfilling mandatory requirements (e.g. Criteri Ambientali Minimi (CAM) provided by the Italian legislation). The aim of the declaration is also to provide **transparency and comparability**, but they don’t deal with direct comparison with targets or other products.

EPDs can cover several product categories, which are constituted by products that can fulfil equivalent functions. Based on this variety, some documents provide sets of rules, requirements, and guidelines for the development of EPDs into a certain category. Such documents are the Product Category Rules (PCR).

EPDs are usually open source and quite easily accessible through websites. Considering that the case study is located in two different parts of Europe, websites referring to both national and European resources were consulted.

In particular, the following databases contained useful materials:

- EPD Italy [80]
- International EPD system [81]
- Institut Bauen und Umwelt e.V. (IBU) – EPD programme [82]
- ÖKOBAUDAT – Informationsportal Nachhaltiges Bauen [83]

Keeping the study as general as possible implies a quite complicated operation, because of two main reasons:

i) Despite the attention to environmental issues is growing, there still are **not that many Environmental Product Declarations available**. This applies especially to those products that could appear to be negligible because they don't constitute the main elements of a construction, but they are in any case important for a meticulous evaluation. For instance, EPD for fasteners were found only regarding self-tapping screws. Consequently, some approximations had to be done, and the same EPD had to be used also for staples and other kind of connecting elements.

ii) Another determinant aspect is that EPDs imply in their redaction a lot of assumptions and approximations, mainly related to the **local context** in which the document is addressed. For example, in the transportation section, a certain distance from the construction site is considered. The problem arises when different assumptions are made, as it is expectable since different EPDs are made in different contexts and by different professionals. The same reasoning applies to maintenance scenarios, which are influenced by weather conditions.
Hence, the use of differing assumptions can affect the comparability of the EPDs results.

Figure 48 Logos of EPD Italy [80], International EPD System [81], IBU [82]

Here are the conditions in which different EPDs produce comparable outputs, according to EN 15804: [84]

- Same functional requirements;
- Environmental quality and technical quality of any composite parts, components or products excluded are the same;
- Quantities of excluded material are the same;
- Excluded processes, modules or life cycle stages are the same;
- Influence of the product system on the aspects and effects of the building operation is considered;
- Elemental fluxes associated with material inherent properties are fully and consistently considered as described in the standard.

Therefore, EPDs can be compared only if the assumptions made are close enough to each other.

As soon as these difficulties are overcome, the method adopted for the evaluation gets to its simpler stage: data from the EPDs of different specific products for the same element are gathered and the average of their interesting output values is calculated.
Then, all the outputs of components that are included in each wall are summed up and the final results are obtained. The two retrofitting technologies can be in this way compared in terms of kg of CO_2 equivalent, as well as with other indicators. The comparison can be straightforward by highlighting the ratio between the impacts of one concrete wall over one timber wall. Another result explaining the comparison could be the difference between the impacts, showing for instance the spared emissions in the case the less impactful retrofit is chosen, compared to the other technology.

In the end, it is necessary to remind that EPDs cover only one of the three pillars of sustainability: the environmental field. Tools suitable for an overall sustainability evaluation should consider also the social and economical spheres, while this assessment is strictly environmental.
Nevertheless, criteria in the Green Building Rating Systems are not always facing all the three aspects at the same time, which are instead considered all together in the complexity of sustainability certification, making use of the combination of criteria. That is why a study of the impact only in environmental terms could still be useful for a criterion proposal.

4 Numerical investigations and results

In the following paragraphs the results obtained in the various steps of the procedure, executed as described in the previous chapter, will be reported also defining more in detail what are the assumptions made in the study.

4.1 Case study definition

Since the ideation of the case study does not involve any numerical query, here in the following table are summarized the characteristics of the building under investigation.

Building use destination	Office building	
Year of construction	1978	
Location 1 (Italy)	Zambrone (coordinates: (38.70, 15.99))	
Location 2 (Germany)	Aachen (coordinates: (50.78, 6.08))	
PGA (475 years return period)	Location 1	0,24g
	Location 2	0,12g
Structural material	Reinforced concrete	
Structural typology	Moment Resisting Frame (MRF)	
Number of bays (6 m span each)	Longitudinal direction	4
	Transversal direction	2
Number of floors (3 m storey height)	5	
Materials properties	Steel FeB32	Elastic modulus: E = 200 GPa Density: ρ = 7700 kg/m^3 Characteristic yield stress: f_y = 320 MPa Characteristic tensile stress: f_u = 500 MPa Average yield stress: $f_{y,m}$ = 430 MPa Average tensile stress: $f_{u,m}$ = 645 MPa
	Concrete C25/30	Elastic modulus: E = 31 GPa Density: ρ = 2500 kg/m^3 Characteristic concrete cylinder strength: f_{ck} = 25 MPa Poisson's ration ν = 0,2 Average concrete cylinder strength: f_{cm} = 33 MPa (taken from table 3.1 of Eurocode 2) [69] Shear modulus: G = 12,92 GPa

4.2 Dimensioning of existing structure

This paragraph will cover the definition of the geometrical characteristics of the idealized existing Moment Resisting Frame structure in reinforced concrete. The operation will be done accounting for the fundamental load combination in both the locations (Zambrone, Italy and Aachen, Germany), with the addition of a modal response spectrum analysis for the Italian case study. In the computation of seismic loads, a response spectrum defined in Italian legislations from the year 1975 is adopted.

The analysis starts with the definition of all the loads to which the structure is subject.

4.2.1 Definition of loads

First of all, it is necessary to identify the critical loads searching for the decisive load combination for ULS:

$$F_{Ed} = \sum_{j\geq 1} \gamma_{G,j} G_{k,j} \;\; "+" \; \gamma_P P \; "+" \; \gamma_{Q,1} Q_{k,1} "+" \; \sum_{i\geq 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \qquad (1)$$

The loads due to the self-weight of the structure are automatically computed by the software. The way in which the masses of the structure are defined can vary. It can be done though the density and volume of each component or through the definition of masses depending on a selected load combination.
The former approach is used during the dimensioning of the structure by means of the fundamental load combination, whereas the latter is involved in the computation of inertial masses to be considered in case of seismic actions. Therefore, apart from the computations regarding the fundamental load combinations, the definition of masses using applied loads will be exploited throughout the whole study.

Some of the other loads are common for both the locations, such as:

1) **Permanent loads**, due to:
   - Horizontal structural partitions: reinforced concrete bidirectional slab. Assumed thickness of 16 cm. Density of 2500 kg/m^3. Hence **g_{slab} = 4 kN/m^2**
   - Flooring components: leveling mortar and floor tiles, other finishings. Assumed **g_{floor} = 1,8 kN/m^2**
   - Vertical external partitions: perimetral multi-layer walls composed by bricks, plasters, insultation. Assumed a load of $g_{ext.walls}$ = 4 kN/m^2. Multilplied by the net floor height between beams (h_{net} = 2,5) and by a reduction coefficient equal to 0,9 to consider the presence of voids (windows and doors) into these walls. Obtaining then **$G_{ext.walls}$ = 9 kN/m**. This linear load needs to be applied directly on the perimetral beams of the frame, except from the roof beams.

2) **Variable** (live) **loads** (referring to the Eurocodes):
   - Occupancy loads for floors of residential buildings (category A): $\mathbf{q_{occupancy,A} = 2\ kN/m^2}$
   - Occupancy loads for floors of office buildings (category B) $\mathbf{q_{occupancy,B} = 3\ kN/m^2}$
   - Occupancy loads for non-accessible roofs: the code imposes the use of the same loads applied in floor for the cases in which the roof is accessible only for maintenance purposes. It will be respectively $\mathbf{q_{roof,A} = 2\ kN/m^2}$ and $\mathbf{q_{roof,B} = 3\ kN/m^2}$ for categories A and B.
   - Movable partitions: single layer wall of bricks with plaster applied on both faces. The areal load due to the components (density multiplied by wall thickness) considered is 1,5 kN/m^2, which is multiplied by an interstorey net height of 2,84 m, obtaining $\mathbf{Q_k = 4{,}26\ kN/m}$. According to the Eurocode 1 [63], linear loads from moveable partitions can be distributed over the floor area if they are within certain ranges, considering the fact that they could be moved during the lifetime of the structure. In this case, the value is over the maximum allowed to be redistributed for the Eurocode and should be considered as a linear load in the effective position of the partition inside the building. To keep a simpler approach, reference was made to the Italian normative NTC2018 [85], which enables to convert linear loads due to moveable partitions up to 5 kN/m into distributed loads. In the case study, the final value of areal load $\mathbf{q_k = 2\ kN/m^2}$.

All the mentioned distributed loads were applied on computational areas defined into the software, one for each floor. In this way, the distributed loads are conveyed in a bidirectional way to the frame elements on which the area is defined.

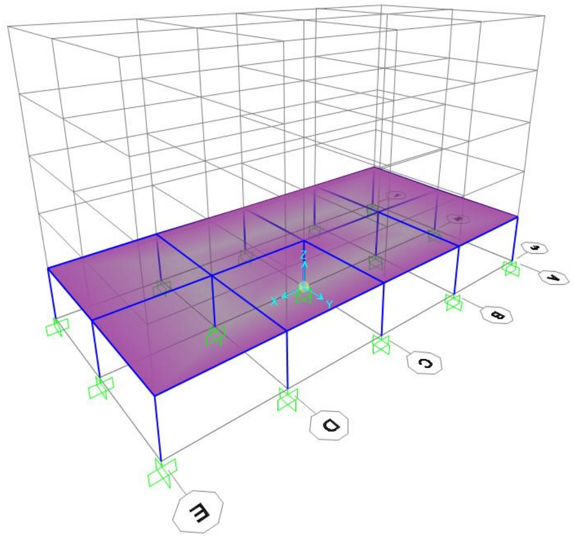


Figure 50 Example of area defined for the application of distributed loads to be transmitted into the reinforced concrete frame

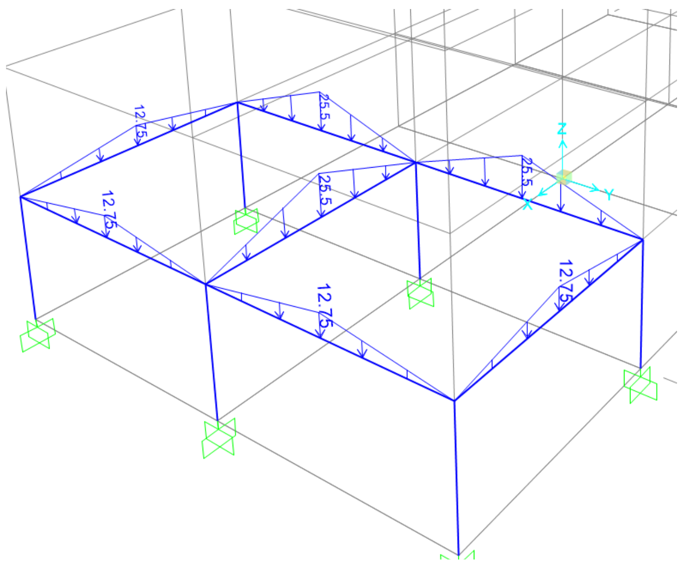


Figure 49 Example of bidimensional distribution of areal loads (variable loads for category B: occupancy and moveable partitions). The distribution is triangular as expected

In the same way in which areas are defined on **each floor**, all the joints included in the area of each floor are constrained creating a **diaphragm**. Thanks to this constraint, the nodes of the whole floor are imposed to have the same vertical displacements, as if the slab was infinitely stiff against bending moments.

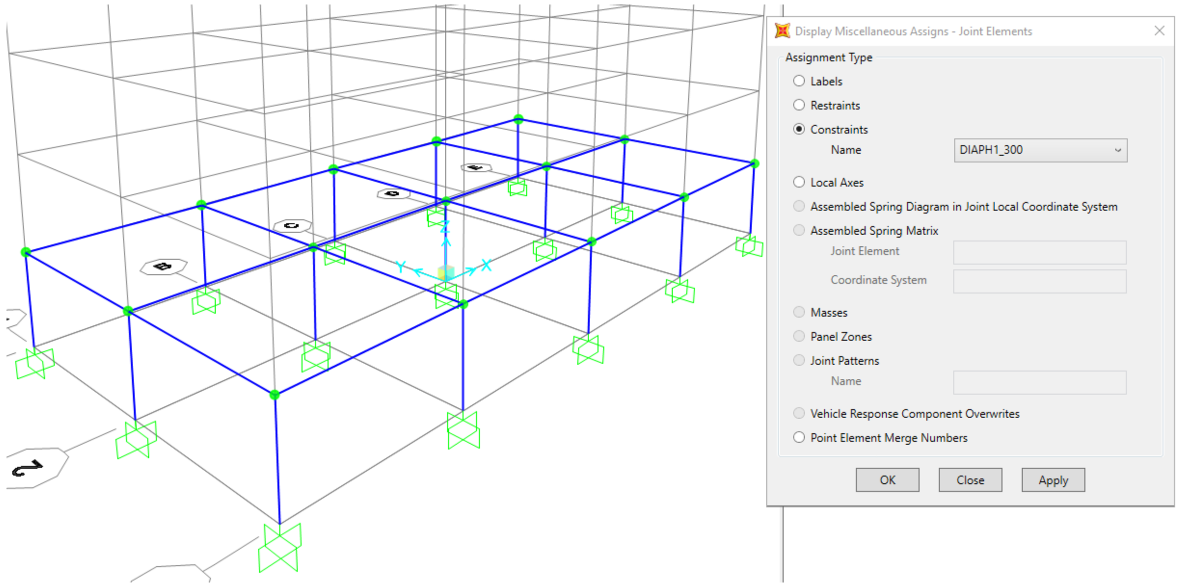


Figure 51 Imposition of the diaphragm for the first floor. All the nodes indicated

Moreover, for the variable loads due to occupancy that repeat over successive floors, the decrement factor given by *formula (2)* is computed: $\alpha_n = \frac{2+(n-2)\Psi_0}{n}$.
It is used to assume the non-contemporaneity of such loads on all the interested floors. Finally, the values of distributed variable loads for the two possible categories on the various floors are obtained, considering both the occupancy and the moveable partitions, with the application of the non-contemporaneity coefficient:

Floor	n (number of floor above the element considered)	Ψ_0	α_n	Category A [kN/m^2]	Category B [kN/m^2]
1	4	0,7	0,85	3,4	4,25
2	3	0,7	0,9	3,6	4,5
3	2	0,7	1	4	5
4	1	0,7	1	4	5
5	-	-	-	2	3

Then, some **other loads** depend on the location. Such loads are influenced by geographical factors and in the case study, those loads are related to wind, snow, and earthquakes (which are not present in the fundamental load combination but are going to be used in the seismic load combination for the dimensioning of the Italian case).

3) **Variable loads, depending on geographical context**. In the **first location** (Zambrone, Italy), these loads were computed with reference to the Italian code NTC2018 [85], which is the Italian national annex of the Eurocodes. In particular:

- The **snow load** is calculated depending on the zonation applied to the country, in addition to the altitude above sea level, some coefficients for thermal and exposition effects and a shape factor for the roof configuration. The reference expressions is:

$$q_s = q_{sk}\mu_i C_E C_t \qquad (44)$$

  Since the building is located in the south of Italy, where the snow is hardly present, the value of q_s is expected to be quite low. This can be affirmed even if the shape of the roof is assumed to be planar, which does not help in the removal of snow by gravity, hence has an higher value of μ_i compared to other configurations.

  Without going deeper in further details, the value of the distributed snow load is: **$q_{s,IT}$ = 0,49 kN/m^2**.

- Regarding the **wind loads**, a similar computation could be applied, concerning many parameters which depends especially on the location of the structure and the topography of the area. An important observation is that since the exposure coefficient varies with the height of the part of building considered, also the pressure on the building due to wind varies with the vertical coordinate. Once the various classes and categories are set, it is possible to compute the wind loads with SAP2000, selecting the reference normative. The resultant wind loads are applied on the center of each diaphragm, and are displayed directly as the product of the wind pressure multiplied by the area of influence of the corresponding floor:

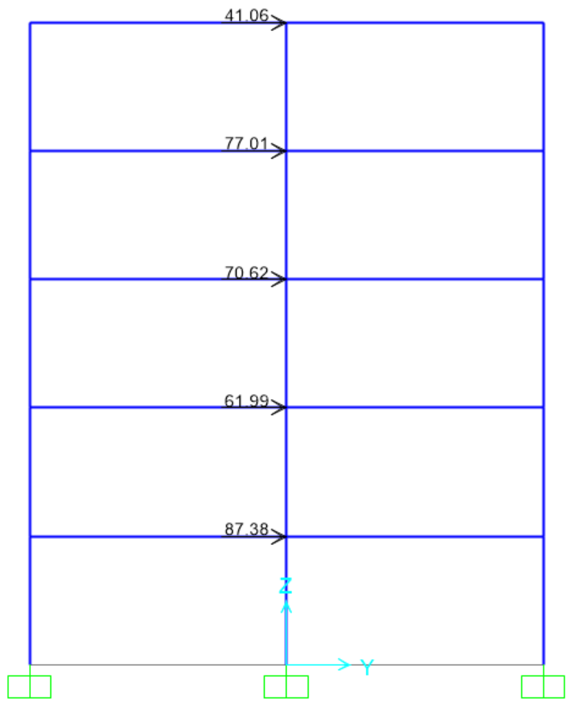


Figure 52 Example of concentrated wind loads (in kN) computed by the software, in the transversal direction of the building. Italian case.

4) **Variable loads, depending on geographical context**. In the **second location** (Aachen, Germany), they were computed with reference to the general indications of the Eurocodes, since the use of the national annex was completed for language barriers.
   - The snow loads were calculated in a very similar way to the Italian case. The value is expected to be higher than in the south of Italy. Indeed, the areal load obtained is $\mathbf{q_{s,DE} = 0{,}71\ kN/m^2}$.
   - With the same procedure applied in the Italian case study, the wind load was automatically computed by SAP2000 indicating the use of Eurocodes for their determination, after setting all the necessary parameters.
     The results obtained in terms of concentrated loads are lower than in the Italian case, probably because of the position of Zambrone near the coast.

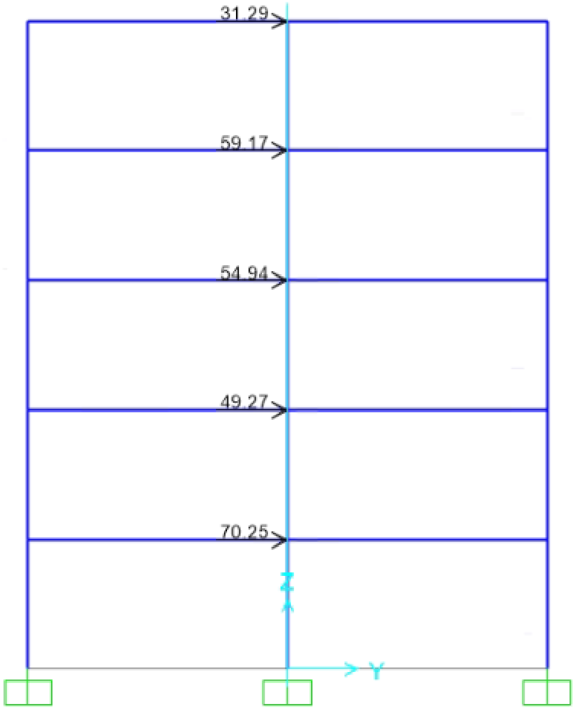


Figure 53 Concentrated wind loads (in kN) on the center of each floor's diaphragm computed by the software, in the transversal direction of the building. German case.

- **Seismic loads** for seismic load combination. Definition of the response spectrum according to **DM 3-3-1975** [65] for the dimensioning in **Zambrone**.
  As shown in the methodology chapter, the Italian case study is dimensioned also with a modal response spectrum analysis. The reference spectrum is obtained from a code of the mid-1970s, and the process for defining it is hereby depicted.
  Firstly, it was necessary to define the expression giving the spectrum:

$$a/g = C\ R \qquad (45)$$

In *formula (45)*:

- a is the spectral acceleration applied to the building in correspondence of a certain vibration period;
- g is the gravity acceleration;
- R is called response coefficient and it is a function depending on the vibration period of the structure T, which is the variable on the horizontal axis of the defined spectrum;
- C is the coefficient of seismic intensity, given by:

$$C = \frac{S-2}{100} \qquad (46)$$

Where the seismicity grade S is associated with the national seismic zones defined in 1974. In the case of Zambrone, S was equal to 12.
The only further specification regards R, which can be defined by two different formulas depending on the value of the fundamental vibration period T_0:

if $T_0 \leq 0,8$ s $\qquad R = 1,0 \qquad (47)$

if $T_0 > 0,8$ s $\qquad R = \frac{0,862}{T_0}\frac{2}{3} \qquad (48)$

With these definitions it is possible to print the response spectrum for the location of the Italian structure. It will be used in the modal response spectrum analysis to check the dimensioning of the existing building for the Italian case study, with the procedure illustrated in the *paragraph {3.3}.*

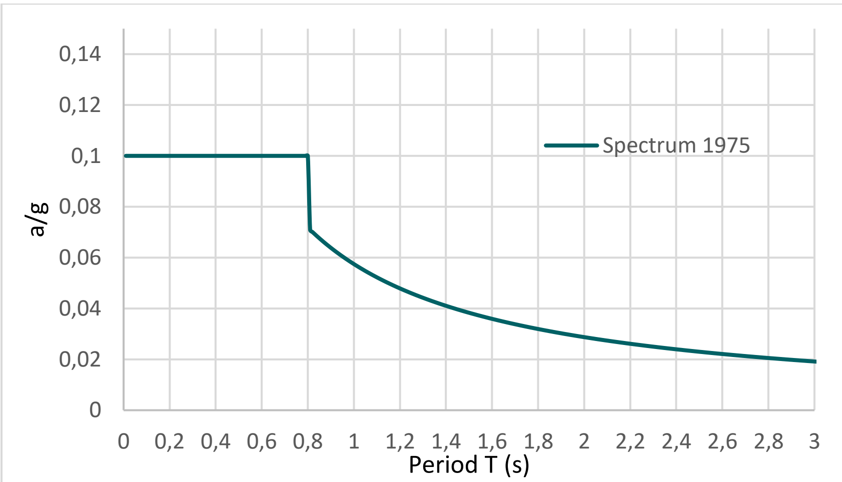


Figure 54 Response spectrum used for the dimensioning

Once all the involved loads were defined, they were introduced in the model and applied on the respective elements. Then, all the possible load combinations deriving from the fundamental load combination were defined in the model, considering alternatively the various variable loads as main and secondary variable loads. Thereby, the coefficients applied to the different variable loads were modified to obtain all the possible load combinations. At this point it was possible to operate the design on the software, as it will be shown in the next paragraphs, for the two possible locations.

4.2.2 Dimensioning in location 1: Zambrone (Italy)

The Italian case study was dimensioned considering directly both the possibilities involving the fundamental load combinations and the seismic load combinations, applying the response spectrum previously defined.

Since the consideration of the two load combinations implies a different mass source in the two cases, the checks were operated in two separate steps: at first the fundamental load combination was applied, assuming the masses as a result of the volume and the density of the elements, then the seismic loads were considered with the inertial masses of the structure defined as in *formula (5)*:

$$m = \sum G_{k,j} \text{"+"} \sum \Psi_{E,i} Q_{k,i} \qquad (5)$$

By consequence, the only loads considered where the gravity loads and the variable loads related to occupancy, for which the $\Psi_E = \varphi\, \Psi_2 = 0{,}8 \cdot 0{,}3 = 0{,}24$

In all the other variable loads, either φ or Ψ_2 were null according to the values provided in tables by the Eurocodes, obtaining $\Psi_E = 0$.

Then, after defining all the loads, applying them in the model and defining the load combinations derived by the fundamental load combination, the analysis was run and the "**concrete design**" by SAP2000 was executed. In the first place, the reinforcement of the columns' cross-sections was still to be determined, using the function of SAP2000 that indicates the necessary amount after the run of the design.

Remembering that in the first phase of the research both residential and office use categories were considered for the building, here are the results obtained with the application of the fundamental load combination considering the category A (residential):

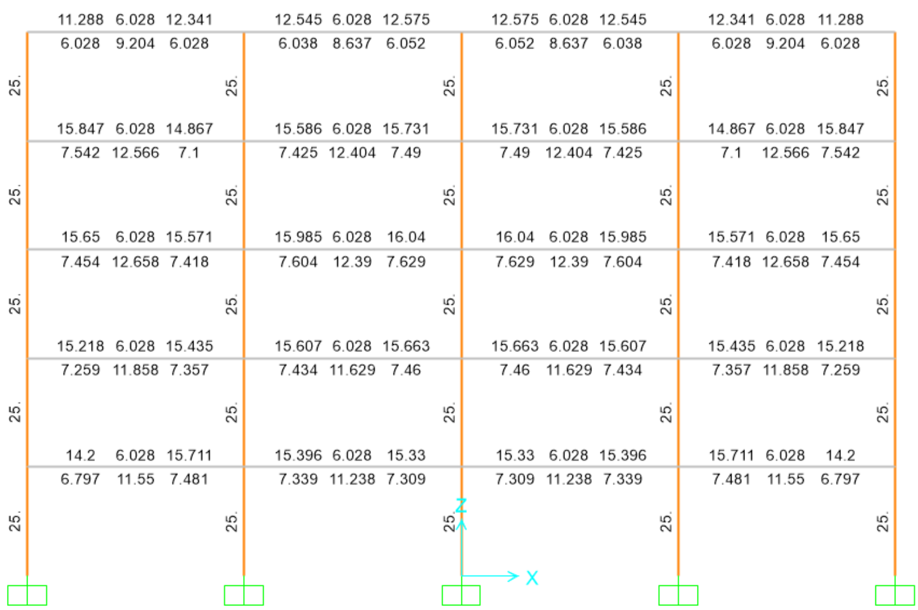


Figure 55 Rebars required in the central longitudinal frame, with the use category A.

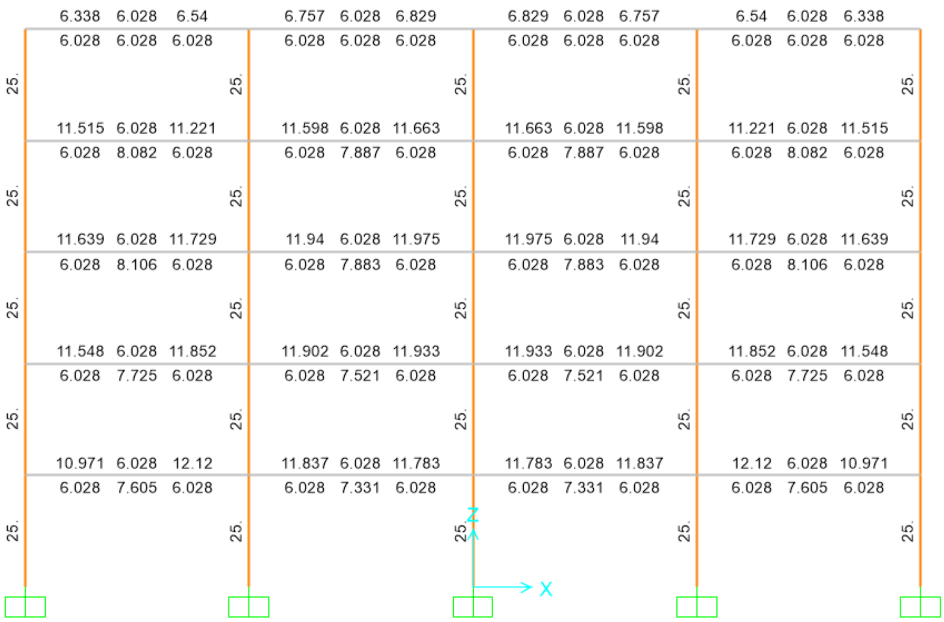


Figure 56 Rebars required in the external longitudinal frames, with the use category A.

Figure 57, reported on the right, shows the central transversal frame, in which are present the highest stresses on beams. Precisely, the beams that require the most reinforcement are the ones on the third floor for the **midspan** section, with a required area of **reinforcement** at the of **12,67 cm²**, and the ones in the second floor for the end-sections of the elements, with a **top-reinforcement**, resisting the negative bending **at the node**, equal to **16,90 cm²**.

Also in the case of category B the central frame showed the maximum requirements. For comparison, the results obtained in the same frame but for the offices use destination are reported below.

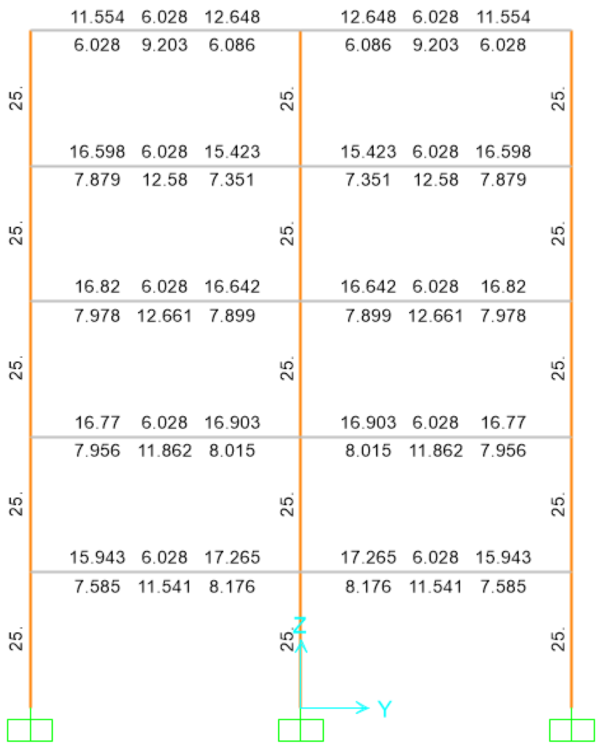


Figure 57 Rebars required in the central transversal frame, with the use category A.

As it was expected, since the variable loads related to occupancy are higher, in the case of **category B** the requirements for steel rebars in the beams are higher. The maximum values are **20,64 cm²** at the **bottom of midspan** and **18,68 cm²** in the proximity of the **nodes**, at the **top** of the section.

Therefore, it is evident how the study can be conducted on the sole category B, which involves higher solicitations of the elements compared to the residential case.

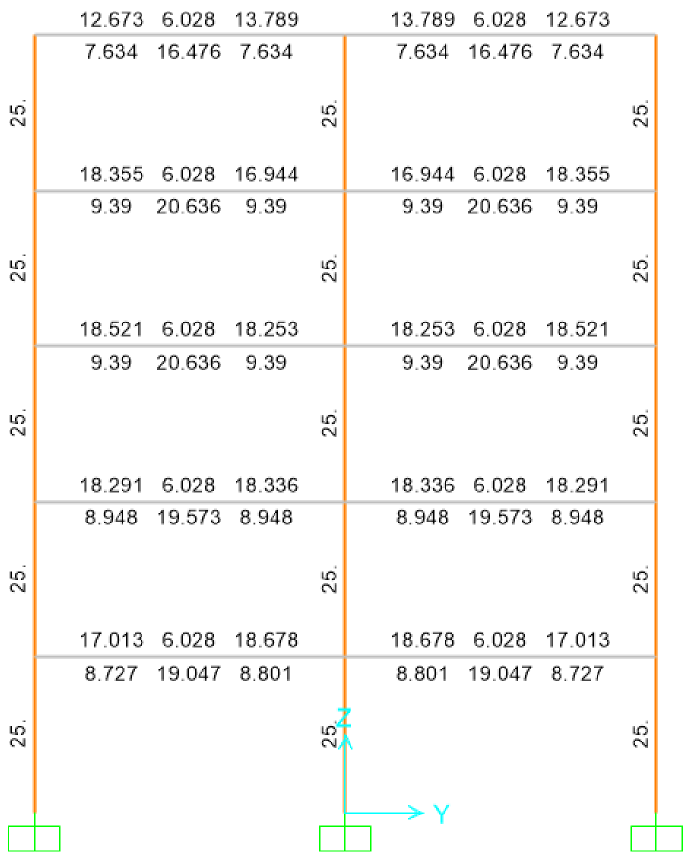


Figure 58 Rebars required in the central transversal frame, with the use category A.

For what concerns the columns, the reinforcement necessary according to the design function of SAP2000 is equal to 1/100 of the concrete area of the cross section. It is never higher than this minimum threshold due to the loading conditions applied to the structure, not so heavy for 50x50 cm^2 columns.

Afterwards, it was possible to investigate the amounts of rebars required by the application of **seismic load combinations**. Like in the previous case, the settings in SAP2000 concrete design tool are kept as predefined, including the nominal curvature method for second order analysis, except the utilization factor limit imposed to 0,9 instead of 0,95.

With this analysis the beams are much less exploited and have definitely lower requirements in terms of reinforcement, while the contrary is verified for the columns, in which the amount of rebars necessary for the elements to be verified is much higher. Considering the longitudinal frames, the most solicited frames are the external ones, with a maximum value of **59,16 cm^2** in the **columns** at the edges of the ground floor. Instead, all the required longitudinal reinforcements in the columns of the central longitudinal frame require lower amounts of steel bars.

Consequently, the amount of rebars in the various elements can be designed, extracting excel files from the model which indicate the values obtained for each element in the analysis. It was in this way identified which elements were most solicited by the various load combinations.

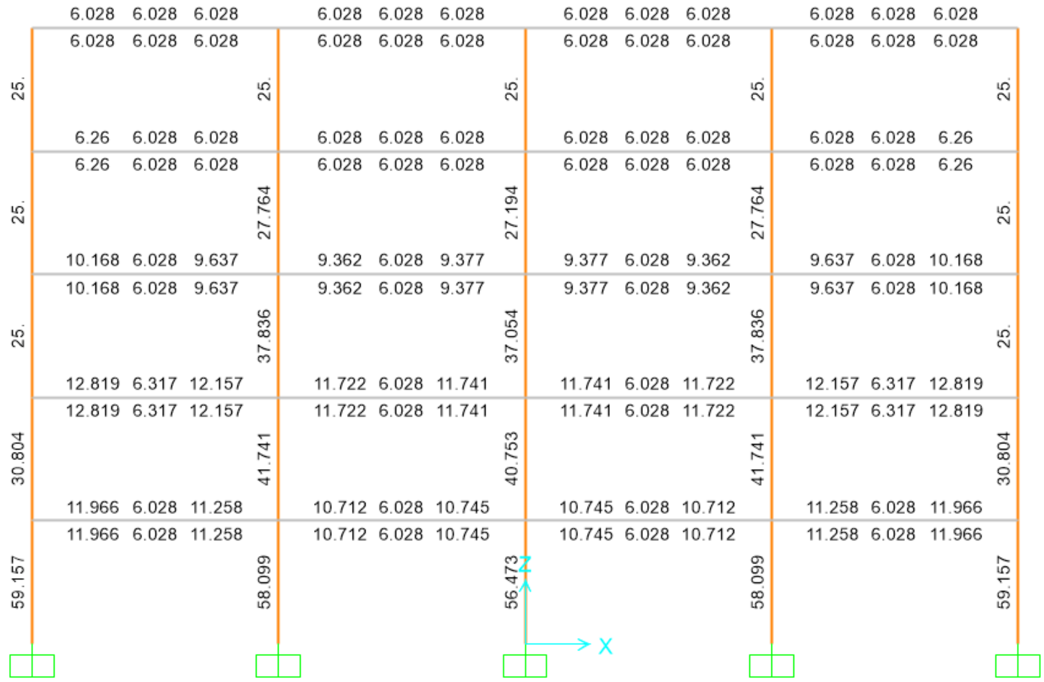


Figure 59 Rebars required in the external longitudinal frames, applying the seismic load combination.

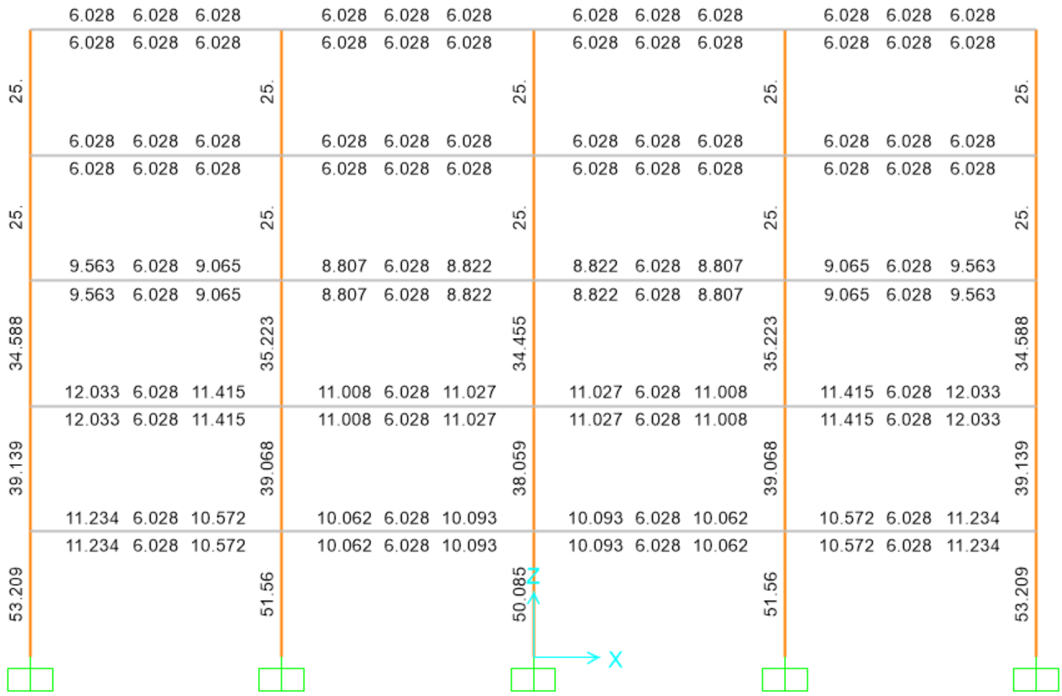


Figure 60 Rebars required in the central longitudinal frame, applying the seismic load combination.

Considering a design carried out on the most exploited elements, that gives an output which is then replicated for all the other elements of the structure, the following tables shows the design that was adopted for the beams and the columns:

Element	Cross-section [cm^2]	Distance from node	Top rebars area required [cm^2]	Top rebars configuration (diameter in mm)	Top rebars area adopted [cm^2]
Beams	50x30	x = 0 cm	18,64	6 Ø20	18,85
		x = 100 cm	9,08	5 Ø16	10,05
		x = 300 cm	6,027	3 Ø16	6,032

Element	Cross-section [cm^2]	Distance from node	Bottom rebars area required [cm^2]	Bottom rebars configuration (diameter in mm)	Bottom rebars area adopted [cm^2]
Beams	50x30	x = 0 cm	13,69	7 Ø16	14,07
		x = 100 cm	9,39	6 Ø16	12,06
		x = 300 cm	20,64	7 Ø20	21,99

Element	Cross-section [cm^2]	Distance from node	Longitudinal rebars area required [cm^2]	Rebars configuration (diameter in mm)	Longitudinal rebars area adopted [cm^2]
Columns	50x50	All along the element	59,16	10 Ø28	61,57

The reinforcement defined so far is only regarding the longitudinal directions of the elements. Some observations about the **transversal reinforcement** for both the columns and the beams need to be developed. Since the common construction practice of the last century, even in the second half, did not take much into consideration the importance of the confining reinforcement as it is done nowadays, the study will proceed with some assumptions in that direction.

For the columns, for which the reinforcement can be defined in SAP2000, it is assumed a really low amount of transversal rebars, simulated in the model with a diameter of 8 mm and with a spacing between consecutive bars in the longitudinal direction of the column of 50 cm.

For what concerns the beams, the values used are more similar to the current common practice, but still not sufficient according to the design values extracted by the software. This has to be related to the fact the building is undergoing very high shear stresses due to applied seismic loads, which need to be completely faced by the frame since there are no other components acting against the horizontal loads. The results obtained in terms of transversal reinforcement area per meter of beam's length would be then unreasonable, especially for a building constructed in the 1970s. Therefore, even if the verification given by the ratio between the required and adopted rebars is not satisfied, the values of adopted transversal reinforcement are the following:

Element	Cross-section [cm^2]	Range of distance from node	Transversal rebars required [cm^2/cm]	Transversal rebars configuration (diameter in mm)	Transversal rebars adopted [cm^2/cm]
Beams	50x30	0 – 100 cm	0,145	Ø8/20 cm	0,025
		100 - 200 cm	0,127	Ø8/25 cm	0,020
		200 - 300 cm	0,040	Ø8/40 cm	0,013

At this point, it was possible to insert the specified amounts of rebars for the columns in the model and operate the analysis with the cross-section properties set to "**reinforcement to be checked**". In this way, the result of the calculation will be displayed by the model in terms of **demand/capacity PPM ratio**, which is required to be lower than 0,9.

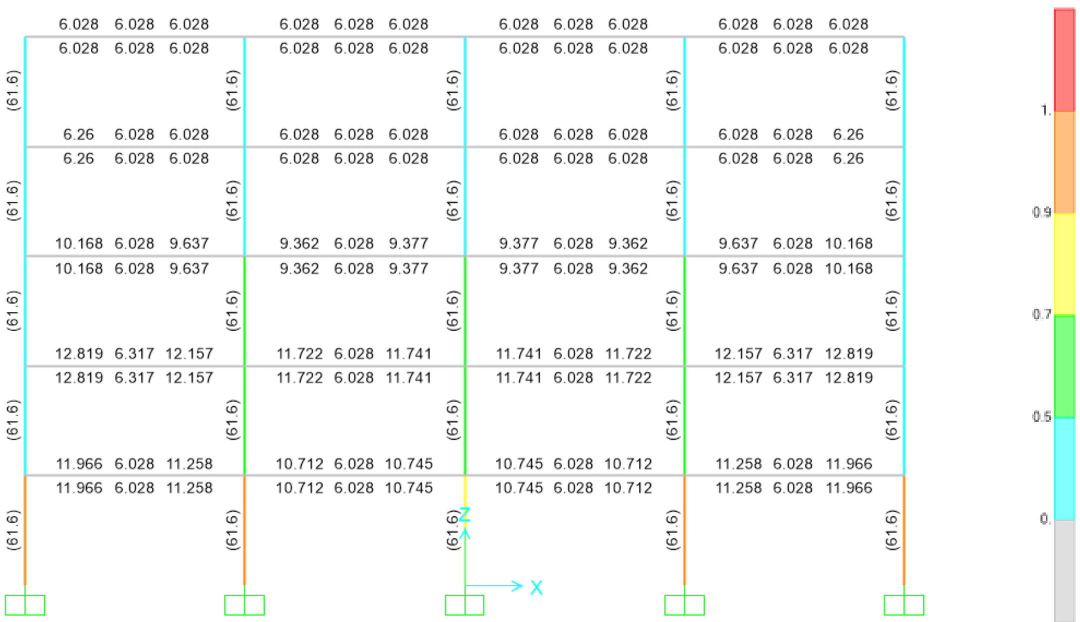


Figure 61 Check of demand/capacity ratio for columns in the external longitudinal frames, applying the seismic load combination.

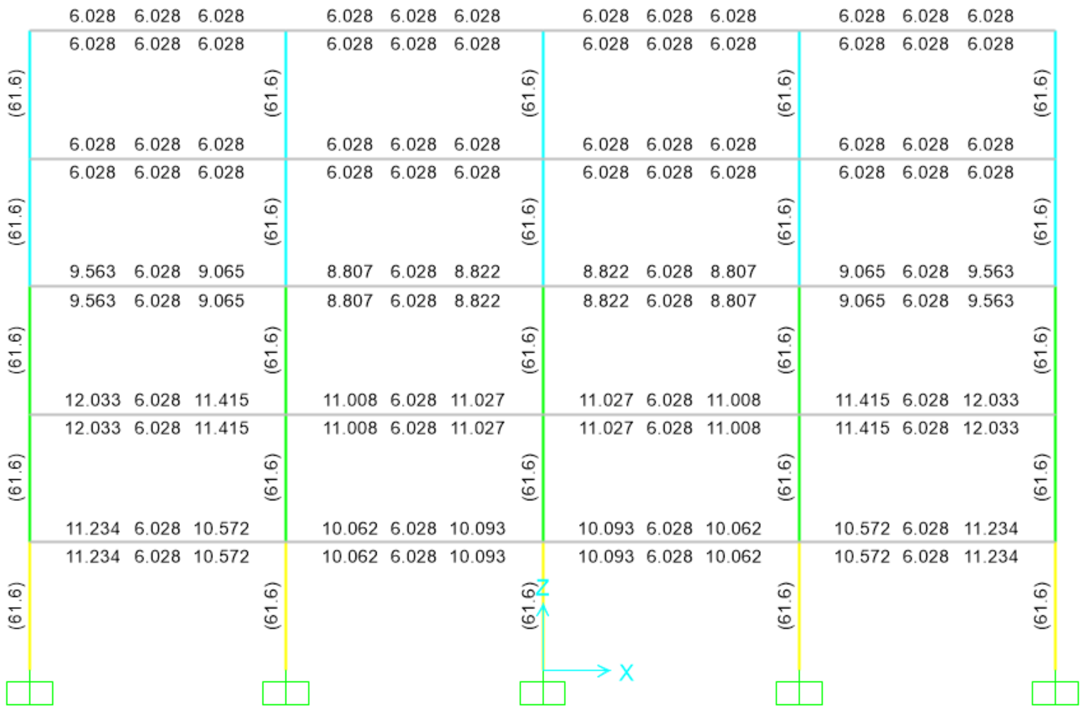


Figure 62 Check of demand/capacity ratio for columns in central longitudinal frame, applying the seismic load combination.

As in the design phase, the most stressed columns are the ones in the four vertexes, for which the exploitation ratio is equal to 0,87.

Instead, for what concerns the beams the verification was carried out simply dividing the amount of rebars provided in the design by the one required according to the analysis performed with the fundamental load combination, as show in *formula (4)*:

$$check = \frac{Area\ of\ reinforcement\ required}{Area\ of\ reinforcement\ defined\ in\ the\ element} = \frac{A_{s,required}}{A_{s,element}} \qquad (4)$$

In this way it is also possible to show how large is the margin that separates the chosen configuration from the limit situation, after which the check would not be satisfied anymore.

Element	Cross-section [cm²]	Distance from node	Top rebars area required [cm²]	Top rebars area adopted [cm²]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	18,64	18,85	0,989
		x = 100 cm	9,08	10,05	0,903
		x = 300 cm	6,027	6,032	0,999

Element	Cross-section [cm²]	Distance from node	Bottom rebars area required [cm²]	Bottom rebars area adopted [cm²]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	13,69	14,07	0,973
		x = 100 cm	9,39	12,06	0,779
		x = 300 cm	20,64	21,99	0,939

Even if some of the ratios are near 1, which is the limit value to satisfy the check, it is important to recall that the building is assumed to be designed in the end of the 1970s, and with existing buildings the actual detailing is often not very precise and not exceeding the minimum required. Furthermore, the approach used for design in that period was the one related to admissible tensions, which for sure would give different results than the limit state methodology. Therefore, little margins are also accepted, and the selected amount of reinforcement is confirmed. This is valid also in the case of transversal reinforcement already discussed where the implemented rebars are not enough compared to the results in SAP2000.

Finally, all the elements of the existing structure are defined. Their configuration will be then checked against the current seismic loads acting on the structure according to Eurocode 8 [61] specifications.

4.2.3 Dimensioning in location 2: Aachen (Germany)

The same procedure adopted for the Italian case was applied also in the second location, with the exception of the analysis under seismic loads. Consequently, the dimensioning of the structure located in Aachen was performed only considering the fundamental load combinations, for which the mass of the structure was deducted by the elements' volume and density.

Since there are no seismic loads applied in the dimensioning of the structure, a smaller cross-section for the columns was assumed as a first hypothesis: 40x40 cm^2, against the 50x50 cm^2 adopted for the Italian case study. The beams are the same: 50x30 cm^2.

This paragraph will present the results obtained with the application of the loads for the German location and directly considering category B, which will be investigated throughout the research. Despite the procedure is the same adopted for the Italian location, with also the utilization factor limit set to 0,9 instead of 0,95 in the design settings, namely in these settings a difference was encountered. Indeed, selecting the country as Germany in the analysis configuration, the default second order method of analysis becomes nominal stiffness instead of nominal curvature that was automatically adopted setting the country as Italy.

In any case, here are the required rebars adopting the “reinforcement to be designed” analysis mode:

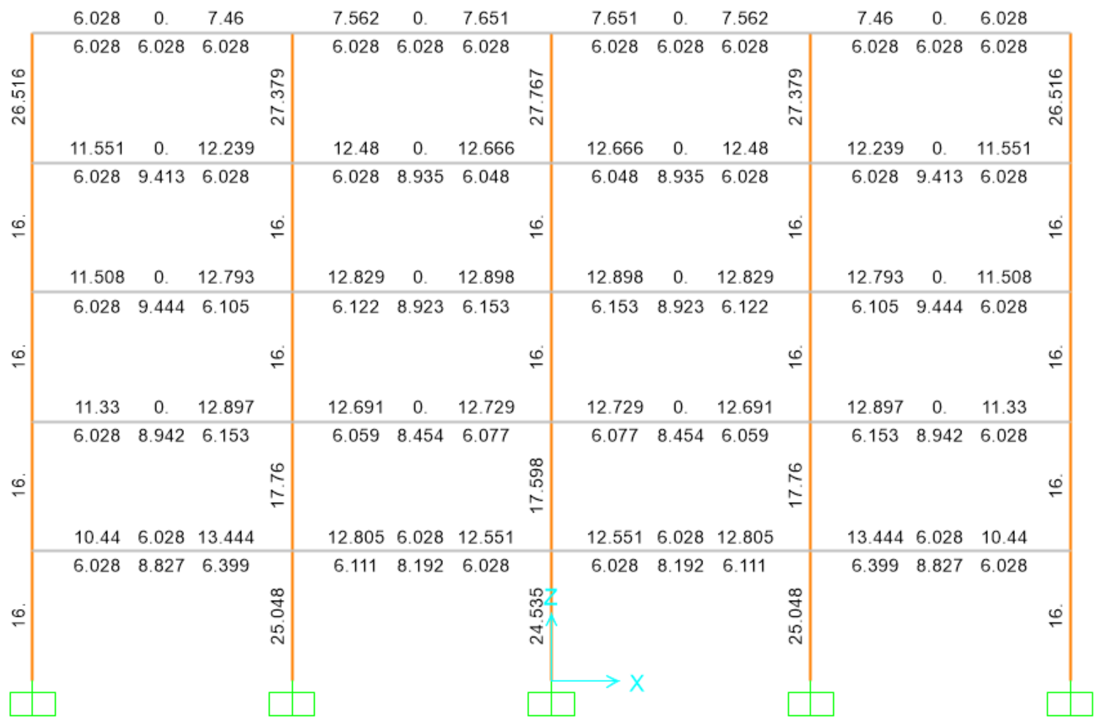


Figure 63 Rebars required in the external longitudinal frames.

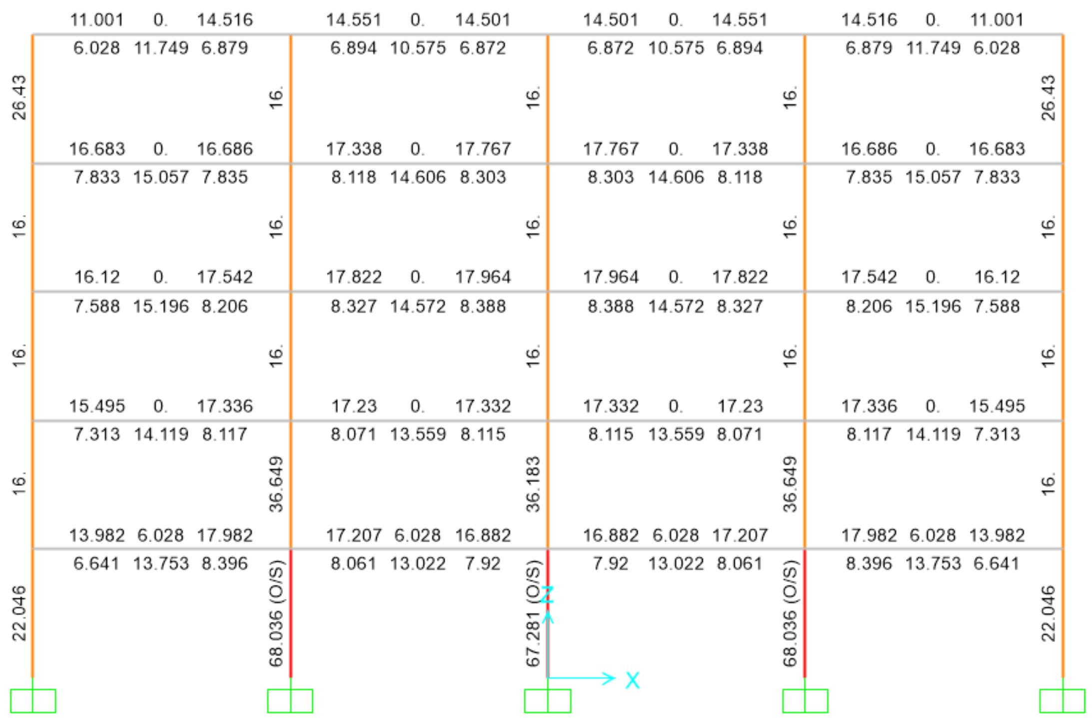


Figure 64 Rebars required in the central longitudinal frames, with the most loaded columns.

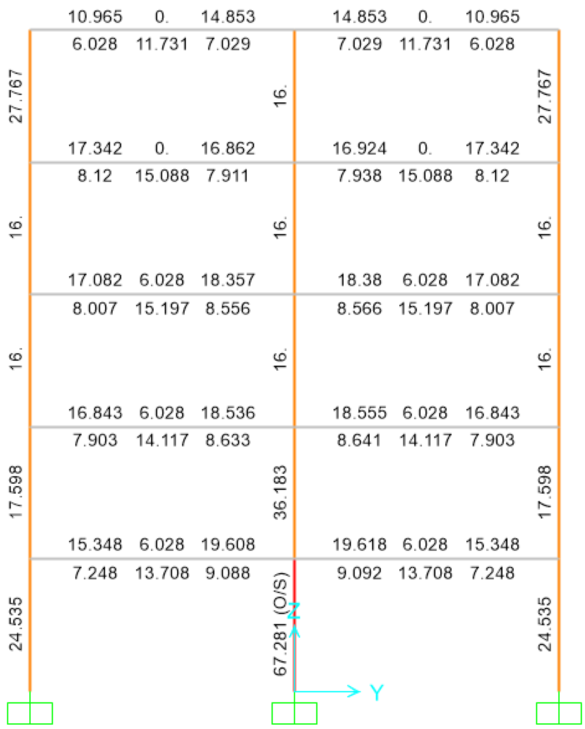


Figure 65 Rebars required in the central transversal frame, with the most loaded beams.

As it is possible to notice in *figures 64 and 65*, some columns are displayed in red since the amount of rebars required in them is higher than the maximum allowed by normative. It was decided to neglect this problem and dimension the columns' reinforcement according to elements that are less solicited than those columns. The reason was always to simulate a design in the 1970s that was not perfect and was referred to different methodologies from the limit state required by the current Eurocodes.

Then, the elements' detailing in terms of steel reinforcement was defined, in an analogous way as in the Italian structure. It might be useful to highlight that in these analyses the properties of the materials adopted in the building are the characteristic ones, since it is a dimensioning, as if it was done before the construction. Instead, assessing the behavior of the existing structure at the current time, the average properties of the materials will be considered, as prescribed by Eurocode 8-3 [71]. Starting from the longitudinal rebars:

Element	Cross-section [cm^2]	Distance from node	Top rebars area required [cm^2]	Top rebars configuration (diameter in mm)	Top rebars area adopted [cm^2]
Beams	50x30	x = 0 cm	19,62	7 Ø20	21,99
		x = 100 cm	6,028	3 Ø16	6,032
		x = 300 cm	6,028	3 Ø16	6,032

Element	Cross-section [cm²]	Distance from node	Bottom rebars area required [cm²]	Bottom rebars configuration (diameter in mm)	Bottom rebars area adopted [cm²]
Beams	50x30	x = 0 cm	9,09	5 Ø16	10,05
		x = 100 cm	6,028	3 Ø16	6,032
		x = 300 cm	15,197	5 Ø20	15,71

Element	Cross-section [cm²]	Distance from node	Longitudinal rebars area required [cm²]	Rebars configuration (diameter in mm)	Longitudinal rebars area adopted [cm²]
Columns	40x40	All along the element	36,65	8 Ø24	36,19

And regarding the transversal rebars, the same assumptions made in the Italian case were adopted for both vertical and horizontal elements:

Element	Cross-section [cm²]	Range of distance from node	Transversal rebars required [cm²/cm]	Transversal rebars configuration (diameter in mm)	Transversal rebars adopted [cm²/cm]
Beams	50x30	0 – 100 cm	0,145	Ø8/20 cm	0,025
		100 - 200 cm	0,127	Ø8/25 cm	0,020
		200 - 300 cm	0,040	Ø8/40 cm	0,013

and Ø8/50 cm for the columns in height.

Finally, the verifications on the columns were performed with the software, while the checks on the beams were carried out manually dividing the required rebars by their designed value, as it was done for location 1.

Again, the transversal reinforcement required by SAP2000 was unreasonable for an existing building and it was set to a value that does not satisfy the requirement but that might be more realistic. In the end, also the German case elements are completely defined and can be checked against the current seismic loads for Aachen.

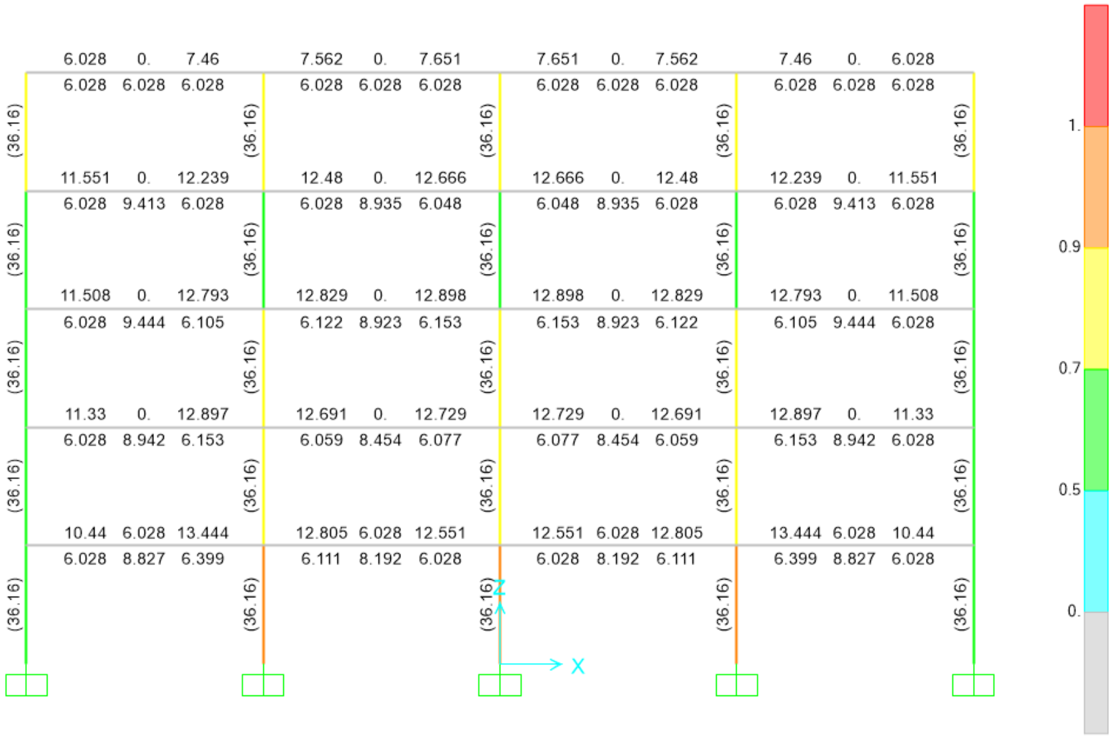


Figure 66 Check of demand/capacity ratio for columns in external longitudinal frames.

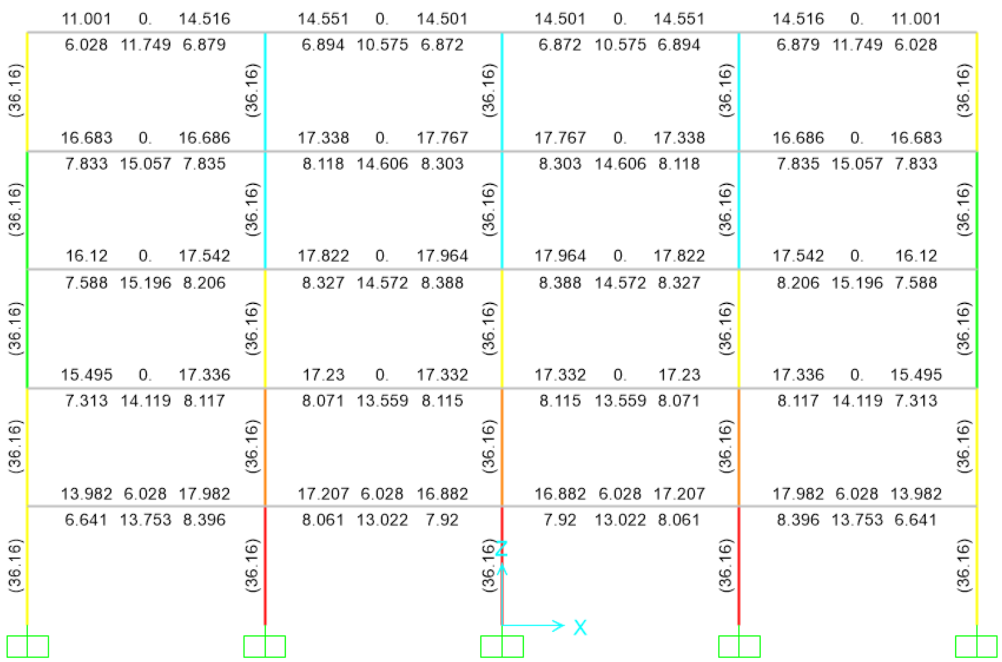


Figure 67 Check of demand/capacity ratio for columns in the central longitudinal frame.

As usual, for the beams the checks were conducted manually with requirement/adoption of reinforcement ratio.

Element	Cross-section [cm²]	Distance from node	Top rebars area required [cm²]	Top rebars area adopted [cm²]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	19,62	21,99	0,892
		x = 100 cm	6,028	6,032	0,999
		x = 300 cm	6,028	6,032	0,999

Element	Cross-section [cm²]	Distance from node	Bottom rebars area required [cm²]	Bottom rebars area adopted [cm²]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	9,09	10,05	0,904
		x = 100 cm	6,028	6,032	0,999
		x = 300 cm	15,197	15,71	0,967

4.3 Structural verification against current seismic loads

In the following paragraphs the study goes on with the verification of the dimensioned existing structures against the current seismic loads, using the modal response spectrum analysis.

To do so, it is necessary to define the response spectra for the two different locations, and then apply them using the methodology discussed in {3.3}. The checks are expected to be non-verified, even if the analyses are conducted on the structure considering the average properties of the materials.

Such assumption is due to the Eurocode regarding seismic analysis of existing structures and the properties should be also divided by the confidence factor, depending on the grade of knowledge of the building under study. Since the fictitious structures were dimensioned and the materials were directly assumed during the research, the confidence factor used is the maximum possible, equal to 1. For this reason, the analyses are carried out simply using the average properties of the materials adopted.

But first, it is necessary to investigate the modes of vibration of the two structures located in Italy and in Germany respectively.

4.3.1 Modal analysis and vibration modes for both locations

Before proceeding with the modal response spectrum analysis, it was necessary to identify the vibration modes. The first three modes of vibration are often the most influential on the analysis.

Since the structure has a fixed geometry in which there are four longitudinal spans and two transversal spans, all with the same distancing between the columns, it is expected to have less overall stiffness in the transversal direction.

This was confirmed by the modal analyses, in which the first mode of vibration reflects a transversal displacement while the second is related to longitudinal displacements. As it often happens, the third mode of vibration was a torsional one.

The observations are valid for both the locations considered in the study.

The first three vibration modes for the Italian location are summarized in the following table and showed by the next pictures.

Vibration mode	Type of vibration	Vibration period [s]	Vibration frequency [Hz]
1	Transversal displacement	0,81	1,24
2	Longitudinal displacement	0,76	1,32
3	Torsional displacement	0,71	1,41

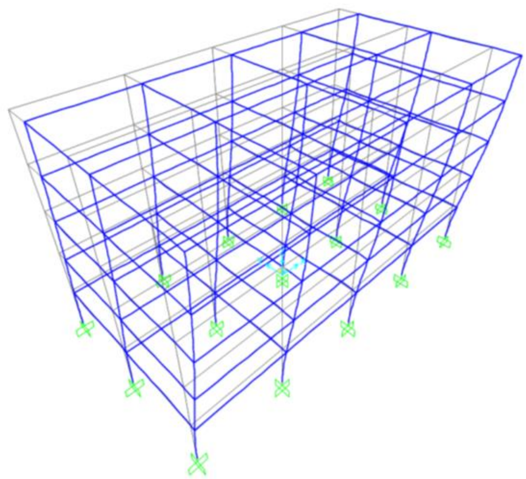


Figure 68 First mode of vibration, Italian

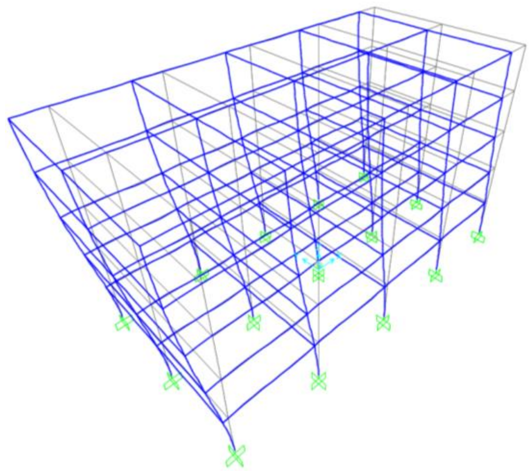


Figure 69 Second mode of vibration, Italian case

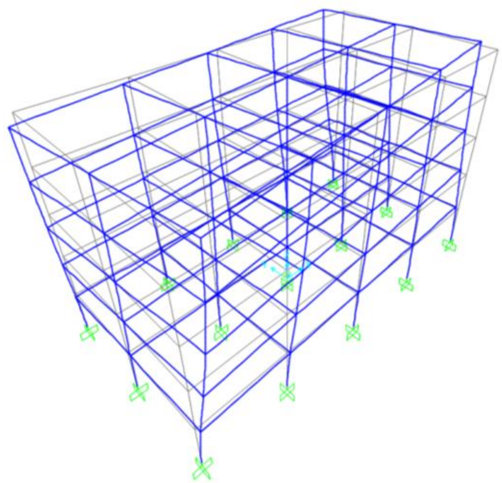


Figure 70 Third mode of vibration, Italian case

Instead, in the German case study the results are:

Vibration mode	Type of vibration	Vibration period [s]	Vibration frequency [Hz]
1	Transversal displacement	0,85	1,18
2	Longitudinal displacement	0,8	1,25
3	Torsional displacement	0,74	1,34

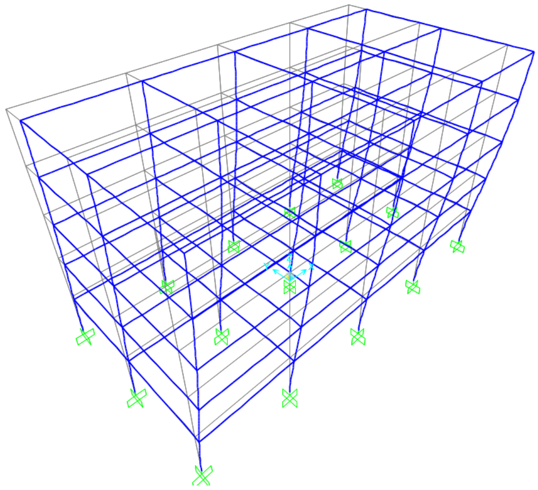


Figure 71 First mode of vibration, German

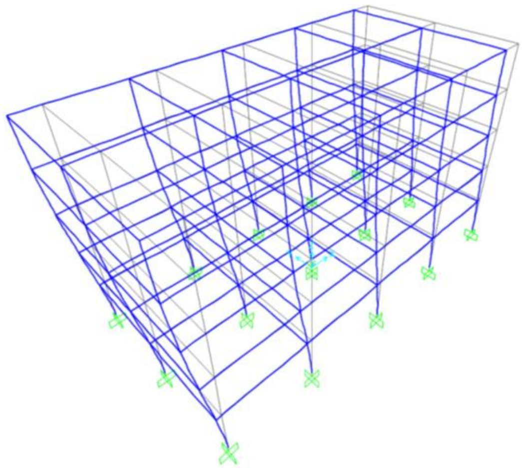


Figure 72 Second mode of vibration, German case

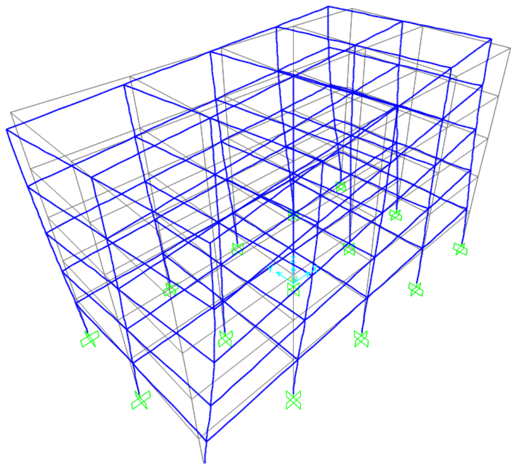


Figure 73 Third mode of vibration, German case

It can be noticed that in the structure dimensioned in Aachen the periods are higher. Such a result was expected since the columns have a reduced cross-section compared to the structure dimensioned in Zambrone, with a deriving lower stiffness of the overall structure.

4.3.2 Response spectra in the two locations

The response spectra were defined in the two locations. As already mentioned, in the German case the spectrum is a type B spectrum, while in the Italian case a type A spectrum (due to the higher seismicity of the country) was defined according to the Italian normative NTC2018 [85].

Starting from Aachen, the peak ground acceleration was determined with the use of some online maps by Dlubal [86]. It indicated a value of 1,06 m/s^2, which could be also described as 0,106g. For the study, an even higher value was implemented: $a_{gR} = 0{,}12g = 1{,}2m/s^2$.

According to some maps derived by the German national annex of Eurocode 8 [87], the soil class was assumed as C-T, where the two letters indicate characteristics of shallow and deep soil strata. C indicated a bedrock with alterations constituted by loose rocks, with dominant shear wave velocities between 150 m/s and 350 m/s, while T is used for shallow sedimentary basins and transition zones.

Compared to the definition of the response spectrum by the Eurocode, there are some differences due to the German national annex, as it happened for the soil class. Here is an example of how the spectrum should be defined in Germany:

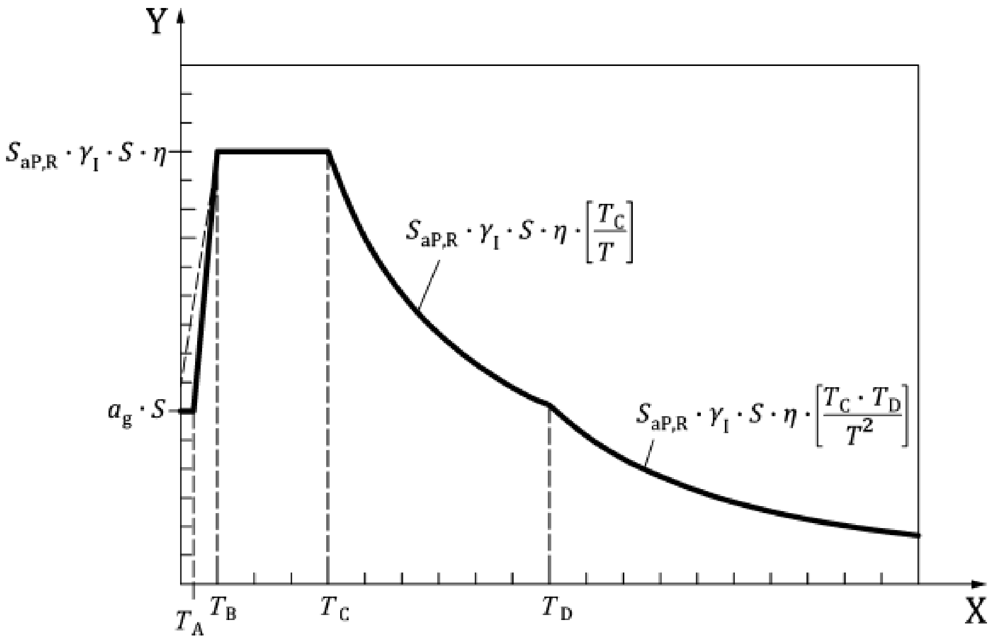


Figure 74 Example of response spectrum according to German national annex [87].

Therefore, it is necessary to determine the parameter $S_{aP,R}$ with the formula

$$S_{aP,R} = 2{,}5a_{gR} = 3\ m/s^2 \qquad (49)$$

Then, using the value in some tables it is possible to determine the soil parameter S, which in this case is equal to $S = 1{,}1$.

Also the periods T_B, T_C, T_D can be extracted by tables for the return period of 475 years, depending on the soil classification. In the case study they can be assumed as:

$T_B = 0{,}1;\qquad T_C = 0{,}4;\qquad T_D = 2{,}0.$

The parameter γ_1 is related to the use of the building and its strategical value. In the investigated structure the building has a normal destination of use, with a subsequent category II and a value of $\gamma_1 = 1$.

Finally, the factor η can be taken as 1 for the elastic response spectrum, or $\eta = 1/q$ in the application of the q-method, as it will be done for the research.

It is relevant to recall that the behavior factor q is a choice of the designer, considering the ductility expected for the structure. It can also be determined with the use of a pushover analysis, but as a first-approach a value of 1,5 seems to be reasonable for a reinforced concrete moment resisting frame located in Germany. Such value will decrease the entity of the actions implemented with the modal response spectrum analysis, by reducing the accelerations related to the periods in the response spectrum.

In the end, the design spectrum to be used in the modal response spectrum analysis for the structure placed in Aachen was obtained. It can be also compared to the elastic response spectrum, which is 1,5 times greater than the design spectrum.

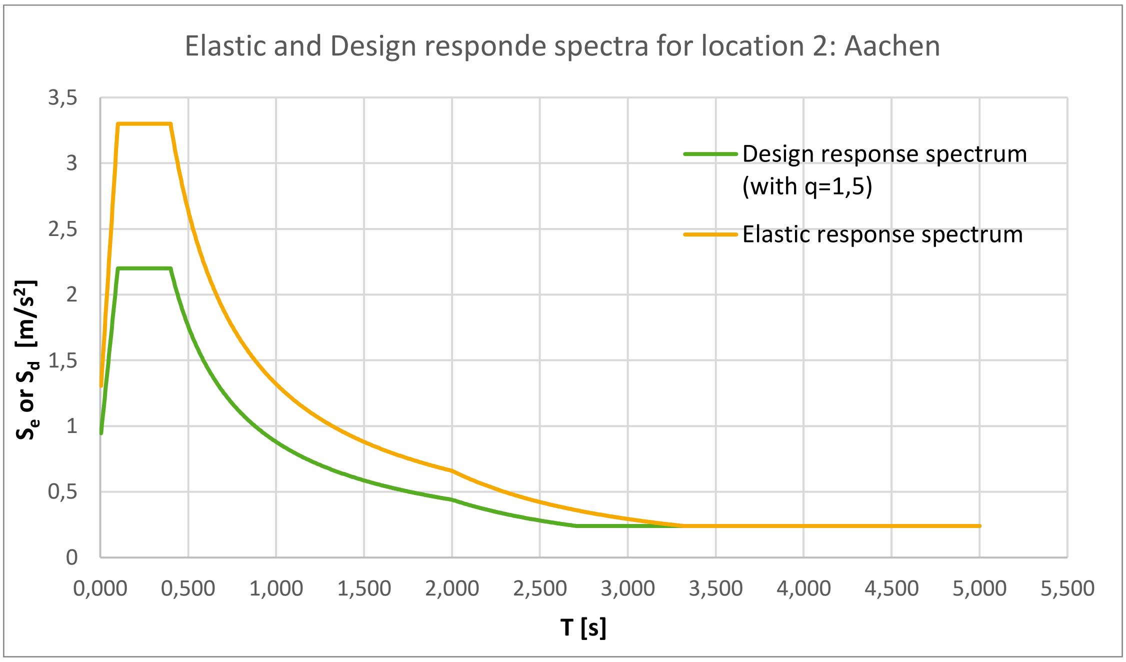


Figure 75 Elastic and design response spectra applied in the verification of the German structure with the modal response spectrum analysis

It can be noticed how for very high periods the response spectra have a minimum threshold, equal to the peak ground acceleration a_{gR} multiplied by a coefficient whose recommended value is $\beta_0 = 0{,}2$. Therefore, the minimum value of the spectra is 0,24 m/s^2.

Going on with the location 1, Zambrone, the peak ground acceleration was extracted by the maps published by the National Institute for Geophysics and Vulcanology (INGV) [88]. Since the geographical area is the south of Italy, the value of the reference PGA is quite high:

$$a_{gR} = 0{,}24g = 2{,}4\, m/s$$

It can be multiplied by the importance factor, which is also in this case $\gamma_1 = 1$, obtaining the peak ground acceleration used for the definition of the response spectrum $a_g = 2{,}4\, m/s$.

Then, apart from the PGA, two other parameters are related to the site considered. Firstly, the maximum value of amplification factor of the horizontal spectral acceleration $F_0 = 2{,}42$

and secondly the start period of the constant velocity proportion of acceleration spectrum $T_C^* = 0{,}365$.

Once the soil category is determined, C in the case study (deposits of medium-densified coarse-grained soils or medium-consistent fine-grained soils, with equivalent shear velocities between 180 m/s e 360 m/s), it is possible to calculate other parameters useful in the definition of the spectrum. Deriving from T_C^*, the parameter $C_c = 1{,}05\ (T_C^*)^{-0{,}33} = 1{,}464$ (formula related to the particular soil class) is used for the determination of the period

$$T_C = T_C^* C_c = 0{,}534\ s \qquad (50)$$

From which also the period $T_B = T_C/3 = 0{,}178\ s$ is obtained.

Instead, $T_D = 4a_g + 1{,}6 = 4 \cdot 0{,}24 + 1{,}6 = 2{,}56\ s$

where the peak ground acceleration is expressed with reference to the gravity acceleration.

The response spectrum is defined by the following formula in the Italian National Annex (which corresponds to the Italian normative NTC2018):

$$0 \leq T \leq T_B \ : \ S_e(T) = a_g S \eta F_0 \left[T/T_B + \frac{1}{\eta F_0}(1 - T/T_B) \right] \qquad (51)$$

$$T_B \leq T \leq T_C : \ S_e(T) = a_g S \eta F_0 \qquad (52)$$

$$T_C \leq T \leq T_D : \ S_e(T) = a_g S \eta F_0 [T_C/T] \qquad (53)$$

$$T_D \leq T : \qquad S_e(T) = a_g S \eta F_0 [T_C T_D / T^2] \qquad (54)$$

Therefore, the only parameter left is the one related to soil S, which can be calculated as the product between two coefficients:

- the stratigraphic coefficient, whose formula depend on the soil class. For class C: $S_s = 1{,}7 - 0{,}6 F_0 a_g = 1{,}7 - 0{,}6 \cdot 2{,}42 \cdot 0{,}24 = 1{,}35$
  (where a_g is expressed with reference to g);
- the topographic coefficient assumed as $S_s = 1$ in normal conditions.

Consequently, the soil parameter is $S = S_s S_t = 1{,}35$

Finally, the elastic response spectrum can be defined. The design response spectrum is obtained dividing the elastic one by the behavior factor q, which was fixed as q=3, considering a higher ductility for the building located in Italy compared to the structure dimensioned in Aachen.

The comparison of elastic and design response spectra, considering a return period of 475 years, is shown in the following graph.

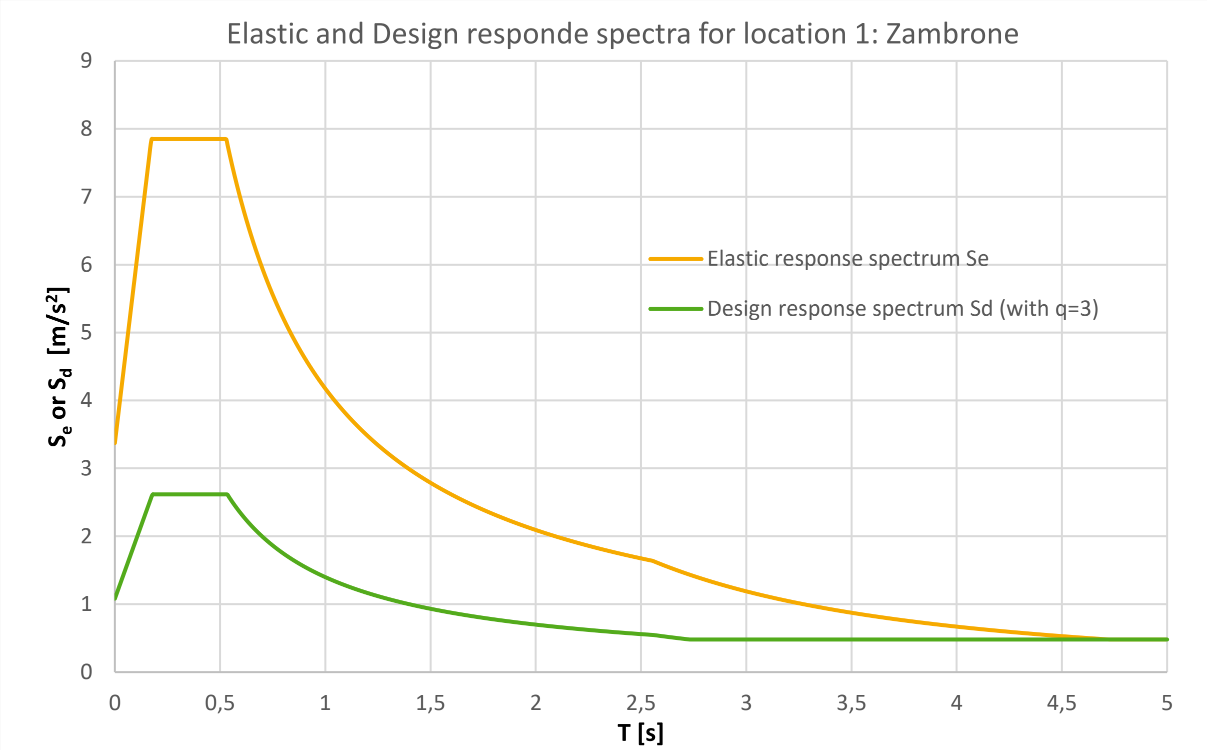


Figure 76 Elastic and design response spectra applied in the verification of the German structure with the modal response spectrum analysis

Comparing the spectra for the two locations, it is shown how the seismic actions in the south of Italy are way bigger than in Aachen, which is already one of the most seismic areas in Germany. In fact, the maximum accelerations for the elastic spectra are 3,3 m/s^2 in the German location, against the 7,85 m/s^2 for Zambrone.

Then, applying the respective behavior factors, the design spectra are more similar. Anyway, the Italian design spectrum is still higher, even with a q equal to two times the behavior factor for the German case. The maximum acceleration values are 2,62 m/s^2 and 2,2 m/s^2 respectively for location 1 (Italy) and location 2 (Germany).

Both the spectra are referring only to return period of 475 years, related to the verifications at the Significant Damage limit state. Indeed, the necessity of retrofitting will be investigated only with reference to that limit state. Instead, the retrofitting measures efficiency will be investigated on three different limit states: Near Collapse, Significant Damage and Damage Limitation, respectively coupled with 2475 years, 475 years and 225 years return periods.

4.3.3 Verifications against current seismic loads in location 1: Zambrone (Italy)

Applying the modal response spectrum analysis on the existing structure previously dimensioned, using the design response spectrum just defined, it is possible to check the elements of the structure. The analysis on SAP2000 was executed using the setting "reinforcement to be checked" in the definition of the columns' cross-section.

For what concerns the beams, such elements will be again verified comparing manually the required amount of reinforcement with the rebars provided during the design of the existing structure.

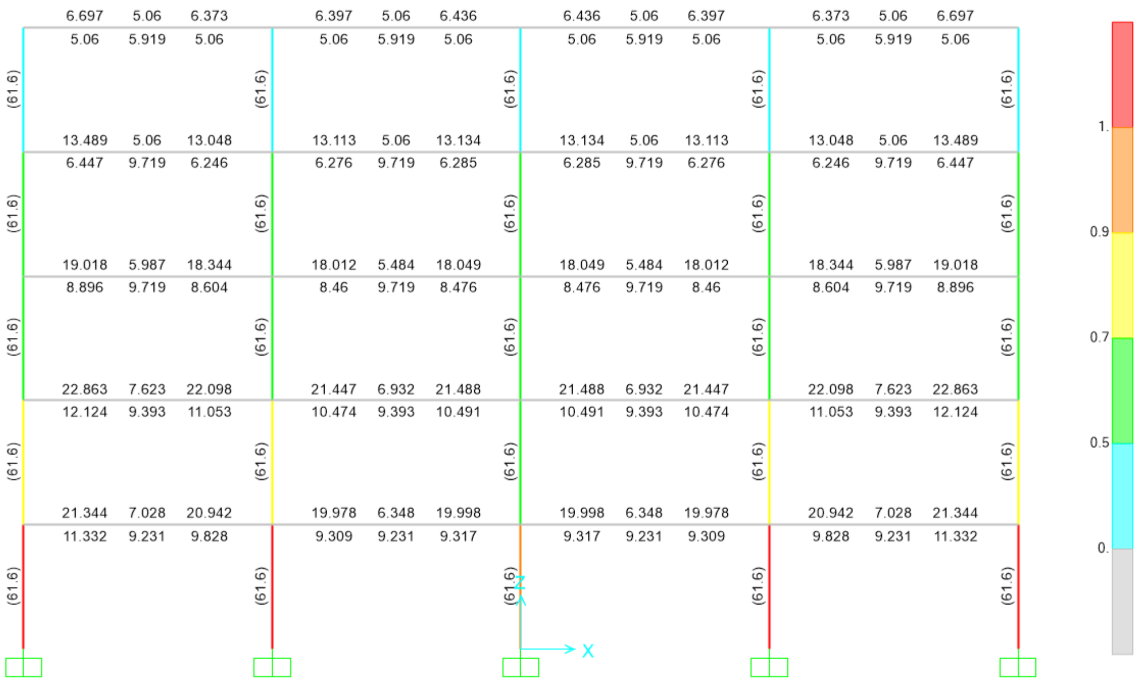


Figure 77 Check of demand/capacity ratio for columns in external longitudinal frames, required amount of rebars in the beams (in cm[2]).

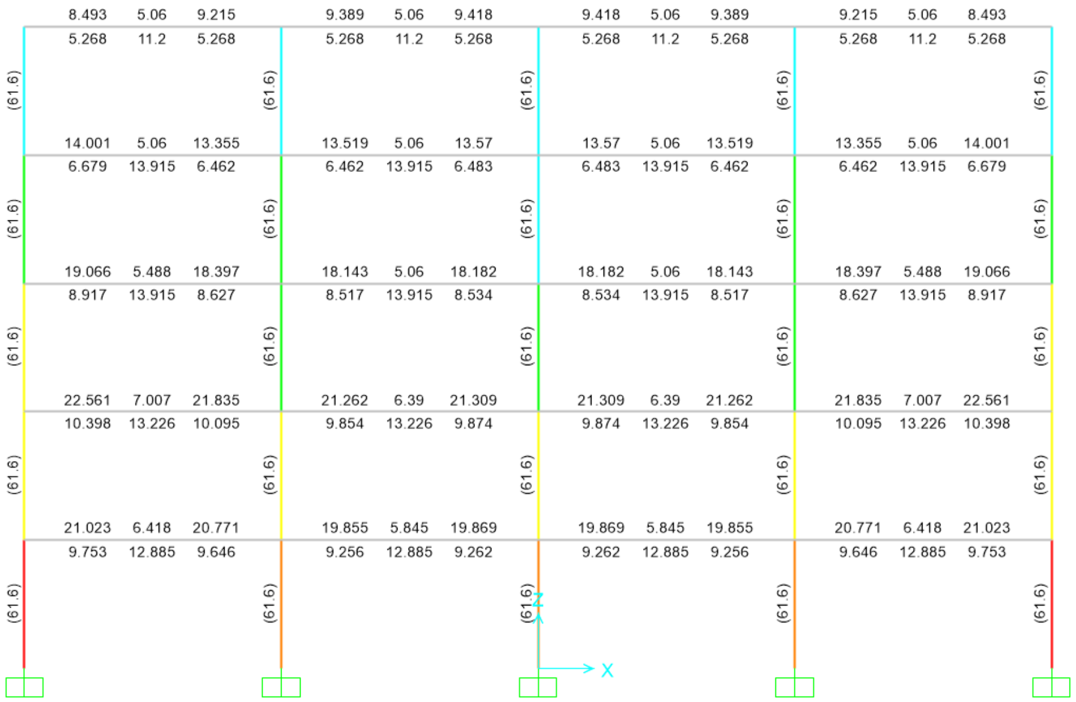


Figure 78 Check of demand/capacity ratio for columns in the central longitudinal frame, required amount of rebars in beams (in cm[2]).

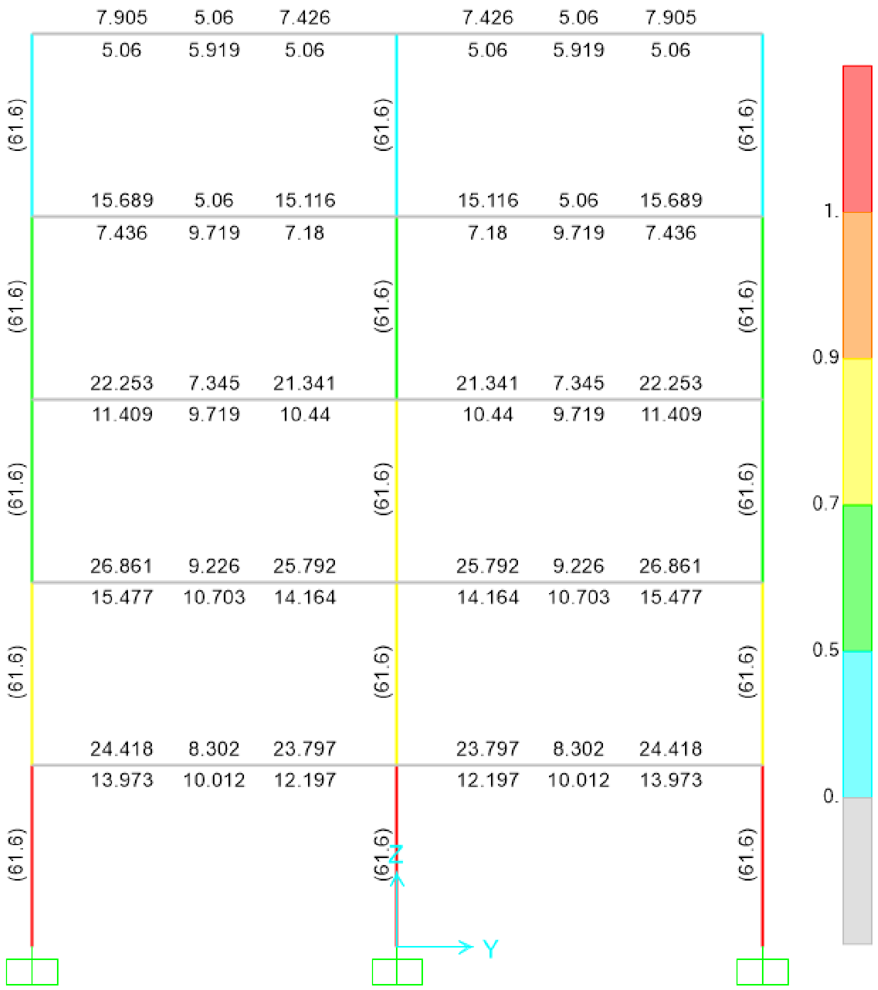


Figure 79 Check of demand/capacity ratio for columns in the most external transversal frames, maximum amount of required rebars in the beams (in cm^2, on the second floor).

As clearly shown by the colors in the model, ten columns on the base floor do not satisfy the requirements on the demand/capacity ratio. The most stressed elements are the one in the corners, where the demand reaches a value equal to 1,095 times the capacity.

This would be already enough for assessing the necessity for retrofit of the structure against seismic loads.

In addition, many beams would require a reinforcement that exceeds the one provided during the dimensioning of the existing structure. Extracting the necessary tables from the model and identifying the highest values of required rebars in the cross-sections of the beams at various distances from the nodes, the ratios between required and implemented reinforcement areas (for the most critical elements) were obtained:

Element	Cross-section [cm²]	Distance from node	Top rebars area required [cm²]	Top rebars area adopted [cm²]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	26,86	18,85	1,425
		x = 100 cm	14,28	10,05	1,421
		x = 300 cm	5,83	6,032	0,967

Element	Cross-section [cm²]	Distance from node	Bottom rebars area required [cm²]	Bottom rebars area adopted [cm²]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	15,48	14,07	1,100
		x = 100 cm	12,57	12,06	1,042
		x = 300 cm	13,91	21,99	0,633

An observation can be drawn about the fact that the in the mid-span cross sections the beams are still verified: such part of the beam is the one solicited especially by the gravity loads and thereby the reinforcement depends mainly on the application of fundamental load combination. Considering that the model that undergoes these checks has the average materials' properties, it is reasonable to obtain lower requirements in those parts, compared to the ones in the dimensioning using characteristic properties.

On the other hand, even if the strength of the materials is higher, it is not enough to face the greater loads due to the application of the current response spectrum. Hence, the verifications that are not satisfied indicate the necessity for retrofitting.

4.3.4 Verifications against current seismic loads in location 2: Aachen (Germany)

The application of the modal response spectrum for the second location gave some results that were rather unexpected, at least for what regards the vertical elements. Indeed, from a first glance at the analysis results on the model, the whole structure seems to be verified against the application of the design response spectrum. This is due to the use of average properties in the study of existing structures.

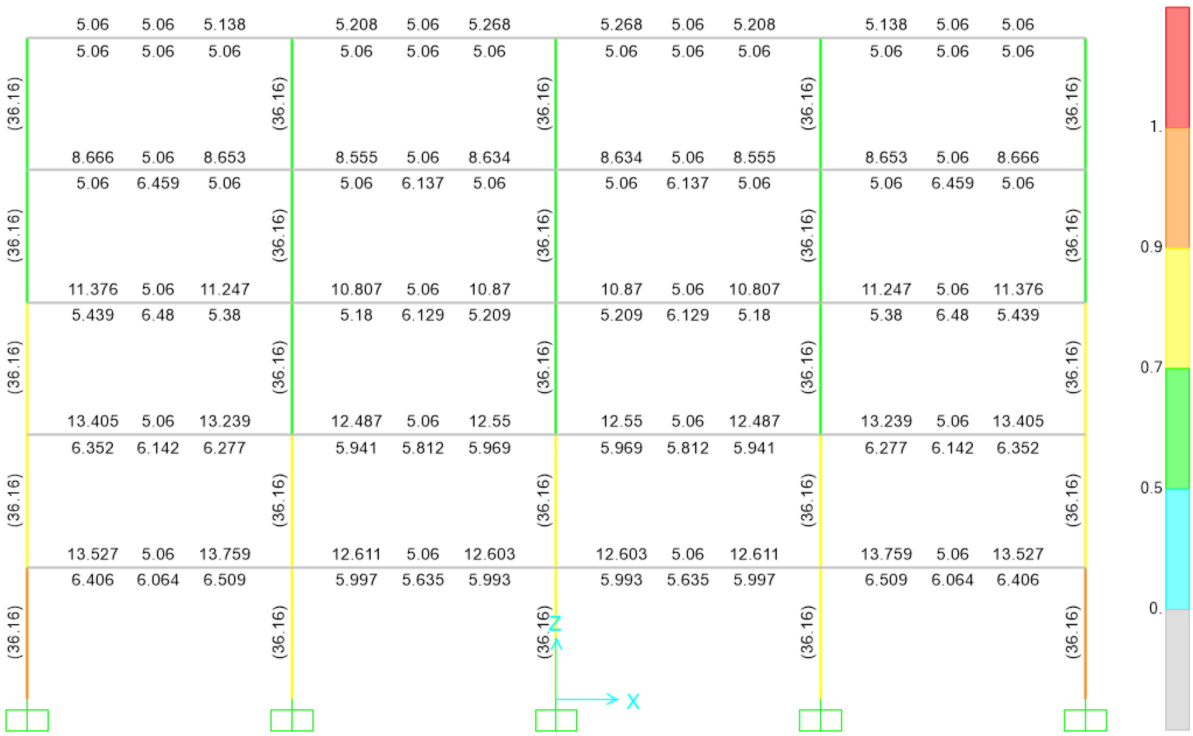


Figure 80 Check of demand/capacity ratio for columns in the external longitudinal frames, required amount of rebars in beams (in cm^2).

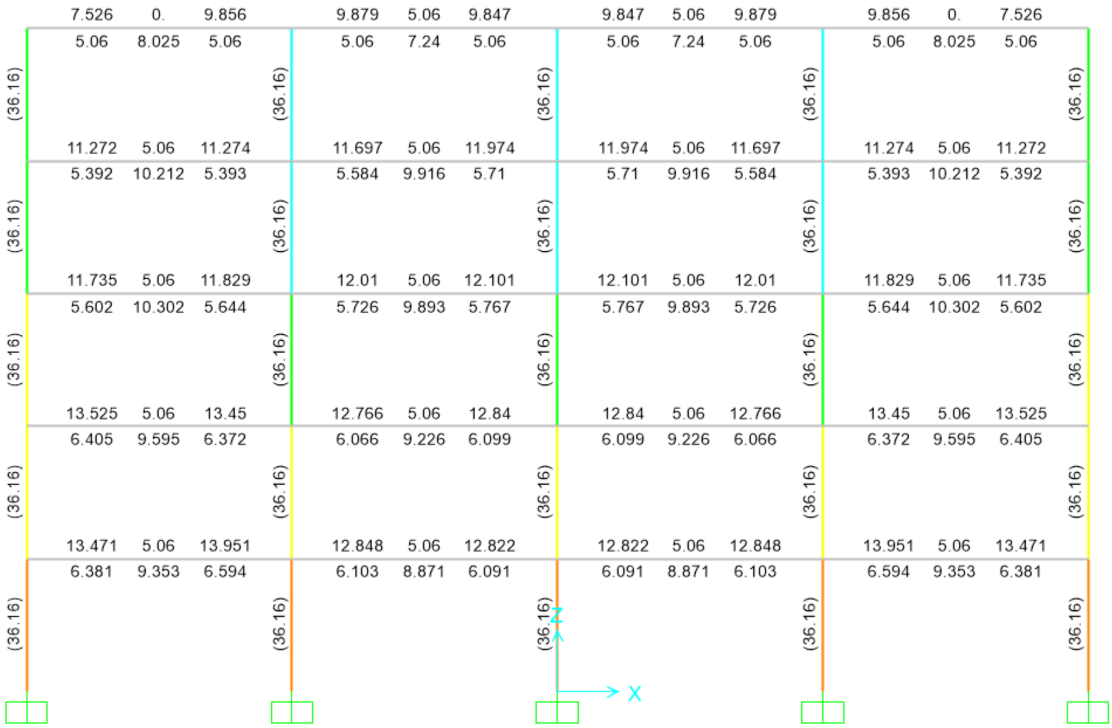


Figure 81 Check of demand/capacity ratio for columns in the central longitudinal frame, required amount of rebars in beams (in cm^2).

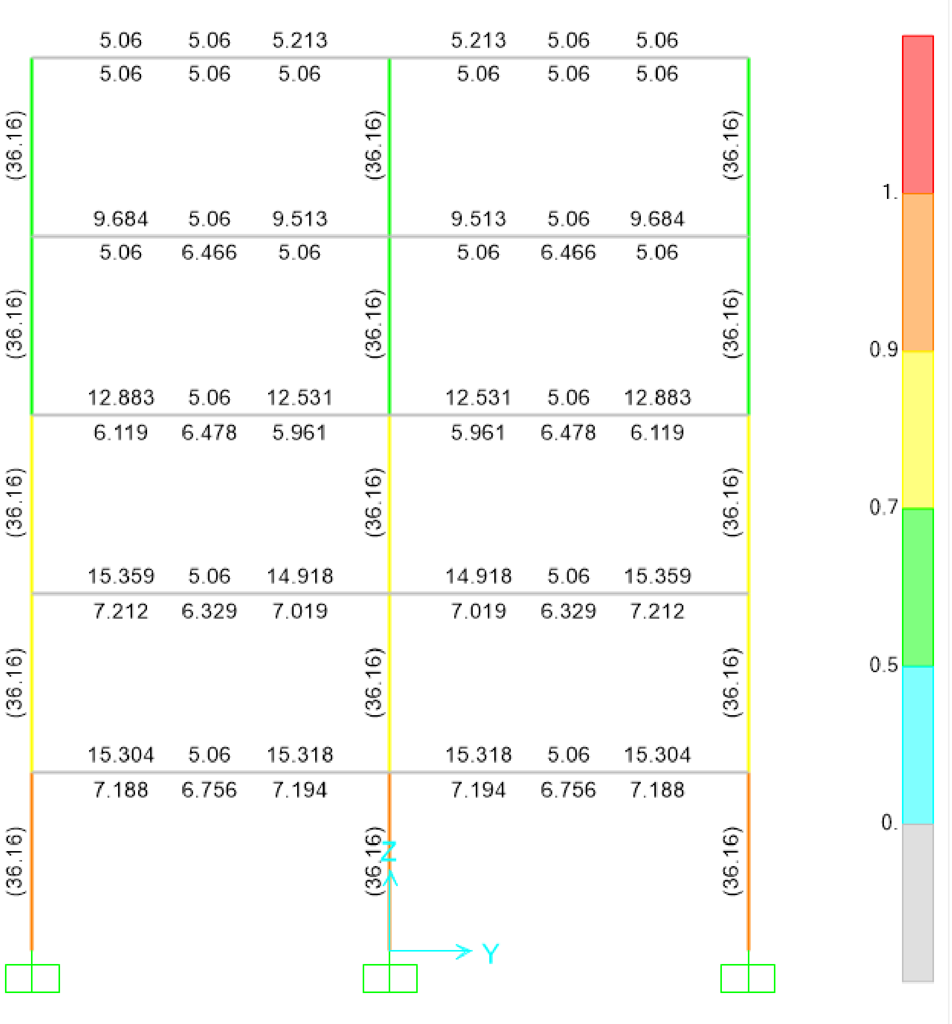


Figure 82 Check of demand/capacity ratio for columns in the most external transversal frames, maximum amount of required rebars in the beams (in cm^2, on the second floor).

Meanwhile, not all the verifications are satisfied in the case of the longitudinal reinforcement that is required for the beams with the application of the current seismic loads:

Element	Cross-section [cm^2]	Distance from node	Top rebars area required [cm^2]	Top rebars area adopted [cm^2]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	15,36	21,99	0,698
		x = 100 cm	7,52	6,032	1,247
		x = 300 cm	0,00	6,032	0,000

Element	Cross-section [cm^2]	Distance from node	Bottom rebars area required [cm^2]	Bottom rebars area adopted [cm^2]	Rebar areas ratio required/adopted
Beams	50x30	x = 0 cm	7,21	10,05	0,717
		x = 100 cm	7,08	6,032	1,174
		x = 300 cm	10,30	15,71	0,656

In this case, the newly adopted seismic loads are not impacting excessively on the beams but they are still causing some unverified checks. In fact, it looks like the seismic actions enlarged the areas in proximity of the nodes that are subjected to relevant stresses, contrarily to the effect registered during the dimensioning of the existing structure with the only fundamental load combination. Consequently, the low amount of rebars at 1 m distance from the nodes is not sufficient anymore for resisting the applied stresses.

By consequence, a **retrofit intervention is necessary** also in the German case study, despite the columns where verified, at least against the load combination used for ductile members.

4.4 Design of retrofit interventions

At this point, the need for structural retrofit is established in both the locations. Following the analyses run for the verifications against the current seismic loads, it is possible to dimension the concrete shear walls to be inserted into the existing structures.

For both the Italian and German case study the geometrical characteristics of the walls are set as: 20 cm thickness, 2 m width and 3 m height. What will differ between the two cases, is the amount of reinforcement adopted in the wall's cross-section.

The design and verification of requirements for the concrete shear walls was carried out after the execution of a first analysis with the walls already inserted into the model. In fact, in the dimensioning some values of forces acting on the walls are needed. The preliminary analysis is performed adopting the option "reinforcement to be designed" for the wall's cross-section. The walls were defined in the Italian and German case differently, but in both cases the dimensioning was made just one time. The same wall was applied then also changing the configuration of the retrofit in terms of number of concrete wall moduli and positioning into the existing structure.

A more precise evaluation could impose the iteration of reinforced concrete wall detailing every time the configuration of the retrofit undergoes any change, but this was not done in the study for the sake of simplicity. For the aim of the research, reaching a reasonable dimensioning of the wall module should be sufficient to estimate precisely enough the environmental impact due to the refurbishment with the implementation of RC shear walls.

Differently from the case of the reinforced concrete intervention technology, the Light Timber Frame shear walls are already defined and only need to be placed in the frames of the structure, in order to absorb horizontal loads and stiffen the building.

Considering that the timber intervention is studied in terms of average properties of the materials, as investigated in the laboratory activities of RWTH Aachen, also the reinforced concrete walls will be designed and evaluated considering the average properties of the concrete class C30/37 and of the steel reinforcement B500C. This had to be done to enable a comparison between the two solutions, despite the Eurocode requiring the analysis of new elements with their characteristic features, also in the case of a retrofit.

The evaluation of the performance of the existing structure will be carried out also with the pushover analysis and the N2 method, in addition to the already performed verifications with the modal response spectrum analysis.

4.4.1 Design of concrete shear walls in location 1: Zambrone (Italy)

The procedure for the dimensioning of the concrete shear walls starts with the check of their slenderness. Its limit value is calculated with:

$$\lambda_{lim} = 20ABC/\sqrt{n} \qquad (22)$$

In which $A = 0{,}7$, $B = 1{,}1$ and $C = 1{,}7 - r_m = 1{,}7 - M_{01}/M_{02} = 1{,}28$.

The values of the first order end moments were obtained from an analysis performed with shear walls inserted in all the floors, in four different spans of the structure. Moreover, the selected values were at the base (M_{02}) and at the top (M_{01}) of the ground floor shear wall modulus in the transversal frames, which was the most stressed compared to the wall's moduli applied to the upper floors and in the longitudinal direction.

Then, to calculate the limit slenderness, only the relative nominal force is missing: $n = N_{Ed}/A_c f_{cd} = 0{,}078$

where in the formula of f_{cd} the characteristic value was substituted by the average value:

$f_{cd} = 0{,}85 f_{cm}/\gamma_m = 0{,}85 \cdot 38/1{,}5 = 21{,}53\ MPa$

Consequently,

$$\lambda_{lim} = 20ABC/\sqrt{n} = 69{,}66$$

Instead, the slenderness of the element was calculated through the effective length and the radius of gyration:

$$\lambda = l_0/i \qquad (23)$$

With:

$l_0 = 2l$ in the case of cantilever, which was assumed as it is the most preventive possibility, even if the walls are somehow constrained also at the top;

$i = \frac{d\sqrt{3}}{2}$ in the case of a rectangular cross section, with d equal to the thickness of the wall. In the end, $\lambda = 34{,}64$ was definitely lower than the limit slenderness. Hence, the dimensioning of the shear wall can be done without accounting for second order effects.
Then, for the dimensioning it was necessary to calculate the bending moment acting at the base of the structure, considering also the eccentricity of the applied axial action. Such eccentricity was calculated as 1/400 of the wall's effective length. In the end, the product of the axial action by the eccentricity was subtracted to the bending moment acting at the base of the wall.

In the end: $M_{Ed} = M + e_i N_{Ed} = \ 3253{,}18\ kNm$.

Using the charts for the dimensioning of vertical elements taken by the Eurocodes, based on the ratios $M_{Ed}/(bh^2 f_{ck})$ and $N_{Ed}/(bhf_{ck})$. The following graph was adopted to determine the quantity $A_s f_{yk}/(bhf_{ck})$, since the ratio between the thickness of the wall (h=20 cm) and the centroid of bars in half a section (d_2=4 cm) is exactly 0,2.

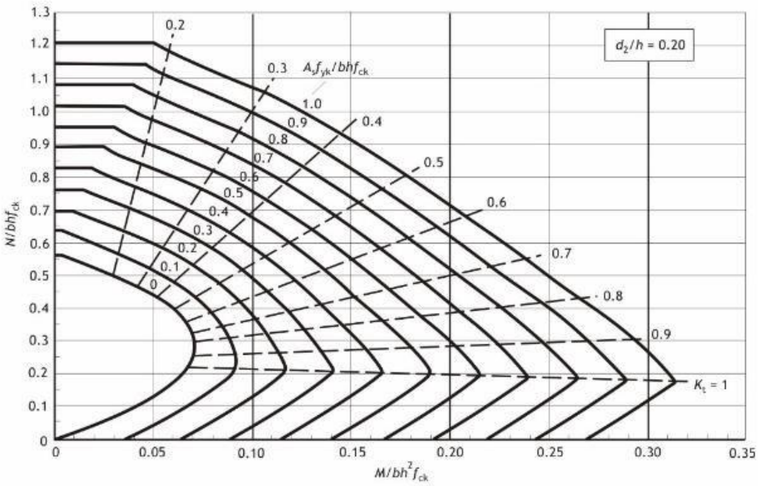


Figure 83 Design chart for columns with d_2/h=0,2, from Eurocode 2 [69]

Finally, the amount of longitudinal rebars in the wall was identified as:

$$A_s = 91{,}20\ cm^2$$

Which satisfies the requirement of $A_s \geq 0{,}002 A_c = 8\ cm^2$.

Then, also taking into consideration the minimum diameter of the reinforcement equal to 12 mm, it was chosen to use 15 longitudinal bars of diameter 20 mm on each face of the wall, for a total of 94,25 cm^2. By consequence, the spacing between longitudinal rebars is 13 cm.

The horizontal reinforcement of the wall was dimensioned as a function of the vertical one. In fact, it should be around 1/4 of the longitudinal rebars.
Also in the horizontal direction, 15 bars were placed on each face of the wall, but in this case with a diameter of 10 mm. The spacing is 20 cm, below the maximum limit of 40 cm.
The assumed nominal cover is 30 mm. No links between the reinforcements are required since the area of vertical rebars (94,25 cm^2) exceeds $0{,}2A_c = 80\ cm^2$.

4.4.2 Design of concrete shear walls in location 2: Aachen (Germany)

In the dimensioning of the reinforced concrete shear walls for Aachen, the same approach already used for the first location is adopted.
Considering different actions derived from the model after the introduction of shear walls, compared to the Italian case, the limit slenderness is changing, but the slenderness of the element is always the same since the geometry was set to be the equal in the two locations.

The slenderness check is verified also in the German case study:

$$\lambda = 34{,}64 \leq 55{,}74 = \lambda_{lim}$$

Then, using the same design chart as in the case study from Zambrone, an area of required steel reinforcement in the vertical direction equal to 48 cm^2 was obtained.
To satisfy this requirement, 12 reinforcement steel bars on each face of the wall were placed, each with a diameter of 16 mm and spaced from the others 15 cm. The total area of the reinforcement was in this way 48,25 cm^2.

Again, the horizontal reinforcement is 1/4 of the longitudinal one. To respect this provision, 12 rebars on each face with a diameter of 8 mm were defined. The spacing between them is 25 cm. The nominal cover is 30 mm also in this case, while the links to keep a good positioning of the reinforcement bars is required since the amount of longitudinal rebars is lower than 0,2 times the concrete cross-section.

4.4.3 Application of shear walls in the existing structure for both locations

The definition of the positions for the retrofitting walls had to consider criteria such as keeping the most symmetry possible in the building, in order to avoid torsional actions due to differences in stiffness in the frames. This restricted the possibilities in terms of intervention, while a facilitating factor was the absence of constraints related to neighboring buildings or particular technologies used for the façades of the building. In addition, the internal distribution of spaces was neglected, considering only the frame structure. These assumptions allowed to place the walls in any frame portal of the building (meaning by portal the space created by two columns and the beam connecting them).

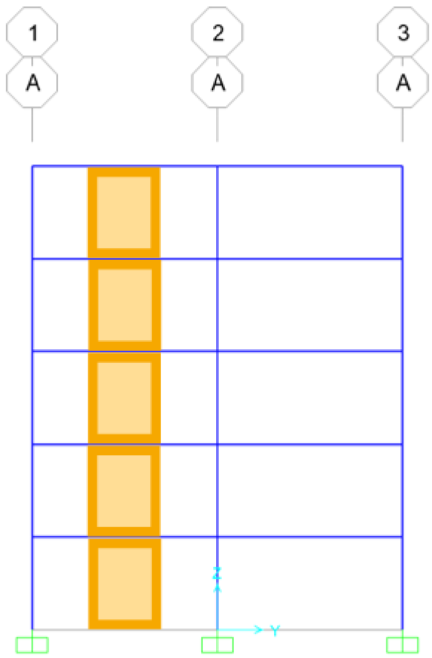


Figure 84 Positioning of shear walls over the five floors in a transversal frame.

The first positioning involved walls placed in four positions of the building, over all the five floors. It was done in the same way in both the locations and with the two retrofit technologies.

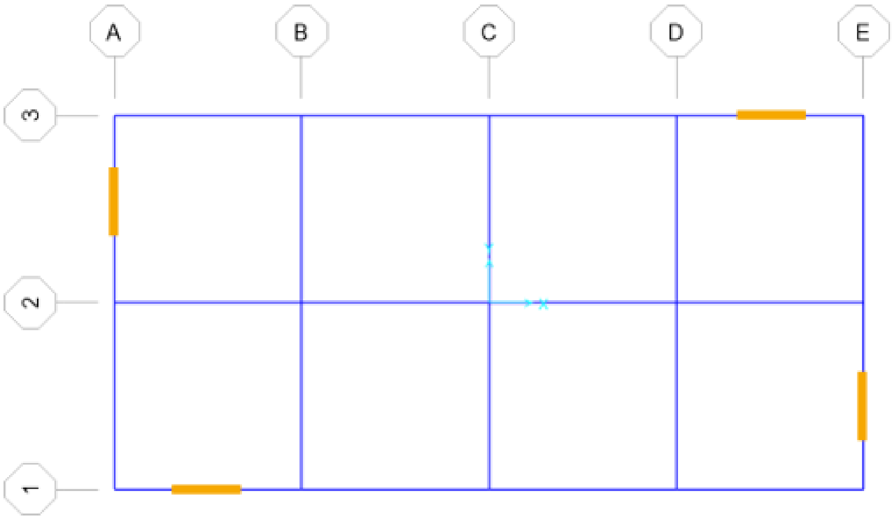


Figure 85 Positioning of the shear walls (yellow segments) in plan view.

Then, in all the investigated cases it was observed that the main problems in terms of stability and plastic hinges formation were encountered in the first three floors of the building. This kind of result was quite expectable since the most stressed parts of the walls are the ones towards the base of the building, which are contrasting the overturning moment due to the horizontal actions like the seismic ones.

Consequently, with the aim of reducing the impact of the intervention under many aspects (i.e.: construction works, economic impact, environmental impact), it was decided to operate with the introduction of **shear walls only on the first three storeys** of the structure, as it is shown in *figure 86*.

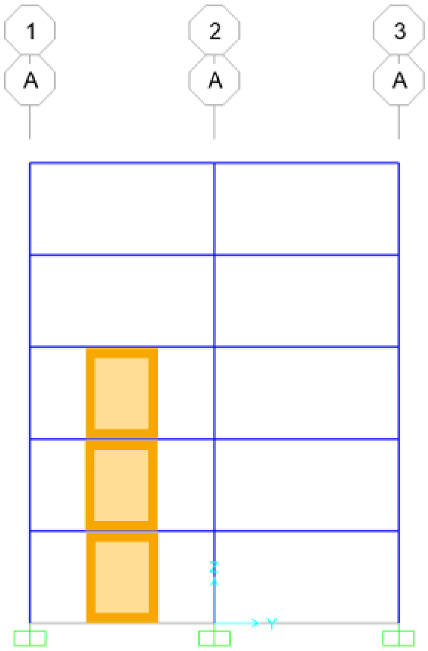


Figure 86 Example of wall configuration on just three floors.

The new configurations adopted, with walls only on the lower floors, were proved to be enough to have acceptable structural resistance.

Going on with the design of the retrofit made for **location 1**, Zambrone, where the seismic loads are much higher, the first analyses clearly stated the need for additional walls, in more than four portals of each considered floor.

For what concerns **reinforced concrete shear walls**, it was decided to double the number of elements involved, occupying eight portals for each of the first three floors, instead of four portals. The representation of this solution is depicted in *figure 87 and figure 88*.

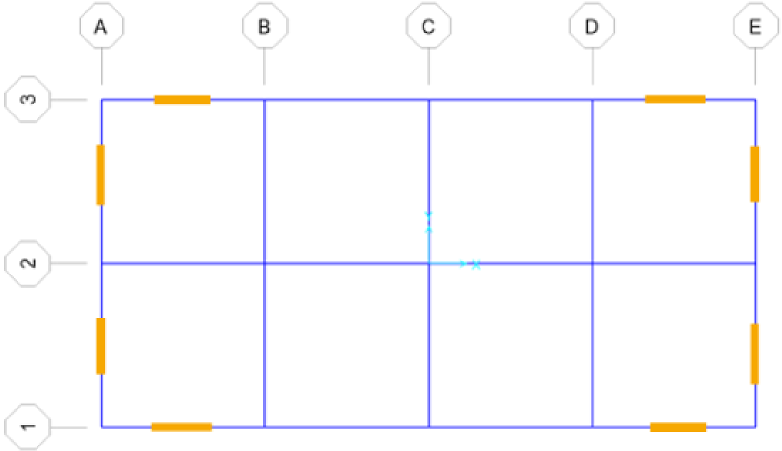


Figure 87 Positioning of reinforced concrete shear walls (yellow segments) in plan view, for the Italian case.

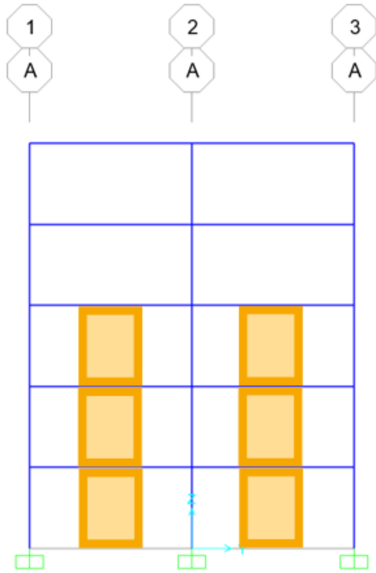


Figure 88 Example of walls configuration on an external transversal frame.

Then, for **light frame timber shear walls**, even a higher number of elements was needed.

In addition, instead using just one element of width 1,25 m, two consecutive power walls where implemented. This application of two moduli together was shown in the description of the shear walls and will be adopted in all the portals in which timber shear walls are positioned for the Italian case.

To have structural responses comparable to the ones given by concrete shear walls, the timber elements with a width of 2,5 m had to be applied into six portals for each direction, in each considered floor. Therefore, two walls are placed in each longitudinal frame, while three of the five transversal frames are equipped always with two walls.

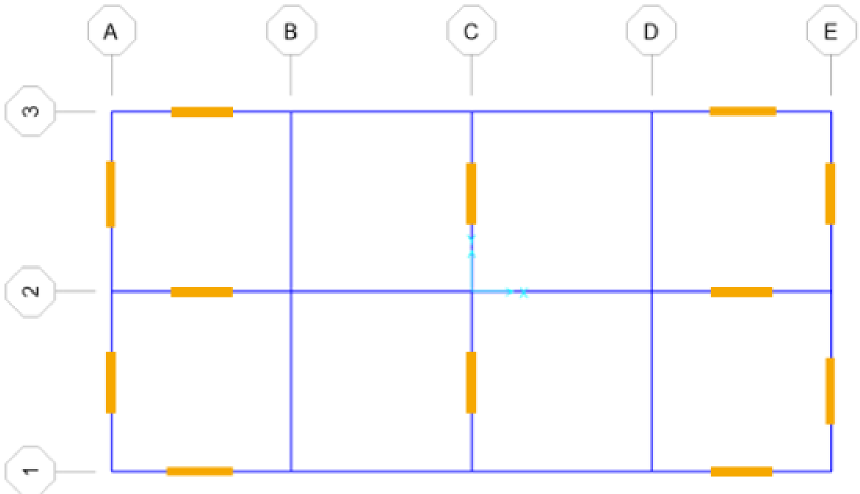


Figure 89 Positioning of light frame timber shear walls (yellow segments) in plan view, for the Italian case.

Passing to the retrofit applied to the German structure, in location 2 the forces to be contrasted are lower than in the Italian case. Apart from the design response spectrum itself that is already advantageous for Aachen compared to Zambrone, the structure is also lighter, thanks to smaller cross-sections in the columns. This results in seismic actions that can be faced more easily by the structure and thereby the extent of the intervention is restricted.

Indeed, for **reinforced concrete shear walls**, the final configuration adopted imposes the presence of elements in only four portals for each of the lower three floors. This positioning is the same adopted in one of the first hypotheses made during the design of the retrofit, as illustrated in *figures 84 and 86* previously reported.

Going on with the **Light Frame Timber shear walls**, the number of walls was doubled compared to the application of the reinforced concrete intervention. The positioning is the same adopted for the Italian case using RC shear walls (*figures 87 and 88*). But differently from the first location, in this case only one timber wall element was necessary, obtaining a width of each wall equal to 1,25 m (instead of 2,50 m as in the Italian structure).

For brevity only the evaluations of the final configurations, with the description of the defined plastic hinges, will be displayed in the thesis. Nevertheless, the process to reach such result was long and included many tests and analyses, with different positions of walls, definition of hinges and setting of parameters for the application of pushover analyses.

4.5 Evaluation of structural results

The study of the retrofit interventions, with the different techniques and in the two locations, was carried out by means of pushover analyses performed separately in the two main directions of the structural frame. Subsequently, after obtaining the performance curves of the various structural configurations, it was possible to evaluate the results in terms of actual capacity, evaluating the performance points for the Significant Damage limit state, thanks to the application of the N2 method. Finally, the outputs were compared with the requirements imposed by structural instability or by the activation state of plastic hinges in the structure. In this way, it was possible to identify whether the failures happened for displacements and base shear forces lower or higher than the values associated to limit states requirements. In the former case, the verification would not be satisfied.

But before diving into the actual process for the verification, it is necessary to identify the properties assigned to the plastic hinges, especially for the horizontal components. They vary considering the diversities associated to the two locations, overall in terms of resistance of the elements depending on the defined amount of rebars.

4.5.1 Definition of plastic hinges

Initially, the hinges inserted in the elements for the application of the pushover analysis were set to be defined automatically by SAP2000. They were placed towards the edges of all the elements composing the structure, for both beams and columns.
But in this way, since the beams reinforcement required is defined after every analysis performed by the software, it was impossible to have hinges in the horizontal elements that reproduced the behavior of the existing elements with a fixed amount of rebars.
To overcome this issue, the hinges for the beams were defined manually into the models, after determining their properties according to the American standard FEMA 273 [76].

Instead, the columns hinges, since the presence of reinforcement was properly defined in the software, were computed automatically by SAP2000 extracting the properties from tables in the standard ASCE 41-13. The same was done for the hinges to be placed in the reinforced concrete shear walls.

The hinges for the vertical elements were defined with three possible degrees of freedom: axial (P), and the moments in the two main directions (M2, M3). For the beams, only the main bending moment (M3) was taken into account for the formation of plastic hinges.

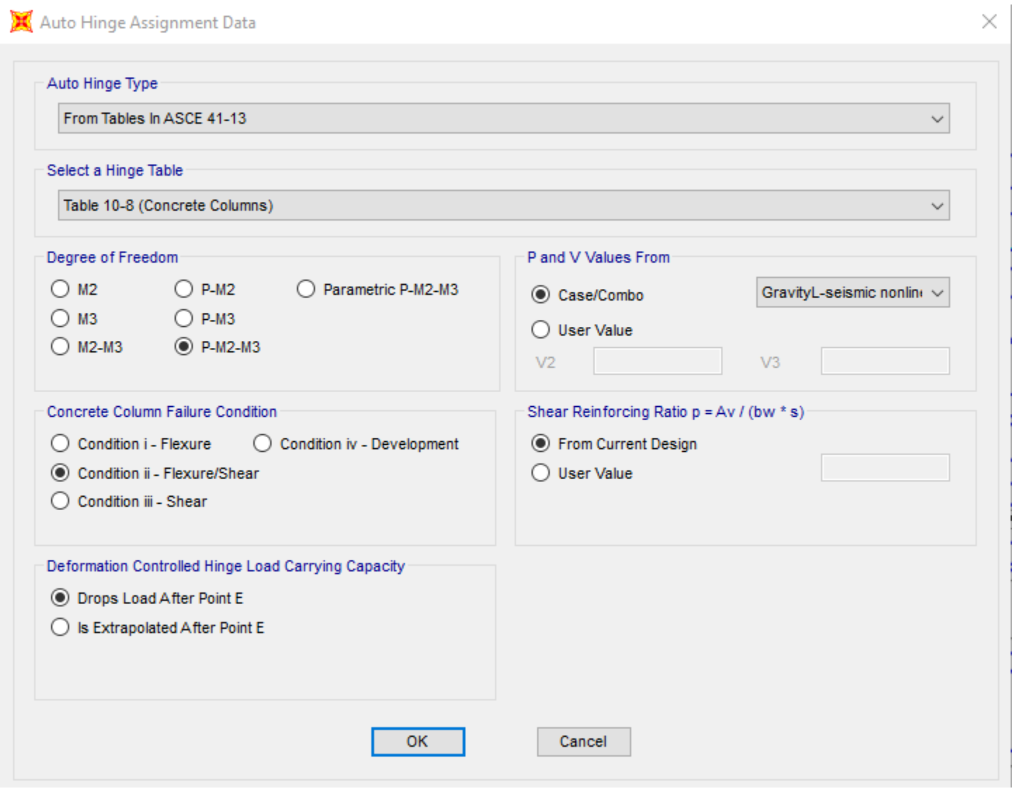


Figure 90 Settings for the definition of plastic hinges for columns and shear walls in SAP2000.

The definition of the moment-rotation curve for the plastic hinges in the case of the beams was conducted identifying the values from a table of the mentioned standard. To do so, different parameters were considered:

- The **ratios between the areas of steel rebars and the area of the concrete section**, respectively considering the reinforcements at the top and at the bottom of the section: ρ and ρ'. It is important to notice that the selected section is the one on the edge of the beam, the most reinforced part will be the top of the beam.
- The **same ratio** but in the case in which a **balanced failure** of the section. So, when the compressed part of concrete reaches its ultimate deformation ($\varepsilon_{u,c}$) to which corresponds the compressive strength (f_c') and at the same time the reinforcement is yielded by tensile stresses ($f_{y,s}$), with the coupled yielding deformation of steel ($\varepsilon_{y,s}$). It can be obtained by:

$$\rho_{bal} = 0{,}85\beta_1 \frac{f_c'}{f_{y,s}} \frac{\varepsilon_{u,c}}{\varepsilon_{u,c} + \varepsilon_{y,s}} \qquad (55)$$

  The factor $\beta_1 = 0{,}8$ is due to the shape assigned to the compressive stresses, identified as a rectangular stress distribution over 80% of the concrete's compressed area. All the values of strength adopted are the average values, since the case study refers to the mean properties of the materials.
- The **resistance of the section against shear actions**, calculated as the sum of the resistance provided by concrete and the one provided by closed stirrups, V.
- The width of the element b_w.
- The **distance** between the **intrados** of the beam and the steel **reinforcement in tension** d.

All the mentioned parameters were used to identify the input values for the tables determining the properties of the moment-curvature curves for the beams' hinges. It is important to notice that such values were calculated transforming all the quantities in the American unit of measures, passing from cm to inches, from kN to lbf (pound-force) and from MPa to psi (pounds per square inch).

Then, in addition to the assumed conforming transversal reinforcement, the entering values in the hinge's definition table are given by:

$$\frac{\rho - \rho'}{\rho_{bal}} \qquad (56)$$

$$\frac{V}{b_w d \sqrt{f_c'}} \qquad (57)$$

Moreover, since the table defines distinctively the curve only in certain ranges and the results obtained were mid-way between such values, it was necessary to operate some interpolations.

The last parameter necessary for the complete definition of the hinges is the bending moment capacity at yielding of the steel reinforcement $M_{y,Rd}$. It was calculated considering the maximum tensile force absorbed by the rebars in the existing configuration:

$$T_y = f_y A_s \qquad (58)$$

And then multiplying such value for the distance between the reinforcement and the neutral axis of the section (x), which was assumed to be in the center. Consequently:

$$M_{y,Rd} = x\, T_y \qquad (59)$$

Finally, the beams' hinges were defined for the structures in the two locations.
The properties of the hinges in Italian case show greater values in terms of rotations, both for the curve definition and in the identification of the failure points. This result was expected since the defined reinforcement in the beams is bigger in the first location than in the structure situated in Aachen. The same happens for the resisting bending moment at yielding conditions.

The acceptance criteria are named after the American normative: Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP) but they are equivalent to the limit states defined by the Eurocodes, respectively Damage Limitation (DL), Significant Damage (SD) and Collapse Prevention (CP).

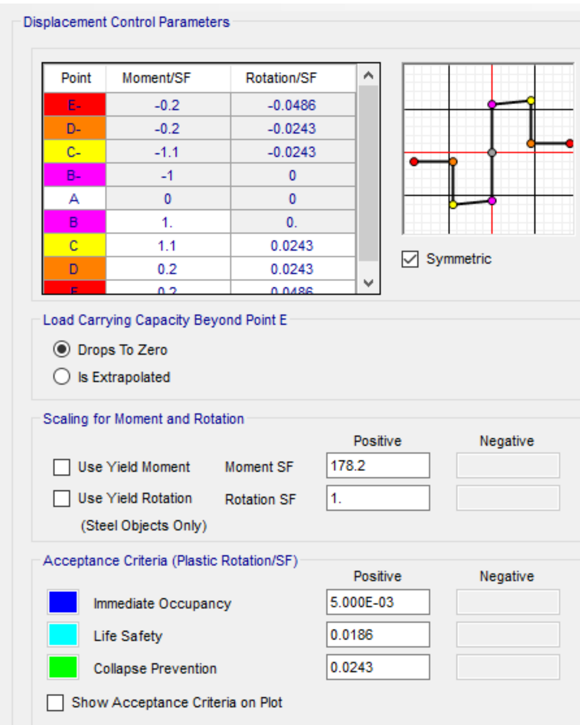


Figure 92 Definition of the moment-rotation curves and values of rotation corresponding to limit states, in location 1.

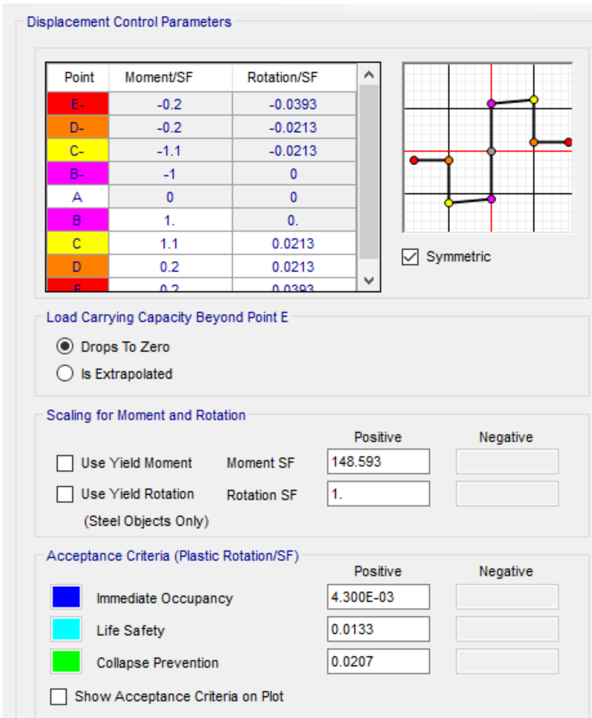


Figure 91 Definition of the moment-rotation curves and rotation corresponding to limit states, in location 2.

4.5.2 Positioning of plastic hinges

Starting from the existing building models, in all their elements the hinges were placed at the beginning and the end, towards the edges.

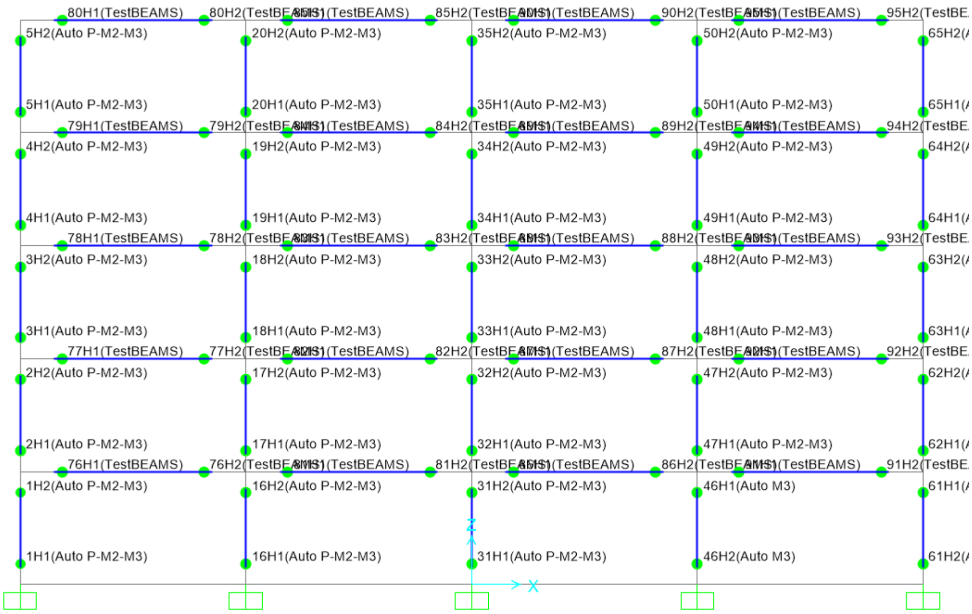


Figure 93 Position of the plastic hinges in the element in the existing structures, longitudinal frame.

The relative positioning of the hinge near the start and the one near the end of the elements, with respect to their length, was 0,05 and 0,95. Therefore, in the columns the first hinge was placed at 15 cm from the base and 15 cm from the top, while in the beams they were located at 30 cm from each of the two edges.

Considering the retrofit interventions carried out with reinforced concrete shear walls, also in those elements the hinges were placed at relative distance 0,05 and 0,95. In addition, the beams in correspondence of the portal subjected to refurbishment were modified for the application of the equivalent frame method. In fact, the part of beam embedded into the walls was defined as a new cross-section in the model, with extremely high flexural stiffness. The rest of the beam was kept as the existing element, with the plastic hinges only on the edges next to the columns, without any hinge positioned near the wall. Besides, the distance from the edge of beams was kept 30 cm, implying an updated relative distance in that part of beam equal to 0,15 (the length of each beam part on the side of the walls is 200 cm).

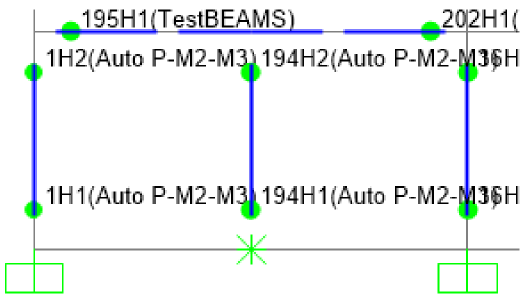


Figure 94 Detail of the reinforced concrete shear wall modeling, showing the subdivision of the beams in three elements: two parts of existing beam on the sides, one part of extremely stiff beam in the center, corresponding to the wall.

Figure 95 Position of the plastic hinges (green dots) in the elements of the reinforced concrete shear wall retrofit, case study in location 1, Zambrone. Presence of two shear walls in a longitudinal frame.

Figure 96 Position of the plastic hinges (green dots) in the elements of the reinforced concrete shear wall retrofit, case study in location 2, Aachen. Presence of one shear wall in a longitudinal frame.

In the end, for the cases of light timber frame shear wall retrofits, the new elements do not require any hinge since their nonlinearity is considered using plastic multilinear links. The hinges in the components of the existing reinforced concrete frame were kept in the same positions as specified for the study of the existing building.

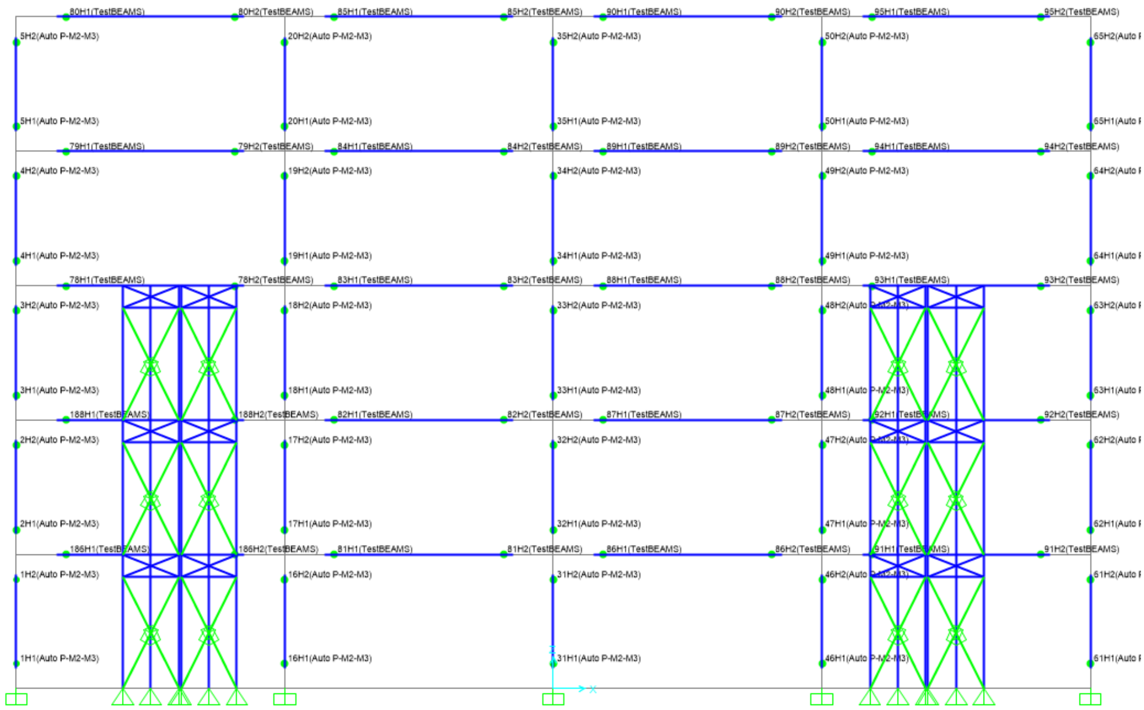


Figure 97 Plastic hinges (represented by the green dots) positioned in the case of timber retrofit of the structure in location 1, Zambrone.

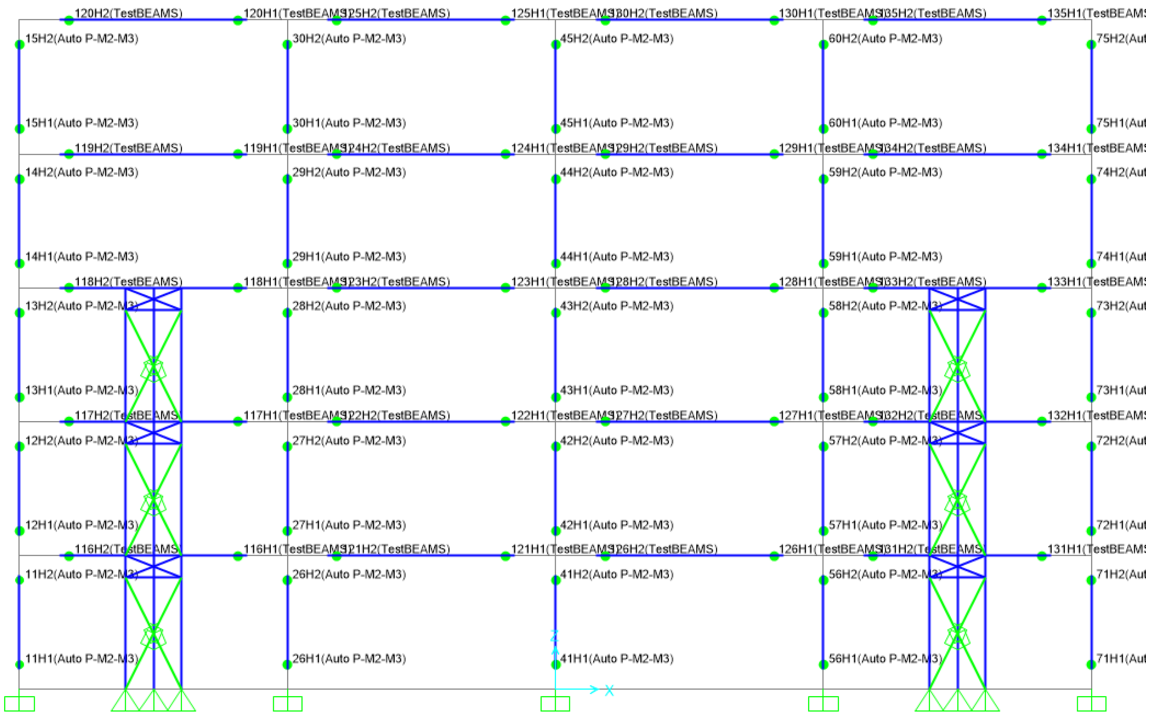


Figure 98 Plastic hinges (represented by the green dots) positioned in the case of timber retrofit of the structure in location 2, Aachen.

4.5.3 Pushover analysis and capacity curves

The various pushover analyses in the existing building and in the retrofitted structures were performed separately in the longitudinal and transversal direction of the building. This measure allowed lower instability during the computation of the steps in the analyses.
Other relevant settings are the initial conditions for the application of the horizontal load pattern, the load pattern itself, the way in which the nonlinearity is accounted for, the definition of the target displacement and the configurations of the results' saving.

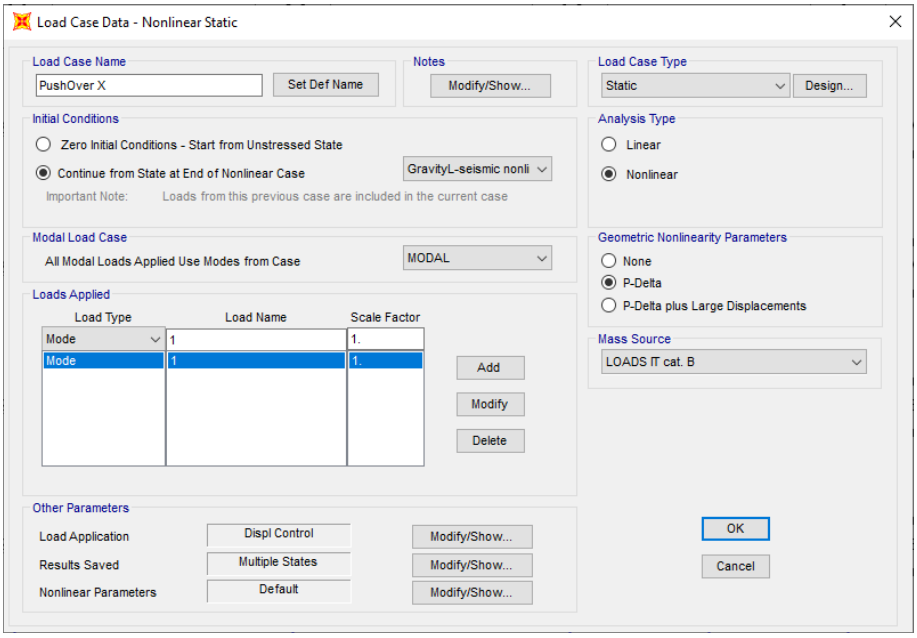


Figure 99 Settings adopted in the definition of pushover load cases.

As for the initial conditions, a nonlinear load case with the gravity loads for the seismic case was created in the model. The results of this analysis were then used as starting point for the pushover load case.

The horizontal load pattern was derived from the shape of the modes of vibration of the structure in the corresponding direction. It was computed directly by the software.

The geometric nonlinearity parameters are associated to the P-Delta method, through an option present in SAP2000.

The analysis was performed in displacement control and the **target displacement** in the corresponding direction was set as **30 cm**, selecting a node from the roof (fifth floor).

The analyses are completed only if the target displacement is reached or the maximum number of saved steps is fixed. Such number was fixed to 100, while the minimum number of saved steps was set to 50.

If it was not possible to reach convergence, obtaining too many null steps, the analysis ended with an error but still saving the results obtained up to that point.

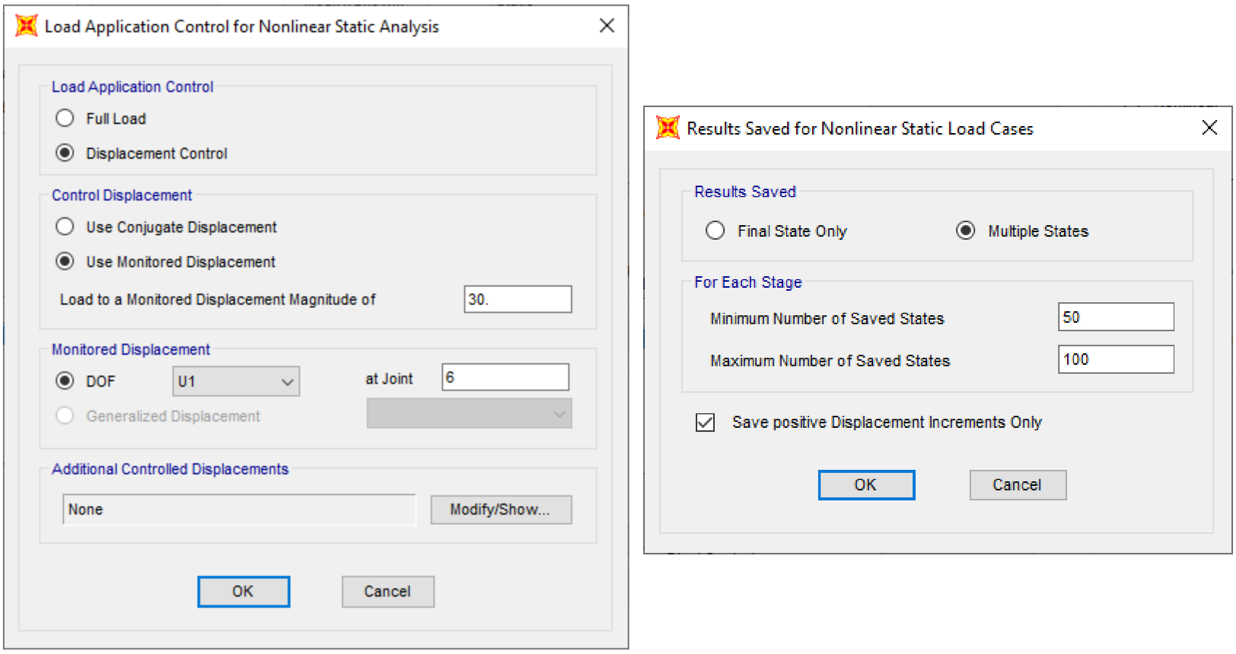


Figure 100 Settings for the displacement control (on the left) and saving of the results (on the right).

Then, running the pushover analyses on the existing and retrofitted structures, in both the locations, it was possible to obtain the corresponding capacity curves. Looking at the results on the model in terms of displacements, it was identified at which steps the plastic hinges overcame the value imposed for the Significant Damage limit state. Therefore, the **capacities** of the various building configurations **at the SD failure** were **determined**, in terms of base shear and roof displacement that could be bear by the structure.

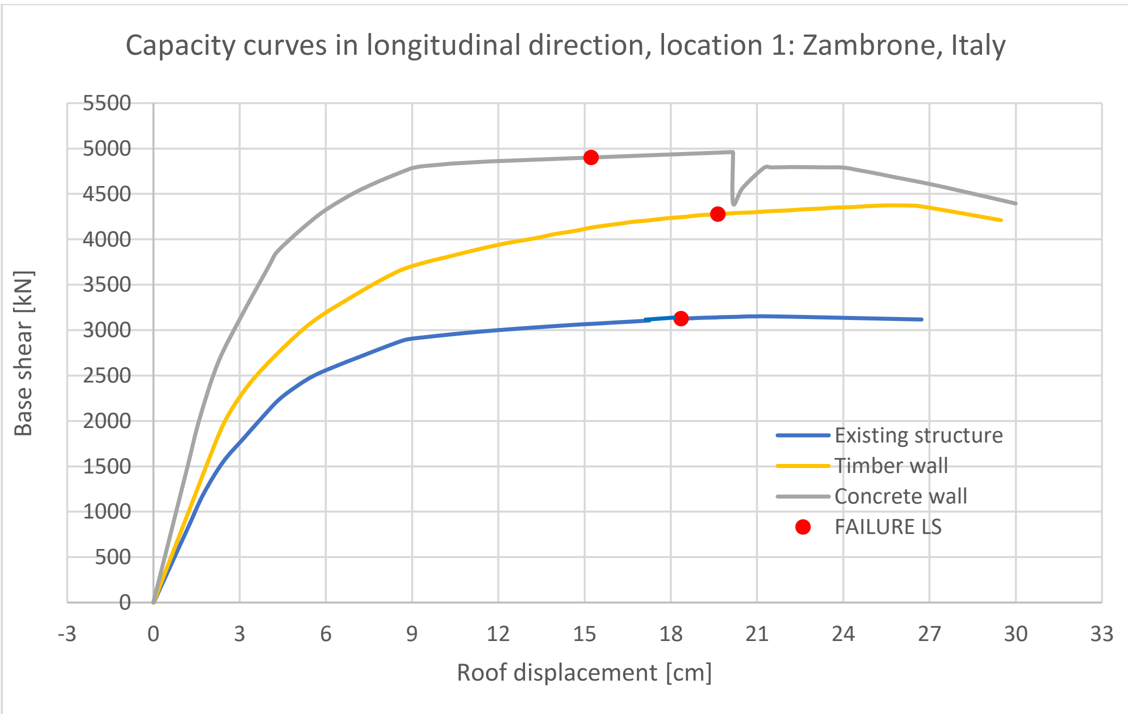


Figure 101 Capacity curves and failure points of the existing structure and after the two possible retrofit interventions, longitudinal direction, first location.

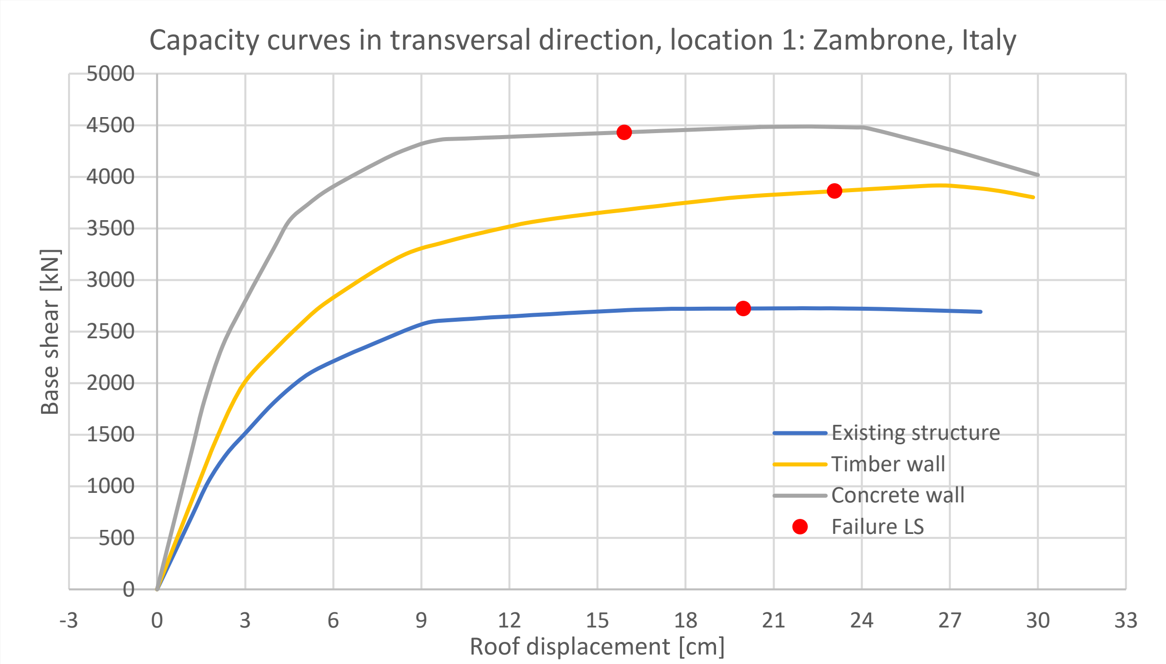


Figure 102 Capacity curves and failure points of the existing structure and after the two possible retrofit interventions, transversal direction, first location

The results in the first location confirm some aspects that were expected. For instance, the concrete retrofit makes the structure stiffer than the timber retrofit. On the other hand, the light timber frame shear walls confer to the building a much higher ductility with respect to the reinforced concrete solution. Also, the capacity in the longitudinal direction is higher than the one in transversal direction, since the frames that resist the horizontal loads are composed by more elements in the former case.

Instead, something that could seem unpredictable is the lower displacement at failure registered for the concrete retrofit, but it is still justified given the higher stiffness of the system, which can be accompanied by a more brittle behavior.

Another aspect that can be highlighted is the shape of the curve for the structure retrofitted with concrete shear wall, which has an irregularity around the displacement of 20 cm, dropping the value of base shear and then regaining it. This can be due to a sudden change in stiffness for the rotation in correspondence of the plastic hinges, in fact in those steps some hinges overcome the Near Collapse rotation.
At the same time, it shows how easily this kind of analysis could be unstable and even coupled with unexpected outcomes.

Continuing with the German intervention, in this case the timber retrofit is more resistant than the concrete one. Such result was not expected initially, but it could be reconducted to the different ratio of timber/concrete walls adopted with respect to the Italian case. In fact, for the structure located in Zambrone, such ratio was equal to 3 timber walls placed every 2 concrete walls, while in Aachen it was selected as 2 timber walls for each concrete wall.

However, the timber walls in the second location are composed by only one wooden frame of 1,25 m width, against the two elements used in Italy. This may indicate that distributing the single elements, instead of creating larger timber walls, could be better if the aim of the intervention is a stiffening of the building.

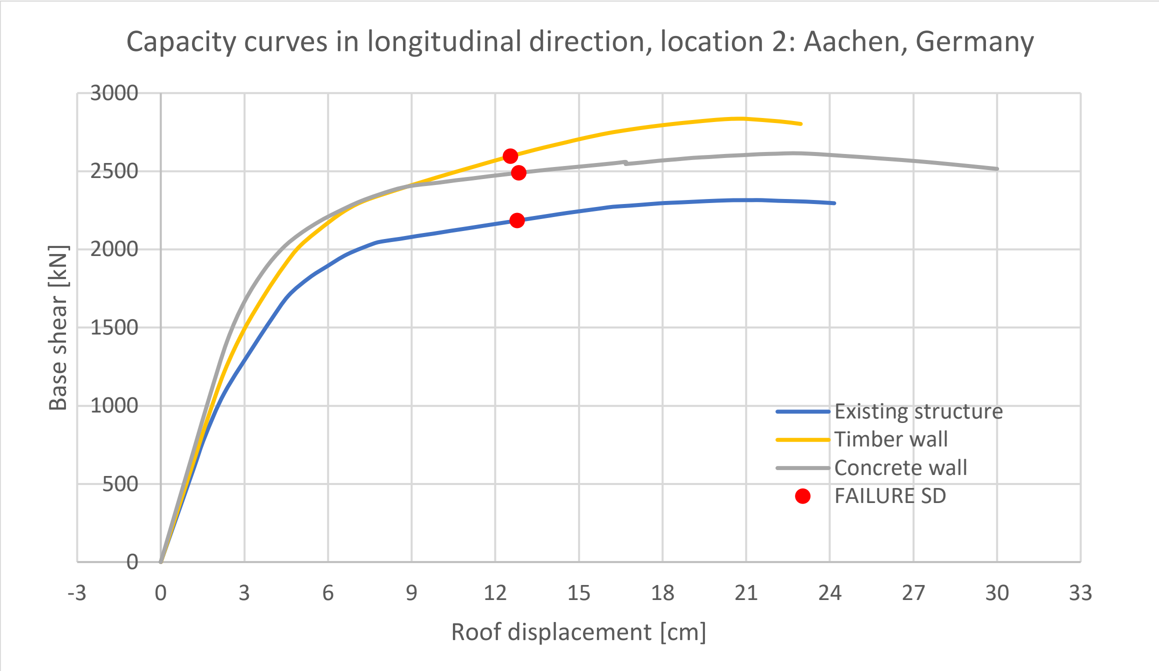


Figure 103 Capacity curves and failure points of the existing structure and after the two possible retrofit interventions, longitudinal direction, first location.

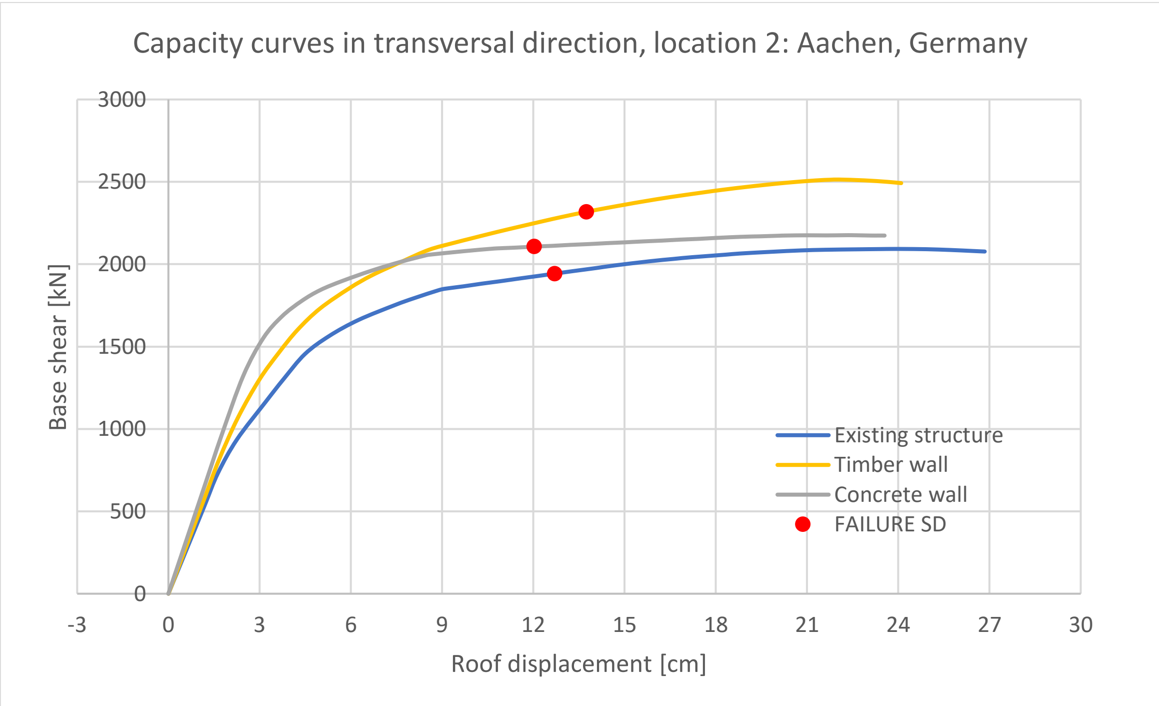


Figure 104 Capacity curves and failure points of the existing structure and after the two possible retrofit interventions, transversal direction, second location.

In any case, the reported curves refer to the actual multi-degree of freedom (MDOF) systems. The next step of the analysis is to transform them into the equivalent single degree of freedom (SDOF) systems' curves, which will be used to identify the performance point, in combination with the elastic response spectrum of the corresponding location. The performance points obtained with the N2 method will be confronted with the points in which the SD failure was observed during the pushover analyses.

4.5.4 Application of the N2 method

To apply the N2 method, it is necessary to transform the studied MDOF into an equivalent SDOF. In order to carry out such transformation, the lamped masses of each floor m_i of the structure were gathered from the models.

Then, in each direction (longitudinal or transversal), the shape of the relative vibration modes at each degree of freedom Φ_i (corresponding to each floor) were normalized to the highest floor.

Multiplying the mass of each floor by its mode shape and summing all of them, it is possible to obtain the modal mass of the equivalent SDOF, in the two directions:

$$\mathrm{m}^* = \sum \mathrm{m_i}\Phi_\mathrm{i} \qquad (60)$$

In the next step, the modal participation factor is calculated for the longitudinal and transversal direction separately, with the formula:

$$\Gamma = \frac{\mathrm{m}^*}{\sum m_i \Phi_i^2} \qquad (61)$$

The displacement and base shear proper of the SDOF are obtained simply scaling the capacity curves obtained by the pushover analysis with Γ.

The capacity curve of the SDOF is then approximated by a bilinear shape since the graphical procedure of N2 method requires a post yield stiffness equal to zero. This operation is done respecting the requirement of having the same energy absorption of the original curve. The last point of the curve is fixed, assigning to the simplified curve the yielding force F^* and the ultimate displacement d^*, while the displacement associated with the yielding of the curve is calculated with the *formula (36)*, already mentioned in *paragraph {3.5}*:

$$d_y^* = 2(d_m^* - E_m^*/F_y^*) \qquad (36)$$

In which E_m^* is the energy dissipated, calculated as the area under the original curve.

It is possible to observe that with this determination of the bilinear curve, the stiffness in the elastic domain is often lower than in the original curve. On the contrary, if the decrease of the capacity curve in its final part is very high, the elastic stiffness can become much greater than the original one. In the cases in which it became too much, instead of fixing yielding force as the last point, it was fixed as the maximum base shear in the capacity curve.

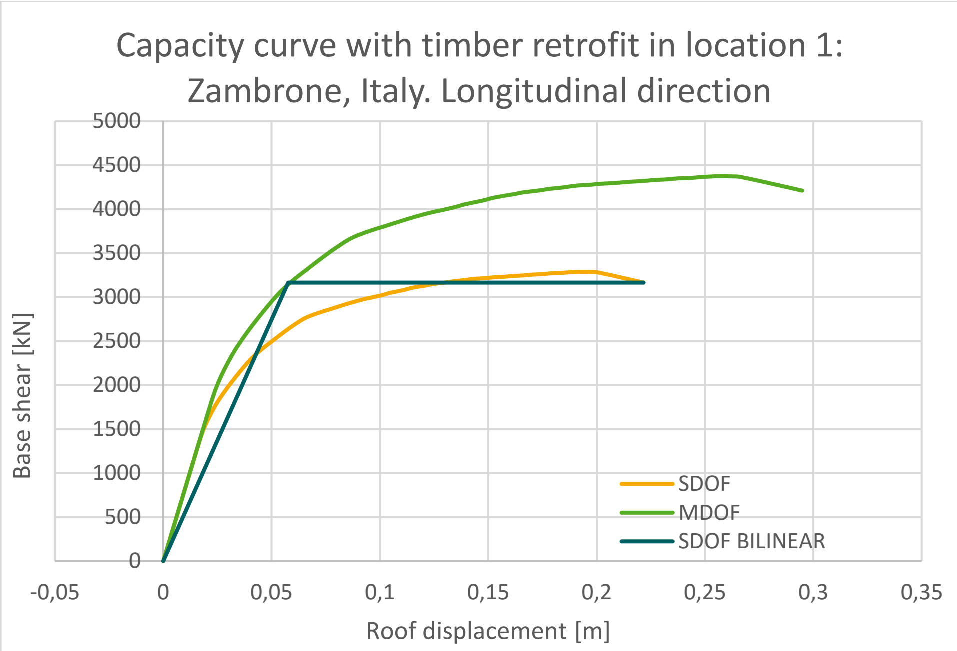


Figure 105 Example of transformation from MDOF to equivalent SDOF, and then to the bilinear curve for SDOF, in the force-displacement domain

With the aim of comparing the new capacity curves with the elastic response spectrum, the base shear is divided by the mass of the equivalent SDOF, obtaining the acceleration S_a of the SDOF. The capacity curve in the $S_a – S_d$ is known and it can be used in combination with the Acceleration – Displacement Response Spectrum (ADRS) to identify the performance point of the structure for the considered limit state.

In fact, the elastic response spectrum related to the Significant Damage (associated with the return period of 475 years) limit state needs to be translated from the period domain to the displacement domain, using the formula:

$$S_{de} = \frac{S_{ae} T^2}{4\pi^2} \qquad (25)$$

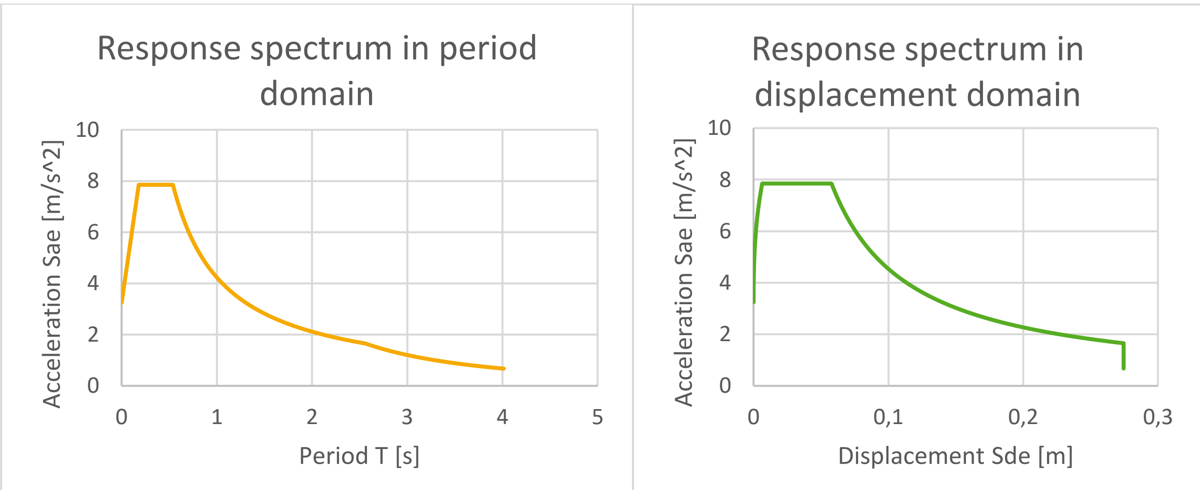


Figure 106 Difference between the acceleration-period response spectrum and the ADRS

Finally, the $S_a - S_d$ bilinear curve for the equivalent SDOF is approximated, is placed in the same domain as the ADRS, to get the performance point.

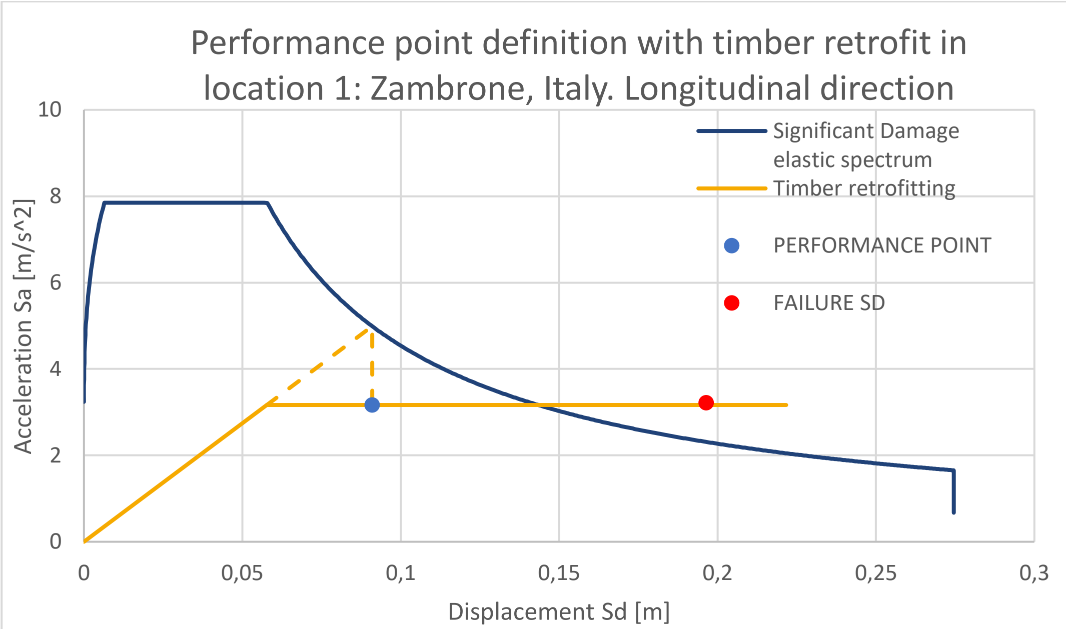


Figure 107 Example of application of N2 method for the first location, with timber retrofit

As shown in the previous graph, the performance point has the displacement in correspondence of the intersection between the elastic part of the bilinear curve (or its continuation) and the ADRS, since the elastic period $T^* \geq T_C$.
Then, the acceleration is either the one in the intersection, if it happens directly on the elastic branch, or the one associated with the equivalent SDOF yielding (horizontal part of the bilinear curve).
The inclination of the elastic branch is given by the elastic period of the system, which can be calculated through the *formula (38)* as:

$$T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}} \qquad (38)$$

If this point is located at a lower displacement than the failure identified in the model, due to the exceeding of the Significant Damage rotation for the first time during the pushover analysis, then the structure is verified. This means that the seismic loads coupled with the considered limit state are not strong enough to put the structure in a state of crisis.

In the end, the displacement in the original MDOF system can be obtained multiplying the displacement of the performance point by the modal participation factor:

$$D_t = d_{pp}\Gamma \qquad (62)$$

The outputs of the pushover analysis coupled with the N2 method were obtained for both the locations, considering the existing building and the two retrofitting techniques studied, separately for the two main directions of the building.

4.5.5 Evaluation of results in location 1: Zambrone, Italy

The graphs below show the results obtained by the pushover analysis coupled with the N2 method for the Italian case study, considering the two main directions of the building. The results are shown for the three structural configurations (existing building, reinforced concrete shear walls retrofit, light timber frame shear walls retrofit) to enable a comparison between their results.

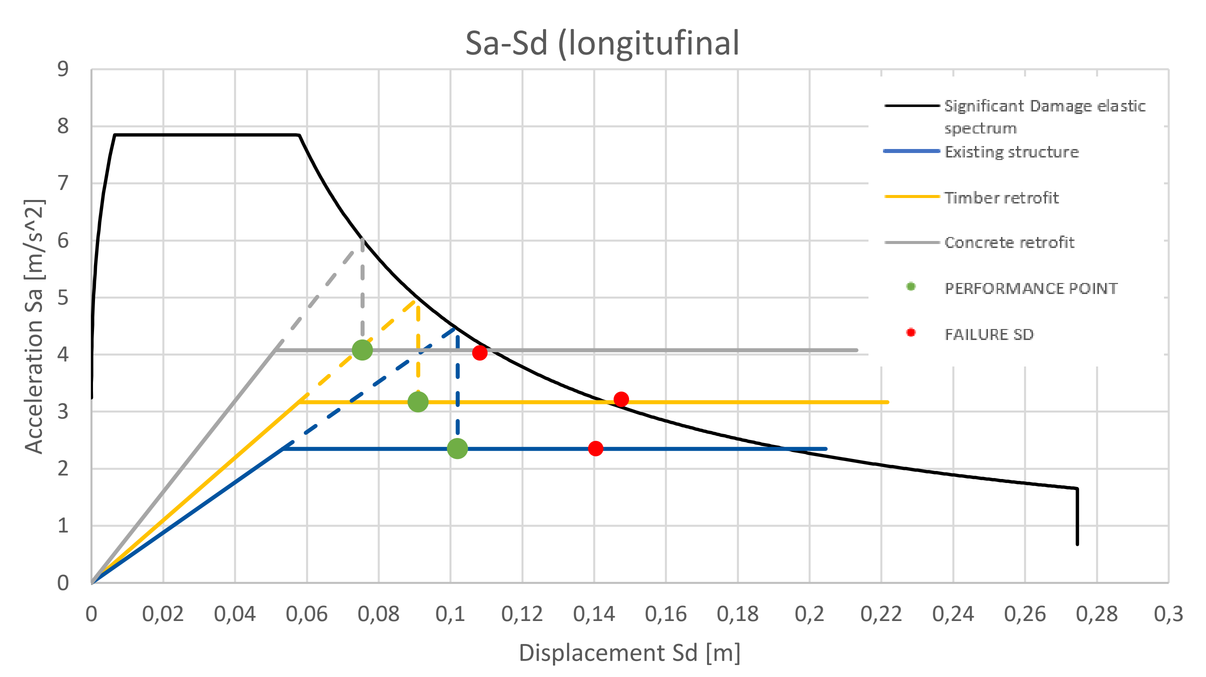


Figure 108 Structural evaluation in longitudinal direction

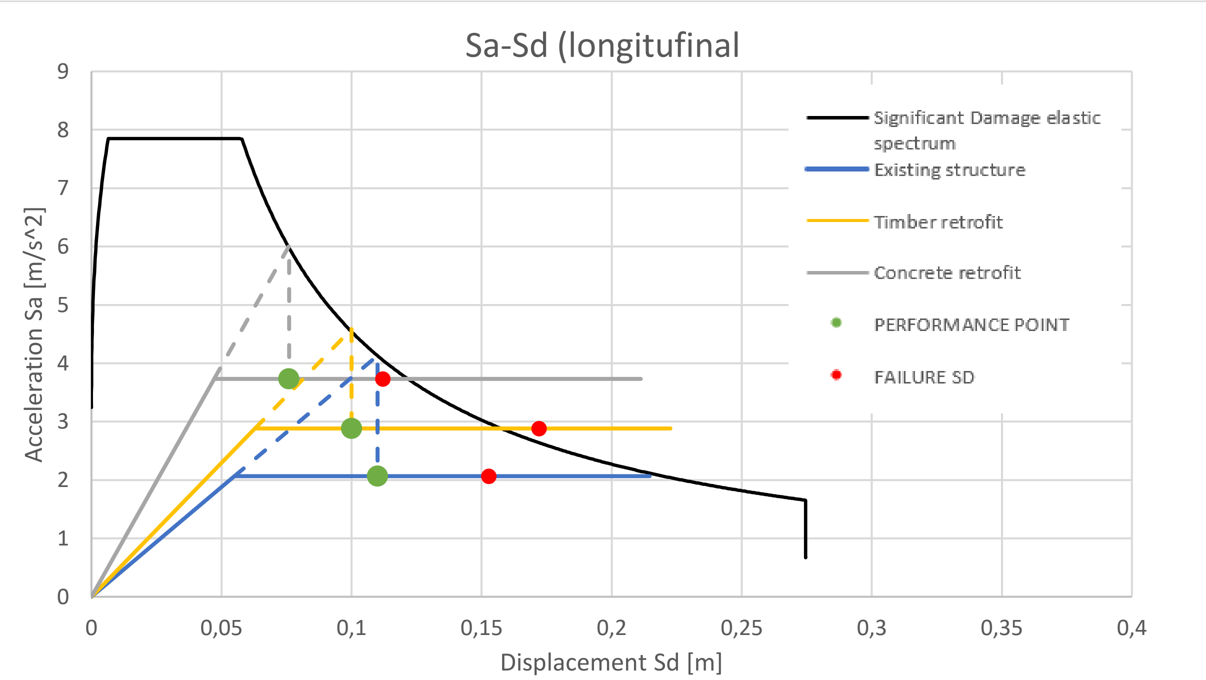


Figure 109 Structural evaluation in longitudinal direction

As it is easy to notice in *figures 108 and 109*, the performance points precede by far the point of failure at Significant Damage limit states, in all the cases.
This is the base for a very relevant observation: evaluating the behavior of the existing structure and its resistance against seismic loads with pushover analysis and N2 method, no retrofit intervention would be required. This would allow to save money, environmental impacts and disfunction of the building to perform the refurbishment. Instead, with the assessment carried out with the modal response spectrum analysis, the building needed absolutely a retrofit, in order to sustain the loads due to the SD limit state response spectrum, even after the application of a rather high behavior factor (q=3).
This point can be a good starting base for a potential criterion proposal for the Green Building Rating Systems, preferring more sophisticated analysis methods rather than simplified ones.
Then, assuming that an intervention is necessary, both the techniques exploiting reinforced concrete and timber were performing more than sufficiently, reaching better results than in the existing structure. So, both the retrofit are valid and could implemented, but they are prone to different results: on the one hand the reinforced concrete shear walls allow a higher stiffness in this case, with higher forces resisted, on the other hand the timber walls allow an higher ductility of the structure.

Therefore, if the retrofit choice was done purely following structural considerations, the technology would be selected based on the main goal of the refurbishment: reaching an higher ductility or an higher resistance to horizontal loads.

4.5.6 Evaluation of results in location 2: Aachen, Germany

In the second location the same procedure was applied, obtaining the results displayed in the *figures 110 and 111*.

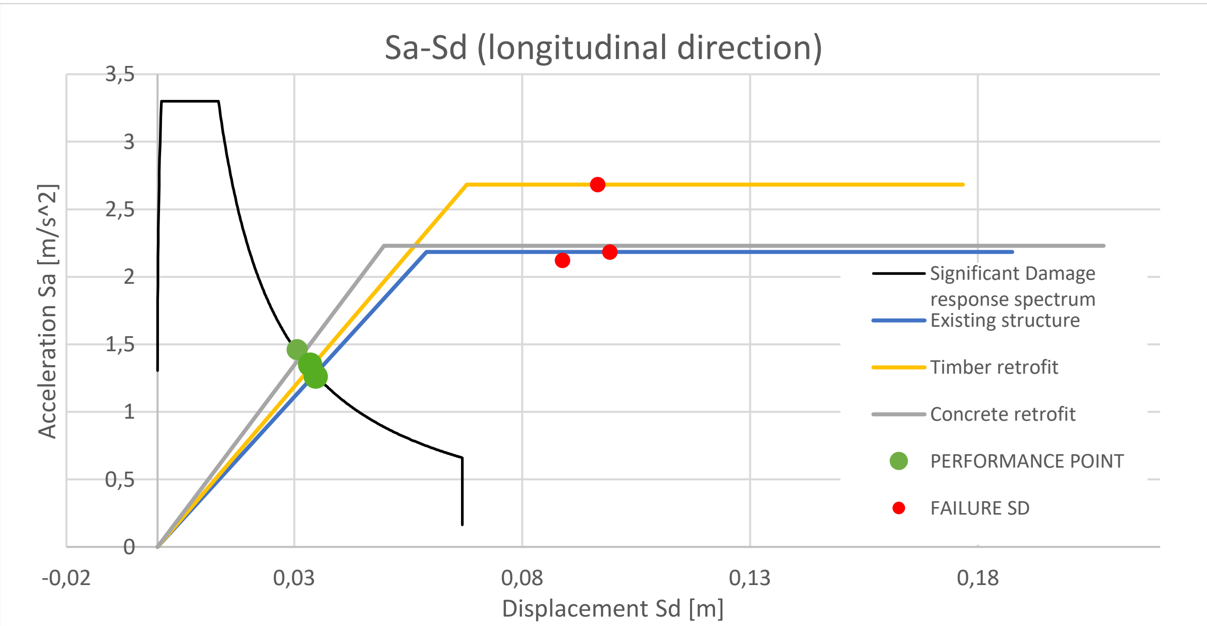


Figure 110 Structural evaluation in longitudinal direction

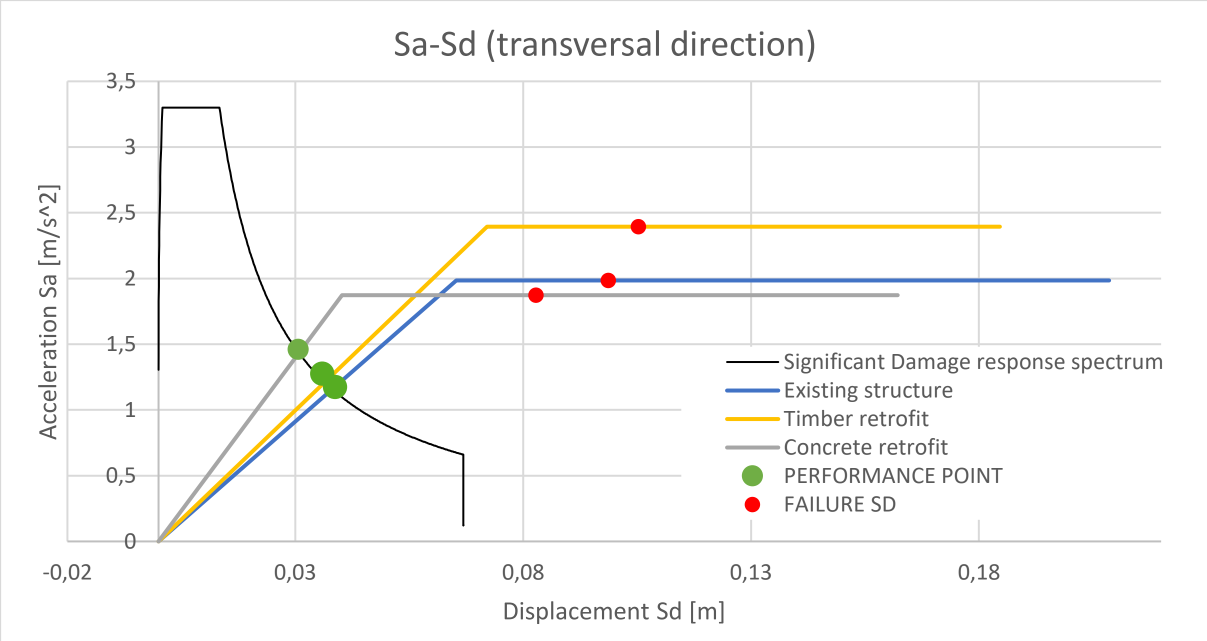


Figure 111 Structural evaluation in longitudinal direction

Also the existing structure situated in Aachen, according to this evaluation, is already verified for the SD limit state. Again, with this results no retrofit would be necessary, differently from the results obtained in the modal response spectrum analysis.

Consequently, both the refurbishment methods would be acceptable and could be used if the necessity for an intervention was assumed.

An interesting aspect of this case is the fact that the structural behavior is elastic. This is due to the very low requirement imposed by the elastic response spectrum associated with the German location.
The reinforced concrete solution is once again stiffer in the elastic branch with respect to the timber solution, but the latter reaches higher values of yield acceleration. However, since the performance point is in the elastic domain in all the cases, this higher resistance to lateral loads would not be shown in the case of seismic actions with the magnitude associated to 475 years of return period.

4.5.7 Results in the two locations in the original MDOF

Finally, to sum up the results obtained in the two locations and with different structural configurations, the following tables show the characteristics of the equivalent SDOFs. In addition, the performance points reconducted to the original MDOF are reported, in terms of displacement at the roof and base shear that would interest the structure in case a 475 years return period earthquake happened.

The first table refers to the Italian location, Zambrone:

Type of structure	Equivalent SDOFs						MDOF		
	longitudinal direction						longitudinal direction		
	m_x* [tonnes]	**Γ_x**	**T*** [s]	**S_{ay}*** [m^2/s]	**S_{ae}** [m^2/s]	**R_μ** $= S_{ae}/S_{ay}$*	Total **m** [tonnes]	**$D_{t,x}$** [m] $= S_{de}\,\Gamma_x$	**$F_{v,x}$** [kN]
Existing	1015,797	1,307	0,947	2,348	4,321	1,840	1638,76	0,137	3039,679
LTF retrofit	999,747	1,330	0,848	3,166	4,981	1,222	1654,66	0,121	3946,895
RC retrofit	864,145	1,408	0,704	4,077	5,840	1,432	1680,14	0,109	4850,511
	transversal direction						transversal direction		
	m_y* [tonnes]	**Γ_y**	**T*** [s]	**S_{ay}*** [m^2/s]	**S_{ae}** [m^2/s]	**R_μ** $= S_{ae}/S_{ay}$*	Total **m** [tonnes]	**$D_{t,y}$** $= S_{de}\,\Gamma_y$ [m]	**$F_{v,y}$** [kN]
Existing	1009,625	1,307	1,025	2,066	4,131	1,759	1654,662	0,144	2684,185
LTF retrofit	983,657	1,340	0,928	2131,784	4,553	1,939	1654,66	0,130	3572,819
RC retrofit	846,437	1,421	0,706	3,732	5,683	1,394	1680,14	0,114	4378,080

In the following table the data and results for the location 2, Aachen, are displayed:

Type of structure	Equivalent SDOFs					MDOF		
	longitudinal direction					longitudinal direction		
	m_x* [tonnes]	**Γ_x**	**T*** [s]	**S_{ay}*** [m^2/s]	**S_{ae}** [m^2/s]	Total **m** [tonnes]	**$D_{t,x}$** [m] $= S_{de}\,\Gamma_x$	**$F_{v,x}$** [kN]
Existing	823,009	1,288	1,033	2,184	1,275	1261,52	0,045	1674,079
LTF retrofit	813,847	1,299	1,000	2,683	1,382	1261,54	0,041	2180,118
RC retrofit	811,610	1,445	0,938	2,230	2,182	1633,42	0,029	2047,858
	transversal direction					transversal direction		
	m_y* [tonnes]	**Γ_y**	**T*** [s]	**S_{ay}*** [m^2/s]	**S_{ae}** [m^2/s]	Total **m** [tonnes]	**$D_{t,y}$** $= S_{de}\,\Gamma_y$ [m]	**$F_{v,y}$** [kN]
Existing	819,078	1,287	1,140	1,985	1,114	1261,52	0,051	1447,858
LTF retrofit	804,392	1,305	1,090	2,394	1,313	1261,54	0,043	2004,988
RC retrofit	799,304	1,452	0,921	1,873	1,419	1633,42	0,029	1789,141

It is relevant to highlight that in the german location, the value of the reduction factor R_μ was not considered since the performance point is in the elastic branch of the S_{ae} - S_{de} curve. In such case, it would be lower than 1.

Another observation concerns the lower forces that can be sustained in the transversal direction, due to the lower number of portals (only 2 spans instead of 4 covered in the longitudinal direction). Consequently, in that direction the performance point is characterized by an higher displacement.

4.6 Evaluation of environmental impacts

The evaluation of the environmental impacts was conducted with a simple methodology, but quite time demanding. In fact, many Environmental Product Declarations (EPDs) for the various components were gathered and used together to assess the impacts, with the purpose of obtaining from the research results as general as possible.
If this kind of method would be adopted in the practice of a retrofit intervention, it would still be simple and also imply a rather short time for the calculation. In fact, it wouldn't be necessary to derive the outcomes by many EPDs for the same material, but just by the EPDs of the components actually implemented in the retrofitting intervention.

Another difficult part of the process was to identify EPDs that considered the same modules and life cycle stages, otherwise a comparison would not be possible. This issue would present itself in a limited extent in real practice, only for the comparison between the different solutions that could be adopted in the retrofit and not between all the various products used for defining the impact of same component, as in this study.
In addition, the confronted declarations must have in common the same functional requirements, but this should not be a problem since the elements considered are very similar.

The final results were evaluated in terms of Global Warming Potential (**GWP**), with its unit of measure kg of $CO_{2,eq}$, total use of renewable and non-renewable primary energy resources (**PERT** and **PENRT**) expressed in MJ, use of net fresh water (**FW**) in m^3. These are only 4 of the many parameters described by EPDs, but they are the most intuitive and easy to understand, other than being particularly relevant for environmental impacts.

4.6.1 Environmental impacts of light timber frame shear walls

The study of these components was carried out splitting the walls in their main components: OSB sheathing, solid timber studs, and fasteners (staples, screws and plates).

The results for a single panel of 1,25 m width were evaluated in the same way for both the locations, since the geographical scope of the considered EPDs was often not defined.

Based on the geometry and the construction details available, the volumes or masses of the components (depending on the declared unit used in the EPDs of that kind of element) were calculated and used then to evaluate the environmental impacts. For each element of timber power wall, the volume of OSB panels and studs is respectively 0,141 m^3 and 0,171 m^3 while the total mass of fasteners is 9,44 kg.

For the **OSB panels**, three different EPDs were gathered:

- SWISS KRONO OSBPlatten [89]
- Oriented Strand Board, OSB [90]
- EGGER OSB-boards [91]

EPD product	System boundaries	Geographical scope	Time validity	Declared unit	Source
EGGER OSB-boards [89]	A1-A3, C3, D	/	2018-2023	1 m3	IBU
Oriented Strand Board, OSB [90]	A1-A3, A5, C3, D	/	2021-2026	1 m3	IBU
OSB NORBORD [91]	A1-A5, C1-C3, D	/	2022-202X	1 m3	International EPD system

Regarding the **solid timber studs**, four EPDs were eligible for the study:

EPD product	System boundaries	Geographical scope	Time validity	Declared unit	Source
KVH STRUCTURAL TIMBER [92]	A1-A3, A5, C2, C3, D	/	2018-2023	1 m3	IBU
EGGER TIMBER [93]	A1-A3, C1-C4, D	/	2021-2026	1 m3	IBU
GLUED SOLID TIMBER [94]	A1-A3, C1-C4, D	/	2021-2026	1 m3	IBU
STANDARD AND SPECIAL SAWN TIMBER [95]	A1-A5, C1-C3, D	Global	2022-2027	1 m3	International EPD system

In the end, all the **fasteners** were approximated with the use of a single EPD, produced for screws. This was done to overcome the lack precise declarations for the different fastening elements (screws, staples, nails, plates).

EPD product	System boundaries	Geographical scope	Time validity	Declared unit	Source
KVH STRUCTURAL TIMBER [96]	A1-A3, C1-C4, D	/	2022-2027	1 kg	IBU

Since most of the considered EPDs did not account for the phases A4 and A5, the only common phases in all the chosen declarations are: A1-A3, C3, D.

Such life-cycle phases deal with: raw material supply, transport and manufacturing (A1, A2 and A3, in the product stage), waste processing (C3, in the end of life stage), and finally reuse/recovery/recycling potential (D: benefits and loads beyond the system boundaries).

4.6.2 Environmental impacts of reinforced concrete shear walls

In the case of concrete, many EPDs were available and also indicated the geographical scope. Hence, it was possible to diversify the results for the Italian and German locations, in addition to considering different amounts of rebars for the dimensioned walls. In the Italian case, the volume of concrete is 1,167 m^3 in each wall, while the mass of reinforcement steel is 255,466 kg. For the second location, each RC shear wall is composed by 1,183 m^3 of concrete and 134,470 kg of steel.

Starting from **concrete C30/37**, here are the gathered EPDs:

EPD product	System boundaries	Geographical scope	Time validity	Declared unit	Source
CALCESTRUZZO PRECONFEZIONATO [97]	A1-A3, C1-C4, D	Europe	2022-2027	1 m3	EPD Italy
MISCELE DI CALCESTRUZZO RICICLATO (BWR) [98]	A1-A5, C1-C4, D	Italy	2021-2026	1 m3	EPD Italy
MISCELE DI CALCESTRUZZO RICICLATO (Gasser Markus) [99]	A1-A5, C1-C4, D	Italy	2021-2026	1 m3	EPD Italy
READY MIXED CONCRETE C30/37 (Iston) [100]	A1-A5, C1-C4, D	Global	2022-2027	1 m3	International EPD system
READY MIXED CONCRETE C30/37 (Lafarge) [101]	A1-A5, B1-B7, C1-C4, D (A1-D)	Global	2021-2026	1 m3	International EPD system
GENERIC READY-MIXED CONCRETE [102]	A1-C4	/	2018-2023	1 m3	IBU
RECYCLING READY MIX CONCRETE C30/37 [103]	A1-A3, A4, C1-C3, D	Germany	until 2022	1 m3	Oekobaudat
RECYCLING READY MIX CONCRETE C30/37 [104]	A1-A3, A4, C1-C3, D	Germany	until 2022	1 m3	Oekobaudat

It is important to notice that the “Generic ready-mixed concrete” [99] was initially considered, but then it was neglected since it does not take into account the phase D of the life-cycle.

Then going on with the **steel reinforcement B500C**, the first three EPDs from the following table above were used for the calculation of impacts in Italy. The fourth and the fifth where used in both the locations. Meanwhile, the last three were used only in the German case, despite having a general or not specified geographical scope. This last choice was made since the production sites of the corresponding products are located in Lituania, Poland and Germany, in the order in which they are shown in the table.

The same reasoning was applied to the first two declarations of steel rebars selected, which have a global geographical scope but derive from Italian production plants, hence they were adopted only for the assessment of the Italian retrofit.

EPD product	System boundaries	Geographical scope	Time validity	Declared unit	Source
HOT-ROLLED REINFORCING STEEL [105]	A1-A4, C1-C4, D	Global, Italy	2022-2025	1 ton	EPD Italy
HOT-ROLLED REINFORCING STEEL FOR CONCRETE [106]	A1-A4, C1-C4, D	Global, Italy	2016-2027	1 ton	EPD Italy
ACCIAI LAMINATI A CALDO TONDO IN ROTOLI, TONDO IN BARRE [107]	A1-A3, C1-C4, D	Italy	2022-2025	1 ton	EPD Italy
Hot rolled concrete steel [108]	A1-A4, C2-C3, D	Europe, Serbia	2021-2026	1 ton	International EPD system
HOT-DRAWN REINFORCING STEEL FOR CONCRETE [109]	A1-A4, C1-C4, D	Global	2016-2026	1 ton	EPD Italy
Steel rebars [110]	A1-A4, C1-C4, D	Europe	2021-2026	1 kg	International EPD system
XCarb® Recycled and renewably produced Reinforcing steel in bars and coils [111]	A1-A3, C3-C4, D	/	2021-2026	1 ton	IBU
Schöck Combar® [112]	A1-A3, A5, C2-C4, D	/	2021-2027	1 kg	IBU

Considering the necessity of group many materials in the composition of the same wall, and above all the fact that the timber and reinforced concrete walls need to be compared, the only life-cycle stages common to all the exploited EPDs are the following: A1-A3, C3, D.

Thereby, even though the assessment could seem quite restricted, especially if compared to the potentialities of some EPDs, the results were evaluated considering only those stages.

4.6.3 Methodology for the results interpretation

The evaluation of the results was made following three different logics.

1) The first type of assessment is for only one wall element, without considering the contribution of the last phase, related to the reuse, recovering and recycling potential. This was done to adopt an approach more focused on the short term, which might be more appealing for the people that have decisional power.
   This was done because in that stage the values are usually negative, showing the positive impacts due to a circular use of material. In this way it will be possible to highlight how much the environmental performance can get better for each retrofit, in the case a proper use of the materials that reach their end of life is made.
2) The second visualization of results considers also the last stage of the life cycle, always accounting for just one wall elements. In this way, the comparison between cases with and without material reuse, recovering and recycling is allowed.

3) In the end, the third type of results aims to show the outcomes obtained for the whole building, taking into account both the locations in which the structure is studied. Consequently, the impact of the overall refurbishments will be displayed, with the implementation of different numbers of shear walls depending on the site.

4.6.4 Single wall element results in location 1: Zambrone, Italy

Starting from the Italian case study located in Zambrone, the *figure 112* shows the impact of a single wall element, considering the first two kinds of results' representations.

The elements considered are:

- a shear wall with the dimensions common for both locations 1 and 2 (200 cm width, 300 cm height and 20 cm thickness), with the dimensioned amount of steel rebars for the Italian retrofit;
- a timber wall module with a width of 125 cm, composed as described as in *paragraph {3.4}.*

Starting from the **Global Warming Potential** (GWP):

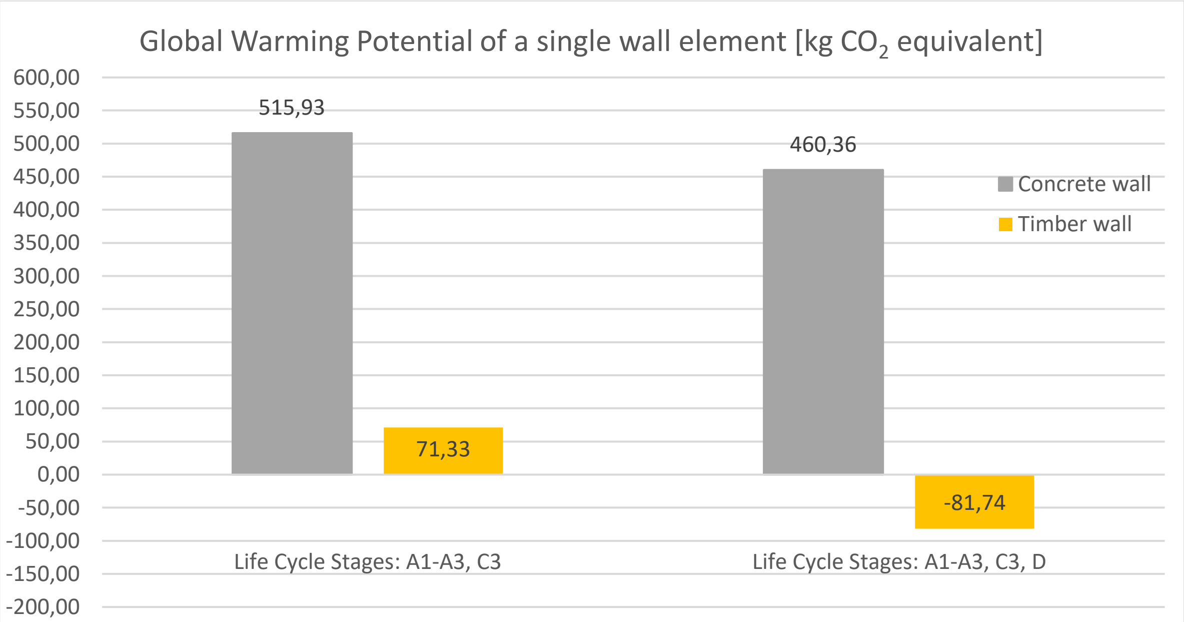


Figure 112 GWP of a single wall element in the first location: Zambrone, Italy

It is clear how the timber wall allows a lower impact in terms of GWP. In the first case, without the recycling and reuse phase, the amount of kg of CO_2 equivalent spared using one timber wall element is:

$$\Delta GWP \ = GWP_{RC} - GWP_{timber} = 515{,}93 - 71{,}33 = 444{,}6\ kg\ CO_2eq$$

In the case in which also the stage D is considered, the difference is even higher:

$$\Delta GWP_{with\ D} \ = GWP_{RC,with\ D} - GWP_{timber,with\ D} = 460{,}36 - (-81{,}74) = 542{,}1\ kg\ CO_2eq$$

The value of $GWP_{timber,with\ D}$ could seem a bit curious since it is negative, but it simply means that in the end, the benefits associated with a proper reuse of this material are higher than the impacts caused by adopting it in the first place. In addition, the EPDs concerning wood elements often present negative values also in the product stage, since in that phases the CO_2 absorbed by the trees in their life is counted with an improving impact on the emissions of the final product.

Considering one more time the results on the left, an amount of **7,23 timber wall elements** has the **same impact of** just one **reinforced concrete wall**. For the results accounting for the phase D, the ratio is not meaningful since the result for wooden elements is negative.

It is also true that to obtain a comparable structural improvement, many more timber elements are required than RC shear walls. Later it will be analyzed whether the timber solution in the overall retrofit will be still advantageous environmentally speaking.

Going on with the **use of primary energy resources**, it is shown how the considered timber elements exploit many more renewable resources compared to the production of concrete and steel rebars. If also the final stage is considered, the value of PENRT (total use of primary non-renewable energy resources) is even negative, since it considers the avoided impacts thanks to material reuse.

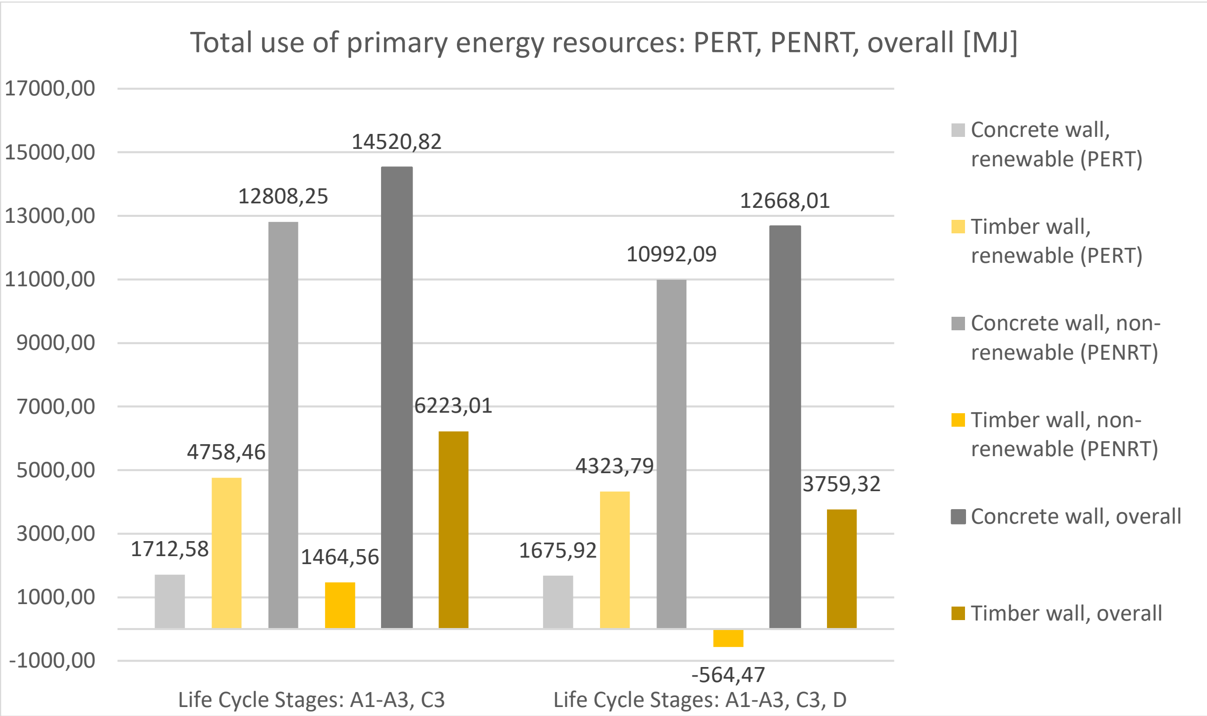


Figure 113 Total use of primary energy resources of a single wall element. Location: Zambrone, Italy

In the end, the overall sparing of energy resources associated with timber walls rather than reinforced concrete walls is:

$$\Delta energy\ = energy_{RC} - energy_{timber} = 14{,}52\ -\ 6{,}22\ =\ 8{,}30\ GJ$$

And considering also the phase D it becomes:

$$\Delta energy \ = energy_{RC,with\,D} - energy_{timber,with\,D} = 12{,}67 \ - \ 3{,}76 \ = \ 8{,}91\ GJ$$

If the results considered regarded only the non-renewable shares, the differences would be even higher.

Finally, the amount of timber walls that consume the same energy as their reinforced concrete homologous is **3,37**, also considering the stage beyond the system boundaries.

To conclude the observations on the single wall elements for the first location, the last parameter considered is the **use of net fresh water** (FW):

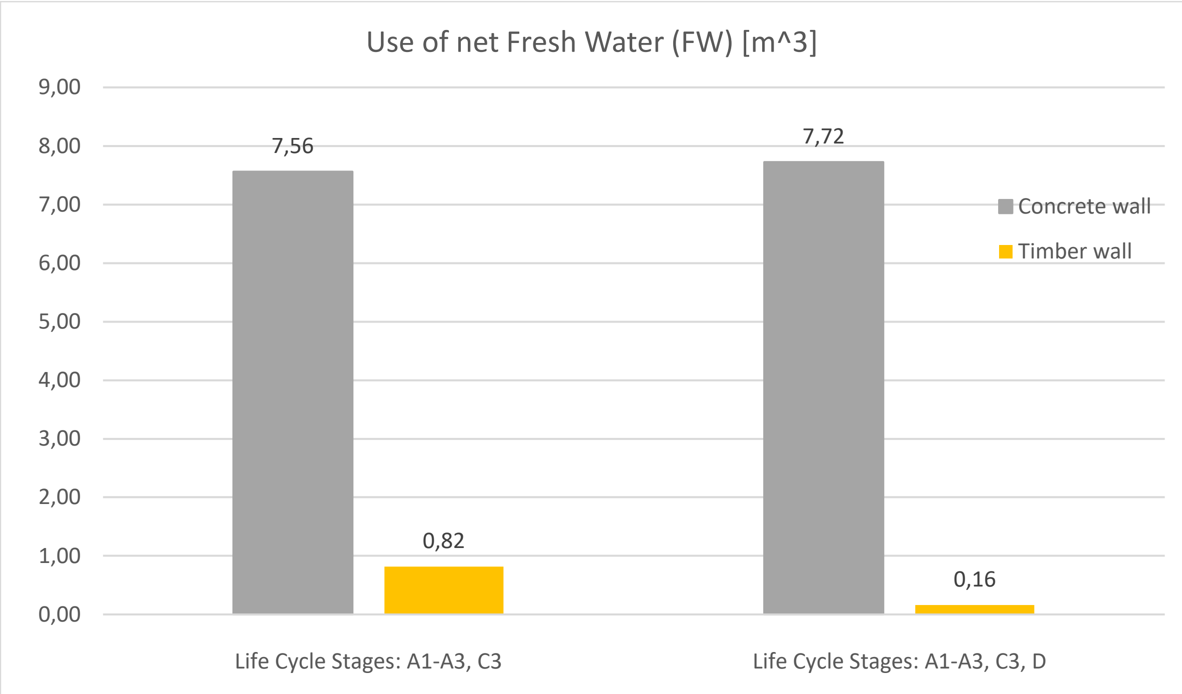


Figure 114 FW of a single wall element in the first location: Zambrone, Italy

Again, the values show evidently how the reinforced concrete walls have higher impacts. In addition, a particular effect is present in the concrete walls, with a FW that becomes higher with the addition of the reuse, recovery and recycling potential. This means that those operations require more water than the one spared by recycling the products that compose the RC wall.

Considering also the phase D, **7,56 m^3** of water can be spared using a timber element instead of a concrete element. With the same approach to the results, more than **48 wooden walls** imply the same FW of just one concrete wall.

4.6.5 Single wall element results in location 2: Aachen, Germany

Highlighting the fact that the timber elements were computed in the exact same way in the two locations, giving the same results, the difference from the Italian case will depend only on the impacts associated to reinforced concrete shear walls.

A relevant observation is that in all the parameters accounting for the phase D implies higher impacts. Such phenomenon is due to the fact that in the steel rebars EPDs used for Germany, the recycling of steel (usually around 90% of the total steel considered) appears to be more impactful than the production of new steel. In fact, the values coupled with that phase should be given by the difference between the harmfulness of the recycling procedure and the one of new production, which is in the end positive in many of the considered declarations. Therefore, on a one-element comparison scale, the environmental advantages of using timber become even higher than keeping the calculation into the system boundaries.

Showing the same type of results as in the Italian case, but for the second location, the ratio between the GWP of one RC element and one timber element becomes lower: **6,97 timber walls emit as much as one concrete wall**. This is given by the lower amount of steel reinforcement adopted in the same wall geometry for the structure in Aachen compared to the equivalent in Zambrone.

Instead, the spared kilograms of equivalent CO_2 are increasing it the phase D is also considered, calculated with the difference:

$$\Delta GWP_{with\ D} = GWP_{RC,with\ D} - GWP_{timber,with\ D} = 587{,}07 - (-81{,}74) = 668{,}81\ kg\ CO_2eq$$

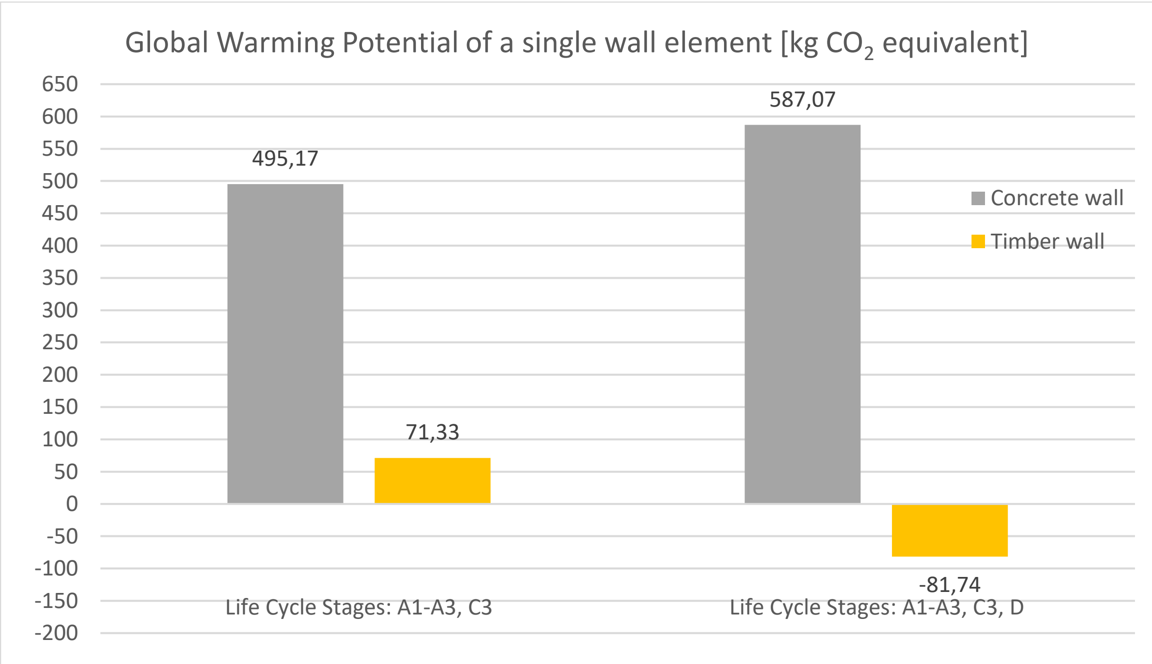


Figure 115 GWP of a single wall element in the second location: Aachen, Germany

The same qualitative observations applied for the GWP are valid also for the **total use of primary energy resources**. The EPDs used for the reinforced concrete walls in Germany present much worse results than the ones used in Italy. By consequence, also the overall energy use is much greater than in the first location, even if the required reinforcement is lower. The ratio between the two elements shows that **9,32 timber walls** consume the same energy **as one RC shear wall** element. The spared energy, considering all the analyzed phases is extremely high:

$$\Delta energy = energy_{RC,with\ D} - energy_{timber,with\ D} = 35{,}03 - 3{,}76 = 31{,}27\ GJ$$

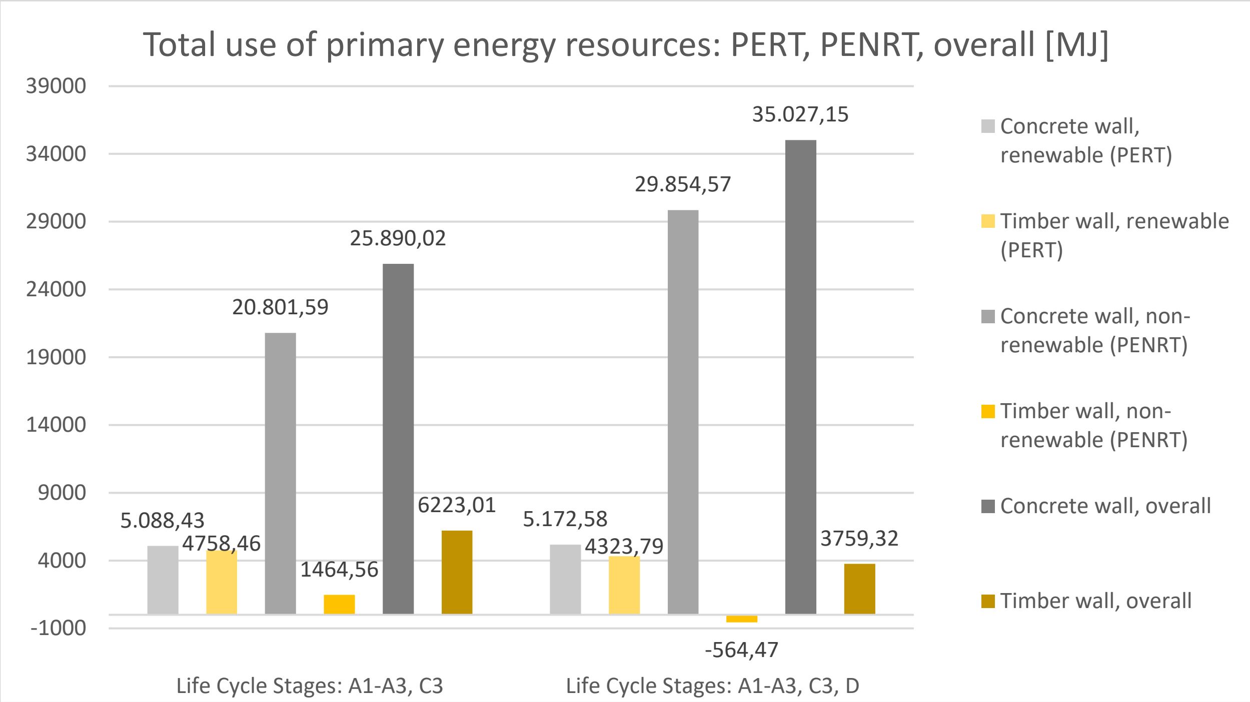


Figure 116 Total use of primary energy resources of a single wall element. Location: Aachen, Germany

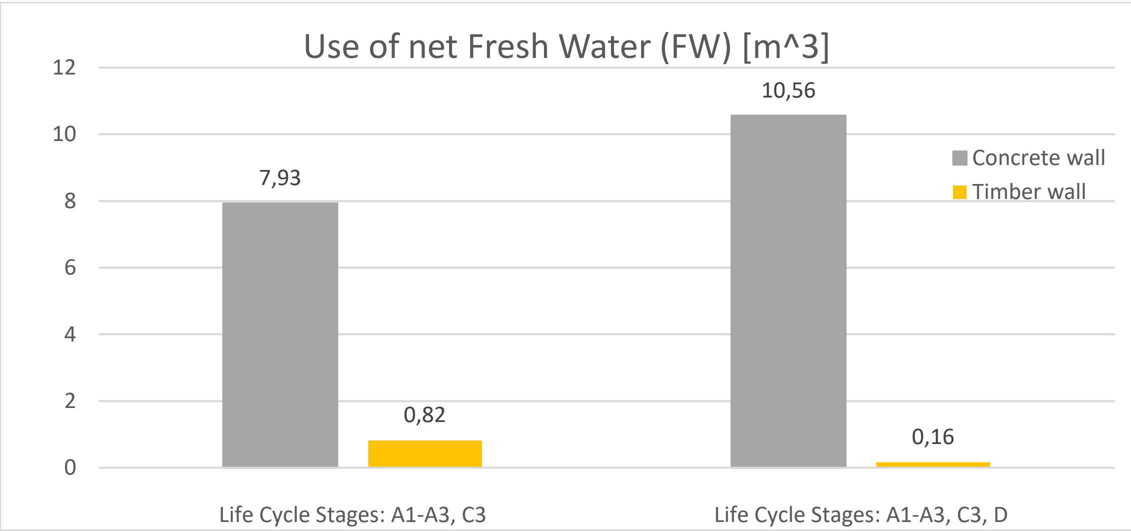


Figure 117 FW of a single wall element in the second location: Aachen, Germany

The final observations are inspired by *figure 117*, which enables to study the results in terms of **use of net fresh water** for the German case. Taking into account also the final stage, the **ratio** reinforced concrete single element over timber single element is **66**.

By using a wooden wall instead of a concrete one, a maximum of **10,40 m^3 of FW** could be spared.

4.6.6 Overall retrofit results in both the locations

The following graphs report on the left the results obtained for the Italian case study, while on the right the structure based in Aachen is considered. The evaluations are considering the phases of the life cycle A1, A2, A3, C3 and D.

It is useful to recall that in the Italian case 24 RC shear walls were adopted, while the number of light timber frame elements of 1,25 m width was 72. In Aachen, the reinforced concrete walls were 12 and the timber walls used were 24.

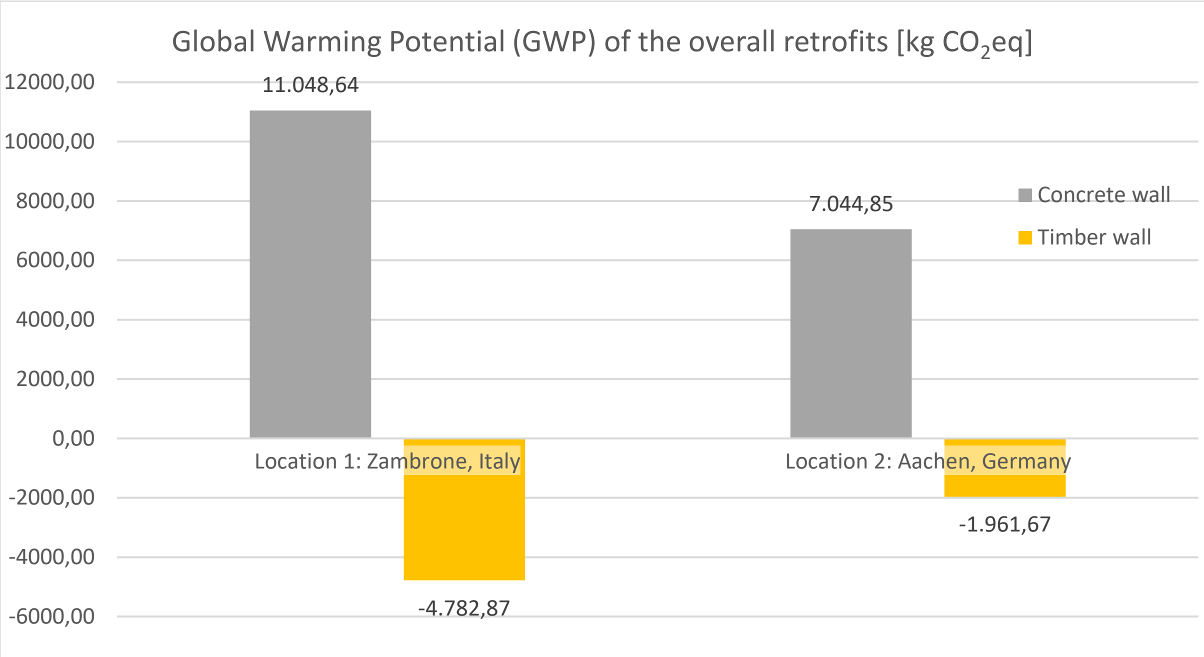


Figure 118 GWP of the overall retrofits, for both timber and reinforced concrete, in both the locations

Looking at the results in terms of **GWP**, it is fair and square how the interventions carried out with timber technology have lower environmental impacts than their reinforced concrete counterparts.
In both the locations, if the materials are exploited in the right way beyond the system's boundaries, the timber refurbishment will have more benefits than loads.

The differences in the two locations are respectively:

$$\Delta GWP_{IT} = GWP_{RC,IT} - GWP_{timber,IT} = 11{,}05 - (-4{,}78) = 15{,}83\ tonnes\ CO_2eq$$

$$\Delta GWP_{DE} = GWP_{RC,DE} - GWP_{timber,DE} = 7{,}04 - (-1{,}96) = 9{,}00\ tonnes\ CO_2eq$$

The comparison between the two sites shows that, especially when the retrofit is a quite extensive intervention, the sparing in terms of equivalent CO_2 can be massive.

Then, looking at the energetical issues, the Italian case study shows a very little difference in terms of total use of overall primary energy resources, relatively to the huge energy involved. But a high value for renewable resources (PERT) is much more valuable than a similar amount of PENRT in terms of environmental impact.

Moreover, in the second location the impact of reinforced concrete walls appears to be even greater, despite the number of RC elements is half the amount utilized in Zambrone.

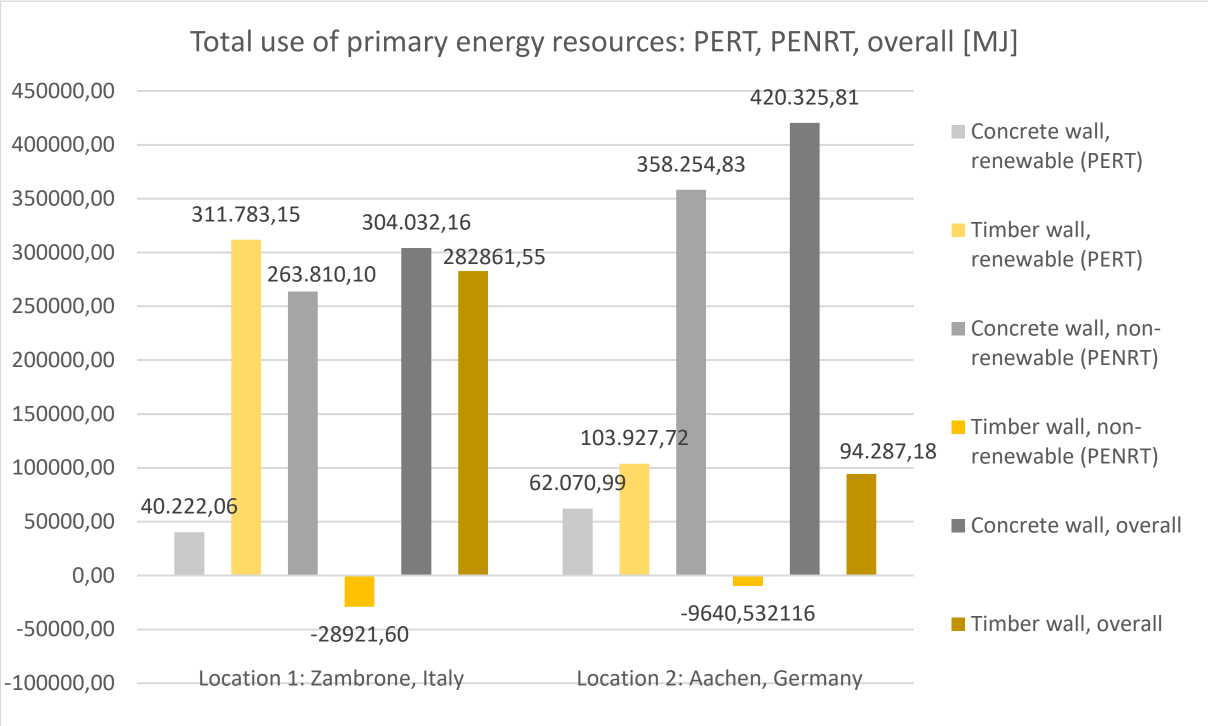


Figure 119 Total use of primary energy resources of the overall retrofits, for both timber and reinforced concrete, in both the locations.

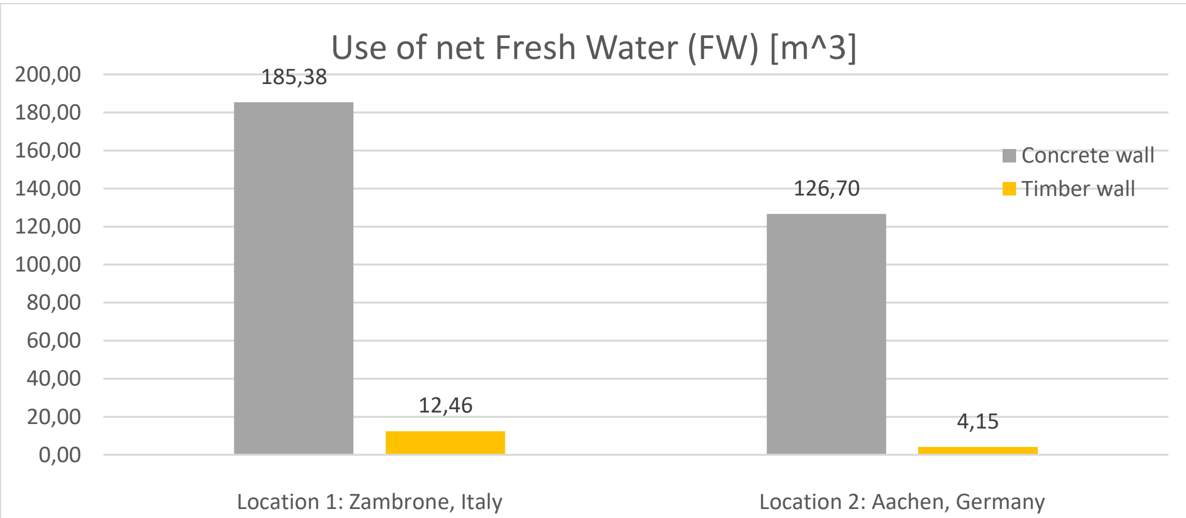


Figure 120 FW of the overall retrofits, for both timber and reinforced concrete, in both the locations.

Finally, comparing the use of net fresh water, the amount of savings that are involved with the choice of a timber retrofit are another time very high, as it can be noticed by the results shown in *figure 120.*

In the first location, the spared water for the total intervention would be **172,92 m^3**, while in the German case they would sum up to **122,55 m^3**.

4.6.7 Final considerations on the obtained results

The first observation that can rise from the results is the big difference between the single reinforced concrete wall element in the two locations. The results obtained look a bit unexpected, especially considering that in the second location the amount of reinforcement was even lower.
Therefore, those results should be treated and considered very carefully.

In any case, even if there could be some lack of precision due to the approximations made during the environmental assessment, the results are reflecting what was expected qualitatively. Namely, a lower impact due to the timber retrofit, which was predictable especially considering a single element.

In addition, many uncertainties that are intrinsically related to EPDs, such as the transport of the products to the construction site or the transport to the landfill at the end of the life-cycle, were neglected in the evaluation. In fact, those passages are accounted for in the phases A4 and C2, which were not considered in this assessment.
Besides, the stage B (use stage) of products' life cycle is basically never used in construction products' EPDs. Hence, the simplifications due to the phases neglected were many, but probably not that much impactful on the reliability of the results.

To conclude, it can be pointed out that in a real case retrofit, specific choices in terms of materials and products are undertaken. Consequently, the possible uncertainties due to the average of values given by different EPDs would not be present, as instead happened in this research, like in the case of the German RC walls. Therefore, this kind of approach could be a good way to evaluate the environmental impacts of various construction possibilities and it could represent a useful tool in order to follow the greener path.

5 Conclusions

The main conclusions about this research are related to two different aspects: the use of different seismic analysis methods and the implementation of different materials in construction. The final paragraph will explain the concept that could be implemented for a proposal of a new criterion in the Green Building Rating Systems.

5.1 Use of linear or nonlinear seismic analysis methods

The study has assessed how the results in terms of structural behavior can be different by applying diverse methods of analysis.

Starting from the modal response spectrum analysis, for sure the method can be quite fast, simple to be applied and quite easy to be understood.
As it always happens, especially in engineering, gaining in simplicity involves also some drawbacks. In the case of seismic analysis, the use of linear methods limits the possibilities in terms of materials potentialities, not accounting for their nonlinear behavior.
This obstacle can be partly overcome with the use of other parameters, like the behavior factor q, accounting for those “hidden” properties of the elements, which would not be exploited with linear analyses.

But, as it was depicted by the study, the use of more sophisticated methods can allow to obtain more precise results, exploiting in the best way possible the properties of the materials. This might come along with savings in terms of costs, environmental impacts, and possibly also social impacts. The optimization of construction materials is always a goal pursued by designers, and the application of nonlinear methods in the seismic analysis is a valuable tool in this sense.
Following the results of this thesis, the retrofit intervention was not even necessary according to the pushover analysis coupled with N2 method. This was valid even in the Italian case, where the modal response spectrum analysis showed many deficiencies of the structure with respect to the Significant Damage limit state.

Thereby, using nonlinear methods, although requiring more computational efforts and deeper knowledge of the analysis methods, could be a good solution for enhancing the sustainability in construction practices. Nevertheless, it is the duty of the designer to be always aware of what the analysis really means and to know until which point the methods can be pushed and exploited. It is important also to recognize the eventual necessity of adopting simpler and safer methods, when the knowledge of the tools and operational conditions is not deep enough to operate with confidence.

5.2 Comparison of construction materials for retrofit

Avoiding the apparent need for a retrofit intervention, by using nonlinear analysis methods would lower the impacts of the refurbishment to none. This could have happened in the case study, according to the results obtained with the pushover analysis associated to the N2 method.
But in the case in which a retrofit necessity was anyway assessed, it is important to operate with consciousness, trying to lower the environmental impacts as much as possible, in order to enhance the sustainability of the construction sector.

One of the most relevant choices that can be made by designers, always keeping in mind the context in which the retrofit is operated and the possible constraints, is the selection of appropriate construction materials and technologies.
Despite reinforced concrete is still the most widespread construction material, there are solutions which can impact much less on the environment, maybe with a higher economical effort, but still presenting many other advantages.

The study highlighted how light timber frame shear walls can keep up with the traditional reinforced concrete shear walls from the structural point of view, and they are by far more competitive regarding sustainability features. Indeed, the adopted technology involving wood, with a long-term perspective, can even have environmental impacts of equivalent CO_2 lower than zero. Instead, traditional materials like the observed reinforced concrete are not nearly as performing under a sustainability point of view. On the contrary, they weight hugely on the global environmental emissions and impacts.

Following the results discussed, it is possible to affirm that the studied light timber frame shear walls, defined as “power walls”, can provide good structural performances and at the same time have a positive impact on the environment, especially if compared with other construction technologies.

5.3 Proposal of a new criterion for Green Building Rating Systems

The certification schemes analyzed are not covering the seismic design of buildings. The concept at the base for the possible implementation of a new criterion, considering the impact of a proper seismic analysis on the building, is hereby discussed.

5.3.1 Green Building Rating Systems outside Europe

Thanks to further research implemented on the Green Building Rating Systems, some considerations about other schemes which are not common in Europe, were made.

In particular, the Japanese certification Comprehensive Assessment System for Built Environment Efficiency (**CASBEE**) has some criteria related to seismic issues. Considering the high seismicity to which Japan is subjected, it seems reasonable to find reliable criteria in this GBRS. In this protocol, the "improvement of earthquake resistance (including seismic isolation and vibration damping)" constitutes a factor to possibly earn some points. The criterion associated to it basically awards point whether the structural resistance exceeds the requirements by codes with a certain margin. The two thresholds fixed are 25% of exceedance and 50%.

A similar approach is used by the American scheme **RELi**, which is "a rating system and leadership standard that takes a holistic approach to resilient design" [114]. It recommends, among other actions, a 20% increase in mapped spectral response acceleration parameters [115].

Figure 121 Logos of CASBEE [113] and RELi [114], respectively on the left and right

Nevertheless, a similar approach could be safer in case of seismic actions applied to the structure but there would be additional environmental impacts due to a possible excessive requirement of construction materials to respect the increased requirements. This excessive exploiting of materials would involve some steps back considering the enhancement of constructions' sustainability. For this reason, a concept for the proposal of a new criterion for GBRSs was investigated accounting for other aspects.

5.3.2 Different structural materials as a factor for criterion proposal

Since the Sustainable Building Assessment Schemes are very general in their application and they were not developed for particular solutions or technologies, awarding the use of a particular material for refurbishment does not look like an option for a criterion proposal.

Despite it was shown how the timber elements can be more sustainable than concrete, the GBRSs are very general and need to consider any possible design scenario. For this reason, it would not be feasible to consider in absolute terms one solution better than the other.

Too many parameters would be involved in such an assumption. For instance, the transport of timber products to countries which do not dispose of wood, but might have other raw materials and resources, could have a higher impact than using other technologies developed in place.

Hence, the consideration of a certain construction material for the criterion proposal is out of discussion.

5.3.3 Different seismic analysis methods as a factor for criterion proposal

Contrarily to the case of different materials, using nonlinear methods does not involve particular considerations on possible impacts, if not to the computational effort sustained by the designers. As it was shown by the study, obtaining more precise structural results could help in avoiding unnecessary interventions. In a more general case of design of new structures, the use of nonlinear analysis could enable an optimization of materials, which would also cause lower environmental and economic costs.

Therefore, the new criterion that could be proposed would **develop around the use of nonlinear analysis**, which might increase the computational effort and be more time consuming in the design procedure, but can give **more accurate results than the linear methods**.

In particular, considering seismic design, the study of the structure through pushover analysis, with the structural evaluation performed with the N2 method, **could be awarded by a certain amount of points in the various Green Building Rating Systems**, or it could encouraged by protocols like Level(s).

The number of points awarded would need to be calibrated to all the other criteria already present in the certifications. Probably the **geographical context** in which the design is carried out could make a difference in the weight of a similar criterion in the certification, having a greater relevance in areas subjected to high seismicity.

Maybe with slightly more effort and less simple understanding, but still feasible, would be the consideration of the **extent of the intervention** for assigning a proportional number of points to the use of nonlinear analysis. In alternative, the score could be attributed directly by the documentation of the effectively spared CO_2 equivalent obtained with the use of nonlinear methods, instead of linear analyses.

To conclude, it seems relevant to highlight that the results of this study were considering a certain type of building, in specific locations. Hence, there is a probable need for a **generalization** of this research before really implementing a criterion based on it. However, with the results obtained both in terms of comparison of seismic analysis methods and choice of construction materials to be used, this master thesis could be a solid base for future developments and further studies.

6 Outlook on possible developments

The research carried out so far gives some inputs for the possible proposal of a new criterion related to seismic issues, to be inserted into Green Building Rating Systems.
Such protocols are well established methods, composed by many criteria ranging in different fields of building sustainability, all over its life-cycle.
Before the launch of a new protocol, or even before the addition of new concepts into the existing certifications, deep studies are carried out and committees work with the aim of verify their effectiveness.
Therefore, this master thesis has the purpose of **suggesting a possible field for further development of GBRSs**, but wider research on the topic should be carried out before considering the criterion adequate for any certification scheme.

For instance, the **results** obtained with both modal response spectrum analysis and pushover analysis coupled with the N2 method could be analyzed **for other cases**, regarding new buildings or retrofit interventions, involving different construction materials from reinforced concrete and timber. Once enough data will be gathered about the possible advantages related to the use of nonlinear analysis methods, especially in sustainability terms, the criterion could be really implemented in the sustainable building assessment schemes.

Another way in which this research topic could be enriched is the **consideration of other limit states during the structural assessment**. Indeed, this master thesis concentrated on the evaluation of results against the Significant Damage limit state seismic actions, but it might be interesting to consider the cases of Damage Limitation and Collapse Prevention limit states. Above all the former, considering a return period of 225 years for the seismic actions, may present interesting results for what concerns the avoid of damages and consequently refurbishments on existing structures. A good behavior in this sense would enhance the sustainability and the resiliency of the considered building.

A last consideration regards the **comparison between retrofit interventions and demolition** with a consequent reconstruction of the building. For sure, which one has lower environmental impacts depend on the conditions of the structure and how large the retrofit would be. In any case, it is a concept that was not investigated during this research, and the limit between a proper refurbishment and an additional environmental impact could be questioned considering both the seismic analysis methods previously implemented. Such an assessment could represent a relevant tool in terms of decision-making, and it could even have some potential as a prerequisite to be respected for applying the proposed seismic criterion in a GBRS. Indeed, if the refurbishment was the worst option in terms of sustainability, it would not be adequate to incentivize a similar intervention by awarding points for the use of nonlinear methods in the retrofit’s design.

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