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Structural analysis of existing reinforced concrete buildings

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1. INTRODUCTION

The age of the Italian building heritage combined with the total absence of seismic regulation at the time of construction are some of the factors that make it at high seismic risk. Where seismic risk means the product between dangerousness, vulnerability and exposure (R = P x V x E), where the danger measures the probability that a certain event (as the earthquake) occurs in a given time interval; the vulnerability represents the resistance characteristics of the object and, in the case of constructions, expresses the predisposition of the structure to be damaged by a seismic event; the exhibition indicates the value (not only economic) of the exhibited good, including human lives.

The fundamental aspects of the law regarding existing reinforced concrete structures will be explained, Chapter 8 of the NTC2018 will be examined in detail, with particular interest offered to safety analyses and all that derives from it, such as seismic vulnerability analysis.

The vulnerability analysis proposed by the legislation is a long and expensive process, it requires a lot of work and a large outlay of money due to the high level of detail required to achieve adequate levels of knowledge for the analysed construction. This procedure involves the use of machinery and laboratory tests for the characterization of materials and all construction details, as well as on-site surveys and historical archival research that requires dilated times and burdensome costs.

All owners of large real estate assets often need structural interventions on a small number of buildings compared to the total, perhaps due to the lack of sufficient economic funds for the entire estate, this requires to be able to identify those structures on which there's a need of intervention with greater urgency in a rapid, little invasive and economic way: with this objective the fulcrum idea of this thesis is born, to create a method, a simplified procedure for the analysis of seismic vulnerability of the existing structures in reinforced concrete. A methodology that will not replace the one defined by the NTC2018 but which will serve to have a first and approximate sampling and differentiation of the hundreds of real estate units owned by the companies.

The work is divided into phases, starting from the analysis of different projects dating back to different constructive eras, to get to the creation of a database of geometric informations depending on the age of construction, from this database are extracted the useful informations for the construction of a "virtual reference model", complementary to the real structure, for the various years taken into consideration. It ends with the validation of the new procedure and then with the comparison of the results between the project of a 1987 Turin apartment building realistically modelled on calculation software, and the corresponding virtual model built maintaining only the plan and a few other details, going to take all useful information (geometric quantities) from the previously created database.

2. EXISTING STRUCTURES IN REINFORCED CONCRETE

"An existing building is defined as the structure that has, on the date of presentation of the safety assessment or intervention project, the fully constructed structure", that's the definition of existing structure given by the Italian standards for construction.

Italian cities are characterized by very different types of buildings, from the monumental building built in Roman times, to historical buildings which still today constitute the nucleus of life at the civil level for residences and small businesses; to newer buildings in masonry or reinforced concrete, sometimes even reworked over time with non-criterion interventions.

An ancient building heritage, the absence of seismic criteria in the design as not required at the time of construction, architectural modification interventions without the proper structural checks, are some of the factors that make the Italian building heritage at high seismic risk.

With reference to the 2001 census, out of a total of 27,291,993 structures (corresponding to 11,226,595 buildings), more than 60% were found to have been built before 1971. The distribution is fairly homogeneous on the national territory, with a peak higher than the 80% in Liguria, followed by Piedmont and Tuscany and a minimum around 50% in Sardinia.

Regarding the construction type, the 2001 census tells us that the Italian heritage consists of 6,903,982 brick buildings (61.50%), 2,768,205 in reinforced concrete (24.66%) and 1,554. 408 with other features.

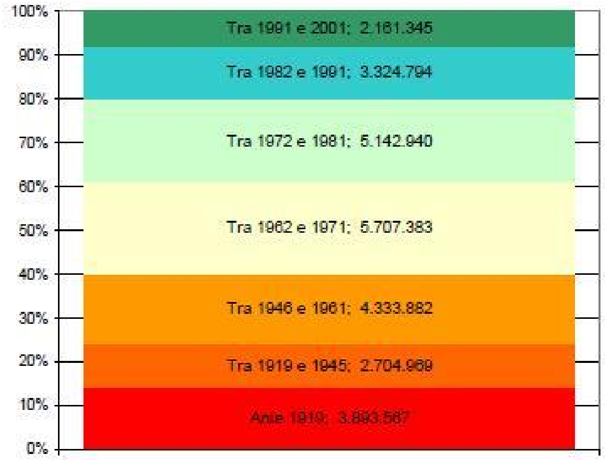


Figure 1-classification by time of Italian buildings

The identification of the seismic zones, in Italy, took place at the beginning of the 20th century through the instrument of the royal decree, issued following the destructive earthquakes of Reggio Calabria and Messina on December 28th 1908. Since 1927 the affected areas have been divided into two categories, in relation to their degree of seismicity and their geological constitution. Therefore, the seismic map in Italy was nothing but the map of the territories affected by the strong earthquakes after 1908, while all the territories struck before that date , most of the seismic areas of Italy, were not classified as seismic and , consequently, there was no obligation to build in compliance with anti-seismic regulations. The list originally consisted, therefore, in the cities of Sicily and Calabria seriously damaged by the earthquake of 1908, which was modified after each seismic event simply adding the new damaged municipalities.

A turning point was given by the law of 2 February 1974, n. 64, concerning *"Measures for constructions with particular prescriptions for seismic zones",* which completely replaced the law of 25 November 1962, n. 1684, as well as of the law of 5 November 1971, n. 1086, laying down rules for the regulation of reinforced, normal and prestressed concrete conglomerates and metal structures.

In fact, only in 1974, through the law n. 64, a new national seismic regulation was established which established the reference framework for the seismic classification methods of the national territory, as well as the drafting of technical standards. Then, only in 2008, to make anti-seismic design mandatory throughout Italy through the Technical Standards for Construction.

These data provide a clear measure of the antiquity of Italian residential assets and the inadequacy of modern anti-seismic design requirements. Add to this the change in seismic zoning, and consequent application of adequate design standards, that the Italian territory has undergone in recent years.

Italy is a country characterized by high seismicity that is distributed throughout the country with different levels of severity. The most recent regulations have acknowledged the presence of a widespread seismic hazard that does not spare areas that the previous classifications declared exempt from a probability of seismic events of some importance. This circumstance has accentuated the presence on the Italian territory of buildings, even fairly recent ones, that although constructed according to the law, do not meet the requirements of the seismic zones.

2.1 MAIN PROBLEMATICS OF EXISTING BUILDINGS

Fortunately, the above does not mean that, following an earthquake, our entire building stock is not able to withstand since, the good building rules have been applied since ancient times, if around 185 BC. we wrote:

"Exactly as the well connected truss that tightens the walls does not break down if there is an earthquake, so a firm heart in the well-matured decision is not discouraged at the critical moment." (Ben Sirach, Ecclesiastes, chap. 22, ver. .16).

Buildings with a reinforced concrete structure characterize the Italian construction heritage over the last 80 years and are widespread throughout the national territory, even in small towns. Depending on the location of the building and the period of construction, this could have been built in compliance with the seismic regulations in force at the time, which, over time, have improved the degree of analysis and reduced the vulnerability of the buildings themselves. The evolution of the seismic hazard assessment of areas of the Italian territory over the years can lead, for new buildings, to the risk of being subjected to a higher seismic action than expected in the design phase. In the case of the L'Aquila earthquake, the damage analysis showed the high seismic vulnerability of the buildings affected, despite the seismic actions associated with the most significant recordings for the central area of L'Aquila were comparable to those expected for the area from regulations that were to apply at the time of construction. The shortcomings that have most influenced the seismic response of buildings and which can be taken as a reference for the analysis of other similar situations, and therefore referable to the entire Italian architectural heritage, are:

- Configuration of the structure inadequate for the structure of the building;
- Arrangement of non-structural elements inadequate to the geometric characteristics of the structure (presence of "soft plan");
- Presence of frames in only one direction;
- Presence of "strong" beams and "weak" columns;
- Presence of squat columns, crisis due to shear force on the columns with reduced free height due to partial interaction with the infill walls;
- Inadequate construction details: bracket of columns and knots, lengths of anchoring and overlapping of reinforcements;
- Insufficient quality of the materials, first of all of the concrete, both in terms of resistance and casting;
- Risk of local collapse for non-structural elements;
- Overturning of the infill panels out of the plane;

- Low resistance of the foundation system that could lead to a global collapse of the structure, often due to the fact that the foundation beams are not well connected one to the other;
- Absence of adequate seismic joints between neighbor structures;
- Absence of rigid plane;
- Wrong reinforcement arrangement and percentage of steel not adequate for the purposes;

The remarkable influence of the regularity of the geometry and the distribution of the masses of a building, regardless of the material used, on the seismic response of the buildings is now widely recognized, so much so that the most important regulations in the world explicitly consider the effects of regularity irregularity due to both the structural configuration and the unfavorable arrangement of non-structural elements. The regularity, to be analyzed both in plan and in height, is linked to a series of different factors that can be identified in simplicity, symmetry, compactness, distribution of resistances and stiffness, distribution of structural elements and distribution of non-structural elements.

To have a better comprehension of what it’s wrote, we can proceed with an example: a very particular case of irregularity is that of a building of L'Aquila, characterized by a stepped configuration, a very articulated volume and a completely open ground floor. Even structurally, the building was characterized by a non-regular structure, consisting of flat frames arranged in the transverse direction in correspondence of the first four floors and frames arranged in longitudinal direction in correspondence of the upper floors. According to the project, the parallel frames had to be connected to each other by thick beams arranged orthogonally to the frames themselves, but no trace was found of these beams in the inspections carried out on the building.

Figure 2-collapsed building in L'Aquila-example 1

The cases of open plan and presence of frames in only one direction are among the possible causes of irregularity, together with, for example, not compact and non-symmetrical plan configurations and asymmetrical positions of stairwells and elevators.

Figure 3-collapsed building in L'Aquila-example 2

The presence of frames in only one direction is typical of buildings designed for vertical loads only (as required in the case of the absence of a seismic zone or before the seismic rules) and is linked to the mono-directional texture of the floors; in the frames on which the slab does not rest directly, thick beams may be present, but in some cases the transverse beams are absent. A situation is therefore configured in which in the direction parallel to the weaving of the floors, the resistance to horizontal seismic actions is very low or absent.

Another feature that can weaken the earthquake resistance of a building is the presence of an open or weak floor, which is configured in the case of the pilot at the ground floor. These conditions occur both in design and later, for new functional requirements and subsequent restructuring. In these cases it is very important to analyze the architectural changes that are performed not only in structural terms of resistance to vertical loads, but also and above all of a double effect, of displacement of the masses and of weakening of the transverse resistance due to absence, partial or total, of plugging. The absence of infill panels on the ground floor has caused many collapses at various earthquakes in the last 40 years (San Salvador 1986, Athens 1999, L'Aquila 2009). The figures below show a case occurred during the L'Aquila earthquake of 2009, for which it is clear that the presence of an open plan constituted the element of weakness of the structure with the consequent collapse of the wing sustained by this and the collapse of the infill at the same level in the contiguous block, with almost irrelevant damage in the rest of the building both on the lower floors and on the upper floors.

Figure 4-collapsed building in L'Aquila-example 3

Negative effect can also be caused by the presence of "squat" columns, or rather columns having a free length of inflection reduced by the presence of infill only for a band: the presence of infill for a limited height works by contrast where it is present, subjecting to concentrated shear stress in the short free stretch the column, generally not verified or armed for this type of solicitation.

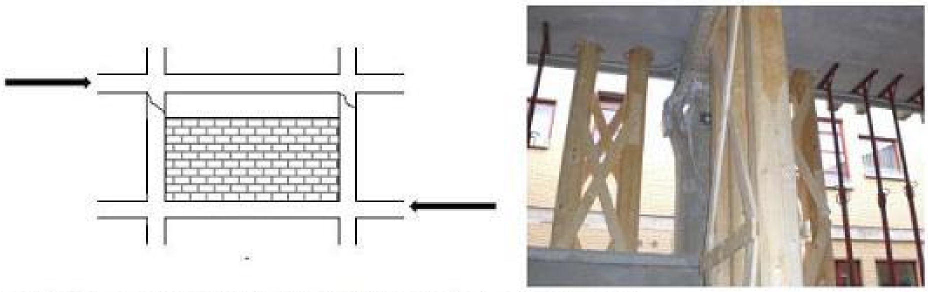


Figure 5-scheme of a "squat column"

To underline important concept of what's written, we have had a real revolution in the design of reinforced concrete buildings: from a design where actions were represented by the only vertical loads (gravity), to a design that takes into account also other types of actions, those are the horizontal ones and so the seismic force that a structure must supply in case of earthquakes, for these reasons law and criteria had to be changed during a long process that is going on also in the present days.

3. SAFETY ASSESSMENT

As stated in chapter 8 of the NTC2018, the safety assessment of an existing structure is a quantitative procedure, aimed at determining the extent of the actions that the structure is able to sustain with the minimum level of security required by the legislation in force.

The increase in the level of safety is obtained by modifying the global structural conception with interventions also local. The safety assessment must make it possible to determine whether:

- the use of the building can continue without interventions;
- the use must be modified (downgrading, change of destination and imposition of limitations and cautions in the use);
- it is necessary to increase structural safety, through interventions.

The safety assessment must be carried out only when one of the following situations occurs:

- great reduction of the resistant or deformation capacity of the structure or of some of its parts due to: significant degradation and decay of the mechanical characteristics of the materials, significant deformations consequent also to problems in the foundation;
- damage caused by environmental actions (earthquake, wind, snow and temperature), by exceptional actions (impacts, fires, explosions) or by anomalous operating and use situations;
- proven serious design or construction errors;
- change of the intended use of the building or parts of it, with significant variation of the variable loads or passage to a higher class of use;
- exeshearion of not explicitly structural interventions, if they interact, even partially, with elements having a structural function and, consistently, reduce their capacity and modify their rigidity;
- whenever structural interventions are carried out (repair or local interventions, improvement or adaptation);
- works carried out in the absence or discrepancy of the housing title, where necessary at the time of construction, or in contrast with the technical standards for buildings in force at the time of construction.

If the circumstances referred to in the preceding points concern limited portions of the construction, the safety assessment can also be carried out only on the elements involved and on those interacting with them, bearing in mind their function in the structural complex, given that the changed local conditions do not affect substantially on the overall behavior of the structure. In assessing safety, to be carried out whenever structural improvements or adaptations are performed, the designer must explain in a specific report, expressing them in terms of the relationship between capacity and demand, the levels of safety prior to the intervention and those achieved with it.

If it is necessary to carry out the safety assessment of the construction, the verification of the foundation system is mandatory only if there are conditions that can give rise to phenomena of global instability or if one of the following conditions occurs:

- in the building there are important failures attributable to subsidence of foundations or disruptions of the same nature that have occurred in the past;
- are possible phenomena of overturning and sliding of the construction due to: unfavorable morphological conditions, modifications made to the terrain profile near the foundations, project seismic actions;
- liquefaction phenomena of the foundation soil due to project seismic actions are possible.

In order to verify the existence of the before mentioned conditions, reference will be made to the documentation available and specific investigations may be omitted only if, in the explicitly justified judgment of the professional in charge, sufficient knowledge is available on the volume of significant land and on the foundations to carry out previous evaluations.

The evaluation of the safety and the planning of the interventions on the existing buildings can be carried out with reference to the SLUs only, except for the class of use IV constructions, for which the verifications to the SLE specified in the legislation are also required; in the latter case, reduced performance levels may be adopted. For the seismic combination the verifications to the SLU can be performed with respect to the condition of safeguarding human life (SLV) or to the condition of collapse (SLC).

In the verifications with respect to the seismic actions the safety level of the construction is quantified through the ratio ξ_E between the maximum bearable seismic action of the structure and the maximum seismic action that would be used in the project of a new construction; the extent of the other actions simultaneously present is the same as for new buildings, except for what emerged with regard to permanent vertical loads following the investigations conducted and except for the possible adoption of specific measures restricting the use of the building and, consequently, on the variable vertical loads.

It is convenient remembering the definition of "risk" as the product between Hazard, Vulnerability and Exposure (R = P x V x E), where the Dangerousness measures the probability that a certain event (es the earthquake) occurs in a certain interval of time; Vulnerability represents the resistance characteristics of the object and, in the case of constructions, expresses the predisposition of the structure to be damaged by a seismic event; the Exposition indicates the value (not only economic) of the exhibited good, including human lives.

The restriction of use can change from portion to portion of the construction and, for the i-th portion, is quantified through the ratio ξ_I between the maximum value of the vertical variable overload that can be supported by that part of the construction and the value of the variable vertical overload that would be used in the design of a new building. It is necessary to adopt measures restricting the use of the building and to proceed with improvements or adaptations in the event that the verifications relating to actions controlled by man, mainly permanent loads and other service actions, are not met.

3.1 SESMIC RISK AND SEISMIC VULNERABILITY

The seismic risk is an indicator that allows us to evaluate the set of possible effects in terms of expected damage that an earthquake can produce in a given time interval, in a given area, in relation to its probability of occurrence and its degree of intensity. It is the result of the interaction between the natural event (earthquake) and the main characteristics of goods and lives exposed.

The seismic risk of a territory can be schematically evaluated as a combination of danger (P), vulnerability (V) and exposure (E):

$$R = P \times V \times E.$$

Seismic hazard is defined as the probability that, in a given area and in a certain interval of time, an earthquake will occur that exceeds a threshold of intensity, magnitude or fixed peak acceleration; the danger is a physical characteristic of the territory and represents the frequency and strength with which earthquakes occur (seismicity of the site).

The exhibition indicates the possibility that a territory suffers more or less damage in economic terms, loss of human lives and architectural and cultural assets.

The seismic vulnerability is the predisposition of a building to suffer damage and collapse. The more vulnerable a building is (by type, inadequate design, poor quality of materials, construction methods and poor maintenance), the greater the consequences on the structure. In order for buildings to have a low vulnerability, the current legislation requires compliance with anti-seismic criteria, requiring that structures show a ductile response to telluric stress.

If on the one hand it is not possible to act to change the seismic hazard of a territory and very little can be done to modify the exposure to seismic risk, on the other hand we have many possibilities to reduce the vulnerability of buildings and thus implement policies of prevention and safety of buildings.

The procedure for assessing the safety of existing buildings proposed by the Technical Standards has the purpose of estimating the vulnerability of existing structures and studying the most appropriate restoration interventions.

In technical terms the seismic vulnerability of a structure is represented by an indicator that relates the capacity of the structure to resist and the request in terms of resistance or displacement of the earthquake.

The procedures for assessing the seismic vulnerability of buildings can be carried out with different degrees of depth and complexity of calculation: from more qualitative estimates, based on the survey of the main characteristics of the building constituent elements, to complex numerical analyzes using calculation methods that they can be linear and non-linear.

The design process for the assessment of seismic vulnerability and therefore the estimate of the seismic vulnerability index of a building follows the NTC and can be summarized in the following steps:

- Historical-critical analysis: it is the tool that guides the designer in the reconstruction of the current state of stress in the light of the changes and events that have affected the building over time.
- Cognitive survey: the current state of the construction is defined by plane-altimetric, structural surveys and the damage and deformation state of the structure.
- Mechanical characterization of materials: evaluation of the resistance capacity of materials through surveys carried out on site or in the laboratory.
- Definition of knowledge levels and consequent confidence factors: reductive coefficients of the mechanical properties of materials are gradually defined as the degree of depth of the investigations increases; it goes from the level of knowledge 1 (lc1), the minimum allowed, to the level of knowledge 3 (lc3), the maximum allowed.
- Structural analysis and determination of the vulnerability of the existing structural system.
- Proposal of possible interventions and evaluation of the optimal cost / benefit ratio.

In fact one of the main novelties of the NTC2018 concerns the introduction of the ζ_E coefficient to evaluate the seismic vulnerability of an existing structure.

The seismic vulnerability index, or rather the seismic risk indicator, is a numerical value that is used to summarize the results of a seismic vulnerability assessment, at least from a numerical point of view.

$$\zeta_E = \frac{maximum\ horizontal\ action\ suistanable\ from\ the\ existing\ structure}{project\ seismic\ action\ in\ case\ of\ a\ new\ construction}$$

The seismic risk indicator is given by the ratio between the building's resistant capacity and the demand in terms of resistance or displacement envisaged by the Technical Regulations, therefore the outcome of the verification is positive (building that meets the requirements of the Technical Regulations) if the indicator is greater than or equal to 1, negative if less than 1.

In the context of a safety assessment, in relation also to the construction type of the building, the checks to be carried out are different and the vulnerabilities can be multiple. The indicator therefore summarizes the numerical vulnerabilities in a single "easy to read" value, which is not however to be considered exhaustive since in the numerical checks vulnerabilities such as the fall of chimneys or other non-structural elements are not included.

To underline the concepts the coefficient ζ_E is given by the ratio between the maximum bearable seismic action of the structure (the capacity of the structure) and the seismic action of the project that would be used in the case of a new construction (the demand envisaged by the Regulation).

With the new NTC 2018 the vulnerability index has been assigned the ζ_E nomenclature and limits have been set on the values it can assume depending on the type of intervention.

3.1.1 CLASSIFICATION OF STRUCTURAL INTERVENTION

The Italian legislation distinguishes between the following categories of intervention:

- repair or local interventions: interventions involving individual structural elements and which, in any case, do not reduce pre-existing safety conditions;
- improvement interventions: interventions aimed at increasing pre-existing structural safety, without necessarily reaching the safety levels set by the standard;
- adjustment interventions: interventions aimed at increasing pre-existing structural safety, achieving the safety levels set by the standard.

For improvements and adaptations, the exclusion of provisions in the foundation must in all cases be explicitly justified by the designer, through a verification of the suitability of the foundation system. If the intervention provides for the insertion of new elements that require specific foundations, the latter must be verified with the general criteria as required for new buildings. For assets of cultural interest falling in areas declared at risk of seismic activity, pursuant to paragraph 4 of the art. 29 of the Legislative Decree of 22 January 2004, no. 42 "Code of cultural heritage and landscape", it is in any case possible to limit oneself to improvement interventions by carrying out the related safety assessment.

3.1.1.1 REPAIR OR LOCAL INTERVENTION

Interventions of this type will concern individual parts and elements of the structure. They must not significantly change the overall behavior of the building and are aimed at achieving one or more of the following purposes:

- restore, with respect to the configuration prior to damage, the initial characteristics of damaged elements or parts;
- improve the resistance and ductility characteristics of elements or parts, even undamaged;

- prevent local collapse mechanisms;
- modify an element or a limited portion of the structure.

The project and the safety assessment can only refer to the parts or elements involved, documenting the structural deficiencies found and showing that, with respect to the configuration prior to damage, degradation or variant, substantial changes are not produced to the behavior of the other parts and of the structure as a whole and that the interventions do not lead to a reduction in pre-existing security levels.

The safety report may be limited only to the parties involved in the intervention and to those interacting with them, it must document the structural deficiencies found, resolved or persistent, and indicate any consequent limitations on the use of the building. In the case of local reinforcement interventions, aimed at improving the mechanical characteristics of structural elements or limiting the possibility of local collapse mechanisms, it is necessary to assess the increase in the level of local security.

3.1.1.2 IMPROVEMENT INTERVENTION

The safety assessment and the intervention project must be extended to all the parts of the structure potentially affected by behavior changes, as well as to the structure as a whole. For the seismic combination of actions, the value of ζ_E may be less than unity. With the exception of specific situations relating to cultural heritage, for class III buildings for scholastic use and class IV the value of ζ_E, following the improvement interventions, must in any case be no less than 0.6, while for the remaining class III buildings and those of class II the value of ζ_E, again following the improvements, must be increased by a value not less than 0.1. In the case of interventions that involve the use of insulation systems, to check the insulation system, one must have at least $\zeta_E = 1.0$.

3.1.1.3 ADJUSTEMENT INTERVENTION

The construction adjustment intervention is mandatory when it means:

- raising the building (A);
- expand the construction through works structurally connected to it and such as to significantly alter its response (B);
- make changes in the intended use that lead to increases in global vertical loads in the foundation of more than 10%, assessed on the basis of the characteristic combination, including only gravity loads. The obligation to proceed to the local verification of the individual parts and elements of the structure remains, however, even if they involve limited parts of the building (C);

- carry out structural interventions aimed at transforming the building through a systematic series of works that lead to a structural system different from the previous one; in the case of buildings, carry out structural interventions that transform the structural system through the use of new vertical load-bearing elements on which at least 50% of the overall gravitational loads refers to the individual floors (D).
- make changes to the use of the class that lead to class III buildings for school or class IV use. In any case, the project must refer to the entire construction and must report the checks of the entire post-intervention structure (E).

In cases (A), (B) and (D), for the verification of the structure, one must have ζ_E = 1.0. In cases (C) and (E) we can assume ζ_E = 0.80. The obligation to proceed to the local verification of the individual parts and elements of the structure remains, however, even if they involve limited parts of the building. A change in the height of the building due to the construction of summit curbs or changes in the roof that do not increase the living area is not considered an extension, based on condition (A). In this case it is not necessary to proceed with the adjustment, unless one or more of the conditions referred to in the previous points are present.

3.2 DEFINITION OF THE REFERENCE MODEL FOR ANALYSIS

"In existing buildings the situations that can be concretely found are the most different and it is therefore impossible to provide specific rules for all cases. Consequently, the safety assessment model must be defined and justified by the designer, on a case-by-case basis, in relation to the expected structural behavior, taking into account the general indications set out below.", that's what the Italian standards for construction (NTC2018) states.

The main problem we have to face with an existing structure is the lack of information, often both about the structure and the used materials, without considering the construction details as anchoring length and overlapping of reinforcement.

How to face the analysis or the assessment of an existing building?

How to design a project of seismic adjustment?

During past years designer used two different approaches: on one hand their analysis where based on too conservative assumptions, and so constructor need to face with high costs for the intervention; while on the other hand they used a very low conservative approach, and this leads to a bad level of security.

This fact underlines the necessity to have an in-depth study of the current situation of the studied existing building, to have the greater and higher possible number of information about structural components, non structural elements and used materials.

Here what the Italian standard for construction says in the Chapter 8 :

"For the purposes of a correct identification of the existing structural system and its state of stress it is important to reconstruct the realization process and the subsequent modifications suffered over time by the building, as well as the events that affected it"

To do this we can follow a certain procedure:

- Geometric survey (plans, elevations, sections) and structural survey (identification of the type of load-bearing structure: frame, shear wall, mixed type);

- Survey for the construction details (size of structural elements, quantity, arrangement and type of steel reinforcement bars, connections, floors, roofs);

- Definition of mechanical properties of the materials used (compression strength of concrete, tensile strength of reinforced concrete bars, masonry strength of internal and external walls);

- Definition of a numerical model representative of the actual current state of the analyzed building.

3.2.1 HISTORICAL-CRITICAL ANALYSIS

Knowledge of the history of a building is an indispensable element, both for assessing current safety and defining interventions and forecasting their effectiveness. The analysis begins with finding all the documents available on the origins of the building such as, for example, drawings and design reports of the first construction of the building and any subsequent interventions, elaborations and surveys already produced, possible testing reports and concerns:

- the construction period;
- the techniques, the construction rules and, if existing, the technical standards of the construction period;
- the original form and subsequent modifications;
- the traumas suffered and the alterations of the surrounding conditions;

- deformations, instability and cracking patterns, with indications, where possible, of their evolution over time;
- previous consolidation interventions;
- the urban and historical aspects that regulated the development of the building aggregate of which the building is part.

Generally speaking, it is also useful to know the pathologies or building deficiencies highlighted by similar buildings in terms of type and time of construction.

Ultimately, this phase must allow us to interpret the current condition of the building as the result of a series of static events and transformations that overlapped over time.

To correctly identify the structural system and its state of stress, it is important to reconstruct the construction process and the subsequent changes that the building has undergone over time, as well as the events that affected it.

The first point is to have an historical analysis of the building and so we need to rebuilt the realization process and all the modifications that the structure had during time, to do this we have to proceed with archival research aimed at identifying the building history, the different building phases, the urban and historical development of the neighborhood where the building is located, the subsequent architectural and structural changes.

After that we also need a study on all the earthquakes those affect the area of interest to have an effective index on the behavior of the structure.

3.2.2 STRUCTURAL-GEOMETRIC SURVEY

Now we can proceed with the geometric and structural survey: we need to find the original schemes (in the municipal offices) of the structure, as plans and sections of the structural components, and to take samples where possible; and then proceed with an in situ survey to asses what's written or drawn in the projects and testing the sampled material.

The geometric-structural survey will have to refer to the overall geometry, both of the construction and of the structural elements, including the relationships with any structures in adherence. In the survey the changes that occurred over time, as derived from the historical-critical analysis, will have to be represented and recognized.

The survey must identify the resistant organism of the construction, also bearing in mind the quality and the state of conservation of the materials and the constituent elements. Disruptions must also be detected, in place or stabilized, paying particular attention to the identification of crack patterns and damage mechanisms, those could be easily identified in the structure.

It is the photograph of the de facto state that claims to be closer to being an X-ray of critical elements and construction details. The survey contains the characterization of the structural elements, of the non-structural ones, of the state of conservation of the construction, of the crack pattern and defines the survey plan to be carried out on the structure.

The NTC 2018 give particular emphasis to the construction details, placing them at a level of greater importance than the mechanical characteristics of the materials. The survey of construction details is aimed at obtaining the following information:

- quantity of longitudinal reinforcement in beams, columns, walls and its arrangement;
- quantity of bent reinforcing bars that contribute to the shear strength, present in the beams;
- quantity and details of transversal reinforcement in the critical areas and in the beam-column nodes;
- amount of longitudinal reinforcement that contributes to the negative moment of T-beams, present in the floors;
- support lengths and constraint conditions of the horizontal elements;
- thickness of the covers;
- length of the overlapping areas of the bars and their anchors;

3.2.3 EFFECTIVE RESISTANCE OF MATERIALS

The Chapter 8 of the Italian technical standards for construction (NTC2018) states that:

"To achieve adequate knowledge of the characteristics of the materials and their degradation, we will rely on documentation already available, on-site visual checks and on experimental investigations".

For what concerns materials we need to prove that the real in situ characteristics of concrete and steel are similar or equal to those given by the designer, to do this we proceed with some tests, those could be destructive or non-destructive.

The purpose of investigating materials is to determine the mechanical, physical and chemical characteristics of the materials making up the various structural elements of buildings in general. The purpose of this investigation is to obtain information on the "health" of the structures. This need for investigation is determined by preserving the structures from degradation phenomena that can be chemical (interaction and attack of chemical agents), physical (thermal cycles, thermo-hygrometric variations), accidental (fires, explosions, seismic events). In accordance with the regulations in force in the construction field, prior to any type of intervention, knowledge of the structure is essential, which can be obtained at various levels, determined by the study of geometry, structural details and materials. The regulations state that in the absence of specific original project drawings it is essential to carry out in-situ surveys and checks, defined by type of construction and level of knowledge established.

3.2.3.1 DESTRUCTIVE TESTS

EXTRACTION OF CONCRETE SAMPLES (CORES):

This test, regulated by EN 12504 - 1: 2002, is performed in correspondence of the main structural elements of a structure (columns, beams in elevation and foundation, reinforced walls) and is useful for the evaluation of the average strength of the concrete by compression test on cylindrical specimens of suitable dimensions

(H / D> 1) taken directly from the analyzed element by means of a suitable core drill.

Figure 6-extraction of a core

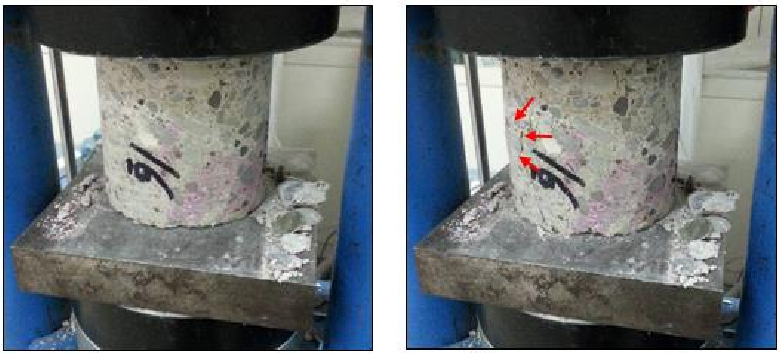


Figure 7-compression test on concrete

CARBONATION TESTS

It consists of a colorimetric test, regulated by EN 9944: 1992, performed on a concrete sample taken directly from the analyzed element. A solution is used to determine the depth of carbonation responsible for the corrosion of the reinforcing bars.

EXTRACTION OF REINFORCEMENT BARS

This test allows the evaluation of the mechanical characteristics of the steel by tensile test of specimens of reinforcing bars present in the structural elements of a building (columns, beams in elevation and foundation, reinforced walls, floor joists). The phases of this test are: the demolition of the concrete cover layer, the welding of the restoration bar and the subsequent shearting of the sample to be analyzed (UNI EN ISO 6892: 2009)

VISUAL INVESTIGATIONS FOR CHARACTERIZATION OF STRUCTURAL ELEMENTS

It is carried out on columns, elevation and foundation beams and allows the determination of the arrangement, diameters and state of preservation of reinforcing bars. The procedure consists in removing the layer of concrete cover and in the various measurements and surveys useful for reconstructing the investigated element.

3.2.3.2 NON-DESTRUCTIVE TESTS

SCLEROMETRIC TESTS

Returns the average compression value of the concrete through the use of a particular instrument called the sclerometer. This instrument is formed by a cylinder with a steel striking mass, driven by a spring that contrasts a percussion rod in direct contact with the surface to be analyzed. The sclerometer is placed on the surface and then pressed until it reaches the rebound index through which the concrete compressive strength is obtained (UNI EN 12504-2)

Figure 8- sclerometer instrument

ULTRASONIC TESTS

Through this non-destructive test the strength of the concrete is estimated starting from the ultrasonic waves. In particular, through the propagation speed of these waves, the speed of impulse diffusion is calculated and with good approximation it is possible to go back to the mechanical strength of the concrete considering the correlation between Young's modulus, wave propagation speed and the strength of the concrete itself. The standard that defines the correct exeshearion of the test is UNI EN 12504-4.

SONREB METHOD

This method of investigation is based on the combination of ultrasonic tests (SONic) and rebound tests (REBound) and allows to trace the strength of the concrete starting from the speed of the ultrasonic waves and the rebound index. The combination of the two methods has numerous advantages, reducing the percentage of errors that would occur if the two methods were used individually. In literature there are various expressions through which it is possible to estimate the strength of the concrete starting from the rebound index and the speed of propagation of the ultrasonic waves.

PACOMETRIC SURVEY

The pacometer is the digital instrument that allows to detect, in a non-destructive way, the presence of reinforcements inside reinforced concrete structural elements. Its use is also necessary for the preparation of destructive tests that would be influenced by the presence of steel bars (core drilling, bar extractions)

Figure 9- pacometer instrument

THERMOGRAPHIC SURVEY

This technique is very useful in the context of structural investigations, in fact it is possible to establish at best a plan of destructive investigations, given the return of thermal images at full field that allow the identification of defects, cavities, hidden construction elements, variation of warping thicknesses of the floors. The current technology also allows the localization of all the discontinuities that can cause structural malfunctions.

3.2.4 LEVEL OF KNOWLEDGE

In the past, identical safety coefficients were used for the design of new buildings and the rehabilitation of existing buildings, despite the different level of knowability, both in terms of structure and materials. With O.P.C.M. (ordinance president of the council of ministers) 3274/2003 and O.P.C.M. 3431/2005, Ministerial Decree 14/01/2008 there is the introduction of knowledge levels that allow to assume different safety coefficients, reductive of the resistances, (confidence factors) according to the degree of uncertainty on the building.

To each level of knowledge corresponds a Confidence Factor to be used in the analysis of the structure to reduce the resistance of the materials or amplify the actions.

On the basis of the analyzes carried out in the cognitive phases reported above, the "levels of knowledge" of the various parameters involved in the model will be identified and the related confidence factors defined, to be used in safety audits. For the purposes of choosing the type of analysis and the values of the confidence factors, the following three levels of knowledge are distinguished, sorted by increasing information:

- LC1: intended as achieved when the historical-critical analysis has been carried out commensurate with the level considered, the geometry of the structure is known based on the original drawings (making a visual sample survey to verify the actual correspondence of the built to the drawings) or to a survey, since construction drawings are not available, the construction details have been derived on the basis of a simulated project and with limited in-situ surveys on the reinforcements and on the connections present in the most important elements (the data collected must be such as to allow local checks resistance), since no information is available on the mechanical characteristics of the materials (coming from construction drawings or test certificates) the usual values of the construction practice of the time have been adopted, validated by limited in-situ tests on the most important elements ; the corresponding confidence factor and FC = 1.35. Safety evaluation is generally performed by linear, static or dynamic analysis; the information collected must allow the development of a suitable structural model.

- LC2: intended as achieved when the historical-critical analysis has been carried out commensurate with the level considered, the geometry of the structure is known based on the original drawings (making a visual sample survey to verify the actual correspondence of the built to the drawings) or to a relief, the construction details are known, or partially from the original construction drawings integrated by limited in situ investigations on the reinforcements and on the connections present in the most important elements, or following an extensive in situ investigation (the data collected must be such as to allow, in the case of a linear analysis, local resistance checks, or the development of a non-linear structural model, the mechanical characteristics of the materials are known based on construction drawings, supplemented by limited in situ tests (if the values obtained from the in situ tests are less than the corresponding values indicated in the project drawings, extensive tests are performed i n situ), or with extensive in situ tests; the corresponding confidence factor and FC = 1.2. Safety evaluation is performed using linear or non-linear analysis methods, static or dynamic; the information collected on the dimensions of the structural elements, together with those concerning the structural details, must allow the development of a suitable structural model.

- LC3: intended as achieved when the historical-critical analysis has been carried out commensurate with the level considered, the geometry of the structure is known based on the original drawings (making a visual sample survey to verify the actual correspondence of the built to the drawings) or to a relief, the construction details are known, or from the original construction drawings integrated by limited in situ investigations on the reinforcements and connections present in the most important elements, or following an exhaustive in situ investigation (the data collected must be such as to allow, in the In the case of a linear analysis, local resistance checks, or the development of a non-linear structural model, the mechanical characteristics of the materials are known based on the construction drawings and original test certificates, supplemented by limited tests in situ (if the values obtained from the in situ tests are less than the corresponding values indicated in the original certificates of test, exhaustive tests are performed in situ), or with exhaustive in situ tests; the corresponding confidence factor and FC = 1. Safety evaluation is performed using linear or non-linear analysis methods, static or dynamic; the information collected on the dimensions of the structural elements, together with those concerning the structural details, must allow the development of a suitable structural model.

The aspects that define the levels of knowledge are:

- structure geometry,
- construction details,
- material properties,
- connections between the different elements
- and their presumable collapse modalities.

Tabella C8.5.IV – *Livelli di conoscenza in funzione dell'informazione disponibile e conseguenti metodi di analisi ammessi e valori dei fattori di confidenza, per edifici in calcestruzzo armato o in acciaio*

Livello di conoscenza	Geometrie (carpenterie)	Dettagli strutturali	Proprietà dei materiali	Metodi di analisi	FC (*)
LC1	Da disegni di carpenteria originali con rilievo visivo a campione; in alternativa rilievo completo ex-novo	Progetto simulato in accordo alle norme dell'epoca e ***indagini limitate*** in situ	Valori usuali per la pratica costruttiva dell'epoca e ***prove limitate*** in situ	Analisi lineare statica o dinamica	1,35
LC2		Elaborati progettuali incompleti con ***indagini limitate*** in situ; in alternativa ***indagini estese*** in situ	Dalle specifiche originali di progetto o dai certificati di prova originali, con ***prove limitate*** in situ; in alternativa da ***prove estese*** in situ	Tutti	1,20
LC3		Elaborati progettuali completi con ***indagini limitate*** in situ; in alternativa ***indagini esaustive*** in situ	Dai certificati di prova originali o dalle specifiche originali di progetto, con ***prove estese*** in situ; in alternativa da ***prove esaustive*** in situ	Tutti	1,00

(*) A meno delle ulteriori precisazioni già fornite nel § C8.5.4.

Figure 10- table 8.5IV of NTC2018

To underline the importance of this factors, an in-depth study of the state of affairs leads us to have minor corrective factors, factors that we will apply to the resistances of our materials and of our structure and therefore possible lower costs as well as greater safety in the opposite case.

3.2.4.1 CONSTRUCTIVE DETAILS CHECK

To access the real level of information we have we can follow a certain scheme:

- LIMITED

They are used to verify the correspondence between the reinforcements or the characteristics of the connections actually present and those reported in the construction drawings, or obtained through the simulated project. The quantity and arrangement of the reinforcement is verified for at least 15% of the elements

- EXTENDED

They are used when the original construction drawings are not available as an alternative to the simulated project followed by limited checks, or when the original construction drawings are incomplete. The quantity and arrangement of the reinforcement is verified for at least 35% of the elements

- INCLUSIVE

They are used to verify the correspondence between the reinforcements or the characteristics of the connections actually present and those reported in the construction drawings, or obtained through the simulated project. The quantity and arrangement of the reinforcement is verified for at least 50% of the elements

In the control of the achievement of the percentages of elements investigated for the purpose of surveying the construction details, any repetitive situations are taken into account, which allow to extend to a wider percentage the checks carried out on some structural elements forming part of a series with obvious repeatability characteristics , for the same geometry and role in the structural scheme

Situation details (such as obvious structural symmetries, particular regularity etc.) can lead to a decrease in the number of elements to be investigated.

3.2.4.2 TESTS ON MATERIAL CHECK

To access the real level of information we have we can follow a certain scheme also for what concern material properties:

- LIMITED

They are used to complete information on the properties of the materials obtained or from the regulations in force at the time of construction or from the nominal characteristics shown on the construction drawings or from original test certificates.

1 specimen of concrete. for 300 square meters of the building floor;

1 sample of reinforcement per floor of the building.

- EXTENDED

They are used to obtain information in the absence of both the construction drawings and the original test certificates, or when the values obtained from the limited tests are lower than those reported in the original drawings or certificates.

2 specimens of concrete. for 300 square meters of the building floor;

2 samples of reinforcement per floor of the building.

- INCLUSIVE

They are used to obtain information in the absence of both the construction drawings and the original test certificates, or when the values obtained from the limited tests are lower than those reported in the original drawings or certificates, and an accurate level of knowledge is desired (LC3).

2 specimens of concrete. for 300 square meters of the building floor;

2 samples of reinforcement per floor of the building.

For the purposes of material testing, it is permissible to replace some destructive tests, no more than 50%, with a larger number, at least triple, of non-destructive tests, single or combined, calibrated on destructive ones. The number of specimens can be varied, increasing or decreasing, depending on the homogeneity characteristics of the material.

3.2.5 THE ACTIONS

The evaluation of the security of an existing building is necessarily a function of the actions to which the building is subjected. The actions to be considered are both static and seismic.

In defining the loads on the structure it is advisable to consider as permanent loads those that actually affect the structure and overloads as indicated by the Technical Standards for the construction of 2018.

Always for permanent loads, following a thorough geometric-structural investigation, it is possible to adequately reduce the partial safety coefficients. The interaction of the actions will have to be evaluated, as well as for the new structures according to the specifications of the Technical Standards for the construction of 2018. Also partial safety factors will be given by NTC2018.

3.2.6 SKILLS FOR THE CONSTRUCTION OF A REALISTIC MODEL ON THE SOFTWARE

The structure model must be three-dimensional and adequately represent the effective spatial distributions of mass, rigidity and resistance, with particular attention to situations in which horizontal components of the seismic action can produce significant vertical inertia forces (large light beams, protrusions). If a non-dissipative behaviour model or a dissipation model is used that uses the coefficient of behaviour q, elastic constituent laws will be used for the materials. If a dissipative behaviour model is adopted, taking explicitly into account the dissipative capacity, the constitutive link used to model the non-linearity of the material must be justified, also in relation to the correct representation of the energy dissipated in the hysteresis cycles. The geometric non-linearities, if significant, will be taken into consideration for both behaviours. In representing the rigidity of structural elements, it is necessary to take into account cracking.

If specific analyses are not performed, the bending and shear stiffness of the masonry, of the reinforced concrete, of the steel-cement elements can be reduced by up to 50% of the stiffness of the corresponding non-cracked elements, taking due account of the considered limit state and of the influence of permanent axial stress. With the exception of specific assessments and provided that the openings present do not significantly reduce the rigidity, the flat horizontal surfaces can be considered infinitely rigid in their average plane provided that they are made of reinforced concrete or brick-cement with a reinforced concrete slab with at least 40 mm thick or in a mixed structure with a reinforced concrete slab at least 50 mm thick connected to structural elements in steel or wood by means of shear-off connectors of suitable dimensions. In defining the model, non-structural elements not specifically designed as collaborators (such as fills and partitions) can be represented only in terms of mass; their contribution to the behaviour of the structural system in terms of rigidity and resistance will only be taken into account if it has negative effects on safety.

The following is a list of the most used software for modulating the structures and their analysis in compliance with the Italian Standards for NTC2018 constructions:

3.3 SEISMIC ANALYSIS

Standards establish that we need to distinguish:

- capacity of a structural element or of a structure: the set of stiffness, strength and ductility characteristics expressed by them, when subject to a predetermined set of actions;
- question on a structural element or on a structure: the set of stiffness, resistance and ductility characteristics required of them by a predetermined set of actions.

The check against the various limit states is carried out by comparing capacity and demand; in the absence of specific indications on the matter, the verification is considered to be carried out positively when the requirements of stiffness, strength and ductility are met, for the structural elements, and for stability and functionality, for non-structural elements and systems.

Buildings subject to seismic action, not equipped with appropriate isolation and dissipative devices, must be designed in accordance with one of the following structural behaviors:

- non dissipative structural behavior ($q=1,5$)
- dissipative structural behavior ($q\geq 1,5$).

For non-dissipative structural behavior, in the evaluation of the application all the members and connections remain in the elastic or substantially elastic range; the demand deriving from the seismic action and from the other actions is calculated, according to the limit state referred to, but independently from the structural typology and without taking into account the non-linearity of material, through an elastic model.

For dissipative structural behavior, in the evaluation of the demand a high number of members and connections evolve in the plastic field (plastic hinges), while the remaining part of the structure remains in the elastic or substantially elastic field; the demand deriving from the seismic action and from the other actions is calculated, according to the limit state referred to and the structural typology, taking into account the dissipative capacity linked to the non-linearity of material. If the dissipative capacity is taken into account implicitly through the behavior factor q, an elastic model is adopted; if the dissipative capacity is taken into account explicitly, an adequate constitutive law is adopted in the plastic or intermediate field.

A dissipative structural behavior construction must be designed to achieve one of the two Ductility Classes (CDs):

- High Ductility Class (CD "A"), with high dissipative capacity;
- Average Ductility Class (CD "B"), with average dissipative capacity.

The difference between the two classes lies in the number of zones that will be plasticized provided, in the design phase, both locally and globally.

Both for the CD "A" and for the CD "B", the typical procedures of "in capacity" design are used. This design has the purpose of ensuring ductile behavior for the dissipative structure and operates as follows:

- distinguishes elements and mechanisms, both local and global, in ductile and fragile;
- aims to avoid fragile local breaks and the activation of fragile or unstable global mechanisms;
- aims to localize the energy dissipations by hysteresis in areas of ductile elements identified for this purpose and designed, called "dissipative" or "ductile", consistent with the adopted structural scheme.

These goals can be considered as achieved by designing the SLV resistance capacity of fragile, local and global mechanisms, so that it is greater than that of the ductile elements alternative to them. To ensure compliance with this inequality, at both local and global level, the effective capacity of the ductile elements / mechanisms is increased by means of a suitable coefficient γ_{Rd}, called "over-resistance factor"; starting from this increased capacity the capacity of the undesired fragile elements is dimensioned, alternative to the ductile ones. For each structural type:

- it is necessary to ensure, even on a deductive basis starting from the γ_{Rd} over-resistance factors to be used in capacity planning at the local level, an adequate γ_{Rd} oversensitivity factor of the fragile global mechanisms. Where not explicitly specified in this standard, this factor must be at least equal to 1.25;
- the γ_{Rd} over-resistance factors to be used in capacity planning at the local level for the different structural elements and the individual checks.

The foundation structures and the related structural elements must be designed on the basis of the request transmitted to them by the structure above them, attributing to them non-dissipative structural behavior, regardless of the behavior attributed to the structure on them.

The quantity of the demand with which compare the capacity of the structure can be assessed using one of the models described above and by adopting one of the methods of analysis illustrated below. The analysis methods are divided into linear and non-linear, depending on the characteristics of the structure and the adopted behavior model. In the case of linear analysis, the seismic demand for structures with both non-dissipative and dissipative behavior can be reduced by using a suitable behavior factor q, which will reduce the acceleration values given by the response spectrum. The values attributable to q vary as a function of the structural behavior (dissipative or non-dissipative) and of the limit state considered, binding to the entity of plasticization, which accompany each limit state. For each of the limit states and the analysis methods considered, the following table shows:

- for the linear analysis, the structural behavior, the modeling methods of the seismic action and the limits to be attributed to the behavior factor q, according to the considered limit state;

- for non-linear analysis, structural behavior, modeling of seismic action.

STATI LIMITE		Lineare (Dinamica e Statica)		Non Lineare	
		Dissipativo	Non Dissipativo	Dinamica	Statica
SLE	SLO	q = 1.0 § 3.2.3.4	q = 1.0 § 3.2.3.4	§ 7.3.4.1	§ 7.3.4.2
	SLD	q ≤ 1,5 § 3.2.3.5	q ≤ 1,5 § 3.2.3.5		
SLU	SLV	q ≥ 1,5 § 3.2.3.5	q ≤ 1,5 § 3.2.3.5		
	SLC	---	---		

Table 1 - NTC2018

3.3.1 DIFFERENCES BETWEEN LINEAR AND NON-LINEAR ANALYSIS

The analysis of structures subject to seismic action can be linear or non-linear.

3.3.1.1 LINEAR ANALYSIS

The linear analysis can be used to calculate the seismic demand in the case of both non-dissipative and dissipative structural behavior. In both cases, the seismic demand is calculated, whatever the modeling used for the seismic action, referring to the design spectrum obtained, for each limit state, assuming for the behavior factor q, the limits reported in the standard with values of the basic factors q_0 given below.

In the case of dissipative structural behavior, the value of the behavior factor q, to be used for the considered limit state and in the direction considered for the seismic action, depends on the structural type, its degree of hyperstability and the design criteria adopted and holds account, conventionally, of the dissipative capacities of the material. The structures can be classified as belonging to a typology in a horizontal direction and to another typology in the horizontal direction orthogonal to the previous one, using the corresponding behavior factor for each direction. The upper limit q_{lim} of the behavior factor related to the SLV is calculated by the following expression:

$q_{lim} = q_0 \times K_R$

where is it:

q_0 is the basic value of the behavior factor at the SLV, whose maximum values are shown in the table depending on the Ductility Class, the structural typology, the coefficient λ and the ratio α_u / α_1 between the value of the seismic action for the which plasticization occurs in a number of dissipative zones such as to make the structure a mechanism and that for which the first structural element reaches yielding in bending;

K_R is a factor that depends on the characteristics of regularity in height of the construction, with a value of 1 for regular constructions in height and equal to 0.8 for non-regular constructions in height.

For plan regular constructions, if a non-linear analysis is performed for its evaluation, for the α_u / α_1 ratio, the values indicated in the standard can be adopted, different for the different construction types. For non-plan constructions in plan, values of α_u / α_1 equal to the average between 1.0 and the values provided from time to time for the different construction types can be adopted.

The value of q used for the vertical component of the seismic action at the SLV, unless adequate supporting proofs are given, is q = 1.5 for any structural type and material.

3.3.1.2 NON-LINEAR ANALYSIS

The non-linear analysis can be used both for structural systems with non-dissipative behavior, and for structural systems with dissipative behavior and takes into account the non-linearity of material and geometries. In structural systems with dissipative behavior the constitutive bonds used must also take into account the reduction of resistance and residual resistance, if significant.

3.3.2 DIFFERENCES BETWEEN STATIC AND DINAMIC ANALYSIS

In addition to the fact that the analysis is linear or non-linear, the analysis methods are articulated also in relation to the fact that the balance is treated dynamically or statically. The linear reference analysis method to determine the effects of the seismic action, for both dissipative and non dissipative structural behaviors, is the modal analysis with response spectrum or "dynamic linear analysis". In it the balance is treated dynamically and the seismic action is modeled across the design spectrum. As an alternative to the modal analysis, more refined analysis techniques can be adopted, such as step integration, modeling the seismic action through temporal histories of ground motion. For the constructions whose seismic response, in any main direction, does not depend significantly on the higher vibrating modes, it is possible to use, for both dissipative and non dissipative structural behaviors, the lateral forces method or "linear static analysis". In it the equilibrium is treated statically, the analysis of the structure is linear and the seismic action is modeled across the spectrum of. Finally, to determine the effects of the seismic action one can perform non-linear analyzes; in them the balance is treated, alternatively:

- dynamically ("dynamic non-linear analysis"), modeling the seismic action, using temporal histories of the ground motion;
- statically ("non-linear static analysis"), modeling the seismic action, using static forces grown monotonously.

Both for linear dynamic analysis and for static linear analysis, the accidental eccentricity of the center of mass must be taken into account. For buildings, the effects of this eccentricity can be determined by applying static loads consisting of torques of a value equal to the horizontal result of the force acting on the floor multiplied by the accidental eccentricity of the center of gravity of the masses with respect to its calculation position .

3.3.3 MODAL ANALYSIS OR LINEAR DYNAMIC ANALYSIS

The most used analysis is the modal analysis both for simplicity and eases of use and for the proved rightness of the results.

Dynamic linear analysis consists of:

- in the determination of the vibration modes of construction (modal analysis);
- in the calculation of the effects of the seismic action, represented by the design response spectrum, for each of the identified vibration modes;
- in the combination of these effects. All modes with significant participant mass must be considered.

In this regard, it is appropriate to consider all modes with a participating mass of more than 5% and a number of modes whose total participating mass exceeds 85%. For the combination of the effects related to the individual modes, a complete quadratic combination of the effects related to each mode must be used, such as that indicated in the expression:

$$E = \sqrt{\sum_j \sum_i \rho_{ij} \cdot E_i \cdot E_j}$$

with:

E_j value of the effect related to mode j;

ρ_{ij} correlation coefficient between mode i and mode j, calculated with formulas of proven validity as a function of some factors such as the viscous damping of modes i and j and the ratio between the inverse of the periods of each pair i-j of modes (T_j / T_i).

3.3.4 STATIC NON-LINEAR ANALYSIS OR PUSH-OVER

Non-linear static analysis requires that a structural non-linear equivalent system is associated with the real structural system. In the event that the equivalent system has a degree of freedom, the gravitational loads are applied to the equivalent structural system and, for the considered direction of the seismic action, in correspondence with the horizontal elements of the construction, horizontal forces proportional to the inertia force having resultant (base shear) F_b. These forces are scaled so that the horizontal displacement d_c of a control point coinciding with the center of mass of the last level of the construction grows monotonously, both in a positive and negative direction and until the conditions of local or global collapse are reached. Alternative control points, such as the ends of the last level plan, should also be considered, when the coupling of translations and rotations is significant. The diagram F_b - d_c (force-displacement) represents the capacity curve of the structure, a fundamental parameter of this analysis. At least two distributions of inertial forces must be considered, falling one in the main distributions and the other in the secondary distributions described below.

Main distributions:

- if the fundamental vibrating mode in the considered direction has a mass participation of not less than 75%, one of the following two distributions applies:

distribution proportional to static forces, using as a second distribution a) of secondary distributions,

distribution corresponding to an acceleration pattern proportional to the shape of the fundamental way of vibrating in the considered direction;

- in all cases, the distribution corresponding to the course of the plane forces acting on each horizontally calculated in a linear dynamic analysis can be used, including in the considered direction a number of modes with participation of total mass not lower than 85%. The use of this distribution is mandatory if the fundamental period of the structure is greater than 1.3 TC.

Secondary distributions:

- distribution of forces, derived from a uniform pattern of accelerations along the height of the construction;
- adaptive distribution, which changes with the increase of the displacement of the control point as a function of the plasticization of the structure;
- multimodal distribution, considering at least six significant ways.

3.4 COMPLIANCE WITH THE REQUIREMENTS IN RESPECT OF THE LIMIT STATES AND VERIFICATIONS

For all primary and secondary structural elements, the non-structural elements and the installations must verify that the value of each project request is lower than the corresponding value of the project capacity. The checks of the primary structural elements (ST) are carried out, as summarized in the table, depending on the Class of Use (CU):

- in the case of non-dissipative structural behavior, in terms of rigidity (RIG) and resistance (RES), without applying the specific rules of construction details and capacity planning;
- in the case of dissipative structural behavior, in terms of stiffness (RIG), resistance (RES) and ductility (DUT) (when required), applying the specific rules of construction details and capacity planning.

Verifications of secondary structural elements are carried out only in terms of ductility. The verifications of the non-structural elements (NS) and of the plants (IM) are carried out in terms of functioning (FUN) and stability (STA) depending on the Class of Use (CU).

STATI LIMITE		CU I	CU II			CU III e IV		
		ST	ST	NS	IM	ST	NS	IM(*)
SLE	*SLO*					RIG		FUN
	SLD	RIG	RIG			RES		
SLU	*SLV*	RES	RES	STA	STA	RES	STA	STA
	SLC		DUT(**)			DUT(**)		

Table 2- NTC2018

The checks at the limit state of prevention of collapse (SLC), unless specific indications, are carried out only in terms of ductility and only if the checks in ductility are expressly requested.

3.4.1 STRUCTURAL ELEMENTS (ST)

3.4.1.1 STIFFNESS CHECKS (RIG)

The condition in terms of rigidity on the structure is considered to be satisfied if the consequent deformation of the structural elements does not produce damage on the non-structural elements such as to make the building temporarily unusable. In the case of civil and industrial buildings, if the temporary uninhabitability is due to excessive inter-floor displacements, this condition can be considered satisfied when the inter-floor displacements obtained by the analysis in the presence of the seismic action of the project corresponding to the SL and the CU considered are below the limits indicated below.

For the CU I and II the SLD is referred to and must be:

- for infill walls rigidly connected to the structure, which interfere with its deformability:

$$qd_r \leq 0.0050 \text{ h for fragile infills}$$

$$qd_r \leq 0.0050 \text{ h for ductile infills}$$

- for infill walls designed so as not to be damaged as a result of inter-space d_{rp} displacements, due to their intrinsic deformability or to the connections to the structure:

$$qd_r \leq d_{rp} \leq 0.0100 \text{ h}$$

where is it:

d_r is the displacement of inter-floor, that is the difference between the displacements of the upper floor and of the lower floor;

h is the height of the floor.

For the CUs III and IV we refer to the SLO and the inter-space displacements must be less than 2/3 of the limits previously indicated.

3.4.1.2 RESISTANCE CHECKS (RES)

It must be verified that the individual structural elements and the structure as a whole possess a capacity of resistance sufficient to satisfy the demand at the SLV. The resistance capacity of the members and connections is evaluated in accordance with the rules. For structures with dissipative behavior, the capacity of the members is calculated with reference to their ultimate behavior. For structures with non-dissipative behavior, the capacity of the members is calculated with reference to their elastic or substantially elastic behavior. The resistance of the materials can be reduced to take into account the degradation due to cyclical deformations, justifying it on the basis of specific experimental tests. In this case, the values specified in Chapter 4 of the NTC2018 are attributed to the partial safety factors on the γ_M materials for exceptional situations.

3.4.1.3 DUCTILITY CHECKS (DUT)

It must be verified that the individual structural elements and the structure as a whole possess a capacity in ductility:

- in the case of linear analysis, consistent with the behavior factor q adopted and the relative displacements;
- in the case of non-linear analysis, sufficient to satisfy the demand in ductility highlighted by the analysis.

In the case of linear analysis the verification of ductility can be considered satisfied, respecting for all structural elements, both primary and secondary, the specific rules for construction details specified in this chapter for the different construction types; these rules are to be considered additional with respect to the provisions of Chapter 4 of the NTC2018 and to what is imposed by the rules of capacity planning, compliance with which is in any case obligatory for the primary structural elements of the dissipative behavior structures. For structures with dissipative behavior, if the specific rules of construction details are not complied with, it will be necessary to proceed with ductility checks.

3.4.2 NON-STRUCTURAL ELEMENTS (NS)

3.4.2.1 STABILITY CHECKS (STA)

For non-structural elements, precaution must be adopted to avoid the possible expulsion under the action of the seismic force corresponding to the SL and the CU considered.

3.4.3 PLANTS (IM)

3.4.3.1 FUNCTIONAL CHECKS (FUN)

For the plants, it must be verified that the structural displacements or the accelerations (depending on whether the plants are more vulnerable to the effect of the first or second ones) produced by the actions related to the SL and the CU considered are not such as to produce interruptions of use of the facilities themselves.

3.4.3.2 STABILITY CHECKS (STA)

For each of the main plants, the various functional elements that make up the plant, including the structural elements that support and connect them, between themselves and the main structure, must have sufficient capacity to sustain the demand corresponding to the SL and the CU considered.

3.5 RE.SIS.TO® - Total Seismic Resistance

With reference to the seismic hazard of the Italian territory, a modern approach to building management must know the vulnerability of the existing building heritage, and in particular of the public as it is subject to greater exposure. It is not always possible to carry out in-depth and precise evaluations, at least in the preliminary phase when the first objective is necessarily to first establish the priorities for intervention. In fact, they require considerable economic resources and time and are often incompatible with the real availability of the various properties. Therefore, we often resort to procedures based on some empirical data and qualitative judgments, which if not framed in a single defined analysis procedure do not allow us to obtain comparable data and information.

At the regulatory level, since 2003 (OPCM 3274), a series of laws and circulars have been followed aimed at assessing the seismic vulnerability of Italian strategic works; the initial request for an in-depth analysis was subsequently accompanied by the possibility of carrying out preliminary assessments of "Level 0", at least able to provide knowledge of the general characteristics of the buildings. To date, however, very few administrations have conducted the necessary vulnerability analyses on the entire real estate portfolio. It is therefore advisable to suggest, in order to take an effective step forward with respect to the preparation of the "0" level sheets, an alternative method that allows at least to identify an order of priority of the interventions, if they are more in-depth investigations and complete with seismic vulnerability or real interventions. The reliability of an "intermediate" level method requires that at least some quantitative assessments are carried out, delegating all the rest to the expert's technical judgment. In the privately managed building sector, a similar requirement is expressed by banks and insurance companies that need tools that certify the value of buildings (and any insurance premium) with parameters that are also based on safety and not just on efficiency criteria energy.

Below we illustrate a quick method of assessing the seismic vulnerability of buildings. It can be applied, in two different versions, to buildings in reinforced concrete and masonry, obtaining sufficiently homogeneous vulnerability assessments between the two categories. The preferred field of application of this methodology is represented by buildings that constitute building patrimony of significant numerical consistency, of which we want to define an indicative but uniform seismic vulnerability within the population of buildings under examination, in order to make the choices strategic necessary for the definition of a priority ranking for the next phase, which consists in the development of complete vulnerability studies or seismic improvement / adjustment interventions.

The proposed methodology leads to the definition of an acceleration to the ground of collapse of the building through the evaluation of the resistant shear of the same, floor by floor. This latter quantity is evaluated using simplified mechanical considerations. The transition from the theoretical calculation scheme to the actual conditions of the building, which can highlight possible structural criticalities identified during inspections but not analyzed in detail, takes place using a reductive coefficient, obtained starting from the parameters contained in the vulnerability sheets of the National Earthquake Defense Group (GNDT 1994). This allows an assessment of the aspects characterized by greater empiricism according to methodologies recognized at national level and already applied on different occasions.

The methodology proposed with reference to reinforced concrete buildings will be illustrated below.

The requested information refers mainly to the geometry of the buildings being studied, to an estimate of the types of materials used in the structures, to simplified analysis of the loads and, in the case of reinforced concrete constructions, to the knowledge of the reinforcement bars of at least one column type for construction plan.

This method was initially used to assess the seismic vulnerability of all strategic buildings owned by the Province of Bologna, which were classified using a series of intervals specific to the proposed method (called RE.SIS.TO® - Seismic Resistance Total), for comparison between the building collapse acceleration and the calculation acceleration for the area where the building is erected.

3.5.1 THE METHODOLOGY

For the assessment of seismic vulnerability with the proposed method, the necessary work is divided into three successive phases:

- Search for technical information, in order to obtain a plausible picture of the state of the building;
- Evaluation of the seismic vulnerability of the system, intended as an estimate of the seismic acceleration that leads to the collapse of the building;
- Comparison between the building collapse acceleration and the calculation acceleration for the area where the building is erected.

Knowledge of the current state of the building assumes fundamental importance in the evaluation of the safety checks of the building and, obviously, in the planning of any subsequent interventions of adaptation and seismic improvement. The knowledge phase of the property consists of the following steps:

- Research of historical documentary material relating to buildings, to interventions carried out in the course of its history and at present, available in digital format (rarely available except for buildings built in the last twenty years) or on paper; it is essential, at this stage, the active participation of the staff of the managing body of the building heritage;
- Inspection to verify the correspondence between the graphic drawings (previously collected) and the actual state, to identify the real resistant systems present, to make a sample measurement of the dimensions of the structural elements (walls, beams, columns etc.) and a visual inspection of the typology of horizontal (important aspect for the evaluation of the masses of the building).

As far as materials are concerned, two different approaches are followed for masonry and reinforced concrete constructions. In reinforced concrete buildings, an assay is carried out in a column per floor to assess the type of reinforcement (number and diameter of the irons), a series of pacometric measurements to verify the position of the reinforcing bars and of sclerometer bars to estimate the strength class of concrete.

Since the masonry and reinforced concrete structures are characterized by different collapse mechanisms, two different models have been developed for defining the resistant capacity:

for buildings in c.a. we proceed with the estimate of the resistant shear of the n-th column of the

i-th plane, indicated with $V_{pil,\ n,\ i}$, which is calculated starting from the resistant moment of its weakest end section. In the context of this expeditious methodology, however, it is not possible to have a survey of all the significant sections of all the columns. In the procedure described, clearly approximated, at least the reinforcement of a "standard" column per floor is expected. In particular, for this column it is necessary to know the geometric dimensions, the amount of longitudinal reinforcement, and an estimate of the mechanical properties of the materials (compression strength of the concrete and tensile strength of the steel). On the basis of this information it is possible to calculate the resistant moment of the column type, $M_{pil,\ type,\ i}$, obtained through a classic interaction domain M-N for an assigned value of normal effort (because calculated). The calculation of the resistant moment of the standard column must be performed in both reference directions of the column, which generally coincide with the reference directions, x and y, of the building. To determine the resistant shears along the two directions $V_{x\ pil,\ type,\ I}$ and $V_{y\ pil,\ type,\ i}$, in the case of a framed structure entirely cast in situ the column is considered a static interlocking - sliding interlocking scheme (except for particular system configurations resistant to horizontal actions); this assessment must be made clearly for each floor of the building. In the case of columns in isostatic configuration (as for example for prefabricated structures), an evaluation of the shear resistant to the different floors is not necessary, but it must be calculated exclusively at the base of the building, considering for the columns the static scheme of an embedded shelf to the base. Starting from the resistant shears of the type column, $V_{x\ pil,\ type,\ i}$ and $V_{y,pil,\ type,\ i}$, it is possible to derive those of all the columns of the generic i-th plan ($V_{x\ pil,\ n,\ i}$ and $V_{y\ pil,\ n,\ i}$) using the simplifying hypothesis of direct proportionality between the resistant shear and the moment d inertia of the section of single columns. This approach neglects the variability linked to the significant contribution given by the real amount of reinforcement but has proved to be sufficiently reliable. Analytically translated, this means that the resistive shears of the generic nth column of the i-th plane in the x and y directions will be given respectively by the following relations:

$$V^{x}_{pil,n,i} = V^{x}_{pil,tipo,i}\frac{J_{y,n,i}}{J_{y,tipo,i}} \quad V^{y}_{pil,n,i} = V^{y}_{pil,tipo,i}\frac{J_{x,n,i}}{J_{x,tipo,i}};$$

where:

$J_{x,\ n,\ i}$, $J_{y,\ n,\ i}$ are respectively the moments of inertia of the section of the generic column n-ish around the x and y axes;

$J_{x,\ type,\ i}$, $J_{y,\ type,\ i}$ are respectively the moments of inertia of the typical column section around the x and y axes.

In the presence of reinforced concrete partitions, a standard partition must be identified for each floor and the resistant shear must be assessed for it, as indicated in the NTC (2008). Starting from the resistant shear of the type shear wall, $V_{setto,\ type,\ i}$, we obtain those of all the shear wall of the generic i-th plane by using the simplifying hypothesis of direct proportionality between the shear and the area of the section of the single shear wall.

The resistant shears of the i-th plane in the x and y, $V_{x\ r,\ i}$ and $V_{y\ r,\ i}$,, directions are therefore obtained as the summation of the resistant shears of all the columns and partitions belonging to the plane. It is adopted as the resistant shear of the generic i-th floor, $V_{r,\ i}$ the minimum between that in the x direction and the one in the y direction. It is possible to compare the shear resistant to all planes with the stress shear, obtained by applying to the structure a distribution of equivalent static forces obtained considering a unitary spectral acceleration.

The shearing stress to the generic i-th plane $V_{s,\ i}$ is equal to the sum of the forces applied to the planes above.

The relationships between the resistant shears of the plane $V_{r,\ i}$ and the corresponding plane shears acting V_s, define the structural performance of the individual floors of the building in terms of accelerations on the structural masses, expressed as a fraction of g. The different ratios thus obtained allow to identify the weaker plane (the one with the minimum value of this ratio) and to define the building resistance in terms of spectral acceleration ($S_{a,\,c}$).

This acceleration value is however strongly conventional, since it does not consider the real complexity of the construction in question. The adaptation of the conventional capacity to a realistic value is carried out by making use of ten of the eleven parameters contained in the II level seismic vulnerability sheet (GNDT 1994). Once the vulnerability classes relating to the building in question have been defined, the generic parameter will univocally correspond to a single score, pi, and a unit weight. Therefore, for each parameter, Ki is equal to pi. The reductive coefficient, Crid, is determined through the:

$$C_{rid} = \prod_{i=1}^{10}\left(1-\alpha\cdot\frac{K_i}{K_{pegg}}\right)$$

To define the intensity of the earthquake that the structure can withstand without collapsing, the spectral value of the acceleration must be transformed into the value of the maximum acceleration of the ground (PGAc). Taking into account that the calculated spectral acceleration value is a linear static value, the relation that links Sa, c and PGAc is the following:

$$PGA_c = \frac{S_{a,c}}{\alpha_{PM}\cdot\alpha_{AD}\cdot\alpha_{DT}\cdot\left(\frac{1}{\alpha_{DUC}}\right)}$$

where is it:

α_{PM} is the modal participation coefficient, which is 1.00 for buildings with only one floor above ground and 0.80 in other cases;

α_{AD} is the spectral amplification coefficient and is assumed to be 2.50;

α_{DT} is a coefficient that takes dissipative phenomena into account. If the resistant contribution of the infills is significant compared to that of the main resistant system and is considered in the analysis, α_{DT} assumes unitary value, otherwise if this contribution is ignored αDT is set equal to 0.80;

α_{DUC} is the structural factor and can assume values between 2.00 and 3.00 on the basis of the work rates of the materials under static actions, in accordance with the NTC Circular (2009).

Finally, demand and capacity are compared in terms of acceleration, or the relationship between PGA_c and PGA_d is made. The lower this ratio is, the more the building presents structural vulnerabilities in the presence of a seismic event.

3.5.2 RE.SIS.TO® CLASSIFICATION

In order to improve the immediacy of the perception of the results of the analysis and taking into account the level of approximation of the calculations described, a simplified classification called RE.SIS.TO® is introduced, with which the evaluated buildings are framed within five homogeneous categories in terms of seismic vulnerability, and therefore due to intervention criticalities. The belonging to a category is returned graphically by means of a chromatic scale (using the colors from red to green), as shown in the figure.

Figure 11- resisto classification 1

Buildings are initially assigned one of the classes, from I (low vulnerability) to V (high vulnerability), depending on the relationship between capacity and demand, in terms of ground acceleration; the intervals of this ratio for each class are shown in the table.

PGA_c/PGA_d	Classe di resistenza
0% - 25%	V
25% - 50%	IV
50% - 75%	III
75% - 100%	II
> 100%	I

Figure 12- resisto classification 2

Any local vulnerabilities / criticalities of the building emerged from the inspections, which may represent weak points in the behavior of the structure under earthquake but were not adequately exploited in the previous analysis phase, come into play at this stage; in fact, in the presence of considerable critical elements in a building, this must be recognized as a superior class (therefore pejorative) to that which results from the only relationship between PGA. This makes it possible, albeit in a simplified way, to evaluate for example also local phenomena such as the out of plane overturning of masonry walls, great lack of brackets in the columns (if known), particularly serious geometric or structural irregularities.

On the basis of the procedure described, extensive building heritage can be classified quickly and it will be possible to reason in terms of criticality of intervention. The buildings found in Class V RE.SIS.TO®, for example, will have priority over all others and will therefore be the first on which the scheduled checks will be carried out and on which any local and / or repair work will be scheduled. seismic improvement. To facilitate comparative simplicity, a summary sheet must be completed for each evaluated building, which contains a summary of the most significant data of the assessment, or general information on the body of the building (structural type, description, number of floors ..), parameters of vulnerabilities, capacity parameters and demand parameters. This allows you to quickly assess the condition of a building and the possible presence of anomalous values.

Among the qualifying aspects of the method we can highlight:

- the request for resources and limited time for the application of the procedure, factors of great importance when speaking of public bodies that have to manage dozens and dozens of properties; just think that, if well organized, the inspection does not require more than half a day for the body of the building and the processing of the data and the drawing up of the reports a couple of working days;
- the minimum invasiveness of in situ investigations, limited to some visual inspection and to pacometric and sclerometric surveys (in the case of systems in c.a.);
- the use of recognized tools (when possible) and transparent and flexible procedures for quantitative assessments;
- the ability to couple purely mechanical aspects to "expert" assessments of a geometric-qualitative nature attributable to shared criteria contained in the procedures for compiling the GNDT vulnerability sheets, consolidated over the years;
- the generation of quantitative results able to be subsequently reworked in the light of other socio-economic criteria

4. NEW PROCEDURE FOR THE ANALYSIS OF EXISTING BUILDINGS

In the previous chapter the Italian legislation for the definition of the safety level of reinforced concrete structures, or the analysis of seismic vulnerability and all that concerns it, was illustrated in detail. The incredibly demanding nature of this procedure has often been emphasized, in fact both from a temporal point of view (historical analysis, construction of a model and checks) and from the economic point of view (laboratory tests for the mechanical characterization of materials) the legislation provides for complex and in-depth procedures for the achievement of sensible confidence factors for the case in question. The calculation of the seismic vulnerability coefficient of an existing building in reinforced concrete is a complicated and time-consuming operation, in short not suitable for those structures on which an intervention is urgent.

For all those bodies, public and private, which have and maintain a large real estate assets, and which want to guarantee their usability and security, it is necessary to offer a rapid and rapid method of analysis for determining the priority of intervention from a point of view. of the vulnerability, so as to allow analyses that at the same time offer a high veracity of the results and a speed of operation. For example, to make the concept clearer: when a company owns one hundred properties but can intervene (due to a limited budget) only on a part of these, the owners will need to understand which structures of those owned are in critical situation, or rather those on which it is urgent to intervene both for problems of the structure and for example to enter into deadlines given by state bonuses; to do this it would take a very long period of time following the Italian legislation and a large economic outlay to contract out the vulnerability analyses to subordinate professionals. To avoid this, reference can be made to simplified analyses which guarantee guaranteed reliability, thanks to standards maintained in previous works.

In this panorama the method proposed by the current thesis is inserted: offering the possibility of having information on the vulnerability of a structure starting from a few and essential starting data, without the need for intrusive and exhaustive tests and all the legislative process described above ; with the aim of shearting times and costs of a more thorough and accurate analysis, but still ensuring a high degree of reliability of the results provided, so as to be able to choose in a short period of time on which structure to intervene with precedence over the remaining ones.

The starting data would be, on the contrary to what happens today, easy to find and in small times, without having to go and damage the structure by taking samples for laboratory tests. The procedure is based on a few initial information, it can be divided into:

- Geometries, such as the layout of the structure and its volume, as well as the position of the walls in the said plant and the construction type of the floors (masonry, steel);
- Based on time (not finding a better definition): in fact the other great and fundamental starting information is based on the age of construction, on this depend the characteristic quantities of the various structural elements (thanks to the database we will talk about in chap. 4.3) and, above all, the materials used by the construction companies of the time, with their own characteristic resistances, these instead can be found in the anthology as shown in the image below.

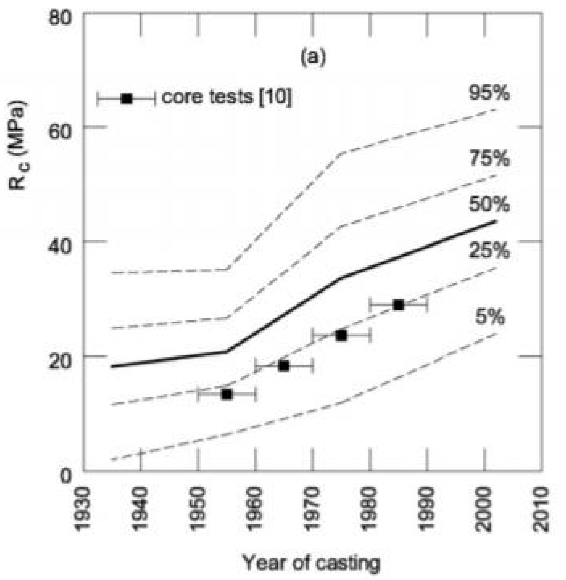


Figure 13- Characteristic resistances for cls

Type of steel	Yield strength – f_y MPa (ksi)	Tensile strength – f_u MPa (ksi)
Common steel	-	280 ~ 500 (40.6 ~ 72.5)
Aq.42	≥ 230 (≥ 33.4)	420 ~ 500 (60.9 ~ 72.5)
Aq.50	≥ 270 (≥ 39.2)	500 ~ 600 (72.5 ~ 87.0)
Aq.60	≥ 310 (≥ 45.0)	600 ~ 700 (87.0 ~ 102)
FeB32	≥ 320 (≥ 46.4)	≥ 500 (≥ 72.5)
FeB38	≥ 380 (≥ 55.1)	≥ 460 (≥ 66.7)
FeB44	≥ 440 (≥ 63.8)	≥ 550 (≥ 79.8)

Figure 14- characteristic resistances for steel

- Location, in fact remains a necessary and fundamental condition to know where the building is located, it depends on the acceleration that the earthquake imposes on the system according to the spectral information that can be found in the anthology, a factor that is of fundamental importance during the analysis seismic structure of a building.

To be able to create a method of such importance and usefulness, different working phases followed, which were briefly summarized in the index below and then explained in detail in the following chapters:

- Research of the material through the office of the Civil Engineers of the Piedmont Region: selection of a dozen civil housing projects in the area of Turin and its surroundings bearing date between 1970 and the early 2000s;
- For each of these projects we proceeded with a sampling on a structural level, with particular reference to geometry and construction details (such as the reinforcement configuration);
- Creation of a database based on the geometric and construction differences classified according to the age of construction of the building and therefore in the legislation in force at the time;
- Through the interpolation of these data we proceeded to calculate the quantities and standard quantities with reference to the different construction periods and therefore to the different design regulations used;

4.1 RESEARCH AND SELECTION OF THE PROJECTS

The first step towards the creation of a database based on geometric dimensions and construction details was to find and retrieve some projects for condominiums for residential purposes; this was possible only thanks to the collaboration of the ministry of infrastructures, more precisely it was necessary to go to the Civil Engineers office located in Corso Bolzano 44 to consult the protocols that deal with the organized registration of all those design practices from 1970 to follow; once the numbers of the files were selected they proceeded with the recovery of the same, kept within the historical archive of the Piedmont Region and with the selection of the projects that most agreed with the requirements imposed by the methodology.

The requisites that these structures respect are summarized below:

- Type of construction in reinforced concrete;
- Structural compactness and regularity of the forms;
- Height between three and eight floors;
- Presence of all the calculation and design documents (a parameter that proved to be the most difficult due to the lack of accuracy in past legislation);

- Area of location of the building in the municipality of Turin or in its first belt;
- No excessive complexity of the structure;

Finally, the projects selected were six, differentiated by construction period and therefore normally respected at the time of design, these are listed in the table below:

YEAR	IDENTIFICATION	UBICATION	N° OF FLOORS
1975	(1)	Torino	7 p.f.t.
1981	(2)	San Mauro T.se	5 p.f.t.
1987	(3)	Torino	5 p.f.t.
1993	(4)	Torino	4 p.f.t.
1999	(5)	Torino	5 p.f.t.
2003	(6)	Torino	7 p.f.t.

Table 3- Selected projects

The main objective was to have as homogeneous as possible structures available, such as plan structure and elevation. Another important point is that of the time reference which, as can be seen, has been abundantly satisfied, given that the analysed period is about thirty years, equally divided, in fact the distance between a project and the next is about six years.

The structures on the list have been designed to meet the requirements imposed by the law in force at the time of writing, which is defined to be the same in all six cases examined, namely the number 1086 drawn up in 1971; a norm that today we will call antiquated, but which for that period marked decisive changes, was in fact one of the first in which anti-seismic legislation was discussed, which is still being completed today.

Usually we find all the calculation and the static analysis for the design of the structure (really less than what we need to exhibit for the presentation of the projects) and all the drawings: the plan, the prospects, the cross sections of the structural elements and the reinforcement arrangement of beams, columns, slabs and shear walls.

4.2 SAMPLING OF GEOMETRIC CHARACTERISTICS

Once the projects were obtained it was necessary to classify the buildings from a structural and geometric point of view, in fact the ultimate goal of this analysis is to create a database that can collect geometric information for the various building eras, thus differentiating the building heritage existing.

The characteristics that have been investigated are divisible into four macro-areas:

- Beams;
- Columns;
- Slabs;
- Shear wall.

For each of these structural elements we proceeded with the creation of Excel files that would collect useful information for analysis, these can be summarized in all those geometric sizes of the various sections and in the percentage of reinforcement present within the concrete, then for the beams, the following information was collected as an example:

- Beam span (light; L);
- Height of the beam (H);
- Thickness of the beam (B);
- span / height ratio (L / H);
- Base / height ratio (B / H);

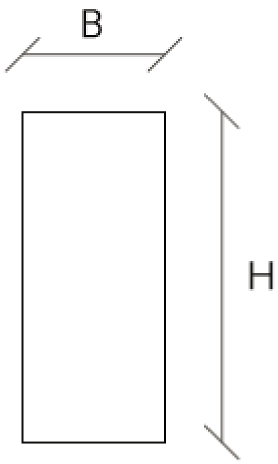


Figure 15- Transversal scheme of a beam

This with regard to the geometry, from the point of view of the reinforcement, on the other hand, it is still necessary to distinguish between the upper and lower one; both evaluated in the three critical points of the selected element, as the section on the first support, the section in the span and that on the second support of the beam.

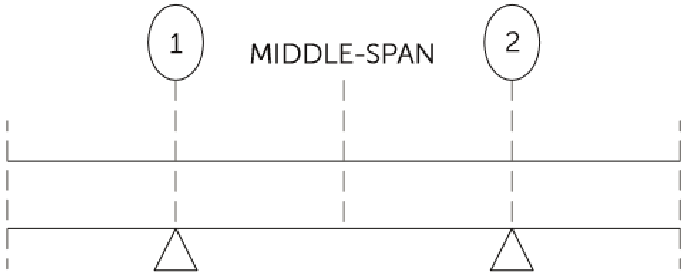


Figure 16- Longitudinal scheme of a beam

As far as the columns are concerned, the geometric sizes selected are:

- Inter-floor height of the column;
- Size in the direction parallel to the warping of the beams (X1);
- Size perpendicular to the beam warping (X2);

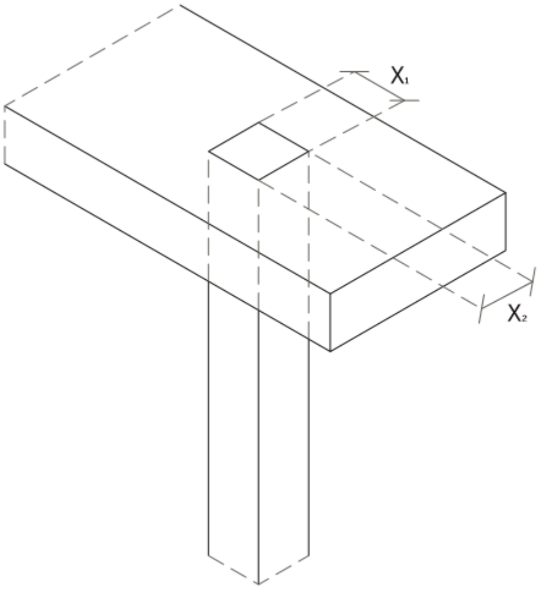


Figure 17- Scheme of a column

While the discourse concerning reinforcement is the same as for beams.

Different was the approach taken for the floors, all in masonry, from the geometric point of view in fact these are given by the repetition of a base section called the joist, with a single direction of warping, characterized by:

- Span of the slab (joist length);
- Width of the joist (b);
- Thickness of the covering jet (s);
- Distance between the various joists;
- Height of the joist (H);

as far as the reinforcement is concerned, however, this was found to be constant along the entire length of the floor, with an identical reinforcement in all the portions of the structure as regards some constructions, while for others it changed from floor to ceiling and it was therefore necessary to proceed with the calculation of some averages.

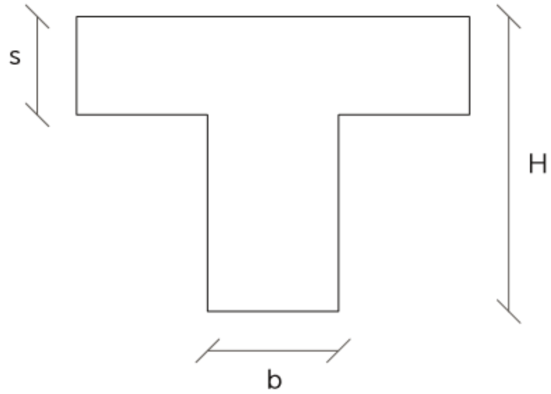


Figure 18- Scheme of a joist

Finally the shear wall inside the building were sampled, usually used as an elevator shaft or stairwell; these have the same structure for all the height of the building and the fundamental parameter, in addition of course to the geometric characterization of the building and its reinforcement reinforcement (therefore the sizes in the two fundamental directions), is the location with respect to the plan of the condominium; in fact the shear wall, being a highly rigid element, has a decisive influence on all the structural analyzes to be conducted.

To record the position of these shear wall we proceeded with the calculation of their barycentre with respect to an origin placed in one of the corners of the plant, then trying to refer this quota to the entire length of the construction. In fact, the parameter that is returned will be a number between 0 and 1, with 0.5 representing the middle, while the closer you get to 0 the more it will be moved to the left and vice versa for the right. This operation will be done both in x and y direction.

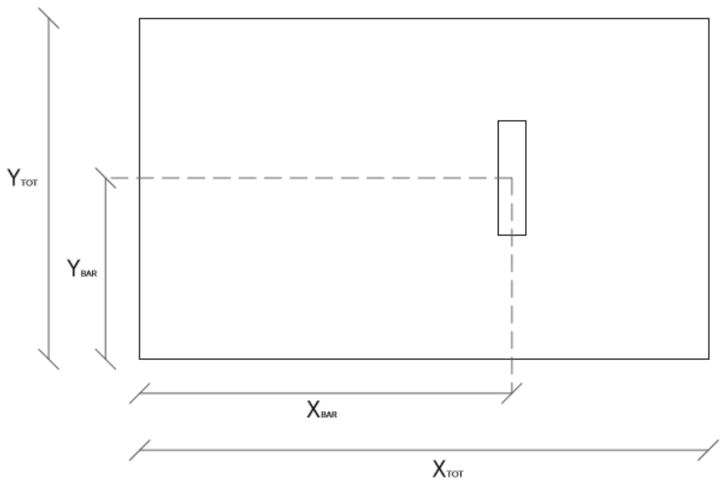


Figure 19- shear walls in the plan view

All the results of this sampling are collected in the annexes found at the end of the thesis.

4.3 CREATION OF A DATABASE

The creation of the database was the next and central step of this thesis. Once the fundamental data were obtained through the direct counting of the various projects, it was necessary to organize the material in an intuitive and easy way to read: on the basis of the analysed structural element all the sizes classified from year to year were collected on the same file , until obtaining the variation that these have undergone in the reference time to the variation of the respected regulations.

To better understand what has been said, we proceed with the actual explanation of what the work has been: we start with an Excel spreadsheet (one for each structural element) compiled during the previous phase for each project analysed, reporting the sizes for each first specify beams, columns, walls and floors; after which comes the creation of a file (one for each structural element) to collect instead all the data that you have of the reference structural element (eg beam), keeping the differences evident and then dividing them according to the different projects and then to the different building eras. In this way you will have four spreadsheets that contain all the measurements taken from the various elements. It will then be a matter of extrapolating some essential data, as we shall see in the next chapter. The original spreadsheets are available in the annexes to the fund (Annex 1). Here below only a excerpt of the classification of a beam (only for what concern the geometric characteristics of the cross section, then there are spreadsheets with information about the reinforcement):

YEAR	N° BEAM	DIRECTION	SPAN[cm]	B[cm]	H[cm]	SPAN/H	B/H
1975	T101	X	330	40	22	15	1.818182
1975	T102	X	315	40	22	14.31818	1.818182
1975	T103	X	310	40	22	14.09091	1.818182
1975	T104	X	350	40	22	15.90909	1.818182
1975	T105	X	340	40	22	15.45455	1.818182
1975	T106	X	315	40	22	14.31818	1.818182
1975	T107	X	285	40	22	12.95455	1.818182
1975	T108	X	305	40	22	13.86364	1.818182
1975	T109	X	315	40	22	14.31818	1.818182
1975	T110	X	405	40	22	18.40909	1.818182
1975	T111	X	330	40	22	15	1.818182
1975	T112	X	320	40	22	14.54545	1.818182
1975	T113	X	390	70	22	17.72727	3.181818
1975	T114	X	265	50	22	12.04545	2.272727

1975	T115	X	320	50	22	14.54545	2.272727
1975	T116A	X	165	50	22	7.5	2.272727
1975	T116B	X	220	50	22	10	2.272727
1975	T117	X	300	50	22	13.63636	2.272727
1975	T118	X	320	50	22	14.54545	2.272727
1975	T119	X	270	50	22	12.27273	2.272727
1975	T120	X	245	50	22	11.13636	2.272727
1975	T121	X	320	50	22	14.54545	2.272727
1975	T122	X	455	80	22	20.68182	3.636364
1975	T123	X	225	70	22	10.22727	3.181818
1975	T124	X	375	70	22	17.04545	3.181818
1975	T125	X	305	50	22	13.86364	2.272727
1975	T126	X	260	50	22	11.81818	2.272727
1975	T127	X	290	50	22	13.18182	2.272727
1975	T128	X	270	40	22	12.27273	1.818182
1975	T129	X	270	40	22	12.27273	1.818182
1975	T130	X	285	40	22	12.95455	1.818182
1975	T131	X	280	40	22	12.72727	1.818182
1975	T132		250	40	22	11.36364	1.818182
1975	T132BIS	X	285	40	22	12.95455	1.818182
1975	T133	X	240	40	22	10.90909	1.818182
1975	T134	X	270	40	22	12.27273	1.818182
1975	T135	X	265	50	22	12.04545	2.272727
1975	T136	X	295	50	22	13.40909	2.272727
1975	T137	X	410	50	22	18.63636	2.272727
1975	T138	Y	588	50	22	26.72727	2.272727
1975	T139	Y	622	50	22	28.27273	2.272727
1975	T140	Y	633	50	22	28.77273	2.272727
1975	T141	Y	577	50	22	26.22727	2.272727
1975	T142	X	425	50	22	19.31818	2.272727
1975	T143	X	420	55	22	19.09091	2.5
1975	T144	X	590	55	30	19.66667	1.833333
1975	T145	X	540	65	22	24.54545	2.954545
1975	T146	X	380	65	22	17.27273	2.954545
1975	T147	X	445	65	22	20.22727	2.954545

Table 4- Example of beams classification

As we can see in 1975 the frame was only in one direction, in fact all, more or less, the beams are in same direction.

4.4 CALCULTION OF STANDARD MISURES AND CHARACTERISTICS

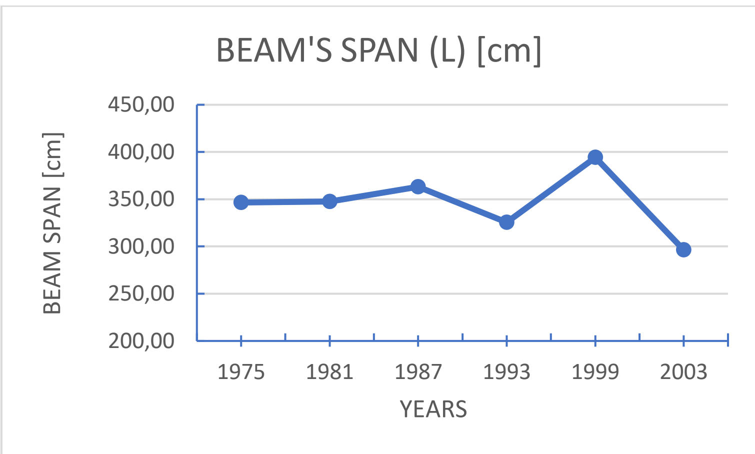
Once obtained calculation sheets for the four structural elements analysed, we proceed with the calculation of some reference values for each of the characteristics: this step occurs thanks to the creation of probability curves, from which an average or a fashion is extracted depending on the cases, for each of the geometrical characteristics searched and therefore different for each of the elements (of all the measurements that have to do with the width of the beams of the 1975 project a statistical distribution is created of which the average will then be calculated, this was used method for all those analysed characteristic quantities); these weighted averages will be used as characteristic values of that year for that determined size of the reference structural element, thus trying to create, through interpolation, a database of all the most frequent geometric quantities used in the year of interest. In fact, if I have a reference value for 1987 and one for 1993, by interpolation I will be able to obtain the value in the year of construction of my building, for example 1990.

4.4.1 BEAMS

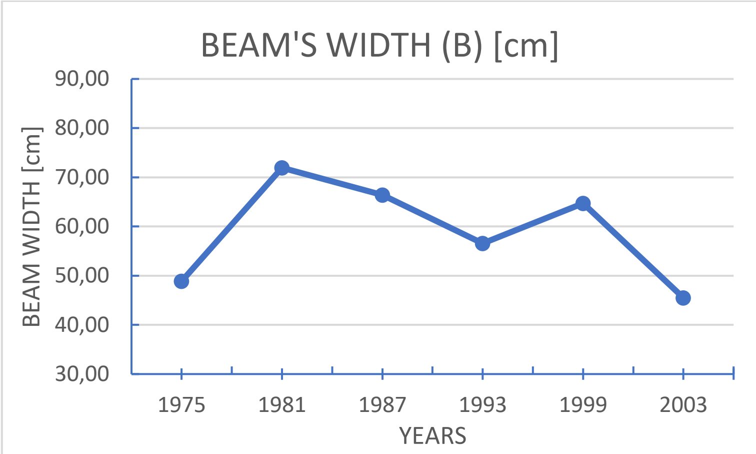
Regarding the beams, statistical distributions have been created for each year for the following quantities:

- Span (L);
- Height (H);
- Thickness (B);
- L / H;
- B / H;
- % of reinforcement on the first support;
- % of reinforcement in the span;
- % of reinforcement on the second support.

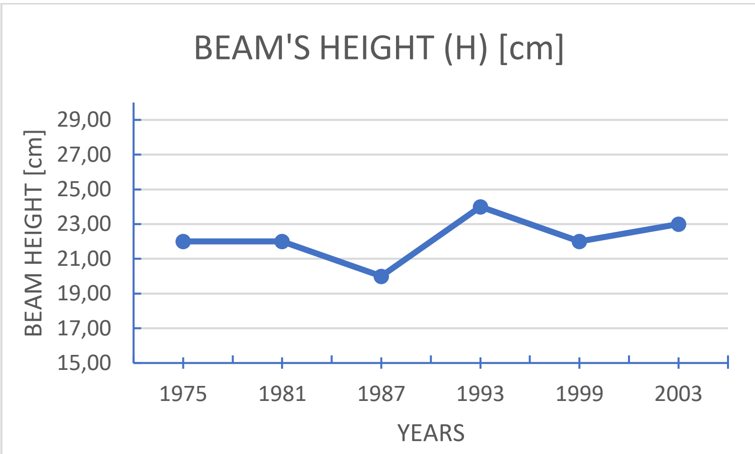
The results of this probabilistic analysis were then reported on some graphs that show the variation of these quantities depending on the age of construction and therefore, as has been said several times, by the legislation respected for the design of the structure.



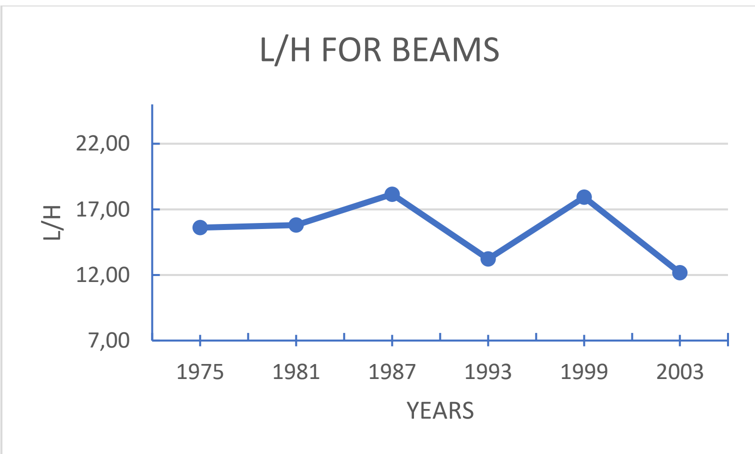
YEAR	SPAN (L) [cm]
1975	346.63
1981	347.52
1987	363.07
1993	325.43
1999	394.16
2003	296.40



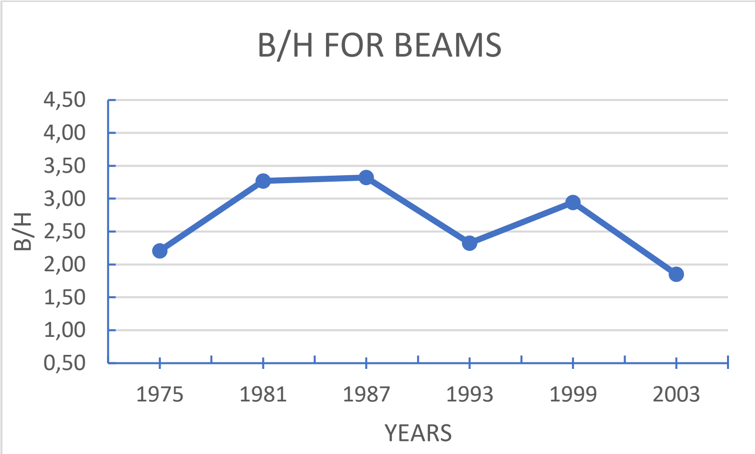
YEAR	B[cm]
1975	48.88
1981	71.94
1987	66.43
1993	56.59
1999	64.74
2003	45.50



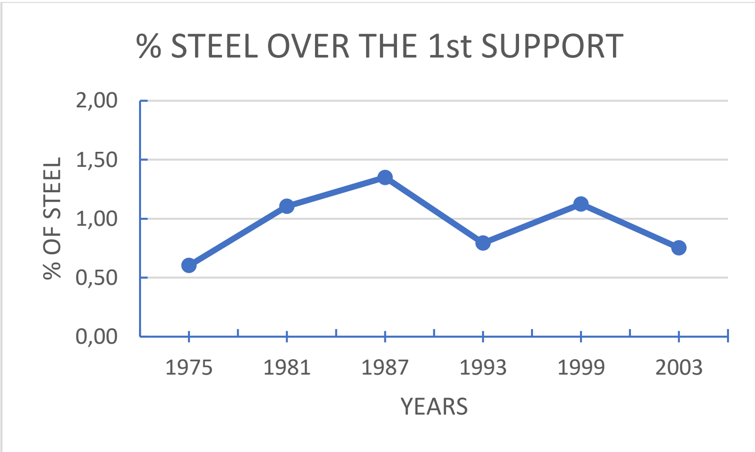
YEAR	H[cm]
1975	22.00
1981	22.00
1987	20.00
1993	24.00
1999	22.00
2003	23.00



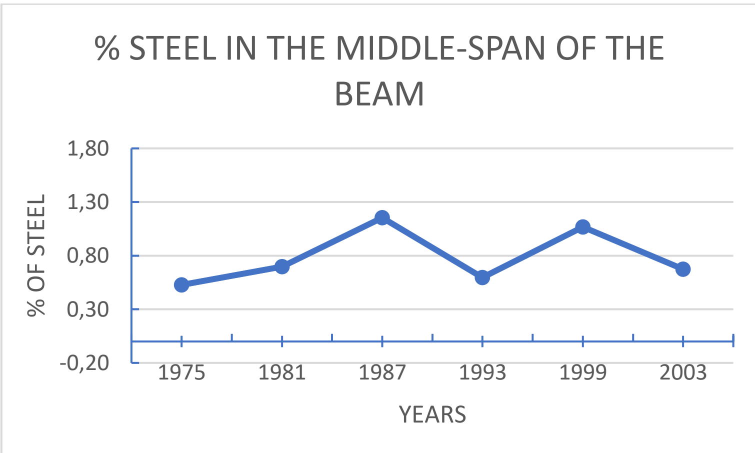
YEAR	L/H
1975	15.61
1981	15.80
1987	18.15
1993	13.22
1999	17.92
2003	12.16



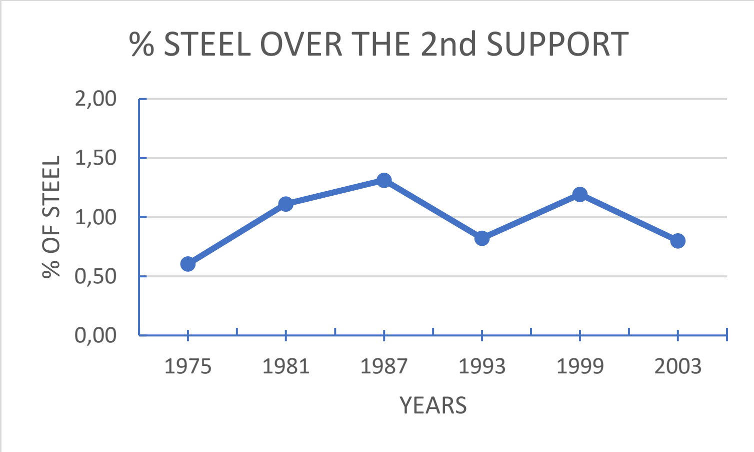
YEAR	B/H
1975	2.21
1981	3.27
1987	3.32
1993	2.33
1999	2.94
2003	1.85



YEAR	% REINFORCEMENT ON SUPPORT 1
1975	0.60
1981	1.10
1987	1.35
1993	0.79
1999	1.12
2003	0.75



YEAR	% REINFORCEMENT IN MIDDLE-SPAN
1975	0.53
1981	0.70
1987	1.15
1993	0.60
1999	1.07
2003	0.67



YEAR	% REINFORCEMENT ON SUPPORT 2
1975	0.60
1981	1.11
1987	1.31
1993	0.82
1999	1.19
2003	0.80

For what concern transversal reinforcements things are different; today's legislation calls for a denser presence of stirrups in the critical areas of the beam and columns, in general their intersection point, the node. According to the regulations in force at those times this consideration was not made: in fact, in all the projects analysed, the stirrup was not only made constantly along a beam, but was also constant throughout the construction, once a scheme was adopted, the same was respected for all the beams of the condominium. In the first three cases the brackets were not even of the mandatory type today, the closed bracket, but open brackets, less safe because they guarantee less resistance to shear.

YEAR	TRANSVERSAL REINFORCEMENT [cm]
1975	ϕ 6 / 25
1981	ϕ 6 / 20
1987	ϕ 6 / 15
1993	ϕ 6 / 15
1999	ϕ 8 / 14
2003	ϕ 8 / 20

Table 5- Transversal reinforcement of beams

What is written in the table above means for example for the year 1975: "stirring at a constant distance equal to 25 cm with 6 mm diameter brackets".

The shear resistance was in fact ensured by the use of the bent steels, that is steel bars that were positioned in the upper part of the beam in the node, and then, bent at 45 °, to be in the lower portion in the span, so as to ensure in the bent part the due shear strength, in the part that is not bent, the resistance to the two greatest moments acting on the beam, so at the node and in the span. This is the other big difference compared to today's practices, where bent steels are no more used, in favour of easier process by carpenters.

4.4.2 COLUMNS

Below is an explanation of the work carried out for the columns of the structures: starting from the database created previously some probabilistic distributions were studied and then the values that were needed more frequently were calculated, the average was calculated for the following characteristics:

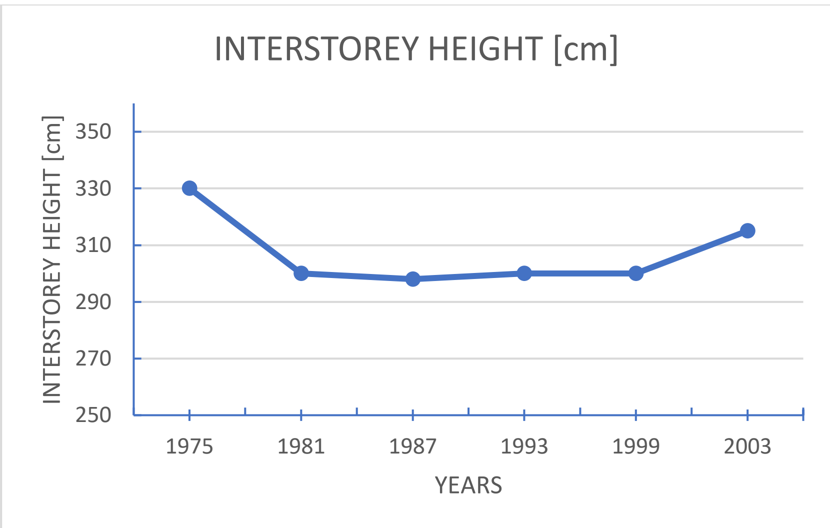
- Inter-floor height (H);
- Thickness in the main direction (X1);
- Thickness in the secondary direction (X2);
- % longitudinal reinforcement within the column section;

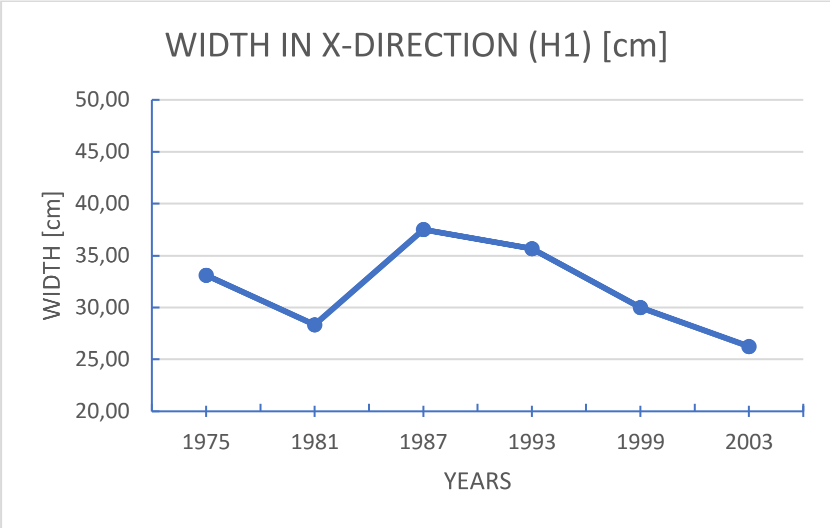
these were considered the fundamental parameters and of interest for the proposed methodology.

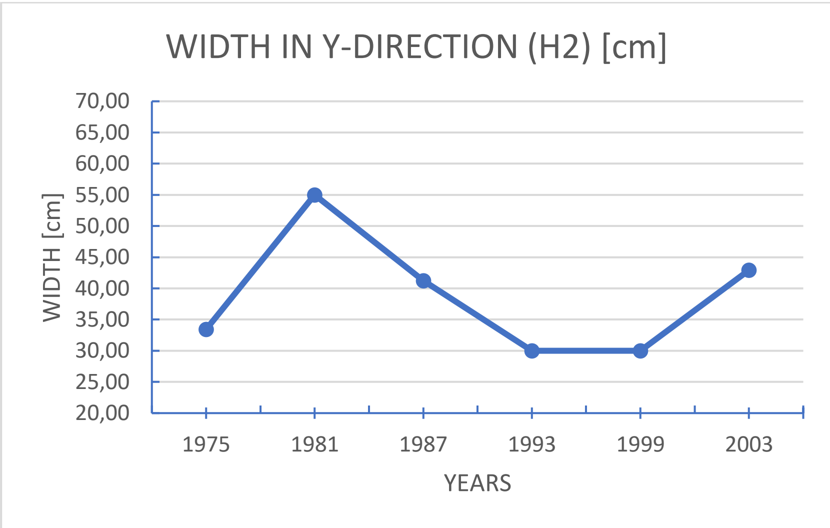
A big difference compared to what was said for the trusses is that they have divided the columns into perimetric and central ones, this to find the differences between the two categories within a standard plan, so as to have a clearer and more detailed measure. The same characteristics were calculated for both categories.

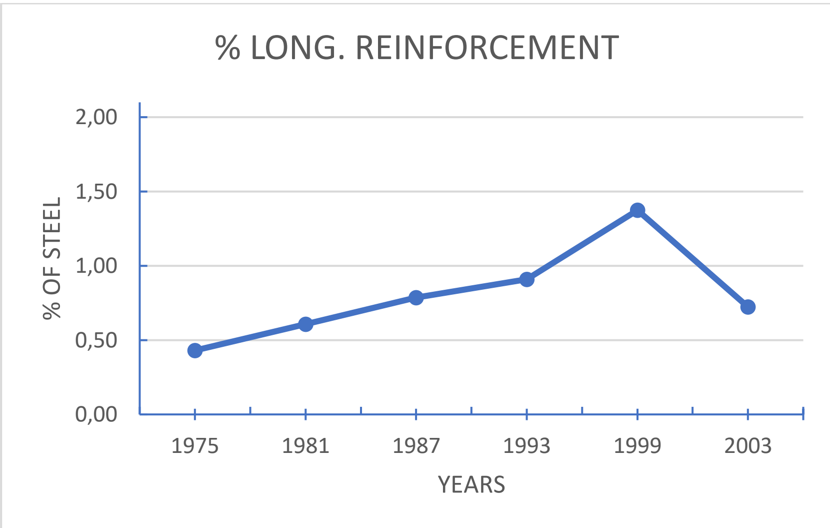
YEAR	POSITION	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]	% LONGITUDINAL REINFORCEMENT
1975	PERIMETRALS	330	33.13	33.44	0.43
1981	PERIMETRALS	300	28.33	55.00	0.61
1987	PERIMETRALS	298	37.50	41.25	0.79
1993	PERIMETRALS	300	35.67	30.00	0.91
1999	PERIMETRALS	300	30.00	30.00	1.37
2003	PERIMETRALS	315	26.25	42.92	0.72

Table 6- Information about perimetrals columns



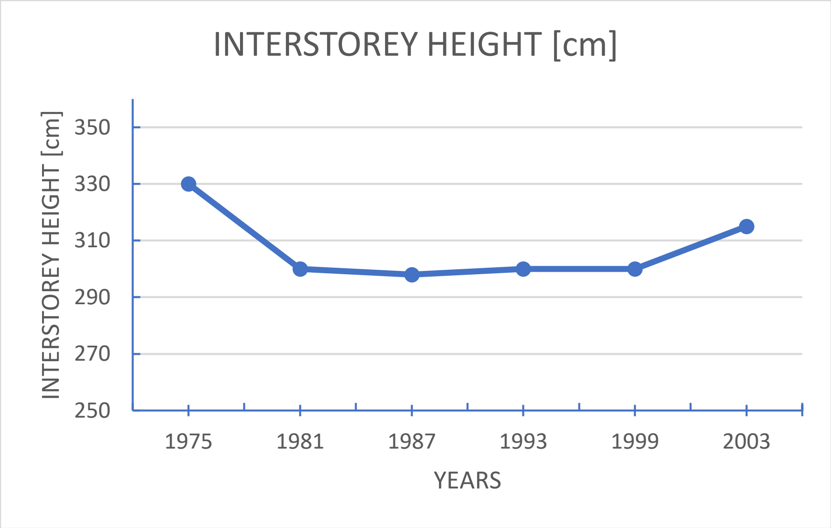

WIDTH IN X-DIRECTION (H1) [cm]
WIDTH [cm]
50,00
45,00
40,00
35,00
30,00
25,00
20,00
1975
1981
1987
1993
1999
2003
YEARS

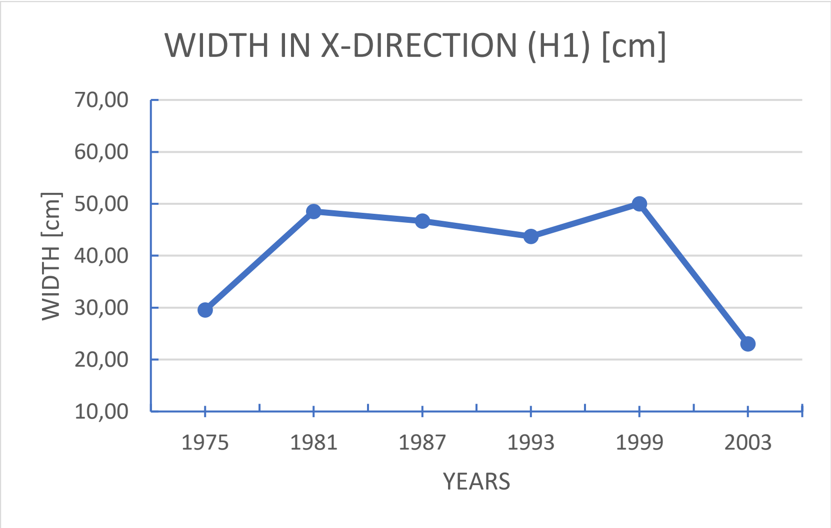

WIDTH IN Y-DIRECTION (H2) [cm]
WIDTH [cm]
70,00
65,00
60,00
55,00
50,00
45,00
40,00
35,00
30,00
25,00
20,00
1975
1981
1987
1993
1999
2003
YEARS

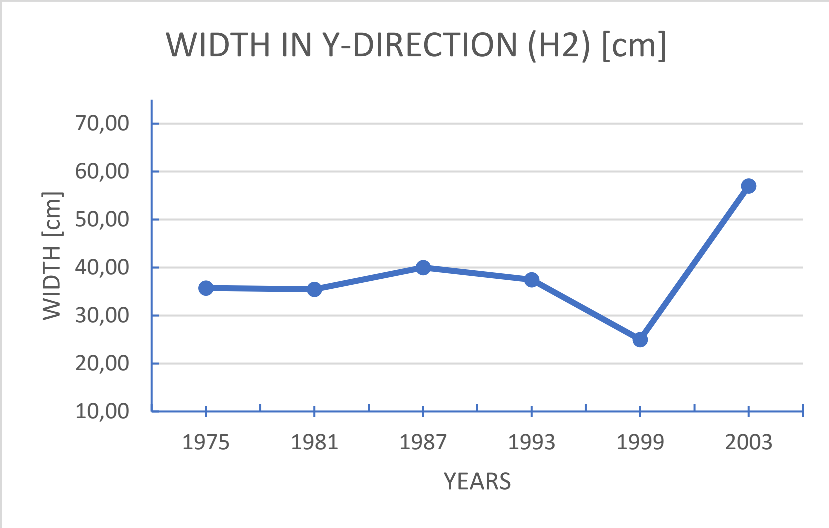

% LONG. REINFORCEMENT
% OF STEEL
2,00
1,50
1,00
0,50
0,00
1975
1981
1987
1993
1999
2003
YEARS

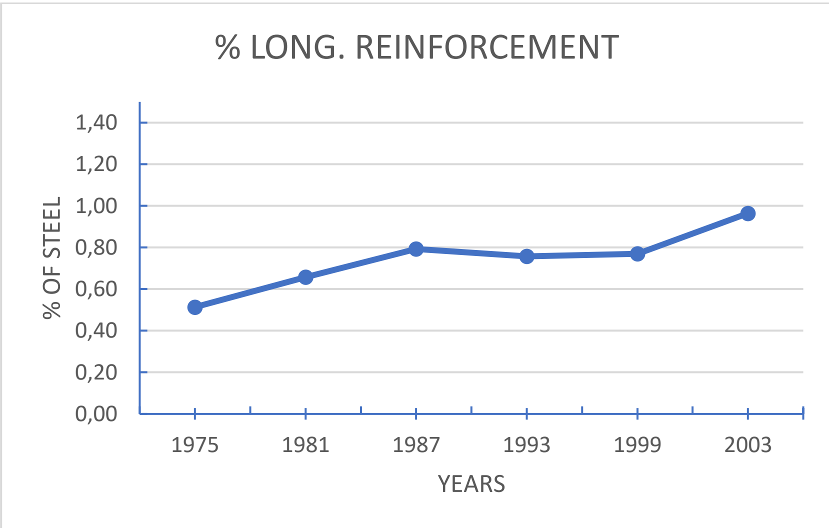
YEAR	POSITION	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]	% LONGITUDINAL REINFORCEMENT
1975	CENTRALS	330	29.52	35.71	0.51
1981	CENTRALS	300	48.50	35.50	0.66
1987	CENTRALS	298	46.67	40.00	0.79
1993	CENTRALS	300	43.75	37.50	0.76
1999	CENTRALS	300	50.00	25.00	0.77
2003	CENTRALS	315	23.00	57.00	0.96

Table 7- Informations about centrals columns









Also of note in this case is the method used for the arrangement of the transverse reinforcements along the element, exactly as for the beams, along the longitudinal sections of the abutment no differences were found, with a spacing between the brackets always constant inside of the construction in question; the scheme now imposed by the regulations on the subject has therefore not been adopted, that is, denser stirruping in the critical areas, paying particular attention to the beams / columns, but a constant and repetitive scheme has been followed for the entire building. The brackets used are in this case closed mesh for the three most recent projects, open instead to the older ones. Once again to underline the great difference in the construction details of the past with respect to the present, thanks to scientific and technological research the changes have been decisive and manifold, especially with regard to the resistance to earthquakes.

4.4.3 SHEAR WALL

The approach adopted for the other structural elements is different, mainly for shear wall and floors. As for the shear wall, whose typical job at the time was that of an elevator shaft and room used as a staircase, contrary to what happens today where they are used as real earthquake-resistant structures. Within a residential building these elements are present in small quantities, a maximum of a couple of units (in the sense that if there are ten columns within the plan of a condominium, there will be one of them, maximum two). These are characterized by different geometrical characteristics, among which the following have been analysed and taken into consideration:

- Coordinates of the center of gravity of the two shear wall that make up the space (x_{BAR} and y_{BAR});
- Distance between the two shear wall that make up a compartment;
- Length of the two shear wall (usually the same);
- Thickness of the two shear wall, (usually the same);
- Reinforcement present in the section (longitudinal and transverse).

The data collected are shown in the tables at the end of the chapter.

To highlight the probably most important characteristic with regard to this structural element, which is not its size nor the percentage of steel present in the section; the shear wall are elements of high rigidity in any case, therefore fundamental for a seismic analysis because they decisively influence the distribution of masses and stiffnesses in the process of seismic analysis of a structure; their decisive parameter is their position within the building plan, this was calculated using the ratio between the coordinates of the center of gravity and the total coordinates of the plant:

Where exactly x_{TOT} and y_{TOT} represent the actual dimensions of the construction.

The previously mentioned measurements will obviously refer to a reference system composed of two Cartesian axes X and Y with the origin placed in one of the vertices of the structure.

These parameters (x and y) will vary between zero and unity, where the first represents the entire left side of the building and the second the one on the right, while the value 0.5 indicates the center.

YEAR	N° SHEAR WALL	X-BAR [cm]	Y-BAR [cm]	WIDTH[cm]	LENGTH[cm]
1975	26	990	725	20	180
1975	27	1155	725	20	180
1975	BUILDING DIMENSIONS	X-TOT [cm]	Y-TOT [cm]	DISTANCE BTW SHEAR WALL[cm]	
1975		2000	1380	165	
1975		PLAN POSITION-X DIRECTION	PLAN POSITION-Y DIRECTION		
1975		0.54	0.53		
1975	N° SHEAR WALL	LONGITUDINAL REINFORCEMENT[cm]	TRANSVERSAL REINFORCEMENT[cm]		
1975	26	8 + 8 ϕ 12	ϕ 6/ 20		
1975	27	8 + 8 ϕ 12	ϕ 6/ 20		
YEAR	N° SHEAR WALL	X-BAR [cm]	Y-BAR [cm]	WIDTH[cm]	LENGTH[cm]
1981	O 1	1058	720	25	160
1981	O 2	1232	720	25	160
1981	BUILDING DIMENSIONS	X-TOT [cm]	Y-TOT [cm]	DISTANCE BTW SHEAR WALL[cm]	
1981		2150	1390	170	
1981		POSITION IN THE PLAN-X DIRECTION	POSITION IN THE PLAN-Y DIRECTION		
1981		0.57	0.52		
1981	N° SHEAR WALL	LONGITUDINAL REINFORCEMENT[cm]	TRANSVERSAL REINFORCEMENT[cm]		
1981	O 1	6 + 6 ϕ 12	ϕ 6/ 15		
1981	O 2	6 + 6 ϕ 12	ϕ 6/ 15		
YEAR	N° SHEAR WALL	X-BAR [cm]	Y-BAR [cm]	WIDTH[cm]	LENGTH[cm]
1987	4	801	1112	20	110
1987	5	1171	1112	20	110
1987	BUILDING DIMENSIONS	X-TOT [cm]	Y-TOT [cm]	DISTANCE BTW SHEAR WALL[cm]	
1987		1972	1380	370	
1987		POSITION IN THE PLAN-X DIRECTION	POSITION IN THE PLAN-Y DIRECTION		
1987		0.49	0.81		
1987	N° SHEAR WALL	LONGITUDINAL REINFORCEMENT[cm]	TRANSVERSAL REINFORCEMENT[cm]		
1987	4	7 + 7 ϕ 12	ϕ 6/ 15		
1987	5	7 + 7 ϕ 12	ϕ 6/ 15		
YEAR	N° SHEAR WALL	X-BAR [cm]	Y-BAR [cm]	WIDTH[cm]	LENGTH[cm]
1993	P 8	800	650	20	195
1993	P 9	980	650	20	195

1993	BUILDING DIMENSIONS	X-TOT [cm]	Y-TOT [cm]	DISTANCE BTW SHEAR WALL[cm]	
1993		1880	1115	180	
1993		POSITION IN THE PLAN-X DIRECTION	POSITION IN THE PLAN-Y DIRECTION		
1993		0.45	0.58		
1993	N° SHEAR WALL	LONGITUDINAL REINFORCEMENT[cm]	TRANSVERSAL REINFORCEMENT[cm]		
1993	P 8	8 + 8 ϕ 12	ϕ 6/ 15		
1993	P 9	8 + 8 ϕ 12	ϕ 6/ 15		
YEAR	N° SHEAR WALL	X-BAR [cm]	Y-BAR [cm]	WIDTH[cm]	LENGTH[cm]
1999	M 401	783	610	20	200
1999	M 402	783	790	20	200
1999	BUILDING DIMENSIONS	X-TOT [cm]	Y-TOT [cm]	DISTANCE BTW SHEAR WALL[cm]	
1999		1410	1046	180	
1999		POSITION IN THE PLAN-X DIRECTION	POSITION IN THE PLAN-Y DIRECTION		
1999		0.56	0.67		
1999	N° SHEAR WALL	LONGITUDINAL REINFORCEMENT[cm]	TRANSVERSAL REINFORCEMENT[cm]		
1999	M 401	8 + 8 ϕ 12	ϕ 6/ 15		
1999	M 402	8 + 8 ϕ 12	ϕ 6/ 15		
YEAR	N° SHEAR WALL	X-BAR [cm]	Y-BAR [cm]	WIDTH[cm]	LENGTH[cm]
2003	1	719	877	20	200
2003	2	904	877	20	200
2003	BUILDING DIMENSIONS	X-TOT [cm]	Y-TOT [cm]	DISTANCE BTW SHEAR WALL[cm]	
2003		1050	1500	185	
2003		POSITION IN THE PLAN-X DIRECTION	POSITION IN THE PLAN-Y DIRECTION		
2003		0.41	0.58		
2003	N° SHEAR WALL	LONGITUDINAL REINFORCEMENT[cm]	TRANSVERSAL REINFORCEMENT[cm]		
2003	1	7 + 7 D 12	ϕ 6/ 15		

Table 8- Informations about shear wall

4.4.4 SLABS

As far as the floors are concerned, the perfect equality of the various joists has been found, which go to make up the classic concrete slab. Within the entire building for the first three projects in temporal order the joists had the same cross section, while the geometric sizes of the sections remain unchanged when the reinforcement used for the most recent ones change. This always underlines the preference on the part of manufacturers and designers of the time to simplify the work to the detriment of safety, given perhaps by long professional experiences.

The arrangement of these joists is in most cases unique, therefore always in the same direction, this is the direction perpendicular to the main one of the beams; for example in those projects where the frames are warped in a single direction (x), the joists are arranged in the direction perpendicular to it (y). Also for this reason it has often been necessary to insert curb-stones to reduce the span and therefore the tension acting on the joists.

The parameters registered for the floors are:

- Thickness of the floor;
- Width of the joist (b);
- Height of the joist (H);
- Coating jet (s);
- Spacing between the various joists;
- Longitudinal reinforcement present in the joist;

these are summarized and shown in the tables below.

YEAR	N° OF SLAB	SLAB THICKNESS [cm]	JOIST WIDTH (b) [cm]	SPACING BETWEEN JOISTS[cm]
1975		30	10	50
1975			COVERING JET (s) [cm]	JOIST HEIGHT (H) [cm]
1975			4	22
1975		LONGITUDINAL REINFORCEMENT	SUPERIOR [cm]	INFERIOR [cm]
1975			2.26	0.79
YEAR	N° OF SLAB	SLAB THICKNESS [cm]	JOIST WIDTH (b) [cm]	SPACING BETWEEN JOISTS[cm]
1981		30	10	50
1981			COVERING JET (s) [cm]	JOIST HEIGHT (H) [cm]
1981			4	22
1981		LONGITUDINAL REINFORCEMENT	SUPERIOR [cm]	INFERIOR [cm]
1981			2.33	1.13

YEAR	N° OF SLAB	SLAB THICKNESS [cm]	JOIST WIDTH (b) [cm]	SPACING BETWEEN JOISTS[cm]
1987		28	10	50
1987			COVERING JET (s) [cm]	JOIST HEIGHT (H) [cm]
1987			4	16
1987		LONGITUDINAL REINFORCEMENT	SUPERIOR [cm]	INFERIOR [cm]
1987	N 201		1.13	1.54
1987	N 202		1.13	1.13
1987	N 203		1.13	1.54
1987	N 205		1	0.79
1987	N 206		1	0.5
1987	N 207		1.29	0.79
1987	N 208		0.79	0.79
1987	N 209		2.51	0.79
1987	N 210		2.26	1
1987	N 211		2.26	1
1987	N 212		1.13	1.13
1987	N 213		2.04	0.5
1987				
1987		MEAN	1.4725	0.958333333
YEAR	N° OF SLAB	SLAB THICKNESS [cm]	JOIST WIDTH (b) [cm]	SPACING BETWEEN JOISTS[cm]
1993		30	10	50
1993			COVERING JET (s) [cm]	JOIST HEIGHT (H) [cm]
1993			4	20
1993		LONGITUDINAL REINFORCEMENT	SUPERIOR [cm]	INFERIOR [cm]
1993	S1		0.79	0.79
1993	S2		0.79	0.79
1993	S3		0.79	0.79
1993	S4		0.79	0.79
1993	S5		0.79	0.79
1993	S6		2.67	0.79
1993	S7		2.67	0.79
1993	S8		1.92	1.13
1993	S9		1.92	0.79
1993	S10		2.67	0.79
1993	S11		2.33	0.79
1993	S12		2.67	1.13
1993	S13		0.79	0.79
1993	S14		0.79	0.79
1993	S15		0.79	0.79
1993	S16		2.67	1.13
1993	S17		2.67	0.79
1993				
1993		MEAN	1.677058824	0.85

YEAR	N° OF SLAB	SLAB THICKNESS [cm]	JOIST WIDTH (b) [cm]	SPACING BETWEEN JOISTS[cm]
1999		30	10	50
1999			COVERING JET (s) [cm]	JOIST HEIGHT (H) [cm]
1999			4	30
1999		LONGITUDINAL REINFORCEMENT	SUPERIOR [cm]	INFERIOR [cm]
1999	1		1.13	1.13
1999	2		1.13	1.13
1999	3		1.13	1.58
1999				
1999		MEAN	1.13	1.28
YEAR	N° OF SLAB	SLAB THICKNESS [cm]	JOIST WIDTH (b) [cm]	SPACING BETWEEN JOISTS[cm]
2003		35	10	50
2003			COVERING JET (s) [cm]	JOIST HEIGHT (H) [cm]
2003			5	25
2003		LONGITUDINAL REINFORCEMENT	SUPERIOR [cm]	INFERIOR [cm]
2003	A		0.79	1.54
2003	B		0.5	0.79
2003	C		0.5	0.5
2003				
2003		MEAN	0.596666667	0.943333333

Table 9- Informations about slabs

5. VALIDATION OF THE NEW PROCEDURE

The proposed new procedure consists in offering a rapid, non-invasive and economic method for identifying the buildings on which there is a greater urgency for structural intervention. The proposed method is based on a geometric sampling and the relative creation of a database in which I can find the variations of some quantities of interest, such as the thickness and height of the beams and the columns: knowing the year of construction of a particular condominium under examination, I derive the standard geometric characteristics to be used in the construction of the virtual model. The core of the procedure is in fact the creation of a frame that is complementary to the real one of the structure under examination, but "standardized": with some dimensions that are taken from the database organized depending on the year of construction of the building and adapted to the plant and to the volume of the structure of interest.

The main purpose of this operation is to shear times and costs in a dizzying manner, while providing a good level of reliability and a first, superficial analysis of the seismic vulnerability of the structure. For a complete analysis the professional in charge must first conduct a historical analysis at the competent offices, inspect the product several times, take dozens of samples depending on the degree of detail to be achieved, bring these samples to special laboratories where they will be subjected to various tests to obtain the characteristic resistances of the materials, all operations which, when added together, take up large amounts of time and money; instead with this new simplified procedure, once the building layout and the cubic volume occupied by it have been obtained, having the database of geometric dimensions available and making some approximation on some structural elements, the professionals will be able to construct the relative "virtual model” and give a first opinion on the structural state and on the earthquake resistance that the structure can withstand; then comparing the results of the simplified analyses relating to the number of buildings in the institution's possession, there will be an important indicator on which of these structures has the greatest need for immediate intervention, once the structure in the most critical conditions has been selected, this will be subjected to the normal legislative and planning procedure required by the NTC2018.

To validate this procedure it was decided to model a condominium in Turin with two different structures: on the one hand the real one, extrapolated from the final projects delivered to the office of civil engineering in 1987, on the other the "virtual structure", built on the bases of the database previously created and illustrated. Submitting the two models to dynamic modal seismic analysis, then proceeded with the comparison of the results; in addition to the fundamental periods and the masses activated by the various modes of vibrating, a parameter considered fundamental in this case is the percentage of earthquake absorbed by the structure with respect to the amount of acceleration to the ground that it should absorb if it were a structure in the phase of design:

$$\zeta_E = \frac{maximum\ horizontal\ action\ suistanable\ from\ the\ existing\ structure}{project\ seismic\ action\ in\ case\ of\ a\ new\ construction}$$

For the seismic analysis of the structure and for the analysis of reinforced concrete beams and columns the electronic computer was used, using the following calculation program: Dolmen Win (R), version 11.0 of 2011 produced, distributed and assisted by Cdm Dolmen srl, based in Turin, Via Drovetti 9 / F. In support of the program, an extensive user manual is provided containing, among other things, a vast series of validation tests both on classic examples of Construction Science, and on particularly demanding structures that can be found in the specialized bibliography. The reliability of the calculation code is guaranteed by the existence of extensive supporting documentation. It is also possible to obtain graphic representations of deformations and stresses of the structure. At the end of the processing, the quality of the solution is also evaluated, based on the equality of external work and deformation energy. Dolmen Win allows a detailed analysis of the behaviour of the entire structure in a linear elastic field, taking into account the stiff behaviour of even complex partitions and floors considered with their actual rigidity. It is also possible to select the degree of refinement of the analysis of complex elements using increasingly detailed meshes.

The structure has been schematized excluding the contribution of elements with negligible rigidity and resistance compared to the main ones. The three-dimensional frame construction, the floors and the vertical walls with high rigidity (elevator shaft, concrete walls) was therefore considered. The foundation plinths are assimilated to elastic constraints of which the stiffness constant is provided. The structure is modelled with the finite element method, applied to three-dimensional systems. The elements used are both one-dimensional (beam with possible internal disconnections), and two-dimensional (triangular and quadrangular plates and membranes). The constraints are considered punctual and inserted through the six elastic stiffness constants, or as rod elements resting on elastic ground.

The analysis of the structure in question was done using the usual methods of Construction Science and in compliance with the laws and regulations in force:

- Law 5/11/1971 n. 1086: Rules for the regulation of reinforced, normal and prestressed concrete conglomerates and metal structures.
- Presidential Decree 6/6/2001 n. 380: Consolidated text of the legislative and regulatory provisions on construction.
- D. M. 14/1/2018: Technical standards for construction.

In accordance with the before mentioned regulations, the following actions were considered in the calculations:

- own structural weights
- permanent loads carried by the structure
- variable loads on the floors, snow, wind

- earthquake-simulating plane forces, obtained through dynamic modal analysis.

The seismic data are the same in both cases, summarized in the following figure:

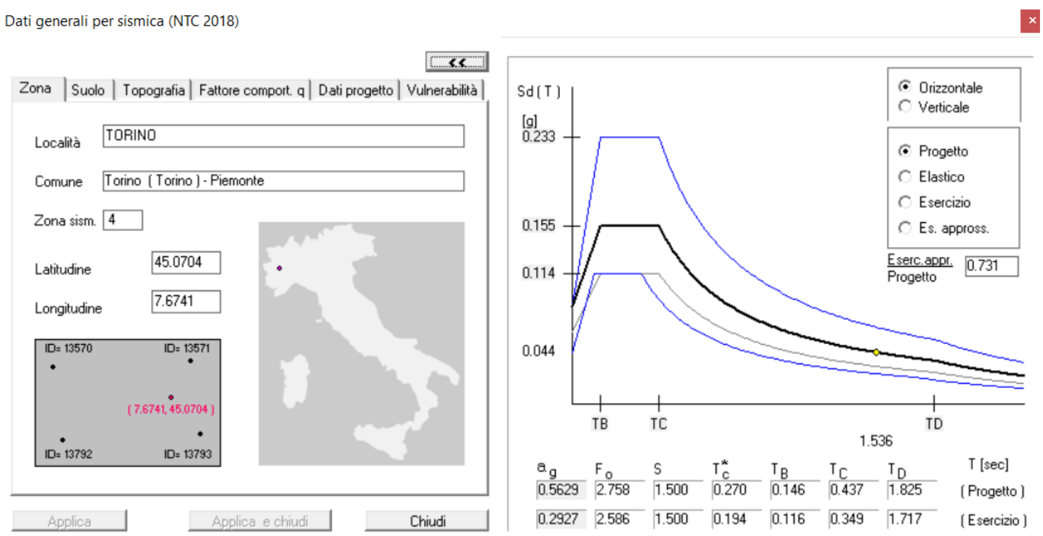


Figure 20- Seismic conditions on Dolmen

We proceed now with the detailed analysis of the two models, and then conclude with a comparison of the results.

5.1 REAL STRUCTURE

Through the office of the Civil Engineering Department of the Piedmont Region a new project was taken, dating back to 1987, it is a condominium located in Torino, six floors above ground plus one basement, construction method with reinforced concrete frames; the reference legislation of the time was the law of 5 November 1971, n.1086.

The building has a rectangular plan, with dimensions of approximately 30.8 x 11.4 m and an overall height of 17.85 m from the ground level plus roof. The structure consists of frames of columns and reinforced concrete beams cast in place of varying sizes, connected to each other by infinitely rigid masonry slabs 25 cm thick, these frames are three and present in only one direction, not connected to each other in the direction orthogonal to the main one. The fact of the absence of connections between the various frames, therefore the lack of frames in the y direction is usual for the buildings of the time, in fact it was not yet mandatory to combine all the resistant elements, given the backwardness of the seismic laws , as the database data shows. The floors will be modelled by the "rigid floor" command, therefore not subject to deformation. The foundations are superficial, consisting of separate plinths (constraint present on Dolmen), while the interlocking constraint for the foundations of the wall against the ground will be used, in fact along the perimeter sections there are walls in reinforced concrete 30 cm thick to contain the thrust of the earth. There are two lift compartments consisting of two walls, each in reinforced concrete 20 cm thick, in a symmetrical position with respect to the center. The inter-plane height of the columns varies according to the planes, is equal to 3.30 m for the ground floor, 3.00 m for the first and 2.95 m for those above. The original drawings where all the dimensions of the various structural elements are shown are available at the bottom of the text (Annex 3). The figures below represent the Dolmen model (plan view for z = 0, axonometry with real dimensions of the various structural elements).

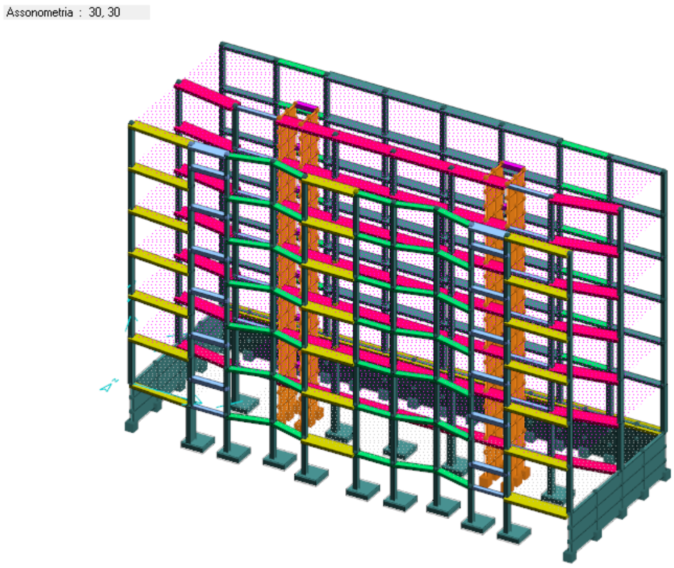


Figure 21- Realistic Dolmen model

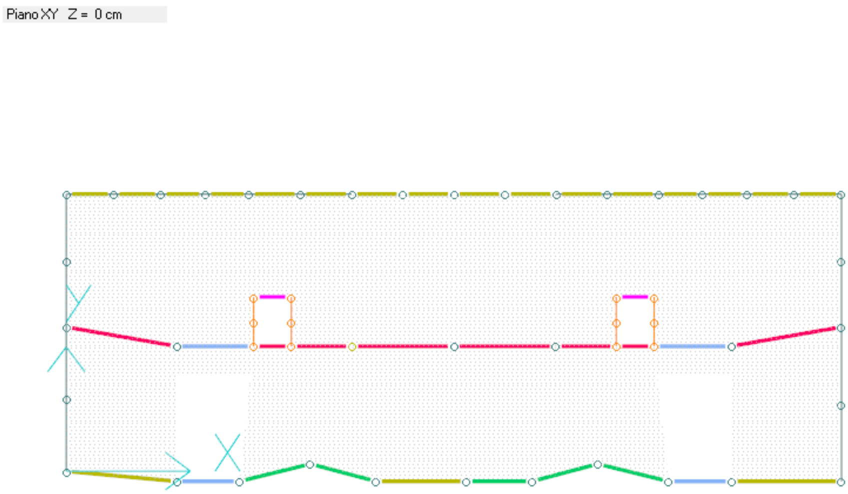


Figure 22- Plan view of the realistic model

Since this is a comparison between the two different structures, the loads have been set identical in the two cases, without taking into account what is reported in the calculation report of the project, but making sure that they had the same values in the two different situations. The roof consists of reinforced concrete beams and was modelled as a load above the floor slab, given its limited contribution compared to the remaining portion of the building; the stairs in the same way were considered only as a load and associated to the beams that surround the space, entrusting to the same elements obviously also the variable load imposed by the legislation. The load conditions used are summarized in the table below.

Schede condizioni

Nuova scheda | Modifica scheda | Duplica scheda | Elimina scheda

Num.	Nome	Coeff.	N° carichi	Categoria in NTC2018	Categoria in norme preced
001)	Peso_proprio________	1	524	Peso proprio	Altro ...
002)	Permanente__________	1	181	Permanente	Altro ...
003)	A:Var_abitazione____	1	14	A:Var abitazione	Altro ...
004)	Neve_(<1000m_slm)___	1	2	Neve (<1000m slm)	Altro ...
005)	p.p._fondazione	1	150	Peso proprio fondaz	Permanente fondaz
006)	perm._fondaz.	1	40	Permanente fondaz	Permanente fondaz
007)	p.p._tetto	1	32	Permanente fondaz	Permanente fondaz
008)	perm._scale	1	16	Permanente	Permanente
009)	var_scale	1	16	A:Var abitazione	Var.abitazione

Figure 23- Loads conditions

Regarding the strengths of the materials, these were extrapolated from the values provided by the graphs depending on the year of construction and knowing that in the project they used steel type FeB44, shown in the figure below:

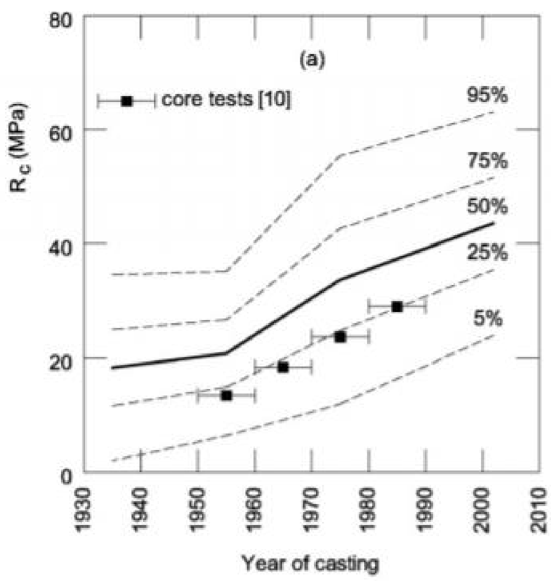


Figure 24- Characterisic resistances of cls

Type of steel	Yield strength – f_y MPa (ksi)	Tensile strength – f_u MPa (ksi)
Common steel	-	280 ~ 500 (40.6 ~ 72.5)
Aq.42	≥ 230 (≥ 33.4)	420 ~ 500 (60.9 ~ 72.5)
Aq.50	≥ 270 (≥ 39.2)	500 ~ 600 (72.5 ~ 87.0)
Aq.60	≥ 310 (≥ 45.0)	600 ~ 700 (87.0 ~ 102)
FeB32	≥ 320 (≥ 46.4)	≥ 500 (≥ 72.5)
FeB38	≥ 380 (≥ 55.1)	≥ 460 (≥ 66.7)
FeB44	≥ 440 (≥ 63.8)	≥ 550 (≥ 79.8)

Figure 25- Characteristic resistances of steel

The modal seismic analysis is not influenced by the quantity of reinforcement present in the section area in a decisive way, but only by the dimensions of the sections themselves and by the material composing them, for this reason it was not necessary to proceed with the definition of the reinforcement inside columns and beams, but it was sufficient to model the before mentioned elements with the correct geometric dimensions.

The results provided by the Dolmen calculation software for both the seismic analysis of modal type (linear dynamic) and the static analysis, which returns the torques not calculated from the dynamic one, will be shown below. The solicitations acting on the various elements of the structure at the bottom of the text (Annex 4) will also be available.

5.1.1 RESULTS OF THE SEISMIC DYNAMIC ANALYSIS FOR THE REAL MODEL

lavoro :\REALE_

```
PARAMETRI DI CALCOLO:

Modello generale
Assi di vibrazione:   X   Y
Combinazione quadratica completa (CQC)

DATI PROGETTO

Edificio  sito in località  TORINO ( long. 7.674   lat. 45.070400 )

Categoria del suolo di fondazione  = C

Coeff. di amplificazione stratigrafica Ss = 1.500

Coeff. di amplificazione topografica   ST = 1.000

S   =  1.500

Vita nominale dell'opera VN     = 50  anni

Coefficiente d'uso CU           = 1.0
```

Periodo di riferimento VR = 50.0

PVR : probabilità di superamento in VR = 10 %

Tempo di ritorno = 474

Coeff. di smorzamento viscoso = 5.0

Valori risultanti per :
ag 0.563 [g/10]
Fo 2.758
TC* 0.270

Fattore di comportamento q = 1.500

Rapporto spettro di esercizio / spettro di progetto = 0.729

CONDIZIONI DI RIFERIMENTO	COEFFICIENTE	PESO RISULTANTE [daN]
1.	1.000	925981.5
2.	1.000	907659.9
3.	0.300	138817.2
8.	1.000	28800.0
9.	0.300	2400.0
7.	1.000	202915.7

*** TABELLA AUTOVETTORI ***

n	PERIODO [sec]	MASSA ATTIVATA %X	%Y	%Z	COEFFICIENTI DI CORRELAZIONE n+1	n+2	n+3	n+4	n+5	n+6	n+7
1	2.364898	0.000	75.071	0.000	0.668	0.066	0.003	0.002	0.001		
2	2.203990	2.581	0.261	0.000	0.099	0.003	0.002	0.002			
3	1.634453	83.386	0.007	0.000	0.006	0.004	0.003				
4	0.519880	8.827	0.000	0.000	0.246	0.077					
5	0.436702	0.477	0.008	0.000	0.261						
6	0.369334	0.000	18.794	0.000							
MASSA TOTALE		95.272	94.141	0.000							

5.1.2 RESULTS OF THE SEISMIC STATIC ANALYSIS FOR THE REAL MODEL

DATI PROGETTO

Edificio sito in località TORINO (long. 7.674 lat. 45.070400)

Categoria del suolo di fondazione = C

Coeff. di amplificazione stratigrafica Ss = 1.500

Coeff. di amplificazione topografica ST = 1.000

S = 1.500

Vita nominale dell'opera VN = 50 anni

Coefficiente d'uso CU = 1.0

Periodo di riferimento VR = 50.0

PVR : probabilità di superamento in VR = 10 %

Tempo di ritorno = 474

Coeff. di smorzamento viscoso = 5.0

Valori risultanti per :
ag 0.563 [g/10]
Fo 2.758
TC* 0.270

Fattore di comportamento q = 1.500

Rapporto spettro di esercizio / spettro di progetto = 0.729

Coeff. lambda = 1.0000
Sd = 0.022 per T1 = 2.365

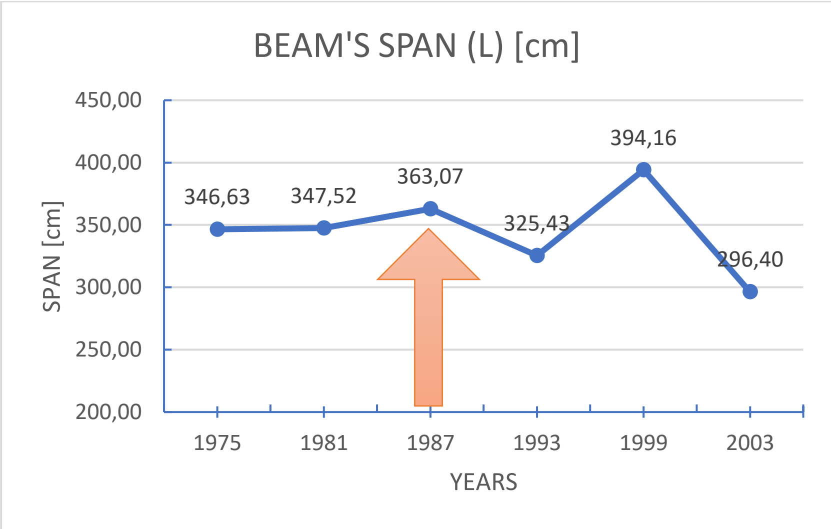
```
Numero condizioni generanti carichi sismici :    6

Cond. 001 : Peso_proprio________    con coeff.     1.000
Cond. 002 : Permanente__________    con coeff.     1.000
Cond. 003 : A:Var_abitazione____    con coeff.     0.300
Cond. 008 :          perm._scale    con coeff.     1.000
Cond. 009 :            var_scale    con coeff.     0.300
Cond. 007 :           p.p._tetto    con coeff.     1.000

Condizioni di carico sismico generate:

Cond. 020 :              Sisma X
Cond. 021 :              Sisma Y
Cond. 022 :      Torcente add. X
Cond. 023 :      Torcente add. Y

Carichi sismici :
```

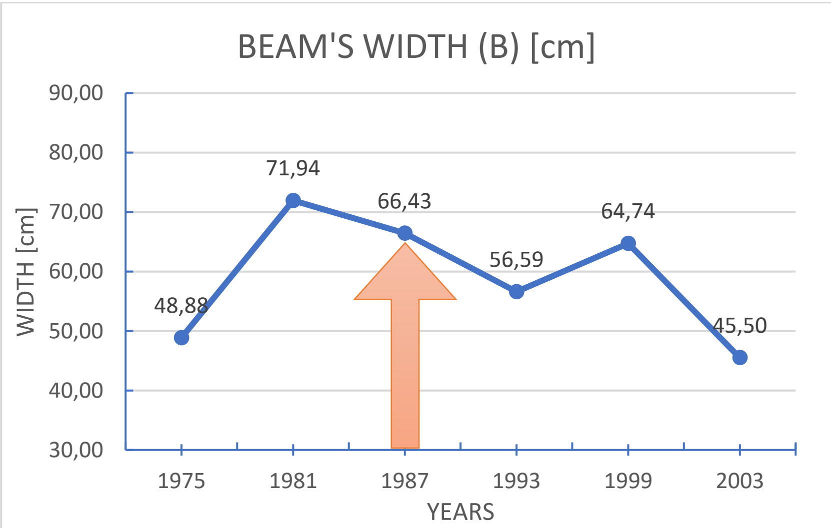
Piani	Pesi	C. distr.	Forze piano	Torc. piano X	Torc. piano Y	Bar. X	Bar. Y
cm	daN		daN	daNcm	daNcm	cm	cm
295.0	37444	0.0046	174	9847	26635	1530.3	495.2
405.0	4290	0.0064	27	266	2165	1535.0	591.4
475.6	11422	0.0075	85	3110	9379	1535.5	197.1
625.0	316851	0.0098	3111	176534	477499	1535.7	558.0
723.3	3884	0.0114	44	430	3500	1532.3	591.4
779.1	10583	0.0122	130	4720	14235	1536.0	193.2
925.0	313136	0.0145	4550	258208	698413	1535.4	558.1
1020.0	3827	0.0160	61	598	4864	1533.6	591.4
1072.0	10505	0.0168	177	6447	19442	1536.5	191.5
1220.0	312846	0.0192	5995	340240	920296	1535.6	558.0
1318.3	3835	0.0207	79	774	6299	1535.0	591.4
1375.5	10489	0.0216	227	8260	24909	1535.5	190.9
1515.0	308684	0.0238	7346	416889	1127620	1554.0	550.6
1613.3	3835	0.0253	97	948	7709	1535.0	591.4
1668.3	10489	0.0262	275	10018	30212	1535.5	190.9
1810.0	312838	0.0284	8895	504769	1365322	1535.7	558.0

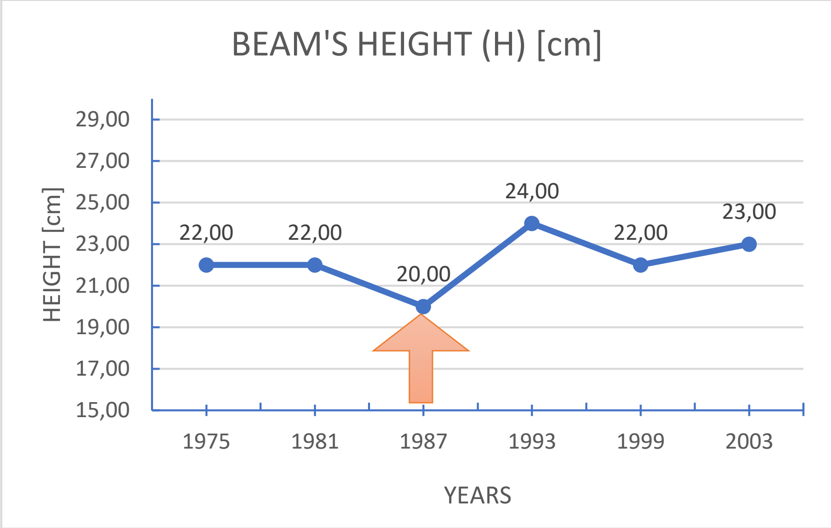
5.2 VIRTUAL STRUCTURE

As already mentioned above, the great news introduced by this method is the creation of a digital model that is complementary to the original one, so that it keeps the external envelope of the real one, but that it also refers to the database created, going to take some geometrical quantities from the data previously collected and organised, in such a way as to create within it a ‘base frame’ that in some cases will be totally different from the original. This base frame will compose the entire skeleton in reinforced concrete of the structure, which will therefore be extremely regular both in plan and height, keeping the beam light always constant, as well as their height and thickness, and like the two dimensions of the columns, which will simply be differentiated according to their position in plan: central or perimeter. The foundation is kept the same as the original, it is therefore of a superficial type and composed of isolated plinths and there will be a wall against the ground 30 cm thick at the level of the basement for which the joint constraint was used for the foundations. Even the two lift compartments composed of the four shear wall in reinforced concrete 20 cm thick are kept in the same position on the floor occupied in the original project. The plan measurements are the same as the real project, therefore a rectangular plan 30.8 m x 11.4 m for a total height of 17.85 m plus roof. The inter-plane height of the columns is kept equal to the original one.

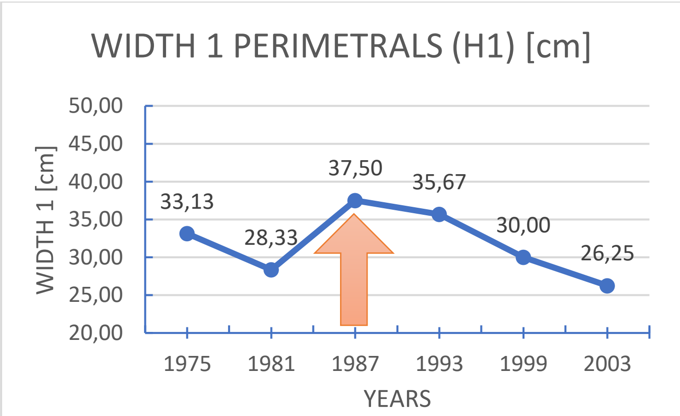
Exactly as in the real project the absence of connection frames was maintained, therefore the frames will in this case be four and only in the x direction, connected to each other by infinitely rigid masonry slabs 25 cm thick, the floors will be modelled through the "rigid floor" command, therefore not subject to deformation. The absence of connections between the various frames, therefore the lack of frames in the y direction, is usual for the buildings of the time, it was not still mandatory to link all the resistant elements, given the backwardness of the seismic laws, in fact, even in the virtual structure the same approach was maintained, confirmed by the data taken from the database.

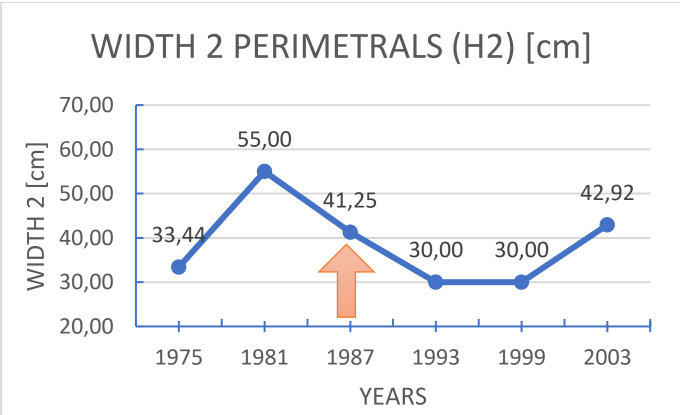


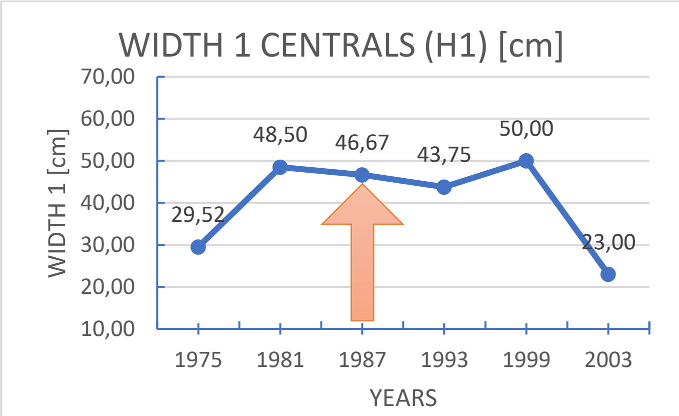


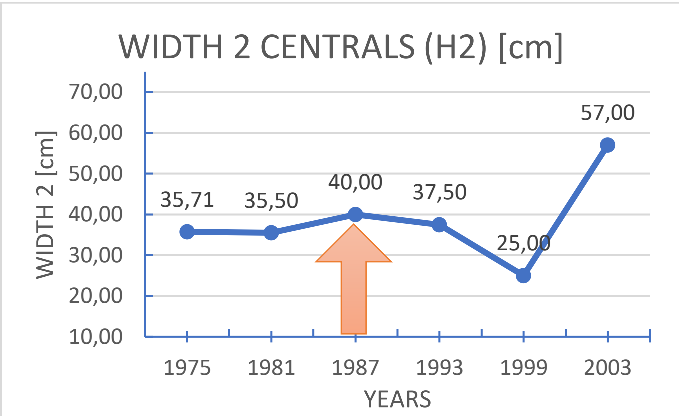


The characteristic dimensions of the two fundamental structural elements that are beams and columns are extrapolated from the database, as shown in the graphs: knowing the year of construction of the building it will be easy to obtain the respective size.









We will therefore have the following dimensions:

- span of beams = 3.60 m;
- beam thickness = 66 cm;
- beam height = 20 cm;
- perimeter column size = 36 x 40 cm;
- central column size = 46 x 40 cm;

The two central beams span in the x direction instead 2 m, since the main requirement was to respect the plan dimensions of the original project.

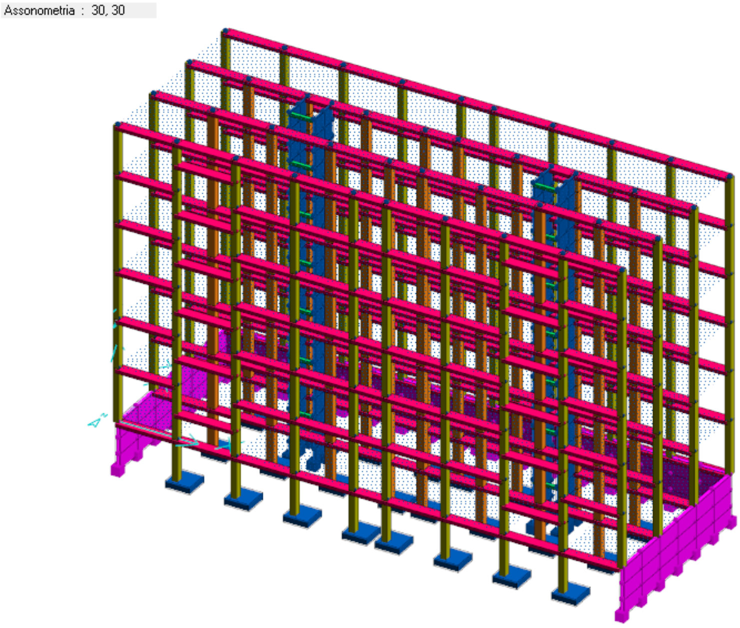


Figure 26- Virtual model on Dolmen

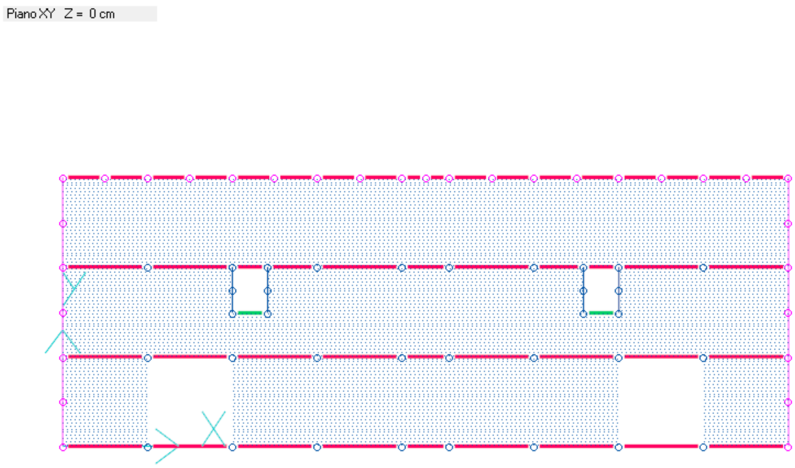


Figure 27- Plan view of the virtual model

As for the stairs and the roof the same approach was maintained as in the model of the real project, they were modelled as additional loads for the rods and floors on which they rest, including the variable loads to be attributed to the two different structural elements (snow and variable scales). The actions and loads have in fact been maintained equal to those adopted in the real model.

The resistances of the materials also underwent the same treatment they had for the real project, then taken from the bibliography graphs (Figure 24 and 25).

Another element that remains unchanged is the seismic conditions: the location, the type of soil, the structure factor and the class of use of the building (Figure 20).

As already mentioned in the previous chapter, the seismic analysis is not influenced by the quantity of reinforcement present in the section area in a decisive way, but only by the dimensions of the sections themselves and by the material composing them, for this reason it was not necessary to proceed with the definition of the reinforcements inside columns and beams, but it was sufficient to model the before mentioned elements with the correct geometric dimensions.

The results provided by the Dolmen calculation software for both the seismic analysis of modal type (linear dynamic) and the static analysis, which returns the torques not calculated from the dynamic one, will be shown below. The solicitations acting on the various elements of the structure at the bottom of the text (Annex 5) will also be available.

5.2.1 RESULTS OF THE SEISMIC DYNAMIC ANALYSIS FOR THE VIRTUAL MODEL

lavoro :\VIRTU_

```
PARAMETRI DI CALCOLO:

Modello generale
Assi di vibrazione:   X   Y
Combinazione quadratica completa (CQC)

DATI PROGETTO

Edificio  sito in località  TORINO ( long. 7.674   lat. 45.070400 )

Categoria del suolo di fondazione  = C

Coeff. di amplificazione stratigrafica Ss = 1.500

Coeff. di amplificazione topografica   ST = 1.000

S   =  1.500

Vita nominale dell'opera VN     = 50  anni

Coefficiente d'uso CU          = 1.0

Periodo di riferimento VR      = 50.0

PVR : probabilità di superamento  in VR   = 10 %

Tempo di ritorno               = 474

Coeff. di smorzamento viscoso   = 5.0

Valori risultanti per :
ag  0.563  [g/10]
Fo  2.758
TC* 0.270

Fattore di comportamento q         = 1.500

Rapporto spettro di esercizio / spettro di progetto   = 0.731
```

CONDIZIONI DI RIFERIMENTO	COEFFICIENTE	PESO RISULTANTE [daN]
1.	1.000	1123316.3
2.	1.000	1904743.0
3.	0.300	133522.2
7.	1.000	233537.9
8.	1.000	31104.0
9.	0.300	2592.0

*** TABELLA AUTOVETTORI ***

n	PERIODO	MASSA ATTIVATA			COEFFICIENTI DI CORRELAZIONE						
	[sec]	%X	%Y	%Z	n+1	n+2	n+3	n+4	n+5	n+6	n+7
1	2.369573	0.000	73.917	0.000	0.314	0.048	0.002	0.002	0.002		
2	2.044820	0.038	0.000	0.000	0.106	0.003	0.003	0.002			
3	1.533943	86.016	0.000	0.000	0.005	0.004	0.004				
4	0.463727	8.895	0.000	0.000	0.590	0.206					
5	0.426694	0.038	0.000	0.000	0.443						
6	0.381490	0.000	19.130	0.000							

```
-----------------------------------------
   MASSA TOTALE  94.987  93.046   0.000 |
-----------------------------------------
```

5.2.2 RESULTS OF THE SEISMIC STATIC ANALYSIS FOR THE VIRTUAL MODEL

```
DATI PROGETTO

Edificio  sito in località  TORINO ( long. 7.674   lat. 45.070400 )

Categoria del suolo di fondazione  = C

Coeff. di amplificazione stratigrafica Ss = 1.500

Coeff. di amplificazione topografica   ST = 1.000

S   =  1.500

Vita nominale dell'opera VN     = 50  anni

Coefficiente d'uso CU           = 1.0

Periodo di riferimento VR       = 50.0

PVR : probabilità di superamento  in VR   = 10 %

Tempo di ritorno                = 474

Coeff. di smorzamento viscoso   = 5.0

Valori risultanti per :
ag  0.563  [g/10]
Fo  2.758
TC* 0.270

Fattore di comportamento q         = 1.500

Rapporto spettro di esercizio / spettro di progetto   = 0.731

Coeff. lambda    =   1.0000
Sd               =    0.022  per T1 =    2.370

Numero condizioni generanti carichi sismici :    6

Cond. 001 : Peso_proprio________   con coeff.     1.000
Cond. 002 : Permanente__________   con coeff.     1.000
Cond. 003 : A:Var_abitazione____   con coeff.     0.300
Cond. 007 :   peso_proprio_tetto   con coeff.     1.000
Cond. 008 :     permanente_scale   con coeff.     1.000
Cond. 009 :            var_scale   con coeff.     0.300

Condizioni di carico sismico generate:

Cond. 018 :              Sisma X
Cond. 019 :              Sisma Y
Cond. 020 :      Torcente add. X
Cond. 021 :      Torcente add. Y

Carichi sismici :
```

Piani cm	Pesi daN	C. distr.	Forze piano daN	Torc. piano X daNcm	Torc. piano Y daNcm	Bar. X cm	Bar. Y cm
295.0	113031	0.0048	543	30939	83589	1542.4	596.5
405.0	7766	0.0066	51	499	4198	1540.0	706.1
464.3	17760	0.0076	134	0	15840	1540.0	0.0
515.0	7766	0.0084	65	635	5339	1540.0	706.1
625.0	504391	0.0102	5132	292503	790271	1539.3	599.6
725.0	7060	0.0118	83	812	6832	1540.0	706.1
777.1	23932	0.0126	303	11504	35722	1540.0	208.3
925.0	493961	0.0151	7438	423953	1145416	1540.1	600.3
1023.3	23666	0.0167	394	14981	46520	1540.0	207.1
1121.7	6942	0.0183	127	1236	10394	1540.0	706.1
1220.0	492678	0.0199	9784	557707	1506787	1539.7	600.1
1318.3	6942	0.0215	149	1453	12217	1540.0	706.1
1369.9	23666	0.0223	528	20055	62276	1540.0	207.1
1515.0	492747	0.0247	12152	692660	1871396	1540.1	600.0
1613.3	6942	0.0263	182	1778	14950	1540.0	706.1
1664.0	23666	0.0271	641	24360	75645	1540.0	207.1
1810.0	492747	0.0295	14518	827534	2235793	1539.9	600.0
1908.3	6942	0.0311	216	2103	17684	1540.0	706.1
1962.8	23666	0.0320	756	28734	89226	1540.0	207.1
2105.0	652541	0.0343	22360	1274508	3443408	1540.2	584.8

5.3 COMPARISON OF RESULTS

To set the comparison between the two models some boundary conditions must be respected, first of all it is necessary to ensure that the two structures are complementary and similar from the structural and geometric point of view, because if the virtual structure differs too much from the real one it is as if we were making a comparison between two totally different structures and the utility of the method would be lost; the other condition necessary to prove the attendance of what has been said so far is the need to prove that the two models have the same resistance and response to an external acceleration set identical, in practice that respond in the same way to the earthquake. Only by demonstrating that, the method will be functional.

The hypothesis number one is validated by several factors, such as:

- the dimensions in plan and in elevation are identical in the two models;
- the two structures have been modelled with the same characteristic resistance values of the materials (steel and concrete);
- the imposed loads have the same values;
- the structural simplifications for modeling are based on the same principles and are identical in the two projects (stairs, balconies, roof);
- the foundation package was modelled in the same way, as the real one found on the original drawings;
- the setting in separate frames was maintained in the virtual model, therefore the absence of connecting beams, with frames present in a single direction;
- the position of the reinforced concrete walls was kept equal to the original project;
- the floors were modelled in the same condition;
- same seismic conditions between the two models.

In practice the only elements of distinction between the two are the dimensions of the structural elements beams and columns, which change with values that were taken from the database previously created depending on the year of construction. The difference in light between the original and "virtual" beams will lead to a change in the number of frames in the two directions, which will increase by one in both cases. The dimensions of the columns will not differ from one to the other, from a plane to the one above, but will remain identical for the entire elevation of the column, while for each floor they will be differentiated only in relation to their position, perimetral or central. The same applies to the beams, which will remain with the same span, height and thickness for the entire building. It is as if a basic unit consisting of four columns, four beams and a slab were built and that this unit was repeated for the entire building volume.

Here below we can see a comparison the two plan views at the foundation level.

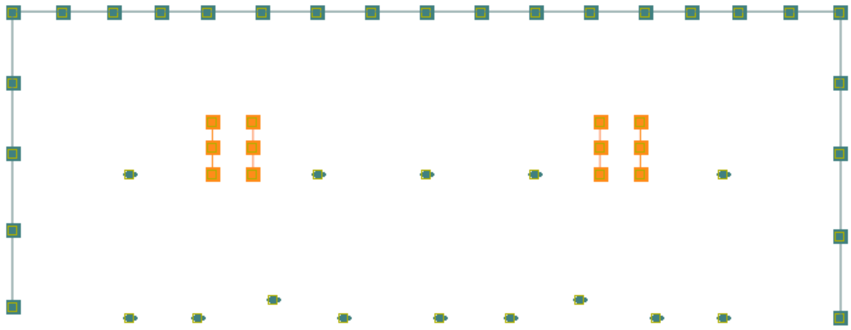


Figure 28- Foundation level of the realistic model

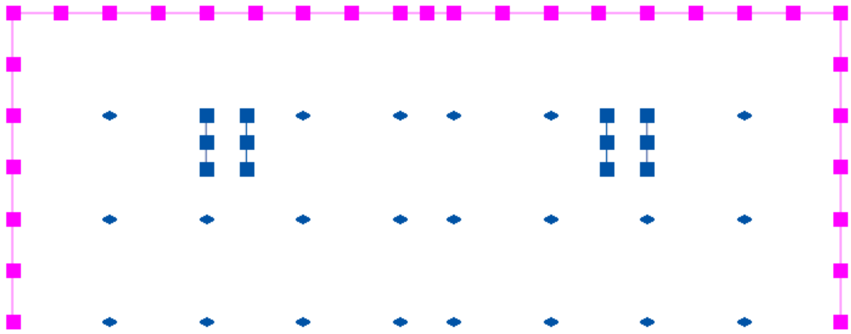


Figure 29- Foundation level of the virtual model

As regards the second of the necessary conditions, the identical response to the earthquake was tested thanks to the dynamic modal analysis carried out on the Dolmen calculation software; once the seismic conditions to be used are indicated, identical in both cases, the program calculates the classic results of the seismic analysis: the various modes of vibration associated with the proper periods of the structure, its frequencies, the masses activated at each different way of vibrating , the correlation coefficients; what interests us most, however, besides the fact that the starting conditions are identical, is the factor ζ_E or the seismic vulnerability index of a structure, the ratio between the acceleration that the structure is able to withstand and the acceleration that should withstand for a new construction project.

$$\zeta_E = \frac{maximum\ horizontal\ action\ suistanable\ from\ the\ existing\ structure}{project\ seismic\ action\ in\ case\ of\ a\ new\ construction}$$

The two coefficients of seismic vulnerability calculated for the two different structures are similar, as demonstrated by the analyses conducted and reported below. This shows that the two structures have the same resistance to the earthquake and therefore the same behaviour towards an acceleration imposed from the outside, identical in the two cases.

REAL MODEL	*VIRTUAL MODEL*
PARAMETRI DI CALCOLO:	PARAMETRI DI CALCOLO:
Modello generale Assi di vibrazione: X Y Combinazione quadratica completa (CQC)	Modello generale Assi di vibrazione: X Y Combinazione quadratica completa (CQC)
DATI PROGETTO	DATI PROGETTO
Edificio sito in località TORINO (long. 7.674 lat. 45.070400)	Edificio sito in località TORINO (long. 7.674 lat. 45.070400)
Categoria del suolo di fondazione = C	Categoria del suolo di fondazione = C
Coeff. di amplificazione stratigrafica Ss = 1.500	Coeff. di amplificazione stratigrafica Ss = 1.500
Coeff. di amplificazione topografica ST = 1.000	Coeff. di amplificazione topografica ST = 1.000
S = 1.500	S = 1.500
Vita nominale dell'opera VN = 50 anni	Vita nominale dell'opera VN = 50 anni
Coefficiente d'uso CU = 1.0	Coefficiente d'uso CU = 1.0
Periodo di riferimento VR = 50.0	Periodo di riferimento VR = 50.0
PVR : probabilità di superamento in VR = 10 %	PVR : probabilità di superamento in VR = 10 %
Tempo di ritorno = 474	Tempo di ritorno = 474
Coeff. di smorzamento viscoso = 5.0	Coeff. di smorzamento viscoso = 5.0
Valori risultanti per : ag 0.563 [g/10] Fo 2.758 TC* 0.270	Valori risultanti per : ag 0.563 [g/10] Fo 2.758 TC* 0.270
Fattore di comportamento q = 1.500	Fattore di comportamento q = 1.500
Rapporto spettro di esercizio / spettro di progetto = 0.729	Rapporto spettro di esercizio / spettro di progetto = 0.731

The fact that the two coefficients are so similar, combined with the structural similarity between the two models, is sufficient to demonstrate the reliability of the simplified procedure for the seismic vulnerability analysis.

What's now demonstrated is that, through the database, is possible to build a reference virtual model, similar but not equal to the real structure, and that the virtual structure has the same seismic response of the real one, cause the coefficient ζ_E is practically the same.

6. CONCLUSIONS

The urgency of an improvement and a seismic adjustment of the structures on the Italian territory is imposed by the age of our real estate, the primary need of Italy in the coming decades is not the construction of new buildings for residential and civil use, but the maintenance of those that are already present in the territory, both for the protection of national artistic beauties and for the safety of citizens, as well as for the eco-sustainability of the construction industry's production cycle.

The methodology proposed in this thesis offers a tool that gives the possibility to the large real estate owners to know the structural state of the properties without an excessive economic and temporal outlay, so as to be able to find those structures on which there is greater urgency of intervention without having to wait for excessive time, without having to use invasive methods for the structure itself and therefore without the obligation to spend money for the priority selection phase. It is not a method that can replace the one indicated by the regulations, but it is a procedure to be added to those already in force, a tool to increase the reliability of the results of a vulnerability analysis.

The results of the research are excellent; this thesis constitutes an excellent starting point which, with the due in-depth analysis and scientific research, could be extended, improved and made legally consistent and reliable. The first point to improve this procedure is the creation of a database that collects information not for six projects but for a greater number, in such a way it would be more complete and reliable. Another improvement for this procedure is to increase the structural construction types those could be analysed, as masonry or steel constructions.

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9. ANNEXES

ANNEX 1- BEAM'S DATABASE

YEAR	N° BEAM	DIRECTION	SPAN[cm]	B[cm]	H[cm]	SPAN/H	B/H
1975	T101	X	330	40	22	15	1.818182
1975	T102	X	315	40	22	14.31818	1.818182
1975	T103	X	310	40	22	14.09091	1.818182
1975	T104	X	350	40	22	15.90909	1.818182
1975	T105	X	340	40	22	15.45455	1.818182
1975	T106	X	315	40	22	14.31818	1.818182
1975	T107	X	285	40	22	12.95455	1.818182
1975	T108	X	305	40	22	13.86364	1.818182
1975	T109	X	315	40	22	14.31818	1.818182
1975	T110	X	405	40	22	18.40909	1.818182
1975	T111	X	330	40	22	15	1.818182
1975	T112	X	320	40	22	14.54545	1.818182
1975	T113	X	390	70	22	17.72727	3.181818
1975	T114	X	265	50	22	12.04545	2.272727
1975	T115	X	320	50	22	14.54545	2.272727
1975	T116A	X	165	50	22	7.5	2.272727
1975	T116B	X	220	50	22	10	2.272727
1975	T117	X	300	50	22	13.63636	2.272727
1975	T118	X	320	50	22	14.54545	2.272727
1975	T119	X	270	50	22	12.27273	2.272727
1975	T120	X	245	50	22	11.13636	2.272727
1975	T121	X	320	50	22	14.54545	2.272727
1975	T122	X	455	80	22	20.68182	3.636364
1975	T123	X	225	70	22	10.22727	3.181818
1975	T124	X	375	70	22	17.04545	3.181818
1975	T125	X	305	50	22	13.86364	2.272727
1975	T126	X	260	50	22	11.81818	2.272727
1975	T127	X	290	50	22	13.18182	2.272727
1975	T128	X	270	40	22	12.27273	1.818182
1975	T129	X	270	40	22	12.27273	1.818182
1975	T130	X	285	40	22	12.95455	1.818182
1975	T131	X	280	40	22	12.72727	1.818182
1975	T132		250	40	22	11.36364	1.818182
1975	T132BIS	X	285	40	22	12.95455	1.818182
1975	T133	X	240	40	22	10.90909	1.818182
1975	T134	X	270	40	22	12.27273	1.818182
1975	T135	X	265	50	22	12.04545	2.272727
1975	T136	X	295	50	22	13.40909	2.272727
1975	T137	X	410	50	22	18.63636	2.272727
1975	T138	Y	588	50	22	26.72727	2.272727
1975	T139	Y	622	50	22	28.27273	2.272727
1975	T140	Y	633	50	22	28.77273	2.272727

1975	T141	Y	577	50	22	26.22727	2.272727
1975	T142	X	425	50	22	19.31818	2.272727
1975	T143	X	420	55	22	19.09091	2.5
1975	T144	X	590	55	30	19.66667	1.833333
1975	T145	X	540	65	22	24.54545	2.954545
1975	T146	X	380	65	22	17.27273	2.954545
1975	T147	X	445	65	22	20.22727	2.954545
1975							
1975		MEAN	346.6327	48.87755	22	15.61008	2.208101

YEAR	N° BEAM	DIRECTION	SPAN[cm]	B[cm]	H[cm]	SPAN/H	B/H
1981	T101	X	435	60	22	19.77273	2.727273
1981	T102	X	485	90	22	22.04545	4.090909
1981	T103	X	278	70	22	12.63636	3.181818
1981	T104	X	195	70	22	8.863636	3.181818
1981	T105	X	375	70	22	17.04545	3.181818
1981	T106	X	350	70	22	15.90909	3.181818
1981	T107	X	257	70	22	11.68182	3.181818
1981	T108	X	260	40	22	11.81818	1.818182
1981	T109	X	272	90	22	12.36364	4.090909
1981	T110	X	395	90	22	17.95455	4.090909
1981	T111	X	285	90	22	12.95455	4.090909
1981	T112	X	175	30	22	7.954545	1.363636
1981	T113	X	215	30	22	9.772727	1.363636
1981	T114	X	365	80	22	16.59091	3.636364
1981	T115	X	325	80	22	14.77273	3.636364
1981	T116	X	222	80	22	10.09091	3.636364
1981	T117	X	372	80	22	16.90909	3.636364

1981	T118	X	335	80	22	15.22727	3.636364
1981	T119	X	295	90	22	13.40909	4.090909
1981	T120	X	390	90	22	17.72727	4.090909
1981	T121	X	272	90	22	12.36364	4.090909
1981	T122	X	475	90	22	21.59091	4.090909
1981	T123	X	312	70	22	14.18182	3.181818
1981	T124	X	272	70	22	12.36364	3.181818
1981	T125	X	452	90	22	20.54545	4.090909
1981	T126	X	440	90	22	20	4.090909
1981	T127	Y	505	60	22	22.95455	2.727273
1981	T128	Y	207	60	22	9.409091	2.727273
1981	T129	Y	470	60	22	21.36364	2.727273
1981	T130	Y	542	50	22	24.63636	2.272727
1981	T132	Y	545	50	22	24.77273	2.272727
1981							
1981		MEAN	347.5161	71.93548	22	15.79619	3.269795

YEAR	N° BEAM	DIRECTION	SPAN[cm]	B[cm]	H[cm]	SPAN/H	B/H
1987	TR201	X	357	70	20	17.85	3.5
1987	TR202	X	275	70	20	13.75	3.5
1987	TR203	15/30	460	70	20	23	3.5
1987	TR204	X	345	70	20	17.25	3.5
1987	TR205	X	400	70	20	20	3.5
1987	TR206	X	394	60	20	19.7	3
1987	TR207	X	435	70	20	21.75	3.5
1987	TR208	X	230	70	20	11.5	3.5
1987	TR209	Y	455	70	20	22.75	3.5
1987	TR210	Y	415	70	20	20.75	3.5
1987	TR211	45	275	70	20	13.75	3.5
1987	TR212	Y	140	60	20	7	3

1987	TR213	Y	422	60	20	21.1	3
1987	TR214	Y	480	50	20	24	2.5
1987							
1987		MEAN	363.071 4	66.4285 7	20	18.1535 7	3.32142 9

YEAR	N° BEAM	DIRECTIO N	SPAN[c m]	B[cm]	H[cm]	SPAN/H	B/H
1993	T1	X	335	50	24	13.9583 3	2.08333 3
1993	T2	X	275	50	24	11.4583 3	2.08333 3
1993	T3	X	350	90	24	14.5833 3	3.75
1993	T4	X	515	90	24	21.4583 3	3.75
1993	T5	X	525	90	24	21.875	3.75
1993	T6	X	350	90	24	14.5833 3	3.75
1993	T7	X	335	50	24	13.9583 3	2.08333 3
1993	T8	X	275	50	24	11.4583 3	2.08333 3
1993	T9	X	189	30	24	7.875	1.25
1993	T10	X	405	80	24	16.875	3.33333 3
1993	T11	X	410	80	24	17.0833 3	3.33333 3
1993	T12	X	450	70	24	18.75	2.91666 7
1993	T13	X	350	70	24	14.5833 3	2.91666 7
1993	T14	X	210	30	24	8.75	1.25
1993	T15	X	335	50	24	13.9583 3	2.08333 3
1993	T16	X	185	50	24	7.70833 3	2.08333 3
1993	T17	X	290	40	24	12.0833 3	1.66666 7
1993	T18	X	415	70	24	17.2916 7	2.91666 7
1993	T19	X	360	70	24	15	2.91666 7
1993	T20	X	335	50	24	13.9583 3	2.08333 3

1993	T21	X	185	50	24	7.708333	2.083333
1993	T22	X	335	50	24	13.95833	2.083333
1993	T23	X	195	50	24	8.125	2.083333
1993	T24	X	440	70	24	18.33333	2.916667
1993	T25	X	345	70	24	14.375	2.916667
1993	T26	X	420	60	24	17.5	2.5
1993	T27	X	365	60	24	15.20833	2.5
1993	T28	Y	269	50	24	11.20833	2.083333
1993	T29	Y	274	50	24	11.41667	2.083333
1993	T30	Y	274	50	24	11.41667	2.083333
1993	T31	Y	239	50	24	9.958333	2.083333
1993	T32	Y	280	50	24	11.66667	2.083333
1993	T33	Y	381	50	24	15.875	2.083333
1993	T34	Y	432	60	24	18	2.5
1993	T35	Y	357	40	24	14.875	1.666667
1993	T36	45°	210	40	24	8.75	1.666667
1993	T37	Y	345	50	24	14.375	2.083333
1993	M37	Y	225	50	24	9.375	2.083333
1993	T38	Y	230	25	69	3.333333	0.362319
1993	T39	Y	325	25	69	4.710145	0.362319
1993	T40	Y	350	60	24	14.58333	2.5
1993	T41	Y	274	60	24	11.41667	2.5
1993	T42	Y	450	60	24	18.75	2.5
1993	M42	Y	225	60	24	9.375	2.5
1993							
1993		MEAN	325.4318	56.59091	24	13.2169	2.327075

YEAR	N° BEAM	DIRECTION	SPAN[cm]	B[cm]	H[cm]	SPAN/H	B/H
1999	T301A	Y	305	70	22	13.86364	3.181818
1999	T301B		425	70	22	19.31818	3.181818
1999	T301C		218	70	22	9.909091	3.181818
1999	T301D		423	70	22	19.22727	3.181818
1999	T326		615	80	22	27.95455	3.636364
1999	T320		300	50	22	13.63636	2.272727
1999	T328		582	80	22	26.45455	3.636364
1999	T308		200	50	22	9.090909	2.272727
1999	T327		520	70	22	23.63636	3.181818
1999	T329		567	70	22	25.77273	3.181818
1999	T305A	X	436	70	22	19.81818	3.181818
1999	T305B		288	70	22	13.09091	3.181818
1999	T305C		288	70	22	13.09091	3.181818
1999	T315		580	50	22	26.36364	2.272727
1999	T323		580	50	22	26.36364	2.272727
1999	T322A		436	50	22	19.81818	2.272727
1999	T322B		288	70	22	13.09091	3.181818
1999	T322C		288	70	22	13.09091	3.181818
1999	T325	45°	150	50	22	6.818182	2.272727
1999							
1999		MEAN	394.1579	64.73684	22	17.91627	2.942584

YEAR	N° BEAM	DIRECTION	SPAN[cm]	B[cm]	H[cm]	SPAN/H	B/H
2003	T54	X	422	70	25	16.88	2.8
2003	T55	X	205	70	25	8.2	2.8
2003	T56	X	422	70	25	16.88	2.8
2003	T57	X	357	85	25	14.28	3.4
2003	T58	X	392	85	25	15.68	3.4
2003	T59	X	142	85	25	5.68	3.4
2003	T60	X	333	70	25	13.32	2.8
2003	T61	X	270	70	25	10.8	2.8
2003	T62	X	327	40	25	13.08	1.6
2003	T63	X	177	40	25	7.08	1.6
2003	T64	X	272	40	22	12.36364	1.818182
2003	T65	X	173	40	22	7.863636	1.818182
2003	T66	X	450	35	22	20.45455	1.590909
2003	T69	Y	274	35	25	10.96	1.4
2003	T70	Y	128	35	25	5.12	1.4
2003	T71	Y	279	35	25	11.16	1.4
2003	T167	Y	150	35	22	6.818182	1.590909
2003	T72	Y	198	35	25	7.92	1.4
2003	T73	Y	299	35	25	11.96	1.4
2003	T74	Y	190	35	25	7.6	1.4
2003							
2003							
2003	COR1	Y	350	35	25	14	1.4
2003	COR2	Y	357	35	25	14.28	1.4
2003	COR3	Y	400	35	25	16	1.4
2003	COR4	Y	350	35	25	14	1.4
2003	COR5	Y	357	35	25	14.28	1.4
2003	COR6	Y	372	35	25	14.88	1.4
2003	COR7	Y	290	35	25	11.6	1.4
2003	COR8	X	616	20	22	28	0.909091
2003	COR9	X	160	35	25	6.4	1.4
2003	COR10	X	180	20	25	7.2	0.8
2003							
2003		MEAN	296.4	45.5	23	12.158	1.850909

YEAR	TOTAL REINF. SUPPORT 1 [cmq]	TOTAL REINF. SPAN [cmq]	TOTAL REINF. SUPPORT 2 [cmq]
1975	6.62	5.49	6.62
1975	4.4	4.4	5.94
1975	2.58	2.58	2.58
1975	2.58	2.58	2.58
1975	2.58	2.58	2.58
1975	2.58	2.58	2.58
1975	2.58	2.58	2.58
1975	2.58	2.58	2.58
1975	2.58	2.58	2.58
1975	2.58	4.84	2.58
1975	7.34	5.49	5.08
1975	6.28	6.84	6.62
1975	10.93	6.44	11.4
1975	5.22	5.22	6.01
1975	5.9	6.37	5.9
1975	6.78	3.23	4.77
1975	4.77	3.23	6.31
1975	5.49	5.49	5.49
1975	6.62	5.49	6.62
1975	7.03	5.9	7.44
1975	4.06	4.06	4.06
1975	5.9	5.9	11.93
1975	12.62	12.62	12.62
1975	11.18	3.14	3.14
1975	16.21	9.67	11.21
1975	7.03	5.9	5.9
1975	4.4	5.53	5.53
1975	5.9	5.9	7.03
1975	4.51	3.72	4.51
1975	6.62	5.49	6.62
1975	4.06	4.06	4.06
1975	7.03	5.9	7.03
1975	16.64	14.63	20.66
1975	3.72	3.72	3.72
1975	5.49	5.49	6.62
1975	4.51	3.72	4.51
1975	4.51	3.72	4.85
1975	5.49	5.49	5.49
1975	13.69	10.61	12.15
1975	7.44	6.31	6.31
1975	9.2	7.19	8.73
1975	8.73	7.19	9.2
1975	6.31	7.44	7.44
1975	9.2	7.66	7.66

1975	6.72	6.72	6.72
1975	13.22	10.14	11.68
1975	10.61	8.6	8.6
1975	5.08	5.08	5.08
1975	8.6	8.6	10.61

YEAR	TOTAL REINF. SUPPORT 1 [cmq]	TOTAL REINF. SPAN [cmq]	TOTAL REINF. SUPPORT 2 [cmq]
1981	15.07	11.05	27.13
1981	29.14	19.09	23.11
1981	10.96	7.88	12.5
1981	11.82	7.2	13.36
1981	14.86	10.24	17.94
1981	17.94	8.7	14.86
1981	14.04	6.34	9.42
1981	7.78	5.52	7.78
1981	10.96	7.88	19.94
1981	21.7	13.06	21.7
1981	19.94	7.88	10.96
1981	6.65	6.65	6.65
1981	6.65	6.65	6.65
1981	21.1	17.08	27.26
1981	24.31	10.24	17.34
1981	16.52	8.1	18.4
1981	20.16	13.06	31.15
1981	33.16	15.07	19.09
1981	10.96	7.88	19.94
1981	23.71	15.07	23.71
1981	19.94	7.88	10.96
1981	21.1	17.08	27.26
1981	23.49	9.42	17.12
1981	17.12	7.88	10.96
1981	21.1	17.08	29.14
1981	27.13	13.06	17.08
1981	8.91	6.65	14.69
1981	16.45	13.06	21.1
1981	21.1	13.06	17.08
1981	13.32	10.24	13.32
1981	15.07	11.05	15.07

YEAR	TOTAL REINF. SUPPORT 1 [cmq]	TOTAL REINF. SPAN [cmq]	TOTAL REINF. SUPPORT 2 [cmq]
1987	16.11	16.11	18.37
1987	17.99	10.17	12.43
1987	20.01	20.01	24.53

1987	20.35	10.17	15.26
1987	17.43	12.91	17.43
1987	11.37	11.37	11.37
1987	31	31	33.26
1987	30.87	30.87	30.87
1987	13.38	13.38	17.9
1987	15.73	7.91	7.91
1987	10.17	7.91	7.91
1987	13	13	18.15
1987	16.52	11.43	11.43
1987	17.06	17.06	17.06

YEAR	TOTAL REINF. SUPPORT 1 [cmq]	TOTAL REINF. SPAN [cmq]	TOTAL REINF. SUPPORT 2 [cmq]
1993	8.91	5.52	10.49
1993	9.13	4.74	13.66
1993	15.02	11.07	23.65
1993	27.17	17.23	34.46
1993	34.46	17.23	27.27
1993	26.01	8.78	13.3
1993	7.89	5.52	9.91
1993	8.55	4.16	6.53
1993	4.74	4.74	4.74
1993	15.34	10.82	24.46
1993	19.84	12.23	15.62
1993	13.08	10.82	18.56
1993	19.59	7.23	11.75
1993	4.74	4.74	4.74
1993	7.89	5.52	8.89
1993	7.53	6.53	6.53
1993	8.47	6.1	8.47
1993	9.94	8.36	13.33
1993	20.68	12.32	14.69
1993	7.89	5.52	9.91
1993	8.55	7.55	7.55
1993	7.89	22.52	11.14
1993	9.78	8.78	13.75
1993	19.83	10.82	21.64
1993	18.05	7.23	13.26
1993	9.94	8.36	12.2
1993	12.76	5.53	7.9
1993	5.74	4.16	5.74
1993	5.74	4.16	5.74
1993	5.74	4.16	5.74
1993	5.74	4.16	5.74
1993	5.74	4.16	8.55

1993	9.91	5.52	7.1
1993	13.08	10.82	13.08
1993	7.89	5.52	9.91
1993	8.55	7.55	5.74
1993	6.53	12.2	12.2
1993	12.2	12.2	12.2
1993	11	11	12.58
1993	19.24	17.66	16.25
1993	6.53	4.16	7.53
1993	7.53	4.16	8.32
1993	7.42	4.84	12.88
1993	12.88	12.88	11.3

YEAR	TOTAL REINF. SUPPORT 1 [cmq]	TOTAL REINF. SPAN [cmq]	TOTAL REINF. SUPPORT 2 [cmq]
1999	15.4	22.02	22.02
1999	22.02	15.4	15.4
1999	15.4	15.4	20.48
1999	20.48	15.4	17.94
1999	20.48	13.86	26.56
1999	12.32	12.32	12.32
1999	17.94	15.4	23.62
1999	12.32	12.32	12.32
1999	21.56	21.56	21.56
1999	16.94	15.4	20.48
1999	12.32	12.32	14.86
1999	16.4	15.4	15.4
1999	15.4	15.4	15.4
1999	12.32	12.32	12.32
1999	13.86	13.86	13.86
1999	12.32	12.32	14.86
1999	17.94	15.4	15.4
1999	15.4	15.4	15.4
1999	12.32	13.86	13.86

YEAR	TOTAL REINF. SUPPORT 1 [cmq]	TOTAL REINF. SPAN [cmq]	TOTAL REINF. SUPPORT 2 [cmq]
2003	13.47	11.93	14.54
2003	14.54	9.14	14.54
2003	14.54	11.93	15.01
2003	13.86	13.86	24.41
2003	22.87	13.16	16.24
2003	11.93	7.91	7.91
2003	12.32	13.86	17.41
2003	17.41	10.78	11.91

2003	5.65	5.65	5.65
2003	4.52	4.52	4.52
2003	6.88	6.88	9.14
2003	7.91	5.65	5.65
2003	4.52	5.65	4.52
2003	8.29	8.29	10.55
2003	6.78	4.52	4.52
2003	4.52	4.52	13.16
2003	13.16	13.16	13.16
2003	4.52	4.52	6.78
2003	6.78	4.52	10.55
2003	10.55	8.29	8.29
2003			
2003			
2003	4.52	4.52	4.52
2003	4.52	4.52	4.52
2003	4.52	4.52	4.52
2003	4.52	4.52	4.52
2003	4.52	4.52	4.52
2003	4.52	4.52	4.52
2003	4.52	4.52	4.52
2003	4.52	4.52	4.52
2003	5.65	5.65	5.65
2003	5.65	5.65	5.65

YEAR	% REINFORCEMENT ON SUPPORT 1	% REINFORCEMENT IN MIDDLE-SPAN	% REINFORCEMENT ON SUPPORT 2
1975	0.752272727	0.623863636	0.752272727
1975	0.5	0.5	0.675000000
1975	0.293181818	0.293181818	0.293181818
1975	0.293181818	0.293181818	0.293181818
1975	0.293181818	0.293181818	0.293181818
1975	0.293181818	0.293181818	0.293181818
1975	0.293181818	0.293181818	0.293181818
1975	0.293181818	0.293181818	0.293181818
1975	0.293181818	0.293181818	0.293181818
1975	0.293181818	0.55	0.293181818
1975	0.834090909	0.623863636	0.577272727
1975	0.713636364	0.777272727	0.752272727
1975	0.70974026	0.418181818	0.740259740
1975	0.474545455	0.474545455	0.546363636
1975	0.536363636	0.579090909	0.536363636
1975	0.616363636	0.293636364	0.433636364
1975	0.433636364	0.293636364	0.573636364

1975	0.499090909	0.499090909	0.499090909
1975	0.601818182	0.499090909	0.601818182
1975	0.639090909	0.536363636	0.676363636
1975	0.369090909	0.369090909	0.369090909
1975	0.536363636	0.536363636	1.084545455
1975	0.717045455	0.717045455	0.717045455
1975	0.725974026	0.203896104	0.203896104
1975	1.052597403	0.627922078	0.727922078
1975	0.639090909	0.536363636	0.536363636
1975	0.4	0.502727273	0.502727273
1975	0.536363636	0.536363636	0.639090909
1975	0.5125	0.422727273	0.512500000
1975	0.752272727	0.623863636	0.752272727
1975	0.461363636	0.461363636	0.461363636
1975	0.798863636	0.670454545	0.798863636
1975	1.890909091	1.6625	2.347727273
1975	0.422727273	0.422727273	0.422727273
1975	0.623863636	0.623863636	0.752272727
1975	0.5125	0.422727273	0.512500000
1975	0.41	0.338181818	0.440909091
1975	0.499090909	0.499090909	0.499090909
1975	1.244545455	0.964545455	1.104545455
1975	0.676363636	0.573636364	0.573636364
1975	0.836363636	0.653636364	0.793636364
1975	0.793636364	0.653636364	0.836363636
1975	0.573636364	0.676363636	0.676363636
1975	0.836363636	0.696363636	0.696363636
1975	0.555371901	0.555371901	0.555371901
1975	0.801212121	0.614545455	0.707878788
1975	0.741958042	0.601398601	0.601398601
1975	0.355244755	0.355244755	0.355244755
1975	0.601398601	0.601398601	0.741958042
1975			
MEAN	0.602710516	0.527435162	0.604764805

YEAR	% REINFORCEMENT ON SUPPORT 1	% REINFORCEMENT IN MIDDLE-SPAN	% REINFORCEMENT ON SUPPORT 2
1981	1.141666667	0.837121212	2.05530303
1981	1.471717172	0.964141414	1.167171717
1981	0.711688312	0.511688312	0.811688312
1981	0.767532468	0.467532468	0.867532468
1981	0.964935065	0.664935065	1.164935065
1981	1.164935065	0.564935065	0.964935065
1981	0.911688312	0.411688312	0.611688312
1981	0.884090909	0.627272727	0.884090909
1981	0.553535354	0.397979798	1.007070707

1981	1.095959596	0.65959596	1.095959596
1981	1.007070707	0.397979798	0.553535354
1981	1.007575758	1.007575758	1.007575758
1981	1.007575758	1.007575758	1.007575758
1981	1.198863636	0.970454545	1.548863636
1981	1.38125	0.581818182	0.985227273
1981	0.938636364	0.460227273	1.045454545
1981	1.145454545	0.742045455	1.769886364
1981	1.884090909	0.85625	1.084659091
1981	0.553535354	0.397979798	1.007070707
1981	1.197474747	0.761111111	1.197474747
1981	1.007070707	0.397979798	0.553535354
1981	1.065656566	0.862626263	1.376767677
1981	1.525324675	0.611688312	1.111688312
1981	1.111688312	0.511688312	0.711688312
1981	1.065656566	0.862626263	1.471717172
1981	1.37020202	0.65959596	0.862626263
1981	0.675	0.503787879	1.112878788
1981	1.246212121	0.989393939	1.598484848
1981	1.598484848	0.989393939	1.293939394
1981	1.210909091	0.930909091	1.210909091
1981	1.37	1.004545455	1.37
1981			
MEAN	1.104370374	0.697230426	1.113288181

YEAR	% REINFORCEMENT ON SUPPORT 1	% REINFORCEMENT IN MIDDLE-SPAN	% REINFORCEMENT ON SUPPORT 2
1987	1.150714286	1.150714286	1.312142857
1987	1.285	0.726428571	0.887857143
1987	1.429285714	1.429285714	1.752142857
1987	1.453571429	0.726428571	1.09
1987	1.245	0.922142857	1.245
1987	0.9475	0.9475	0.9475
1987	2.214285714	2.214285714	2.375714286
1987	2.205	2.205	2.205
1987	0.955714286	0.955714286	1.278571429
1987	1.123571429	0.565	0.565
1987	0.726428571	0.565	0.565
1987	1.083333333	1.083333333	1.5125
1987	1.376666667	0.9525	0.9525
1987	1.706	1.706	1.706
1987			
MEAN	1.350147959	1.15352381	1.313923469

YEAR	% REINFORCEMENT ON SUPPORT 1	% REINFORCEMENT IN MIDDLE-SPAN	% REINFORCEMENT ON SUPPORT 2
1993	0.7425	0.46	0.874166667
1993	0.760833333	0.395	1.138333333
1993	0.69537037	0.5125	1.094907407
1993	1.25787037	0.797685185	1.59537037
1993	1.59537037	0.797685185	1.2625
1993	1.204166667	0.406481481	0.615740741
1993	0.6575	0.46	0.825833333
1993	0.7125	0.346666667	0.544166667
1993	0.658333333	0.658333333	0.658333333
1993	0.798958333	0.563541667	1.273958333
1993	1.033333333	0.636979167	0.813541667
1993	0.778571429	0.644047619	1.104761905
1993	1.166071429	0.430357143	0.699404762
1993	0.658333333	0.658333333	0.658333333
1993	0.6575	0.46	0.740833333
1993	0.6275	0.544166667	0.544166667
1993	0.882291667	0.635416667	0.882291667
1993	0.591666667	0.497619048	0.793452381
1993	1.230952381	0.733333333	0.874404762
1993	0.6575	0.46	0.825833333
1993	0.7125	0.629166667	0.629166667
1993	0.6575	1.876666667	0.928333333
1993	0.815	0.731666667	1.145833333
1993	1.180357143	0.644047619	1.288095238
1993	1.074404762	0.430357143	0.789285714
1993	0.690277778	0.580555556	0.847222222
1993	0.886111111	0.384027778	0.548611111
1993	0.478333333	0.346666667	0.478333333
1993	0.478333333	0.346666667	0.478333333
1993	0.478333333	0.346666667	0.478333333
1993	0.478333333	0.346666667	0.478333333
1993	0.478333333	0.346666667	0.7125
1993	0.825833333	0.46	0.591666667
1993	0.908333333	0.751388889	0.908333333
1993	0.821875	0.575	1.032291667
1993	0.890625	0.786458333	0.597916667
1993	0.544166667	1.016666667	1.016666667
1993	1.016666667	1.016666667	1.016666667
1993	0.637681159	0.637681159	0.729275362
1993	1.115362319	1.023768116	0.942028986
1993	0.453472222	0.288888889	0.522916667
1993	0.522916667	0.288888889	0.577777778
1993	0.515277778	0.336111111	0.894444444
1993	0.894444444	0.894444444	0.784722222

1993			
MEAN	0.793672615	0.595089343	0.823577774

YEAR	% REINFORCEMENT ON SUPPORT 1	% REINFORCEMENT IN MIDDLE-SPAN	% REINFORCEMENT ON SUPPORT 2
1999	1	1.42987013	1.42987013
1999	1.42987013	1	1
1999	1	1	1.32987013
1999	1.32987013	1	1.164935065
1999	1.163636364	0.7875	1.509090909
1999	1.12	1.12	1.12
1999	1.019318182	0.875	1.342045455
1999	1.12	1.12	1.12
1999	1.4	1.4	1.4
1999	1.1	1	1.32987013
1999	0.8	0.8	0.964935065
1999	1.064935065	1	1
1999	1	1	1
1999	1.12	1.12	1.12
1999	1.26	1.26	1.26
1999	1.12	1.12	1.350909091
1999	1.164935065	1	1
1999	1	1	1
1999	1.12	1.26	1.26
1999			
MEAN	1.122766576	1.068019481	1.194817157

YEAR	% REINFORCEMENT ON SUPPORT 1	% REINFORCEMENT IN MIDDLE-SPAN	% REINFORCEMENT ON SUPPORT 2
2003	0.769714286	0.681714286	0.830857143
2003	0.830857143	0.522285714	0.830857143
2003	0.830857143	0.681714286	0.857714286
2003	0.652235294	0.652235294	1.148705882
2003	1.076235294	0.619294118	0.764235294
2003	0.561411765	0.372235294	0.372235294
2003	0.704	0.792	0.994857143
2003	0.994857143	0.616	0.680571429
2003	0.565	0.565	0.565
2003	0.452	0.452	0.452
2003	0.781818182	0.781818182	1.038636364
2003	0.898863636	0.642045455	0.642045455
2003	0.587012987	0.733766234	0.587012987
2003	0.947428571	0.947428571	1.205714286
2003	0.774857143	0.516571429	0.516571429
2003	0.516571429	0.516571429	1.504

2003	1.709090909	1.709090909	1.709090909
2003	0.516571429	0.516571429	0.774857143
2003	0.774857143	0.516571429	1.205714286
2003	1.205714286	0.947428571	0.947428571
2003			
2003			
2003	0.516571429	0.516571429	0.516571429
2003	0.516571429	0.516571429	0.516571429
2003	0.516571429	0.516571429	0.516571429
2003	0.516571429	0.516571429	0.516571429
2003	0.516571429	0.516571429	0.516571429
2003	0.516571429	0.516571429	0.516571429
2003	0.516571429	0.516571429	0.516571429
2003	1.027272727	1.027272727	1.027272727
2003	0.645714286	0.645714286	0.645714286
2003	1.13	1.13	1.13
2003			
MEAN	0.752298026	0.673377655	0.801569735

ANNEX 2- COLUMN'S DATABASE

YEAR	N°COLUMN	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]
1975	PERIMETRALS			
1975	2	330	40	30
1975	3	330	40	30
1975	4	330	40	30
1975	5	330	40	30
1975	6	330	40	30
1975	7	330	40	30
1975	9	330	40	30
1975	10	330	40	30
1975	11	330	40	30
1975	12	330	40	30
1975	13	330	40	30
1975	23	330	30	40
1975	38	330	30	40
1975	44	330	30	30
1975	45	330	30	30
1975	48	330	30	30
1975	49	330	30	30
1975	50	330	30	30
1975	51	330	30	30
1975	52	330	30	30
1975	53	330	30	30

1975	56	330	30	30
1975	57	330	30	30
1975	46	330	20	40
1975	47	330	20	40
1975	54	330	20	40
1975	55	330	20	40
1975	1	330	40	40
1975	8	330	40	40
1975	14	330	40	40
1975	43	330	30	40
1975	58	330	30	40
1975				
1975	MEAN	330	33.125	33.4375
1975				
1975	CENTRALS			
1975	15	330	30	30
1975	16	330	30	30
1975	17	330	30	30
1975	18	330	30	30
1975	20	330	30	30
1975	21	330	30	30
1975	22	330	30	30
1975	24	330	35	35
1975	25	330	35	35
1975	28	330	35	35
1975	29	330	35	35
1975	30	330	35	35
1975	31	330	35	35
1975	32	330	35	35
1975	37	330	35	35
1975	33	330	25	50
1975	36	330	25	50
1975	39	330	20	40
1975	40	330	20	40
1975	41	330	20	40
1975	42	330	20	40
1975				
1975	MEAN	330	29.52381	35.71429

YEAR	N°COLUMN	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]
1981	PERIMETRALS			
1981	F	300	40	25
1981	M	300	25	70
1981	N	300	20	70
1981				

1981	MEAN	300	28.33333	55
1981				
1981	CENTRALS			
1981	C	300	70	25
1981	D	300	70	25
1981	E	300	70	25
1981	G	300	25	70
1981	H	300	40	25
1981	L 1	300	40	25
1981	L 2	300	25	40
1981	P	300	25	70
1981	Q	300	50	25
1981	R	300	70	25
1981				
1981	MEAN	300	48.5	35.5

YEAR	N°COLUMN	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]
1987	PERIMETRALS			
1987	1	298	30	40
1987	2	298	30	50
1987	7	298	30	50
1987	3	298	50	40
1987	6	298	50	40
1987	8	298	30	40
1987	9	298	30	50
1987	14	298	30	50
1987	19	298	40	30
1987	20	298	40	30
1987	23	298	40	30
1987	24	298	40	30
1987	16	298	50	40
1987	17	298	50	40
1987	21	298	30	50
1987	22	298	30	50
1987				
1987	MEAN	298	37.5	41.25
1987				
1987	CENTRALS			
1987	13	298	50	40
1987	10	298	50	40
1987	11	298	50	40
1987	12	298	50	40
1987	15	298	40	40
1987	18	298	40	40
1987				

1987	MEAN	298	46.66667	40

YEAR	N°COLUMN	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]
1993	PERIMETRALS			
1993	1	300	40	25
1993	2	300	40	30
1993	3	300	50	30
1993	4	300	40	30
1993	5	300	40	25
1993	11	300	25	40
1993	12	300	20	40
1993	14	300	20	40
1993	15	300	20	40
1993	16	300	40	25
1993	17	300	40	25
1993	18	300	40	25
1993	19	300	40	25
1993	20	300	40	25
1993	21	300	40	25
1993				
1993	MEAN	300	35.66667	30
1993				
1993	CENTRALS			
1993	6	300	95	20
1993	7	300	30	50
1993	10	300	30	40
1993	13	300	20	40
1993				
1993	MEAN	300	43.75	37.5

YEAR	N°COLUMN	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]
1999	PERIMETRALS			
1999	1	300	30	30
1999	2	300	30	30
1999	3	300	30	30
1999	4	300	30	30
1999	10	300	30	30
1999	12	300	30	30
1999	13	300	30	30
1999				
1999	MEAN	300	30	30
1999				
1999	CENTRALS			

1999	5	300	60	25
1999	6	300	60	25
1999	8	300	40	25
1999	9	300	40	25
1999				
1999	MEAN	300	50	25

YEAR	N°COLUMN	INTERSTOREY HEIGHT[cm]	X1[cm]	X2[cm]
2003	PERIMETRALS			
2003	P1	315	20	60
2003	P3	315	20	60
2003	P6	315	20	60
2003	P8	315	20	60
2003	P4	315	20	45
2003	P5	315	20	45
2003	P9	315	35	30
2003	P10	315	35	30
2003	P13	315	20	35
2003	P16	315	35	30
2003	P25	315	35	30
2003	P26	315	35	30
2003				
2003	MEAN	315	26.25	42.91667
2003				
2003	CENTRALS			
2003	P7	315	25	45
2003	P11	315	15	80
2003	P12	315	15	80
2003	P14	315	15	80
2003	P15	315	15	80
2003				
2003	MEAN	315	23	57

YEAR	CROSS SECTION [cmq]	LONGITUDINAL REINFORCEMENT [cmq]	% LONGITUDINAL REINFORCEMENT
1975			
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667

1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	800	4.52	0.565
1975	800	4.52	0.565
1975	800	4.52	0.565
1975	800	4.52	0.565
1975	1600	4.52	0.2825
1975	1600	4.52	0.2825
1975	1600	4.52	0.2825
1975	1200	4.52	0.376666667
1975	1200	4.52	0.376666667
1975			
1975		MEAN	0.430616319
1975			
1975			
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	900	4.52	0.502222222
1975	1225	6.16	0.502857143
1975	1225	6.16	0.502857143
1975	1225	6.16	0.502857143
1975	1225	6.16	0.502857143
1975	1225	6.16	0.502857143
1975	1225	6.16	0.502857143
1975	1225	6.16	0.502857143
1975	1225	6.16	0.502857143
1975	1250	6.16	0.4928
1975	1250	6.16	0.4928
1975	800	4.52	0.565

1975	800	4.52	0.565
1975	800	4.52	0.565
1975	800	4.52	0.565
1975			
1975		MEAN	0.513524414

YEAR	CROSS SECTION [cmq]	LONGITUDINAL REINFORCEMENT [cmq]	% LONGITUDINAL REINFORCEMENT
1981			
1981	1000	4.52	0.452
1981	1750	10.68	0.610285714
1981	1400	10.68	0.762857143
1981			
1981		MEAN	0.608380952
1981			
1981			
1981	1750	10.68	0.610285714
1981	1750	12.22	0.698285714
1981	1750	13.76	0.786285714
1981	1750	9.04	0.516571429
1981	1000	8.42	0.842
1981	1000	6.78	0.678
1981	1000	6.78	0.678
1981	1750	10.69	0.610857143
1981	1250	6.78	0.5424
1981	1750	10.68	0.610285714
1981			
1981		MEAN	0.657297143

YEAR	CROSS SECTION [cmq]	LONGITUDINAL REINFORCEMENT [cmq]	% LONGITUDINAL REINFORCEMENT
1987			
1987	1200	11.12	0.926666667
1987	1500	12.06	0.804
1987	1500	12.06	0.804
1987	2000	16.08	0.804
1987	2000	16.08	0.804
1987	1200	11.12	0.926666667
1987	1500	12.06	0.804
1987	1500	12.06	0.804
1987	1200	9.24	0.77
1987	1200	9.24	0.77
1987	1200	9.24	0.77
1987	1200	9.24	0.77
1987	2000	16.08	0.804

1987	2000	16.08	0.804
1987	1500	9.24	0.616
1987	1500	9.24	0.616
1987			
1987		MEAN	0.787333333
1987			
1987			
1987	2000	16.08	0.804
1987	2000	16.08	0.804
1987	2000	16.08	0.804
1987	2000	16.08	0.804
1987	1600	12.32	0.77
1987	1600	12.32	0.77
1987			
1987		MEAN	0.792666667

YEAR	CROSS SECTION [cmq]	LONGITUDINAL REINFORCEMENT [cmq]	% LONGITUDINAL REINFORCEMENT
1993			
1993	1000	9.24	0.924
1993	1200	9.24	0.77
1993	1500	9.24	0.616
1993	1200	9.24	0.77
1993	1000	9.24	0.924
1993	1000	9.24	0.924
1993	800	9.24	1.155
1993	800	6.78	0.8475
1993	800	9.24	1.155
1993	1000	9.24	0.924
1993	1000	9.24	0.924
1993	1000	9.24	0.924
1993	1000	9.24	0.924
1993	1000	9.24	0.924
1993	1000	9.24	0.924
1993			
1993		MEAN	0.908633333
1993			
1993			
1993	1900	9.24	0.486315789
1993	1500	9.24	0.616
1993	1200	9.24	0.77
1993	800	9.24	1.155
1993			
1993		MEAN	0.756828947

YEAR	CROSS SECTION [cmq]	LONGITUDINAL REINFORCEMENT [cmq]	% LONGITUDINAL REINFORCEMENT
1999			
1999	900	15.24	1.693333333
1999	900	15.24	1.693333333
1999	900	15.24	1.693333333
1999	900	15.24	1.693333333
1999	900	15.24	1.693333333
1999	900	15.24	1.693333333
1999	900	20.32	2.257777778
1999			
1999		MEAN	1.37
1999			
1999			
1999	1500	9.24	0.616
1999	1500	9.24	0.616
1999	1000	9.24	0.924
1999	1000	9.24	0.924
1999			
1999		MEAN	0.77

YEAR	CROSS SECTION [cmq]	LONGITUDINAL REINFORCEMENT [cmq]	% LONGITUDINAL REINFORCEMENT
2003			
2003	1200	9.24	0.77
2003	1200	9.24	0.77
2003	1200	9.24	0.77
2003	1200	9.24	0.77
2003	900	8.04	0.893333333
2003	900	8.04	0.893333333
2003	1050	6.16	0.586666667
2003	1050	6.16	0.586666667
2003	700	6.16	0.88
2003	1050	6.16	0.586666667
2003	1050	6.16	0.586666667
2003	1050	6.16	0.586666667
2003			
2003		MEAN	0.723333333
2003			
2003			
2003	1125	8.04	0.714666667
2003	1200	12.32	1.026666667
2003	1200	12.32	1.026666667
2003	1200	12.32	1.026666667

2003	1200	12.32	1.026666667
2003			
2003		MEAN	0.964266667

ANNEX 3- DIMENSIONS OF STRUCTURAL ELEMENTS OF THE VALIDATION BUILDING

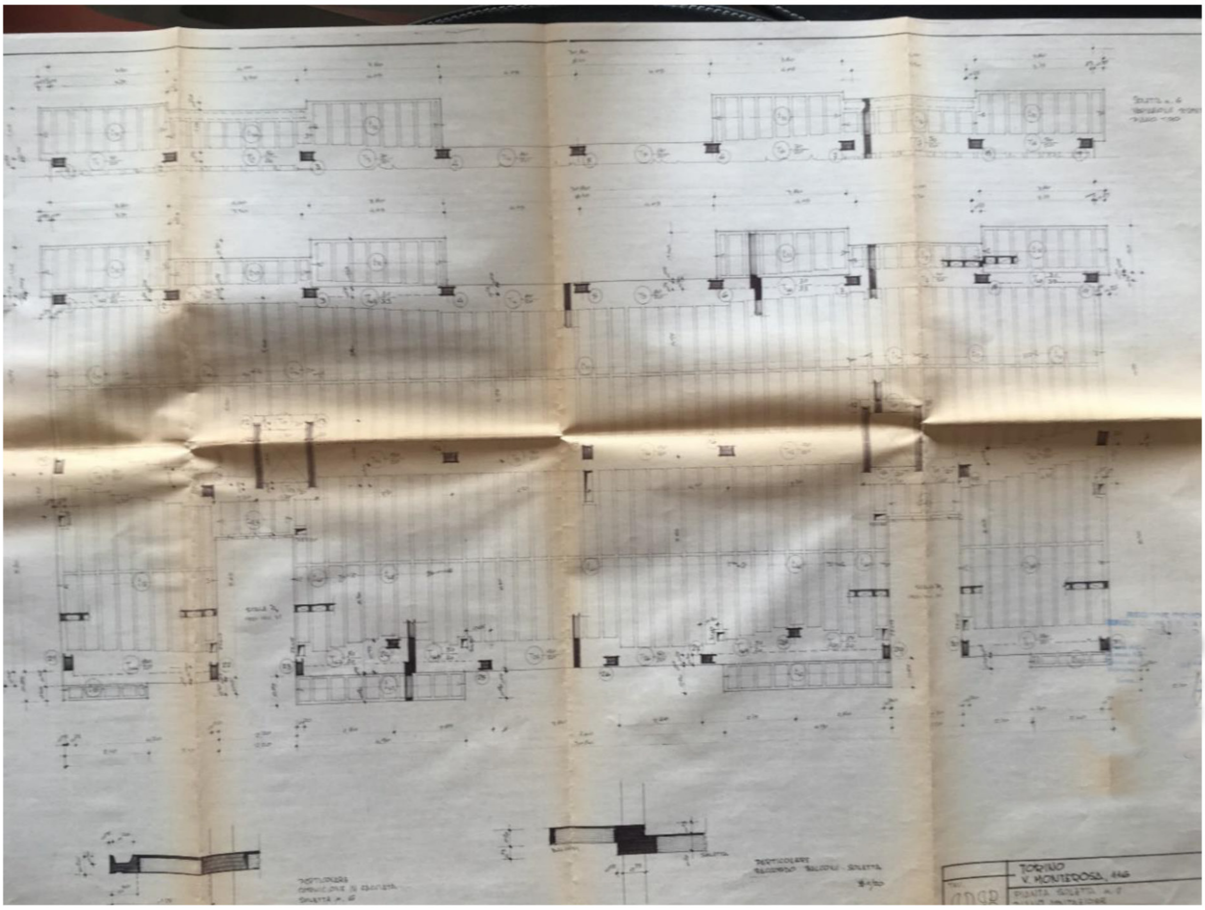
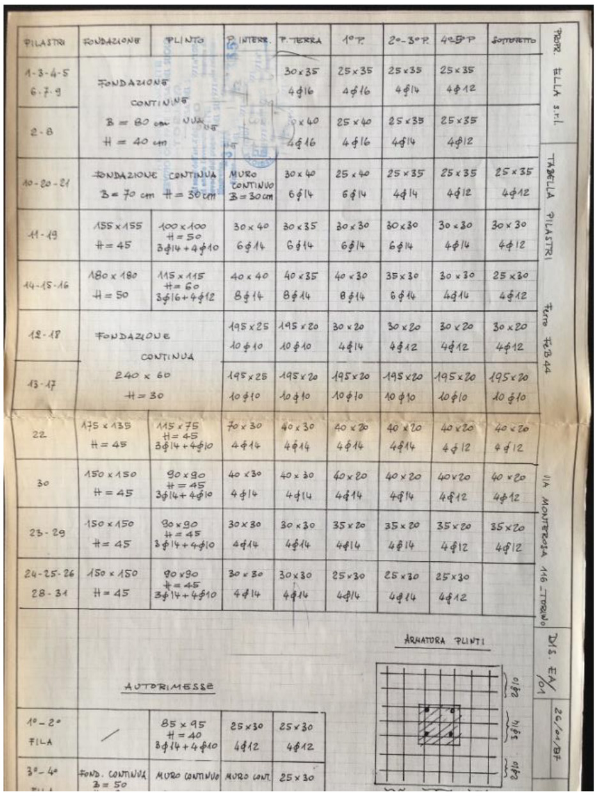


Figure 30- EXAMPLE OF SLAB ARRANGEMENT



PILASTRI	FONDAZIONE	PLINTO	P. INTERR.	P. TERRA	1° P.	2°-3° P.	4°-5° P	SOTTOTETTO
1-3-4-5 6-7-9	FONDAZIONE CONTINUE B = 80 cm H = 40 cm			30x35 4ϕ16	25x35 4ϕ16	25x35 4ϕ14	25x35 4ϕ12	
2-8				[illegible]x40 4ϕ16	25x40 4ϕ16	25x35 4ϕ14	25x35 4ϕ12	
10-20-21	FONDAZIONE CONTINUA B = 70 cm H = 30 cm		MURO CONTINUO B = 30 cm	30x40 6ϕ14	25x40 6ϕ14	25x35 4ϕ14	25x35 4ϕ12	25x35 4ϕ12
11-19	155x155 H = 45	100x100 H = 50 3ϕ14+4ϕ10	30x40 6ϕ14	30x35 6ϕ14	30x30 6ϕ14	30x30 6ϕ14	30x30 4ϕ14	30x30 4ϕ12
14-15-16	180x180 H = 50	115x115 H = 60 3ϕ16+4ϕ12	40x40 8ϕ14	40x35 8ϕ14	40x30 8ϕ14	35x30 6ϕ14	30x30 4ϕ14	25x30 4ϕ12
12-18	FONDAZIONE CONTINUA 240 x 60 H = 30		195x25 10ϕ10	195x20 10ϕ10	30x20 4ϕ14	30x20 4ϕ12	30x20 4ϕ12	30x20 4ϕ12
13-17			195x25 10ϕ10	195x20 10ϕ10	195x20 10ϕ10	195x20 10ϕ10	195x20 10ϕ10	195x20 10ϕ10
22	175x135 H = 45	115x75 H = 45 3ϕ14+4ϕ10	70x30 4ϕ14	40x30 4ϕ14	40x20 4ϕ14	40x20 4ϕ14	40x20 4ϕ12	40x20 4ϕ12
30	150x150 H = 45	90x90 H = 45 3ϕ14+4ϕ10	40x30 4ϕ14	40x30 4ϕ14	40x20 4ϕ14	40x20 4ϕ14	40x20 4ϕ12	40x20 4ϕ12
23-29	150x150 H = 45	90x90 H = 45 3ϕ14+4ϕ10	30x30 4ϕ14	30x30 4ϕ14	35x20 4ϕ14	35x20 4ϕ14	35x20 4ϕ12	35x20 4ϕ12
24-25-26 28-31	150x150 H = 45	90x90 H = 45 3ϕ14+4ϕ10	30x30 4ϕ14	30x30 4ϕ14	25x30 4ϕ14	25x30 4ϕ14	25x30 4ϕ12	

AUTORIMESSE

1°-2° FILA	/	85x95 H = 40 3ϕ14+4ϕ10	25x30 4ϕ12	25x30 4ϕ12
3°-4°	FOND. CONTINUA B = 50	MURO CONTINUO	MURO CONT.	25x30

ARMATURA PLINTI

PROPR. ELLA s.r.l.

TABELLA PILASTRI

Ferro FeB44

V.IA MONTEROSA 116_TORINO

DIS. EA/01

Figure 31- DIFFERENT DIMENSIONS OF COLUMNS

ANNEX 4 – RESULTS FOR THE REALISTIC MODEL

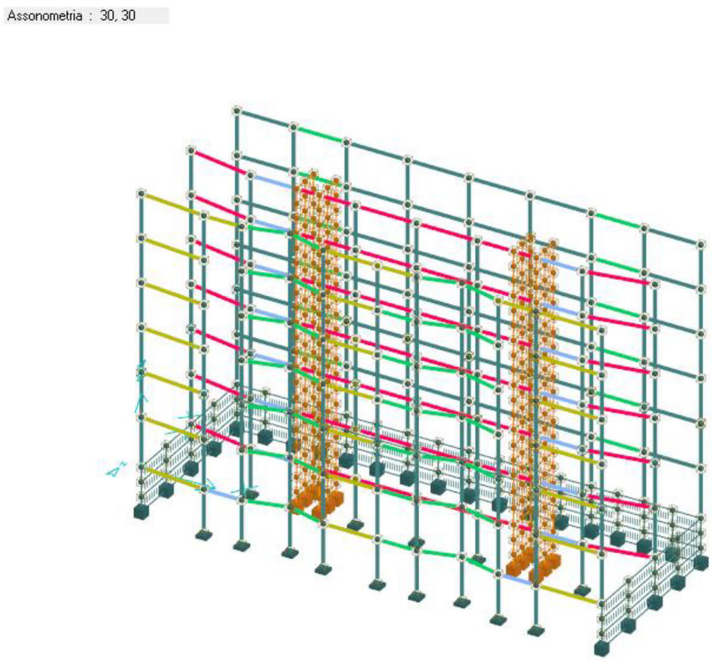


Figure 32- REALISTIC MODEL

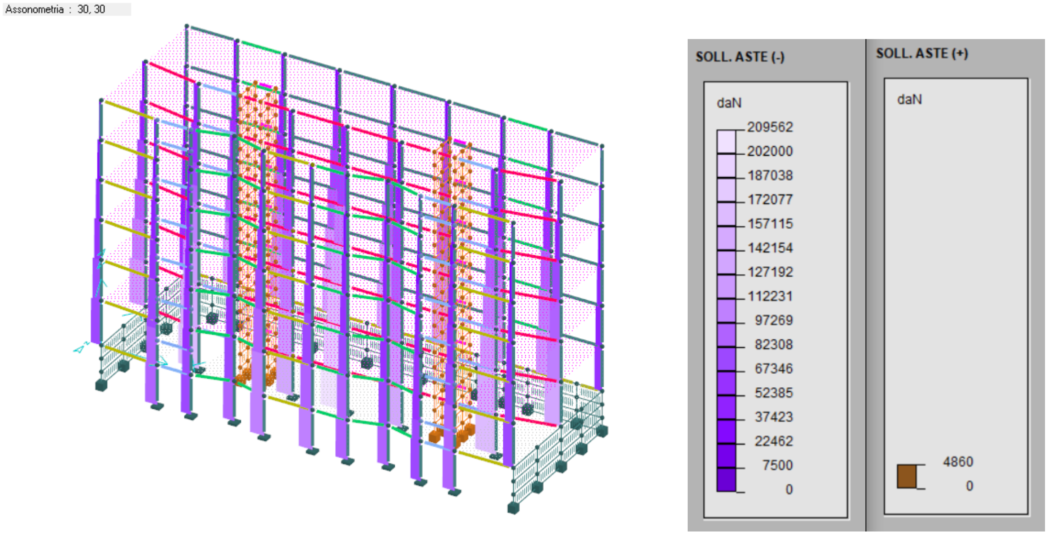


Figure 33- NORMAL FORCE

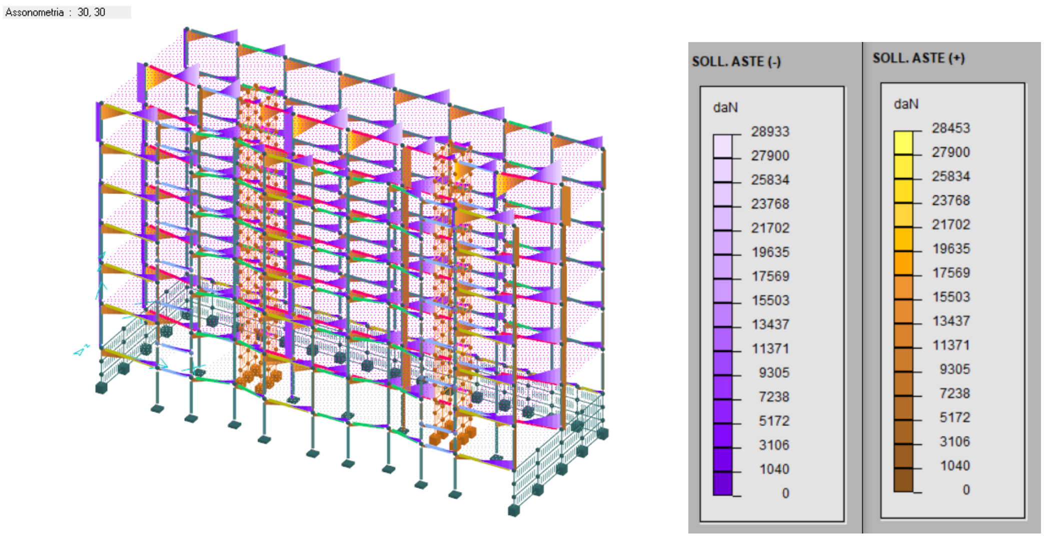


Figure 34- SHEAR TY

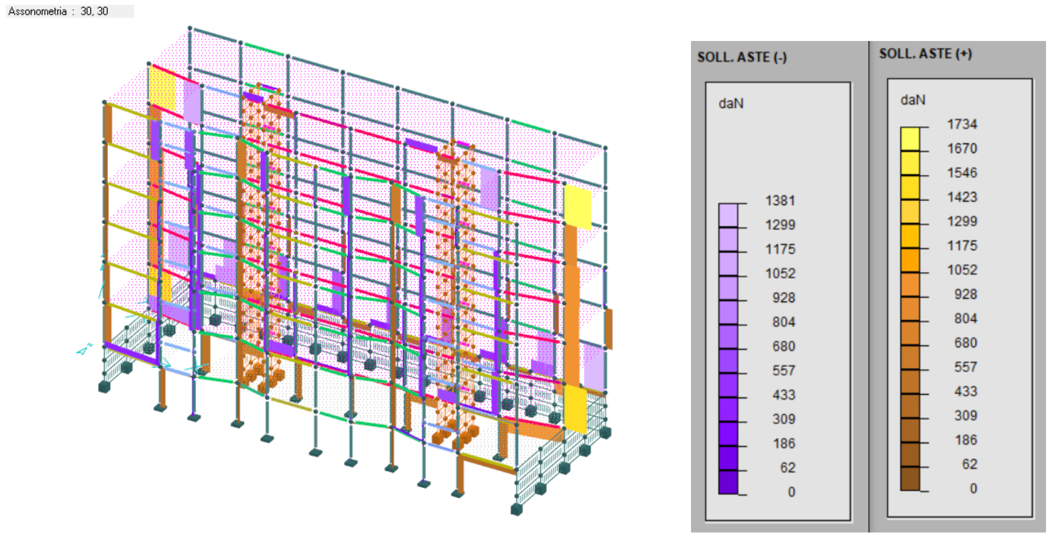


Figure 35- SHEAR TZ

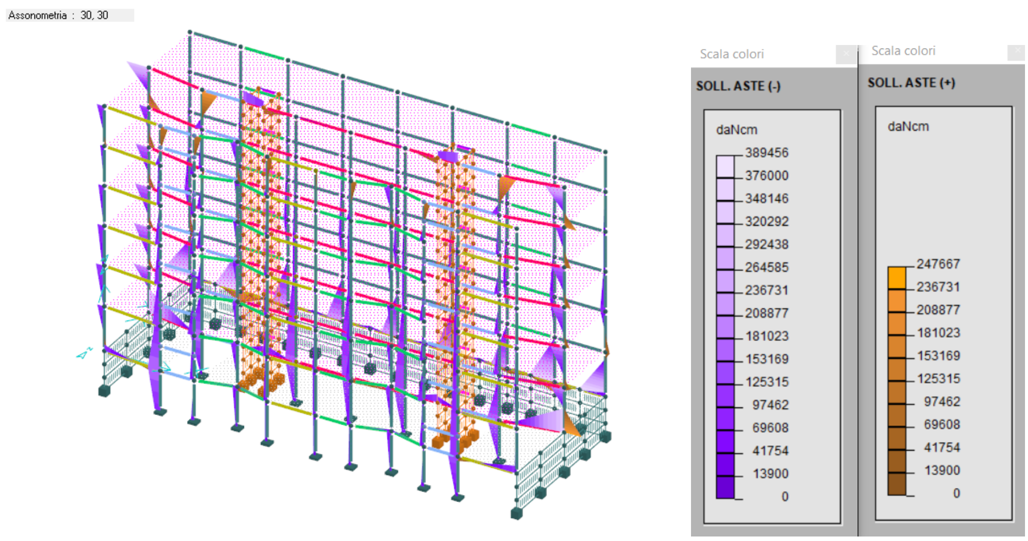


Figure 36- BENDING MOMENT MY

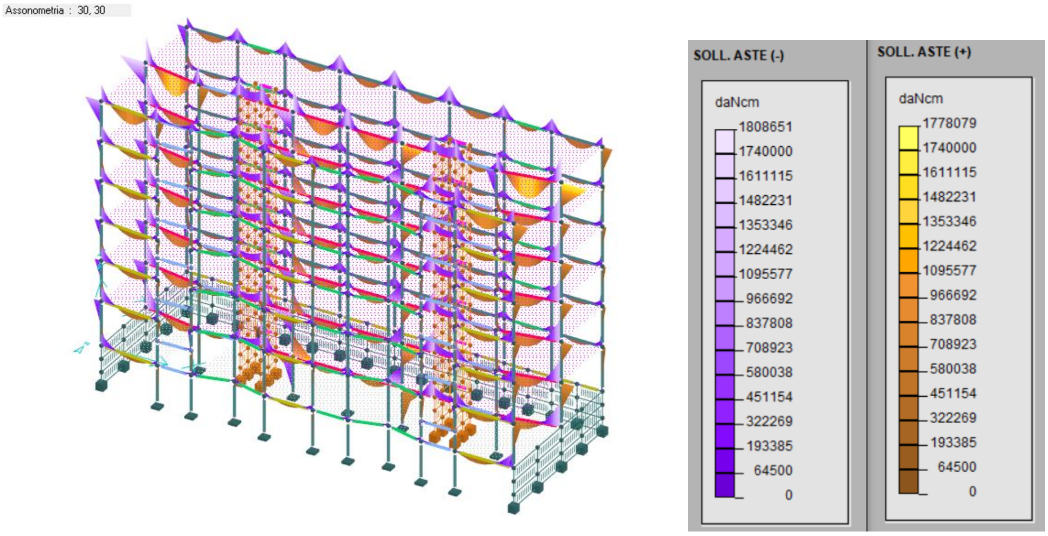


Figure 37- BENDING MOMENT MZ

ANNEX 5 – RESULTS FOR THE VIRTUAL MODEL

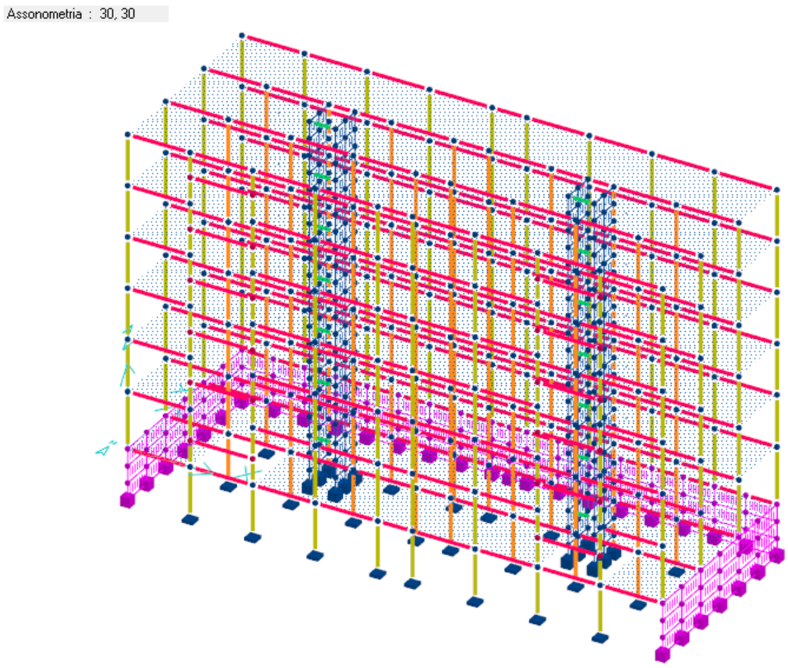


Figure 38- VIRTUAL MODEL

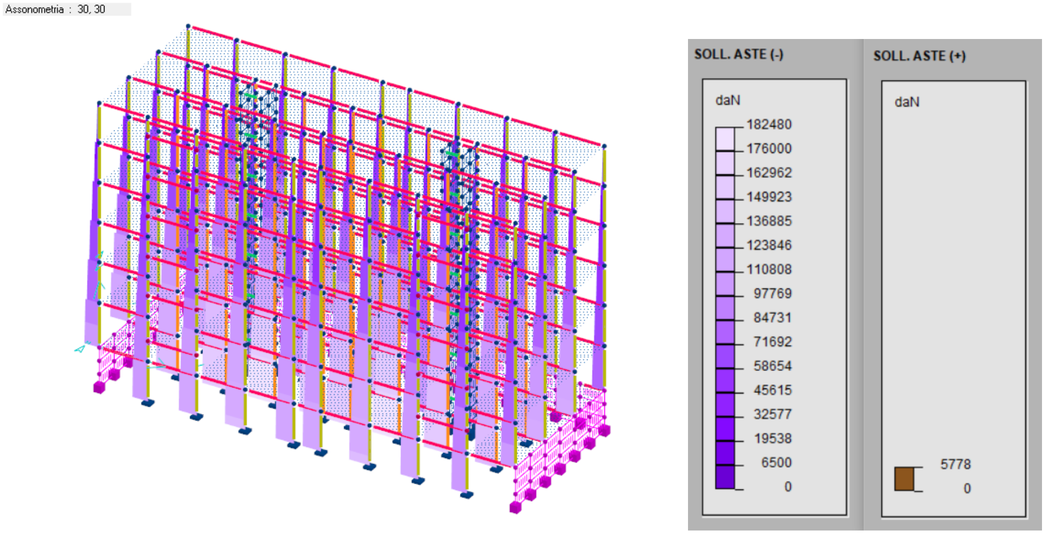


Figure 39- NORMAL FORCE

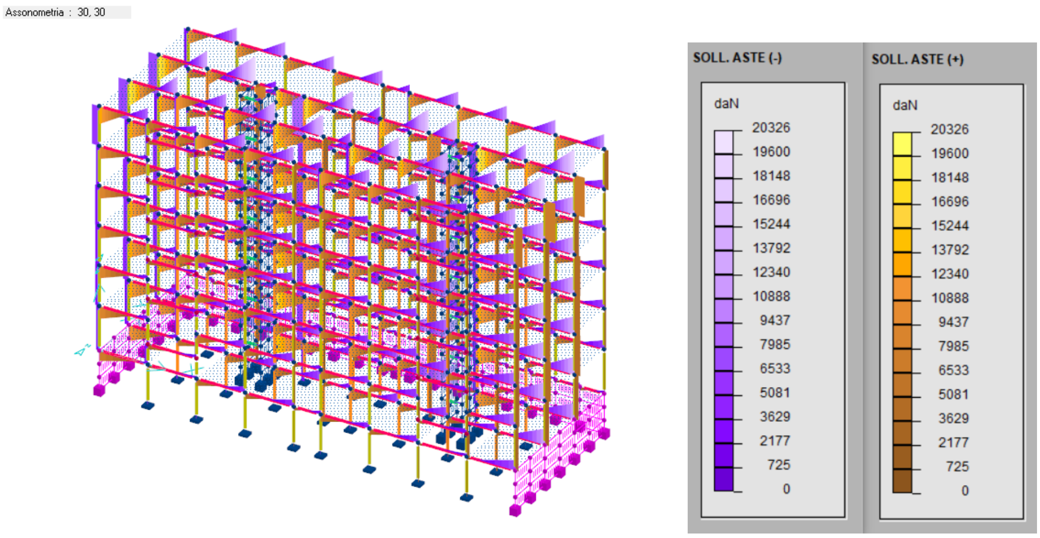


Figure 40- SHEAR FORCE TY

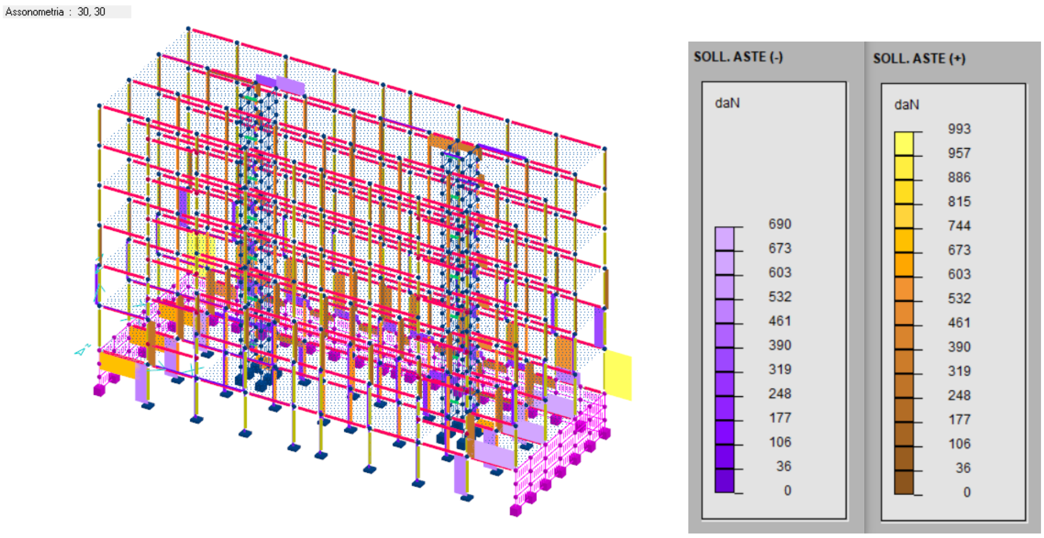


Figure 41- SHEAR FORCE TZ

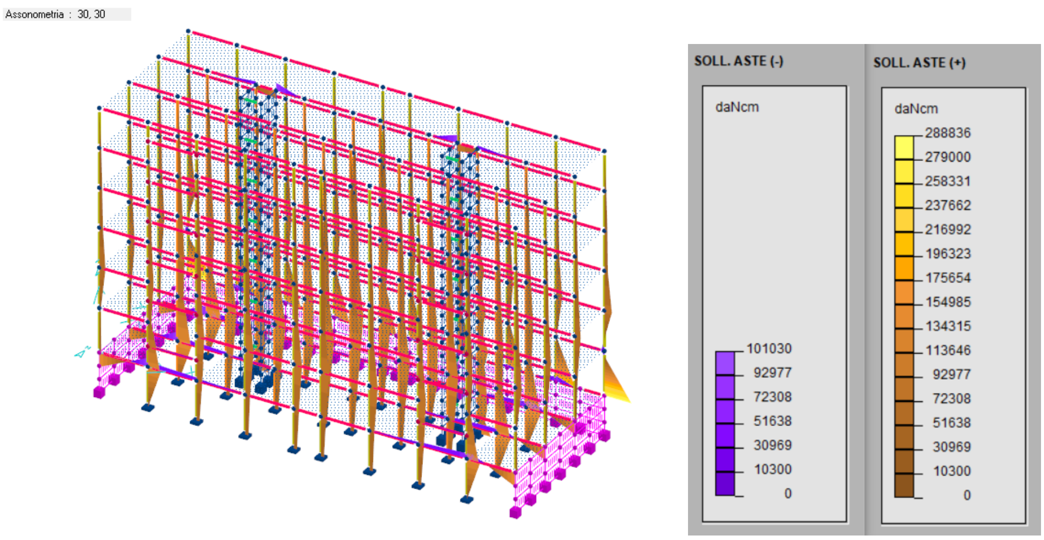


Figure 42- BENDING MOMENT MY

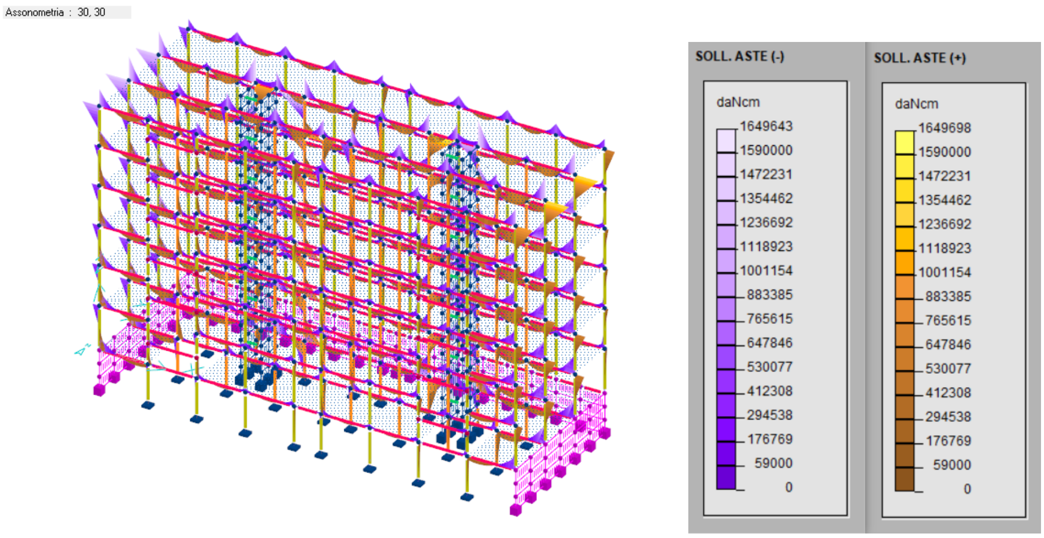


Figure 43- BENDING MOMENT MZ

10. ACKNOWLEDGEMENT

For this small part, I prefer to write in Italian…

Il lavoro affrontato per la stesura di questa tesi è stato per me, oltre che faticoso, fonte di ispirazione e maturazione: ho affrontato con interesse e tenacia questi argomenti, che spero possano entrare a far parte anche del mio futuro prossimo.

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