POLITECNICO DI TORINO

Master of Science program in
BUILDING ENGINEERING

Master's Thesis

Safety assessments of existing RC building

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March 2025

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1 INTRODUCTION

The building undergoing intervention is called "Town Hall" and is located in the **Municipality** of Torre de’ Passeri (PE) on Via Papa Giovanni XXIII; it consists of two building bodies: the main body (US1) and the bathrooms located outside (US2). The intervention in question concerns both building bodies. According to the Technical Standards for Construction referred to in Ministerial Decree 17/01/2018 and the related explanatory Circular no. 07/2019, the seismic upgrade aims to perform technical checks on seismic safety levels. The cognitive and functional analyses necessary for the project's realization involve verifying seismic safety levels through surveys, diagnostic investigation campaigns, and structural studies of the assets subject to the contract.

The geographical coordinates of the structure are:

	WGS84	ED50
LATITUDINE	42.244805	42.245776
LONGITUDINE	13.927504	13.928403

Table 1: coordination of the project

The safety objectives for the building under examination have been defined by the NTC 2018:

-Nominal Life, Buildings with High-Performance Levels (VN) set at 50 years.

-**Usage Class IV**, Buildings with Important Public or Strategic Functions.

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Figure 1 - Aerial View with an indication of the Building

2 DESCRIPTION OF THE STRUCTURE

As mentioned in the preceding paragraph, the structures undergoing structural assessment belong to the Town Hall of Torre de' Passeri. The complex is located in the northern area of the **municipality**: the north boundary is delimited by the railway line, while the southern boundary is Piazza 6.

The main building body features a reinforced concrete load-bearing structure and spans across one basement level and four above-ground floors, reaching a total gross floor area of approximately 1536.84 m2.

This document pertains to the structural units **US1** and **US2,** as depicted in the figure below. The structure extends across three above-ground floors and one basement level.

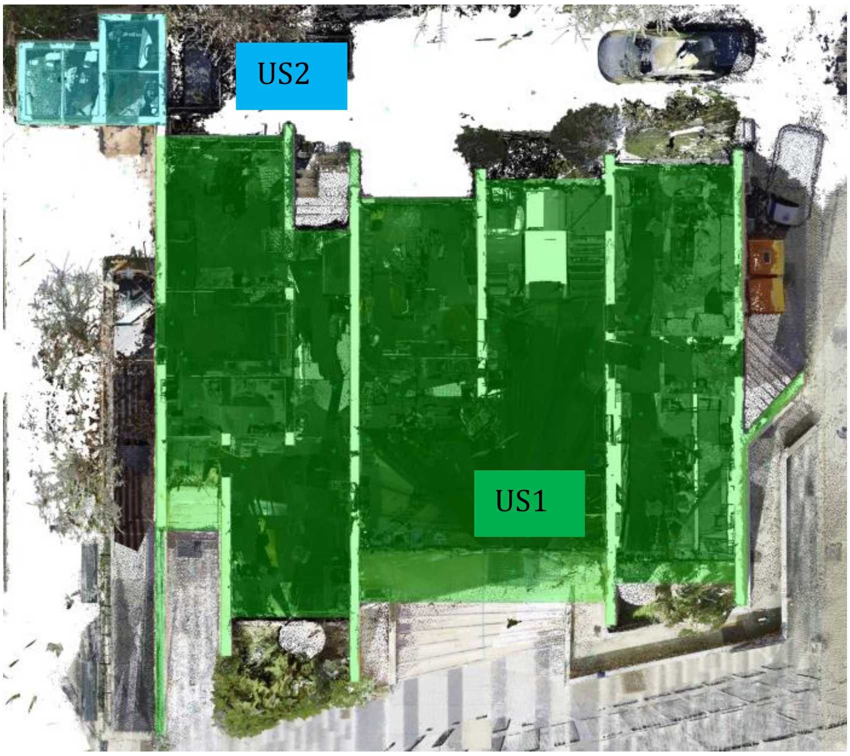


Figure 2 – Identification of structural units

3 ARCHITECTURAL MODELL EX-ANTE

In this section, to enhance clarity and comprehension, I have included architectural floor plans, sections, and views.

Figure 3 – Architectural plan–Underground floor

Figure 4 – Architectural plan–Ground floor

Figure 5 – Architectural plan–First Floor

Figure 6 – Architectural plan–Second Floor

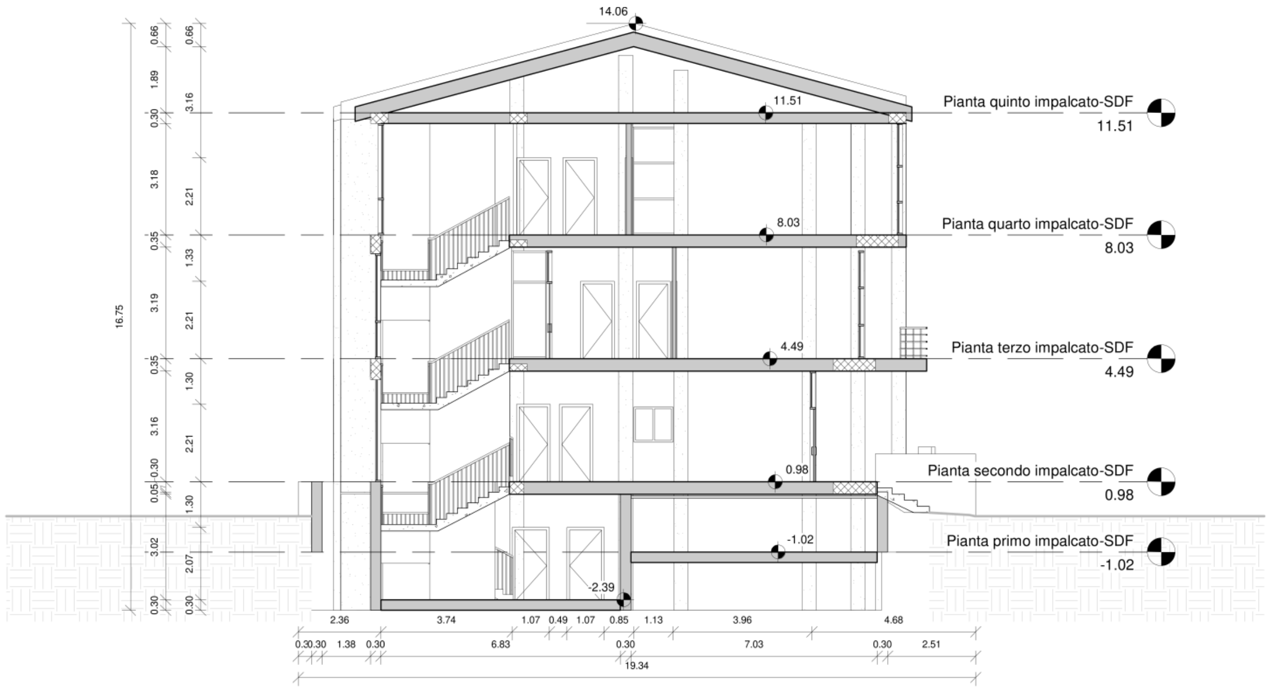


Figure 7– Architectural plan–Section 1-1

Figure 8– Architectural plan–Section 2-2

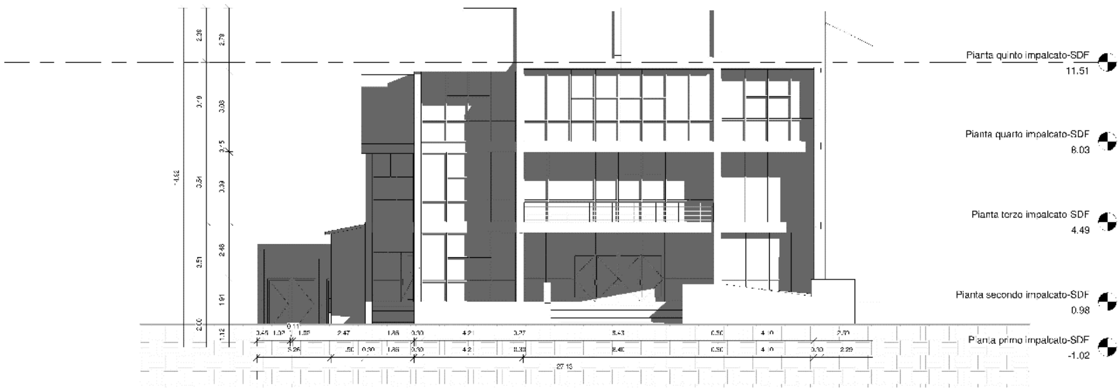


Figure 9– Architectural plan–Northern View

Figure 10– Architectural plan–Southern View

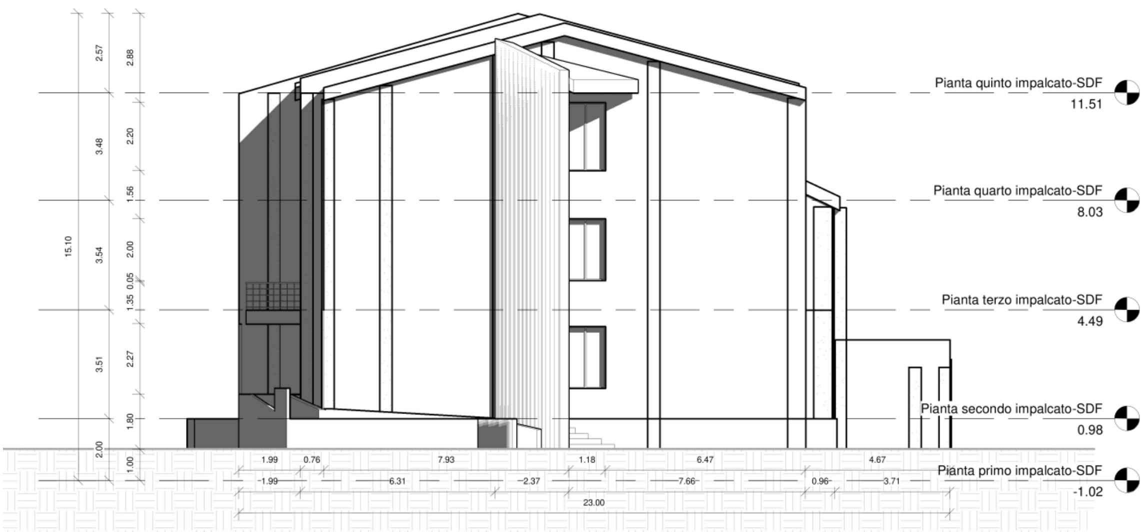


Figure 11– Architectural plan–Eastern View

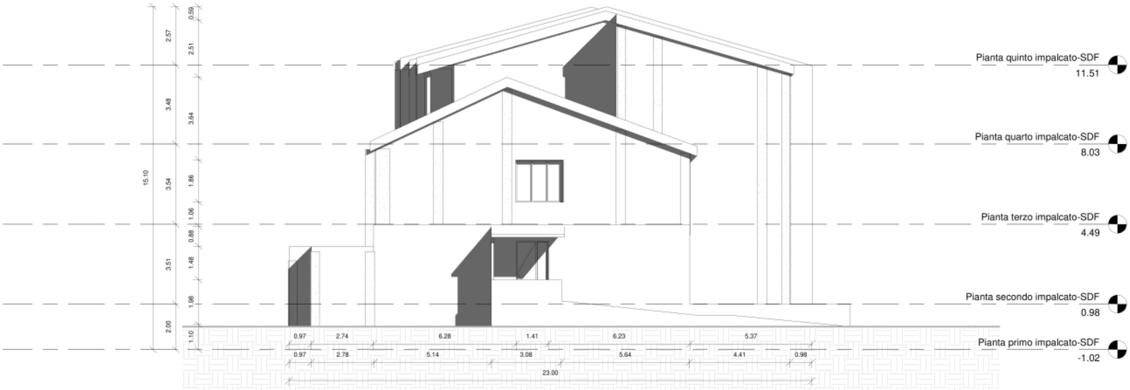


Figure 12– Architectural plan–Western View

4 STRUCTURAL MODEL EX-ANTE

In this section, structural plans have been included to enhance clarity and comprehension.

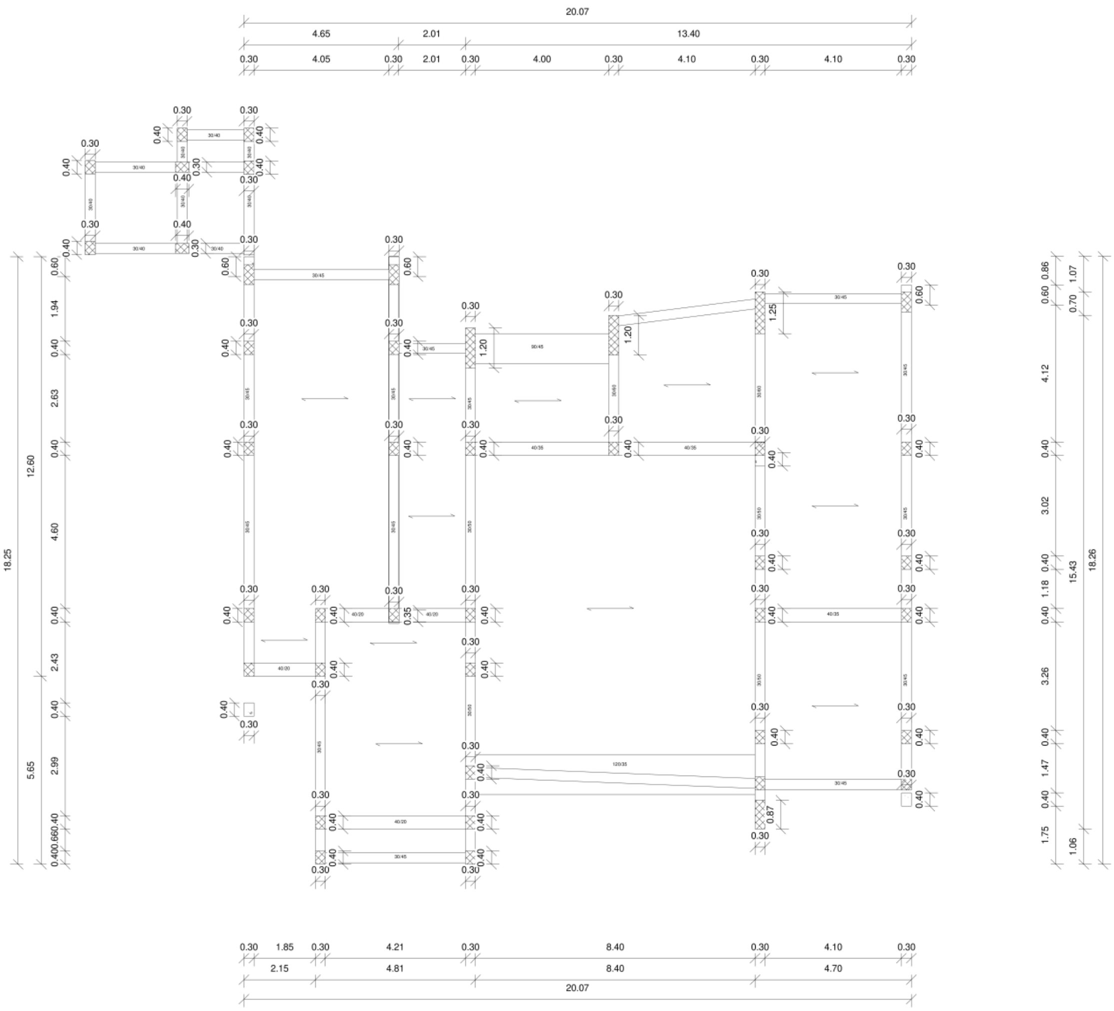


Figure 13– Structural plan– Underground floor slab

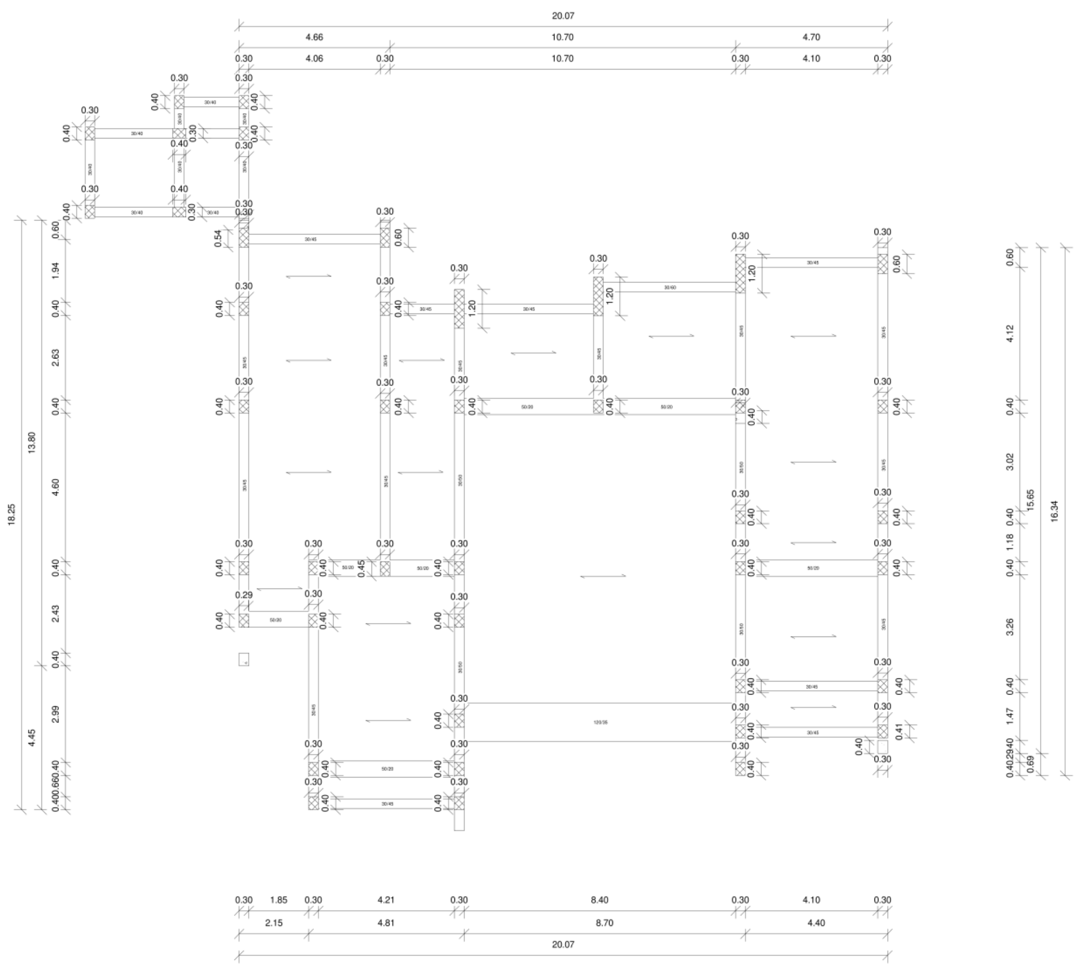


Figure 14– Structural plan– Ground floor slab

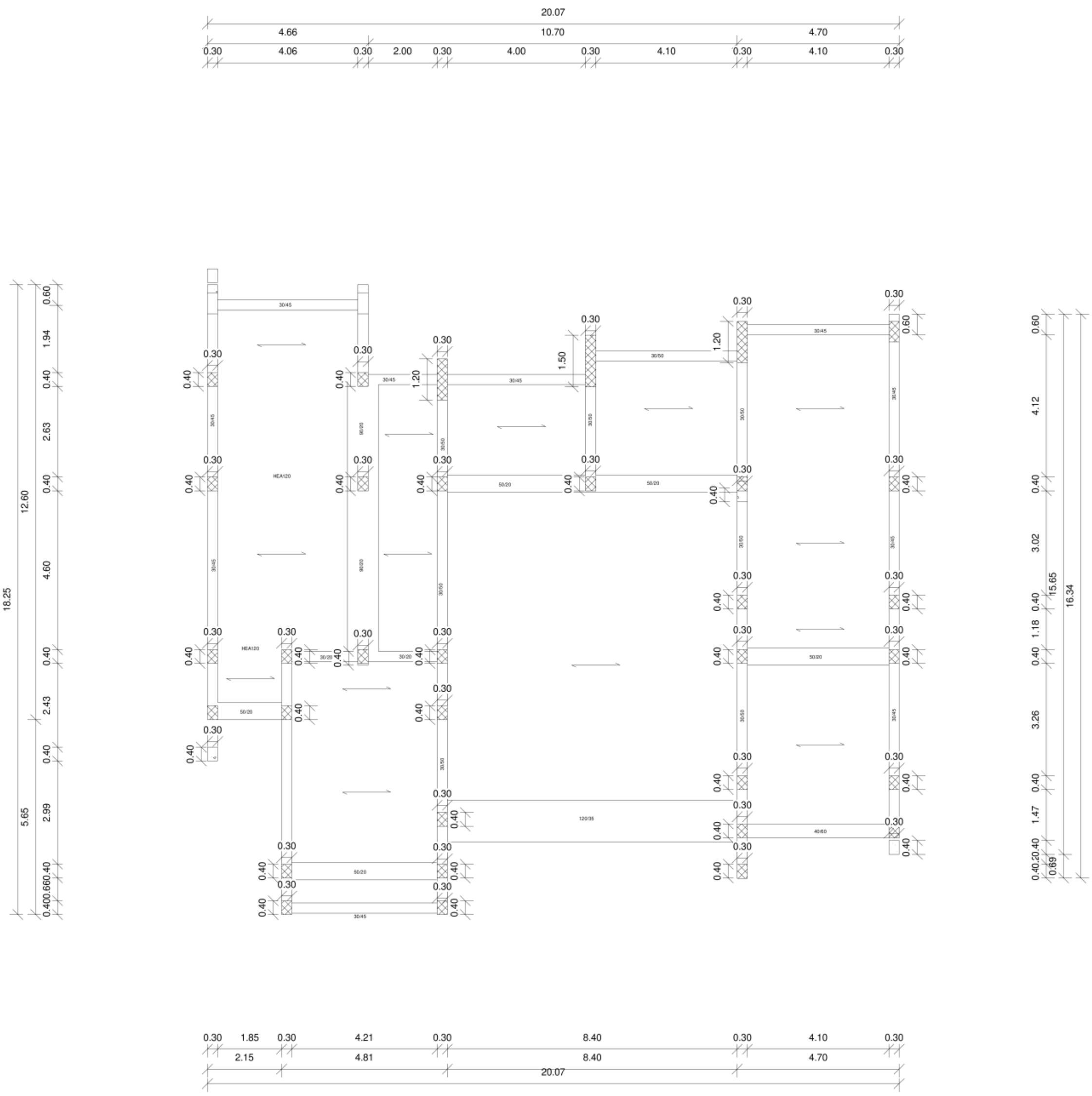


Figure 15– Structural plan– First-floor slab

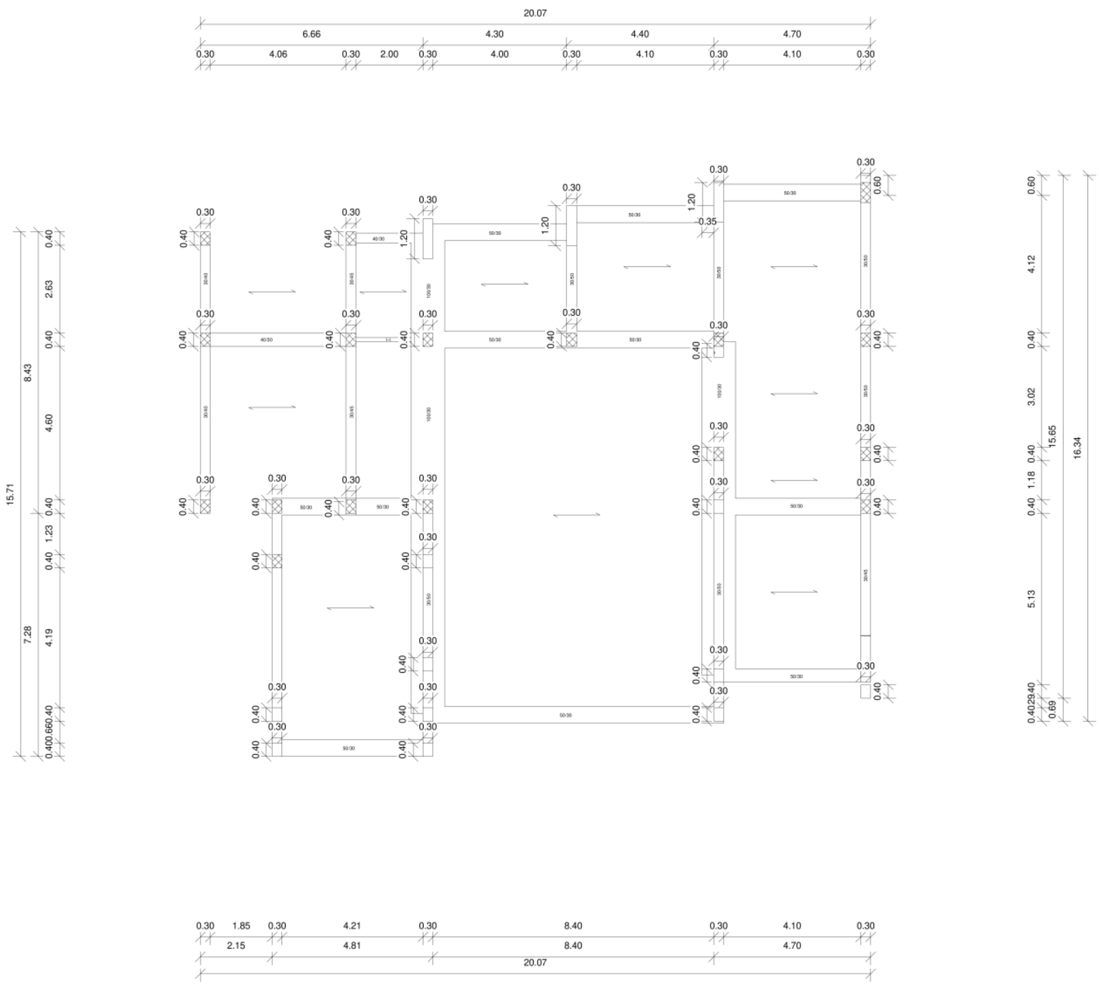


Figure 16– Structural plan–Second-floor slab

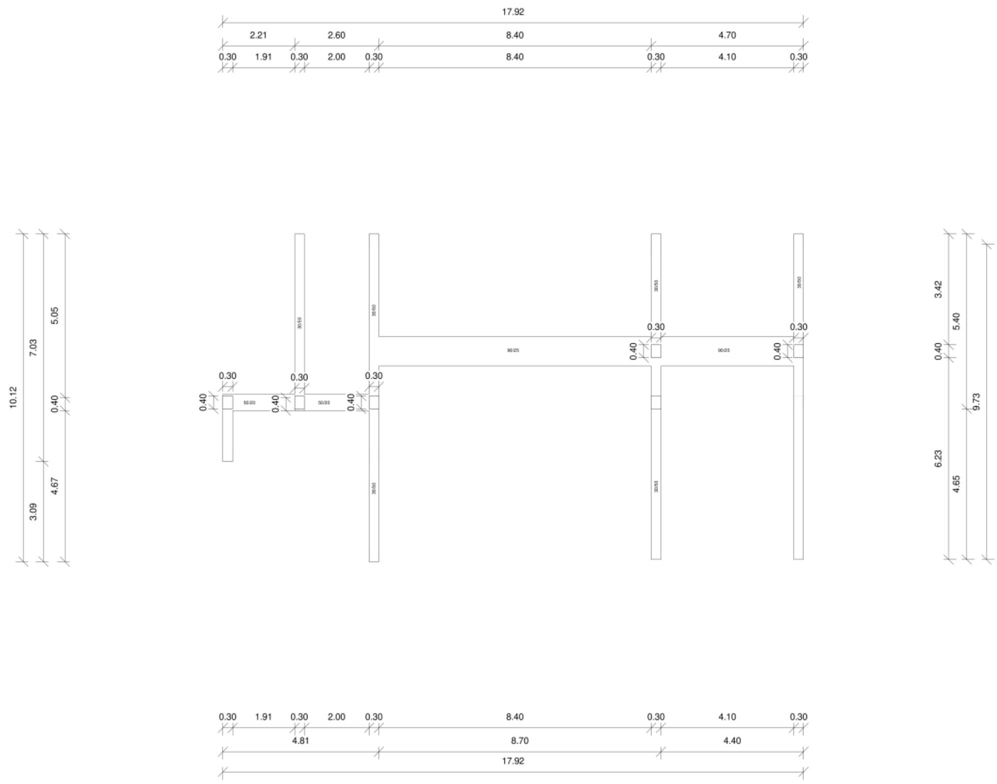


Figure 17– Structural plan–Third-floor slab

5 CLASSIFICATION AND SEISMIC BEHAVIOR OF SOIL

The **seismic classification** of the national territory has introduced specific technical regulations for the construction of buildings, bridges, and other structures in geographical areas characterized by the same seismic risk. Below is the seismic zone for the territory of Torre de' Passeri.

Zona sismica 1	Zona con pericolosità sismica alta. Indica la zona più pericolosa dove possono verificarsi fortissimi terremoti.

Table 2.Seismic zone of the municipality of Torre de'Passeri

The criteria for updating the seismic hazard map have been defined in the Ordinance of the Prime Minister n. 3519/2006, which divided the entire national territory into four seismic zones based on the value of the maximum **horizontal acceleration (a_g)** on rigid or flat ground, which has a 10% probability of being exceeded in 50 years.

Zona sismica	*Descrizione*	*accelerazione con probabilità di superamento del 10% in 50 anni [a_g]*	*accelerazione orizzontale massima convenzionale (Norme Tecniche) [a_g]*	*numero comuni con territori ricadenti nella zona (*)*
1	Indica la zona più pericolosa, dove possono verificarsi fortissimi terremoti.	$a_g > 0,25$ g	0,35 g	703
2	Zona dove possono verificarsi forti terremoti.	$0,15 < a_g \leq 0,25$ g	0,25 g	2.224
3	Zona che può essere soggetta a forti terremoti ma rari.	$0,05 < a_g \leq 0,15$ g	0,15 g	3.002
4	E' la zona meno pericolosa, dove i terremoti sono rari ed è facoltà delle Regioni prescrivere l'obbligo della progettazione antisismica.	$a_g \leq 0,05$ g	0,05 g	1.982

Table 3 - Identification of the Seismic Zone of the Municipality of Torre de' Passeri

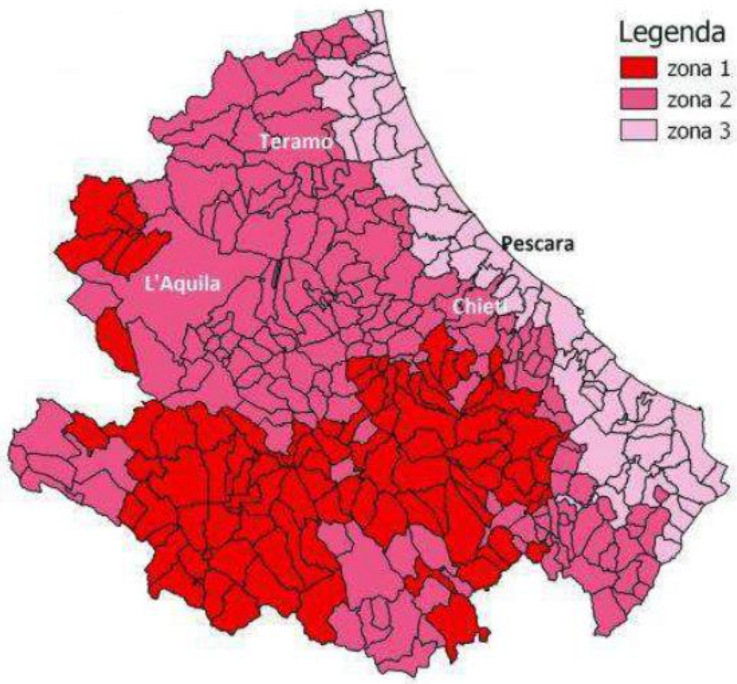


Figure 18 - Seismic Classification of the Abruzzo Region

On the following page, the historical seismicity of the municipality of Torre de' Passeri is reported for seismic events with Magnitude ≥ 4.00 as recorded in the "Parametric Catalog of Italian Earthquakes 2015 DBMI15.

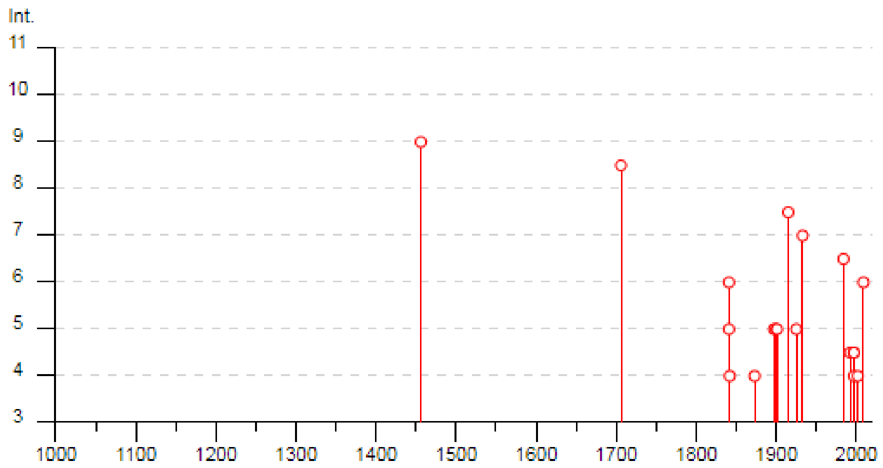


Figure19- Seismic Events Diagram for the Municipality of Torre de' Passeri

Effetti	In occasione del terremoto del				
Int.	Anno Me Gi Ho Mi Se	Area epicentrale	NMDP	Io	Mw
9	1456 12 05	Appennino centro-meridionale	199	11	7.19
8-9	1706 11 03 13	Maiella	99	10-11	6.84
6	1841 06 10	Maiella	11	7	4.96
5	1841 10 18 13	Valle del Pescara	1	5	4.16
4	1842 01 23	San Severino Marche	10	4-5	4.41
4	1873 03 12 20 04	Appennino marchigiano	196	8	5.85
5	1897 04 27 02 17 5	Maiella	27	5	4.21
5	1900 01 29 04 22	Alanno	13	5	4.08
5	1901 10 15 13 55 5	Alanno	10	5	4.22
NF	1908 01 16 10 27	Aquilano	11	4-5	4.12
7-8	1915 01 13 06 52 4	Marsica	1041	11	7.08
NF	1919 10 22 06 10	Anzio	142	6-7	5.22
5	1925 09 24 13 33 4	Molise occidentale	50	7	5.26
7	1933 09 26 03 33 2	Maiella	325	9	5.90
2	1938 08 12 02 28 3	Appennino laziale-abruzzese	55	5-6	4.56
6-7	1984 05 07 17 50	Monti della Meta	911	8	5.86
NF	1990 05 05 07 21 2	Potentino	1375		5.77
NF	1991 05 05 06 33 3	Aquilano	64	6	3.86
4-5	1992 02 18 03 30 0	Chietino	73	5-6	4.11
NF	1992 07 16 05 38 5	Chietino	107	5-6	4.22
4-5	1997 09 26 00 33 1	Appennino umbro-marchigiano	760	7-8	5.66
4-5	1997 09 26 09 40 2	Appennino umbro-marchigiano	869	8-9	5.97
4	1997 10 14 15 23 1	Valnerina	786		5.62
4	2002 11 01 15 09 0	Molise	638	7	5.72
2	2003 06 01 15 45 1	Molise	501	5	4.44
NF	2004 12 09 02 44 2	Teramano	213	5	4.09
6	2009 04 06 01 32 4	Aquilano	316	9-10	6.29

Figure 7 - Locati M., Camassi R., Rovida A., Ercolani E., Bernardini F., Castelli V., Caracciolo C.H., Tertulliani A., Rossi A., Azzaro R., D'Amico S., Conte S., Rocchetti E. (2016). DBMI15, the 2015 version of the Italian Macroseismic Database. Istituto Nazionale di Geofisica e Vulcanologia. doi: http://doi.org/10.6092/INGV.IT-DBMI15

It is observed that the municipality of Torre de’ Passeri falls within seismic hazard zone 918 "Abruzzo Apennines" of seismic hazard zoning ZS9, according to the seismic hazard map (INGV – C. Meletti and G. Valensise, 2004), based on the Ordinance P.C.M. of March 20, 2003, No. 3274.

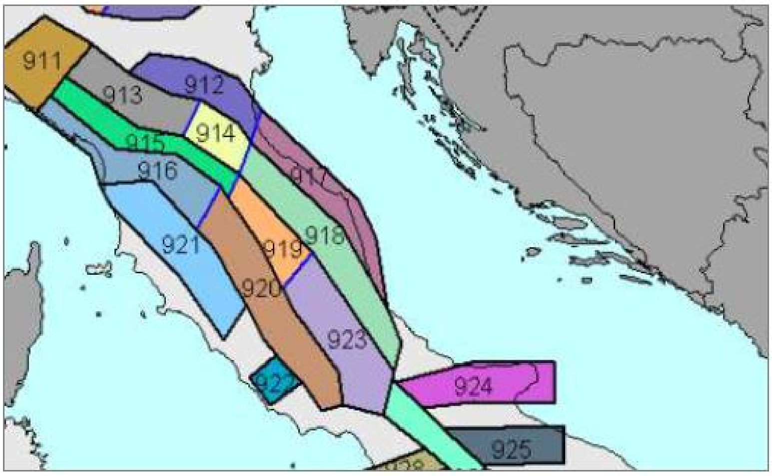


Figure 20- an excerpt from the Seismic Zoning Map ZS9 (by Meletti and Valensise, 2004, http://zonesismiche.mi.ingv.it/).

6 DEFINITION OF BASIC STRUCTURAL MODELLING DATA

6.1 Safety assessment

This report aims to assess the seismic vulnerability of the building under scrutiny. The goal is to identify any structural weaknesses that may arise under the seismic forces specified by current regulations.

We evaluate the structure against three limit states: operational limit state (SLO), damage limit state (SLD), and life safety limit state (SLV).

To analyze the building's seismic vulnerability, we conducted a nonlinear static analysis (Push-over). This method was chosen because linear methods cannot track the evolving dynamic behavior during a seismic event, the development of plasticization mechanisms, or the actual distribution of ductility demands across structural elements. Linear methods typically concentrate these aspects into a single parameter called the "Structure Factor" (q).

7 NOMINAL LIFE, USAGE CLASS, REFERENCE PERIOD

7.1 Nominal Life

The **nominal life (V_n)** of a structural work represents the duration, in years, for which the structure, given regular maintenance, remains fit for its intended purpose. This parameter underscores the expected lifespan of the structure, ensuring its sustained functionality and safety over time. The specific nominal life for various types of structures is detailed in the subsequent table.

TIPI DI COSTRUZIONE		**Vita Nominale V_N** (in anni)
1	Opere provvisorie – Opere provvisionali – Strutture in fase costruttiva	≤10
2	**Opere ordinarie, ponti, opere infrastrutturali e dighe di dimensioni contenute o di importanza normale**	**≥50**
3	Grandi opere, ponti, opere infrastrutturali e dighe di grandi dimensioni o di importanza strategica	≥100

Table 4:Nominal life according to construction type

The Nominal Life (VN) of a building, as defined in § 2.4.1 of the NTC (Italian Building Code), is the duration that must be taken into consideration during the design phase regarding the durability of the structures. It guides the sizing of structures and construction details, the selection of materials, and the implementation of protective measures to ensure the maintenance of strength and functionality. In the design predictions, therefore, if environmental and usage conditions remain within expected limits, extraordinary maintenance interventions to restore the construction's durability will not be necessary until the end of this period. The actual lifespan of the building cannot be assessed during the design phase, as it depends on future events beyond the designer's control. In fact, the vast majority of buildings have had and continue to have, even though subsequent maintenance interventions, a much longer actual lifespan than the nominal life quantified in the NTC. Referring to Table 2.4.1, it is highlighted that, according to the effects of the Decree of the Head of the Department of Civil Protection No. 3685 of October 21, 2003, the strategic nature of a work or its relevance to the consequences of a possible collapse is defined by its usage class.

Considering that the building in question falls under Type 2, a nominal life of ≥50 years will be imposed in accordance with the client's requirements. Therefore, a Nominal Life (VN) of 50 years will be assumed. At the end of the period specified by the VN, the building must undergo vulnerability assessment again.

7.2 Usage Classe

In the event of seismic actions and considering the consequences of operational disruptions or potential collapses, buildings are classified into usage classes as follows:

Class I: Buildings with occasional presence of people, such as agricultural buildings.

Class II: Buildings with normal occupancy levels, without hazardous environmental content or essential public and social functions. Industries with non-hazardous environmental activities. Bridges, infrastructure works, road networks not falling into Class III or Class IV, and railway networks whose interruption does not lead to emergency situations. Dams whose collapse does not result in significant consequences.

Class III: Buildings with significant occupancy levels. Industries with hazardous environmental activities. Extra-urban road networks do not fall into Class IV. Bridges and railway networks whose interruption leads to emergency situations. Dams are significant for the consequences of their potential collapse.

Class IV: Buildings with important public or strategic functions, especially regarding civil protection management in case of disasters. Industries with particularly hazardous environmental activities. Road networks of Type A or B, as defined by D.M. 5 November 2001, No. 6792, "Functional and Geometric Norms for Road Construction," and Type C when part of routes connecting provincial capitals not served by Type A or B roads. Bridges and railway networks are critical for maintaining communication routes, especially after a seismic event. Dams connected to the operation of water supply systems and electricity production plants.

The value of the usage coefficient Cu varies according to the usage class, as shown in the following table:

ClASSO DI USO	I	II	III	IV
COEFFICIENT Cu	0.7	1.0	1.5	2.0

Table 5:Usage coefficient (Tab. 2.4.I NTC 2018)

The building under assessment belongs to **Class IV**, for which the associated usage coefficient **Cu is 2.00.**

7.3 Reference period for Seismic Action

Seismic actions on each building are evaluated in relation to a reference period V_R, derived for each type of construction by multiplying its nominal life V_N by the usage coefficient C_U:

$V_R = V_N \cdot C_U$

Considering that the construction is of Type 2 and the building's usage class is IV, the reference period V_R is determined to be:

$V_R = 50 \cdot 2 = 100$ years

The reference period V_R is of significant importance because, assuming that the seismic action recurrence law follows a Poisson process, it is used to evaluate, given the probability of exceedance PVR corresponding to the considered limit state (Table 3.2.1 of the NTC), the return period TR of the seismic action to be referenced for verification.

8 ON-SITE INVESTIGATIONS

For buildings, to acquire the level of knowledge, the confidence factor (FC), and the properties of materials

Confidence factors, derived from the level of knowledge acquired, are applied to average material strength values obtained from both destructive and non-destructive tests. This process estimates the average material strengths within the considered confidence interval, typically set at 95%. Determining confidence factors for different structural elements or assemblies involves considering uncertainties in material strength estimation and identifying construction details. The acquired level of knowledge from surveys, investigations into structural details, and material tests guide the application of confidence factors to material properties. This approach may vary for different structural elements or groups of elements, and the most suitable analysis method is recommended. In the absence of specific assessments, Table C8.5. IV serves as a reference.

Livello di conoscenza	Geometrie (carpenterie)	Dettagli strutturali	Proprietà dei materiali	Metodi di analisi	FC (*)
LC1		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 e dinamica	1,35
LC2	Da disegni di carpenteria originali con rilievo visivo a campione; in alternativa rilievo completo ex-novo	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

Table 6:Level of Knowledge, Geometries (carpentry), Structural Details, Material Properties, Analysis Methods, Confidence Factor (Fc)

LC1: This level is achieved when a historical-critical analysis commensurate with the level considered has been carried out (referring to § C8.5.1). The structure's geometry is known based on original drawings (verified by a visual survey of a sample to confirm actual correspondence to the drawings) or a survey in case the construction drawings are not available. Detailed construction information is derived from a simulated project (referring to § C8.5.2), and limited on-site investigations on the reinforcements and connections in the most important elements have been conducted (collected data must allow for local resistance checks). In the absence of information on the mechanical characteristics of materials (from construction drawings or test certificates), typical values from the construction practice of the time, validated by limited on-site tests on the most important elements, have been adopted (referring to § C8.5.3); the corresponding confidence factor is FC=1.35. Safety assessment is generally performed using linear, static, or dynamic analysis; the gathered information must allow for the development of a suitable structural model.

LC2: This level is achieved when a historical-critical analysis, tailored to the considered level, has been conducted (as referenced in § C8.5.1). The structure's geometry is determined from original drawings or a survey. Construction details are either known, partially from original construction drawings supplemented by limited on-site investigations into reinforcements and connections of key elements or obtained from extensive on-site investigations (as per § C8.5.2). Mechanical properties of materials are obtained from construction drawings, supplemented by limited on-site tests or extensive on-site tests (as described in § C8.5.3), resulting in a confidence factor of FC=1.2. Safety assessments are conducted using linear or nonlinear, static or dynamic analysis methods. Data collected on structural element dimensions and structural details enable the development of an appropriate structural model.

This level is considered achieved when a historical-critical analysis, tailored to the relevant level, has been conducted (as referenced in § C8.5.1), and the structure's geometry is determined from original drawings or a survey. Construction details are known either from original construction drawings supplemented by limited on-site investigations into reinforcements and connections of key elements or from extensive on-site investigations (as per § C8.5.2). Mechanical properties of materials are obtained from construction drawings and original test certificates, supplemented by limited on-site tests (if values obtained from on-site tests are lower than those indicated in original test certificates, exhaustive on-site tests are conducted) or through exhaustive on-site tests (as described in § C8.5.3). The corresponding confidence factor is FC=1. Safety assessment is carried out using linear or nonlinear, static or dynamic analysis methods. The information collected on structural element dimensions and structural details must enable the development of a suitable structural model.

The material strengths used in the capacity formulas of the elements are derived from the average strengths obtained from available information and additional on-site tests, divided by the FC values indicated in Table C8.5. IV.

FC values can also be evaluated differently for different materials based on statistical considerations conducted on a significant dataset for the elements under consideration and methods of proven validity.

As a purely indicative measure, in Tables C8.5.V and C8.5.VI, the level of investigations (limited, extensive, exhaustive) is linked to the number of surveys of construction details and tests for the assessment of material mechanical characteristics. It is understood that the investigation plan must be appropriately calibrated based on the preliminary analysis (see § C8.5.2.2 and C8.5.3.2) and, therefore, in relation to the level of knowledge to be achieved, directed towards the necessary investigations in the areas of the structure where it is deemed appropriate, both in relation to the static commitment of the different elements and their role in the safety of the structure and in relation to the degree of consistency of the results of preliminary tests and their agreement with what is provided in the original documents.

livello di indagini e prove	Rilievo (dei collegamenti) [a]	Prove (sui materiali) [b],[c],[d]
	Per ogni elemento "primario" (trave, pilastro)	
limitato	La quantità e disposizione dell'armatura è verificata per almeno il 15% degli elementi	1 provino di cls. per 300 m² di piano dell'edificio, 1 campione di armatura per piano dell'edificio
esteso	La quantità e disposizione dell'armatura è verificata per almeno il 35% degli elementi	2 provini di cls. per 300 m² di piano dell'edificio, 2 campioni di armatura per piano dell'edificio
esaustivo	La quantità e disposizione dell'armatura è verificata per almeno il 50% degli elementi	3 provini di cls. per 300 m² di piano dell'edificio, 3 campioni di armatura per piano dell'edificio

Table 7. C8.5.V- Guideline Definition of Survey and Testing Levels for Reinforced Concrete Buildings

EXPLANATORY NOTES TO TABLES C8.5.V AND C8.5.VI

The percentages of elements to be investigated and the number of samples to be extracted and subjected to resistance tests are reported in Tables C8.5.V and C8.5.VI are indicative and should be adapted to individual cases, taking into account the following aspects:

(a) When checking the achievement of the percentages of investigated elements for the survey of construction details, consideration is given to any repetitive situations that allow extending the controls to a wider percentage of checks on certain structural elements that are part of a series with obvious characteristics of repeatability, for equal geometry and role in the structural scheme.

(b) Steel tests aim to identify the steel class used according to the regulations in force at the time of construction. For the purpose of achieving the required number of steel tests to acquire the desired level of knowledge, it is advisable to consider the diameters (in reinforced concrete structures) or the profiles (in steel structures) most commonly used in the main elements, excluding stirrups.

(c) For material tests, it is allowed to replace some destructive tests, up to 50%, with at least triple the number of non-destructive tests, single or combined, calibrated on the destructive ones.

(d) The number of samples reported in Tables C8.5.V and C8.5.VI can be varied, either increased or decreased, depending on the material homogeneity characteristics. In the case of in-situ concrete, these characteristics are often related to the typical construction methods of the construction era and the type of structure, which should be considered when planning the investigation. It will be appropriate, in this regard, to plan for a second campaign of supplementary tests if the results of the first campaign are highly heterogeneous.

8.1 Summery of Qualification of Tests

Below is a summary of the quantification of tests performed for the two structural units, US1 and US2, according to the level of knowledge LC2, as per table C8.5.V of the current NTC 2018 regulations and related explanatory notes.

The survey campaign carried out for the two structural Units includes:

40 ferroscan for checking the construction details of beams, columns, and nodes in reinforced concrete;

5 ferroscan for evaluating the framework of the floors;

9 endoscopies for verifying the stratigraphic sequence of the floors and 4 for vertical elements;

22 extractions of concrete samples (cores);

20 extractions of steel bar samples;

1 load test;

The quantification of tests allows for a comprehensive geometric survey and the acquisition of a level of knowledge equivalent to **LC2**.

8.2 Material Properties

Within the calculation model, the mechanical characteristics of materials were considered using the values obtained from the investigation campaign. Location of the samplings and photographic documentation The hardened concrete samplings, named with the letter "PRC_P or T" and a progressive number identifying the level of belonging and the number of the element from the project numbering, were carried out corresponding to the pillar and beam elements in reinforced concrete of the building in question. Below is a table containing the location of the hardened concrete samplings identified based on the numbering of pillars indicated during the geometric survey, with the unique identification code of the samples, the dimensions of the samplings, and the specimens derived from them.

Figure21- Location of material Investigation_Underground Floor

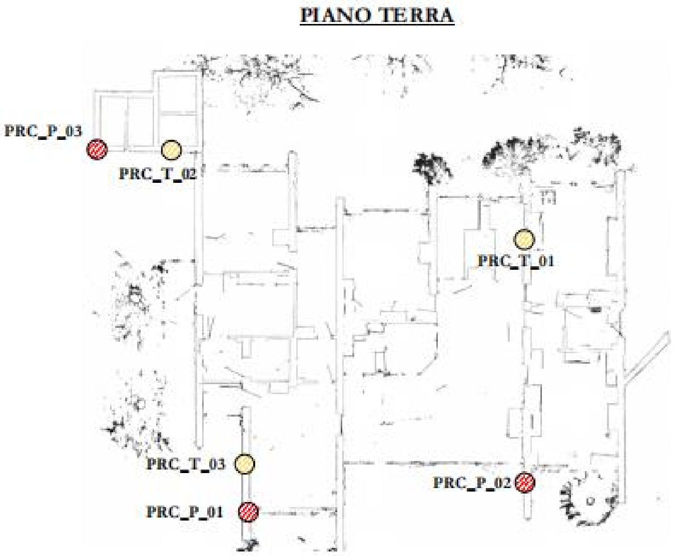


Figure22- Location of material Investigation_Ground Floor

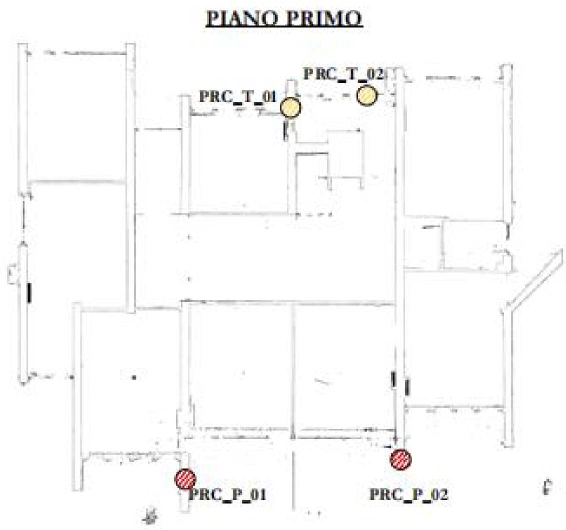


Figure23- Location of material Investigation_First Floor

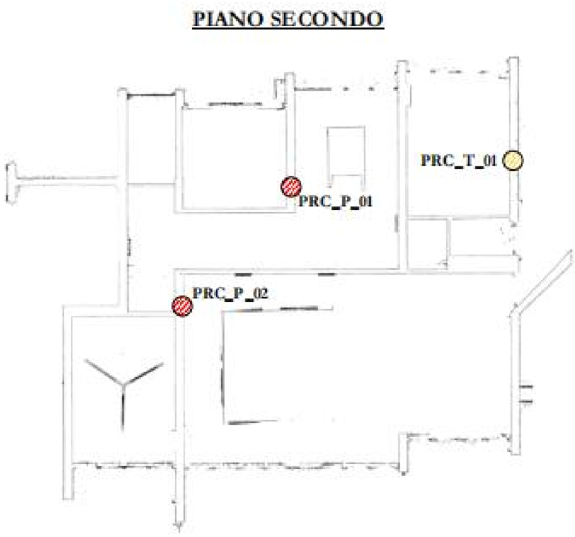


Figure24- Location of material Investigation_Second Floor

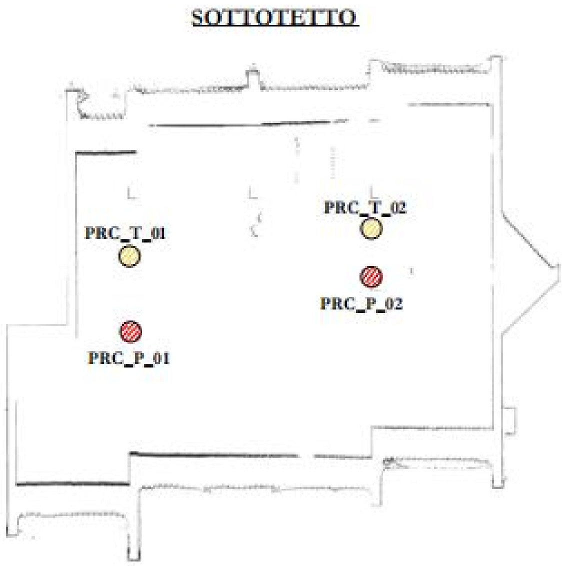


Figure25- Location of material Investigation_Attic

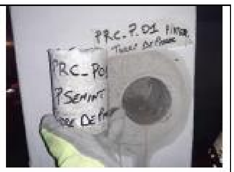
Sigla	Ubicazione / Pos. in Opera	Dimensioni Diam. [mm]	Lungh. [mm]	Anomalie / Armature	Profondità di Carbonatazione [mm]
PRC_T_01	TRAVE PIANO INTERRATO	84,6	133	Nessuna	
PRC_P_01	PILASTRO PIANO INTERRATO	84,6	125	Nessuna	
PRC_P_02	PILASTRO PIANO INTERRATO	84,6	185	Nessuna	
PRC_T_02	TRAVE PIANO INTERRATO	84,6	158	Nessuna	
PRC_S_01	SETTO PIANO INTERRATO	84,6	170	Nessuna	
PRC_P_03	PILASTRO PIANO TERRA	84,6	195	Nessuna	
PRC_T_02	TRAVE PIANO TERRA	84,6	153	Nessuna	
PRC_T_03	TRAVE PIANO TERRA	84,6	125	Nessuna	
PRC_P_02	PILASTRO PIANO TERRA	84,6	150	Nessuna	
PRC_T_01	TRAVE PIANO TERRA	84,6	155	Nessuna	
PRC_P_01	PILASTRO PIANO TERRA	84,6	148	Nessuna	
PRC_P_01	PILASTRO PIANO PRIMO	84,6	178	Nessuna	
PRC_P_02	PILASTRO PIANO PRIMO	84,6	163	Nessuna	
PRC_T_01	TRAVE PIANO PRIMO	84,6	188	Nessuna	
PRC_T_02	TRAVE PIANO PRIMO	84,6	183	Nessuna	
PRC_P_01	PILASTRO PIANO SOTTOTETTO	84,6	148	Nessuna	
PRC_P_02	PILASTRO PIANO SOTTOTETTO	84,6	170	Nessuna	
PRC_T_01	TRAVE PIANO SOTTOTETTO	84,6	135	Nessuna	
PRC_T_02	TRAVE PIANO SOTTOTETTO	84,6	158	Nessuna	
PRC_P_01	PILASTRO PIANO SECONDO	84,6	165	Nessuna	
PRC_P_02	PILASTRO PIANO SECONDO	84,6	148	Nessuna	
PRC_T_01	TRAVE PIANO SECONDO	84,6	155	Nessuna	

Table 8: location and geometric characteristics of the section

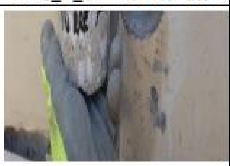
DOCUMENTAZIONE FOTOGRAFICA


PRC_T_01 – P. Interrato


PRC_T_02 – P. Interrato


PRC_P_01 – P. Interrato


PRC_P_02 – P. Interrato


PRC_S_01 – P. Interrato


PRC_T_03 – P. Terra


PRC_P_01 – Sottotetto


PRC_T_02 – P. Primo


PRC_P_02 – Sottotetto


PRC_T_02 – Sottotetto


PRC_P_01 – P. Primo


PRC_P_02 – P. Primo


PRC_T_01 – P. Primo


PRC_P_01 – P. Terra


PRC_T_01 – Sottotetto


PRC_P_01 – P. Secondo


PRC_T_01 – P. Secondo


PRC_T_01 – P. Terra


PRC_P_02 – P. Terra


PRC_P_03 – P. Terra


PRC_T_02 – P. Terra


PRC_P_02 – P. Secondo

Figure26- photographic documentation_Samples of materials

Description and method of test execution

The samples, taken using an electric rotary core drill equipped with a diamond-tipped crown cooled with water, were cut at the bases and then weighed and measured with 1 mm accuracy, verifying conformity with the UNI 12504-1 and UNI EN 12390-3 standards. The tests were carried out in accordance with the UNI EN 12504-1 standard, with the following test conditions, unless otherwise specified:

The samples undergoing testing do not present cracked, indented, or flaked surfaces;

The samples undergoing testing have a length-to-diameter ratio of 1:1;

The samples were placed on the plate of the press, without the interposition of deform-able material and in axis with the load, and were brought to breakage with a load gradient of 0.60 ± 0.20 MPa per second;

The test was performed with the sample surface dry.

Expression of results

The compressive strength of the cylindrical specimen (core) is determined using the following expression:

$f_{core} = N/A$ (MPa)

Where:

N = Break load (N);

A = Area of the reactive section (mm^2)

In accordance with the indications contained in the most recent Guideline on the matter, specifically the "Guidelines for methods of investigation on structures and soils for the projects of repair, improvement, and reconstruction of unusable buildings" issued by the Department of Civil Protection in March 2012, the in-situ cylindrical strength fc, is of the cement conglomerate was calculated using the following formulation from the ACI 214.4R regulation issued by the American Concrete Institute:

$Rc= F_d * f_{core}$

where:

- $R_{c,\ is}$ = In-situ cubic resistance of the sample of cement conglomerate;
- f_{core} = Value of strength resulting from the compression of the concrete specimen;
- F_d = Resistance correction factor that takes into account the disturbance caused to the material during the sampling (coring) phases and assumes a value between 1.00 and 1.10, dependent on

fcore (according to the "Guidelines for the assessment of concrete characteristics in situ" (2017) of the Superior Council of Public Works - Central Technical Service)

8.3 Test Results

Below is a summary of the compressive strength values of the hardened concrete cores taken using the methods already described in this chapter, attached to this Test Report.

Sigla	Ubicazione / Pos. in Opera	Resistenza del Provino(1) [MPa]	Resistenza Cubica(2) [MPa]	Profondità di Carbonatazione [mm]	Note
PRC_T_01	TRAVE PIANO INTERRATO	18,2	19,9	---	---
PRC_P_01	PILASTRO PIANO INTERRATO	16,4	18,0	---	---
PRC_P_02	PILASTRO PIANO INTERRATO	15,2	16,6	---	---
PRC_T_02	TRAVE PIANO INTERRATO	18,7	20,4	---	---
PRC_S_01	SETTO PIANO INTERRATO	29,8	31,6	---	---
PRC_P_03	PILASTRO PIANO TERRA	20,4	22,3	---	---
PRC_T_02	TRAVE PIANO TERRA	17,1	18,7	---	---
PRC_T_03	TRAVE PIANO TERRA	24,5	26,4	---	---
PRC_P_02	PILASTRO PIANO TERRA	16,2	17,7	---	---
PRC_T_01	TRAVE PIANO TERRA	18,4	20,1	---	---
PRC_P_01	PILASTRO PIANO TERRA	12,6	13,8	---	---
PRC_P_01	PILASTRO PIANO PRIMO	12,8	14,1	---	---
PRC_P_02	PILASTRO PIANO PRIMO	17,8	19,4	---	---
PRC_T_01	TRAVE PIANO PRIMO	14,9	16,3	---	---
PRC_T_02	TRAVE PIANO PRIMO	16,1	17,6	---	---
PRC_P_01	PILASTRO PIANO SOTTOTETTO	26,9	28,9	---	---
PRC_P_02	PILASTRO PIANO SOTTOTETTO	25,1	27,1	---	---
PRC_T_01	TRAVE PIANO SOTTOTETTO	27,9	29,8	---	---
PRC_T_02	TRAVE PIANO SOTTOTETTO	22,6	24,6	---	---
PRC_P_01	PILASTRO PIANO SECONDO	21,0	22,9	---	---
PRC_P_02	PILASTRO PIANO SECONDO	17,5	19,1	---	---
PRC_T_01	TRAVE PIANO SECONDO	15,2	16,6	---	---

Table 9:Determination of compressive strength of concrete cores

The document outlines the method for checking how strong the reinforcement bars (rebar) are in a concrete structure's pillars. To get the rebar for testing, small parts of the concrete were removed using ferromagnetic equipment. Reinforcement bars belonging to pillars and beams of various orders of elevation were sampled to undergo tensile testing. Each sample of rebar tested contains a code that includes letters and numbers (for instance, S1) to identify it. "S" stands for the kind of sample, and "1" is just a number to keep track of each test. The purpose of the test is to determine the tensile strength of the material under examination.

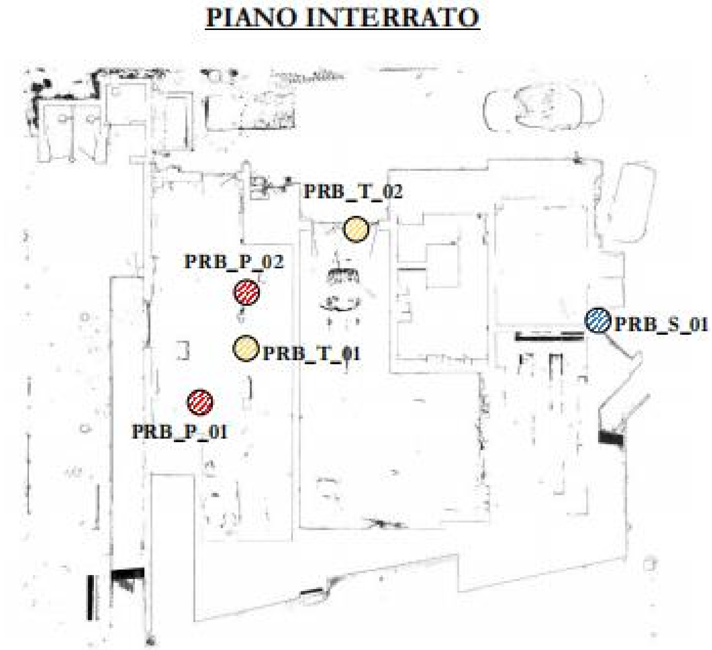


Figure 27- Location of material Investigation_Underground Floor

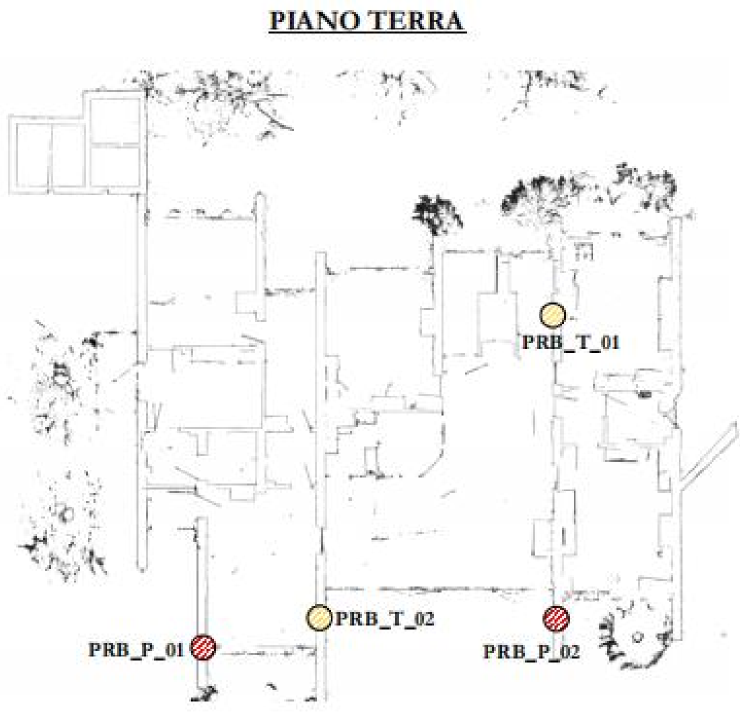


Figure 28- Location of material Investigation_Ground Floor

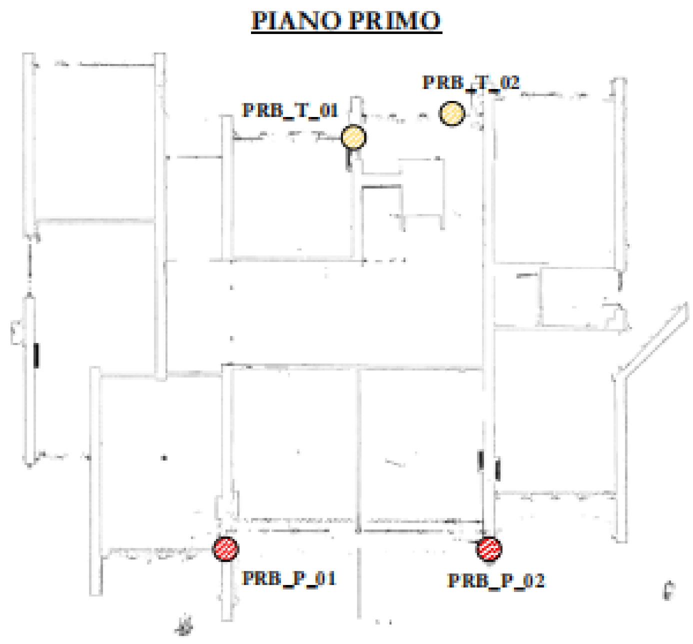


Figure 29- Location of material Investigation_First Floor

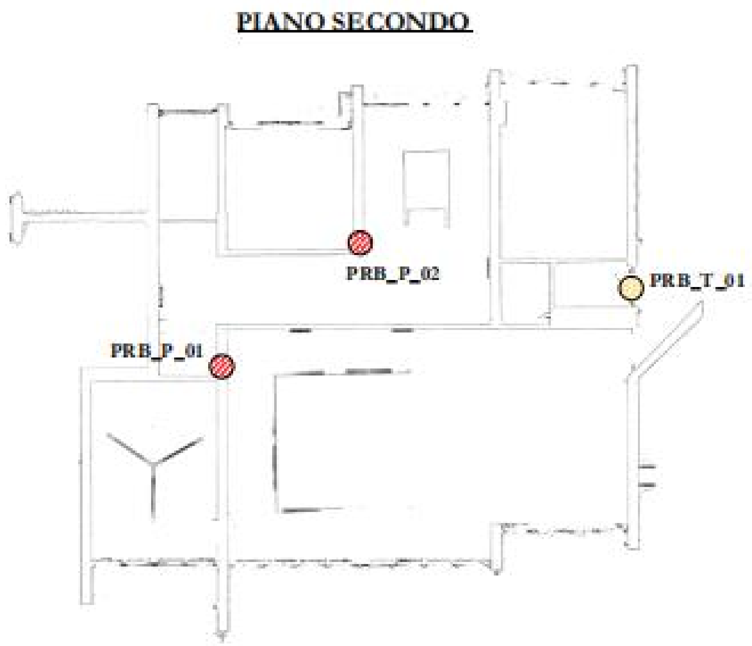


Figure 30- Location of material Investigation_Second Floor

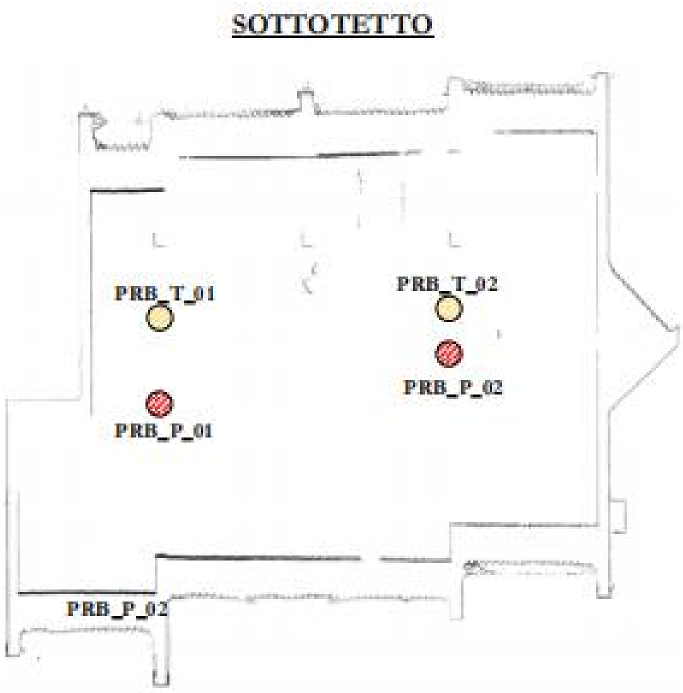


Figure 31- Location of material Investigation Attic

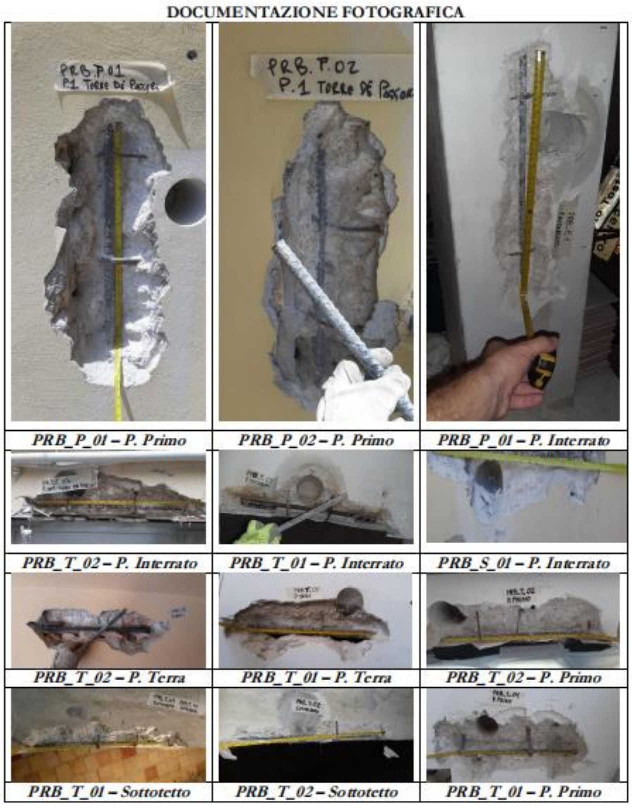


Figure 32 - photographic documentation Samples of materials

Sigla	Tipologia	Posizione in Opera	Ø(2) [mm]	Snervam. [MPa]	Rottura [MPa]	Allungam. A_5 [%]
PRB_T_01	a.m.	Trave piano seminterrato	16,1	531,8	708,9	21,6
PRB_P_01	a.m.	Pilastro piano seminterrato	12,1	544,3	760,8	22,0
PRB_P_02	a.m.	Pilastro piano seminterrato	17,9	535,6	726,8	21,4
PRB_T_02	a.m.	Trave piano seminterrato	14,0	520,0	701,0	21,6
PRB_S_01	a.m.	Setto piano seminterrato	14,1	518,4	676,1	21,2
PRB_P_01	a.m.	Pilastro piano primo	17,6	535,7	779,6	21,4
PRB_P_02	a.m.	Pilastro piano primo	17,6	530,1	752,0	21,0
PRB_T_01	a.m.	Trave piano primo	16,0	467,5	642,8	23,7
PRB_T_02	a.m.	Trave piano primo	16,0	500,9	695,8	22,6
PRB_T_01	a.m.	Trave piano terra	16,0	485,9	651,9	23,0
PRB_T_02	a.m.	Trave piano terra	16,0	499,3	686,4	22,1
PRB_P_01	a.m.	Pilastro piano terra	18,2	514,7	701,9	21,1
PRB_P_02	a.m.	Pilastro piano terra	17,6	508,4	706,5	22,0
PRB_P_01	a.m.	Pilastro piano sottotetto	16,3	511,9	722,4	21,7
PRB_P_02	a.m.	Pilastro piano sottotetto	16,3	520,0	727,5	22,2
PRB_T_01	a.m.	Trave piano sottotetto	14,0	527,5	747,8	23,6
PRB_T_02	a.m.	Trave piano sottotetto	14,0	530,6	749,1	22,5
PRB_P_01	a.m.	Pilastro piano secondo	18,0	517,7	724,9	23,1
PRB_P_02	a.m.	Pilastro piano secondo	18,1	485,7	694,7	24,2
PRB_T_01	a.m.	Trave piano secondo	14,0	501,6	700,1	23,0

Table 10:Determination of tensile strength of reinforcement rebars

In the calculation model, we included the mechanical properties of materials based on the results from our survey. Test results for average concrete cores and reinforcement bars in beams:

Prove su carote in c.a.			
Travi			
Carota	Piano	Resistenza provino (Mpa)	Resistenza cubica (Mpa)
PRC_T_01	Interrato	18.2	19.9
PRC_T_02	Interrato	18.7	20.4
PRC_T_02	Terra	17.1	18.7
PRC_T_03	Terra	24.5	26.4
PRC_T_01	Terra	18.4	20.1
PRC_T_01	Primo	14.9	16.3
PRC_T_02	Primo	16.1	17.6
PRC_T_01	Secondo	15.2	16.6
PRC_T_01	Sottotetto	27.9	29.8
PRC_T_02	Sottotetto	22.6	24.6
Media		**19.36**	**21.04**

Table 11: resistance test on reinforcement concrete cores for beams

Prove sulle barre di armatura			
Travi			
Campione	Piano	Resistenza a snervamento (Mpa)	Resistenza a rottura (Mpa)
PRB_T_01	Seminterrato	531.8	708.9
PRB_T_02	Seminterrato	520	701
PRB_T_01	Terra	485.9	651.9
PRB_T_02	Terra	499.3	686.4
PRB_T_01	Primo	535.7	779.6
PRB_T_02	Primo	500.9	695.8
PRB_T_01	Secondo	501.6	700.1
PRB_T_01	Sottotetto	427.5	747.8
PRB_T_02	Sottotetto	530.6	749.1
Media		**503.70**	**713.40**

Table 12: resistance test on rebars for beams

Test results for concrete cores and reinforcement bars in columns:

Prove su carote in c.a.			
Pilastri			
Carota	Piano	Resistenza provino (Mpa)	Resistenza cubica (Mpa)
PRC_P_01	Interrato	16.4	18
PRC_P_02	Interrato	15.2	16.6
PRC_S_01	Interrato	29.8	31.6
PRC_P_03	Terra	20.4	22.3
PRC_P_02	Terra	16.2	17.7
PRC_P_01	Terra	12.6	13.8
PRC_P_01	Primo	12.8	14.1
PRC_P_02	Primo	17.8	19.4
PRC_P_01	Secondo	26.9	28.9
PRC_P_02	Secondo	25.1	27.1
PRC_P_01	Sottotetto	21	22.9
PRC_P_02	Sottotetto	17.5	19.1
Media		**19.31**	**20.96**

Table 13: resistance test on reinforcement concrete cores for Columns

Prove sulle barre di armatura			
Pilastri			
Campione	Piano	Resistenza a snervamento (Mpa)	Resistenza a rottura (Mpa)
PRB_P_01	Seminterrato	544.3	760.8
PRB_P_02	Seminterrato	535.6	726.8
PRB_S_01	Seminterrato	518.4	676.1
PRB_P_01	Terra	514.7	701.9
PRB_P_02	Terra	508.4	706.5
PRB_P_01	Primo	535.7	779.6
PRB_P_02	Primo	530.1	752
PRB_P_01	Secondo	517.7	724.9
PRB_P_02	Secondo	485.7	694.7
PRB_P_01	Sottotetto	511.9	722.4
PRB_P_02	Sottotetto	520	727.5
Media		**520.23**	**724.84**

Table 14: resistance test on rebars for Columns

9 ACTION ON CONSTRUCTION

The actions considered for the evaluation of the seismic vulnerability of the building are as follows:

a) PERMANENT (G): Actions that act throughout the nominal life of the construction, whose intensity variation over time is so small and slow that they can be considered constant over time. These include:

Self-weight of all structural elements (G1);

Self-weight of all non-structural elements (G2);

b) VARIABLE (Q): Actions on the structure or structural element with instantaneous values that can vary significantly over time:

Long duration: Acts with significant intensity, even if not continuously, for a time not negligible relative to the nominal life of the structure;

Short duration: Actions that act for a brief period relative to the nominal life of the structure.

This category includes, among the most common, variable loads on floors and loads due to snow and wind:

CAT B, Offices:

o CAT B2 (Offices open to the public) qk=3.00 [kN/m2];

o CAT B (Common staircases, balconies, walkways) q_k=4.00 [kN/m2];

o CAT H (Roofs accessible only for maintenance) q_k=0.50 [kN/m2];

o SNOW q_k=0.80 [kN/m2];

9.1 c) SEISMIC (E): Actions deriving from earthquakes. Combination of actions

With reference to the elementary actions previously determined, the following load combinations have been considered:

Fundamental combination used for Ultimate Limit States(SLU):

$$\gamma_{G1} \cdot G_1 + \gamma_{G2} \cdot G_2 + \gamma_P \cdot P + \gamma_{Q1} \cdot Q_{k1} + \gamma_{Q2} \cdot \psi_{02} \cdot Q_{k2} + \gamma_{Q3} \cdot \psi_{03} \cdot Q_{k3} + \ldots$$

Where:

G_1 = value of permanent actions

G_2 = value of non-structural permanent actions

P = Value of pre-stressing actions

Q_{k1} = characteristic value of the basic variable action for each combination

Q_{ki} = characteristic value of the i-th variable action

γ_G = partial coefficient = 1.3 (1.0 if its contribution increases safety)

γ_P = partial coefficient = 0.9 (1.2 if its contribution increases safety)

γ_Q = partial coefficient = 1.5(0.0 if its contribution increases safety)

ψ_{0i} = combination coefficient (Table 2.5.I of NTC2018)

Seismic combination for the Ultimate Limit State of life safeguarding (SLV):

The effects of seismic action will be evaluated considering the masses associated with the following gravitational loads:

$$F_d = E + G_k + P_k + [\ \sum_j (\ \psi_{2j}\, Q_{kj}\ )\ ]$$

Where:

F_d = Design value of each of the actions acting on the structure obtained from its characteristic value Fk as indicated in §2.3 of NTC2018.

E = Value of seismic action for the limit state under consideration.

G_k = Characteristic value of permanent actions.

P_k =Characteristic value of pre-stressing actions.

Q_{kj} = Characteristic values of variable actions, each independent of the others.

ψ_{2j} = Combination coefficient (Table 2.5.I of NTC2018)

Seismic combination for the Damage Limit State (SLD):

The seismic action, derived from the design spectrum for the damage limit state, was combined with the other actions using the following formula:

$F_d = E + G_k + P_k + [\ \sum_j (\ \psi_{2j}\, Q_{kj}\ )\ ]$

Where:

F_d = The design value of each action acting on the structure is obtained from its characteristic value \(F_k \) as indicated in §2.3 of the NTC2018.

E =the seismic action value for the limit state under examination

G_k =characteristic value of permanent actions

P_k =characteristic value of pre-stressing actions

Q_{kj} = characteristic values of independent variable actions

ψ_{2j} =combination coefficient (Table 2.5.I of NTC2018)

Combinations for Serviceability Limit States (SLE):

$G_1 + G_2 + P + Q_{k1} + \psi_{02}\, Q_{k2} + \psi_{03}\, Q_{k3} + \ldots$ rare combination

$G_1 + G_2 + P + \psi_{11} Q_{k1} + \psi_{22} Q_{k2} + \psi_{23} Q_{k3} + \ldots$ frequent combination

$G_1 + G_2 + P + \psi_{21} Q_{k1} + \psi_{22} Q_{k2} + \psi_{23} Q_{k3} + \ldots$ quasi-permanent combination

Where:

G_1 = value of the permanent actions due to the self-weight of all structural elements.

G_2 = value of the permanent actions due to the self-weight of all non-structural elements.

P = value of the pre-stressing actions.

Q_{k1} = characteristic value of the base variable action for each combination

Q_{ki} = characteristic value of the i-th variable action.

ψ_{1i} = coefficient used to define the allowable action values at the 0.95 fractiles of the distributions of instantaneous values.

ψ_{2i} =coefficient used to define the quasi-permanent values of the allowable actions at the mean values of the distributions of instantaneous values.

		Coefficiente γ_F	EQU	A1	A2
Carichi permanenti G_1	Favorevoli	γ_{G1}	0,9	1,0	1,0
	Sfavorevoli		1,1	1,3	1,0
Carichi permanenti non strutturali G_2[(1)]	Favorevoli	γ_{G2}	0,8	0,8	0,8
	Sfavorevoli		1,5	1,5	1,3
Azioni variabili Q	Favorevoli	γ_{Qi}	0,0	0,0	0,0
	Sfavorevoli		1,5	1,5	1,3

[(1)] Nel caso in cui l'intensità dei carichi permanenti non strutturali o di una parte di essi (ad es. carichi permanenti portati) sia ben definita in fase di progetto, per detti carichi o per la parte di essi nota si potranno adottare gli stessi coefficienti parziali validi per le azioni permanenti.

Table 19:Partial coefficient

Categoria/Azione variabile	ψ_{0j}	ψ_{1j}	ψ_{2j}
Categoria A - Ambienti ad uso residenziale	0,7	0,5	0,3
Categoria B - Uffici	0,7	0,5	0,3
Categoria C - Ambienti suscettibili di affollamento	0,7	0,7	0,6
Categoria D - Ambienti ad uso commerciale	0,7	0,7	0,6
Categoria E – Aree per immagazzinamento, uso commerciale e uso industriale Biblioteche, archivi, magazzini e ambienti ad uso industriale	1,0	0,9	0,8
Categoria F - Rimesse , parcheggi ed aree per il traffico di veicoli (per autoveicoli di peso ≤ 30 kN)	0,7	0,7	0,6

Categoria G – Rimesse, parcheggi ed aree per il traffico di veicoli (per autoveicoli di peso > 30 kN)	0,7	0,5	0,3
Categoria H - Coperture accessibili per sola manutenzione	0,0	0,0	0,0
Categoria I – Coperture praticabili	da valutarsi caso per caso		
Categoria K – Coperture per usi speciali (impianti, eliporti, ...)			
Vento	0,6	0,2	0,0
Neve (a quota ≤ 1000 m s.l.m.)	0,5	0,2	0,0
Neve (a quota > 1000 m s.l.m.)	0,7	0,5	0,2
Variazioni termiche	0,6	0,5	0,0

Table 20:Seismic coefficients

9.2 <u>Load Analysis</u>

Classification of the actions by variation in Time.

permanent action (G1, 2) :

Actions acting throughout the design working life of the building, the variation in intensity of which

In the meantime, the time is very slow and modest.

e.g.:

- self-weight (dead loads), carried permanent loads
- prestressing
- Settlement
- earth and water pressure...

 variable action (Q):

For this reason, the variation in magnitude with time is neither negligible nor monotonic.

e.g.:

- imposed loads on building floors and roofs
- wind and snow loads
- temperature variations

 accidental action (A):

usually, of short duration, that is unlikely to occur with a significant magnitude on a given structure

during the design working life, but its consequences might be catastrophic.

e.g.:

- earthquakes and related seismic actions,
- fires, explosions,
- Impacts.

Structural permanent loads (Dead loads) – $g_{1,k}$

The permanent gravitational actions associated with the weights of structural materials are derived from the geometric dimensions and by the weights of the unit of volume of materials from which the structural parts of the construction are made.

MATERIALI	PESO UNITÀ DI VOLUME [kN/m³]
Calcestruzzi cementizi e malte	
Calcestruzzo ordinario	24,0
Calcestruzzo armato (e/o precompresso)	25,0
Calcestruzzi "leggeri": da determinarsi caso per caso	14,0 ÷ 20,0
Calcestruzzi "pesanti": da determinarsi caso per caso	28,0 ÷ 50,0
Malta di calce	18,0
Malta di cemento	21,0
Calce in polvere	10,0
Cemento in polvere	14,0
Sabbia	17,0
Metalli e leghe	
Acciaio	78,5
Ghisa	72,5
Alluminio	27,0
Materiale lapideo	
Tufo vulcanico	17,0
Calcare compatto	26,0
Calcare tenero	22,0
Gesso	13,0
Granito	27,0
Laterizio (pieno)	18,0
Legnami	
Conifere e pioppo	4,0 ÷ 6,0
Latifoglie (escluso pioppo)	6,0 ÷ 8,0
Sostanze varie	
Acqua dolce (chiara)	9,81
Acqua di mare (chiara)	10,1
Carta	10,0
Vetro	25,0

Table 21:specific weight

Non-structural permanent loads (Carried permanent) – $g_{2,k}$

Non-structural permanent loads shall be considered to be loads present on the building during its normal operation, such as those relating to external infills, internal partitions, lightweight concrete layers, insulation, pavements, plasters, false ceilings, systems and more, although in some cases, it is necessary to consider transitional situations in which they are not present.

The proper weights of non-structural materials are derived from geometric dimensions and the weights of the unit of volume of materials from which the non-structural parts of the construction is made.

In principle, in the presence of floor systems, non-structural permanent loads may be assumed as uniformly distributed.

In particular, partitions and light installations in residential and office buildings may generally be assumed as equivalent distributed loads, provided that the floors have adequate transversal distribution capacity.

Below are the different types of flooring and their corresponding load analyses used in the calculation model:

Set of equivalent distributed loads g_2 for internal partitions for building.

Internal partition elements with :

$G_2 \leq 1{,}00$ KN/m;	$g_2=0{,}40$ KN/m^2;
$1{,}00 \leq G_2 \leq 2{,}00$ KN/m;	$g_2=0{,}80$ KN/m^2;
$2{,}00 \leq G_2 \leq 3{,}00$ KN/m;	$g_2=1{,}20$ KN/m^2;
$3{,}00 \leq G_2 \leq 4{,}00$ KN/m;	$g_2=1{,}60$ KN/m^2;
$4{,}00 \leq G_2 \leq 5{,}00$ KN/m;	$g_2=2{,}00$ KN/m^2;

For the floors of residential and office buildings, the weight of the internal partition elements may be assumed as permanent load evenly distributed $g_{2,k}$, provided that the appropriate construction measures are taken to ensure adequate transverse load repartition.

Internal partitions with G2 > 5 kn/m should be considered in their actual position.

Imposed loads on floor systems – $q_{1,k}$

The imposed loads ("sovraccarichi") include loads related to the intended category of use of the building.

The representative models of such actions can consist of:

- vertical uniformly distributed loads q_k
- vertical concentrated loads Q_k
- horizontal uniformly distributed loads H_k

Cat.	Ambienti	q_k [kN/m²]	Q_k [kN]	H_k [kN/m]
A	**Ambienti ad uso residenziale**			
	Aree per attività domestiche e residenziali; sono compresi in questa categoria i locali di abitazione e relativi servizi, gli alberghi (ad esclusione delle aree soggette ad affollamento), camere di degenza di ospedali	2,00	2,00	1,00
	Scale comuni, balconi, ballatoi	4,00	4,00	2,00
B	**Uffici**			
	Cat. B1 Uffici non aperti al pubblico	2,00	2,00	1,00
	Cat. B2 Uffici aperti al pubblico	3,00	2,00	1,00
	Scale comuni, balconi e ballatoi	4,00	4,00	2,00
C	**Ambienti suscettibili di affollamento**			
	Cat. C1 Aree con tavoli, quali scuole, caffè, ristoranti, sale per banchetti, lettura e ricevimento	3,00	3,00	1,00
	Cat. C2 Aree con posti a sedere fissi, quali chiese, teatri, cinema, sale per conferenze e attesa, aule universitarie e aule magne	4,00	4,00	2,00
	Cat. C3 Ambienti privi di ostacoli al movimento delle persone, quali musei, sale per esposizioni, aree d'accesso a uffici, ad alberghi e ospedali, ad atri di stazioni ferroviarie	5,00	5,00	3,00
	Cat. C4. Aree con possibile svolgimento di attività fisiche, quali sale da ballo, palestre, palcoscenici.	5,00	5,00	3,00
	Cat. C5. Aree suscettibili di grandi affollamenti, quali edifici per eventi pubblici, sale da concerto, palazzetti per lo sport e relative tribune, gradinate e piattaforme ferroviarie.	5,00	5,00	3,00
	Scale comuni, balconi e ballatoi	Secondo categoria d'uso servita, con le seguenti limitazioni		
		≥ 4,00	≥ 4,00	≥ 2,00
Cat.	**Ambienti**	**q_k [kN/m²]**	**Q_k [kN]**	**H_k [kN/m]**
D	**Ambienti ad uso commerciale**			
	Cat. D1 Negozi	4,00	4,00	2,00
	Cat. D2 Centri commerciali, mercati, grandi magazzini	5,00	5,00	2,00
	Scale comuni, balconi e ballatoi	Secondo categoria d'uso servita		
E	**Aree per immagazzinamento e uso commerciale ed uso industriale**			
	Cat. E1 Aree per accumulo di merci e relative aree d'accesso, quali biblioteche, archivi, magazzini, depositi, laboratori manifatturieri	≥ 6,00	7,00	1,00*
	Cat. E2 Ambienti ad uso industriale	da valutarsi caso per caso		
F-G	**Rimesse e aree per traffico di veicoli (esclusi i ponti)**			
	Cat. F Rimesse, aree per traffico, parcheggio e sosta di veicoli leggeri (peso a pieno carico fino a 30 kN)	2,50	2 x 10,00	1,00**
	Cat. G Aree per traffico e parcheggio di veicoli medi (peso a pieno carico compreso fra 30 kN e 160 kN), quali rampe d'accesso, zone di carico e scarico merci.	da valutarsi caso per caso e comunque non minori di		
		5,00	2 x 50,00	1,00**
H-I-K	**Coperture**			
	Cat. H Coperture accessibili per sola manutenzione e riparazione	0,50	1,20	1,00
	Cat. I Coperture praticabili di ambienti di categoria d'uso compresa fra A e D	secondo categorie di appartenenza		
	Cat. K Coperture per usi speciali, quali impianti, eliporti.	da valutarsi caso per caso		

* non comprende le azioni orizzontali eventualmente esercitate dai materiali immagazzinati.

** per i soli parapetti o partizioni nelle zone pedonali. Le azioni sulle barriere esercitate dagli automezzi dovranno essere valutate caso per caso.

Table 22:Imposed loads based on building function

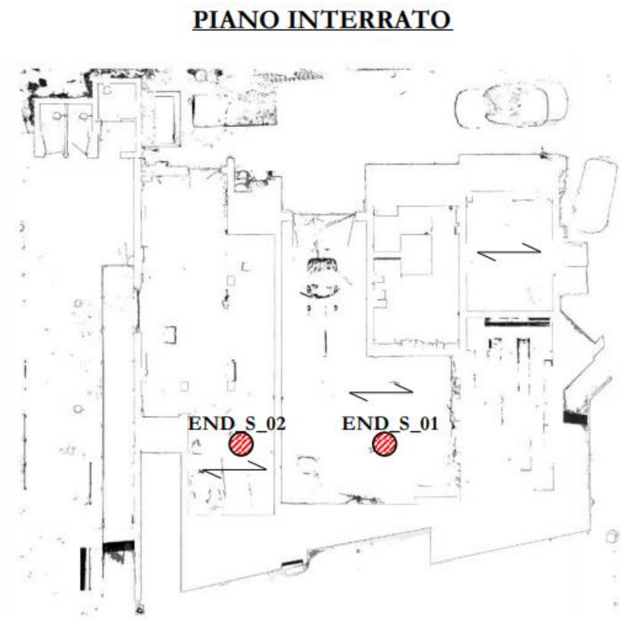


Figure 33- ceiling direction_Underground Floor

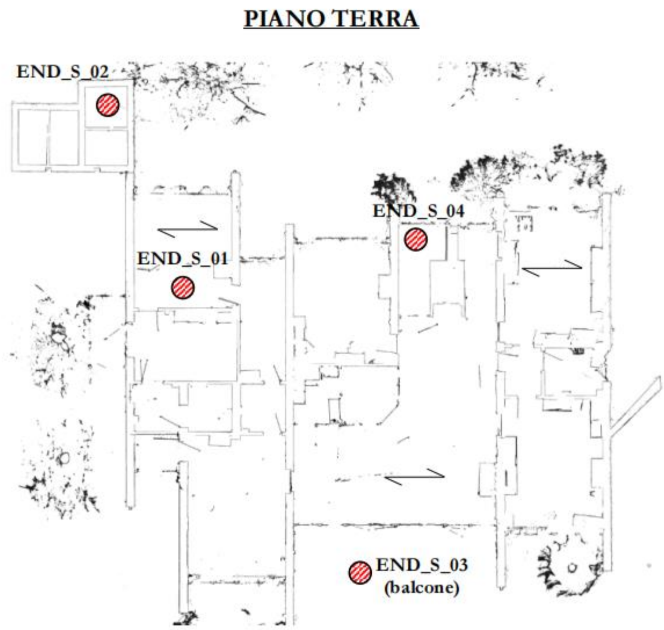


Figure 34- ceiling direction_Ground Floor

PIANO PRIMO

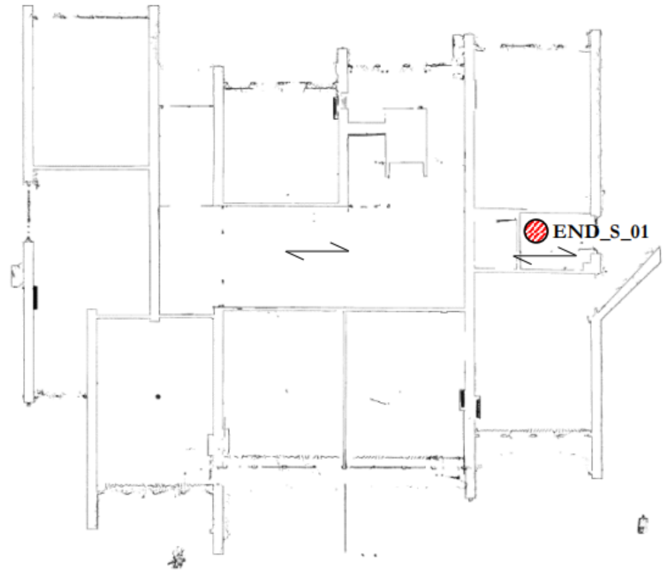


Figure 35- ceiling direction_First Floor

PIANO SECONDO

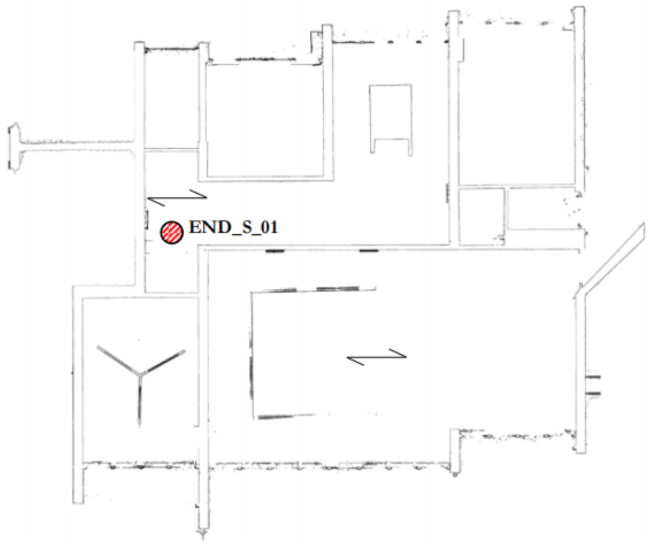


Figure 36- ceiling direction_Second Floor

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Travetti in c.a.p. b= 12 cm
- Armatura travetto:	Trefolo Ø 6 mm
- Interasse:	50 cm
Elemento di alleggerimento:	Coppia di Pignatte a "C" h 8+10 cm innestate tra di loro Camera d'aria 16 cm
Presenza caldana:	Si – 3 cm
Carichi permanenti:	Massetto in cls alleggerito con polistirolo >2,5 cm + piastrella.
Intonaco all'intradosso:	Si – spessore 1 cm

SCHEMA SOLAIO TIPO

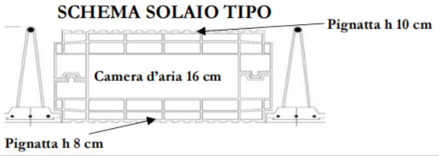
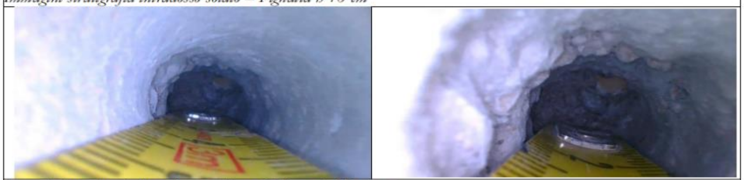


Table 23:Investigation results for first-floor structure of underground slab END_S_01

Immagini stratigrafia intradosso solaio – Pignatta h 8 cm

Immagini stratigrafia intradosso solaio – Camera d'aria 16 cm + Dettaglio pignatte a "C" innestate tra di loro

Immagini stratigrafia intradosso solaio – Pignatta h 10 cm



Immagini stratigrafia intradosso solaio – Caldana + Massetto

Documentazione fotografica solaio – Interasse 50 cm + Larghezza travetto 12 cm + Dettaglio trefolo Ø 6 mm

Figure 37-layers measurement _Underground ceiling (END_S_01)

SOLAIO DI PIANO- PIANO INTERRATO (TIPO 1)

PESO PROPRIO: G_1			**4.13 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.			4.13 kN/m^2

Sovraccarico PERMANENTE: G_2				**1.69 kN/m^2**
- Intonaco	1 cm			0.20 kN/m^2
- Massetto	3 cm	x	16 kN/m^3	0.48 kN/m^2
- Pavimento	1 cm			0.21 kN/m^2
- Tramezzi				0.80 kN/m^2

Sovraccarico VARIABILE - CATEGORIA B2 (Uffici aperti al pubblico): q_k	**3.00 kN/m^2**

SOLAIO DI PIANO						
Solaio laterocementizio con travetti in c.a.p. (34+4) cm						
Tipo di carico	Carico	kN/mc	b (m)	h (m)	i (m)	kN/mq
G1	Laterizi	8.00	0.38	0.18	0.50	1.09
	Travetti	25.00	0.12	0.34	0.50	2.04
	Soletta	25.00	1.00	0.04	1.00	1.00
TOT.						4.13 kN/mq

Table 24:permanent action calculation for first-floor structure of underground slab END_S_01

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Travetti in c.a.p. b= 12 cm
- Armatura travetto:	Trefolo Ø 4 mm
- Interasse:	50 cm
Elemento di alleggerimento:	Pignatta h 14
Presenza caldana:	Si – 3,5 cm
Carichi permanenti:	Massetto in cls alleggerito con polistirolo 8 cm + piastrella.
Intonaco all'intradosso:	Si – spessore 1 cm

SCHEMA SOLAIO TIPO

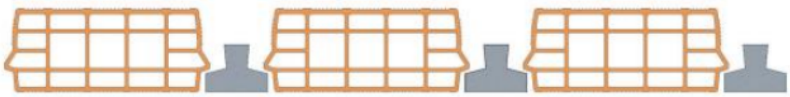
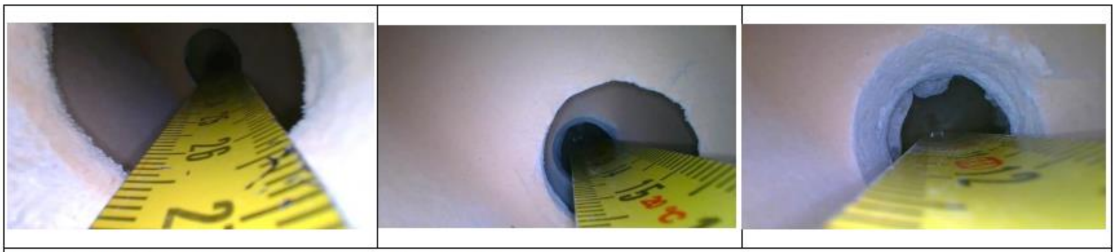
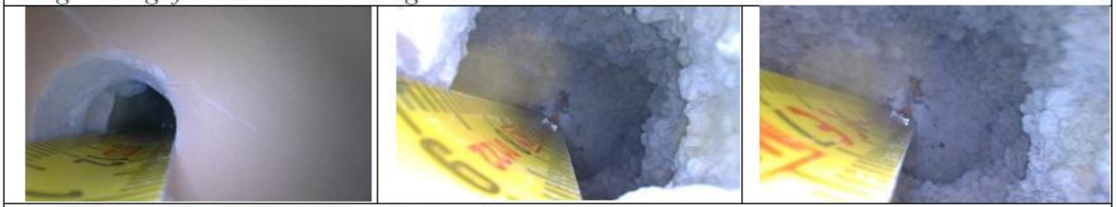


Table 25:Investigation results for first-floor structure of underground slab END_S_02



Immagini stratigrafia intradosso solaio – Pignatta h 14 cm



Immagini stratigrafia intradosso solaio – Caldana 3,5 + Massetto 8 cm

Documentazione fotografica solaio – Interasse 50 cm + Larghezza travetto 12 cm + Dettaglio trefolo Ø 4 mm

Figure 38-layers measurement _Underground ceiling (END_S_02)

SOLAIO DI PIANO - PIANO INTERRATO (TIPO 2)

PESO PROPRIO: G_1				**2.81 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.				2.81 kN/m^2
Sovraccarico PERMANENTE: G_2				**2.68 kN/m^2**
- Intonaco	1 cm			0.20 kN/m^2
- Massetto	8 cm	x	16 kN/m^3	1.28 kN/m^2
- Pavimento	2 cm			0.40 kN/m^2
- Tramezzi				0.80 kN/m^2
Sovraccarico VARIABILE - CATEGORIA B2 (Uffici aperti al pubblico): q_k				**3.00 kN/m^2**

SOLAIO DI PIANO						
Solaio laterocementizio con travetti in c.a.p.						
Tipo di carico	**Carico**	**kN/mc**	**b (m)**	**h (m)**	**i (m)**	**kN/mq**
G1	Laterizi	8.00	0.38	0.14	0.50	0.85
	Travetti	25.00	0.12	0.18	0.50	1.08
	Soletta	25.00	1.00	0.04	1.00	0.88
TOT.						**2.81 kN/mq**

Table 26:permanent action calculation for first-floor structure of underground slab END_S_02

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Travetti in c.a.p. b= 12 cm
- Armatura travetto:	Indagine non eseguita
- Interasse:	50 cm – Passo travetti rilevato con Ferroscan Hilti
Elemento di alleggerimento:	Pignatta h 16
Presenza caldana:	Si – 3 cm
Carichi permanenti:	Massetto in cls alleggerito con polistirolo 10 cm + piastrella.
Intonaco all'intradosso:	Si – spessore 0,5 cm

SCHEMA SOLAIO TIPO

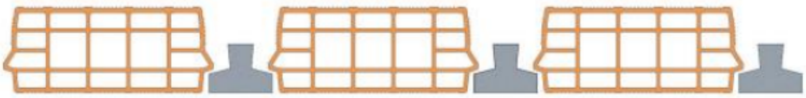
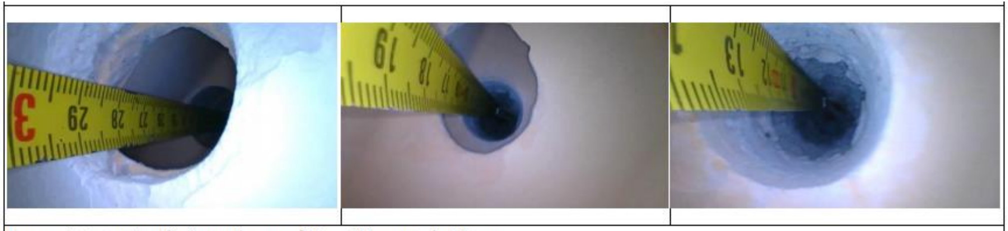
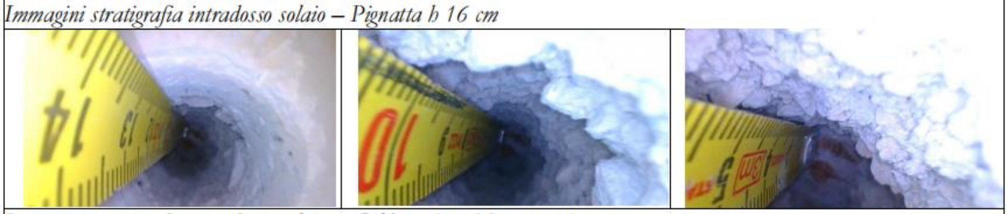


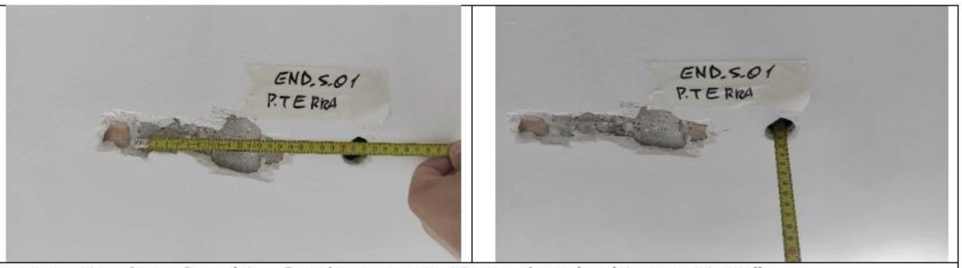
Table 27:Investigation results for second-floor structure of Ground floor slab END_S_01



Immagini stratigrafia intradosso solaio – Pignatta h 16 cm



Immagini stratigrafia intradosso solaio – Caldana 3 + Massetto 10 cm



Documentazione fotografica solaio – Larghezza travetto 12 cm + h totale solaio senza piastrella

Figure 39-layers measurement _Ground Floor ceiling (END_S_01)

SOLAIO DI PIANO - PIANO TERRA (TIPO 1)

PESO PROPRIO: G_1				**3.17 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.				3.17 kN/m^2
Sovraccarico PERMANENTE: G_2				**2.95 kN/m^2**
- Intonaco	0.5 cm			0.15 kN/m^2
- Massetto	10 cm	x	16 kN/m^3	1.60 kN/m^2
- Pavimento	2 cm			0.40 kN/m^2
- Tramezzi				0.80 kN/m^2
Sovraccarico VARIABILE - CATEGORIA B2 (Uffici aperti al pubblico): q_k				**3.00 kN/m^2**

SOLAIO DI PIANO						
Solaio laterocementizio con travetti in c.a.p. (16+4) cm						
Tipo di carico	**Carico**	**kN/mc**	**b (m)**	**h (m)**	**i (m)**	**kN/mq**
G1	Laterizi	8.00	0.38	0.16	0.50	0.97
	Travetti	25.00	0.12	0.20	0.50	1.20
	Soletta	25.00	1.00	0.04	1.00	1.00
TOT.						**3.17 kN/mq**

Table 28: permanent action calculation for the second-floor structure of Ground floor slab END_S_01

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Travetti in c.a.p. b= 12 cm
- Armatura travetto:	Indagine non eseguita
- Interasse:	50 cm
Elemento di alleggerimento:	Pignatta h 16
Presenza caldana:	No
Carichi permanenti:	Massetto in cls 6 cm + Guaina bituminosa sp= 0,5 cm + Tegole di copertura.
Intonaco all'intradosso:	Si – spessore 0,5 cm

SCHEMA SOLAIO TIPO

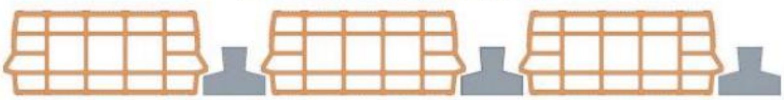
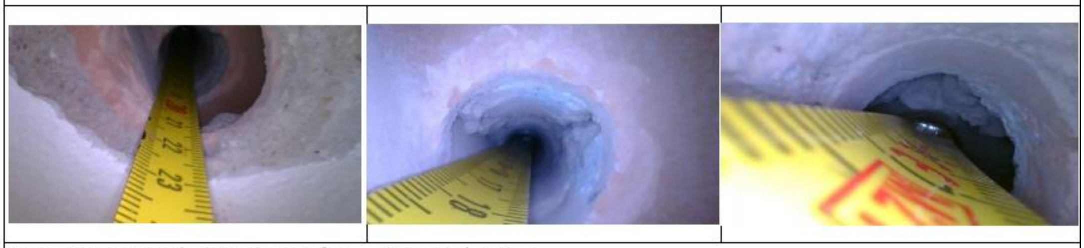


Table 29: Investigation results for second-floor structure of Ground floor slab END_S_02



Immagini stratigrafia intradosso solaio – Pignatta h 16 cm

Immagini stratigrafia intradosso solaio – Massetto 6 cm + Guaina bituminosa sp= 0,5 cm

Documentazione fotografica solaio – Interasse 50 cm + Larghezza travetto 12 cm

Figure 40-layers measurement _Ground Floor ceiling (END_S_02)

SOLAIO DI COPERTURA - PIANO TERRA (BAGNI ESTERNI)

PESO PROPRIO: G_1				**1.93 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.				1.93 kN/m^2

Sovraccarico PERMANENTE: G_2				**2.06 kN/m^2**
- Intonaco	0.5 cm			0.15 kN/m^2
- Massetto	6 cm	x	16 kN/m^3	0.96 kN/m^2
- Guaina bituminosa	0.5 cm	x	10.5 kN/m^3	0.05 kN/m^2
- Tramezzi				0.90 kN/m^2

Sovraccarico VARIABILE - CATEGORIA B2 (Uffici aperti al pubblico): q_k	**3.00 kN/m^2**
Sovraccarico VARIABILE - NEVE Q_k - COPERTURA A FALDE	**0.80 kN/m^2**

SOLAIO DI COPERTURA						
Solaio laterocementizio con travetti in c.a.p.						
Tipo di carico	**Carico**	**kN/mc**	**b (m)**	**h (m)**	**i (m)**	**kN/mq**
G1	Laterizi	8.00	0.38	0.16	0.50	0.97
	Travetti	25.00	0.12	0.16	0.50	0.96
TOT.					**1.93**	**kN/mq**

Table 30: permanent action calculation for the second-floor structure of Ground floor slab END_S_02

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Travetti in c.a.p. b= 12 cm
- Armatura travetto:	Indagine non eseguita
- Interasse:	50 cm – Passo travetti rilevato con Ferroscan Hilti
Elemento di alleggerimento:	Coppia di Pignatte a "C" innestate tra di loro con camera d'aria L'indagine è stata eseguita sulla spalla della pignatta ed è stata rilevata un'altezza di 16 cm a pignatta.
Presenza caldana:	No
Carichi permanenti:	Indagine non eseguita
Intonaco all'intradosso:	Si – spessore 1,5 cm

SCHEMA SOLAIO TIPO

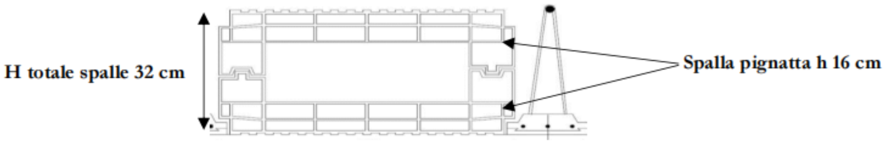
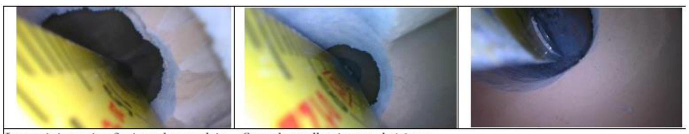


Table 31:Investigation results for second-floor structure of Ground floor slab END_S_03 (balcony)

Immagini stratigrafia intradosso solaio – Prima spalla pignatta h 16 cm
Nota: cerchiato in rosso dettaglio fine spalla prima pignatta – inizio spalla seconda pignatta



Immagini stratigrafia intradosso solaio – Seconda spalla pignatta h 16 cm

Spessore totale 43 cm + Piastrella

Figure 41-layers measurement _Ground Floor ceiling (END_S_03)

BALCONE - PIANO PRIMO

PESO PROPRIO: G_1					**4.87 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.					4.87 kN/m^2

Sovraccarico PERMANENTE: G_2				**1.58 kN/m^2**
- Intonaco	1.5 cm			0.30 kN/m^2
- Massetto	3 cm	x	16 kN/m^3	0.48 kN/m^2
- Pavimento	2 cm			0.40 kN/m^2
- Parapetto				0.40 kN/m^2

Sovraccarico VARIABILE - Scale comuni, balconi, ballatoi: Q_k	**4.00 kN/m^2**

BALCONE						
Solaio laterocementizio con travetti in c.a.p. (32+4) cm						
Tipo di carico	**Carico**	**kN/mc**	**b (m)**	**h (m)**	**i (m)**	**kN/mq**
G1	Laterizi	8.00	0.38	0.32	0.50	1.95
	Travetti	25.00	0.12	0.32	0.50	1.92
	Soletta	25.00	1.00	0.04	1.00	1.00
TOT.						**4.87 kN/mq**

Table 32:permanent action calculation for the second-floor structure of Ground floor slab END_S_03 (balcony)

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Travetti in c.a.p. b= 12 cm
- Armatura travetto:	Indagine non eseguita
- Interasse:	50 cm - Passo travetti rilevato con Ferroscan Hilti
Elemento di alleggerimento:	Pignatta h 12
Presenza caldana:	Si – 3 cm
Carichi permanenti:	Massetto in cls alleggerito con polistirolo >2 cm + piastrella
Intonaco all'intradosso:	Si – spessore 0,5 cm

SCHEMA SOLAIO TIPO

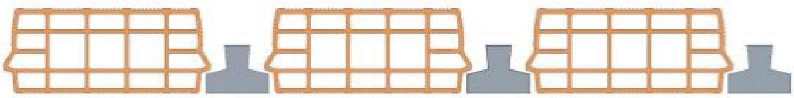


Table 33:Investigation results for third Structural floor- First-floor slab END_S_01.

Immagini stratigrafia intradosso solaio – Pignatta h 12 cm

Immagini stratigrafia intradosso solaio – Caldana 3 cm + massetto in cls alleggerito >2 cm

Documentazione fotografica solaio – Larghezza travetto 12 cm + h totale solaio senza piastrella

Figure 42-layers measurement _First Floor ceiling (END_S_01)

SOLAIO DI PIANO - PIANO PRIMO

PESO PROPRIO: G_1				**2.63 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.				2.63 kN/m^2
Sovraccarico PERMANENTE: G_2				**1.83 kN/m^2**
- Intonaco	0.5 cm			0.15 kN/m^2
- Massetto	3 cm	x	16 kN/m^3	0.48 kN/m^2
- Pavimento	2 cm			0.40 kN/m^2
- Tramezzi				0.80 kN/m^2
Sovraccarico VARIABILE - CATEGORIA B2 (Uffici aperti al pubblico): q_k				**3.00 kN/m^2**

SOLAIO DI PIANO						
Solaio laterocementizio con travetti in c.a.p. (15+4) cm						
Tipo di carico	**Carico**	**kN/mc**	**b (m)**	**h (m)**	**i (m)**	**kN/mq**
G1	Laterizi	8.00	0.38	0.12	0.50	0.73
	Travetti	25.00	0.12	0.15	0.50	0.90
	Soletta	25.00	1.00	0.04	1.00	1.00
TOT.						**2.63 kN/mq**

Table 34:permanent action calculation for third Structural floor- First-floor slab END_S_01

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Travetti in c.a.p. b= 12 cm
- Armatura travetto:	Indagine non eseguita
- Interasse:	50 cm
Elemento di alleggerimento:	Pignatta h 20
Presenza caldana:	Si – 6 cm
Carichi permanenti:	Massetto in cls alleggerito con polistirolo 5 cm.
Intonaco all'intradosso:	Si – spessore 1 cm
SCHEMA SOLAIO TIPO	

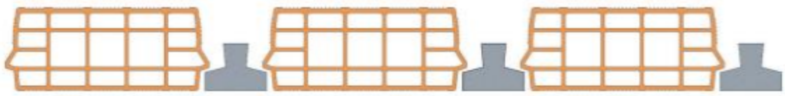


Table 35:Investigation results for fourth structural floor- Sec floor slab END_S_01

Documentazione fotografica solaio – h pignatta 20 cm + h caldana 6 cm

Documentazione fotografica solaio – h massetto in cls 5 cm

Documentazione fotografica solaio–interasse travetti 50 cm

Figure 43-layers measurement _Second Floor ceiling (END_S_01)

SOLAIO DI PIANO - PIANO SECONDO

PESO PROPRIO: G_1				**4.28 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.				4.28 kN/m^2

Sovraccarico PERMANENTE: G_2				**2.20 kN/m^2**
- Intonaco	1 cm			0.20 kN/m^2
- Massetto	5 cm	x	16 kN/m^3	0.80 kN/m^2
- Pavimento	2 cm			0.40 kN/m^2
- Tramezzi				0.80 kN/m^2

Sovraccarico VARIABILE - CATEGORIA B2 (Uffici aperti al pubblico): q_k	**3.00 kN/m^2**

SOLAIO DI PIANO						
Solaio laterocementizio con travetti in c.a.p. (26+6)						
Tipo di carico	**Carico**	**kN/mc**	**b (m)**	**h (m)**	**i (m)**	**kN/mq**
G1	Laterizi	8.00	0.38	0.20	0.50	1.22
	Travetti	25.00	0.12	0.26	0.50	1.56
	Soletta	25.00	1.00	0.06	1.00	1.50
TOT.						**4.28 kN/mq**

Table 36:permanent action calculation for Fourth Structural floor- Sec floor slab END_S_01

SCHEDA SOLAIO	
Tipo di solaio:	Solaio in laterocemento con travetti in c.a.p.
Elementi resistenti:	Coppia di travetti in c.a.p. b= 12 cm disposti in maniera alternata a coppie e singolo distribuiti con uniformità per tutto il solaio
- Armatura travetto:	Indagine non eseguita
- Interasse:	56 cm
Elemento di alleggerimento:	Pignatta h 20
Presenza caldana:	Si – 4 cm
Carichi permanenti:	Tegole di copertura
Intonaco all'intradosso:	No

Table 37:Investigation results for fifth Structural floor- attic slab END_S_01

Documentazione fotografica solaio – h pignatta 20 cm + h caldana 6 cm

Documentazione fotografica solaio–interasse travetti 56 cm

Documentazione fotografica solaio – Dettaglio distribuzione travetti

Figure 44-layers measurement _Attic (END_S_01)

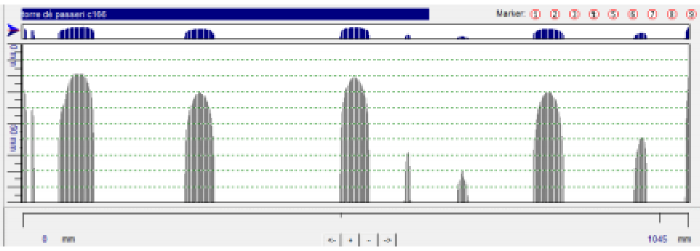
SOLAIO DI COPERTURA - LATEROCEMENTO

PESO PROPRIO: G_1				**3.37 kN/m^2**
Solaio laterocementizio con travetti in c.a.p.				3.37 kN/m^2
Sovraccarico PERMANENTE: G_2				**1.16 kN/m^2**
- Intonaco	1.5 cm			0.30 kN/m^2
- Massetto	4 cm	x	14 kN/m^3	0.56 kN/m^2
- Guaina				0.10 kN/m^2
- Impianti				0.20
Sovraccarico VARIABILE - CATEGORIA H: (Coperture accessibili per sola manutenzione) Q_k				**0.50 kN/m^2**
Sovraccarico VARIABILE - NEVE Q_k - COPERTURA A FALDE				**0.80 kN/m^2**

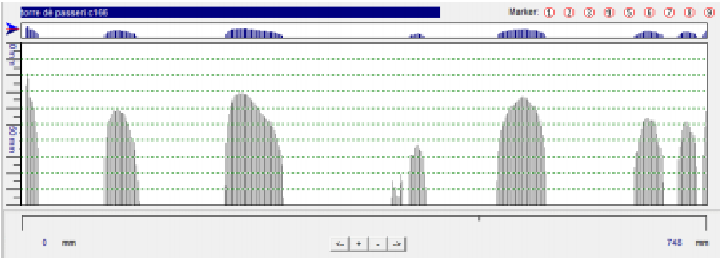
SOLAIO DI COPERTURA						
Solaio laterocementizio con travetti in c.a.p.						
Tipo di carico	**Carico**	**kN/mc**	**b (m)**	**h (m)**	**i (m)**	**kN/mq**
G1	Laterizi	8.00	0.38	0.20	0.56	1.09
	Travetti	25.00	0.12	0.24	0.56	1.29
	Soletta	25.00	1.00	0.04	1.00	1.00
TOT.						**3.37 kN/mq**

Table 38:permanent action calculation for fifth Structural floor- attic slab END_S_01

SCHEDA SOLAIO	
Tipo di solaio:	Pianerottolo intermedio con soletta in c.a. h 20 cm

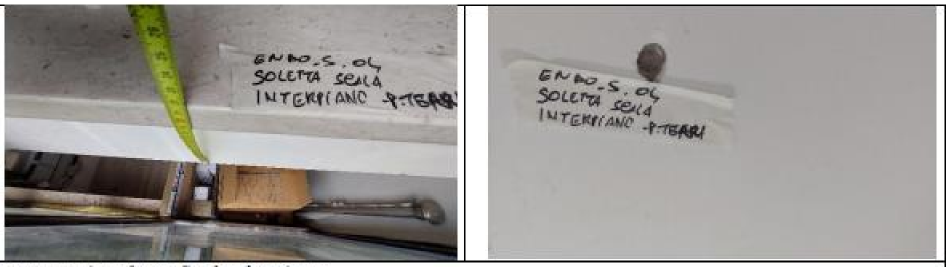


Armatura secondaria
pianerottolo intermedio con soletta in c.a.
Rilevate n°4 barre di armatura - Sansionati 104 cm



Armatura principale
pianerottolo intermedio con soletta in c.a.
Rilevate n°5 barre di armatura - Sansionati 75 cm

Table 39:Investigation results for Second Structural floor- Stairs END_S_04



Documentazione fotografica h soletta in c.a.

Figure 45-layers measurement _Ground floor stairs(END_S_04)

VANO SCALA					
PESO PROPRIO: G_1				**5.00**	**kN/m^2**
- Soletta in cemento armato	20 cm	x	25 kN/m^3	5.00	kN/m^2
Sovraccarico PERMANENTE: G_2				**1.46**	**kN/m^2**
- Intonaco	1.5 cm			0.30	kN/m^2
- Massetto	5 cm	x	16 kN/m^3	0.80	kN/m^2
- Pavimento	2 cm	x	18 kN/m^3	0.36	kN/m^2
Sovraccarico VARIABILE - Scale comuni, balconi, ballatoi: Q_k				**4.00**	**kN/m^2**

Table 40:permanent action calculation for Second Structural floor- Stairs END_S_04

Direction of Slabs

The direction of the slab in the first structural floor -Slab of the underground floor is shown in the figures below :

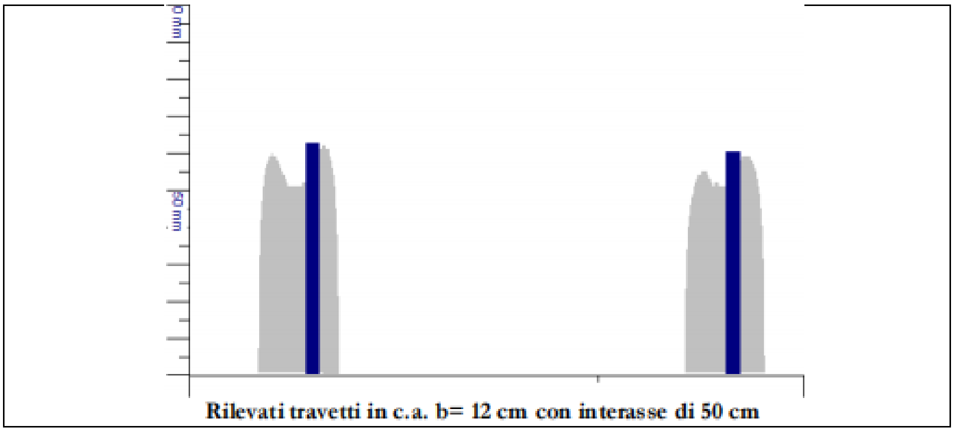


Table 41:Investigation results for first Structural floor-underground floor ORD_S_01

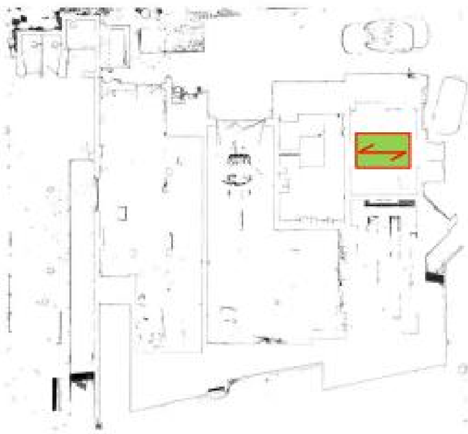


Figure 46-Slab direction First structural floor (ORD_S_01)

The direction of the slab in the second structural floor -Slab of Ground floor are shown in the figures below :

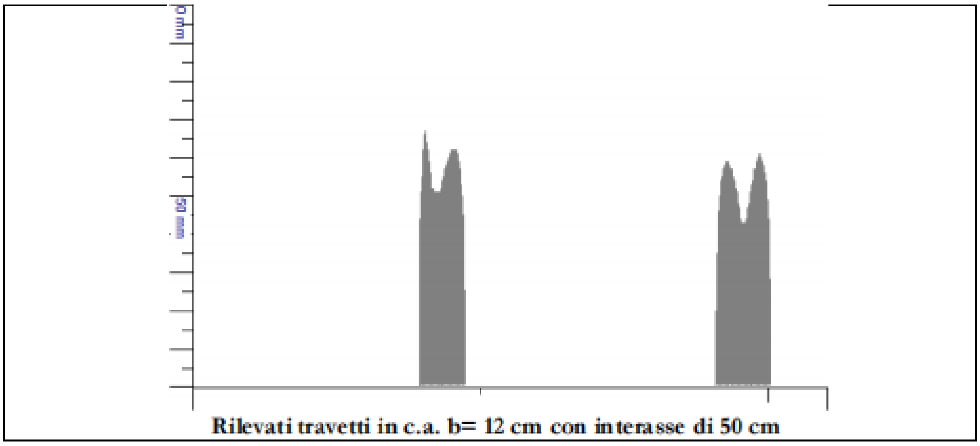


Table 42:Investigation results for Second Structural floor-Ground floor ORD_S_01

Figure 47-Slab direction _Second structural floor (ORD_S_01)

The direction of the slab on the third structural floor and the slab on the First floor are shown in the figures below :

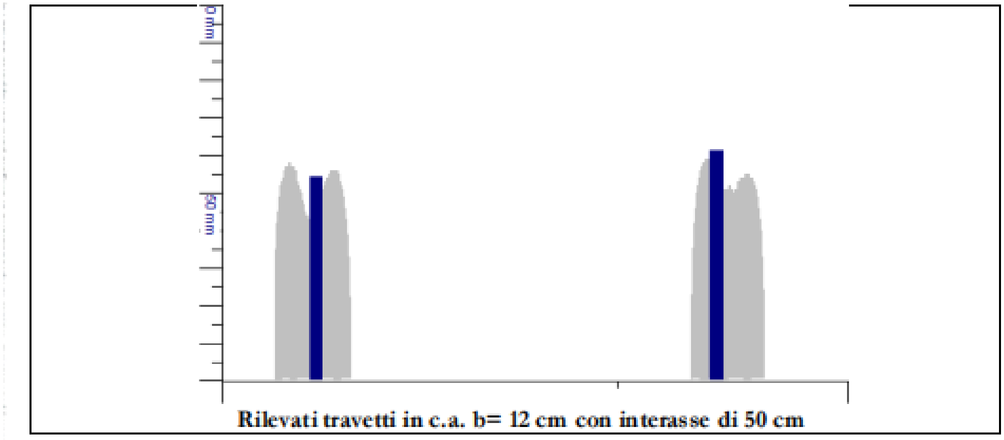


Table 43:Investigation results for third Structural floor-First floor ORD_S_01.

Figure 48-Slab direction _Third structural floor (ORD_S_01)

The direction of the slab in the fourth structural floor -Slab of the second floor is shown in the figures below :

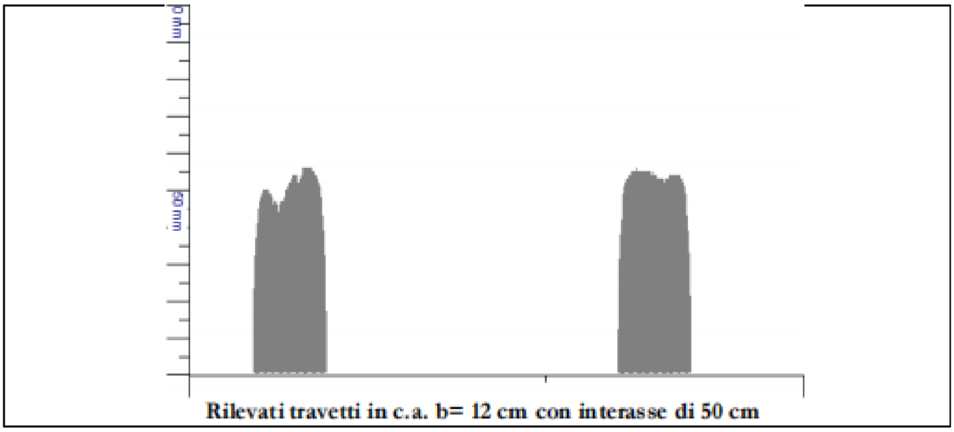


Table 44:Investigation results for the fourth Structural second floor ORD_S_01.

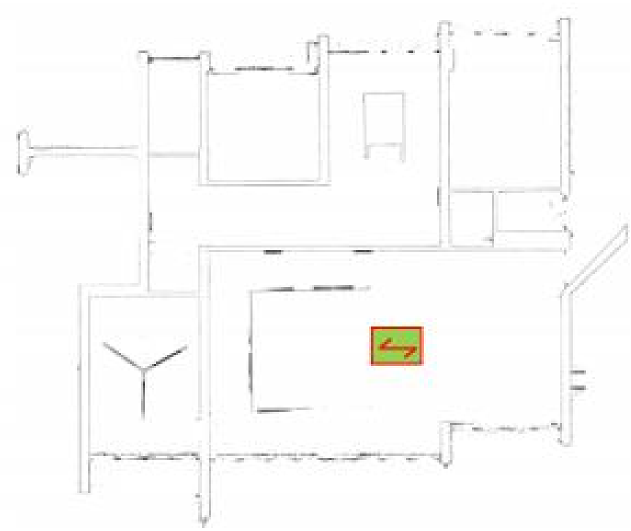


Figure 49-Slab direction _forth structural floor (ORD_S_01)

A load of Walls

For considering the wall loads, it is essential to do an investigation on different floors; the location of the investigations, results, and calculations are reported below:

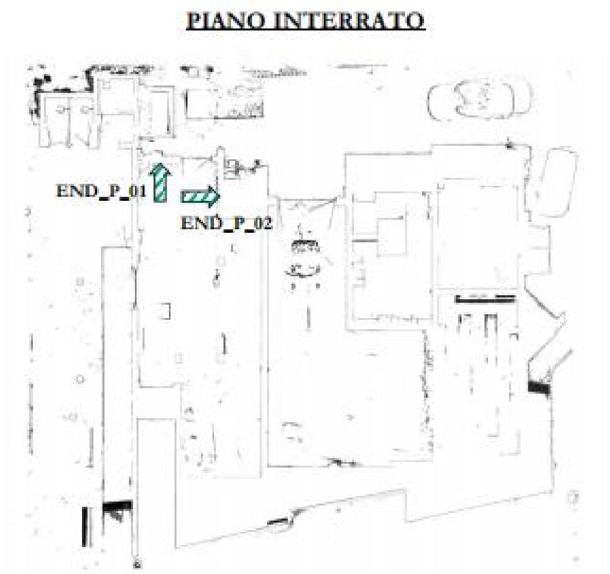


Figure50-Location of investigated walls in the underground floor (END_P_01, END_P_02)

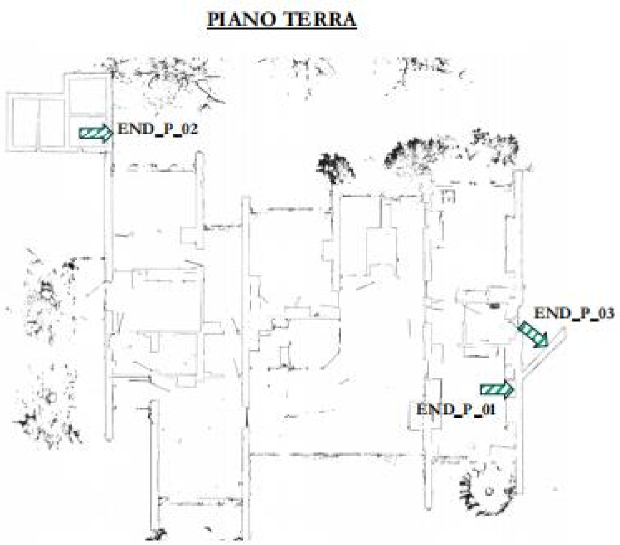


Figure 51-Location of investigated walls in the underground floor (END_P_01, END_P_02, END_P_03)

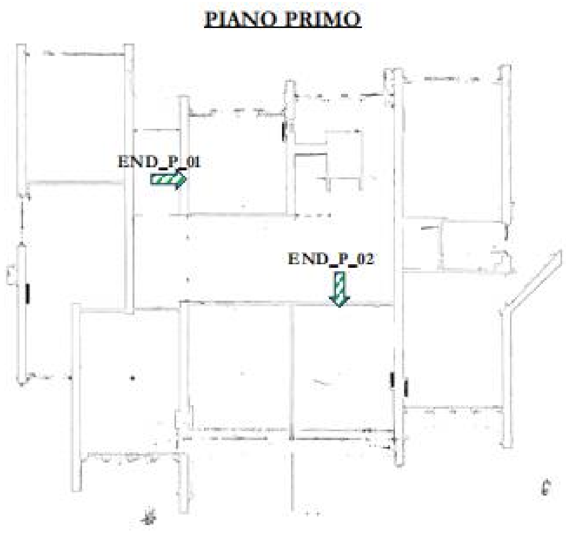


Figure 52-Location of investigated walls on the first floor (END_P_01, END_P_02)

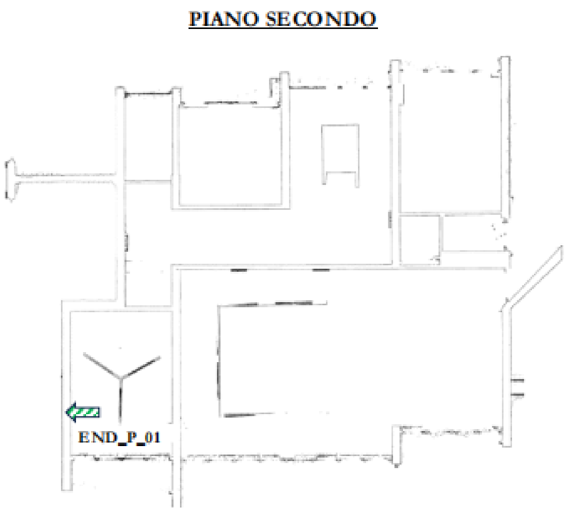


Figure 53-Location of investigated walls on the second floor (END_P_01)

Figure 54-Location of investigated walls in Attic(END_P_01)

RILIEVO DEI DETTAGLI COSTRUTTIVI DELLE PARETI VERTICALI MEDIANTE INDAGINE ENDOSCOPICA

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END_P_01: Piano interrato

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_01	Piano interrato	1,5	8	---	>30	n.i.	42	40,5
	Indagine eseguita da interno verso esterno con perforazione parziale sullo spessore della parete. Rilevato primo paramento in laterizio forato dello spessore di 8 cm, a seguire un secondo paramento in laterizio forato >30 cm							

Immagine stratigrafica inizio primo paramento

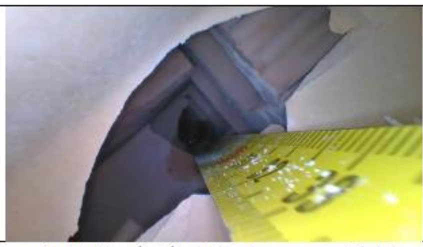

Immagine stratigrafica fine primo paramento + inizio secondo paramento

Immagine stratigrafica secondo paramento

Figure 55-layers measurement for walls _Underground floor(END_P_01)

TAMPONATURA TIPO 1 - PIANO INTERRATO				
PESO PROPRIO: G_1				**4.40 kN/m^2**
- Mattoni forati in laterizio	8 cm	x	11 kN/m^3	0.88 kN/m^2
- Mattoni forati in laterizio	32 cm	x	11 kN/m^3	3.52 kN/m^2
Sovraccarico PERMANENTE: G_2				**0.30 kN/m^2**
- Intonaco interno	1.5 cm			0.30 kN/m^2

Table 45::permanent action calculation for Walls in Underground floor- END_p_01

RILIEVO DEI DETTAGLI COSTRUTTIVI DELLE PARETI VERTICALI MEDIANTE INDAGINE ENDOSCOPICA

Sede Municipale – Piazza 6 Aprile 2009 – 65029 Torre De' Passeri (PE)

END_P_02: Piano interrato

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_02	Piano interrato	1	8	10	8	1	28	28
	Indagine con perforazione totale sullo spessore della parete. Rilevato primo paramento in laterizio forato dello spessore di 8 cm, intercapedine 10 cm, ed infine un secondo paramento in laterizio forato dello spessore di 8 cm.							

Immagine stratigrafica inizio primo paramento

Immagine stratigrafica fine primo paramento

Immagine stratigrafica intercapedine + inizio secondo paramento

Immagine stratigrafica fine secondo paramento

Figure 56-layers measurement _Underground floor(END_P_02)

TAMPONATURA TIPO 2 - PIANO INTERRATO				
PESO PROPRIO: G_1				**1.76 kN/m^2**
- Mattoni forati in laterizio	8 cm	x	11 kN/m^3	0.88 kN/m^2
- Mattoni forati in laterizio	8 cm	x	11 kN/m^3	0.88 kN/m^2
Sovraccarico PERMANENTE: G_2				**0.20 kN/m^2**
- Intonaco interno	1 cm			0.20 kN/m^2
- Intonaco esterno	1 cm			0.20 kN/m^2

Table 46::permanent action calculation for Walls in Underground floor- END_P_02

RILIEVO DEI DETTAGLI COSTRUTTIVI DELLE PARETI VERTICALI MEDIANTE INDAGINE ENDOSCOPICA

Sede Municipale – Piazza 6 Aprile 2009 – 65029 Torre De' Passeri (PE)

END_P_01: Piano terra

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_01	Piano terra	2	8	10	12	2	34	34
	Indagine da interno verso esterno con perforazione totale sullo spessore della parete. Rilevato primo paramento in laterizio forato dello spessore di 8 cm, intercapedine 10 cm, ed infine un secondo paramento in laterizio forato dello spessore di 12 cm.							

Immagine stratigrafica inizio primo paramento

Immagine stratigrafica fine primo paramento + inizio intercapedine

Immagine stratigrafica fine intercapedine + inizio secondo paramento

Immagine stratigrafica fine secondo paramento

Figure 57-layers measurement _Ground floor(END_P_01)

TAMPONATURA TIPO 3 - PIANO TERRA

PESO PROPRIO: G_1				**2.20 kN/m^2**
- Mattoni forati in laterizio	8 cm	x	11 kN/m^3	0.88 kN/m^2
- Mattoni forati in laterizio	12 cm	x	11 kN/m^3	1.32 kN/m^2

Sovraccarico PERMANENTE: G_2		**0.40 kN/m^2**
- Intonaco interno	2 cm	0.40 kN/m^2

Table 47:permanent action calculation for Walls on Ground floor- END_P_01

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END_P_02: Piano terra

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_02	Piano terra	1	24	---	---	1	26	26
	Indagine da interno verso esterno con perforazione totale sullo spessore della parete. Rilevato unico paramento in laterizio forato dello spessore di 24 cm.							

Immagine stratigrafica inizio paramento

Immagine stratigrafica paramento

Immagine stratigrafica fine paramento

Figure 58-layers measurement _Ground floor(END_P_02)

TAMPONATURA TIPO 4 - PIANO TERRA				
PESO PROPRIO: G_1				**2.64 kN/m^2**
- Mattoni forati in laterizio	24 cm	x	11 kN/m^3	2.64 kN/m^2
Sovraccarico PERMANENTE: G_2				**0.40 kN/m^2**
- Intonaco interno	1 cm			0.20 kN/m^2
- Intonaco interno	1 cm			0.20 kN/m^3

Table 48:permanent action calculation for Walls on Ground floor- END_P_02

RILIEVO DEI DETTAGLI COSTRUTTIVI DELLE PARETI VERTICALI MEDIANTE INDAGINE ENDOSCOPICA

Sede Municipale – Piazza 6 Aprile 2009 – 65029 Torre De' Passeri (PE)

END_P_01: Piano primo

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_01	Piano primo	1,5	8	6,5	12	n.i,	30	28
	Indagine con perforazione parziale sullo spessore della parete. Rilevato primo paramento in laterizio forato dello spessore di 12 cm, intercapedine 6,5 cm, ed infine un secondo paramento in laterizio > 7 cm.							

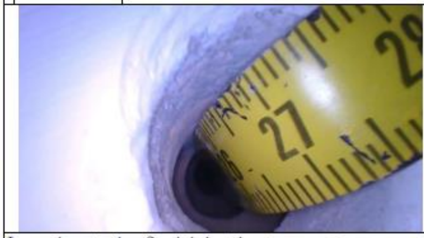


Immagine stratigrafica inizio primo paramento

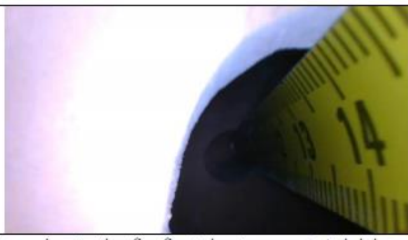


Immagine stratigrafica fine primo paramento + inizio intercapedine

Immagine stratigrafica fine intercapedine

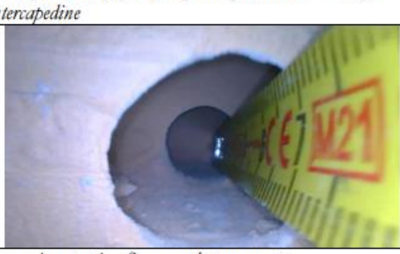


Immagine stratigrafica secondo paramento

Figure 59-layers measurement _First floor(END_P_01)

TAMPONATURA TIPO 5 - PIANO PRIMO

PESO PROPRIO: G_1				**3.04 kN/m^2**
- Mattoni forati in laterizio	8 cm	x	11 kN/m^3	0.88 kN/m^2
- Mattoni pieni in laterizio	12 cm	x	18 kN/m3	2.16 kN/m2

Sovraccarico PERMANENTE: G_2		**0.20 kN/m^2**
- Intonaco interno	1.5 cm	0.20 kN/m^2

Table 49:permanent action calculation for Walls on the First floor- END_P_01

RILIEVO DEI DETTAGLI COSTRUTTIVI DELLE PARETI VERTICALI MEDIANTE INDAGINE ENDOSCOPICA

Sede Municipale – Piazza 6 Aprile 2009 – 65029 Torre De' Passeri (PE)

END_P_02: Piano primo

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_02	Piano primo	1	>7	---	---	n.i.	10	10
	Indagine con perforazione parziale sullo spessore della parete. Rilevato unico paramento in laterizio forato >7 cm							

Immagine stratigrafica inizio paramento

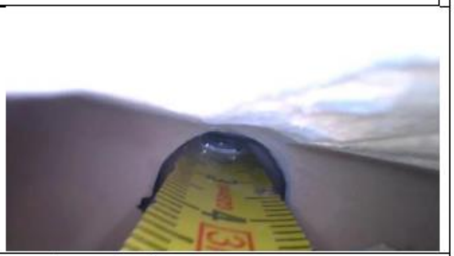


Immagine stratigrafica paramento

Figure 60-layers measurement _First floor(END_P_02)

TRAMEZZATURA - PIANO PRIMO				
PESO PROPRIO: G_1				**0.88 kN/m²**
- Mattoni forati in laterizio	8 cm	x	11 kN/m³	0.88 kN/m²
Sovraccarico PERMANENTE: G_2				**0.20 kN/m²**
- Intonaco interno	1 cm			0.20 kN/m²

Table 50:permanent action calculation for Walls on the First floor- END_P_02

RILIEVO DEI DETTAGLI COSTRUTTIVI DELLE PARETI VERTICALI MEDIANTE INDAGINE ENDOSCOPICA

Sede Municipale – Piazza 6 Aprile 2009 – 65029 Torre De' Passeri (PE)

END_P_01: Piano secondo

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_01	Piano secondo	1	8	11	>11	n.i,	30	31
	Indagine da interno verso esterno con perforazione parziale sullo spessore della parete. Rilevato primo paramento in laterizio forato dello spessore di 8 cm, intercapedine 11cm, ed infine un secondo paramento in laterizio >11 cm.							

Immagine stratigrafica inizio primo paramento

Immagine stratigrafica fine primo paramento

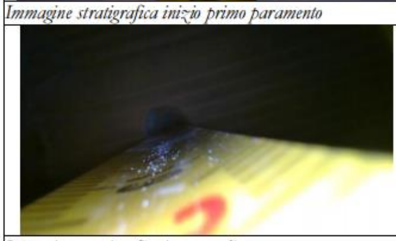


Immagine stratigrafica intercapedine

Immagine stratigrafica secondo paramento

Figure 61-layers measurement _Second floor(END_P_01)

TAMPONATURA TIPO 6 - PIANO SECONDO				
PESO PROPRIO: G_1				**3.04 kN/m^2**
- Mattoni forati in laterizio	8 cm	x	11 kN/m^3	0.88 kN/m^2
- Mattoni pieni in laterizio	12 cm	x	18 kN/m3	2.16 kN/m2
Sovraccarico PERMANENTE: G_2				**0.20 kN/m^2**
- Intonaco interno	1 cm			0.20 kN/m^2

Table 51:permanent action calculation for Walls on the Second floor- END_P_01

RILIEVO DEI DETTAGLI COSTRUTTIVI DELLE PARETI VERTICALI MEDIANTE INDAGINE ENDOSCOPICA

Sede Municipale – Piazza 6 Aprile 2009 – 65029 Torre De' Passeri (PE)

END_P_01: Sottotetto

Sigla indagine	Ubicazione	Spessori (cm)					Spessore parete (cm)	Profondità Indagine (cm)
		Intonaco	I paramento	intercapedine / isolante	II paramento	Intonaco		
END_P_01	Sottotetto	no	6	11	>8	n.i,	non rilevabile	25
	Indagine da interno verso esterno con perforazione parziale sullo spessore della parete. Rilevato primo paramento in laterizio forato dello spessore di 6 cm, intercapedine 11cm, ed infine un secondo paramento in laterizio >8 cm.							

Immagine stratigrafica inizio primo paramento

Immagine stratigrafica fine primo paramento

Immagine stratigrafica intercapedine

Immagine stratigrafica secondo paramento

Figure 62.layers measurement _Attic floor(END_P_01)

TAMPONATURA TIPO 7 - PIANO SOTTOTETTO				
PESO PROPRIO: G_1				**2.46 kN/m^2**
- Mattoni forati in laterizio	6 cm	x	11 kN/m^3	0.66 kN/m^2
- Mattoni pieni in laterizio	10 cm	x	18 kN/m3	1.80 kN/m2
Sovraccarico PERMANENTE: G_2				**0.00 kN/m^2**
- Intonaco interno	0 cm			0.00 kN/m^2

Table 52:permanent action calculation for Walls in the attic floor- END_P_01

9.3 Calculation of seismic action

- Seismic hazard is defined in terms of the maximum expected horizontal acceleration under free-field conditions on a reference rigid site with a horizontal topographic surface and through the ordinates of the elastic response spectrum in acceleration corresponding to it concerning predetermined exceed probabilities P_{V_R}, within the reference period, as defined in § 2.4. These spectral shapes are defined for each exceedance probability within the reference period V_R, starting from the following parameters:
- a_g maximum horizontal acceleration at the site;
- F_0 the maximum value of the acceleration spectrum amplification factor;
- T_c^* The reference value for determining the onset period of the constant velocity segment of the horizontal acceleration spectrum.

The above values can be calculated with reference to four different limit states corresponding to the following exceedance probabilities:

Stati Limite	P_{V_R}: Probabilità di superamento nel periodo di riferimento V_R	
Stati limite di esercizio	SLO	81%
	SLD	63%
Stati limite ultimi	SLV	10%
	SLC	5%

Table 53:Probability of Exceedance P_{VR} as a Function of the Considered Limit State

Having established the reference period of the construction using the following relationship:

$$V_R = V_N \, C_U$$

Where:

- V_N The nominal life of the construction;
- C_U Coefficient of use of the construction.

For each limit state, it is possible to derive the return period T_R of the earthquake using the following:

$$T_R = \frac{-V_R}{\ln\,(1 - P_{V_R})}$$

Based on the return period of the event, the basic seismic hazard parameters are provided from a reference grid available on the website http://esse1.mi.ingv.it/. A point (which identifies the location of the site of interest) within this grid can be processed by weighted averaging of the values taken by the generic parameter at the vertices of the elementary mesh of the reference grid containing the point in question, using the inverses of the distances between the point in question and the four vertices as weights, through the following expression:

$$p = \frac{\sum_{i=1}^{4} \frac{p_i}{d_i}}{\sum_{i=1}^{4} \frac{1}{d_i}}$$

Where:

- p Value of the parameter of interest at the point in question;
- p_i Value of the parameter of interest at the i-th point of the elemental mesh containing the point in question
- d_i It is the distance from the point in question to the i-th point of the aforementioned mesh.

Seismic hazard assessment of the site:

For the site under consideration, we obtain:

Latitudine	42.244811
Longitudine	13.927464
Altitudine [s.l.m.]:	172
Tipo di struttura	Edificio publico
Classo di uso	IV
Vita nominale [anni]	50
Coefficiente d'uso	2.00

Table 54:Site and Structural Information

For the four points of the reference grid, the results are:

Nodes of the reference grid

Site 1 ID: 26979 Lat: 42.2343 Long: 13.8954 Distance: 3000.608

Site 2 ID: 26980 Lat: 42.2340 Long: 13.9629 Distance: 3128.324

Site 3 ID: 26758 Lat: 42.2840 Long: 13.9632 Distance: 5130.764

Site 4 ID: 26757 Lat: 42.2843 Long: 13.8957 Distance: 5055.077

From which, through interpolation for the site under examination, one obtains:

Stato Limite	Tr [anni]	ag [g]	Fo	Tc*[s]
Operatività (SLO)	60	0.092	2.392	0.300
Danno (SLD)	101	0.115	2.402	0.316
Salvaguardia vita (SLV)	949	0.269	2.499	0.360
Prevenzione collasso (SLC)	1950	0.340	2.522	0.374

Table 55:Seismic Design Parameters by Limit State

Response Spectrum simplified approach NTC18.

The simplified approach proposed by NTC18 allows for obtaining response spectra that consider site effects (§3.2.2 NTC18). In particular, these effects are classified into:

-Strati-graphic effects;

-Topographic effects.

For both, the regulations allow the association of a stratigraphic category and a topographic category. The former, based on the examined stratigraphy, is evaluated based on the parameters:

$$V_{S,Eq} = \frac{H}{\sum_{i=1}^{N} \frac{h_i}{V_{S,i}}}$$

Where:

- h_i The thickness of the i-th layer;
- $V_{S,i}$ Shear wave velocity in the i-th layer;
- N Number of layers;
- H Depth of the bedrock, defined as the formation consisting of rock or very stiff soil, characterized by a shear wave velocity not less than 800 m/s
-

The subsoil categories that allow the use of the simplified approach are defined in the following table:

Categoria	Descrizione
A	Ammassi rocciosi affioranti o terreni molto rigidi caratterizzati da valori di velocità delle onde di taglio superiori a 800 m/s, eventualmente comprendenti in superficie terreni di caratteristiche meccaniche più scadenti con spessore massimo pari a 3 m.
B	Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 360 m/s e 800 m/s.
C	Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti con profondità del substrato superiori a 30 m, caratterizzati da

	un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 180 m/s e 360 m/s.
D	Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fina scarsamente consistenti, con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 100 e 180 m/s.
E	Terreni con caratteristiche e valori di velocità equivalente riconducibili a quelle definite per le categorie C o D, con profondità del substrato non superiore a 30 m.

Table 56: subsoil categories

From the results obtained with the MASW tests, a **Vs, eq = 364 m/sec** is obtained. Therefore, the soil is classified seismically as **category B.**

To evaluate topographic effects, in the case of simple configurations (elongated ridges and crests with a height greater than 30 m), the following categories can be referred to:

Categoria	Caratteristiche della superficie topografica
T1	Superficie piana, pendii e rilievi isolati con pendenza media $i \leq 15°$
T2	Pendii con pendenza media $i > 15°$
T3	Colline con larghezza della cresta molto inferiore rispetto alla base e pendenza media di $15° \leq i \leq 30°$
T4	Creste con larghezza della cresta molto inferiore rispetto alla base e pendenza media di $i > 30°$

Table 57: Topographic Surface Characteristics

The topography category of the site under examination is **T1.**

Comparison of simplified spectral methods according to NTC2018 and RSL

Through the regularization procedure proposed in Appendix 1 of Ordinance No. 55 of April 24, 2018, it is possible to transform the response spectrum, resulting from numerical simulations of local seismic response, into a standard-shaped spectrum (according to NTC18). Specifically, this procedure provides parameters for inserting the elastic spectrum into structural analysis software aimed at designing structures in seismic areas.

Below is the comparison for different limit states between the spectrum obtained using the simplified method of NTC2018 and the normalized spectrum obtained from RSL. Since the spectra obtained using the simplified method of NTC2018 are more conservative than those obtained from RSL, the spectra obtained with the simplified method of NTC2018 were used in the calculation model.

SLO

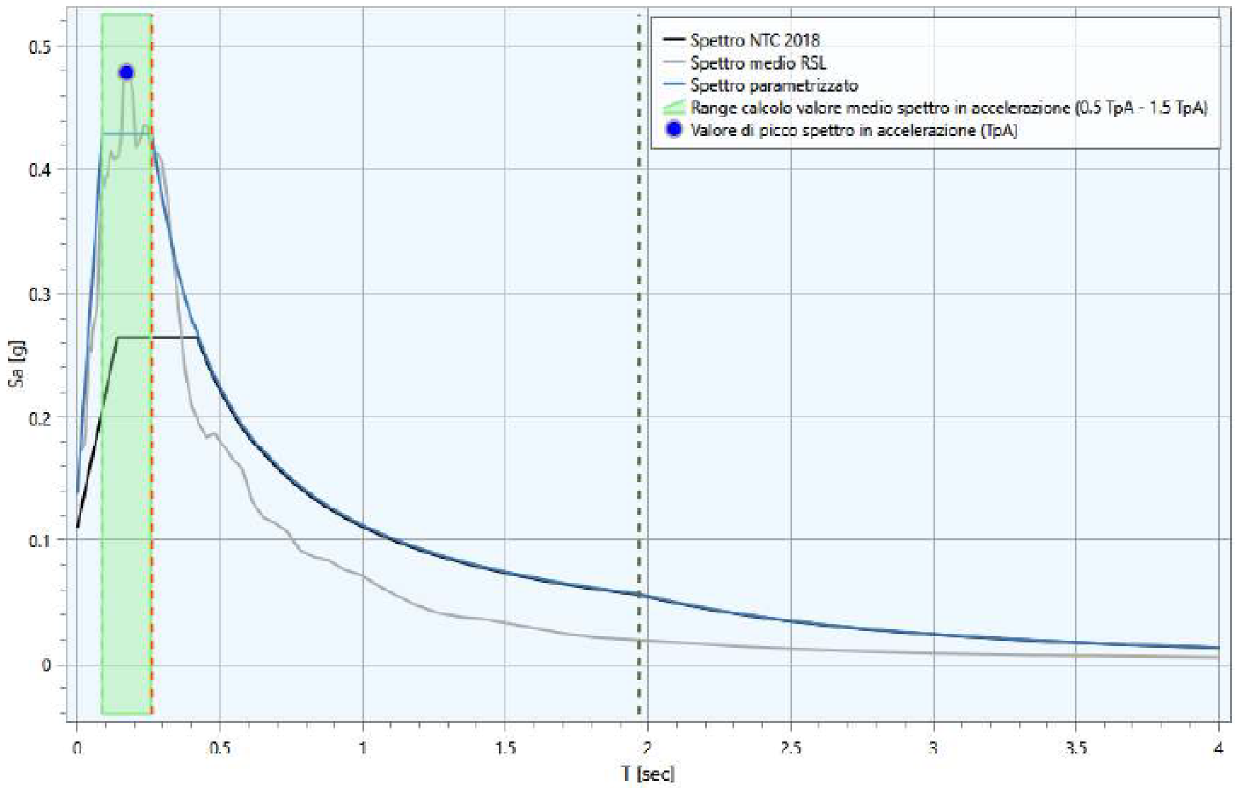


Figure 63:Seismic Spectra Analysis (SLO)

The seismic parameters obtained from the normalization of the spectrum resulting from local seismic response to the **SLO** are as follows:

A_{max}	S	F_0	$T_B\ [sec]$	$T_C\ [sec]$	$T_D\ [sec]$
0.138	1.49	3.13	0.09	0.26	1.97

Table 58:spectrum resulting from local seismic response to the SLO

It is obtained from the structural calculation software that the main mode in the X direction has a period of 0.801 seconds.

SLD

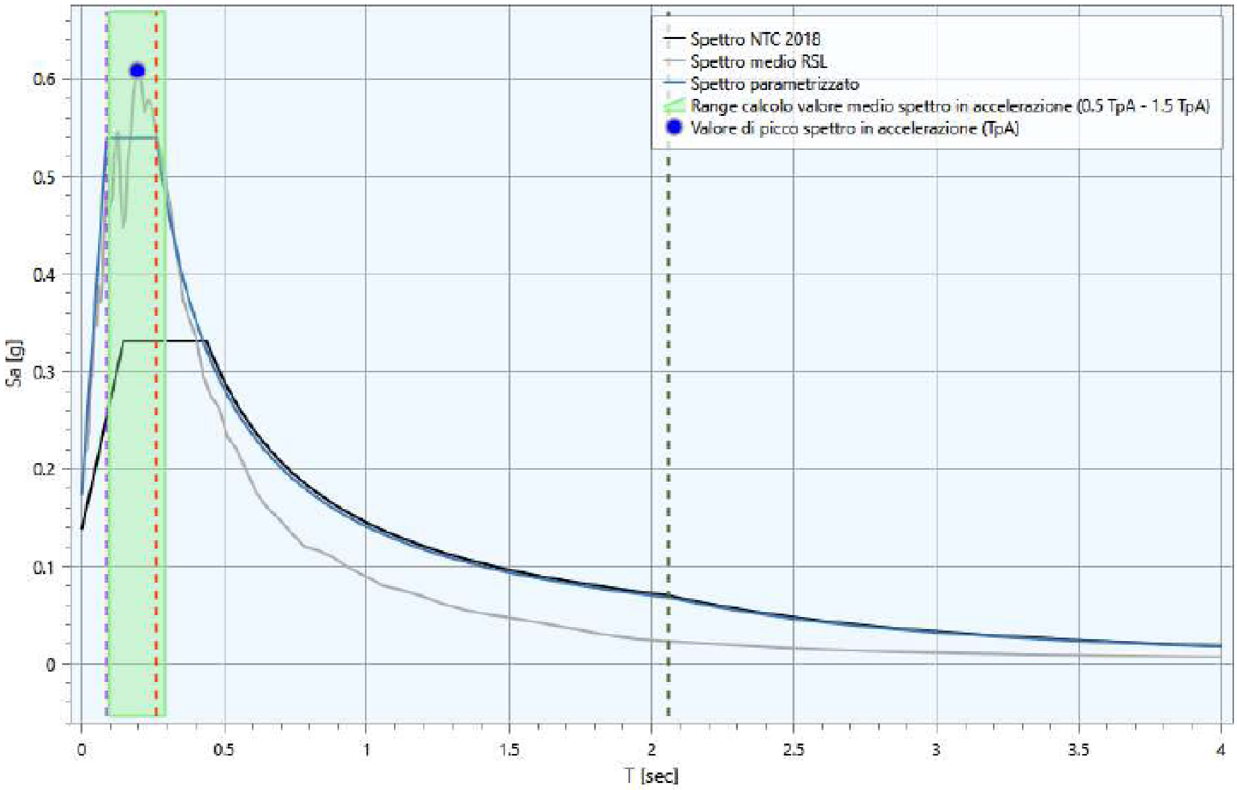


Figure 64:Seismic Spectra Analysis (SLD)

The seismic parameters obtained from the normalization of the spectrum resulting from the local seismic response to the SLD are as follows:

A_{max}	S	F_0	T_B [sec]	T_C [sec]	T_D [sec]
0.173	1.51	3.12	0.09	0.26	2.06

Table 59:spectrum resulting from local seismic response to the SLD

It is obtained from the structural calculation software that the main mode in the X direction has a period of 0.801 seconds.

SLV

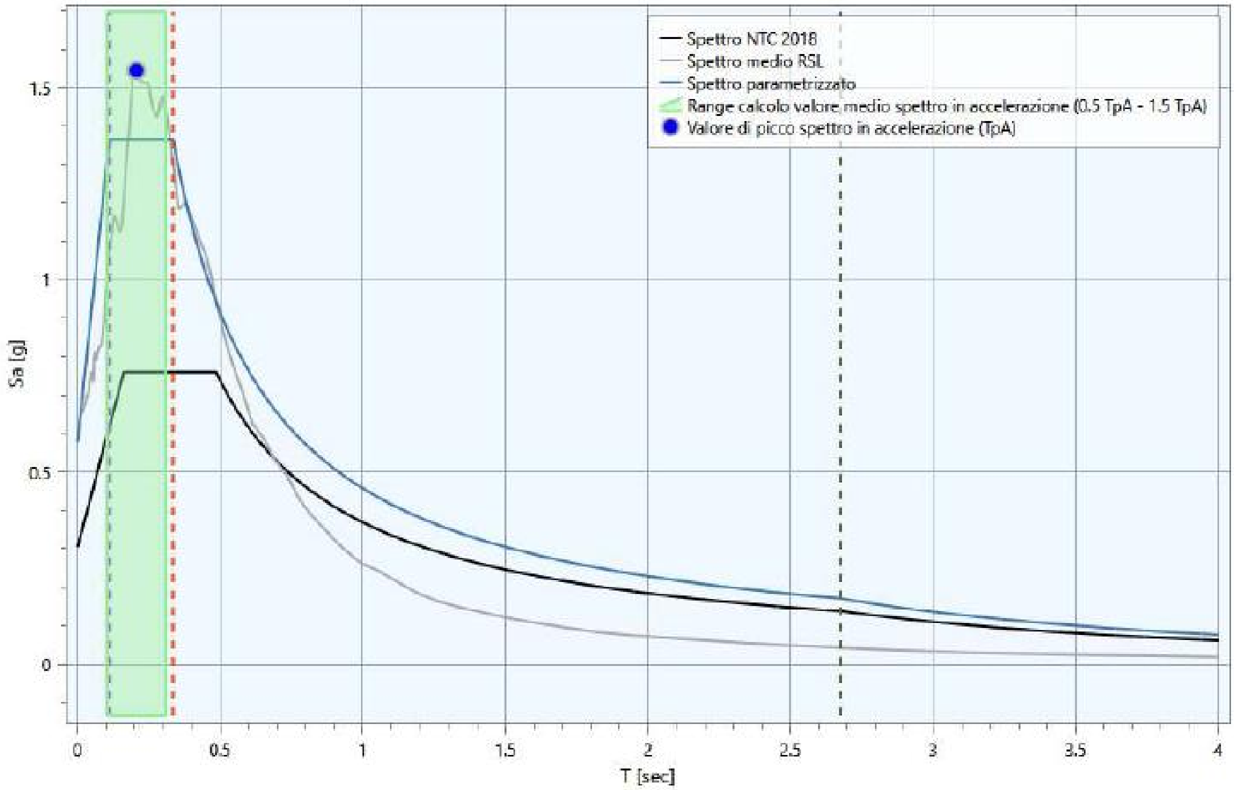


Figure 65:Seismic Spectra Analysis (SLV)

The seismic parameters obtained from the normalization of the spectrum resulting from the local seismic response to the SLV are as follows:

A_{max}	S	F_0	T_B $[sec]$	T_C $[sec]$	T_D $[sec]$
0.578	2.15	2.37	0.11	0.34	2.68

Table 60:spectrum resulting from local seismic response to the SLV

It is obtained from the structural calculation software that the main mode in the X direction has a period of 0.801 seconds.

SLC

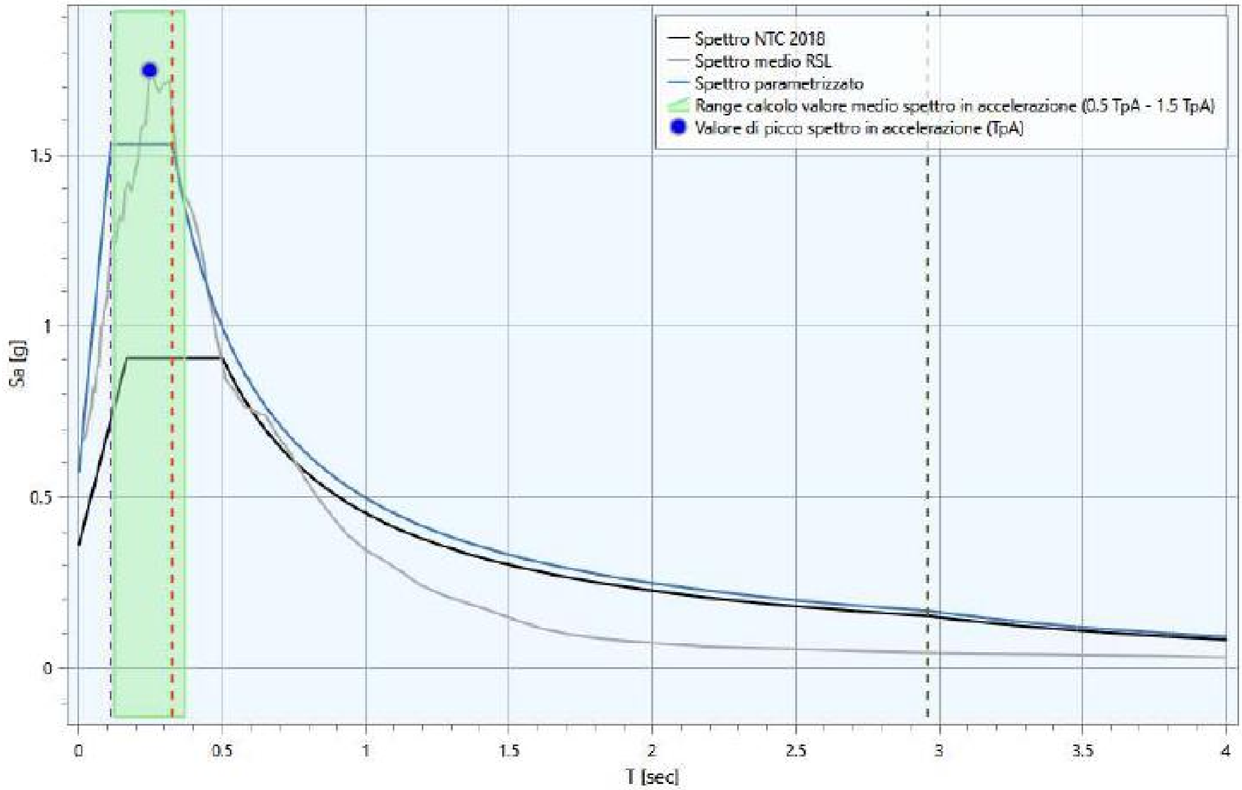


Figure 66:Seismic Spectra Analysis (SLC)

The seismic parameters obtained from the normalization of the spectrum resulting from the local seismic response to the SLC are as follows:

A_{max}	S	F_0	T_B [sec]	T_C [sec]	T_D [sec]
0.573	1.68	2.67	0.11	0.33	2.96

Table 61:spectrum resulting from local seismic response to the SLC

It is obtained from the structural calculation software that the main mode in the X direction has a period of 0.801 seconds.

9.4 Determination of Wind loads

$P = q_b * c_e * c_p * c_d$

Calculation of Reference kinetics pressure(q_b) :

$q_b = V_{b2}$ [3.3.1]

ρ=Air density-1.25 kg/m3

V_b=Reference wind speed (in m/s)

*Calculation of Reference wind speed Vb

$V_b = V_{b,0}.\ C_a = 27*1 = 27$

$V_{b,0}$ =is the basic reference speed at sea level, assigned in Tab. 3.3.I according to the area in which the building is located

C_a= is the altitude coefficient given by the relation:

$C_a=1$ for as a0 [3.3.1.b]

$a_s=175$

$a_0=500$

$k_0 =0.37$

a_0, k_0 = they are parameters provided in Tab. 3.3.I according to the area in which the building stands (Fig. 3.3.1);

a_s = is the altitude above sea level of the site where the building stands.

Tab. 3.3.I -*Valori dei parametri* $v_{b,0}$, a_0, k_s

Zona	Descrizione	$v_{b,0}$ [m/s]	a_0 [m]	k_s
1	Valle d'Aosta, Piemonte, Lombardia, Trentino Alto Adige, Veneto, Friuli Venezia Giulia (con l'eccezione della provincia di Trieste)	25	1000	0,40
2	Emilia Romagna	25	750	0,45
3	Toscana, Marche, Umbria, Lazio, Abruzzo, Molise, Puglia, Campania, Basilicata, Calabria (esclusa la provincia di Reggio Calabria)	27	500	0,37
4	Sicilia e provincia di Reggio Calabria	28	500	0,36
5	Sardegna (zona a oriente della retta congiungente Capo Teulada con l'Isola di Maddalena)	28	750	0,40
6	Sardegna (zona a occidente della retta congiungente Capo Teulada con l'Isola di Maddalena)	28	500	0,36
7	Liguria	28	1000	0,54
8	Provincia di Trieste	30	1500	0,50
9	Isole (con l'eccezione di Sicilia e Sardegna) e mare aperto	31	500	0,32

Table 63: parameter values of Vb,0,a0, K0

Figure 67:Italian wind climatic zone

The location of the site is in zone 3:

$V_{b,0}$ =27 m/s, a_0 =500m , K_s =0.37

The sea level of ex manifattura Tabbachi according to Google map :

a_s= 172 $a_s \leq a_0$ Ca =1

according to the tab 3.1.1

$q_b=V_{b2}$ = ½ *1.25*272 =455.63

Influence of the return period T_R:

$V_R=V_b \cdot C_r$

V_R = Reference wind speed for return periods different from 50 years.

C_R = Return coefficient.

T_R = return period expressed in years.

*For new work in the phase of construction or for transitional phases relating to interventions on existing buildings, the period of return of the action may be reduced as follows:

- for construction phases or transitional phases with an expected duration at the time of the project of between three months and one year, it will be assumed TR =50 years.

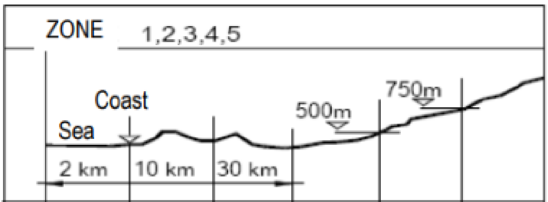
$$c_r = 0.75\sqrt{1 - 0.2 \times \ln\left[-\ln\left(1 - \frac{1}{T_R}\right)\right]}$$

$C_r = 1$

Exposure coefficient C_e:

The exposure coefficient C_e depends on the height z above the ground of the point considered, the topography of the terrain, and the exposure category of the site where the construction stands. In the absence of specific analyses that take into account the direction of origin of the wind and the actual roughness and topography of the land surrounding the building, for heights on the ground not greater than z = 200 m, it is given by the formula:

Topographic coefficient C_t =1 (suggested)



A	- -	IV	IV	V	V	V
B	- -	III	III	IV	IV	IV
C	- -	*	III	III	IV	IV
D	I	II	II	II	III	**

* Categoria II in zona 1,2,3,4
Categoria III in zona 5

** Categoria III in zona 2,3,4,5
Categoria IV in zona 1

Table 64:Zonal Classification Based on Distance from Coast and Elevation

Categoria di esposizione del sito	k_r	z_0 [m]	z_{min} [m]
I	0,17	0,01	2
II	0,19	0,05	4
III	0,20	0,10	5
IV	0,22	0,30	8
V	0,23	0,70	12

Table 65: parameter values of K_r, Z_0, Z_{min}

Roughness class of the soil	Description
A	Aree urbane in cui almeno il 15% della superficie sia coperto da edifici la cui altezza media superi i 15m **Urban areas**
B	Aree urbane (non di classe A), suburbane, industriali e boschive **Suburban areas**
C	Aree con ostacoli diffusi (alberi, case, muri, recinzioni,....); aree con rugosità non riconducibile alle classi A, B, D **Rural areas**
D	Aree prive di ostacoli (aperta campagna, aeroporti, aree agricole, pascoli, zone paludose o sabbiose, superfici innevate o ghiacciate, mare, laghi,....) **Open fields**
L'assegnazione della classe di rugosità non dipende dalla conformazione orografica e topografica del terreno. Affinché una costruzione possa dirsi ubicata in classe A o B è necessario che la situazione che contraddistingue la classe permanga intorno alla costruzione per non meno di 1 km e comunque non meno di 20 volte l'altezza della costruzione. Laddove sussistano dubbi sulla scelta della classe di rugosità, a meno di analisi dettagliate, verrà assegnata la classe più sfavorevole.	

Table 66:Roughness Class of the Soil

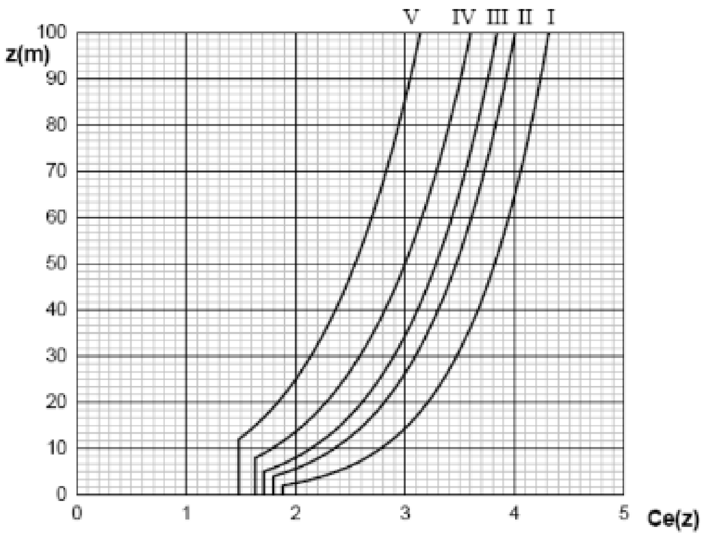


Table 67: Exposure coefficient graph based on the height (Fore C_t=1)

The project is in zone3 with a sea level elevation of 172 m and in the Suburban area (B) Category= IV Kr = 0.22, Z0 =0.30(m), Z_{min} =8 (m)

Z=16.5 , Z_{min} =8 m , $Z \geq Z_{min}$,Ce =0.5

Pressure coefficient cp:

It depends on the type and geometry of the construction and its orientation with respect to the wind direction. The net pressure coefficient C_p is given by the difference between the C_{pe}(external) and the C_{pi} (internal), a difference made considering the most unfavorable condition of the sign.

$C_p = C_{pe} - C_{pi}$

*External pressures coefficients C_{pe}:

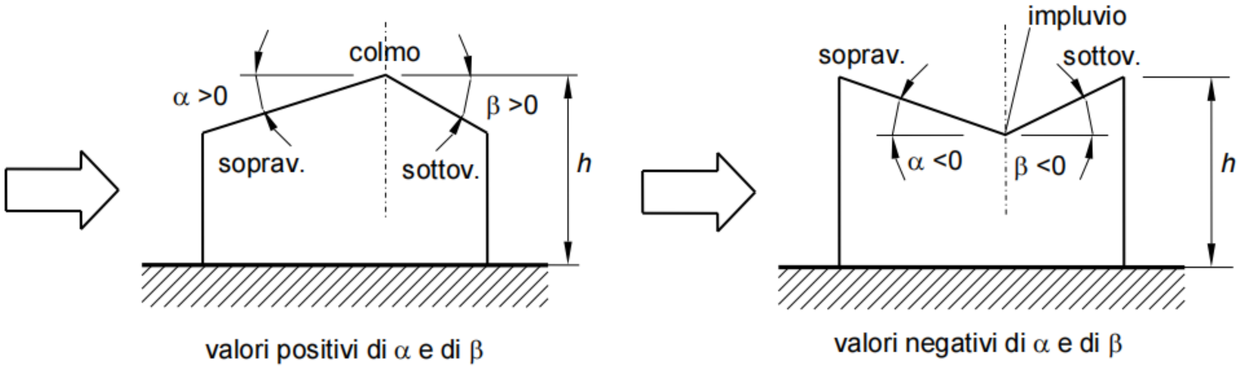


Figure 68: Reference scheme for the double-pitched roof(positive and negative values)

$\alpha \leq -30°$	$c_{pe} = -0{,}8$
$-30° \leq \alpha \leq -15°$	$c_{pe} = -1{,}2 - \alpha/75$
$-15° \leq \alpha \leq 45°$	$c_{pe} = -0{,}8 + \alpha/75$
$45° \leq \alpha$	$c_{pe} = -0{,}2$

Figure 69: pressure coefficient for double pitched roofs (α in °): wind normally directed to the ridge lines

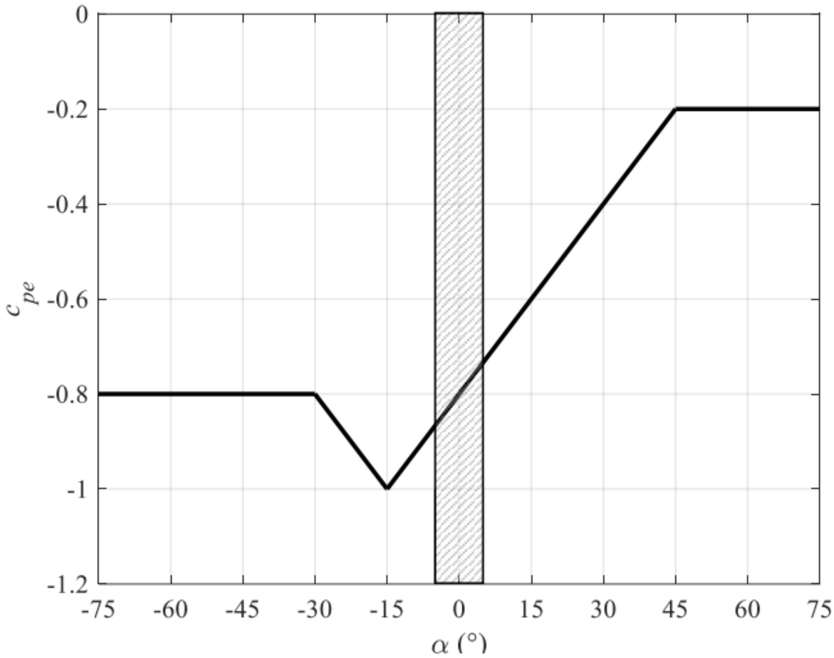


Figure 70: Pressure coefficients for double-pitched roofs: leeward slope with wind direction perpendicular to the ridge.

C_{pe}=-0.8+α/75 $\Rightarrow$-0.8+18/75=-0.56

C_{Pi}=0.3

C_p=0.56-0.3=0.26

C_d (Dynamic Coeff.)

Dynamic Coeff. can be assumed as C_d=1 in buildings of recurrent types, such as buildings of regular shape not exceeding 80 m in height and industrial warehouses

$P=q_b\ c_e\ c_p\ c_d$

P=0.455*1*0.26*1=0.1183

9.5 Determination of snow loads

The snow load on roofs, where applicable, is calculated using the following formula from the regulations:

$q_s=i * a_{sk} * C_E * C_t$ (Refer to §3.3.7)

Where:

q_s = snow load on the roof;

μ_i = shape coefficient of the roof, as specified in (Refer to §3.4.5);

a_{sk} = the characteristic reference value of the snow load on the ground [kN/m²], as specified in (Refer to §3.4.2) of the NTC 2018

for a 50-year return period;

C_E = exposure coefficient, according to (Refer to §3.4.3);

C_t = thermal coefficient, according to (Refer to §3.4.4).

DATA DEFINITION

In accordance with Section 3.4 of the Italian NTC18 regulations, the minimum reference snow load for areas at or below an altitude of 1500 meters above sea level (m a.s.l.) must align with the values specified in the regulatory tables, which are based on a statistical return period of 50 years. When dealing with locations at or above 1500 m a.s.l., it is required to consult local statistical data to determine appropriate snow load values. However, these locally determined snow loads should never be below the minimum values specified for an altitude of 1500 m a.s.l. in the NTC18 regulations.

a_s=Altitude from sea level

a_s=172m

the building is located in Pescara, which is in zone II.

In the context of Italian building regulations regarding snow loads for structures situated at altitudes exceeding 1500 meters above sea level, the minimum snow load values are determined based on local climate and exposure conditions. These values must be at least equivalent to the baseline values established for an elevation of 1500 meters.

For structures under construction or for temporary structures associated with modification or repair works on existing buildings, the standard return period for snow load calculations is modified as follows:

If the construction phase or temporary condition is anticipated to last no more than three months, a reduced return period of at least 5 years is used for snow load calculations.

If the construction phase or temporary condition is expected to last between three months and one year, a longer return period of at least 10 years is applied.

These adjustments to the return period account for the shorter duration of risk exposure due to snow loads during the specified construction or temporary phases.

The exposure coefficient should be used to modify the value of the snow load on roofs based on the specific characteristics of the area where the structure is located. Normally, one adopts Ce=1. The recommended coefficients for the different topographic classes are reported in the table.

As the topographical class is normal, Ce=1.

The angles of the roof are:

$\alpha 1=18$

$\alpha 2=18$

qs=0.8*1*1*1=0.8KN/mq

10 DEFINITION OF GENERAL AND SPECIFIC CRITERIA FOR ASSESSING THE SEISMIC VULNERABILITY AND STATIC ADEQUACY OF BUILDINGS

10.1 Regularity analysis - US1

A building is considered regular in plan if it meets all of the following conditions:

a) The distribution of masses and stiffnesses is approximately symmetrical with respect to two orthogonal directions, and the plan shape is compact, meaning that the outline of each floor is convex; the requirement can be considered satisfied even in the presence of plan recesses when they do not significantly affect the stiffness in the plane of the floor, and for each recess, the area between the floor perimeter and the circumscribed convex line does not exceed 5% of the floor area;

b) The ratio between the sides of a rectangle in which the building is inscribed is less than 4;

c) Each floor has a stiffness in its own plane much greater than the corresponding stiffness of the vertical structural elements so that its deformation in plan negligibly affects the distribution of seismic actions among these elements, and it has sufficient strength to ensure the effectiveness of such distribution.

Analyzing the previous requirements to establish plan regularity, **THE BUILDING IS FOUND TO BE IRREGULAR IN PLAN.**

Height irregularity check

A building is considered regular in height if it meets all of the following conditions:

a) All systems resisting horizontal actions extend throughout the entire height of the building or, if there are parts with different heights, up to the top of the respective part of the building;

b) Mass and stiffness remain constant or vary gradually, without abrupt changes, from the base to the top of the building (mass variations from one floor to another do not exceed 25%, and stiffness does not decrease from one floor to the one above by more than 30% and does not increase by more than 10%); for stiffness purposes, structures with walls or concrete cores, or walls and cores in masonry with constant section height, or braced steel frames, to which at least 50% of the seismic action is allocated at the base, can be considered regular in height;

c) The ratio between capacity and demand at the SLS is not significantly different, in terms of strength, for successive floors (this ratio, calculated for a generic floor, must not differ by more than 30% from the same ratio calculated for the adjacent floor); the last floor of framed structures with at least three floors may be an exception;

d) Any narrowing of the horizontal section of the building occurs continuously from one floor to the next or occurs so that the setback of a floor does not exceed 10% of the corresponding dimension of the floor immediately below nor 30% of the dimension of the first floor. The last floor of buildings with at least four floors is an exception, for which no narrowing limitations are provided.

Analyzing the previous requirements to establish height regularity, **THE BUILDING IS FOUND TO BE IRREGULAR IN HEIGHT.**

10.2 Regularity analysis US2

Regular buildings are a special category whose characteristics can significantly simplify some design and verification choices. The requirement of regularity ensures that the first modes of vibration of the structure are similar to those of a shelf, with almost total involvement of the entire mass and limitations on torsional frequencies. A building is considered regular if it is regular both in plan and in height.

Plan regularity verification

A building is considered regular in plan if it meets all of the following conditions:

a) The distribution of masses and stiffness is approximately symmetrical with respect to two orthogonal directions, and the plan shape is compact, meaning the outline of each floor is convex. The requirement can be considered satisfied even in the presence of plan recesses when they do not significantly affect the stiffness in the floor plane, and for each recess, the area between the perimeter of the floor and the convex circumscribed line does not exceed 5% of the floor area;

b) The ratio between the sides of a rectangle in which the building is inscribed is less than 4;

c) Each floor has **stiffness in its own plane greater than the corresponding stiffness of the vertical structural elements, such that its deformation in plan negligibly influences the distribution of seismic actions among these elements** and has sufficient strength to ensure the effectiveness of such distribution.

Analyzing the above requirements to determine plan regularity, **THE BUILDING IS CONSIDERED TO BE REGULAR IN PLAN.**

Height regularity verification

A building is considered regular in height if it meets all of the following conditions:

a) All systems resisting horizontal actions extend throughout the entire height of the building, or if parts with different heights are present, extend to the top of the respective part of the building;

b) Mass and stiffness remain constant or vary gradually, without abrupt changes, from the base to the top of the building (variations in mass from one floor to another do not exceed 25%, and stiffness does not decrease from one floor to the one above by more than 30% and does not increase by more than 10%); for stiffness purposes, structures with walls or concrete cores, or walls and cores in masonry with constant section height, or steel braced frames, to which at least 50% of the seismic action is assigned at the base, can be considered regular in height;

c) The ratio between capacity and demand at the SLS is not significantly different, in terms of strength, for successive floors (this ratio, calculated for a generic floor, must not differ by more than 30% from the analogous ratio calculated for the adjacent floor); an exception can be made for the last floor of frames of at least three floors;

d) Any narrowing of the horizontal section of the building occurs continuously from one floor to the next or occurs so that the setback of a floor does not exceed 10% of the dimension corresponding to the immediately underlying floor, nor 30% of the dimension corresponding to the first floor. An exception is made for the last floor of buildings with at least four floors, for which no narrowing limitations are provided.

Analyzing the above requirements to determine height regularity, **THE BUILDING IS CONSIDERED TO BE REGULAR IN HEIGHT.**

11 STRUCTURAL MODELING

11.1 Introduction to Modeling and choice of calculation code

The software used for the modeling and subsequent analysis of the building is CDSwin 2022, license number 36779.

The modeling of each structural unit was approached comprehensively through a three-dimensional model. In particular, modal analysis and nonlinear static verification (Pushover) were conducted.

Structural modeling is the process through which the structural system is schematized with a physical-mathematical model composed of sub-models:

Geometric model: geometry of the structural system, defined through the selection of elements and constraints;

Mechanical model: constitutive laws of the structural materials used, defined by mechanical parameters (strength and deformability);

Actions model: "actions that affect the structural system during its design life. The software used for modeling and subsequent analyses of the building is CDSwin 2022, license number 36779."

Geometric Modell – US1 e US2

The geometric model is created in the calculation software in the 'input for floors' section and represents the first operational phase. In this phase, the structure is modeled using the finite element method and various specialized library elements to schematize the various structural elements. A virtual model of the building is then created using parametric objects such as columns, beams, partitions, etc., and the seismic parameters and design criteria of each structural element are defined.

All the information identified from on-site surveys and retrieved from the original project has been entered into the geometric model:

The sections of the structural elements were derived from the laser scanner survey conducted from scratch and from the investigations carried out to investigate the construction details;

The loads on the floors were derived from endoscopies of the floor slabs;

The mechanical characteristics are defined based on the results obtained from destructive and non-destructive tests carried out during the survey campaign.

In the schematization of the floors, they are chosen to be inserted as a load in order to represent their effects on the beams.

The inter-floor slabs are of the concrete block type (with different thicknesses on various floors); the roof for US1 is a pitched concrete block roof with a thickness of 20+4 cm, and for US2, it is a flat concrete block slab, 16 cm thick. The infill walls are of various types, including:

Box infill walls consisting of a first internal layer of perforated brick, an air gap, and a second layer of perforated brick;

Infill walls consisting of a single layer of perforated brick;

Modelling

- As the first step of the modeling part, it is necessary to export the plans files from Revit to Autocad and simplify them as much as possible in order to just have the mean structural elements; below the simplified plans are available :

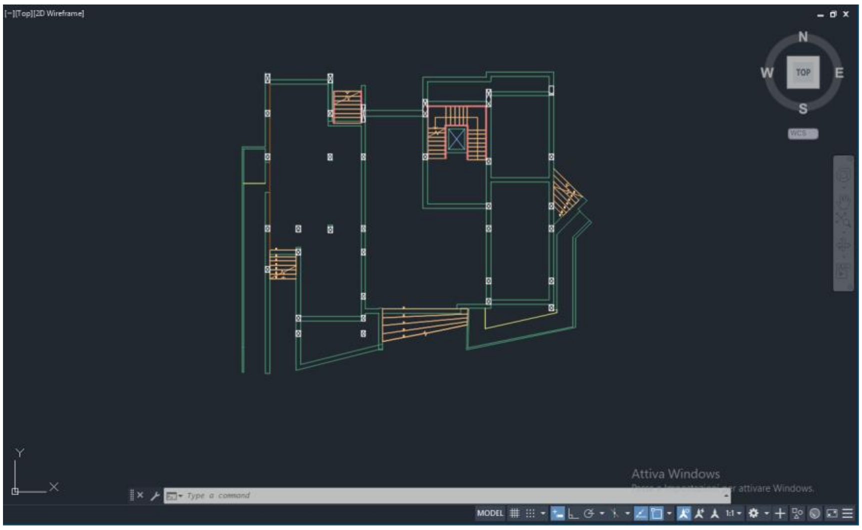


Figure 71. Simplified plan of underground floor

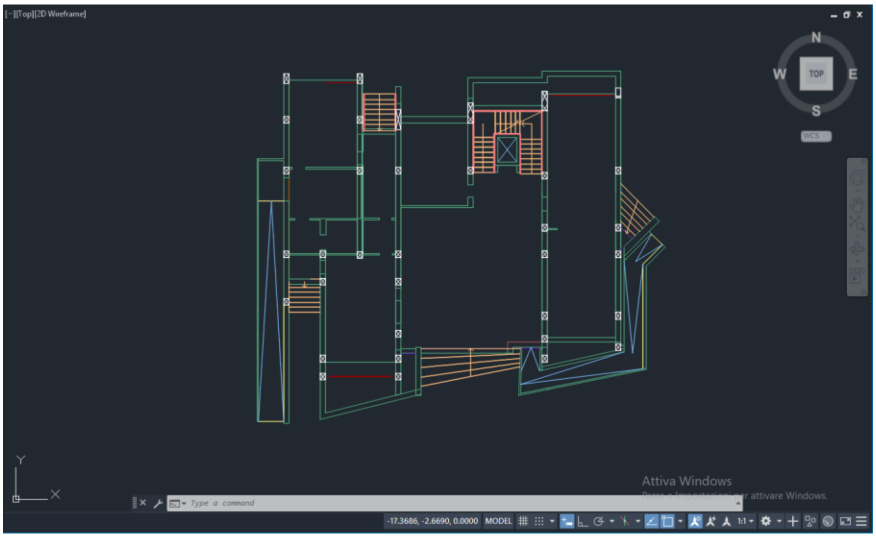


Figure 72. Simplified plan of Ground floor

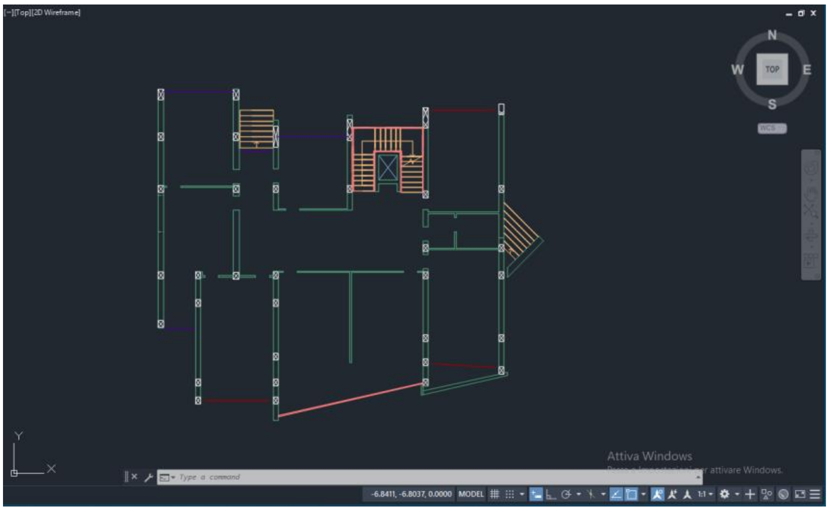


Figure 73. Simplified plan of underground floor

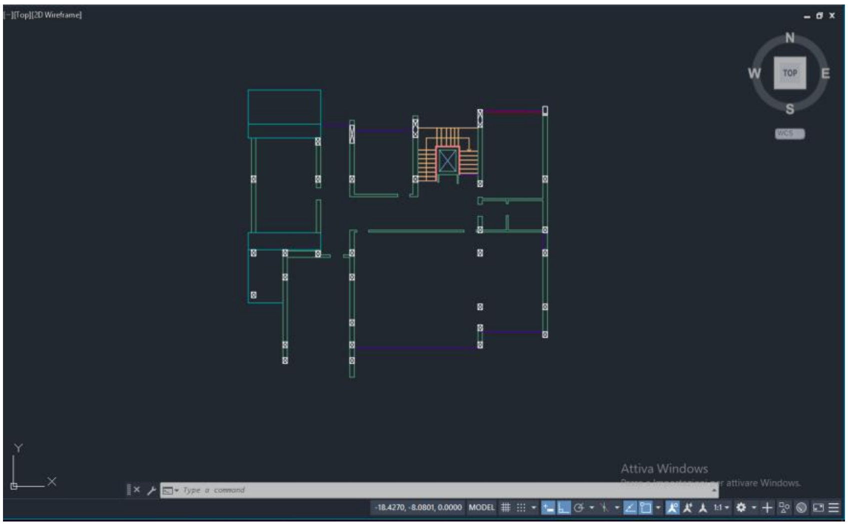


Figure 74. Simplified plan of Second floor

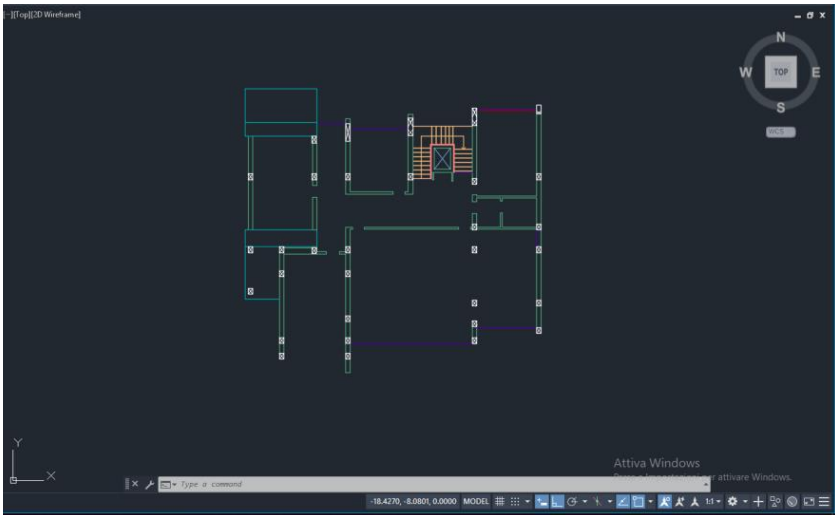


Figure 75. Simplified plan of Attic floor

- In order to coordinate the plans in CDSwin, it is essential to bring a specific point that is similar in all plans to (0,0) coordination; for making reference points more easily, the axis of columns had been drawn by cross-shaped lines.

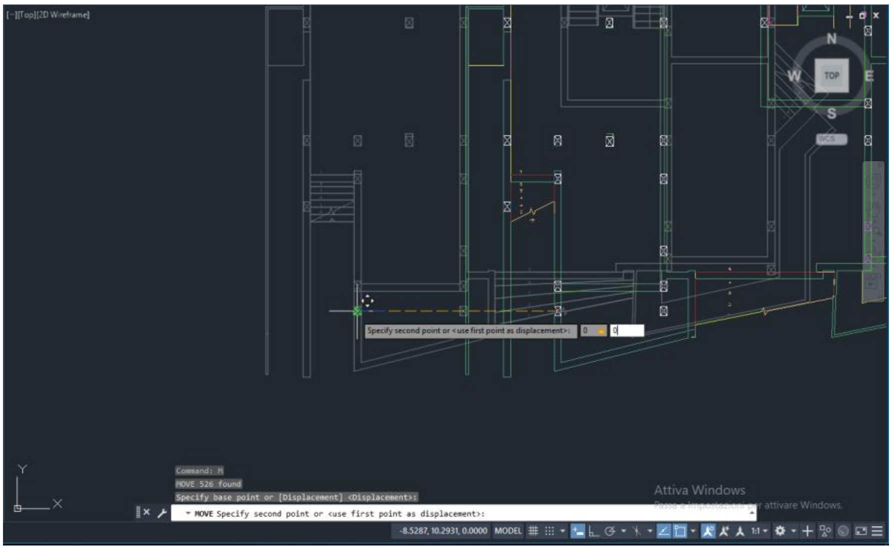


Figure76..moving to coordination 0.0

- To Start a new project: To start a new project, I made a new folder in drive C named EX-ANTE for the project, then in CDS WIN: File /open the project/EX-ANTE.

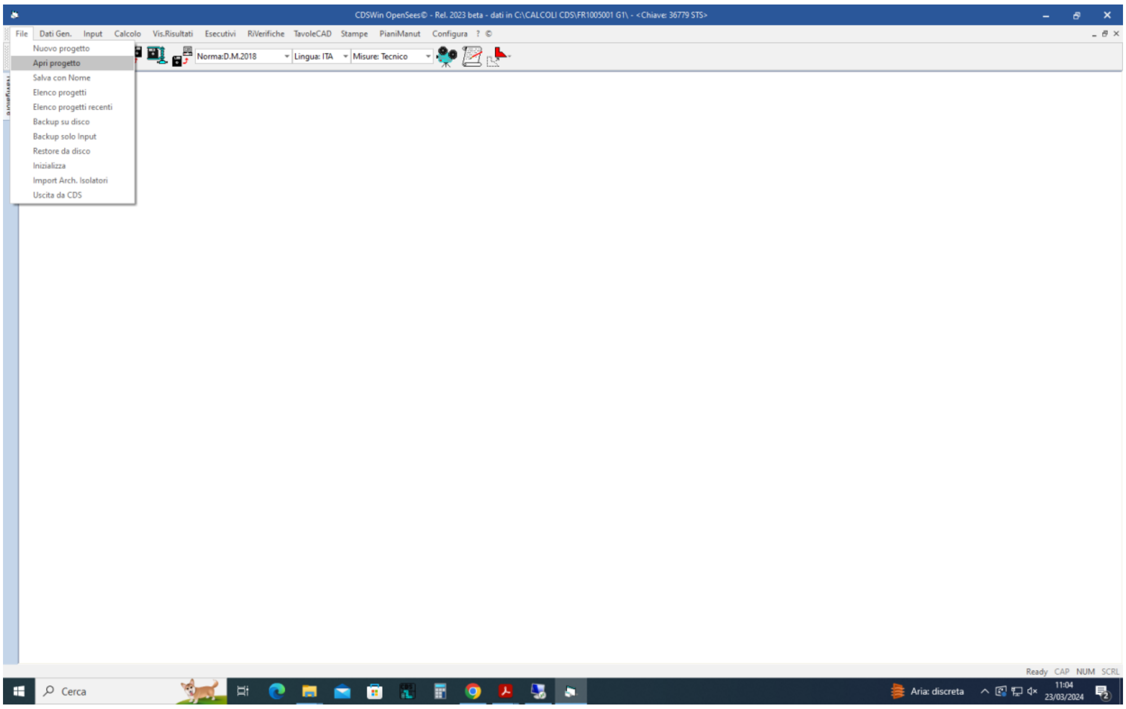


Figure 77..starting a new project in CDSwin.

- In the next step, it is necessary to identify a folder in which the CDSwin file is going to be located:

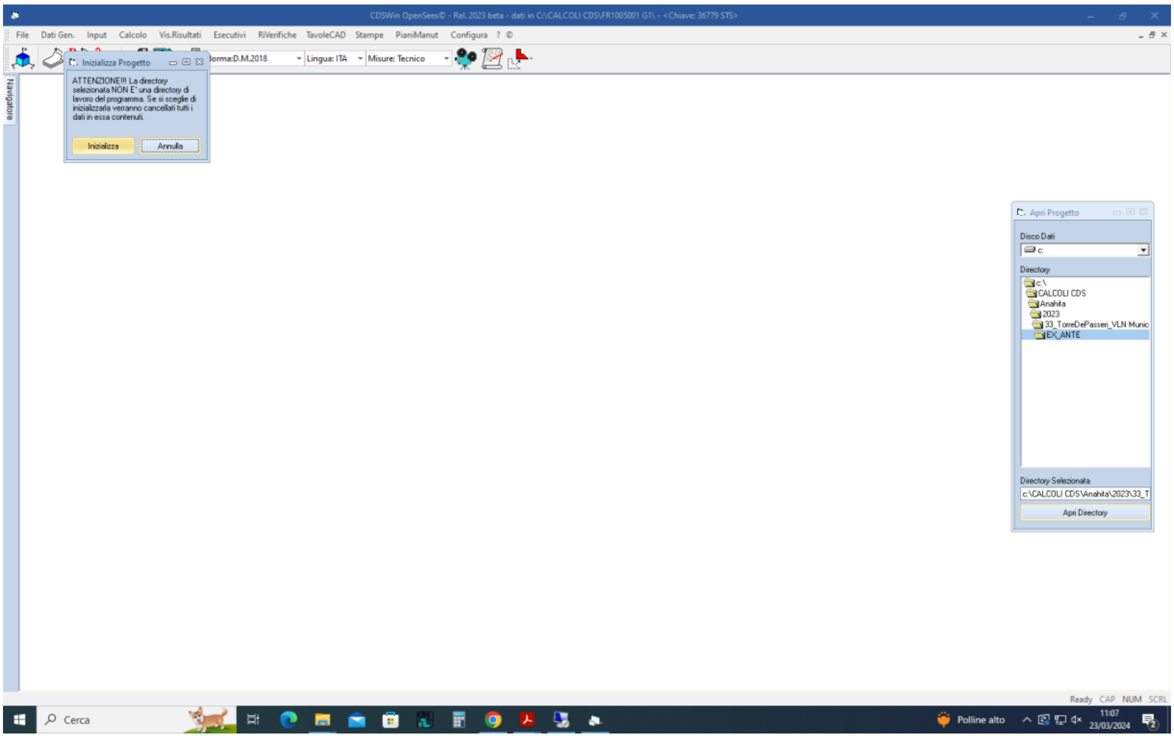


Figure 78.Creating a new folder for starting a new project in CDSwin

- The files that have been exported from Autocad software in DXF format should be pasted on the same folder as the mean file is in :

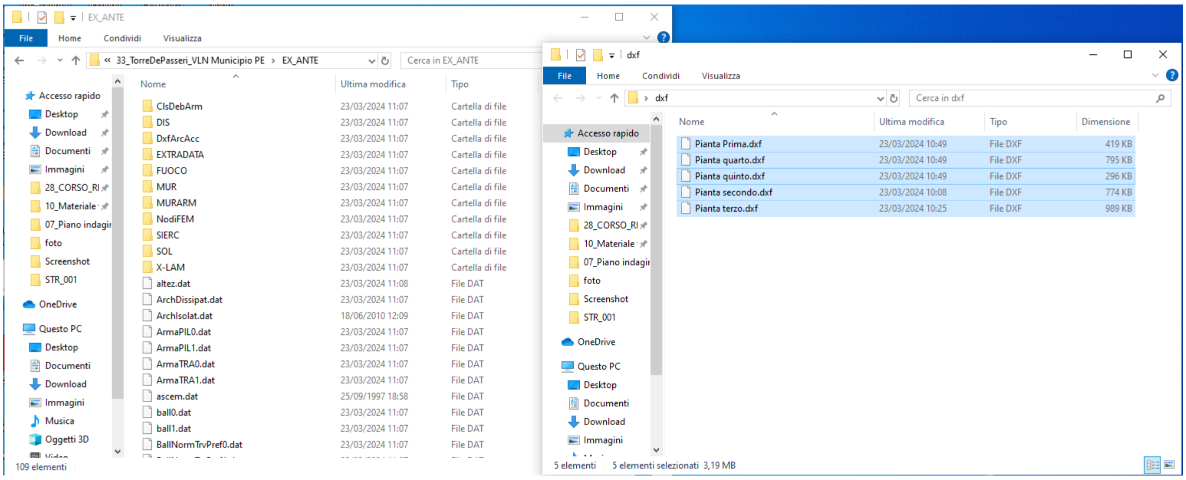


Figure 79.Inserting DXF

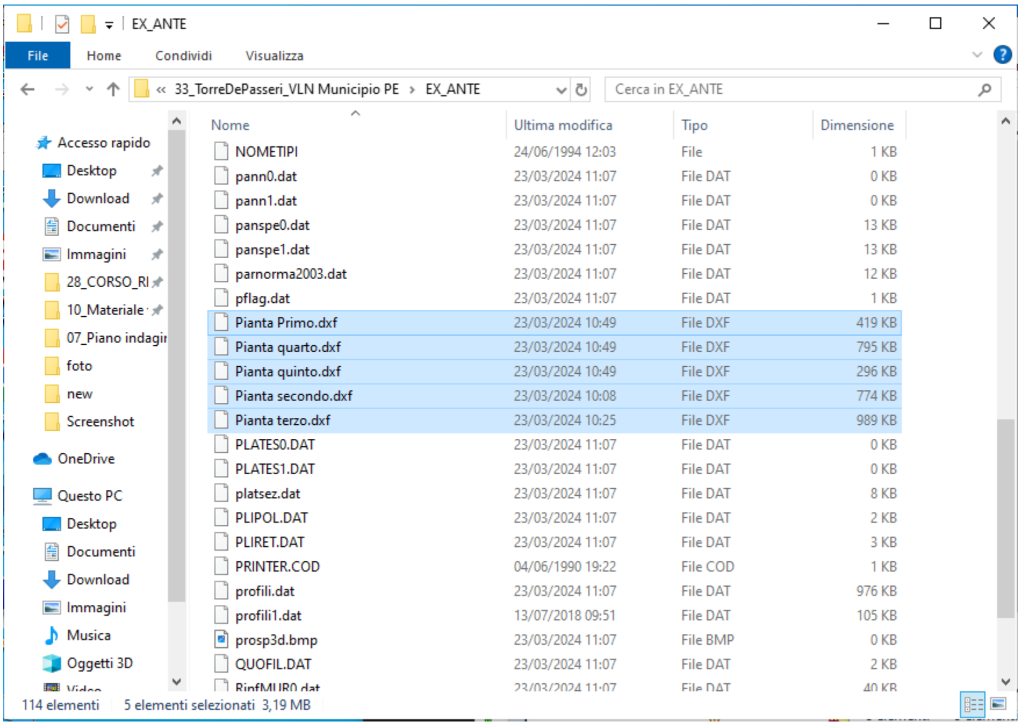


Figure 80. DXF files inserted

- For Making floors in CDSwin, the path is choosing floors (Quotes) from the mean bar and then inserting the name of the floors as numbers and also their height; the name of the dxf files should be inserted in the DXF ARCHIT part:

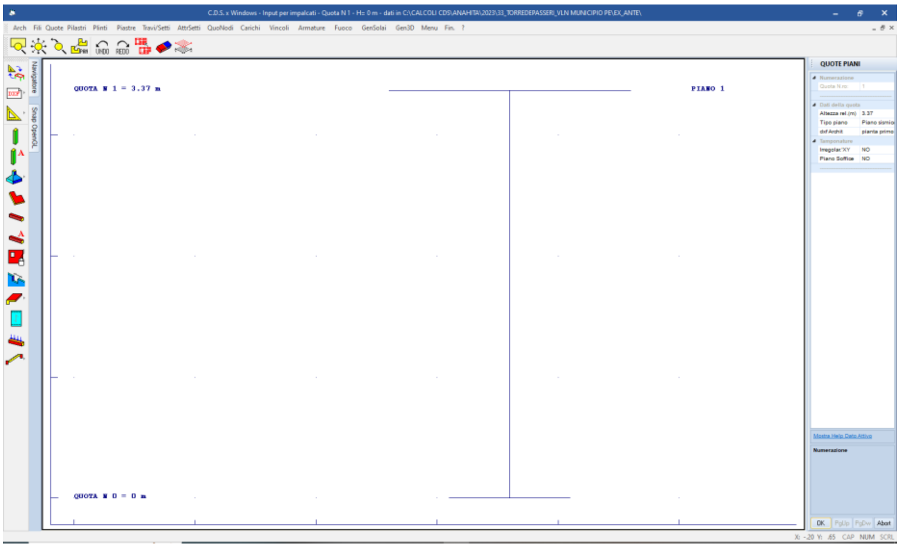


Figure 81. Floor1-Underground floor

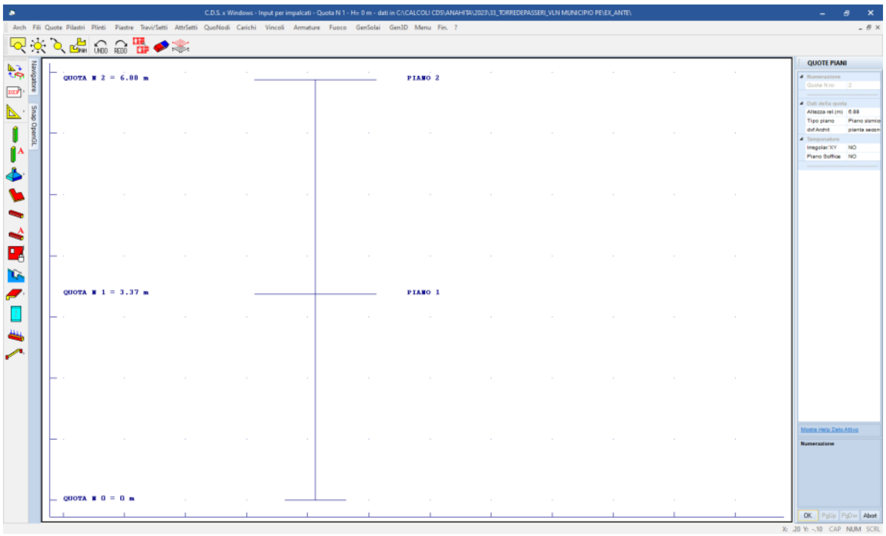


Figure 82. Floor 2-Ground floor

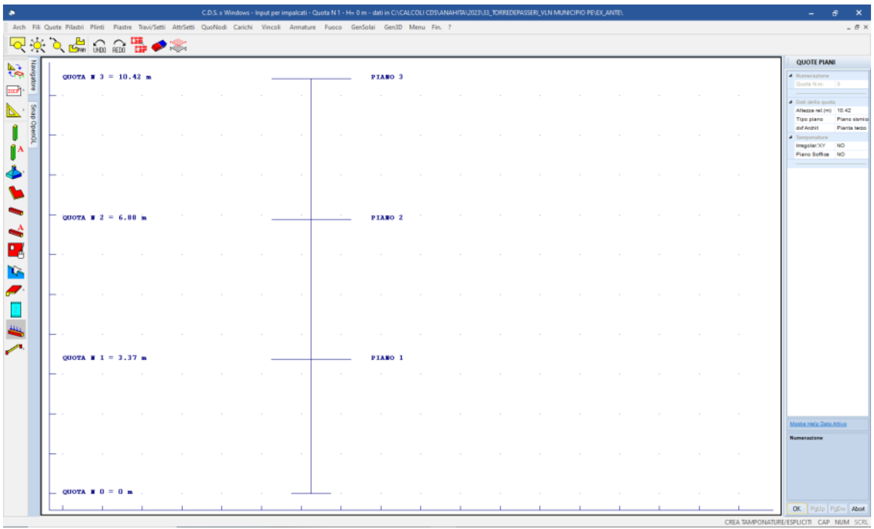


Figure 83. Floor 3-First floor

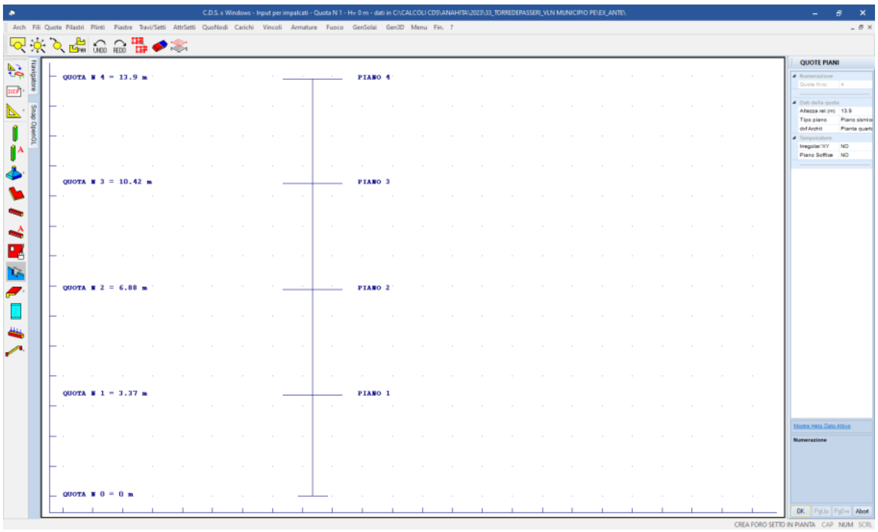


Figure 84. Floor 4-Second floor

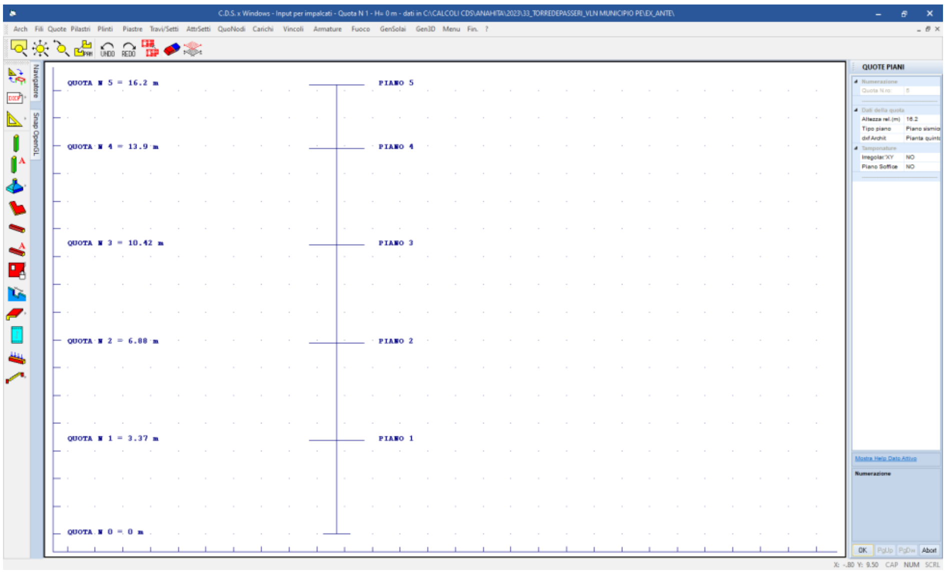


Figure 85. Floor 5-Attic floor

- To put the reference points by going to the points(fili) part and choosing the command DXF, the related plan of the floor will appear, and then it is possible to choose the desired points:

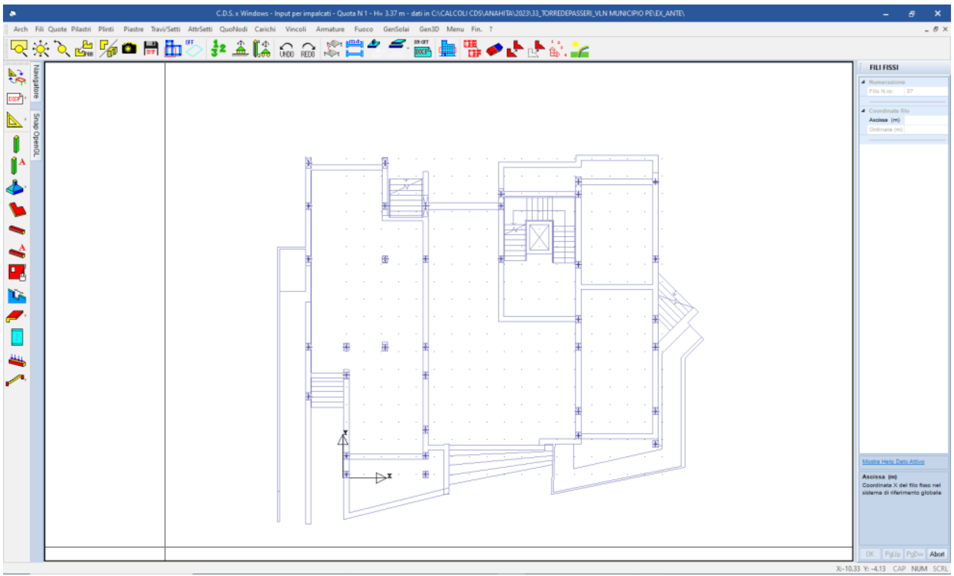


Figure 86:Underground slab in CDS win

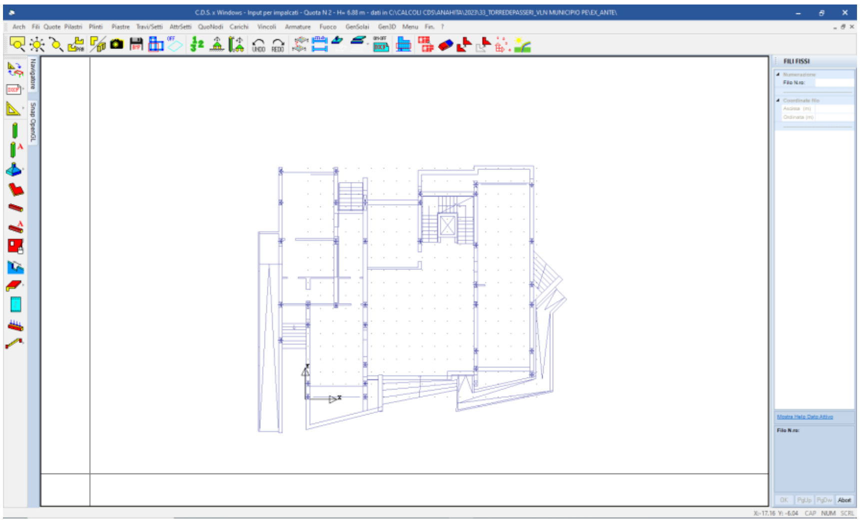


Figure 87:Ground floor slab in CDS win

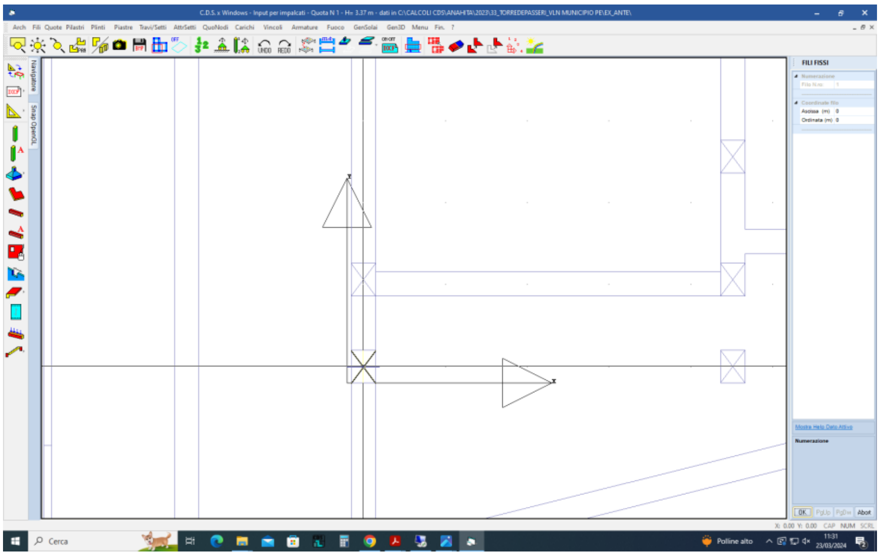


Figure 88.Positioning base points

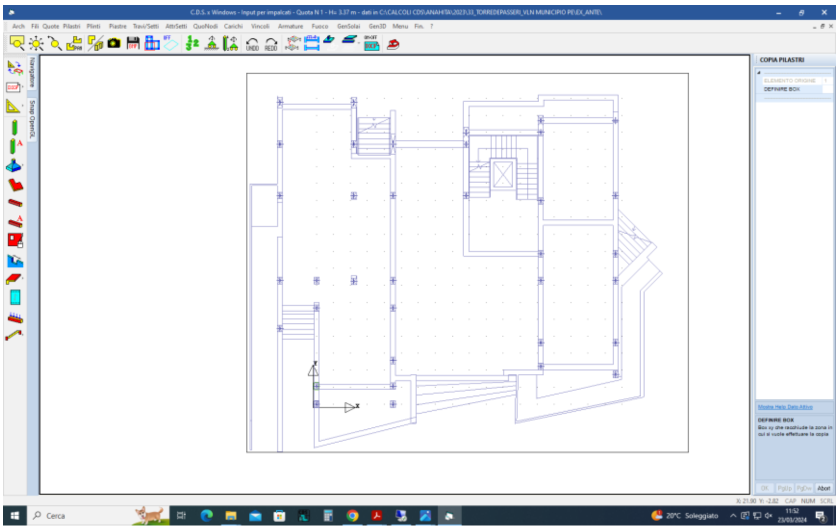


Figure 89. reference points inserted

- To model the columns in the main bar, the column(Pilastri) should be chosen. Then, for having a new section, the path is a section/new section /rectangular shape, and then the dimensions can be inserted; in the following part, it is possible to add the desired angle and position of the column according to a reference point:

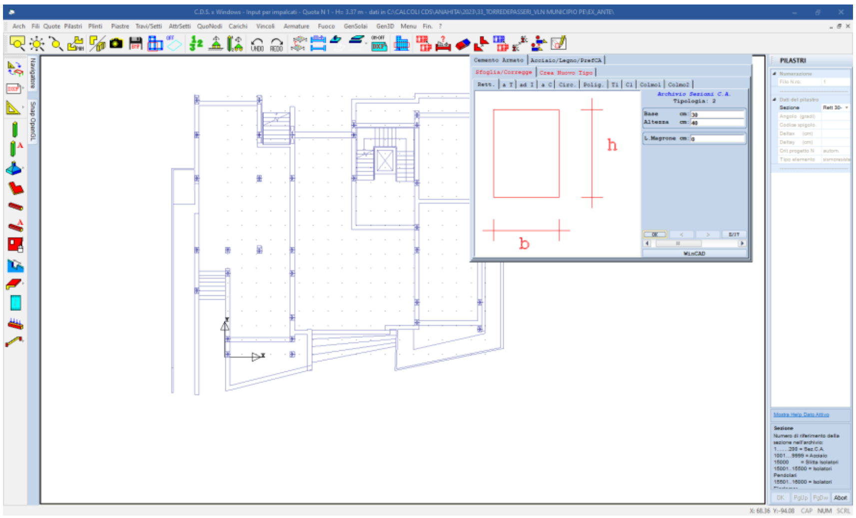


Figure90.Column with section: 30*40

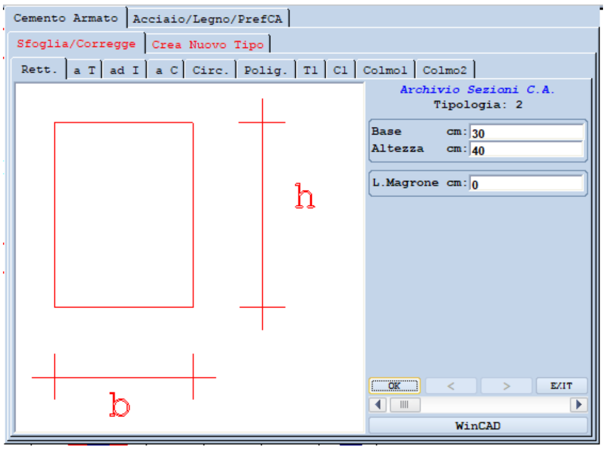


Figure 91.Creation of a New Column with section: 30*40

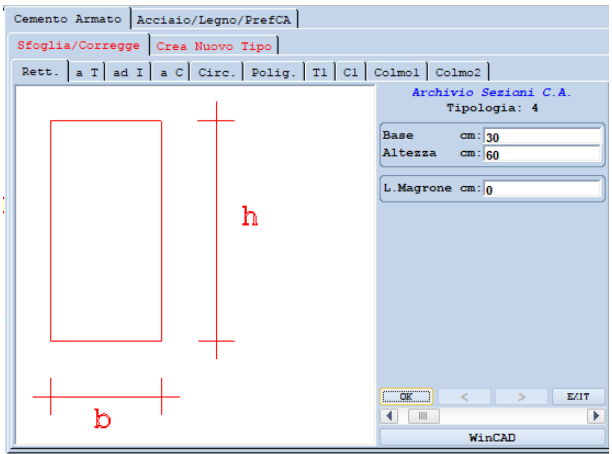


Figure 92.Creation of a New Column with section: 30*60

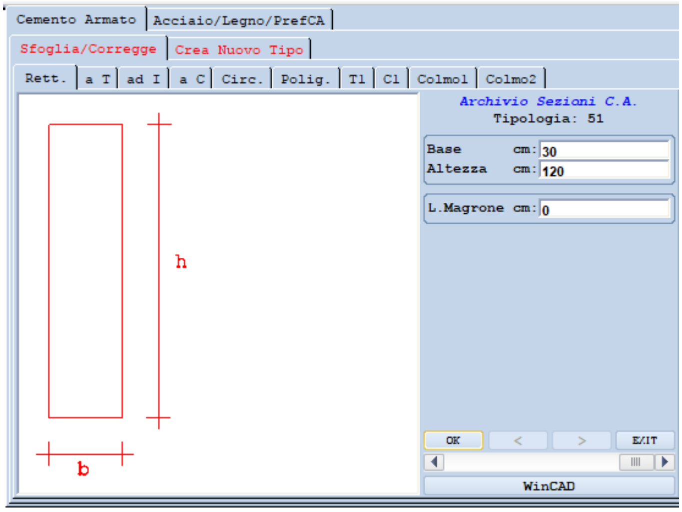


*Figure 93.Creation of a New Column with section 30*120*

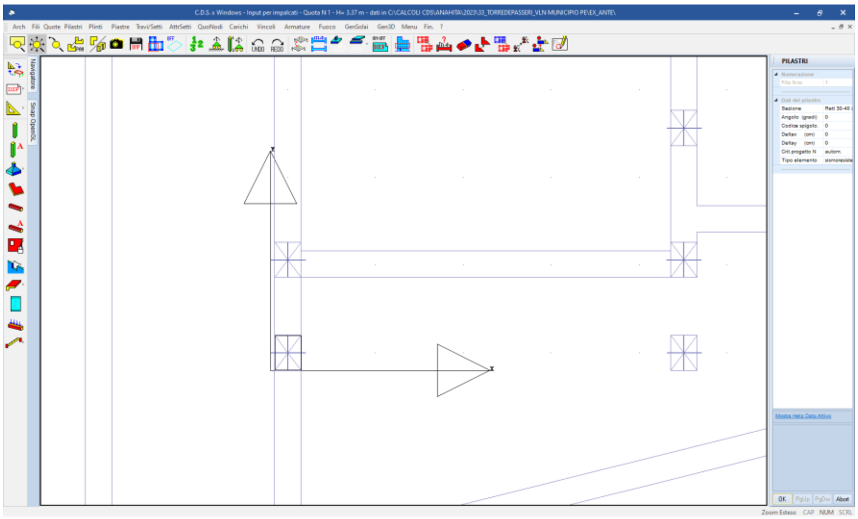


Figure 94:Column positioned

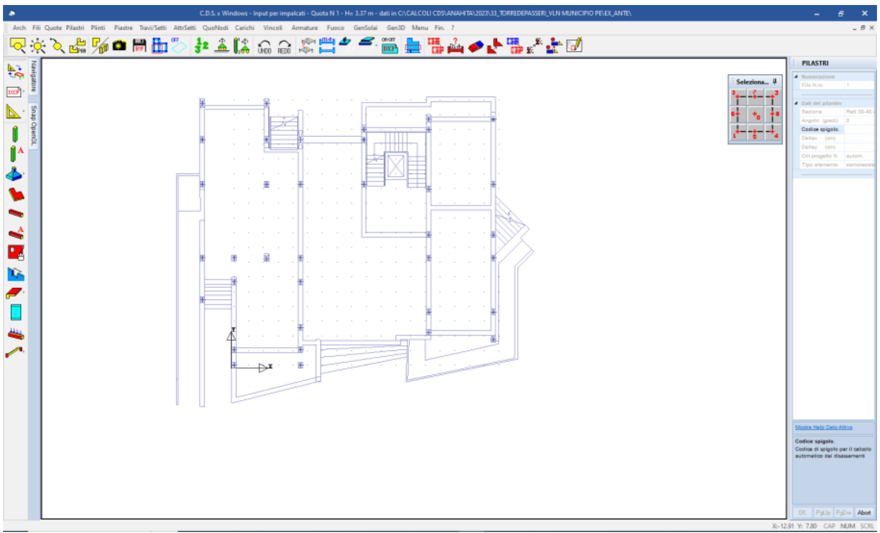


Figure 95. positioned all columns in the plan.

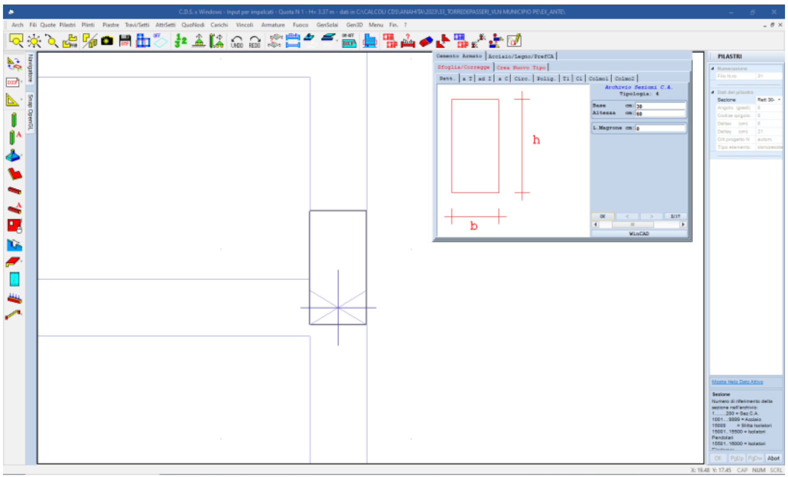


*Figure 96.Column with section: 30*60*

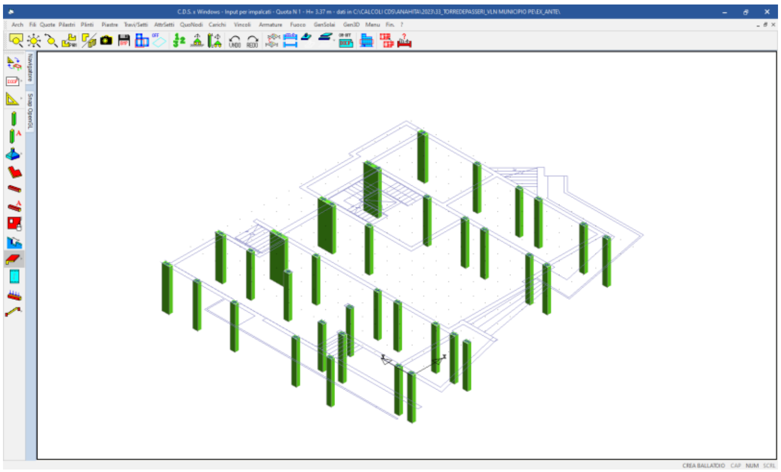


Figure 97.Columns in the one-floor

- For modeling the beams, the path is the main bar/beams(travel); for making the new section, the procedure is like the one for columns. It is necessary to provide two points, initial and final, for determining the position of the beam:

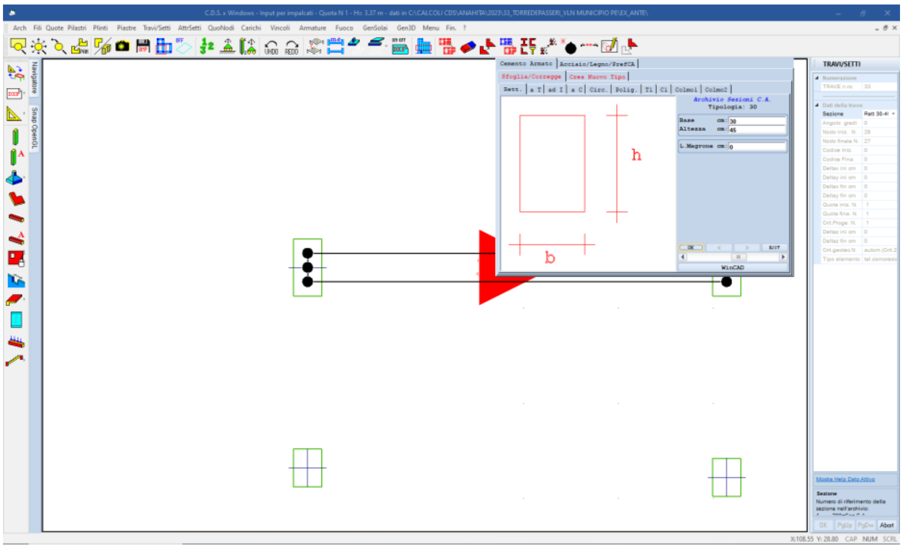


*Figure 98.Creation of New beam with section: 30*45*

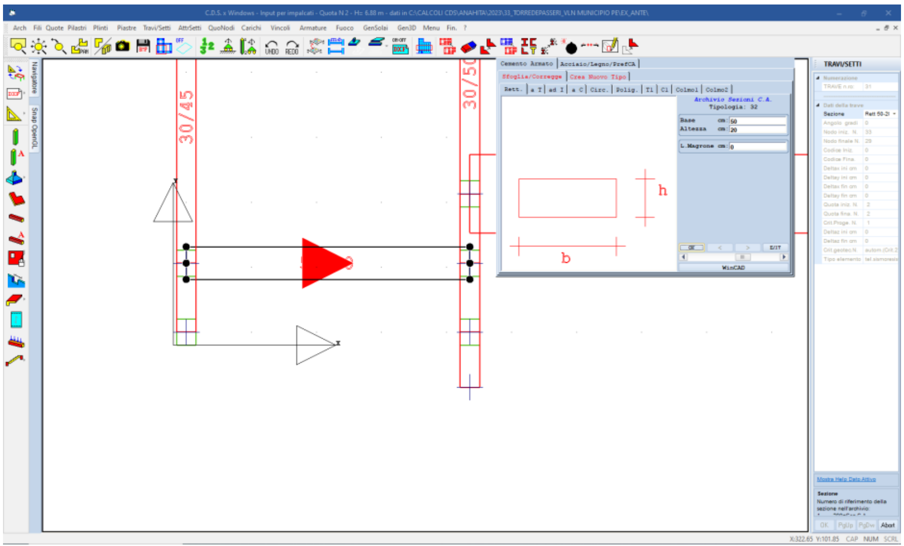


*Figure 99.Creation of New beam with section: 50*20*

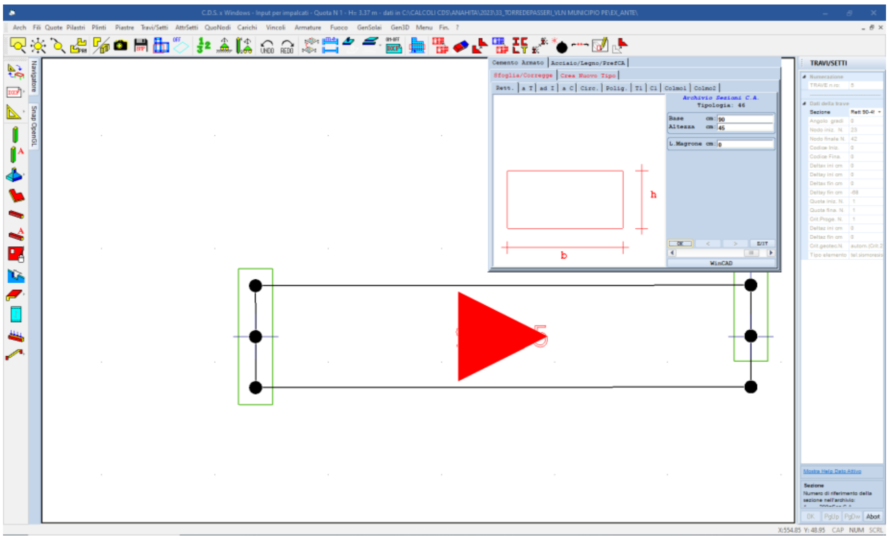


*Figure100.Creation of New beam with section: 90*45*

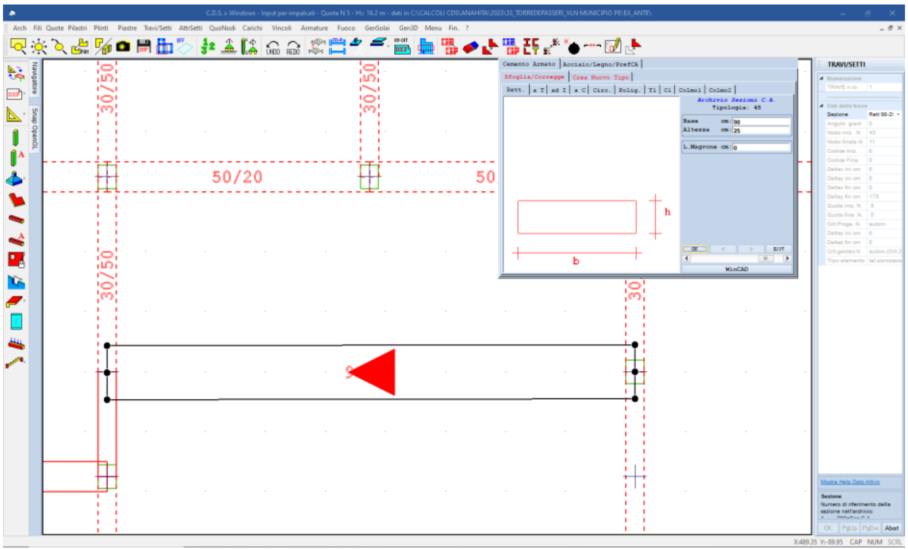


*Figure101.Creation of a New beam with section 90*25*

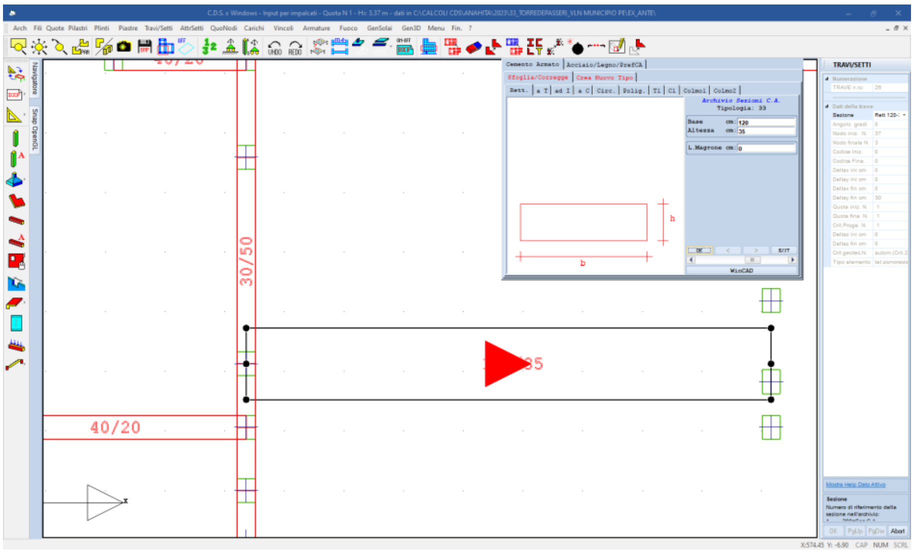


*Figure102.Creation of a New beam with section 120*35*

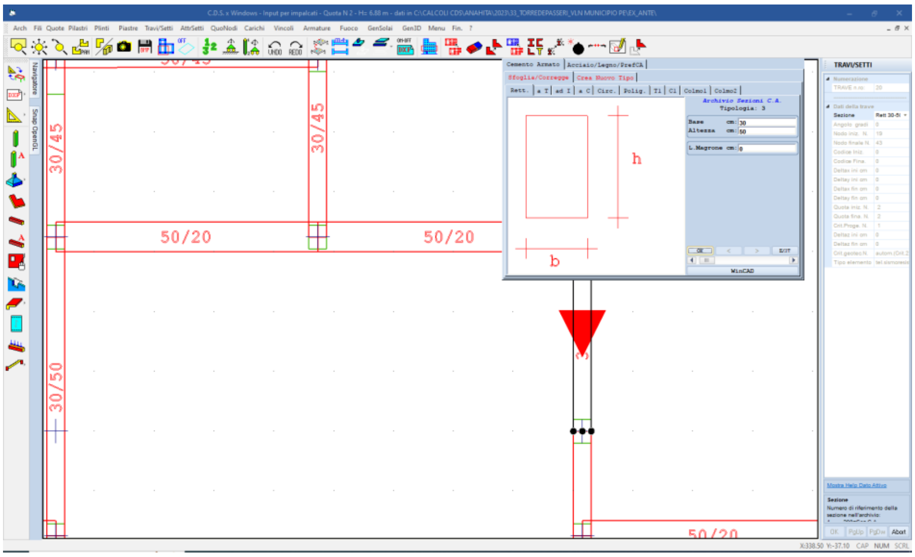


*Figure103.Creation of New beam with section: 30*50*

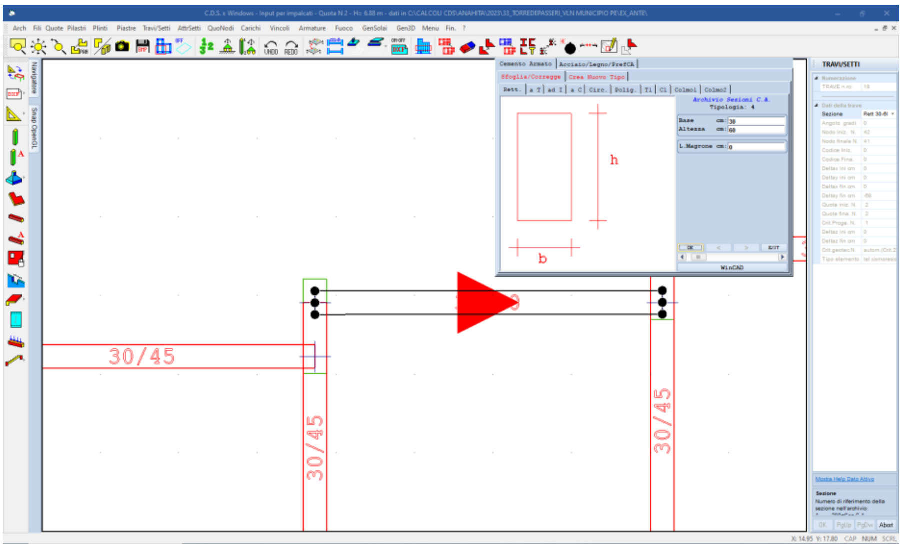


*Figure104.Creation of New beam with section: 30*60*

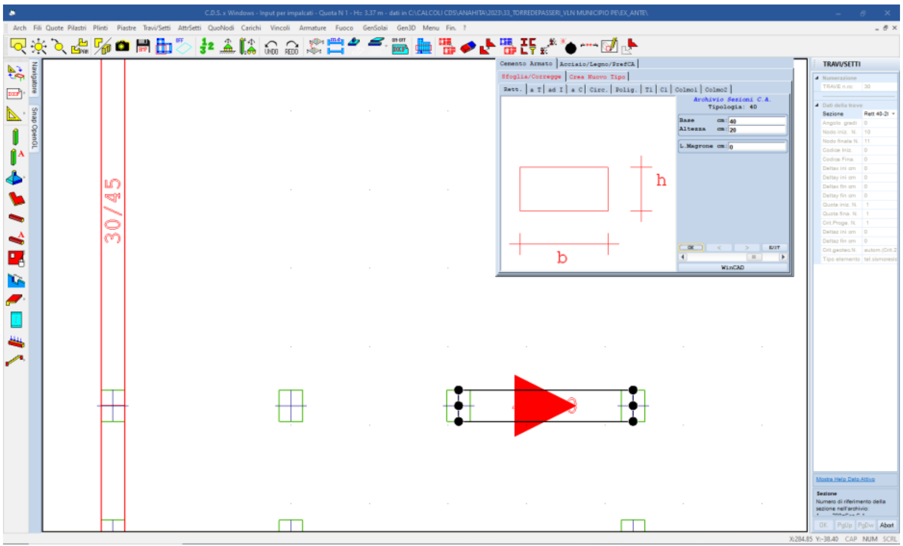


*Figure105.Creation of New beam with section: 40*20*

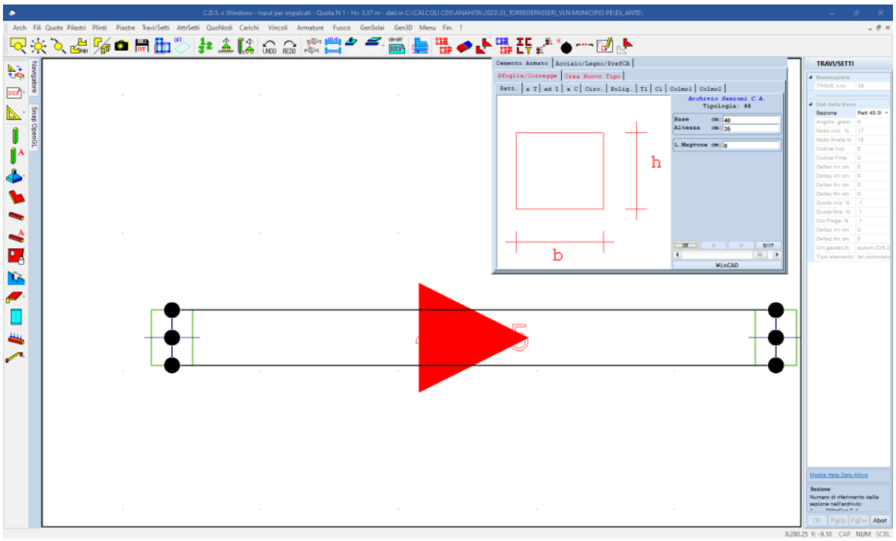


*Figure106.Creation of New beam with section 40*35*

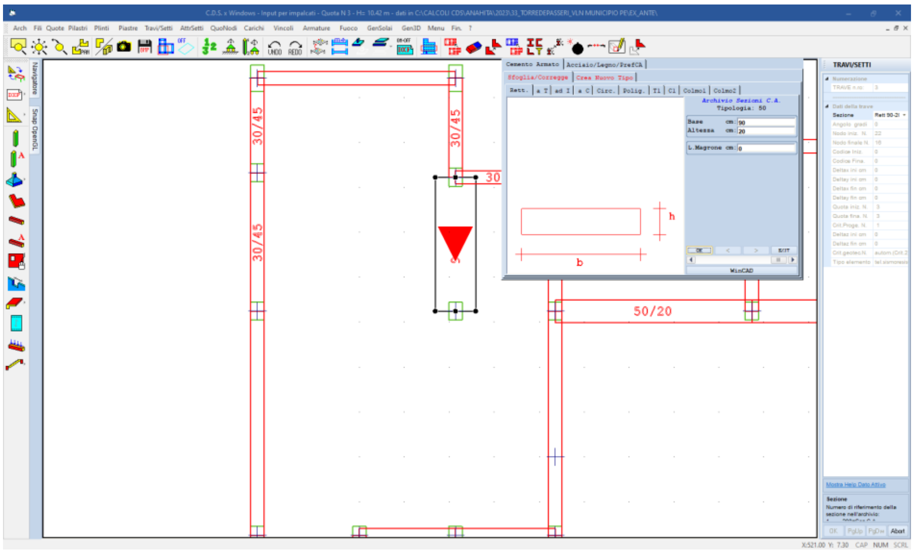


*Figure107.Creation of New beam with section: 90*20*

- Below are the figures for each floor consisting of columns and beams :

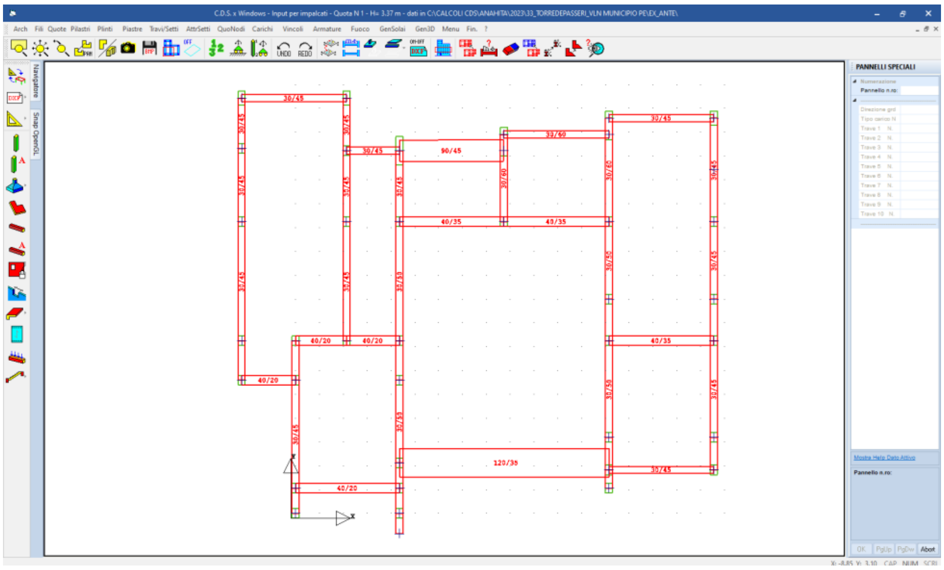


Figure108.Beams of Underground floor slab

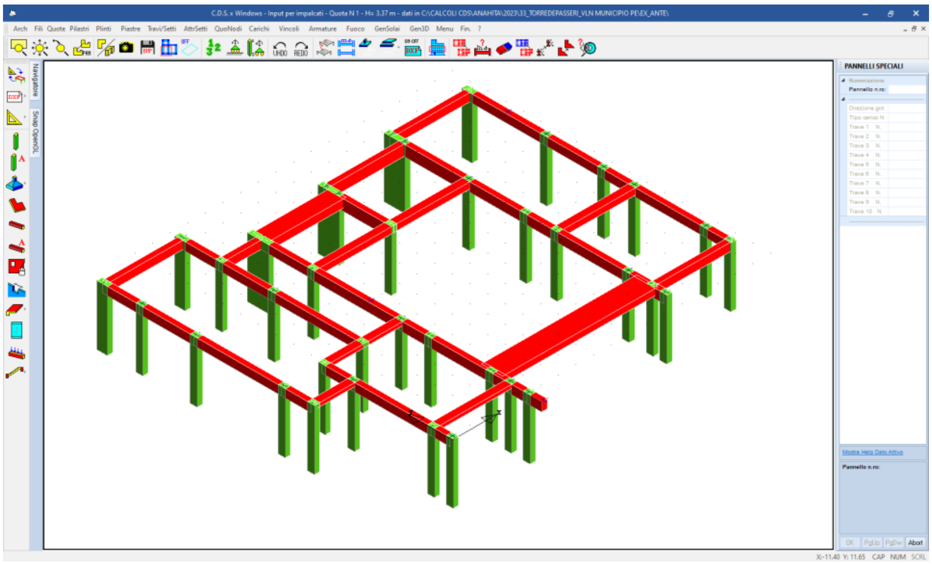


Figure 109 .3D representative of Beams and columns of underground floor

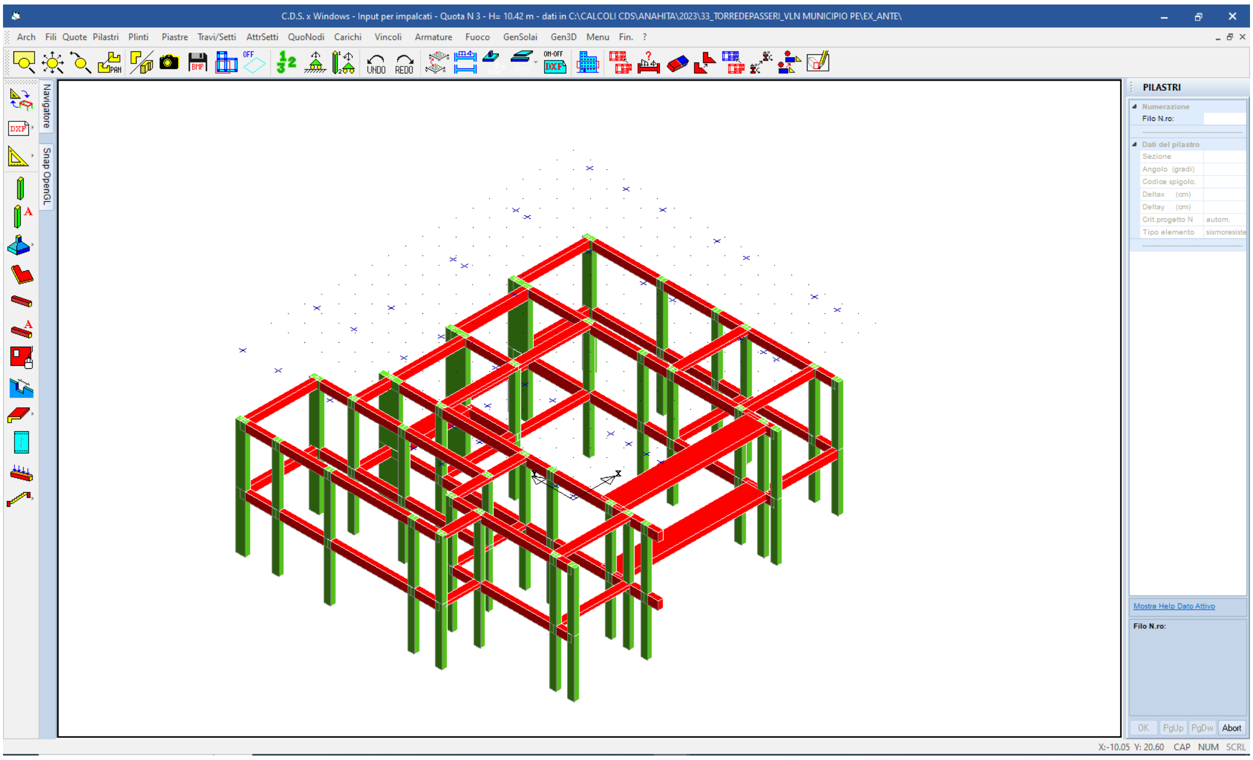


Figure 110.3D is representative of Beams and columns until the Ground floor.

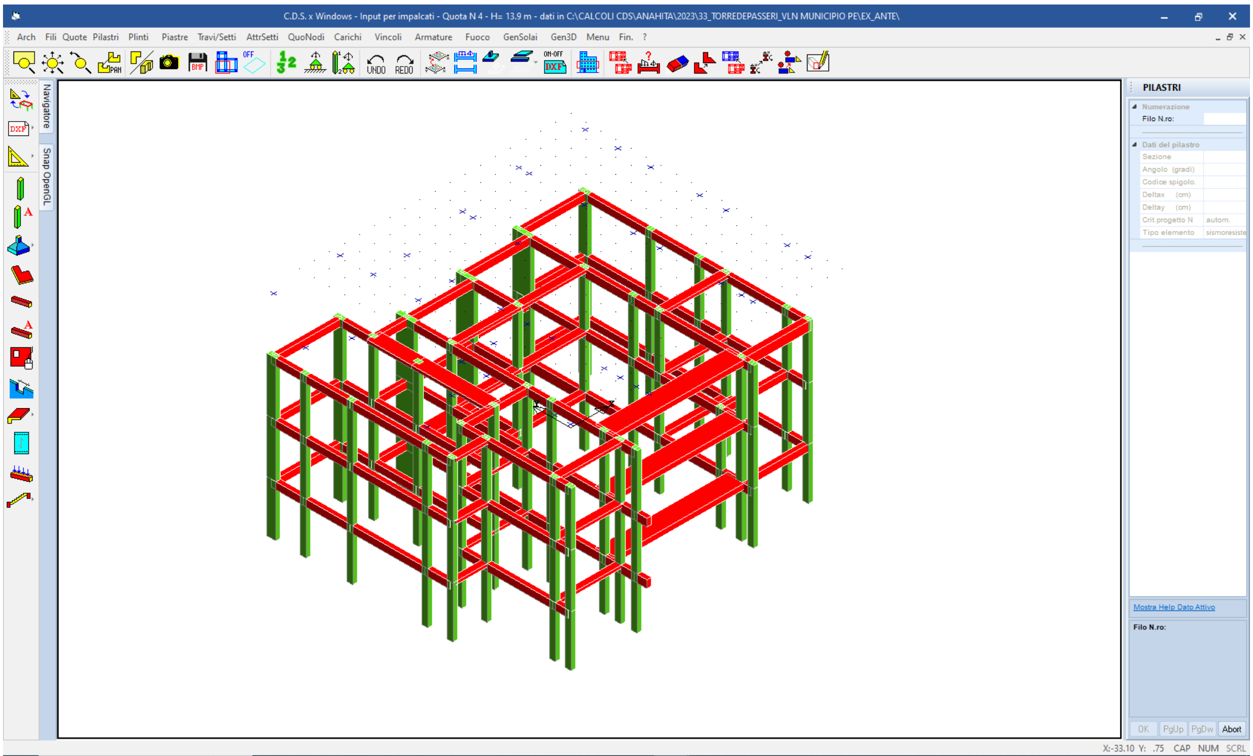


Figure111.3D representative of Beams and columns until the First floor

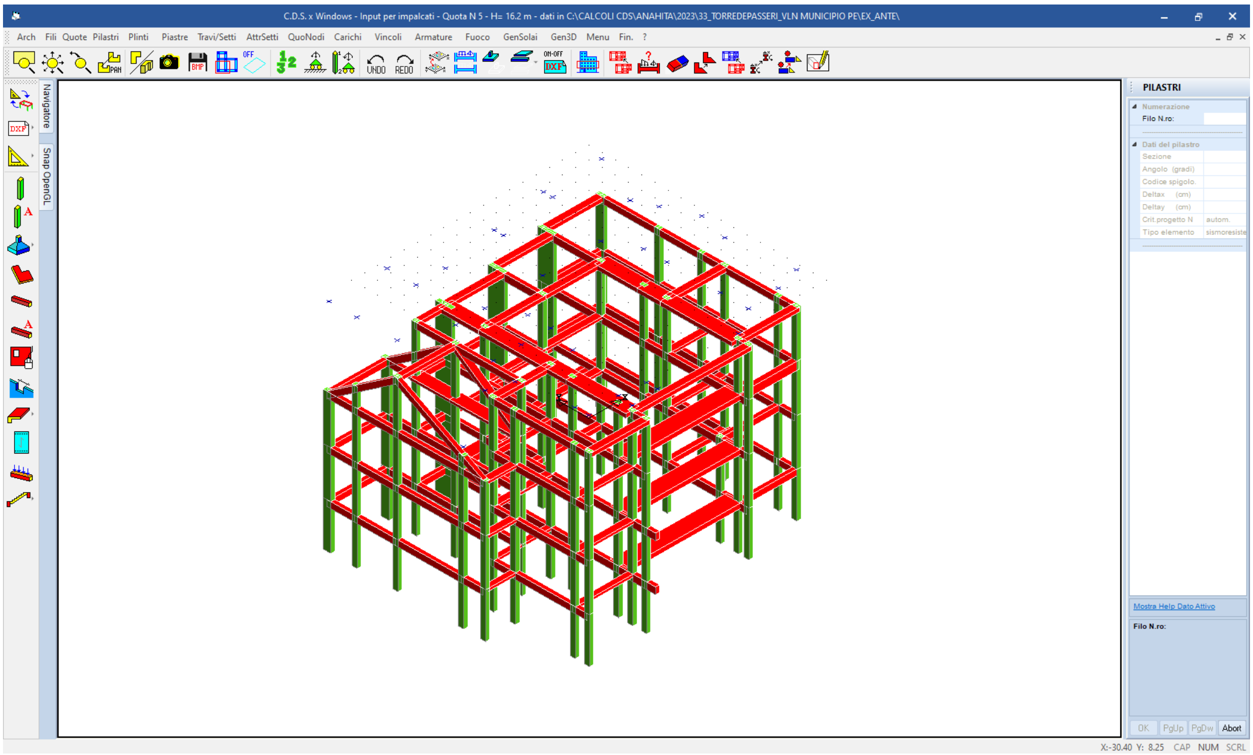


Figure 112.3D is representative of Beams and columns until the First floor.

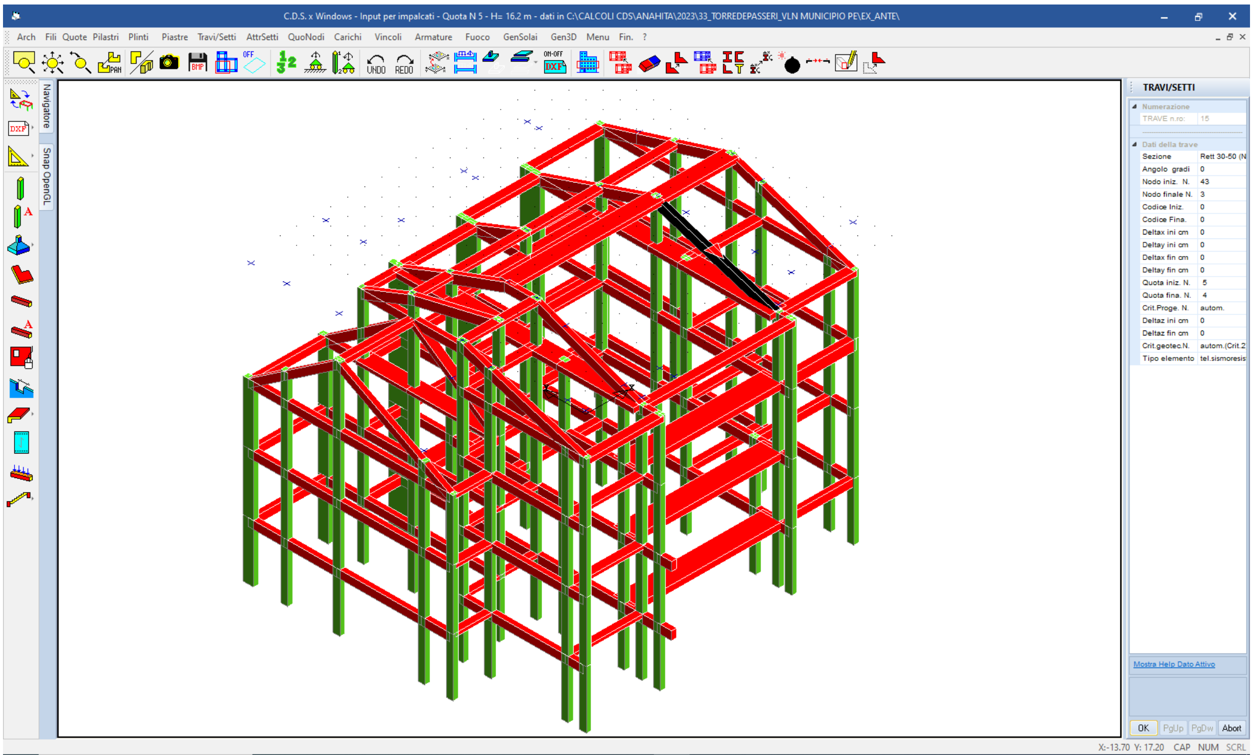


Figure113. 3D representative of Beams and columns

- In the last step of modeling columns and beams, the model should be regenerated.

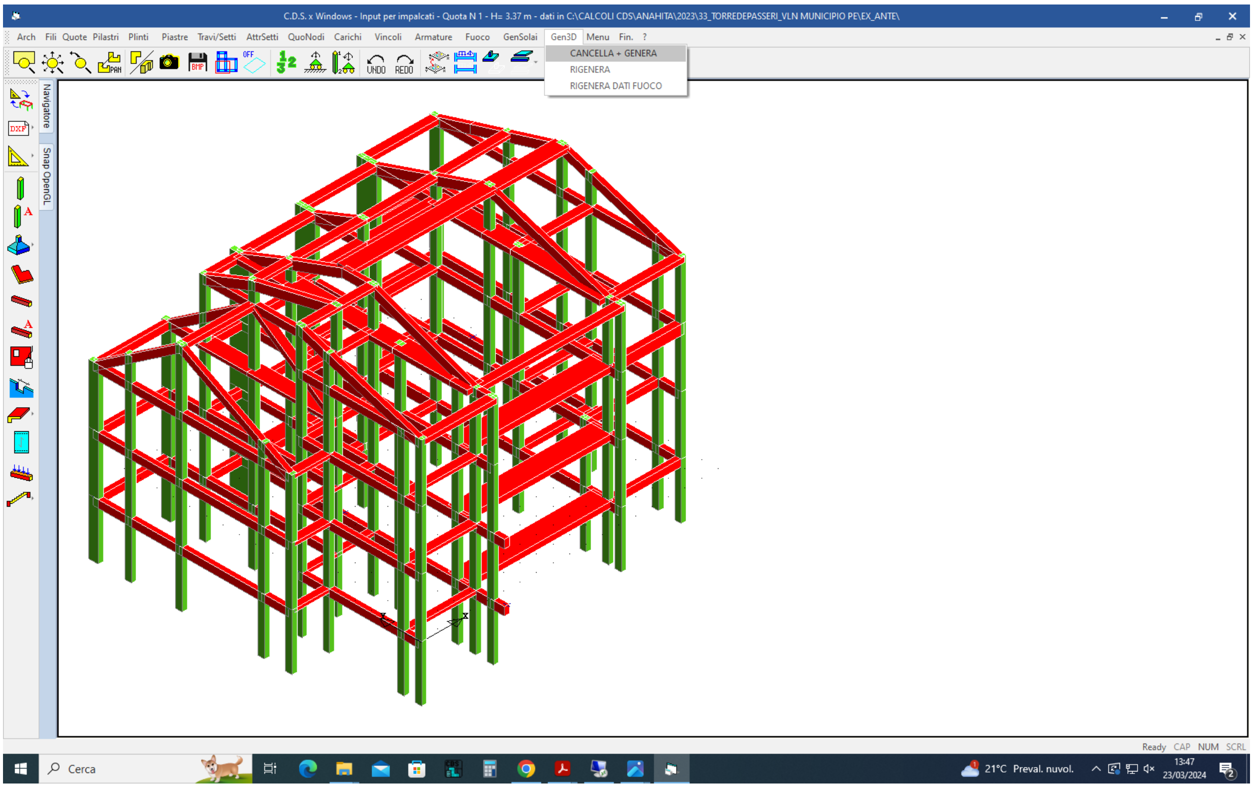


Figure114. Regeneration of Beams and columns

Loads

- For inserting and introducing the loads, the path consists of going to the main bar and load option, and then special panels are chosen. In this part, the calculated loads should be added to the relevant parts, and it's essential to select the initials and final beams.

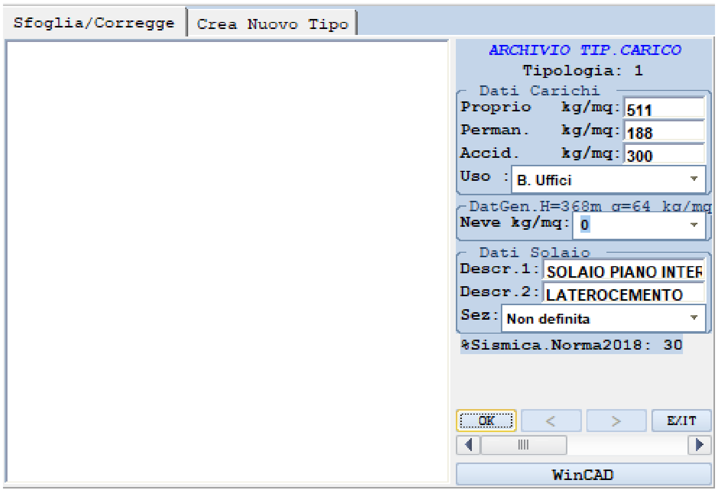


Figure115. Load of the underground floor slab

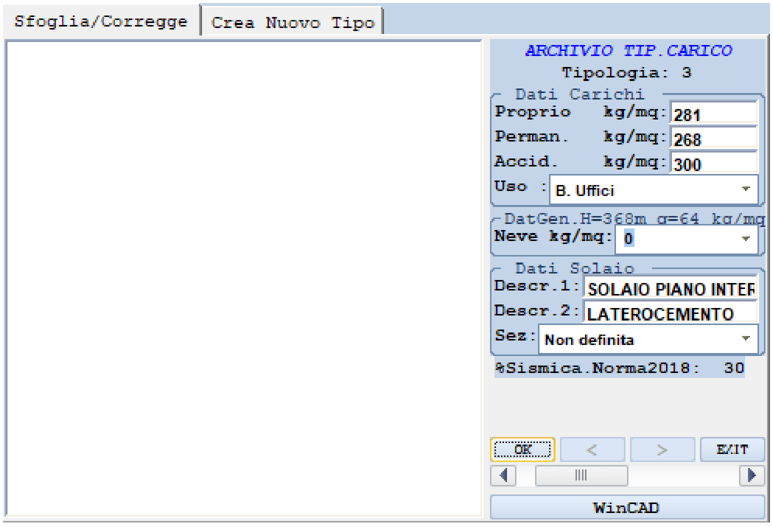


Figure 116. load of the underground floor slab(type2)

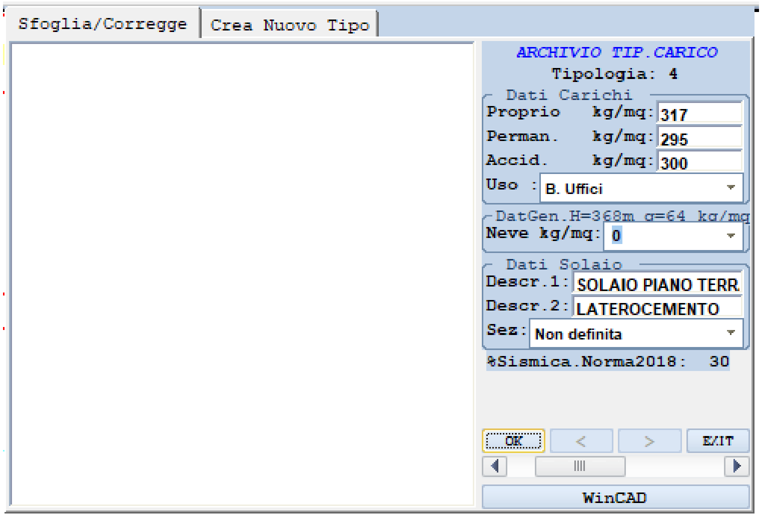


Figure 117. load of the ground floor slab

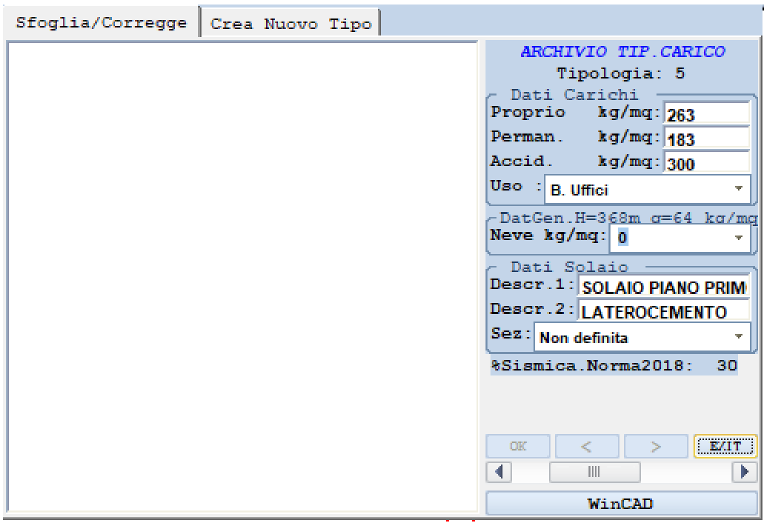


Figure 118. load of the first-floor slab

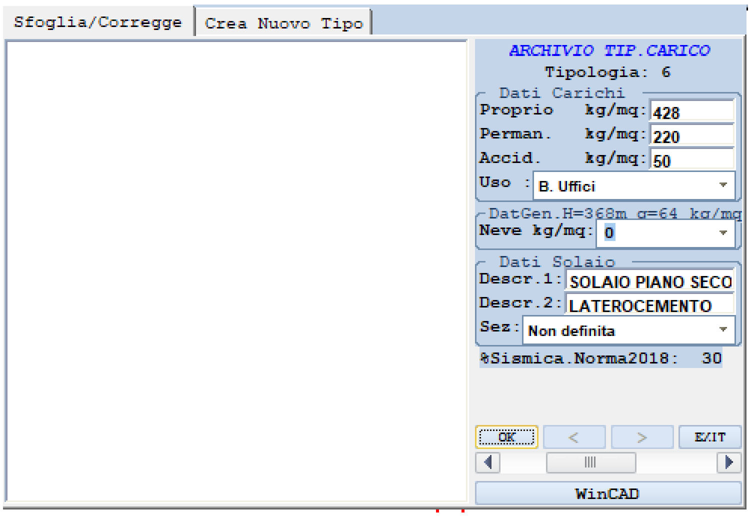


Figure119. Load of the second-floor slab

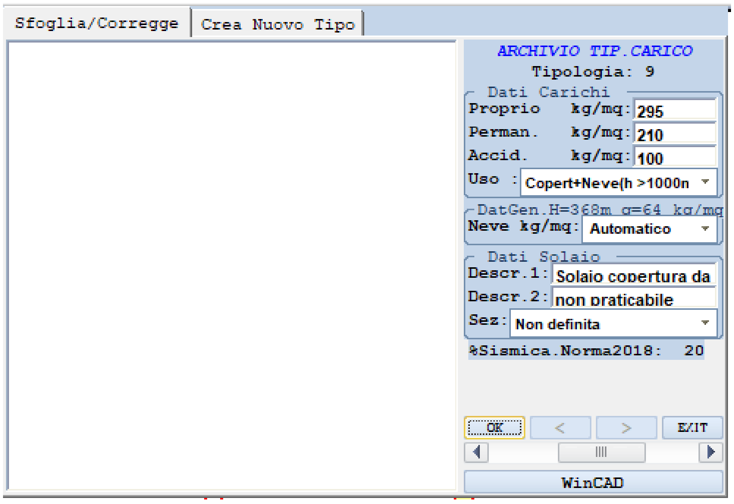


Figure120. Load of the attic floor slab

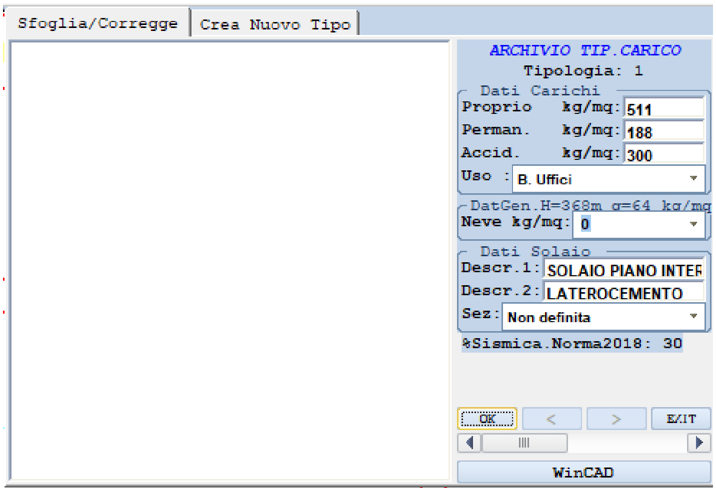


Figure121. Load of the underground floor slab (type2)

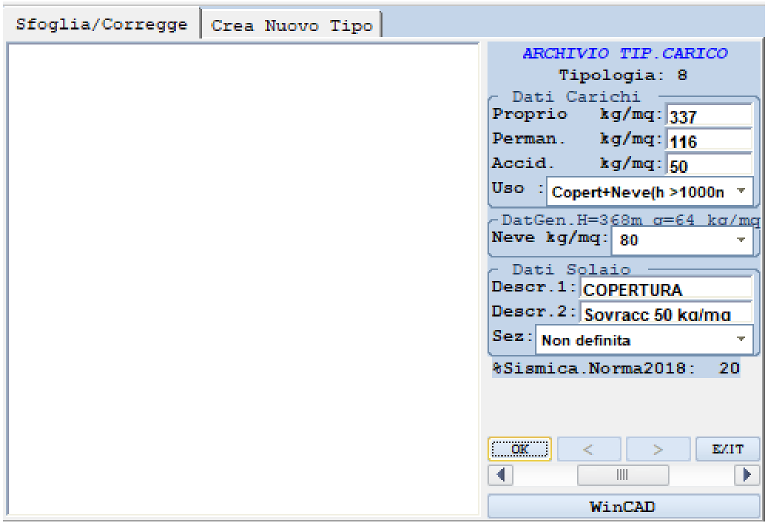


Figure122. load of the UNI2 slab

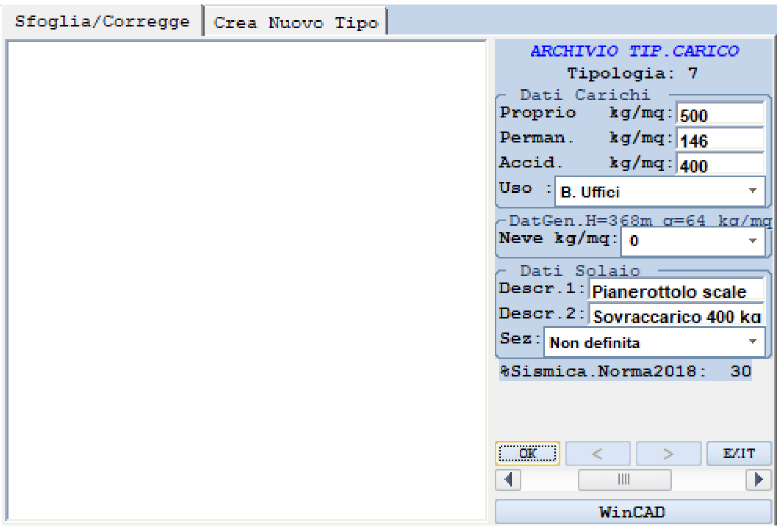


Figure123. load of the scales

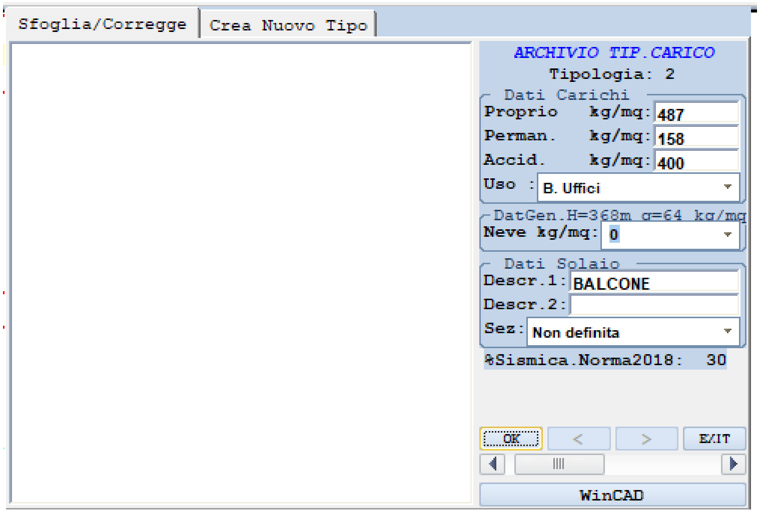


Figure124. Load of the balcony

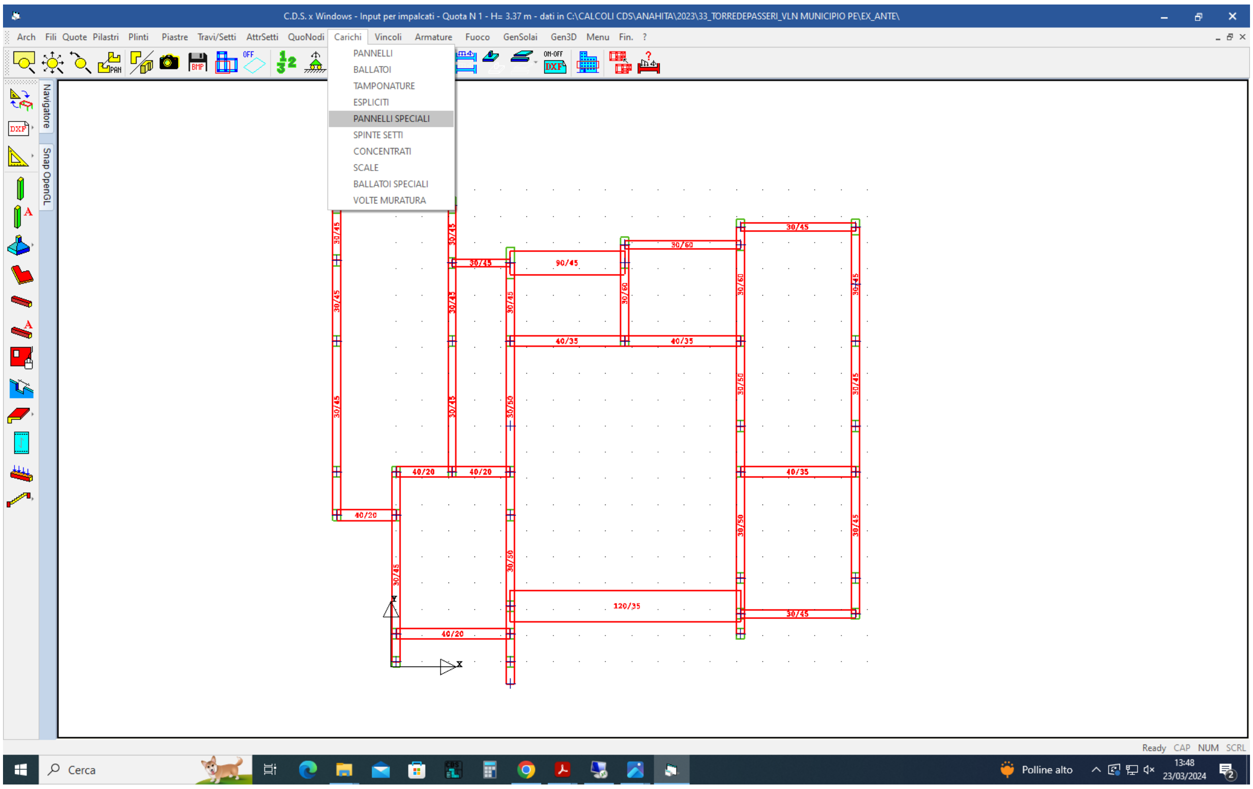


Figure125. Inserting slab loads

- In order to Insert loads of walls, this path should be as follows: loads(carichi)/walls(temperature); after making a new load, the related beam should be chosen for the load to be applied on, and the desirable height should be introduced.

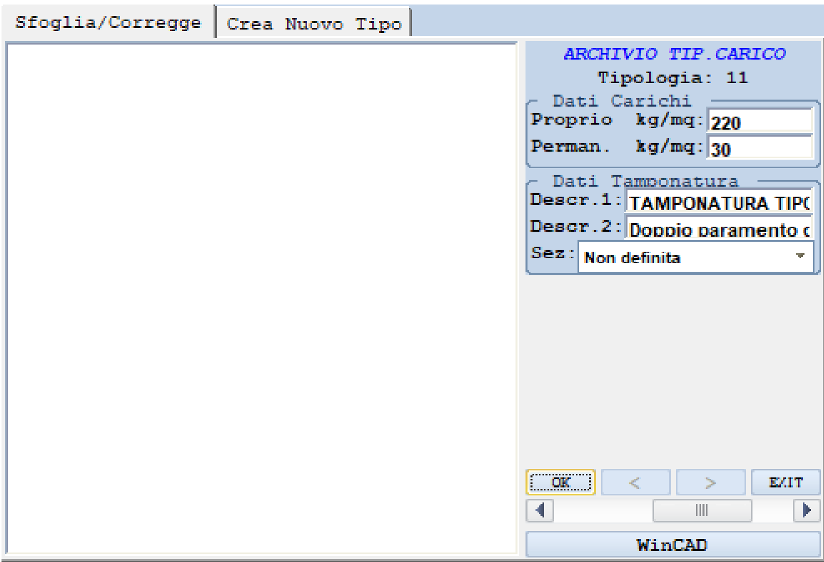


Figure126. Example of wall load creation

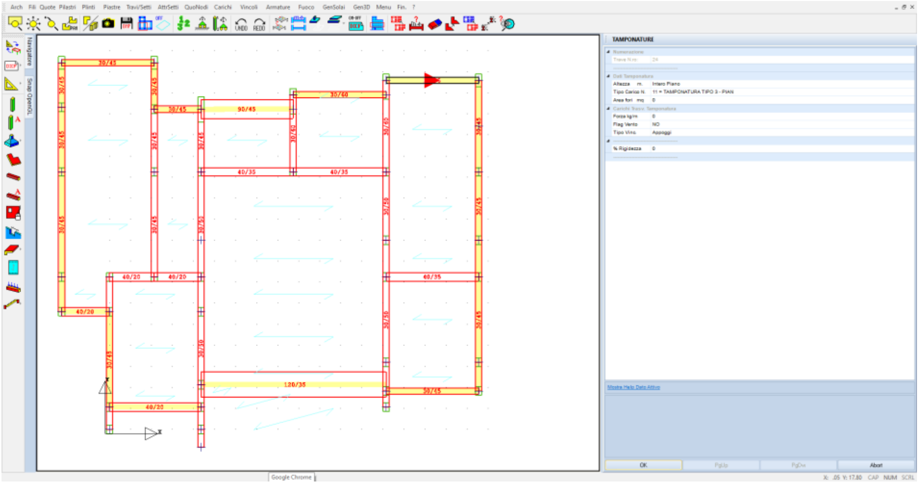


Figure127. Example of wall loads

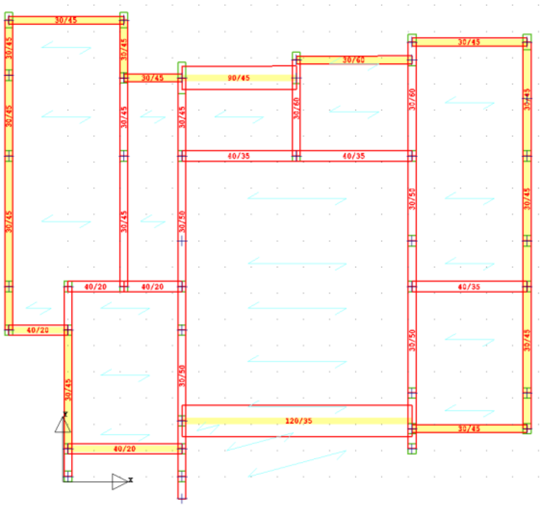


Figure128. Loads of Underground floor slab

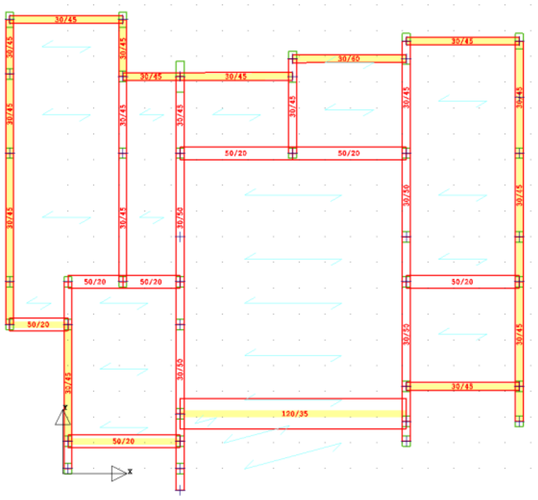


Figure129. Loads of Ground floor slab

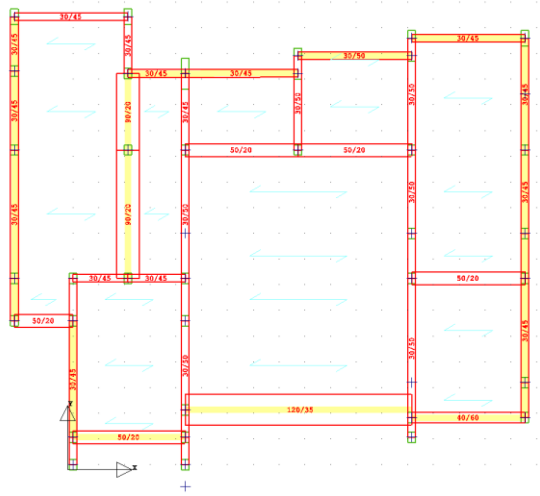


Figure130. Loads of First-floor slab

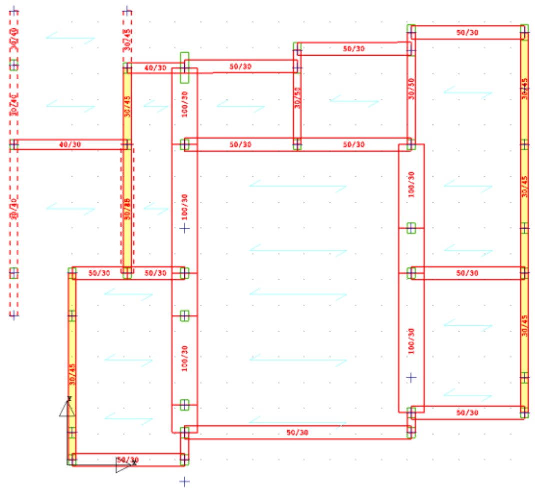


Figure131. Loads of Second-floor slab

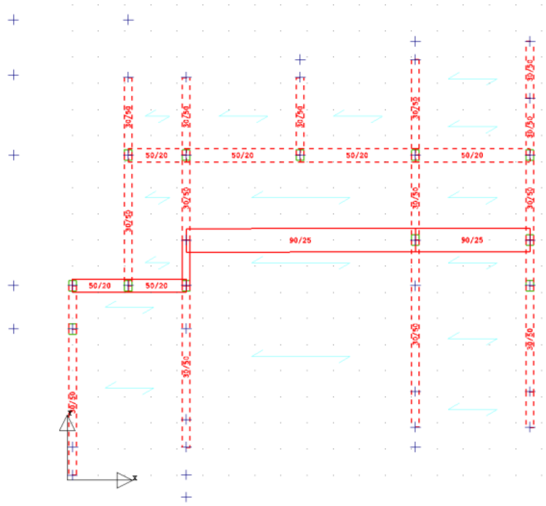


Figure132. Loads of Attic slab

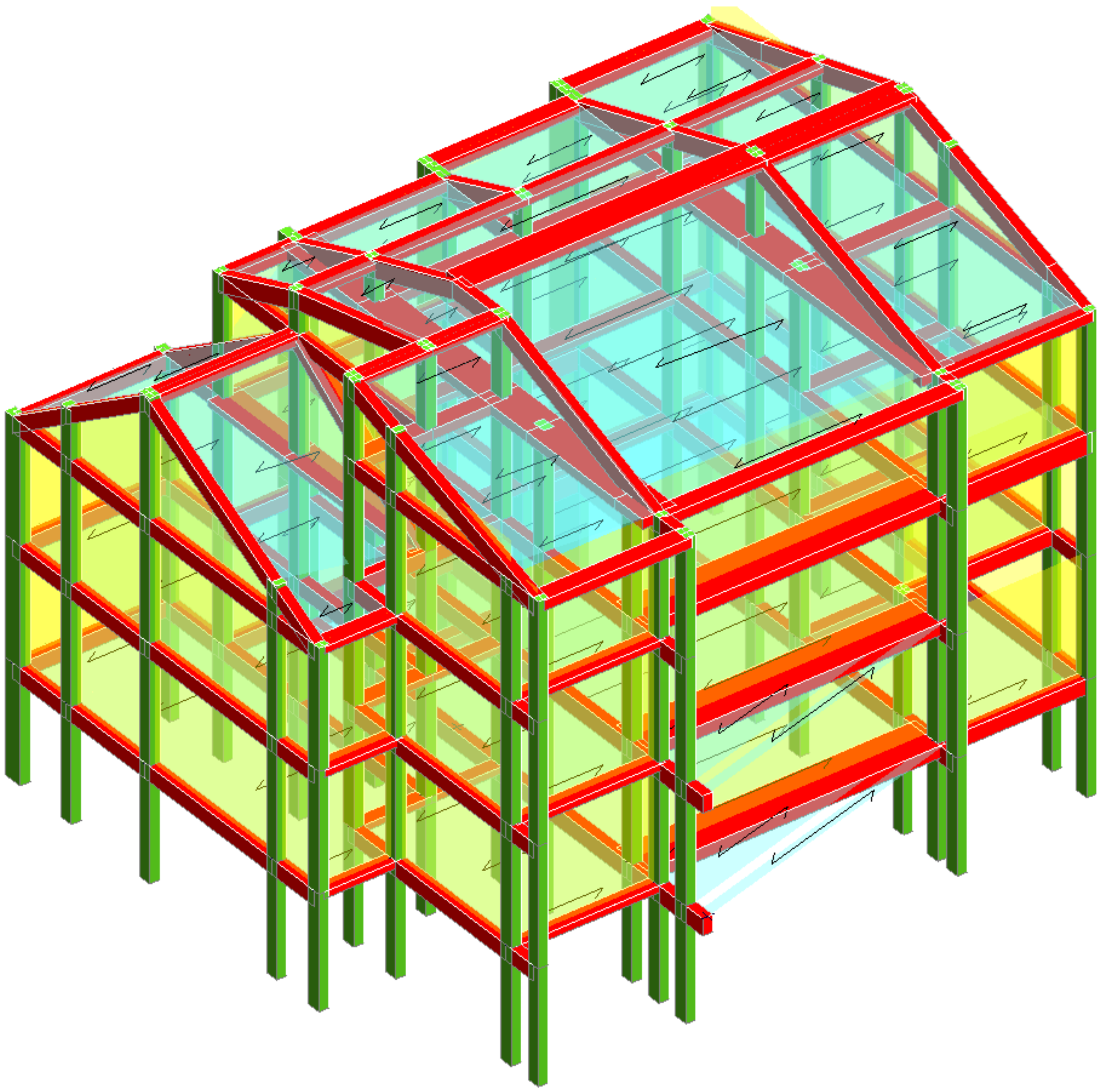


Figure 133– Geometric model US2 - Input for slabs(impalcati)

It is noted that at the basement level, the reinforced concrete walls have been defined in the mechanical model and, therefore, are not visible in the geometric model shown above.

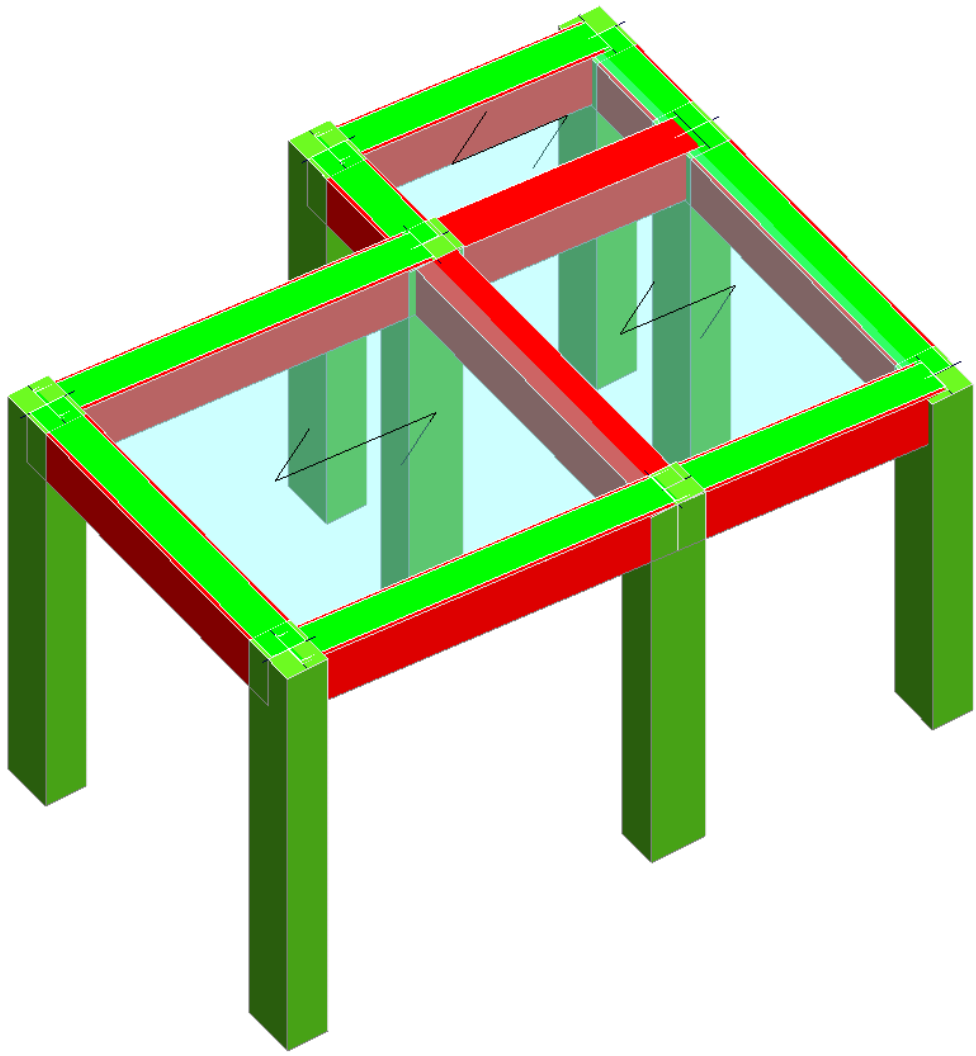


Figure 134 - Geometric model US2 - Input for slabs(impartial)

Mechanical model - US1 and US2

In the calculation software, once the geometric modeling in the 'input for floors' section is completed, the mechanical model is created in the 'spatial input' section. In this phase, the structure is understood as the set of many simple elements whose stress-strain relationship at each point is known exactly. Each element interacts with its neighbors through specific points (the nodes), where congruence and equilibrium are respected. With the finite element method, the mass and inertia of the rods are considered condensed along their own centroidal axis.

The mechanical model describes the relationship between the field of deformations and stresses, or between the generalized characteristics of stresses and the dual displacements, whose compatibility is validated through appropriate criteria or domains of resistance.

Once the nature of a given construction material is defined, the mechanical model that characterizes it consists first of all in the definition of a constitutive law (σ-ε) that interprets in a simplified way the relationship between stresses and strains, identified through specific laboratory tests.

For the sectional verifications of reinforced concrete elements, the following relationships were used:

Parabola-rectangle relationship for concrete;

Perfectly elastic-plastic or limited ductility hardening relationship for steel.

In order to evaluate the capacity of ductile or brittle elements/mechanisms, the properties of existing materials directly obtained from on-site tests and any additional information are used, divided by confidence factors. Furthermore, for the calculation of the resistance capacity of primary brittle elements, the material strengths are divided by the corresponding partial coefficients and confidence factors.

Through the mechanical model, it is possible to schematize the constraints and define the mechanical properties of each individual rod.

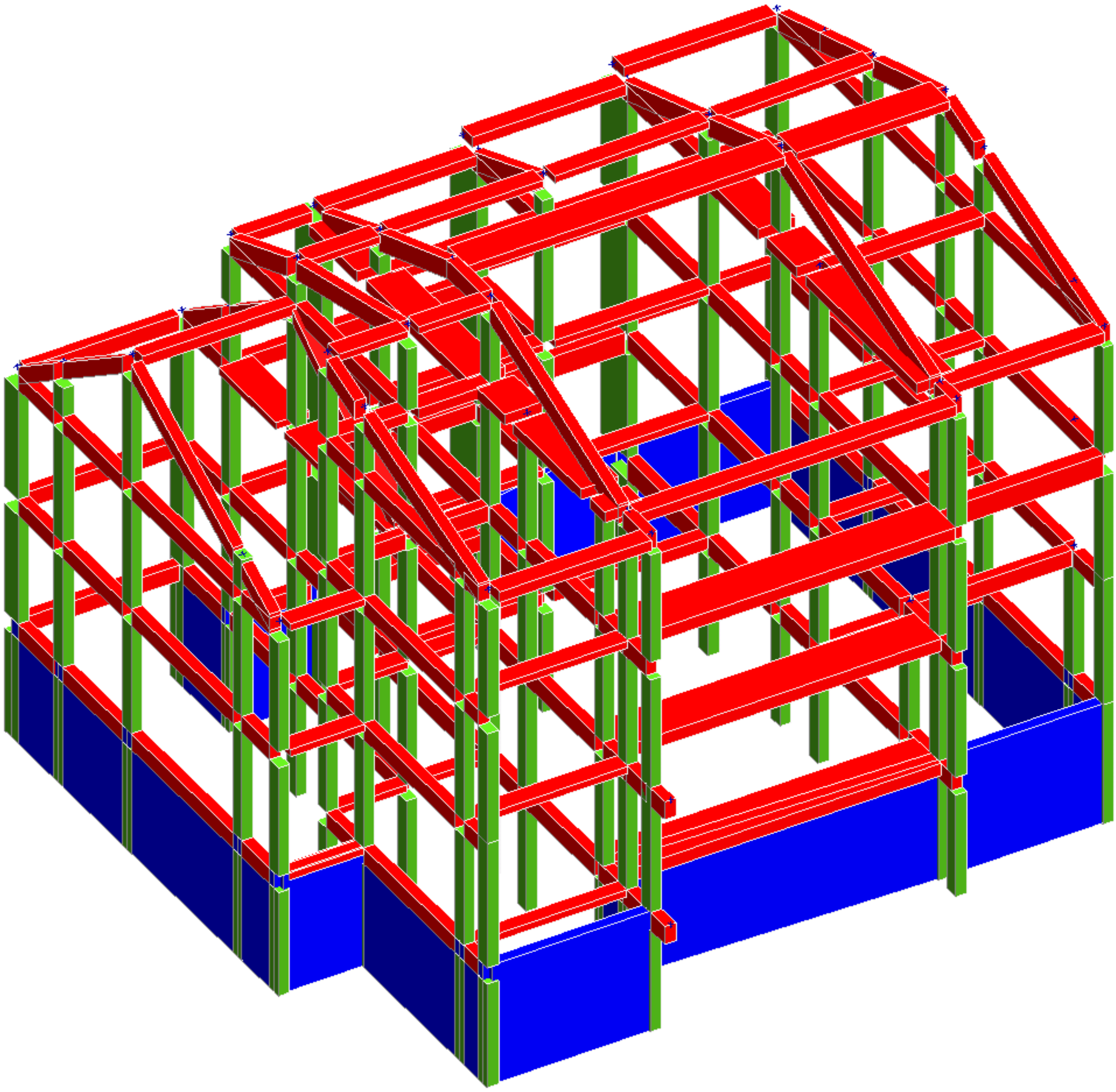


Figure 135 – Mechanical model US2 - Spatial input

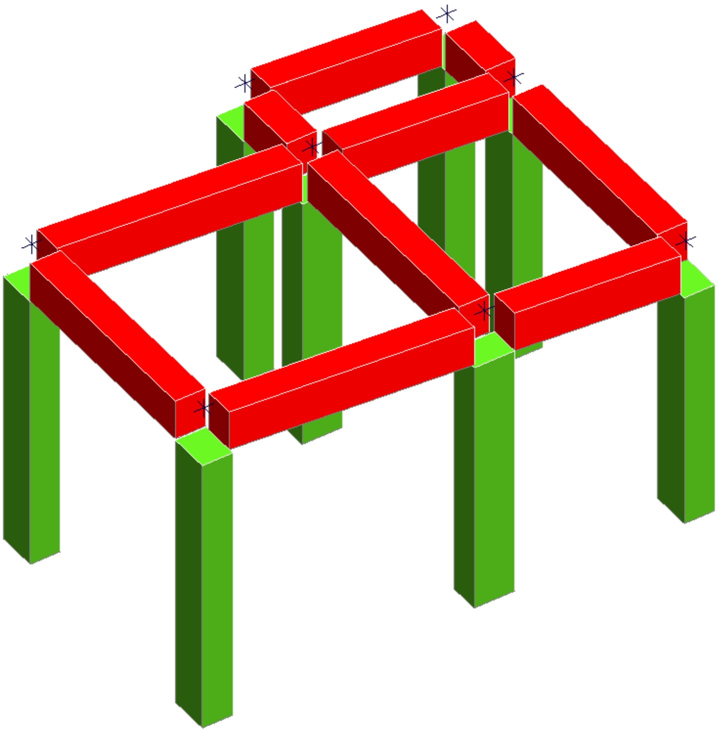


Figure 136 – Mechanical model US2 - Spatial input

12 ADOPTED ANALYSIS METHODS

The analysis methods used are static analysis for evaluating the safety of the structure, linear dynamic analysis to assess the main modes of the structure, and nonlinear static analysis (Push-over) for the overall assessment of the seismic vulnerability of the structure.

12.1 Non-seismic static analysis

The non-seismic static analysis involves evaluating the structure's response to gravitational loads due to the self-weight of the structural elements (G1), non-structural permanent loads (G2), and variable loads (Qk). The fundamental combination used for the ultimate limit states (ULS) is as follows.

$\gamma_G \cdot G{,}1 + \gamma_G \cdot G{,}2 + \gamma_P \cdot P + \gamma_Q \cdot Q_{k,1} + \sum \psi_0 \gamma_Q Q_{k,i}$

The definition of actions complies with what is formulated in §2.5.1.3 and §2.5.2; in particular, Qk,1 is the dominant variable action, while Qk,2, Qk,3, ..., are variable actions that can act simultaneously with the dominant one. The variable actions Qk,j are combined with combination coefficients ψ whose values are provided in §2.5.3, Table 2.5. I

12.2 Dynamic linear analysis

The free oscillations of a linear elastic system can be considered as the superposition of "simple oscillations," each of which corresponds to a well-defined shape or deformation (mode), i.e., such that the ratio of displacements of any two parts remains constant over time. Each oscillation corresponds to a period. The study of the dynamics of the elastic structure through its principal modes is called modal analysis.

The modes of vibration depend on the stiffness and inertial (mass) characteristics of the system and are calculated by solving specific eigenvalue problems.

The analysis considers all vibration modes that provide a significant contribution to the dynamic response of the structure. In this regard, it is helpful to emphasize that the current standard (see § 7.3.3.1) requires that all modes with significant participating mass be considered. This criterion is considered satisfied if the sum of the effective modal masses, for all considered modes, amounts to a significant percentage of the structure (85%) or if all modes with participating mass greater than a minimum percentage (5%) are considered.

Each of the identified vibration modes is associated with a participation coefficient which, in turn, in relation to the design spectrum, allows evaluating the maximum vectors of equivalent static forces related to the various modes.

The maximum probable value of any effect (displacement, stress, etc.) is given by statistical derivation formulas. The most commonly used combinations of seismic responses to obtain maximum effect values are: SRSS (square root of the sum of the squares of the modal responses Ei) and CQC (complete quadratic combination).

The calculation of seismic forces to be applied to the structure depends on the design spectra, i.e., relationships that provide the structural acceleration as a function of some factors, the main ones being the period of the structure, ground acceleration, and soil characteristics.

In the case of modal analysis, the calculation of response parameters is performed with reference to the dynamic characteristics of the structure (its natural vibration modes).

12.3 Non-linear static analysis

Nonlinear static analysis involves applying gravitational loads to the structure and, for the considered direction of seismic action, a system of distributed horizontal forces at each level of the building, proportional to the inertial forces and having a resultant (base shear) Fb. These forces are scaled to monotonically increase, both positively and negatively, until reaching local or global collapse conditions, the horizontal displacement dc of a control point coinciding with the center of mass of the top level of the building (excluding any towers). The Fb - dc diagram represents the capacity curve of the structure.

At least two distributions of inertial forces must be considered, falling into the main distributions (Group 1) and the secondary distributions (Group 2), as illustrated below.

Group 1 - Main distributions:

If the fundamental mode of vibration in the considered direction has a mass participation of not less than 75%, one of the following two distributions is applied:

- Distribution proportional to the static forces as described in § 7.3.3.2, using the a) distribution of Group 2 as the second distribution;

- Distribution corresponding to an acceleration pattern proportional to the shape of the fundamental mode of vibration in the considered direction;

In all cases, the distribution corresponding to the pattern of floor forces acting on each level calculated in a linear dynamic analysis can be used, including in the considered direction a number of modes with a total mass participation of not less than 85%. The use of this distribution is mandatory if the fundamental period of the structure exceeds 1.3 TC.

Group 2 - Secondary distributions:

a) Force distribution, derived from a uniform acceleration pattern along the height of the building;

b) Adaptive distribution, which changes as the displacement of the control point increases depending on the plastic behavior of the structure;

c) Multi-modal distribution, considering at least six significant modes.

Nonlinear static analysis allows determining the capacity curve of the structure, expressed by the relationship Fb - dc, where Fb is the base shear, and dc is the displacement of a control point, which for buildings is typically represented by the center of mass of the top level. For each considered limit state, comparing the capacity curve with the displacement demand allows for determining the level of performance achieved. For this purpose, a real structural system is associated with an equivalent single-degree-of-freedom structural system.

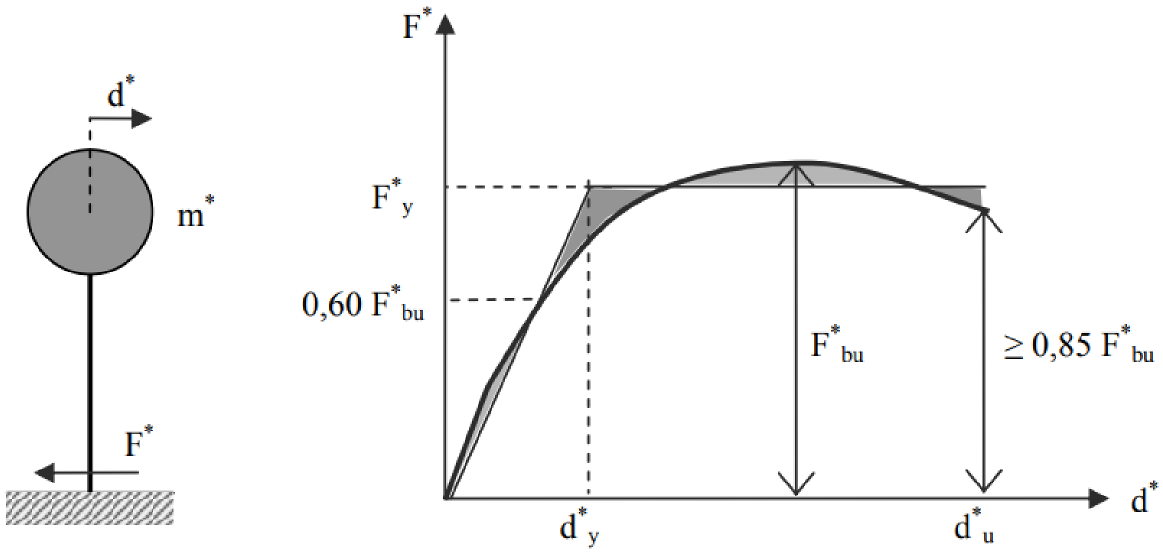


Figure 137 – Equivalent Bilinear System and Diagram

The force F^* and displacement d^* of the equivalent system are related to corresponding quantities Fb and dc of the real system by the relationships:

$$F^* = F_b / \Gamma$$

$$d^* = d_c / \Gamma$$

Where is the "modal participation factor" defined by the relationship:

$$\Gamma = \frac{\varphi^T M \tau}{\varphi^T M \varphi}$$

Where:

ϕ_i is the i-th mode shape vector.

M is the mass matrix of the system.

r is the vector representing the distribution of the applied forces (often the influence vector).

The capacity curve of the equivalent system now needs to be replaced by a bilinear curve with a first elastic segment and a second perfectly plastic segment. Let F_{bu} be the maximum strength of the real structural system and $F_{bu} = F_{bu}/$ the maximum strength of the equivalent system. The elastic segment is identified by requiring it to pass through the point 0.6 F_{bu} on the capacity curve of the equivalent system, and the yielding force Fy* is identified by setting the equality of the areas under the bilinear curve and the capacity curve for the maximum displacement du corresponding to a reduction in strength $\leq 0.15\ F_{bu}$.

The elastic period of the bilinear system is given by the expression:

$$T^* = 2\pi\sqrt{\frac{m^*}{k^*}}$$

Where $m^* = \Phi^T M\tau$ e k^* is the stiffness of the elastic segment of the bilinear curve. In case the elastic period of the building T^* satisfies $T^* \geq T_C$ l, the displacement demand for the inelastic system is assumed to be equal to that of an elastic system with the same period.

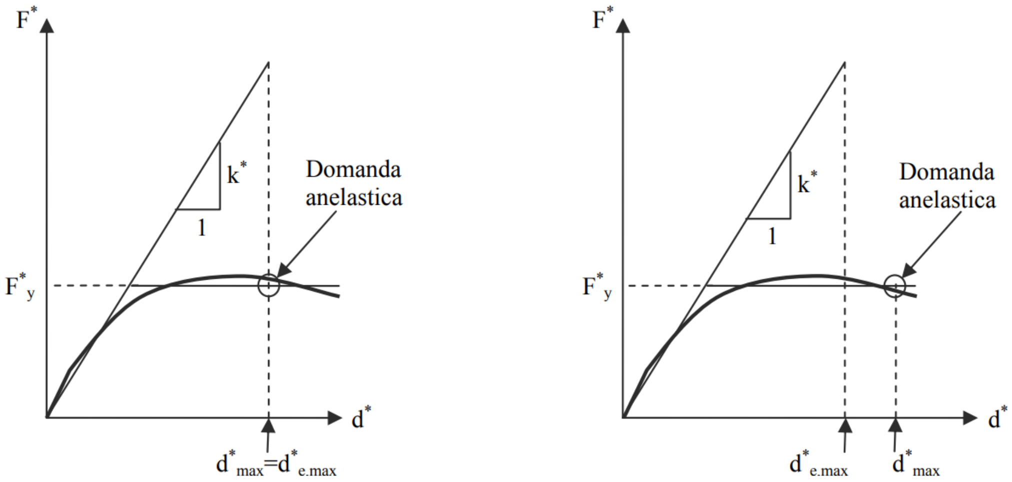
$$d^*_{max} = d^*_{e,max} = S_{De}\left(T^*\right)$$

In the case of $T^* < T_C$, The displacement demand for the inelastic system is greater than that of an elastic system with the same period and is obtained from the latter using the expression:

$$d^*_{max} = \frac{d^*_{e,max}}{q^*}\left[1+\left(q^*-1\right)\frac{T_C}{T^*}\right] \geq d^*_{e,max}$$

where $q^*=Se(T^*)\, m^* / F^*y$

It represents the ratio between the elastic response force and the yielding force of the equivalent system. If $q^* \leq 1$ then $d^*max = d^*e,max$.



Spostamento di riferimento per $T>T_C$ *Spostamento di riferimento per* $T \leq T_C$

The accidental torsional effects are considered as prescribed in § 7.2.6 of the NTC. Once the displacement demands d^*_{max} for the limit state under consideration is found, it is verified that $d^*_{max} \leq d^*_u$ and the compatibility of displacements for ductile elements/mechanisms and resistances for fragile elements/mechanisms are checked. The static nonlinear analysis conducted in the manners prescribed by the NTC may significantly underestimate deformations on the stiffer and more resistant sides of torsionally flexible structures, i.e., structures where the torsional mode of vibration has a period greater than at least one of the main translational modes of vibration. To account for this effect, among the secondary force distributions, the adaptive distribution should be chosen. The seismic action must be applied for each direction, in both possible directions, and the most unfavorable effects resulting from the two analyses must be considered.

13 ASSESSMENT OF SEISMIC SAFETY AND VULNERABILITY EX-ANTE

The assessment of seismic safety and vulnerability was carried out using the calculation methods outlined in the chapter "Adopted Analysis Methods." Before performing a comprehensive safety assessment, it is necessary to evaluate any local criticalities that generate local vulnerabilities. Based on the available documentation, geometric and structural surveys, and on-site investigations, local criticalities such as degradation and/or detachments were identified.

14 CRITICALITIES

- The assessment of structural criticalities generating local vulnerabilities is based on the available documentation, geometric and structural surveys, and on-site investigations. From this assessment, it emerged that:
- All horizontal planes, made of reinforced concrete with a slab thickness of at least 4 cm, can be considered infinitely rigid in their plane, as outlined in paragraph § 7.2.6 of the NTC2018.
- The building materials (concrete and steel) exhibit satisfactory mechanical characteristics.

  Therefore, it can be stated that the building does not present significant criticalities.

  Among the criticalities identified are:
- Some structural elements exhibit visible phenomena of spalling of concrete cover.

Figure 138 - Beam experiencing concrete cover spalling (US1)

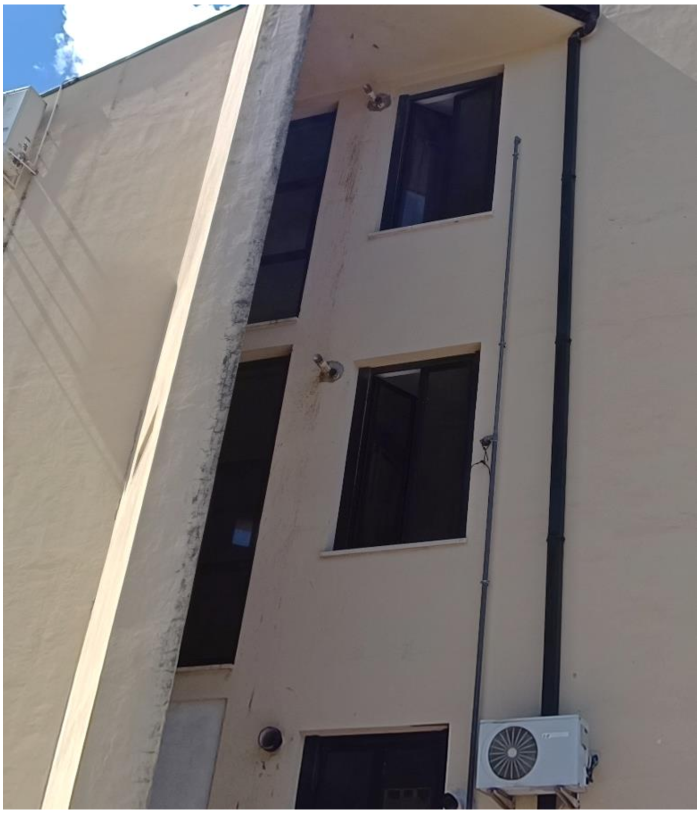


Figure 3 -The partition is experiencing rebar cover expulsion (US1).

- Material degradation due to humidity:

Figure 140- External plaster degradation (US2).

Figure 141- Internal plaster degradation (US2).

Figure 142 - Internal plaster degradation (US2).

Figure 143– Plaster lesions (US2).

Figure 144 - material degradation on the underside of the balcony (US1

15 DISTANCE BETWEEN CONTIGUOUS BUILDINGS ACCORDING TO §7.2.1 OF NTC2018

The distance between contiguous buildings must be such as to avoid hammering phenomena and cannot be less than the sum of the maximum displacements determined for the LLS, calculated for each building according to §7.3.3 (linear Analysis) o il §7.3.4 (nonlinear Analysis).

The distance between two points of facing buildings cannot, in any case, be less than 1/100 of the elevation of the points considered, measured from the foundation's break or the top of the rigid box structure described in §7.2.1, multiplied by $2a_gS/g \leq 1$.

Distanza tra costruzioni contigue NTC2018

Verifica spostamento tra unità strutturali:

Piano Primo
Unità Strutturale 1
Unità Strutturale 2

Dati:

ag	0.578
S	2.15
H_{nodo}	3.2 m
Giunto esistente	70 mm

US1			US2	
Nodo	94		Nodo	6
u (mm)	39.22		u (mm)	1.47
		Verifica nodo 1		
40.69	≤	70	DISTANZA ADEGUATA	

The joint verification is satisfied.

16 NON-SEISMIC STATIC ANALYSIS

In this chapter, the structure's response to gravitational loads due to the self-weight of structural elements (G1), non-structural permanent loads (G2), and variable loads (Qk) will be evaluated. From the visualization of the mechanical model and its respective legend, it is possible to assess which elements pass the verification and which ones do not.

16.1 Results of non-seismic static analysis - US1

SHEAR

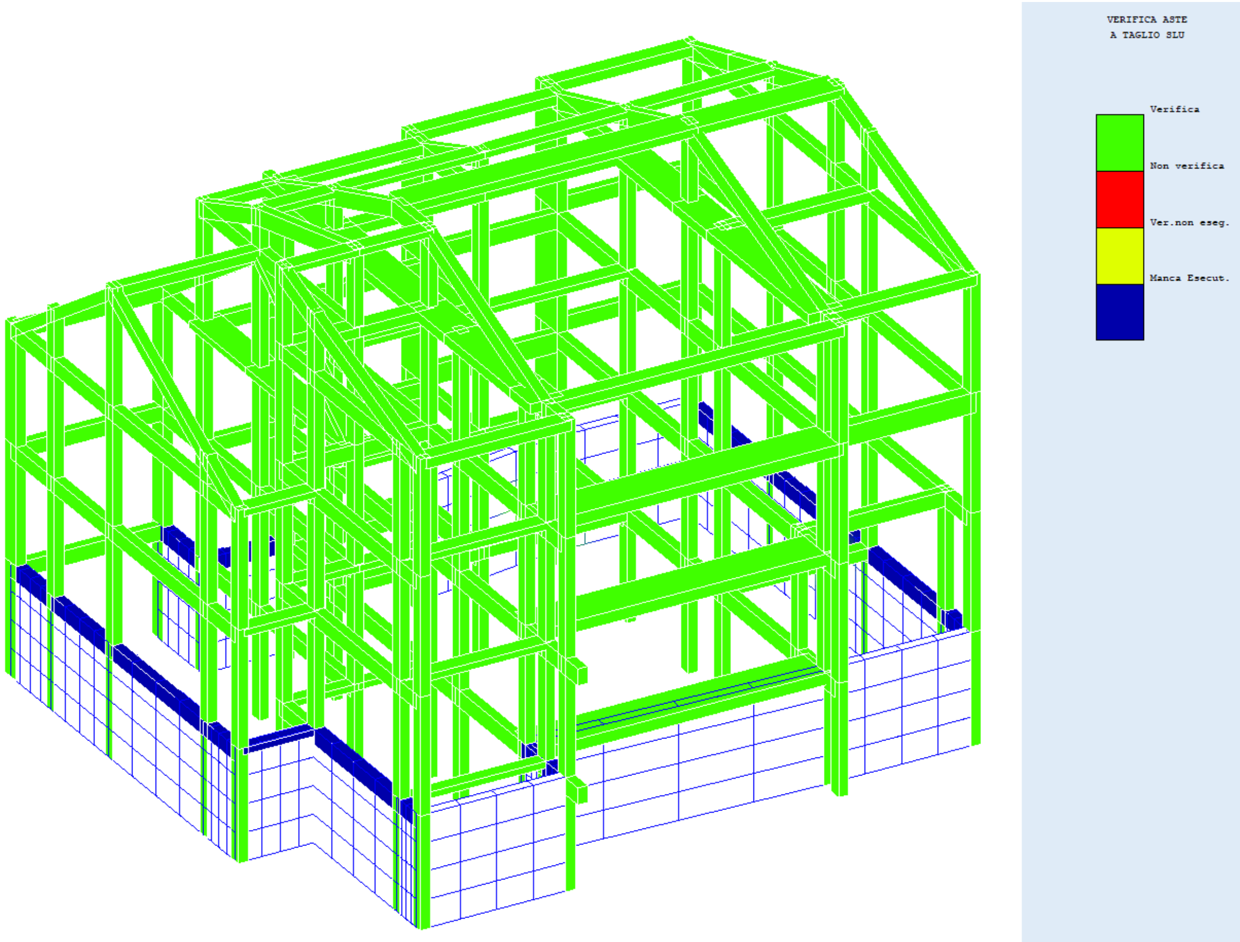


Figure 145 - share verification (US1).

The shear verification is considered satisfied when the Applied Shear is less than the Resistance Shear. In this case, the structural element is identified with the color green.

The shear verification is satisfied.

Flexural bending

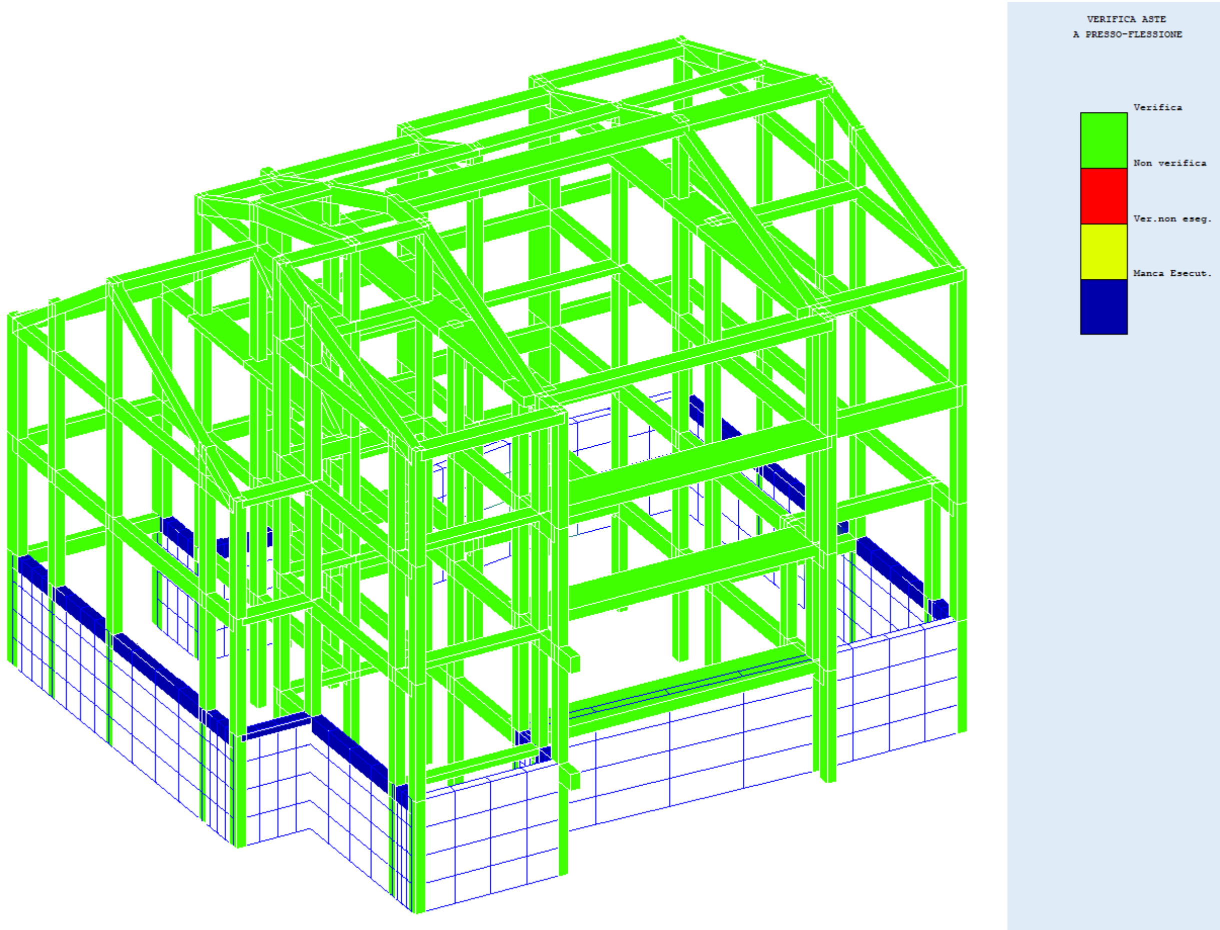


Figure 146 - Flexural bending verification (US1).

The flexural verification is considered satisfied when the applied bending moment is less than the resistance moment. In this case, the structural element is identified with the color green. The flexural verification is satisfied.

16.1 Results of non-seismic static analysis - US2

SHEAR

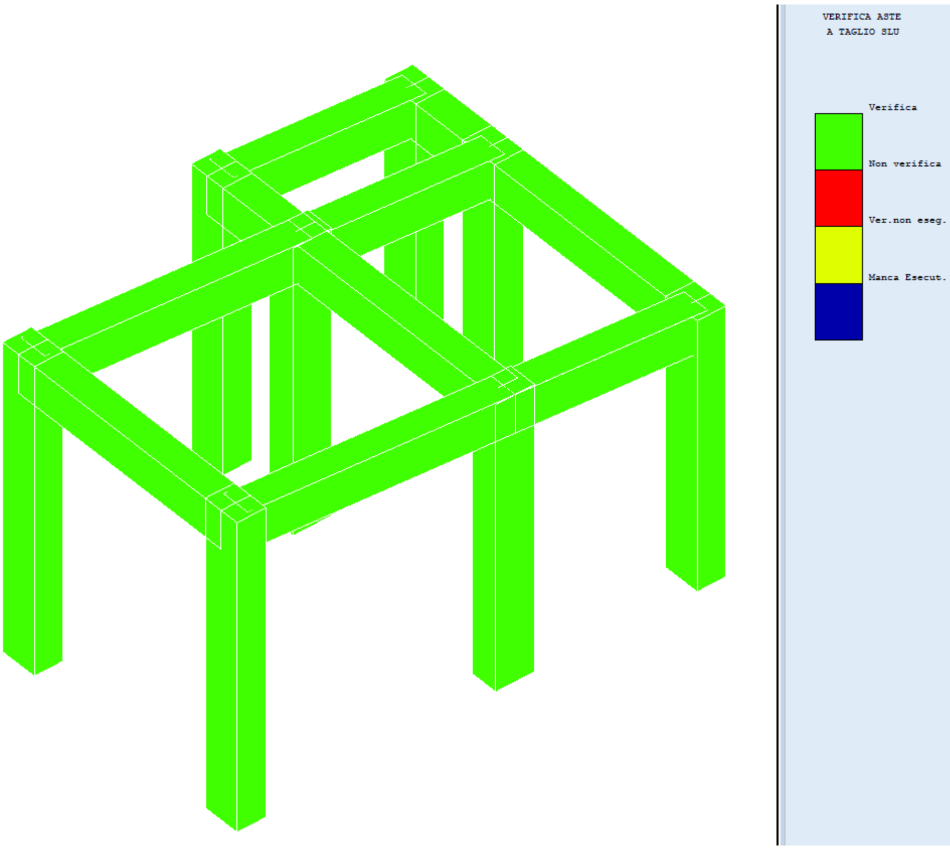


Figure 147 - share verification (US2).

The shear verification is considered satisfactory when the Soliciting Shear is less than the Resistant Shear, in which case the structural element is identified in green. The shear verification is satisfied.

Flexural bending

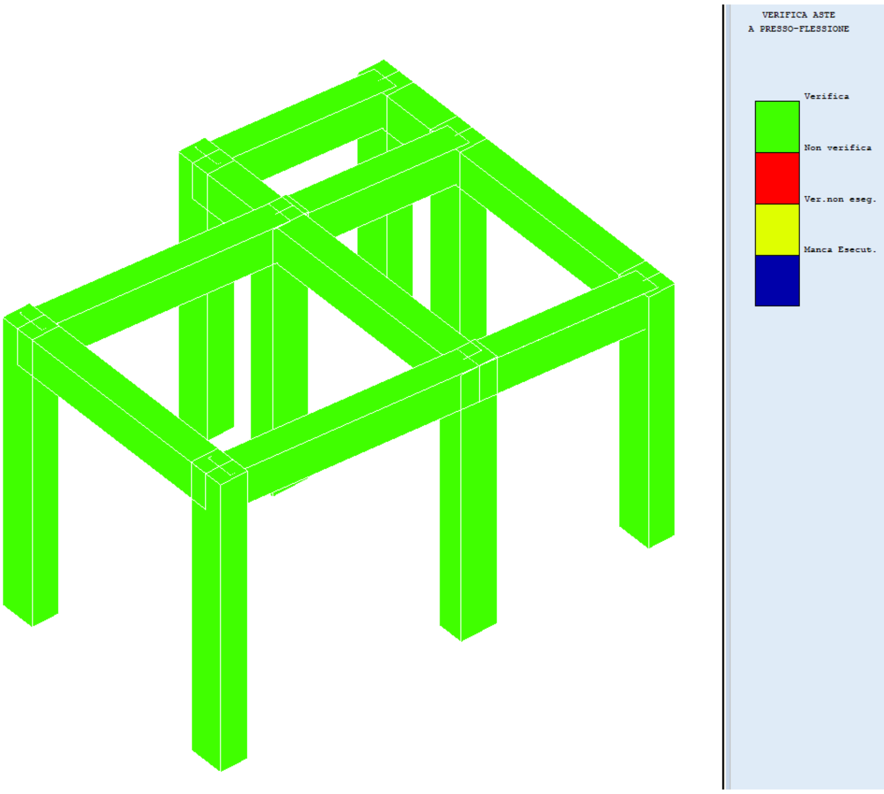


Figure 148- Flexural bending verification (US2).

Flexural verification is deemed satisfactory when the Soliciting Moment is less than the Resisting Moment; in this case, the structural element is identified with green. The flexural verification is satisfied.

17 FLOOR SLAB VERIFICATION

17.1 Load Test

The load test was conducted on a reinforced concrete floor slab, as shown in the following image:

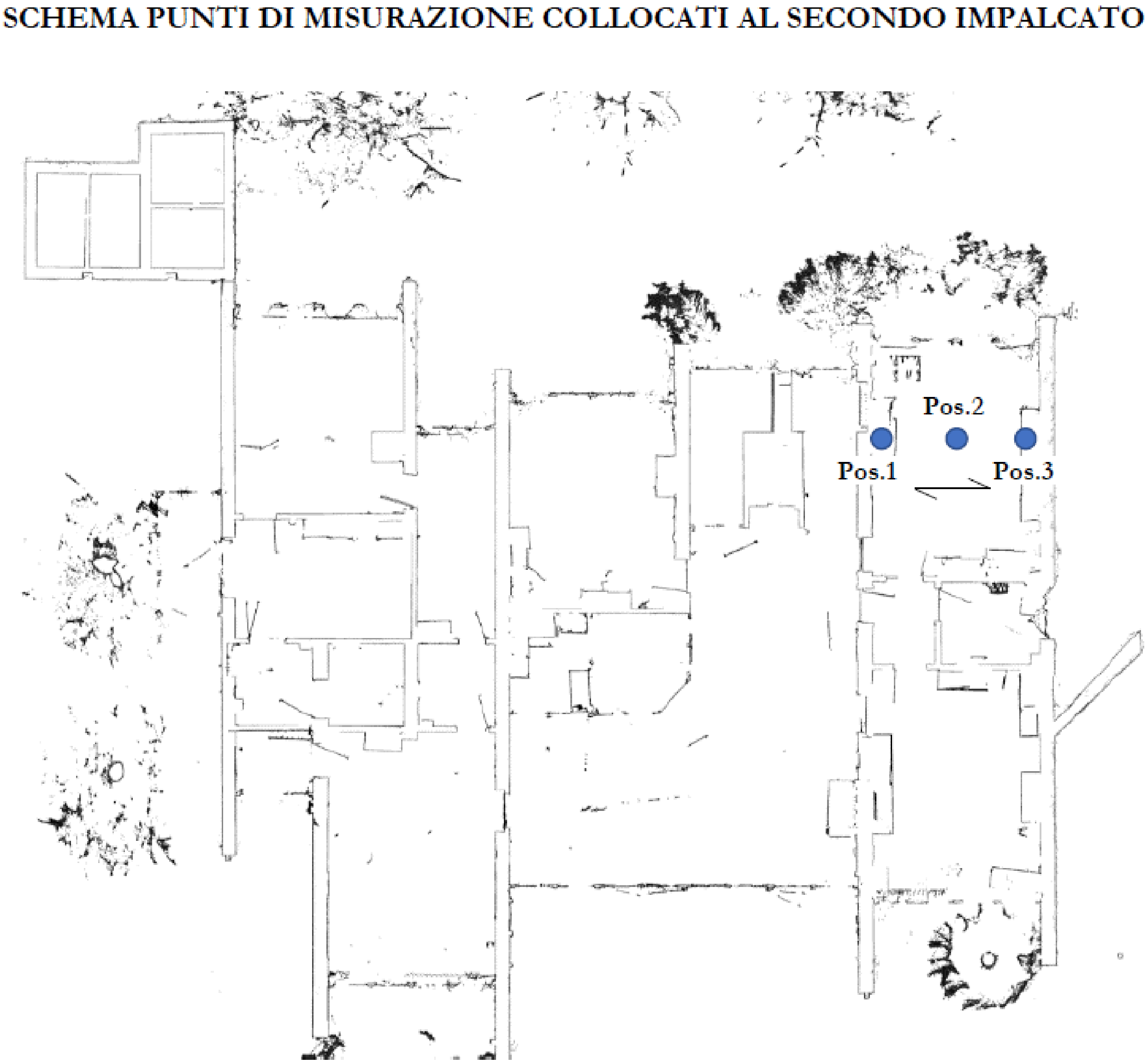


Figure 149 – Ubicazione della prova di carico

Flexible hydraulic load tanks with maximum plan dimensions of 2.40x3.00 m were used for the load test. The load test was performed on the second floor's horizontal plane of the structure. The vertical displacements of the structures were measured using digital comparators – Mitutoyo brand, accuracy 0.01 mm, full scale 25 mm, calibrated – placed on the undersides of the floor directly in contact with the monitoring sections using telescopic rods.

In accordance with NTC2018 C4.1.2.2.2, the limit for long-term deflection of beams and floors, calculated under the quasi-permanent condition of loads, should not exceed 1/250 of the span for safeguarding the appearance and functionality of the structure.

For the case under consideration, the load test resulted in a maximum deformation at midspan of 0.34 mm following the application of 3 kN/sqm, as shown in the load-deflection graph below:

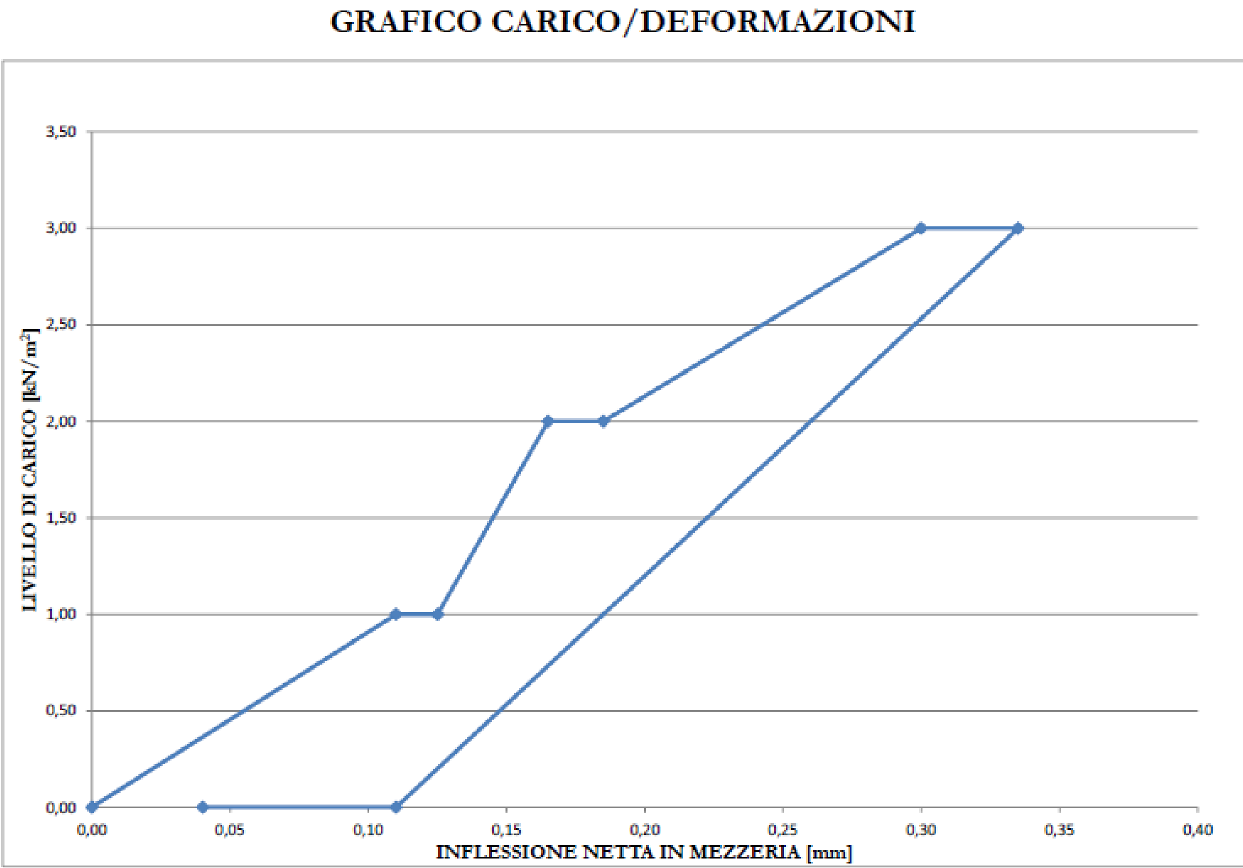


Figure 150 :load-deflection graph

Since the maximum span of the analyzed floor is 4.00 m, you get:

Luce solaio	Limite massimo 1/250*L	Spostamento verticale max misurato δ	Verifica δ < 1/250*L
(m)	(mm)	(mm)	VERIFICATO
4.00	16.00	0.34	

Table 68. Verification of Maximum Vertical Displacement of Floor Slabs

The verification is satisfied and extends to all floors of the same type.

18 VERIFICATION OF NON-STRUCTURAL ELEMENTS

The verification of non-structural elements refers to those elements "with stiffness, strength, and mass significant enough to influence structural response, and those that, although not affecting structural response, are equally significant for the safety and/or well-being of individuals" (cf. § 7.2.3 D.M. 2018).

Floor infills meet the definition of N.T.C. 2018, as they have been modeled in the building since they are elements that do not affect structural response but still need to be considered in the structural analysis with their mass for the correct determination of gravitational and seismic loads.

The verifications of non-structural elements required by D.M. 2018 are illustrated in the following Table 7.3.III, depending on the building's use class (as defined in §2.4.2) and the considered limit state. It can be seen that, for non-structural elements (indicated as "NS"), stability checks (indicated as "STA") are required only for use classes 2 to 4 (excluding only works with occasional presence of people and agricultural buildings) and for the Limit State of Safeguarding Life (SLV) only.

Verifiche richieste per ciascun tipo di elemento (ST = elemento strutturale; NS = elemento non strutturale; IM = impianto), per ciascuna classe d'uso e per ciascun stato limite (cfr. Tabella 7.3.III D.M. 2018)

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(**)		

(*) Nelle CU III e IV, negli impianti sono compresi gli arredi fissi;
(**) Nei casi esplicitamente indicati nel D.M. 2018.

Table 69.Verification Requirements for Each Type of Element, Usage Class, and Limit State (Table 7.3.III D.M. 2018)

Being in use class IV, stability checks (§7.3.6.2 of D.M. 2018) are performed to verify that the non-structural element is not "expelled" under the equivalent seismic action.

18.1 Wall Verification

Non_structural constructive elements refers to the elements with enough stiffness, resistance and mass that can influence the structural response and the safety of people ,even though they don’t be considered as part of the primary structure; These elements must be stronger than the expected seismic forces, which are determined by project designer.

Seismic demand on the non-structural elements can be determined by applying horizontal force F defined is follows:

$F_a=(S_a \,.\, W_a)/q_a$

F, is the horizontal seismic force distributed or acting in the center of mass of the non-natural element, in the most unfavorable direction, resulting from the distributed forces proportional to the mass.

S, is the maximum acceleration, dimensionless with respect to that of gravity, that the structural element undergoes during the earthquake and corresponds to the limit state examined (see §3.2.1),

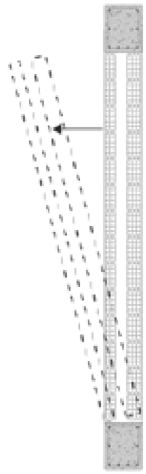
W, is the weight of the element;

q is the behavior factor of the element.

Below is the analysis of the infills, considering seismic action.

EX-ANTE – wall (8+12) cm located in Ground floor

STRATIGRAFIA DELLA TAMPONATURA		
Descrizione degli strati della tamponatura		
Tipologia di tamponatura	**Tamponatura a cassetta**	
	Spessore [cm]	**Peso [kN/m³]**
Intonaco esterno	1.5	30
Mattone forato esterno	12	8
Intercapedine	4	
Mattone forato interno	8	8
Intonaco interno	1	20



Dimensioni del singolo strato da verificare

Resistenza caratteristica a compressione dell'elemento	fbk	20.00	N/mm²
Resistenza della malta		M15	
Resistenza caratteristica a compressione della tamponatura	fk	9.70	N/mm²
Coefficiente di sicurezza del materiale	γ_M	2.00	
Resistenza di progetto a compressione della tamponatura	fd	4.85	MPa

Dimensioni del pannello di tamponatura da verificare (la verifica considera una larghezza unitaria della tamponatura)

Altezza della tamponatura	H	3.50	m
Peso del paramento interno	$W_{a,int}$	2.94	kN/m
Inerzia del paramento interno	I_{int}	0.00004	m⁴/m
Peso del paramento esterno	$W_{a,est}$	4.94	kN/m
Inerzia del paramento esterno	I_{est}	0.00014	m⁴/m
Modulo elastico dei paramenti	E	9700	MPa

Definizione dell'azione sismica

Accelerazione massima del terreno ag su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete non strutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.10	m
Quota del baricentro dell'elemento non strutturale misurata a partire dal piano di fondazion	Z	5.13	m
Coefficiente (Circolare NTC2018 paragrafo C7.3.3.2 formula C.7.3.2)	C1	0.075	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	0.57	sec
Paramento interno			
Massa della muratura	m	0.30	daN/massa
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.321	sec
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	a	0.3	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	b	1.2	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	ap	4	
Accelerazione massima (Circolare n°7_C7.2.3)	Sa(ta)	3.7789	g
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	5.55	kN
Paramento esterno			
Massa della muratura	m	0.50	daN/massa
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.226	sec
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	a	0.3	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	b	1.2	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	ap	4	
Accelerazione massima (Circolare n°7_C7.2.3)	Sa(ta)	3.78	g
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	9.32	kN/m

VERIFICA A RIBALTAMENTO SEMPLICE - paramento interno			
Momento ribaltante (domanda)	M_{RIB}	9.72	kN*m/m
Momento stabilizzante (capacità)	M_{STA}	0.12	kN*m/m
Rapporto M_{RIB} /M_{STA}	D/C	82.66	NON VERIFICATO

VERIFICA A RIBALTAMENTO SEMPLICE - paramento esterno			
Momento ribaltante (domanda)	M_{RIB}	16.32	kN*m/m
Momento stabilizzante (capacità)	M_{STA}	0.30	kN*m/m
Rapporto M_{RIB} /M_{STA}	D/C	55.11	NON VERIFICATO

CALCOLO INDICATORE DI RISCHIO			
Calcola indicatore di rischio Stato di Fatto	α_a	1.21%	
Accelerazione sismica corrispondente all'indicatore di rischio	a_g	0.0070	g

The verification is not satisfied, and therefore, an intervention for anti-toppling of the infills is planned by connecting them to reinforced concrete beams and columns using structural plaster based on pure lime, bi-axial basalt fiber mesh, and helical stainless steel bars.

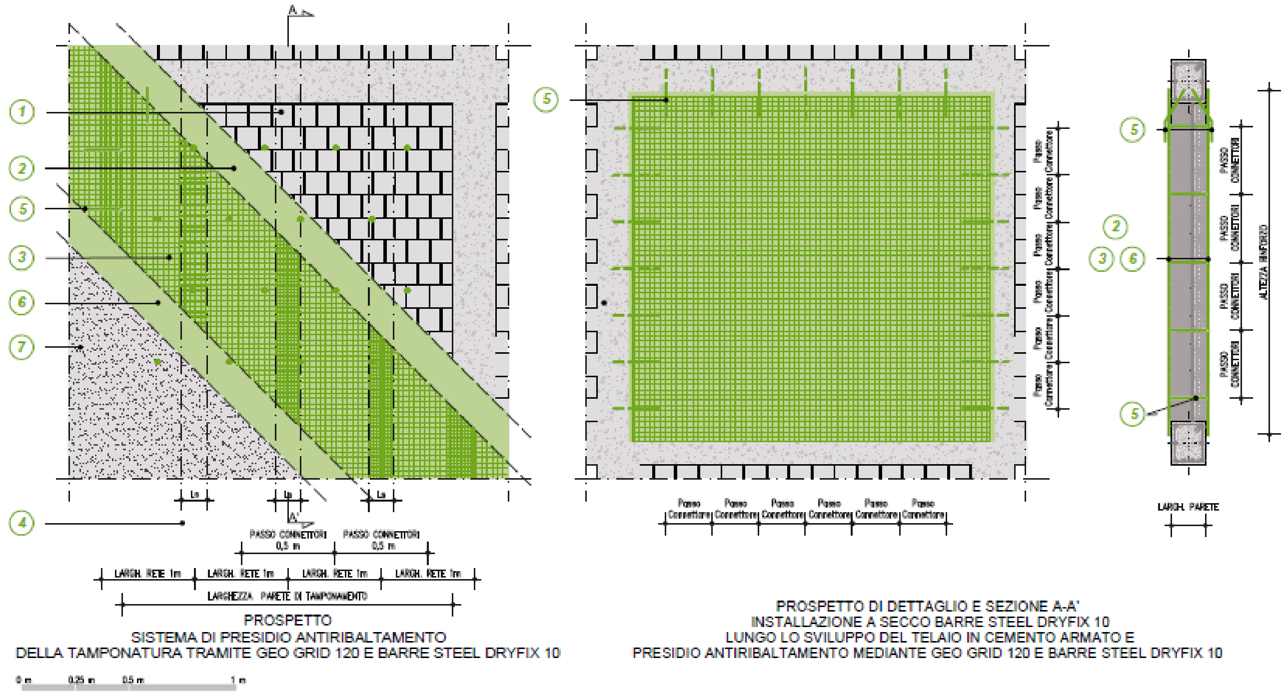
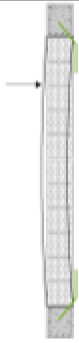


Figure 451 – Detail of the anti-toppling intervention for the infills.

Ex-post

STRATIGRAFIA DELLA TAMPONATURA			
Descrizione degli strati della tamponatura	**Spessore [cm]**	**Peso [kN/m³]**	
Intonaco esterno	1.50	30.00	
Mattone forato	12.00	5.00	
Intonaco interno	2.50	20.00	
Spessore totale della tamponatura	26.50		



Riepilogo dimensioni geometriche e caratteristiche meccaniche della tamponatura			
Altezza del pannello murario	H	3.50	m
Lunghezza pannello murario	L	1.00	m
Peso della muratura totale	Wa	7.88	kN/m
Modulo elastico	E	9700.00	N/mm²
Resistenza di progetto a compressione della muratura	f_d	4.85	MPa
CARATTERISTICHE GEOMETRICHE DEL RINFORZO			
Rinforzo paramento esterno con rete Geo Grid 120 e GeoCalce Multiuso			
Rimozione di intonaco esterno		SI	
Tensione di progetto della rete	σ_d	213.33	Mpa
Connettori tra paramento esterno e telaio in ca.			
Diametro dei connettori	Ø	Steel Dry Fix 8	
Resistenza a taglio di progetto dei connettori		0.76	kN
Passo orizzontale dei connettori al m	s_h	0.5	m
Lunghezza inghisaggio connettori	L_{ans}	50	mm
DEFINIZIONE DELL'AZIONE SISMICA			
Accelerazione massima del terreno su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete nonstrutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.1	m
Quota del baricentro dell'elemento nonstrutturale misurata a partire dal piano di fc	Z	5.13	m
Coefficiente	C1	0.075	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	0.57	sec
Massa della muratura	m	0.80	daN/massa
FORMULAZIONE SEMPLIFICATA PER COSTRUZIONI CON STRUTTURA A TELAI - CIRCOLARE n°7			
Parametro	a	0.3	
Parametro	b	1.2	
Parametro	ap	4	
Momento d'inerzia efficace del paramento esterno	I_{eff}	0.000175	m⁴/m
Periodo fondamentale di vibrazione dell'elemento nonstrutturale	Ta	0.044	sec
Accelerazione massima	Sa(Ta)	1.422	g

VERIFICA DI RESISTENZA A PRESSOFLESSIONE FUORI DAL PIANO DEL PARAMENTO ESTERNO				
Definizione della domanda				
Peso della tamponatura al metro lineare di altezza	Wa/m	1.32	kN/m/m	
Forza Orizzontale (Domanda sismica) (NTC 2018_§7.2.3)	Fa	0.94	kN/m/m	
Schema Statico		SEMINCASTRO		Nell'hp di semincastro è necessario prvedere le Steel DryFix anche alla base del pannello
Momento sollecitante con forza distribuita	$M_{sd\_lat\_est}$	**0.72**	**kN*m/m**	
Sforzo normale agente paramento esterno	$N_{sd\_lat\_est}$	2.30	kN/m	
Definizione della capacità del paramento verso l'interno				
Area paramento esterno	A	0.13	m^2/m	
Tensione di compressione sul paramento esterno	σ_{d_est}	0.018	Mpa/m	
Momento resistente a pressoflessione fuori dal piano verso l'int	M_{rd_est}	**0.15**	**kN*m/m**	
Rapporto Domanda/Capacità	D/C	4.88	MIGLIORAMENTO SISMICO	

VERIFICA A TAGLIO DEI CONNETTORI DI ANCORAGGIO_TELAIO-TAMPONATURA			
Definizione del numero di connettori			
Interasse orizzontale dei connettori	$i_{...}$	0.5	m
Numero di connettori al metro	n°/m	2	
Numero totale di connettori	$n°_{TOT}$	2	
Forza di taglio totale agente sul lato superiore/inferiore della t	**Vsd**	**1.64**	**kN/m**
Resistenza a taglio del singolo connettore	R_1	0.76	kN
Resistenza al taglio totale	V_{Rd}	1.51	**kN/m**
Rapporto Domanda/Capacità	D/C	1.08	MIGLIORAMENTO SISMICO

CALCOLO INDICATORE DI RISCHIO

Calcolo Indicatore di rischio Stato di Progetto

Valutazione della sicurezza per azioni fuori dal piano verso l'interno			
Momento resistente fuori dal piano verso l'interno della tamponatura	Mrd	0.15	kNm/m
Indicatore di rischio	α_u	19%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.111	g
		MIGLIORAMENTO SISMICO	
Valutazione della sicurezza per azioni fuori dal piano verso l'esterno			
Momento resistente fuori dal piano verso l'esterno della tamponatura	Mrd	0.88	kNm/m
Indicatore di rischio	α_u	122%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.708	g
		ADEGUAMENTO SISMICO	

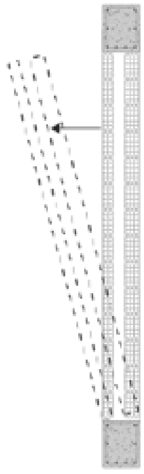
Following the proposed intervention, the verification is satisfied.

EX ANTE – wall (8+12) cm First Floor

STRATIGRAFIA DELLA TAMPONATURA

Descrizione degli strati della tamponatura

Tipologia di tamponatura	Tamponatura a cassetta	
	Spessore [cm]	**Peso [kN/m³]**
Intonaco esterno	1.5	30
Mattone forato esterno	12	8
Intercapedine	4	
Mattone forato interno	8	8
Intonaco interno	1	20



Dimensioni del singolo strato da verificare

Resistenza caratteristica a compressione dell'elemento	fbk	20.00	N/mm²
Resistenza della malta		M15	
Resistenza caratteristica a compressione della tamponatura	fk	9.70	N/mm²
Coefficiente di sicurezza del materiale	γ_M	2.00	
Resistenza di progetto a compressione della tamponatura	fd	4.85	MPa

Dimensioni del pannello di tamponatura da verificare (la verifica considera una larghezza unitaria della tamponatura)

Altezza della tamponatura	H	3.50	m
Peso del paramento interno	$W_{a,int}$	2.94	kN/m
Inerzia del paramento interno	I_{int}	0.00004	m⁴/m
Peso del paramento esterno	$W_{a,ext}$	4.94	kN/m
Inerzia del paramento esterno	I_{ext}	0.00014	m⁴/m
Modulo elastico dei paramenti	E	9700	MPa

Definizione dell'azione sismica

Accelerazione massima del terreno ag su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete non strutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.10	m
Quota del baricentro dell'elemento non strutturale misurata a partire dal piano di fondazion	Z	8.68	m
Coefficiente (Circolare NTC2018 paragrafo C7.3.3.2 formula C.7.3.2)	C1	0.075	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	0.57	sec
Paramento interno			
Massa della muratura	m	0.30	daN/massa
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.321	sec
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	a	0.3	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	b	1.2	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	ap	4	
Accelerazione massima (Circolare n°7_C7.2.3)	Sa(ta)	4.4420	g
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	6.53	kN
Paramento esterno			
Massa della muratura	m	0.50	daN/massa
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.226	sec
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	a	0.3	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	b	1.2	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	ap	4	
Accelerazione massima (Circolare n°7_C7.2.3)	Sa(ta)	4.44	g
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	10.96	kN/m

VERIFICA A RIBALTAMENTO SEMPLICE - paramento interno			
Momento ribaltante (domanda)	M_{RIB}	11.43 kN*m/m	
Momento stabilizzante (capacità)	M_{STA}	0.12 kN*m/m	
Rapporto M_{RIB} /M_{STA}	D/C	97.17	NON VERIFICATO

VERIFICA A RIBALTAMENTO SEMPLICE - paramento esterno			
Momento ribaltante (domanda)	M_{RIB}	19.18 kN*m/m	
Momento stabilizzante (capacità)	M_{STA}	0.30 kN*m/m	
Rapporto M_{RIB} /M_{STA}	D/C	64.78	NON VERIFICATO

CALCOLO INDICATORE DI RISCHIO		
Calcola indicatore di rischio Stato di Fatto	α_a	1.03%
Accelerazione sismica corrispondente all'indicatore di rischio	a_g	0.0059 g

The verification is not satisfied, and therefore, an anti-toppling intervention for the infills is planned by connecting them to reinforced concrete beams and columns using structural plaster based on pure lime, bi-axial basalt fiber mesh, and helical stainless steel bars

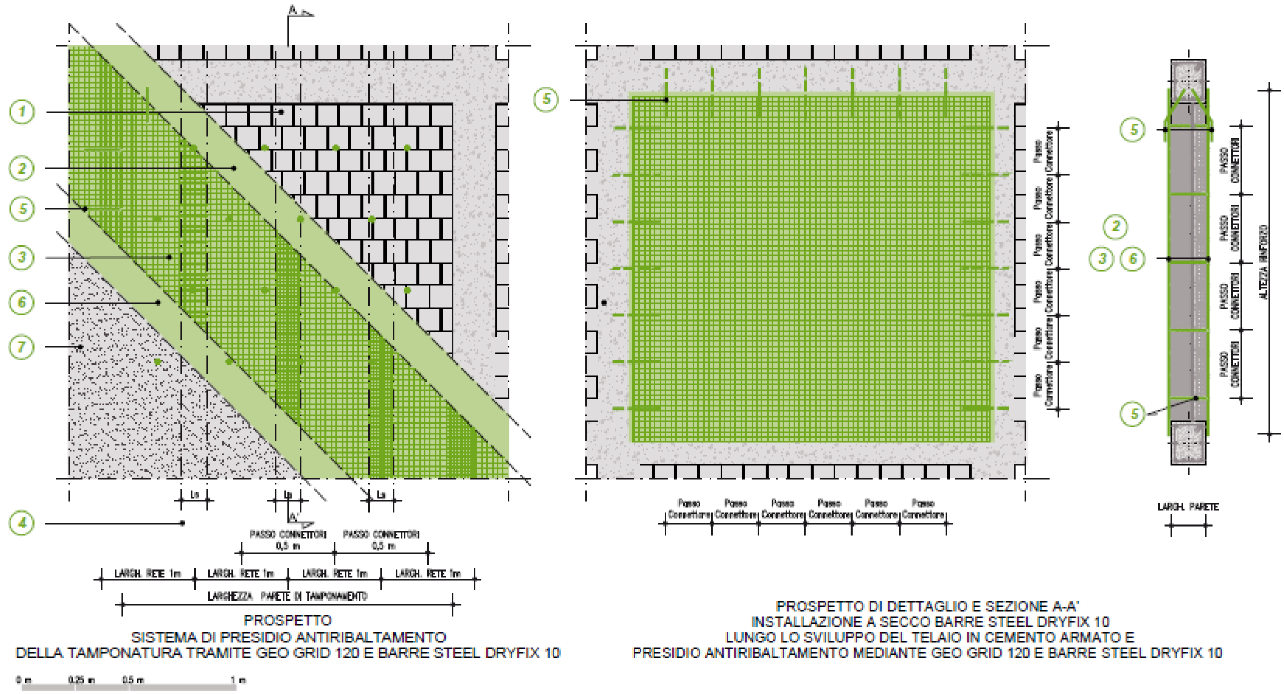
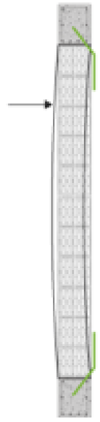


Figure 152 – Detail of the anti-toppling intervention for the infills.

EX-POST

STRATIGRAFIA DELLA TAMPONATURA

Descrizione degli strati della tamponatura	Spessore [cm]	Peso [kN/m³]
Intonaco esterno	1.50	30.00
Mattone forato	12.00	5.00
Intonaco Interno	2.50	20.00
Spessore totale della tamponatura	26.50	



Riepilogo dimensioni geometriche e caratteristiche meccaniche della tamponatura

Altezza del pannello murario	H	3.50	m
Lunghezza pannello murario	L	1.00	m
Peso della muratura totale	Wa	7.88	kN/m
Modulo elastico	E	9700.00	N/mm^2
Resistenza di progetto a compressione della muratura	f_d	4.85	MPa

CARATTERISTICHE GEOMETRICHE DEL RINFORZO

Rinforzo paramento esterno con rete Geo Grid 120 e GeoCalce Multiuso

Rimozione di intonato esterno		SI	
Tensione di progetto della rete	σ_d	213.33	Mpa

Connettori tra paramento esterno e telaio in ca.

Diametro dei connettori	Ø	Steel Dry Fix 8	
Resistenza a taglio di progetto dei connettori		0.76	kN
Passo orizzontale dei connettori al m	s_h	0.5	m
Lunghezza inghisaggio connettori	L_{con}	50	mm

DEFINIZIONE DELL'AZIONE SISMICA

Accelerazione massima del terreno ag su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete non strutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.1	m
Quota del baricentro dell'elemento non strutturale misurata a partire dal piano di for	Z	8.68	m
Coefficiente	C1	ta	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	#VALORE!	sec
Massa della muratura	m	0.80	daN/massa

FORMULAZIONE SEMPLIFICATA PER COSTRUZIONI CON STRUTTURA A TELAI - CIRCOLARE n°7 21/01/2019 _ C7.2.3

Parametro	a	#VALORE!	
Parametro	b	#VALORE!	
Parametro	ap	#VALORE!	
Momento d'inerzia efficace del paramento esterno	I_{eff}	0.000175	m^4/m
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.044	sec
Accelerazione massima	Sa(Ta)	#VALORE!	g

VERIFICA DI RESISTENZA A PRESSOFLESSIONE FUORI DAL PIANO DEL PARAMENTO ESTERNO				
Definizione della domanda				
Peso della tamponatura al metro lineare di altezza	Wa/m	1.32	kN/m/m	
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	#VALORE!	kN/m/m	
Schema Statico		SEMINCASTRO		Nell'hp di semincastro è necessario prvedere le Steel DryFix anche alla base del pannello
Momento sollecitante con forza distribuita	$M_{rd\_tot\_est}$	**#VALORE!**	**kN*m/m**	
Sforzo normale agente paramento esterno	$N_{rd\_tot\_est}$	2.30	kN/m	
Definizione della capacità del paramento verso l'interno				
Area paramento esterno	A	0.13	m^2/m	
Tensione di compressione sul paramento esterno	σ_{0_est}	0.018	Mpa/m	
Momento resistente a pressoflessione fuori dal piano verso l'inter	M_{rd_est}	**0.15**	**kN*m/m**	
Rapporto Domanda/Capacità	D/C	#VALORE!		#VALORE!

VERIFICA A TAGLIO DEI CONNETTORI DI ANCORAGGIO_TELAIO-TAMPONATURA				
Definizione del numero di connettori				
Interasse orizzontale dei connettori	i_{con}	0.5	m	
Numero di connettori al metro	n°/m	2		
Numero totale di connettori	$n°_{TOT}$	2		
Forza di taglio totale agente sul lato superiore/inferiore della trave	**Vsd**	**#VALORE!**	**kN/m**	
Resistenza a taglio del singolo connettore	R_1	0.76	kN	
Resistenza al taglio totale	V_{Rd}	**1.51**	**kN/m**	
Rapporto Domanda/Capacità	D/C	#VALORE!		#VALORE!

CALCOLO INDICATORE DI RISCHIO

Calcolo Indicatore di rischio Stato di Progetto

Valutazione della sicurezza per azioni fuori dal piano verso l'interno			
Momento resistente fuori dal piano verso l'interno della tamponatura	Mrd	0.15	kNm/m
Indicatore di rischio	α_u	16%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.094	g
		MIGLIORAMENTO SISMICO	
Valutazione della sicurezza per azioni fuori dal piano verso l'esterno			
Momento resistente fuori dal piano verso l'esterno della tamponatura	Mrd	0.88	kNm/m
Indicatore di rischio	α_u	104%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.602	g
		ADEGUAMENTO SISMICO	

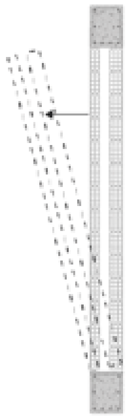
Following the proposed intervention, the verification is satisfied.

EX-ANTE – wall(8+12) cm Second Floor

STRATIGRAFIA DELLA TAMPONATURA

Descrizione degli strati della tamponatura

Tipologia di tamponatura	**Tamponatura a cassetta**	
	Spessore [cm]	**Peso [kN/m³]**
Intonaco esterno	1.5	30
Mattone forato esterno	12	8
Intercapedine	4	
Mattone forato interno	8	8
Intonaco interno	1	20



Dimensioni del singolo strato da verificare

Resistenza caratteristica a compressione dell'elemento	fbk	20.00	N/mm²
Resistenza della malta		M15	
Resistenza caratteristica a compressione della tamponatura	fk	9.70	N/mm²
Coefficiente di sicurezza del materiale	γ_M	2.00	
Resistenza di progetto a compressione della tamponatura	fd	4.85	MPa

Dimensioni del pannello di tamponatura da verificare (la verifica considera una larghezza unitaria della tamponatura)

Altezza della tamponatura	H	3.48	m
Peso del paramento interno	$W_{s,int}$	2.92	kN/m
Inerzia del paramento interno	I_{int}	0.00004	m⁴/m
Peso del paramento esterno	$W_{s,est}$	4.91	kN/m
Inerzia del paramento esterno	I_{est}	0.00014	m⁴/m
Modulo elastico dei paramenti	E	9700	MPa

Definizione dell'azione sismica

Accelerazione massima del terreno ag su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete nonstrutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.10	m
Quota del baricentro dell'elemento nonstrutturale misurata a partire dal piano di fondazione	Z	12.15	m
Coefficiente (Circolare NTC2018 paragrafo C7.3.3.2 formula C.7.3.2)	C1	0.075	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	0.57	sec
Paramento interno			
Massa della muratura	m	0.30	daN/massa
Periodo fondamentale di vibrazione dell'elemento nonstrutturale	Ta	0.317	sec
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	a	0.3	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	b	1.2	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	ap	4	
Accelerazione massima (Circolare n°7_C7.2.3)	Sa(ta)	5.0902	g
Forza Orizzontale (Domanda sismica) (NTC 2018_ §7.2.3)	Fa	7.44	kN
Paramento esterno			
Massa della muratura	m	0.50	daN/massa
Periodo fondamentale di vibrazione dell'elemento nonstrutturale	Ta	0.224	sec
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	a	0.3	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	b	1.2	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	ap	4	
Accelerazione massima (Circolare n°7_C7.2.3)	Sa(ta)	5.09	g
Forza Orizzontale (Domanda sismica) (NTC 2018_ §7.2.3)	Fa	12.49	kN/m

VERIFICA A RIBALTAMENTO SEMPLICE - paramento interno			
Momento ribaltante (domanda)	M_{RIB}	12.95	kN*m/m
Momento stabilizzante (capacità)	M_{STA}	0.12	kN*m/m
Rapporto M_{RIB} /M_{STA}	D/C	110.71	NON VERIFICATO

VERIFICA A RIBALTAMENTO SEMPLICE - paramento esterno			
Momento ribaltante (domanda)	M_{RIB}	21.73	kN*m/m
Momento stabilizzante (capacità)	M_{STA}	0.29	kN*m/m
Rapporto M_{RIB} /M_{STA}	D/C	73.81	NON VERIFICATO

CALCOLO INDICATORE DI RISCHIO			
Calcola indicatore di rischio Stato di Fatto	α_a	0.93%	
Accelerazione sismica corrispondente all'indicatore di rischio	a_g	0.0054	g

The verification is not satisfied, and therefore, an anti-toppling intervention for the infills is planned by connecting them to reinforced concrete beams and columns using structural plaster based on pure lime, bi-axial basalt fiber mesh, and helical stainless steel bars.

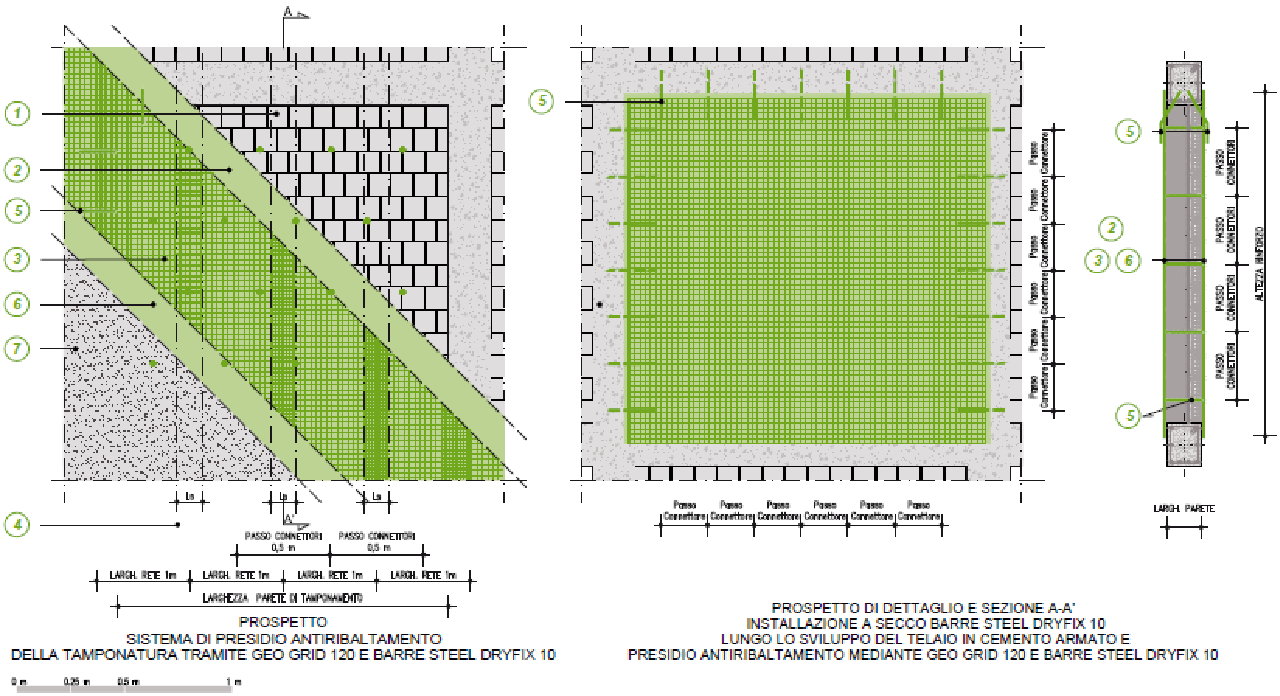
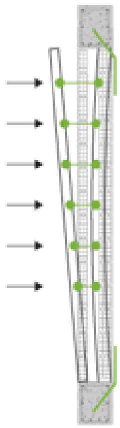


Figure 149 –Detail of the anti-toppling intervention for the infills.

EX-POST

STRATIGRAFIA DELLA TAMPONATURA			
Descrizione degli strati della tamponatura		**Spessore [cm]**	**Peso [kN/m^3]**
Intonaco esterno		1.50	30.00
Mattone forato		12.00	8.00
Intercapedine		4.00	0.00
Mattone forato		8.00	8.00
Intonaco Interno		1.00	20.00
	Spessore totale della tamponatura	26.50	



Riepilogo dimensioni geometriche e caratteristiche meccaniche della tamponatura			
Altezza del pannello murario	H	3.48	m
Lunghezza pannello murario	L	1.00	m
Peso della muratura totale	Wa	7.83	kN/m
Modulo elastico	E	9700.00	N/mm^2
Resistenza di progetto a compressione della muratura	f_d	4.85	MPa
CARATTERISTICHE GEOMETRICHE DEL RINFORZO			
Connettori tra paramento esterno e paramento interno			
Passo verticale dei connettori al m	s_v	0.5	m
Passo orizzontale dei connettori al m	s_h	0.5	m
Diametro dei connettori	Ø	Steel Dry Fix 10	
Resistenza di progetto a trazione dei connettori		0.33	kN
Rinforzo paramento esterno con rete Geo Grid 120 e GeoCalce Multiuso			
Rimozione di intonato esterno		SI	
Tensione di progetto della rete	σ_d	213.33	Mpa
Connettori tra paramento esterno e telaio in ca.			
Diametro dei connettori	Ø	Steel Dry Fix 10	
Resistenza a taglio di progetto dei connettori		0.76	kN
Passo orizzontale dei connettori al m	s_h	0.5	m
Lunghezza inghisaggio connettori	L_{con}	50	mm
DEFINIZIONE DELL'AZIONE SISMICA			
Accelerazione massima del terreno ag su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete non strutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.1	m
Quota del baricentro dell'elemento non strutturale misurata a partire dal piano di for	Z	12.15	m
Coefficiente	C1	0.075	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	0.57	sec
Massa della muratura	m	0.80	daN/massa
FORMULAZIONE SEMPLIFICATA PER COSTRUZIONI CON STRUTTURA A TELAI - CIRCOLARE n°7 21/01/2019 _ C7.2.3			
Parametro	a	0.3	
Parametro	b	1.2	
Parametro	ap	4	
Momento d'inerzia efficace del paramento esterno	I_{eff}	0.000175	m^4/m
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.044	sec
Accelerazione massima	Sa(Ta)	1.906	g

VERIFICA DI RESISTENZA A PRESSOFLESSIONE FUORI DAL PIANO DEL PARAMENTO ESTERNO				
Definizione della domanda				
Peso della tamponatura al metro lineare di altezza	Wa/m	1.32	kN/m/m	
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	1.25	kN/m/m	
Schema Statico		SEMINCASTRO		...ll'hp di semincastro è necessario prvedere le Steel DryFix anche alla base del pannello
Momento sollecitante con forza distribuita	$M_{rd\_tot\_est}$	**0.95**	**kN*m/m**	
Sforzo normale agente paramento esterno	$N_{rd\_tot\_est}$	2.29	kN/m	
Definizione della capacità del paramento verso l'interno				
Area paramento esterno	A	0.13	m^2/m	
Tensione di compressione sul paramento esterno	σ_{0_est}	0.018	Mpa/m	
Momento resistente a pressoflessione fuori dal piano verso l'inter	M_{rd_est}	**0.15**	**kN*m/m**	
Rapporto Domanda/Capacità	D/C	6.50	MIGLIORAMENTO SISMICO	

VERIFICA DEI CONNETTORI DI COLLEGAMENTO TRA I DUE PARAMENTI -Azioni per ribaltamento del paramento interno			
Definizione del numero di connettori			
Forza di trazione massima agente sui conettori	F_{rd_con}	0.356	**kN/m**
Resistenza connettore	Frd	**0.33**	kN/m
Rapporto Domanda/Capacità	D/C	1.07	MIGLIORAMENTO SISMICO

VERIFICA A TAGLIO DEI CONNETTORI DI ANCORAGGIO_TELAIO-TAMPONATURA			
Definizione del numero di connettori			
Interasse orizzontale dei connettori	i_{con}	0.5	m
Numero di connettori al metro	n°/m	2	
Numero totale di connettori	$n°_{TOT}$	2	
Forza di taglio totale agente sul lato superiore/inferiore della trave	**Vsd**	**2.18**	**kN/m**
Resistenza a taglio del singolo connettore	R_1	0.76	kN
Resistenza al taglio totale	V_{Rrd}	1.51	**kN/m**
Rapporto Domanda/Capacità	D/C	1.44	MIGLIORAMENTO SISMICO

CALCOLO INDICATORE DI RISCHIO

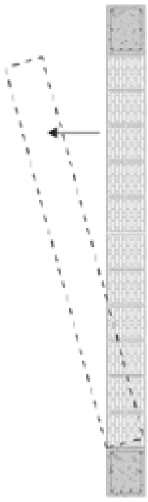
Calcolo Indicatore di rischio Stato di Progetto

Valutazione della sicurezza per azioni fuori dal piano verso l'interno			
Momento resistente fuori dal piano verso l'interno della tamponatura	Mrd	0.15	kNm/m
Indicatore di rischio	α_u	15%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.088	g
		MIGLIORAMENTO SISMICO	

Valutazione della sicurezza per azioni fuori dal piano verso l'esterno			
Momento resistente fuori dal piano verso l'esterno della tamponatura	Mrd	0.88	kNm/m
Indicatore di rischio	α_u	97%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.563	g
		MIGLIORAMENTO SISMICO	

The verification, following the proposed intervention, is deemed satisfactory.

EX-ANTE – Internal Wall



STRATIGRAFIA DELLA TAMPONATURA

Descrizione degli strati della tamponatura

Tipologia di tamponatura	**Singolo paramento**	
	Spessore [cm]	**Peso [kN/m³]**
Intonaco esterno	1	20
Mattone forato esterno	8	8
Intonaco interno	1	20

Dimensioni del singolo strato da verificare

Resistenza caratteristica a compressione dell'elemento	fbk	20.00	N/mm^2
Resistenza della malta		M15	
Resistenza caratteristica a compressione della tamponatura	fk	9.70	N/mm^2
Coefficiente di sicurezza del materiale	γ_M	2.00	
Resistenza di progetto a compressione della tamponatura	fd	4.85	MPa

Dimensioni del pannello di tamponatura da verificare (la verifica considera una larghezza unitaria della ta

Altezza della tamponatura	H	3.50	m
Peso del paramento interno	$W_{a,int}$	0.00	kN/m
Inerzia del paramento interno	I_{int}	0.00000	m^4/m
Peso del paramento esterno	$W_{a,est}$	3.64	kN/m
Inerzia del paramento esterno	I_{est}	0.00004	m^4/m
Modulo elastico dei paramenti	E	9700	MPa

Definizione dell'azione sismica

Accelerazione massima del terreno ag su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete non strutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.10	m
Quota del baricentro dell'elemento non strutturale misurata a partire dal piano di fondazion	Z	8.68	m
Coefficiente (Circolare NTC2018 paragrafo C7.3.3.2 formula C.7.3.2)	C1	0.075	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	0.57	sec
Paramento esterno			
Massa della muratura	m	0.37	daN/massa
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.357	sec
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	a	0.3	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	b	1.2	
Parametro (Circolare n°7_C7.2.3-Tab. C7.2.II)	ap	4	
Accelerazione massima (Circolare n°7_C7.2.3)	Sa(ta)	4.44	g
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	8.08	kN/m

VERIFICA A RIBALTAMENTO SEMPLICE - paramento esterno			
Momento ribaltante (domanda)	M_{RIB}	14.15 kN*m/m	
Momento stabilizzante (capacità)	M_{STA}	0.15 kN*m/m	
Rapporto M_{RIB} / M_{STA}	D/C	97.17	NON VERIFICATO

CALCOLO INDICATORE DI RISCHIO		
Calcola indicatore di rischio Stato di Fatto	α_u	1.03%
Accelerazione sismica corrispondente all'indicatore di rischio	a_g	0.0059 g

The verification is not satisfied, and therefore, it is planned to intervene by anti-toppling the infills by connecting them to reinforced concrete beams and columns using structural plaster based on pure lime, bi-axial basalt fiber mesh, and helical stainless steel bars.

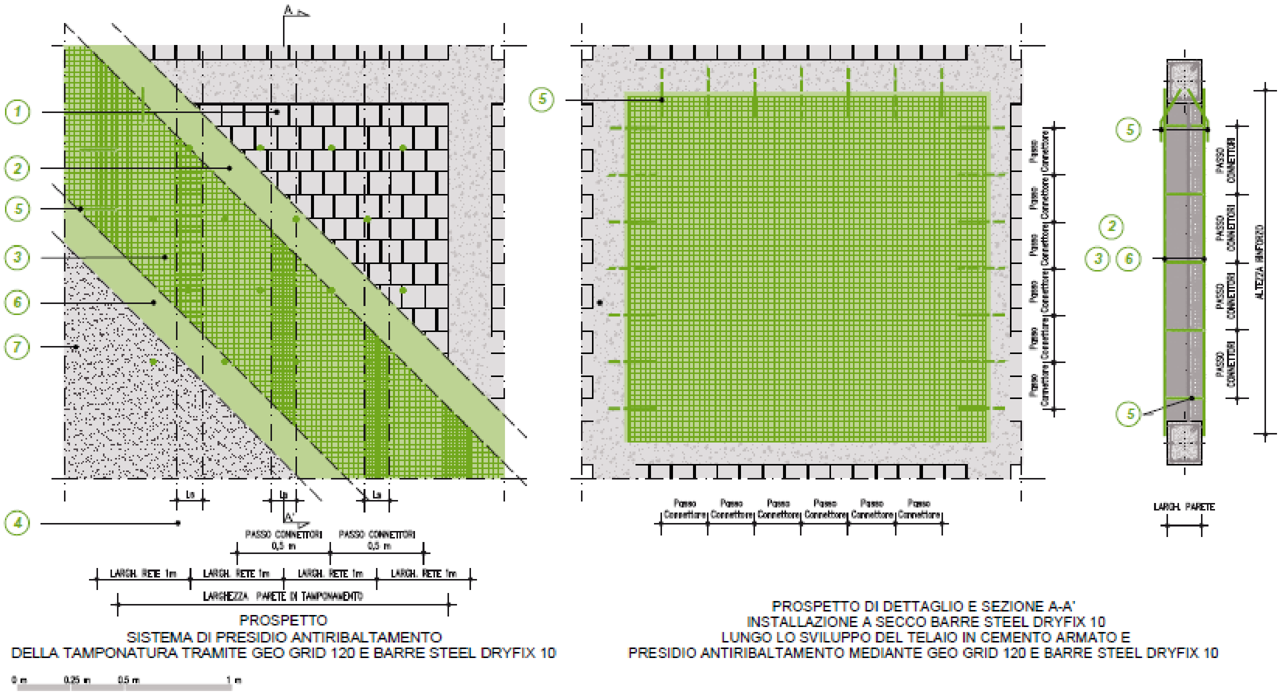
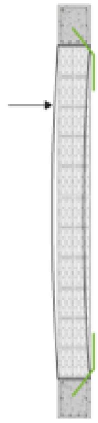


Figure 153 – Detail of the intervention for anti-toppling of the infills.

EX-POST

STRATIGRAFIA DELLA TAMPONATURA		
Descrizione degli strati della tamponatura	**Spessore [cm]**	**Peso [kN/m^3]**
Intonaco esterno	1.00	20.00
Mattone forato	8.00	8.00
Intonaco Interno	1.00	20.00
Spessore totale della tamponatura	10.00	



Riepilogo dimensioni geometriche e caratteristiche meccaniche della tamponatura			
Altezza del pannello murario	H	3.50	m
Lunghezza pannello murario	L	1.00	m
Peso della muratura totale	Wa	3.64	kN/m
Modulo elastico	E	9700.00	N/mm^2
Resistenza di progetto a compressione della muratura	f_d	4.85	MPa

CARATTERISTICHE GEOMETRICHE DEL RINFORZO			
Rinforzo paramento esterno con rete Geo Grid 120 e GeoCalce Multiuso			
Rimozione di intonato esterno		SI	
Tensione di progetto della rete	σ_d	213.33	Mpa
Connettori tra paramento esterno e telaio in ca.			
Diametro dei connettori	Ø	Steel Dry Fix 8	
Resistenza a taglio di progetto dei connettori		0.76	kN
Passo orizzontale dei connettori al m	s_h	0.5	m
Lunghezza inghisaggio connettori	L_{con}	50	mm

DEFINIZIONE DELL'AZIONE SISMICA			
Accelerazione massima del terreno ag su sottosuolo di tipo A	ag/g	0.578	
Coefficiente	F_0	3.13	
Categoria di sottosuolo	Ss	1.22	
Condizione topografica	St	1	
Coefficiente	S	1.22	
Fattore di comportamento della parete non strutturale (Circolare n°7 - Tab. C7.2.I)	qa	2	
Altezza dell'edificio	H	15.1	m
Quota del baricentro dell'elemento non strutturale misurata a partire dal piano di for	Z	8.68	m
Coefficiente	C1	0.075	
Periodo fondamentale di vibrazione della costruzione nella direzione considerata	T1	0.57	sec
Massa della muratura	m	0.37	daN/massa

FORMULAZIONE SEMPLIFICATA PER COSTRUZIONI CON STRUTTURA A TELAI - CIRCOLARE n°7 21/01/2019 _ C7.2.3			
Parametro	a	0.3	
Parametro	b	1.2	
Parametro	ap	4	
Momento d'inerzia efficace del paramento esterno	I_{eff}	0.000057	m^4/m
Periodo fondamentale di vibrazione dell'elemento non strutturale	Ta	0.066	sec
Accelerazione massima	Sa(Ta)	2.067	g

VERIFICA DI RESISTENZA A PRESSOFLESSIONE FUORI DAL PIANO DEL PARAMENTO ESTERNO				
Definizione della domanda				
Peso della tamponatura al metro lineare di altezza	Wa/m	0.94	kN/m/m	
Forza Orizzontale (Domanda sismica) (NTC 2018 _ §7.2.3)	Fa	0.98	kN/m/m	
Schema Statico		SEMINCASTRO		Nell'hp di semincastro è necessario prvedere le Steel DryFix anche alla base del pannello
Momento sollecitante con forza distribuita	$M_{rd\_tot\_est}$	**0.75**	**kN*m/m**	
Sforzo normale agente paramento esterno	$N_{rd\_tot\_est}$	1.65	kN/m	
Definizione della capacità del paramento verso l'interno				
Area paramento esterno	A	0.09	m^2/m	
Tensione di compressione sul paramento esterno	σ_{0_est}	0.019	Mpa/m	
Momento resistente a pressoflessione fuori dal piano verso l'inter	M_{rd_est}	**0.07**	**kN*m/m**	
Rapporto Domanda/Capacità	D/C	10.32	MIGLIORAMENTO SISMICO	

VERIFICA A TAGLIO DEI CONNETTORI DI ANCORAGGIO_TELAIO-TAMPONATURA			
Definizione del numero di connettori			
Interasse orizzontale dei connettori	i_{con}	0.5	m
Numero di connettori al metro	n°/m	2	
Numero totale di connettori	$n°_{TOT}$	2	
Forza di taglio totale agente sul lato superiore/inferiore della trave	**Vsd**	**1.71**	**kN/m**
Resistenza a taglio del singolo connettore	R_1	0.76	kN
Resistenza al taglio totale	V_{Rd}	1.51	**kN/m**
Rapporto Domanda/Capacità	D/C	1.13	MIGLIORAMENTO SISMICO

CALCOLO INDICATORE DI RISCHIO			
Calcolo Indicatore di rischio Stato di Progetto			
Valutazione della sicurezza per azioni fuori dal piano verso l'interno			
Momento resistente fuori dal piano verso l'interno della tamponatura	Mrd	0.07	kNm/m
Indicatore di rischio	α_u	10%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.056	g
		MIGLIORAMENTO SISMICO	
Valutazione della sicurezza per azioni fuori dal piano verso l'esterno			
Momento resistente fuori dal piano verso l'esterno della tamponatura	Mrd	0.58	kNm/m
Indicatore di rischio	α_u	78%	
Accelerazione corrispondente all'indicatore di rischio	a_g	0.451	g
		MIGLIORAMENTO SISMICO	

The verification, following the proposed intervention, is deemed satisfactory.

19 INTERVENTIONS AIMED AT ADDRESSING LOCAL CRITICALITIES.

Interventions aimed at addressing local criticalities include:

Anti-toppling systems for infill walls.

19.1 Anti-toppling prevention of infills by applying a bi-axial network made of natural basalt fiber onto the existing plaster, with a skim coat of pure lime and stitching using stainless steel helical bars.

For non-structural elements, measures must be taken to prevent possible expulsion under the action of Fa (horizontal seismic force distributed or acting at the center of mass of the structural element, in the most unfavorable direction, resulting from forces distributed proportionally to mass) (see §7.2.3) corresponding to the considered SL and CU. (D.M. January 17, 2018 "Technical Standards for Constructions" §7.3.6.2)

Non-structural construction elements refer to those with stiffness, strength, and mass significant enough to influence the structural response and those that, while not affecting the structural response, are still significant for the safety and/or well-being of individuals.

Possible collapse mechanisms of infills:

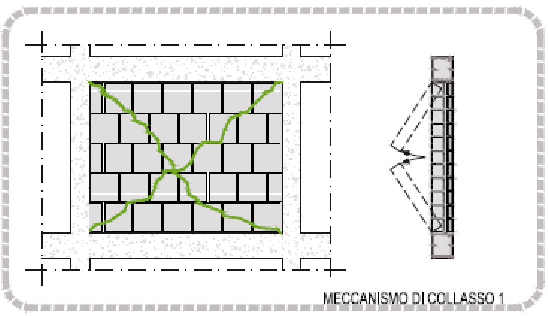


Figure 154 – In-plane collapse mechanism

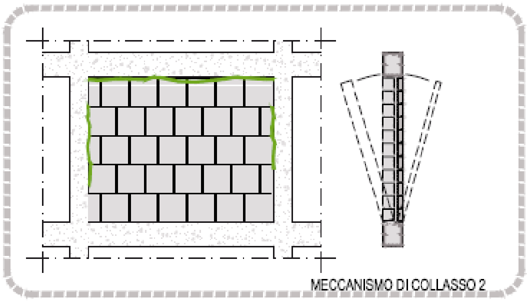
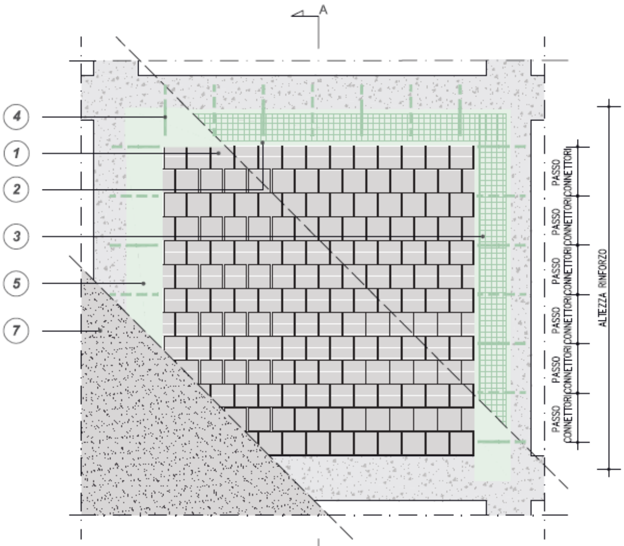
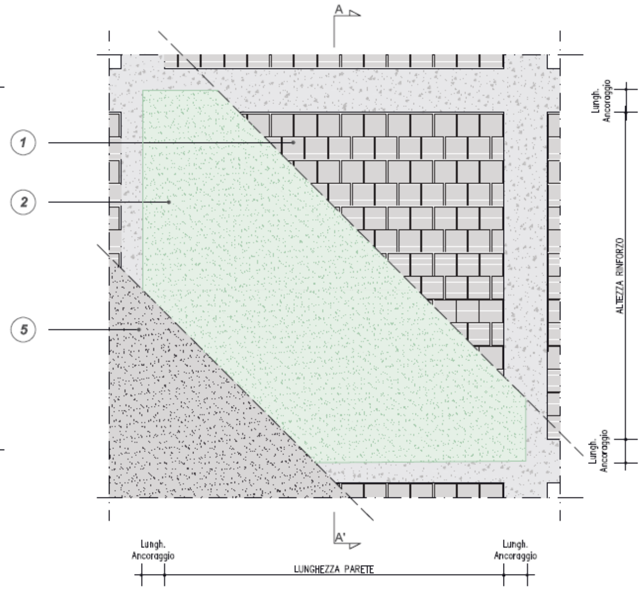


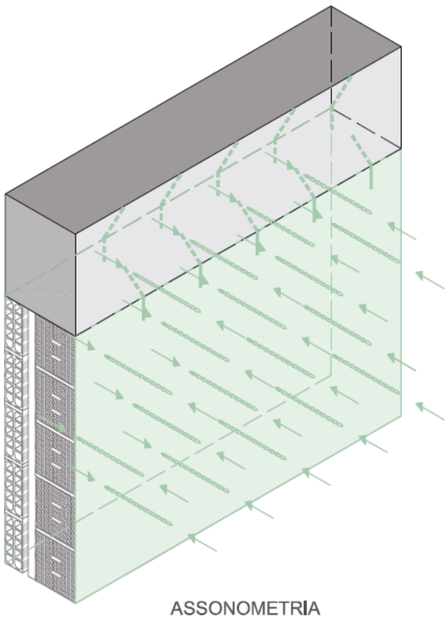
Figure 155 - Out-of-plane collapse mechanism



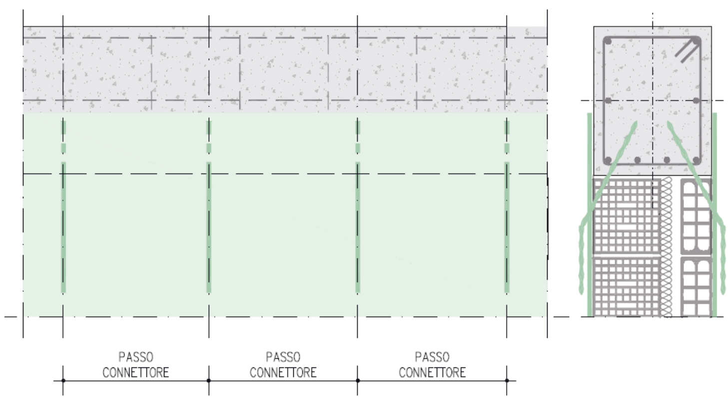
PROSPETTO
RINFORZO DELLA TAMPONATURA TRAMITE GEO GRID 120
IN ABBINAMENTO A GEOCALCE F ANTISISMICO E BARRE ELICOIDALI IN
ACCIAIO INOX



PIANTA
PRESIDIO ANTIRIBALTAMENTO MEDIANTE INTONACO TECNICO
COMPOSITO GEOCALCE® TENACE



ASSONOMETRIA
RINFORZO DELLA TAMPONATURA



To prevent possible collapse mechanisms of the infills, an anti-overturning system for the infills is proposed. This involves applying a biaxial network made of natural basalt fiber with a base plaster of pure lime and stitching it with helical stainless steel bars. The reinforcement will be properly connected to the adjacent structural elements (beams, columns, partitions) near the infills.

20 GLOBAL SEISMIC ANALYSIS EX-ANTE

The Italian legislation (D.M. 17/01/2018) for assessing the seismic resistance of masonry buildings and reinforced concrete buildings allows the application of the following types of analyses:

linear static;

linear dynamic;

nonlinear static (Pushover);

nonlinear dynamic.

- For the purpose of seismic safety assessment, a nonlinear static analysis (PUSHOVER) has been performed.

20.1 Results of the nonlinear static analysis - US1

In this chapter, the seismic vulnerability of the building analyzed in its current state will be addressed. In particular, the values of the risk indices related to the 32 seismic combinations will be reported.

Curve Pushover

Below, we will present the capacity curves obtained from the Pushover analysis, along with the curve corresponding to the most significant risk index. Finally, a summary table of the risk indices related to the structure in question will be provided.

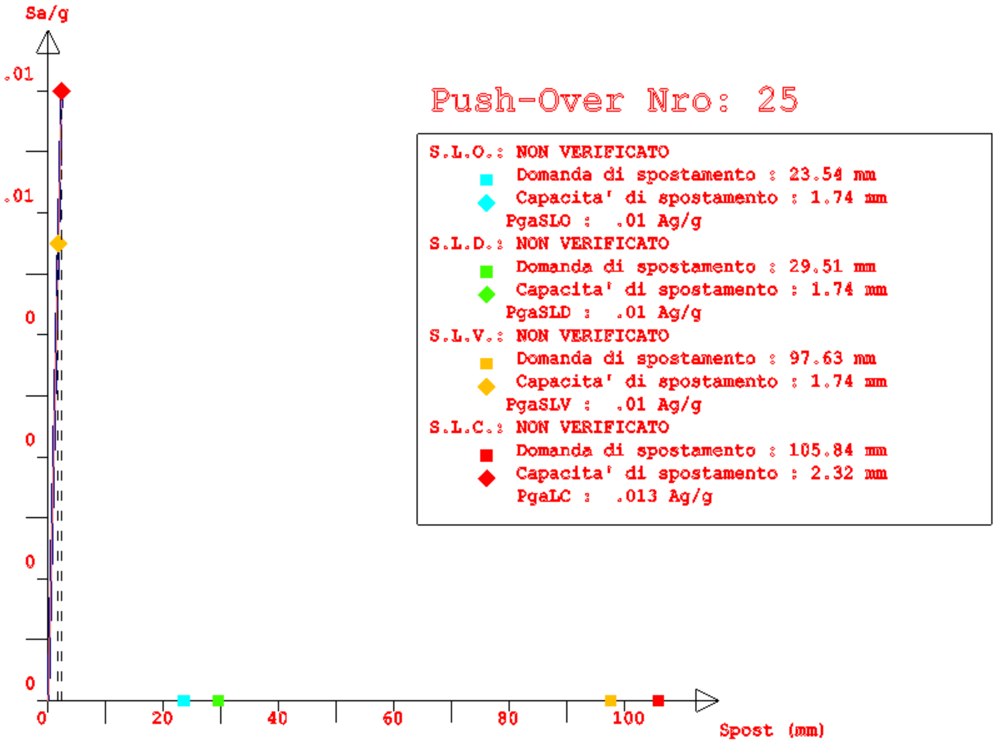


Figure 156– Curve of Pushover n.25

All the curves exhibit a brittle behavior of the building under seismic action due to the failure of the unconfined nodes. For the complete data of each curve, please refer to the tables attached.

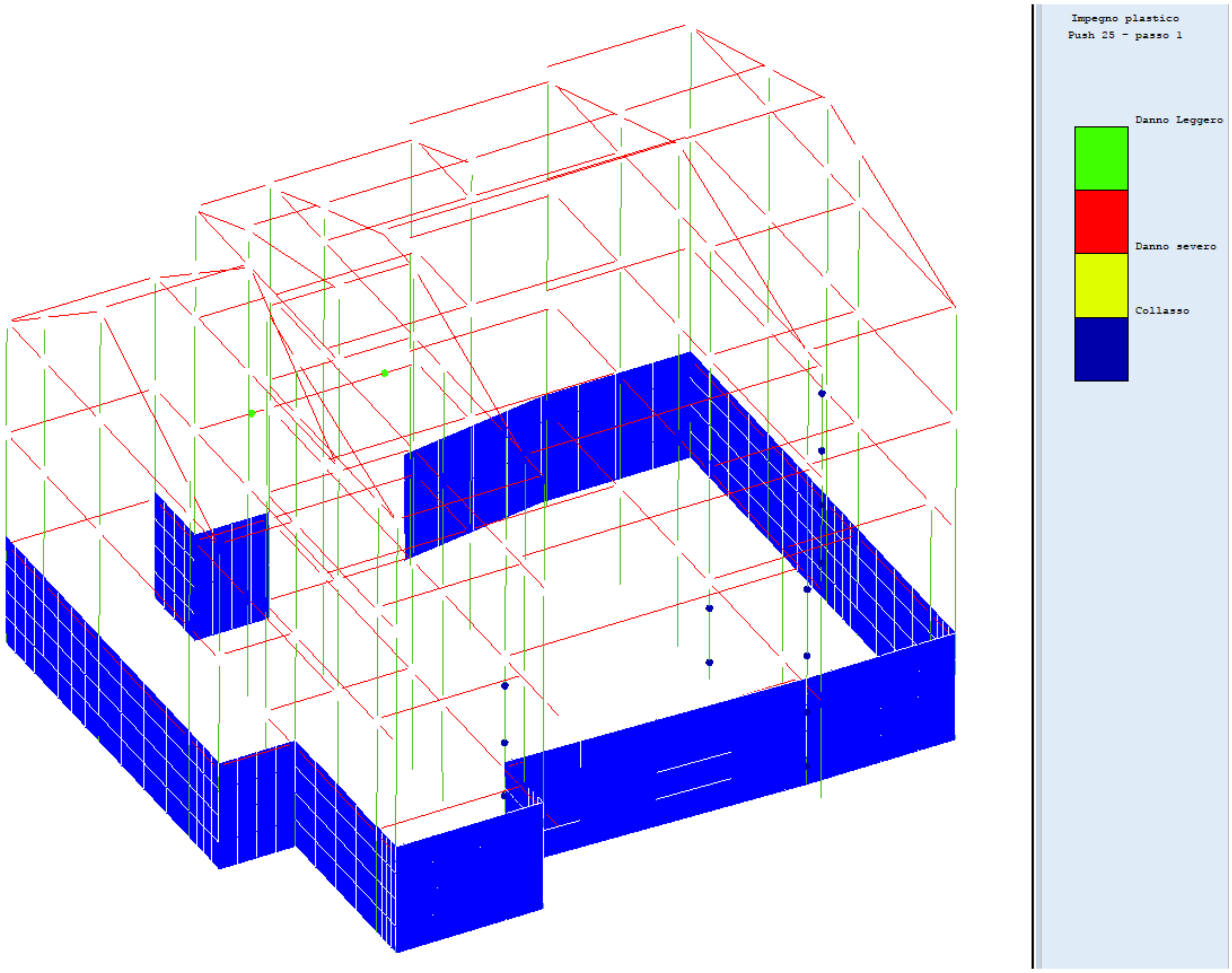


Figure 157– Formation of plastic hinges - deteriorating curve

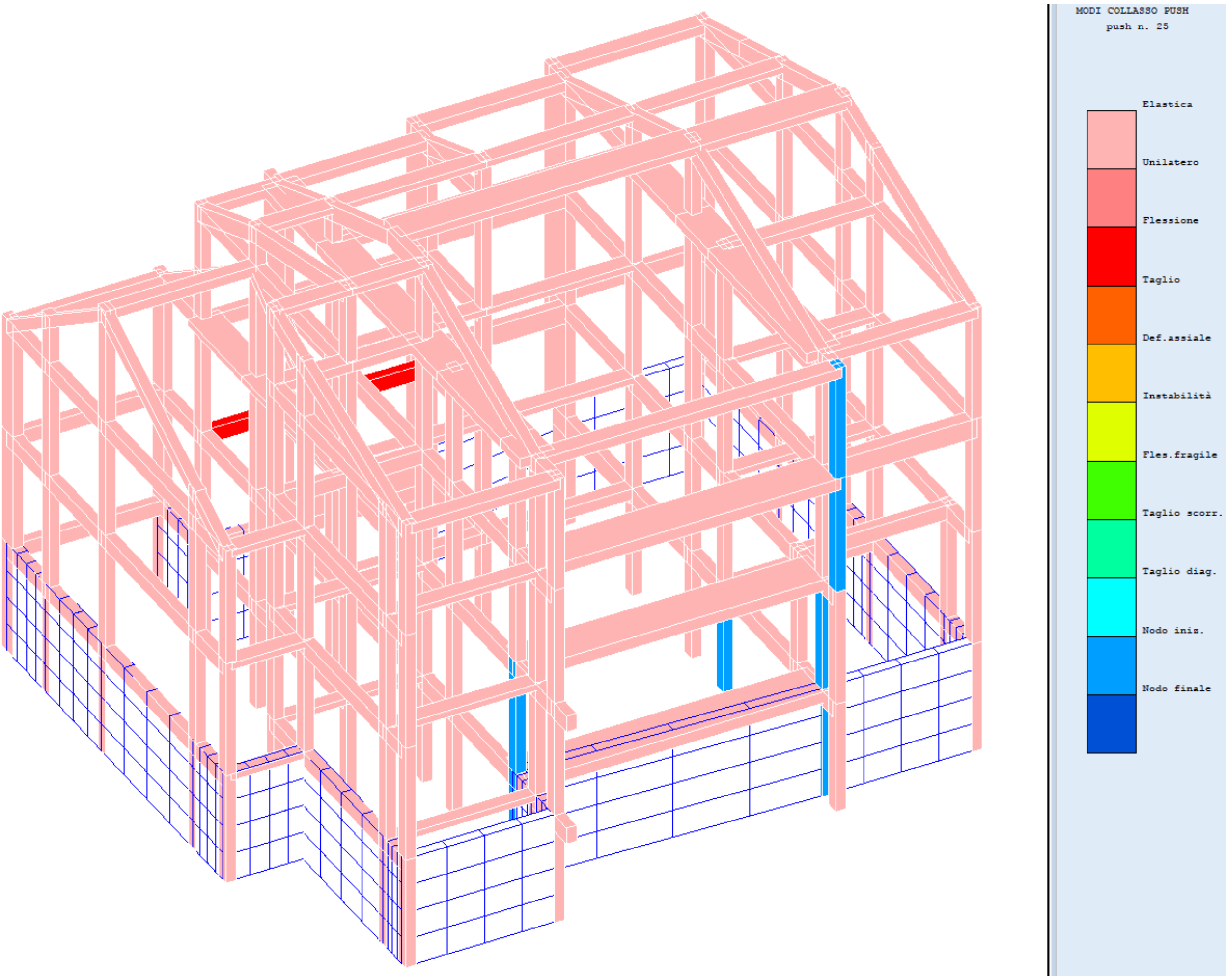


Figure 158– Formation of plastic hinges for collapse beam-column joint - deteriorating curve

Determination of risk indices

The level of seismic vulnerability of buildings is expressed in terms of RISK INDICATORS (ζE), where ζE = CAPACITY/DEMAND. If this value is > 1.00, the building's capacity to withstand seismic action exceeds what is required by regulations (demand). The lower the RI compared to this value, the more deficient the building structures are in withstanding seismic action.

Numero PushOver	PgaSLO/Pga81%	PgaSLD/Pga63%	PgaSLV/Pga10%	PgaSLC/Pga5%
1	.08	.064	.019	.023
2	.081	.064	.019	.024
3	.11	.088	.026	.032
4	.097	.077	.023	.028
5	.099	.079	.024	.029
6	.101	.08	.024	.03
7	.141	.113	.034	.042
8	.138	.11	.033	.04
9	.077	.061	.018	.023
10	.083	.066	.02	.024
11	.113	.09	.027	.033
12	.087	.069	.021	.025
13	.096	.076	.023	.028
14	.103	.082	.024	.03
15	.14	.111	.033	.041
16	.122	.098	.029	.036
17	.076	.061	.018	.022
18	.086	.068	.02	.025
19	.112	.09	.027	.033
20	.079	.063	.019	.023
21	.095	.075	.022	.028
22	.107	.085	.025	.031
23	.14	.111	.033	.041
24	.111	.088	.026	.033
25	.074	.059	.017	.021
26	.088	.07	.021	.026
27	.099	.079	.024	.029
28	.072	.058	.017	.021
29	.092	.073	.022	.027
30	.11	.088	.026	.032
31	.122	.097	.029	.036
32	.102	.081	.024	.03
Min. PgaSL/Pga%	.072	.058	.017	.021

Analyzing the results of the 32 curves in the ADSR plane, it is evident that THE BUILDING IS NOT SEISMICALLY ADEQUATE. The minimum risk indicator is reported with reference to the life safety, damage, and operability limit states in terms of acceleration (PGA):

Stato Limite	ζ_E (PGA_C/PGA_D)
SLV	0.017
SLD	0.058
SLO	0.072
SLC	0.021

20.2 Results of the nonlinear static analysis - US2

In this chapter, the seismic vulnerability of the building analyzed in its current state will be addressed. Specifically, the values of the risk indices related to the 32 seismic combinations will be reported.

Curve Pushover

Below, the capacity curves obtained from the Pushover analysis will be presented, along with the curve corresponding to the most significant risk index. Finally, a summary table of the risk indices related to the structure in question will be provided.

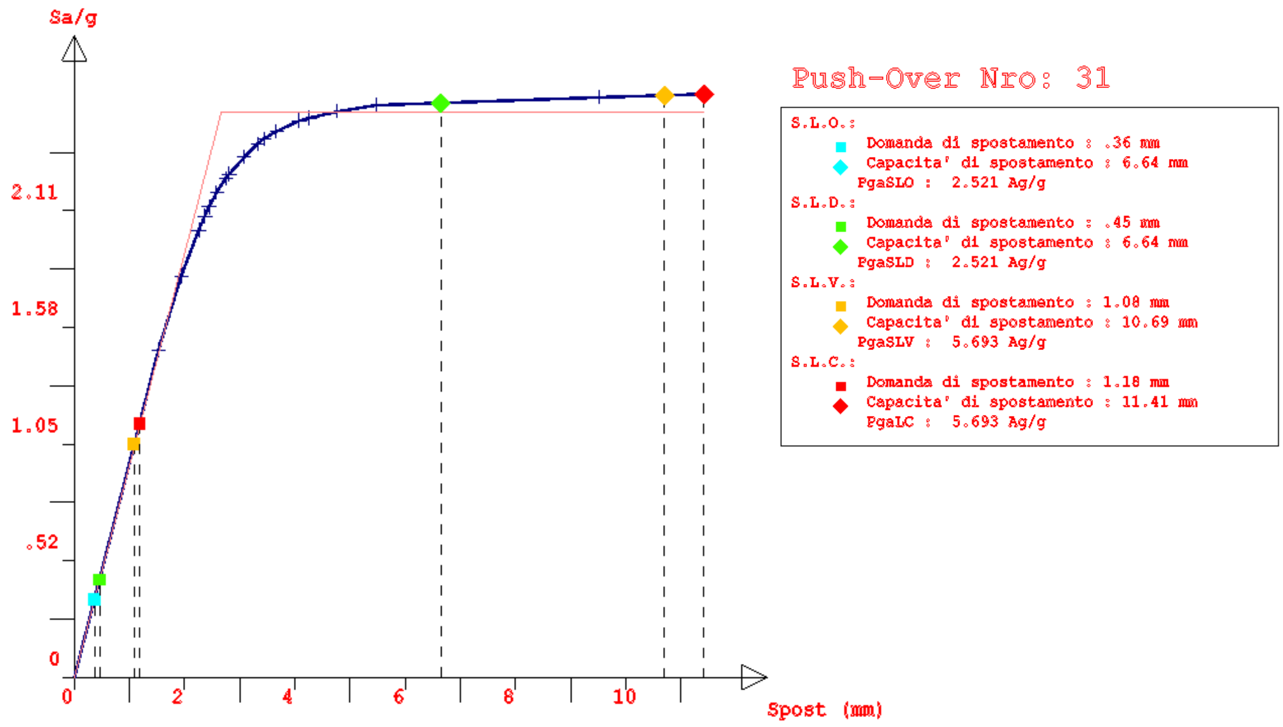


Figure 159 – Curve of Pushover n.31

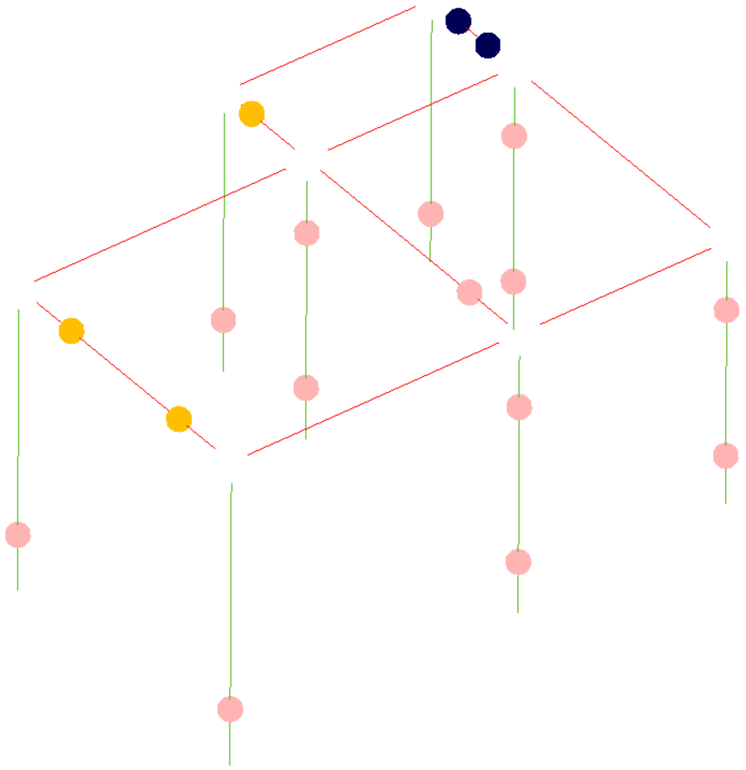


Figure 160–Formation of plastic hinges - deteriorating curve

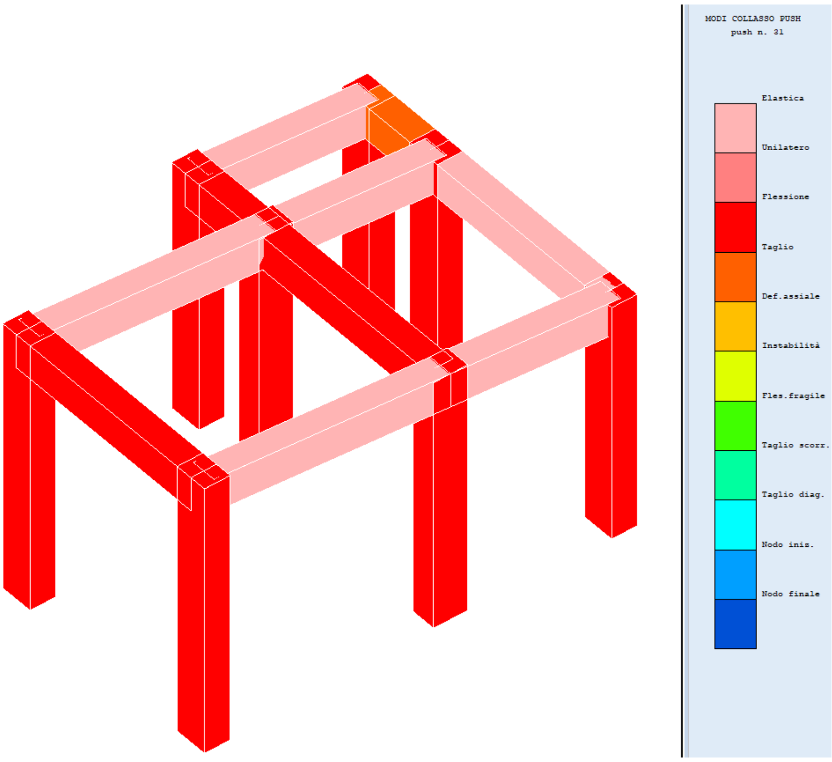


Figure 161–Formation of plastic hinges for beam-column joint collapse - deteriorating curve

Determining the risk indices

The level of seismic vulnerability of buildings is expressed in terms of RISK INDICATORS **(ζ_E)**, where **ζ_E = CAPACITY/DEMAND**, If this value is > 1,00the building's capacity to withstand seismic action exceeds what is required by regulations (demand). The lower the RI compared to this value, the more deficient the building structures are in withstanding seismic action.

Numero PushOver	PgaSLO/Pga81%	PgaSLD/Pga63%	PgaSLV/Pga10%	PgaSLC/Pga5%
1	21.234	18.333	18.576	22.208
2	21.665	20.329	26.321	31.665
3	20.384	16.26	10.261	10.94
4	16.146	12.88	20.37	25.868
5	21.234	18.333	18.576	22.208
6	21.665	20.329	26.321	31.665
7	20.384	16.26	10.261	10.94
8	16.146	12.88	20.37	25.868
9	20.092	17.272	18.32	21.998
10	21.382	23.554	25.334	30.484
11	20.43	16.297	10.664	12.31
12	17.04	13.593	20.36	25.695
13	20.092	17.272	18.32	21.998

14	21.382	23.554	25.334	30.484
15	20.43	16.297	10.664	12.31
16	17.04	13.593	20.36	25.695
17	21.406	17.75	18.293	21.951
18	19.732	21.962	25.336	30.636
19	19.164	15.287	11.945	12.049
20	25.34	20.214	24.314	29.855
21	21.406	17.75	18.293	21.951
22	19.732	21.962	25.336	30.636
23	19.164	15.287	11.945	12.049
24	25.34	20.214	24.314	29.855
25	21.519	18.363	18.731	22.412
26	19.863	22.806	25.478	30.587
27	18.266	14.571	9.85	9.936
28	25.957	20.705	24.28	29.663
29	21.519	18.363	18.731	22.412
30	19.863	22.806	25.478	30.587
31	18.266	14.571	9.85	9.936
32	25.957	20.705	24.28	29.663
Min. PgaSL/Pga%	16.146	12.88	9.85	9.936

Analyzing the results of the 32 curves in the ADSR plane, it is evident that THE BUILDING IS SEISMICALLY ADEQUATE. The minimum risk indicator is reported with reference to the life safety, damage, and operability limit states, in terms of acceleration (PGA):

Stato Limite	ζ_E (PGA_C/PGA_D)
SLV	9.85
SLD	12.88
SLO	16.146
SLC	9.936

21 CONCLUSIONS EX-ANTE

Based on the information provided in the previous chapters and after determining the minimum index resulting from the global analysis, it is considered that the structural unit in question exhibits a high degree of seismic risk.

21.1 Reporting the results of verifications - US1

ANALISI STATICA LINEARE NON SISMICA EX ANTE		
PRESSOFLESSIONE	100 % elementi verificati	VERIFICA SODDISFATTA
TAGLIO	100 % elementi verificati	VERIFICA SODDISFATTA

ANALISI STATICA NON LINEARE		
SLV	ζ_E (PGA_C/PGA_D) = **0.117**	VERIFICA NON SODDISFATTA
SLD	ζ_E (PGA_C/PGA_D) = **0.058**	VERIFICA NON SODDISFATTA
SLO	ζ_E (PGA_C/PGA_D) = **0.072**	VERIFICA NON SODDISFATTA
SLC	ζ_E (PGA_C/PGA_D) = **0.021**	VERIFICA NON SODDISFATTA

The Structural Unit exhibits HIGH SEISMIC RISK due to the brittle failure of the unconfined beam-column joints.

21.2 Reporting the results of verifications - US2

ANALISI STATICA LINEARE NON SISMICA EX ANTE		
PRESSOFLESSIONE	100 % elementi verificati	VERIFICA SODDISFATTA
TAGLIO	100 % elementi verificati	VERIFICA SODDISFATTA

ANALISI STATICA NON LINEARE		
SLV	ζ_E (PGA_C/PGA_D) = **9.85**	VERIFICA SODDISFATTA
SLD	ζ_E (PGA_C/PGA_D) = **12.88**	VERIFICA SODDISFATTA
SLO	ζ_E (PGA_C/PGA_D) = **16.146**	VERIFICA SODDISFATTA
SLC	ζ_E (PGA_C/PGA_D) = **9.936**	VERIFICA SODDISFATTA

The structural unit is deemed seismically adequate with a LOW SEISMIC RISK.

22 ANALYSIS OF INTERVENTIONS FOR REDUCING SEISMIC VULNERABILITY.

Vulnerability is an inherent characteristic of constructions and can be defined as the probability that certain seismic actions correspond to specific levels and types of damage. Although every building is different from the others, it is possible to identify common traits in the modes of damage that can be referenced both during the damage analysis phase and in the phase of hypothesizing alternative interventions to reduce such seismic vulnerability. Their evaluation is of fundamental importance because, when the seismic action of project occurs, the structure as a whole must be able to function as a system in which all ductile elements dissipate, proportionally to their capacity, the energy supplied by the earthquake in the form of inelastic deformations.

23 MAIN TYPES OF SEISMIC DAMAGE IN REINFORCED CONCRETE STRUCTURES.

- Here's a list of the main causes of damage identified in the structural unit that requires interventions:
-
- Brittle failure of beam-column joints without confinement;
- Excessive displacement

24 PRINCIPAL INTERVENTION

The objectives of a consolidation project are to identify the structural behavior of the building, assess the residual structural safety, determine the safety coefficient, and finally identify structural improvement interventions. The choice of type, technique, and extent of the intervention depends on the results obtained in the vulnerability assessment phase. The aim should be to counteract the development of local and/or fragile mechanisms and to improve the overall behavior of the construction.

24.1 Reinforcement of beam-column joints using metal plates

To enhance the node's resistance capacity, reinforcing the node through plating with a shaped steel plate and fixing it with mechanical anchoring is proposed. This intervention:

Ensures confinement with increased strength and ductility of the node.

Does not involve any increase in the geometry of the elements.

Does not entail any increase in the mass and rigidity of the structural elements.

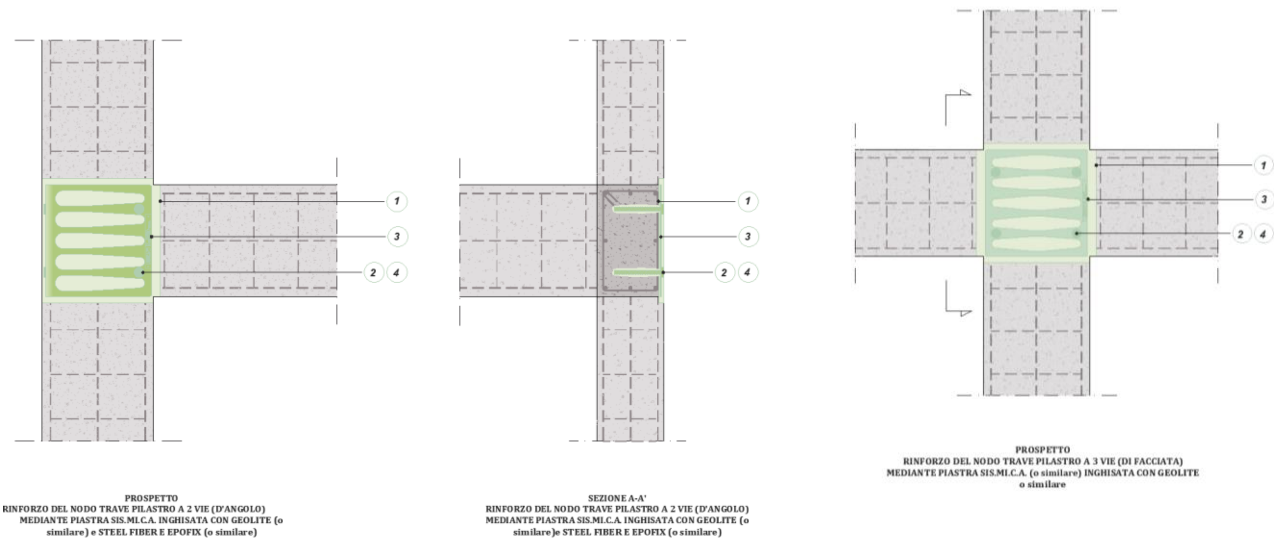


Figure 162– Detail of the beam-column joint intervention (façade and corner)

24.2 Stiffening with steel bracing

Here is a step-by-step description of the proposed intervention for inserting steel bracings:

Removal of Existing Fixtures: Remove any existing fixtures or elements that may obstruct the installation of the steel bracings.

Insertion of Steel Bracing Elements: Install the steel bracing elements in predetermined localized areas of the structure where they will provide the most effective reinforcement against lateral movements. These bracing elements should be designed to absorb a significant portion of the seismic forces.

Connection of Steel Elements to Existing Columns: Connect the steel bracing elements securely to the existing columns using plates and bolt connections. This step ensures proper load transfer and structural integrity.

Refinishing and Reinstallation of Fixtures: After the steel bracing elements are securely installed and connected, refinish the affected areas and reinstall any fixtures or elements that were removed in the first step. This ensures that the structural enhancements blend seamlessly with the existing building aesthetics.

The objective of this intervention is to have the steel bracing elements absorb a significant portion of the seismic forces and enhance the structural rigidity of the building by modifying its vibration modes.

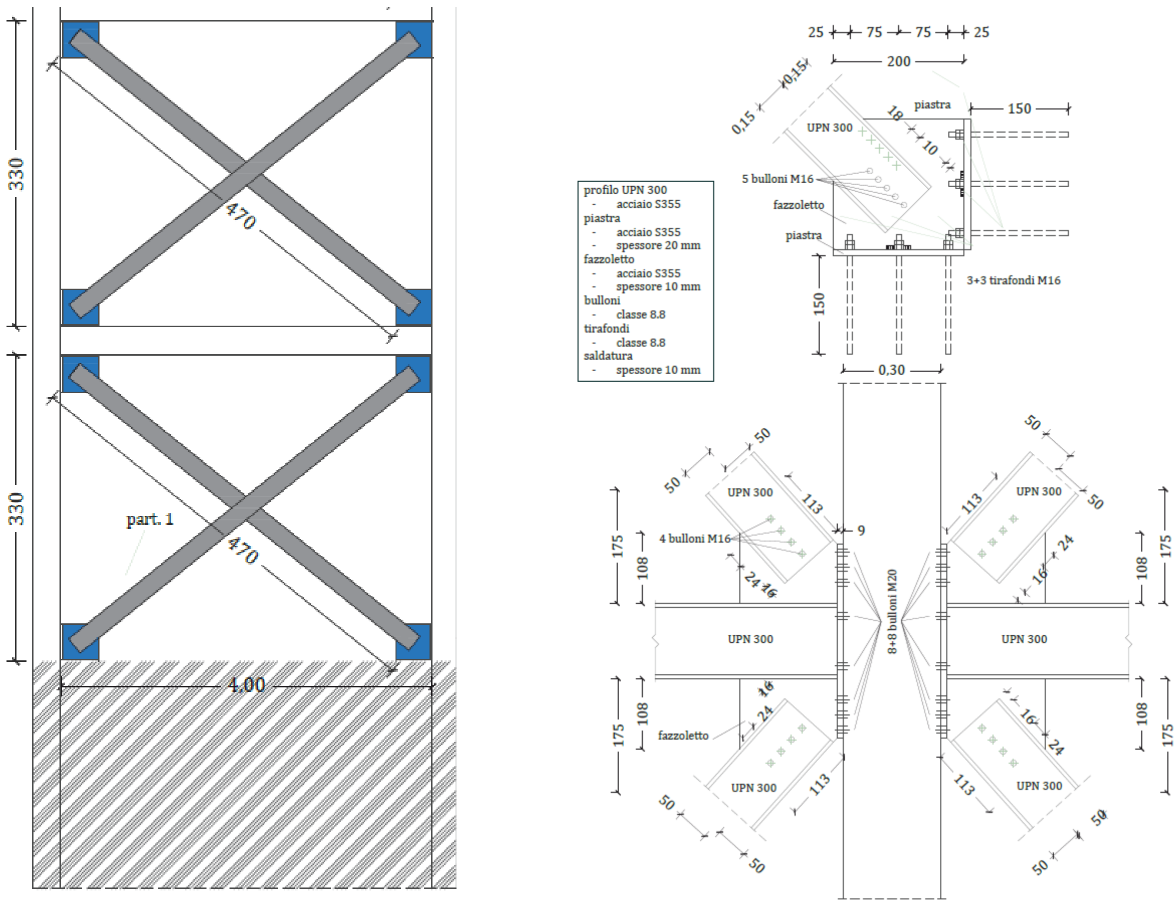


Figure 163 - Detail of reinforcement intervention with steel bracings

24.3 Reinforcement of beams by cladding with galvanized UHTSS steel fiber fabrics with epoxy adhesive

Here is an image illustrating the flexural reinforcement process described:

This reinforcement involves applying galvanized ultra-high tensile strength steel fiber fabric with epoxy adhesive to both the intrados and extrados of the beams to enhance their flexural capacity, reduce deformations under service loads, and limit cracking.

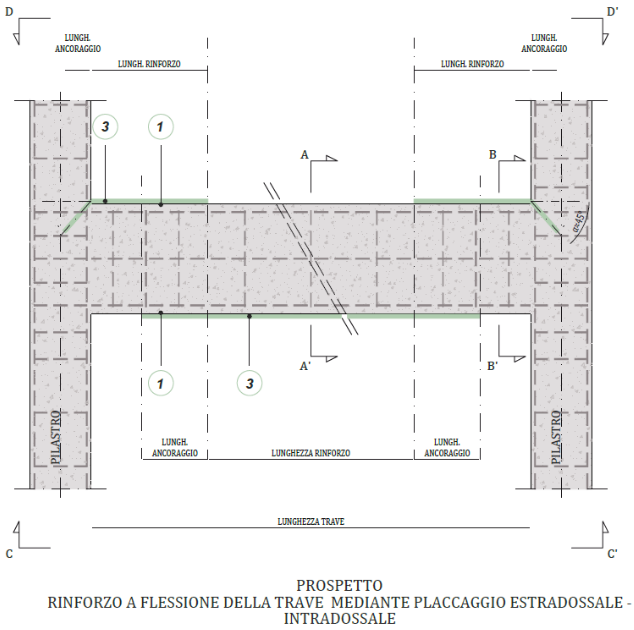


Figure 164–Detail of flexural reinforcement for beams.

Here is an image illustrating the reinforcement process described:

[This reinforcement involves applying galvanized steel fiber fabric and structural mineral thixotropic mortar in a U-wrap configuration around the beams to enhance their shear strength, reduce deformations under service loads, and limit cracking.

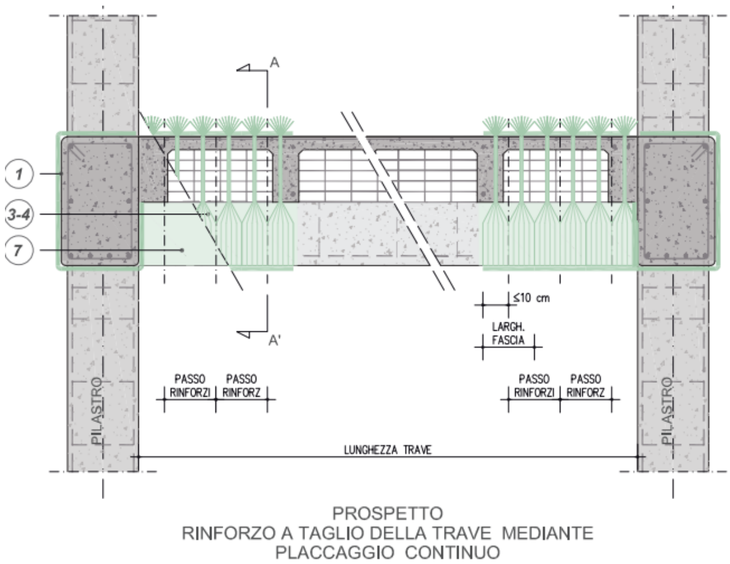


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Figure 165 - Detail of shear reinforcement for beams

24.4 Consolidation of columns with steel jacketing.

This intervention involves the installation of a steel jacket around the structural columns to enhance their shear strength, ductility, joint efficiency, and vertical load-carrying capacity through confinement. The steel jacket, made of S355 steel, will consist of four angle profiles onto which continuous steel plates (stirrups) will be welded.

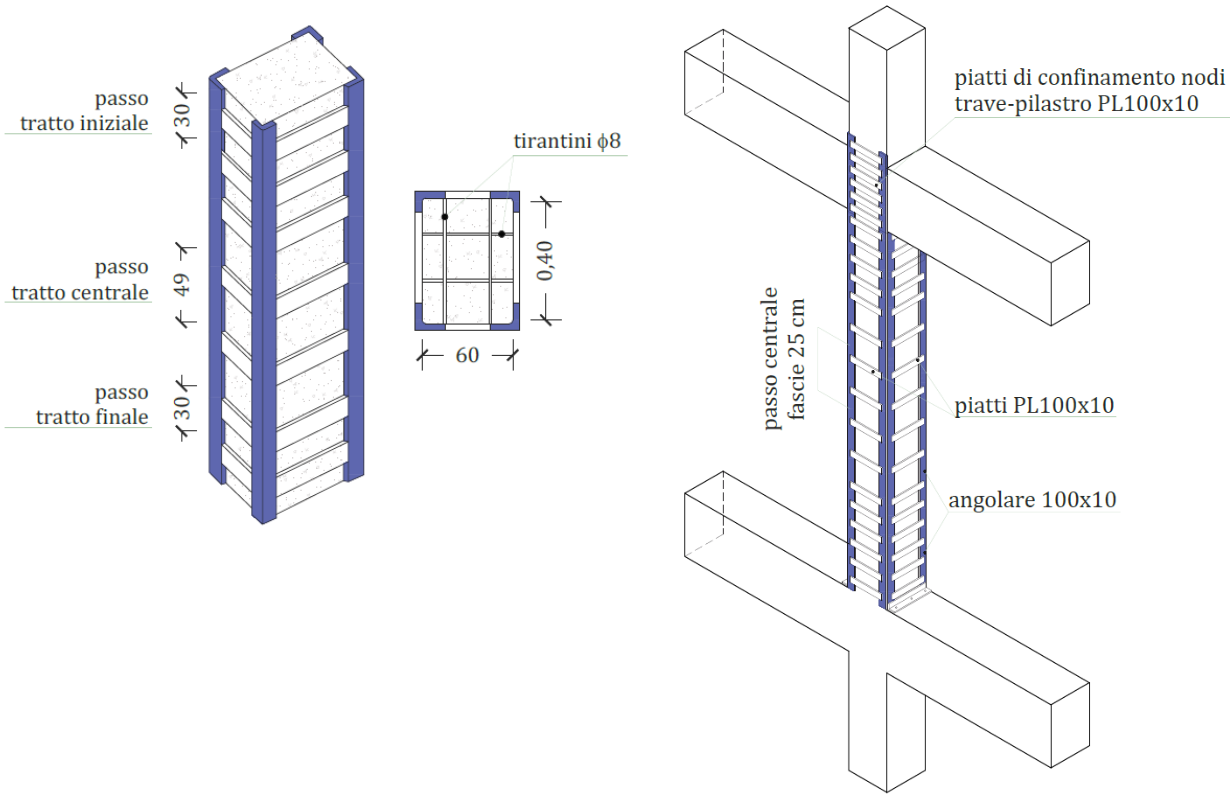


Figure 56 - Detail of steel jacketing for columns.

25 GLOBAL SEISMIC ANALYSIS EX POST

25.1 Results of the nonlinear static analysis US1

Intervention

Below are the results of the overall seismic vulnerability analysis obtained for the structural unit in question following the following interventions:

- Strengthening of beam-column joints;
- Shear and flexural reinforcement of beams;
- Steel jacketing of columns;
- Stiffening with steel bracing.

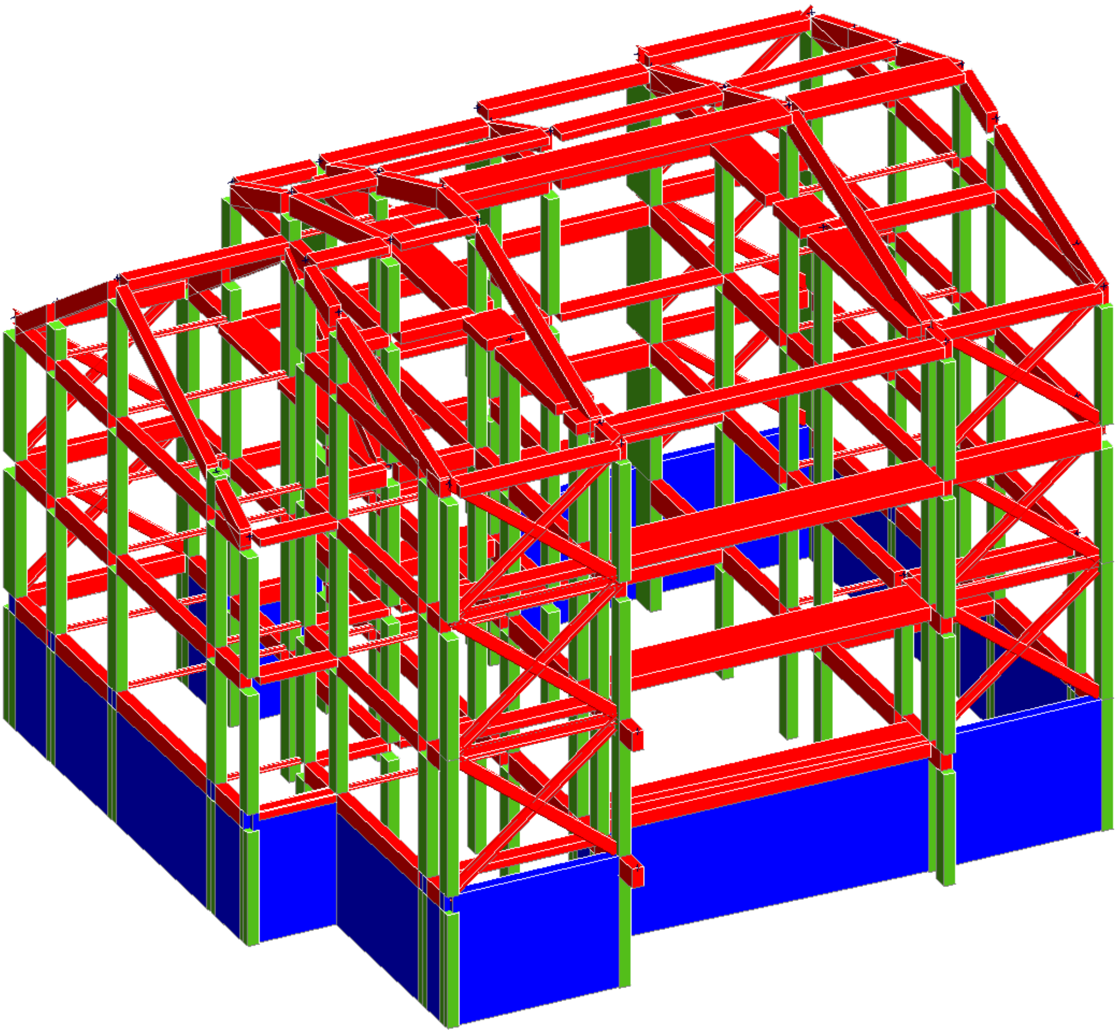


Figure 167 - Mechanical model of Structural Unit 1

Following the intervention, the structure exhibits good behavior under seismic conditions.

Pushover's Curve

Below, the most deteriorating capacity curve obtained from the Pushover analysis will be presented, followed by the failure related to the most significant risk index, and finally, the summary table of risk indices related to the structure in question will be provided.

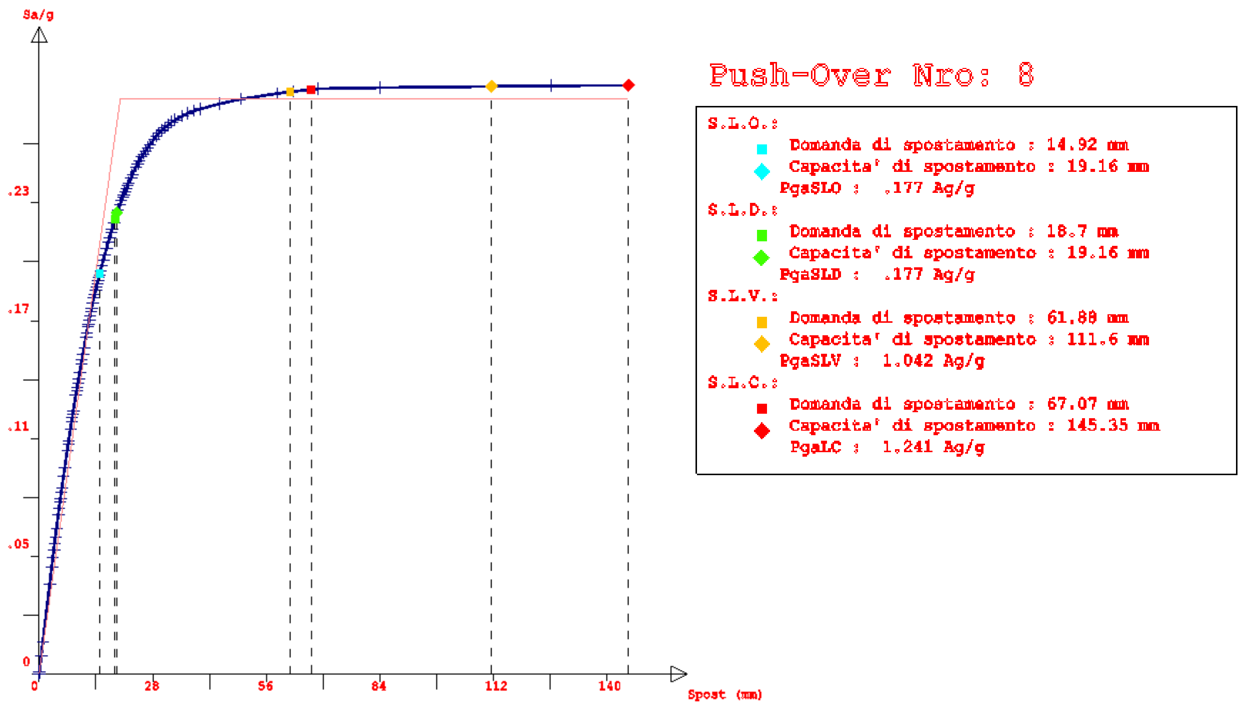


Figure 168 - Curve of Pushover n.8

All the curves demonstrate the building's good seismic action performance. For the complete data of each curve, please refer to the attached tables. The intervention adopted has increased the overall ductility of the structure, reducing the occurrence of fragile mechanisms. The figures indicating the formation of plastic hinges in the structure following the intervention for the same curve are provided.

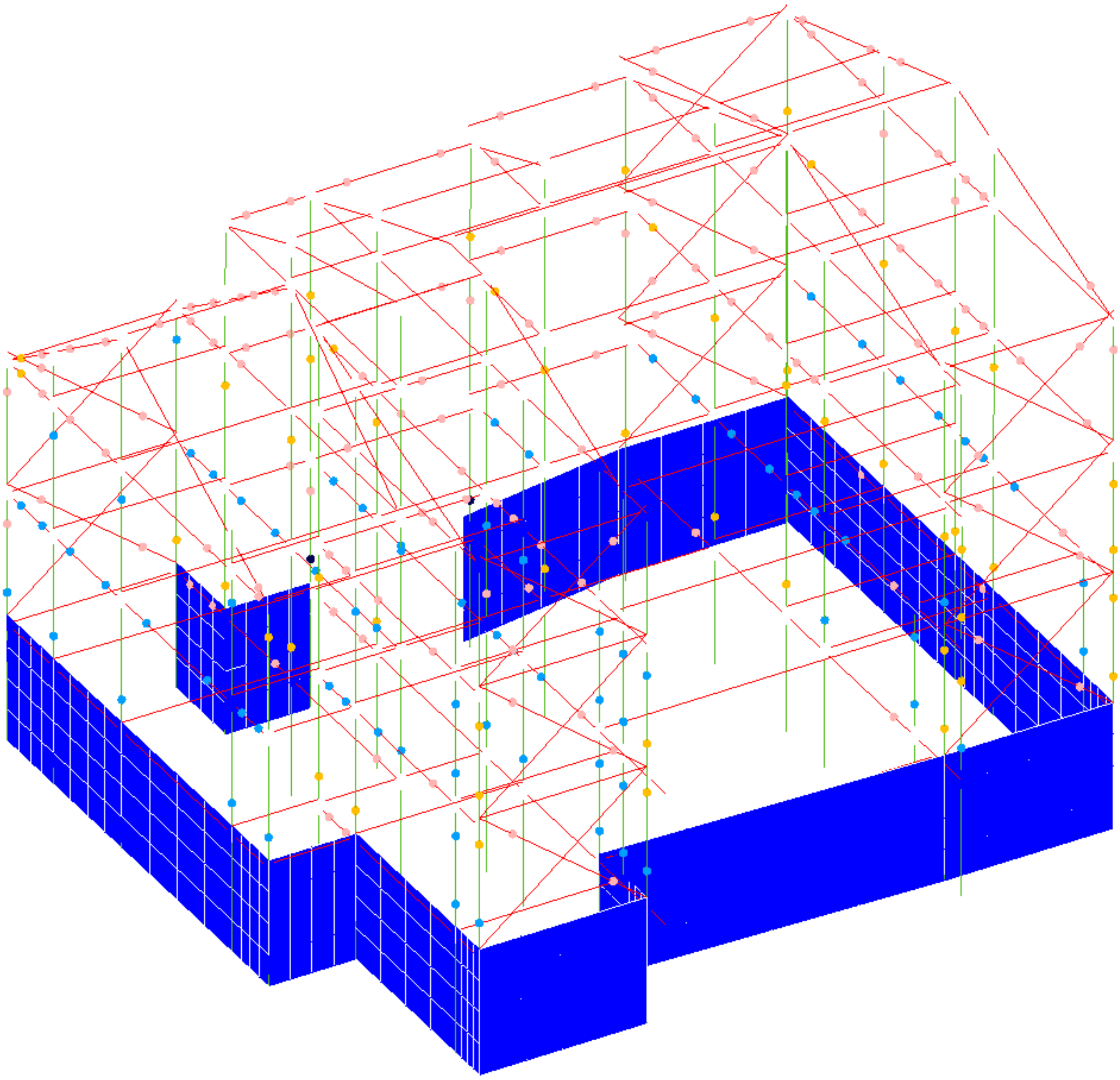


Figure 169- Formation of plastic hinges following the intervention

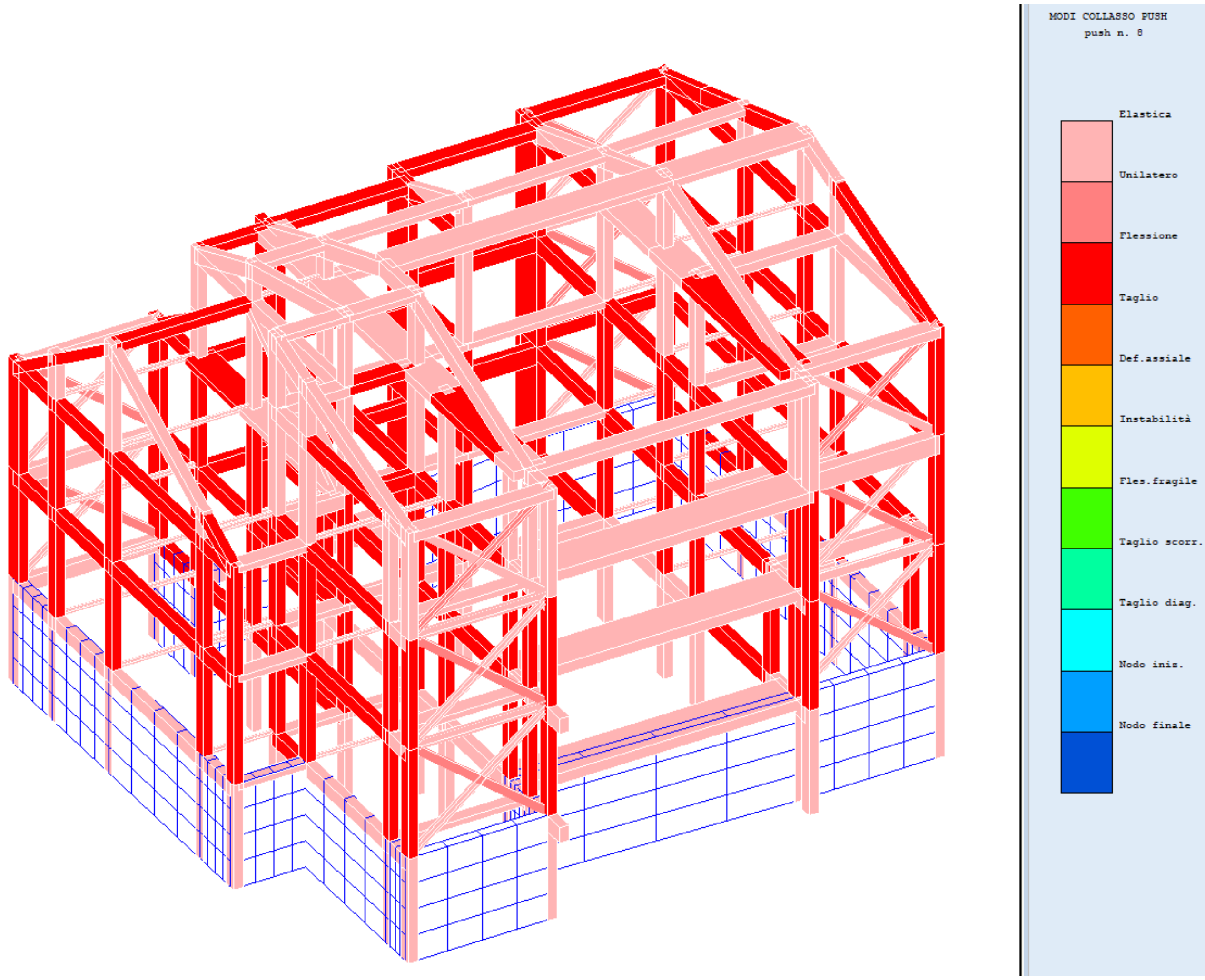


Figure 170 - Formation of plastic hinges

Determination of risk indices

The level of seismic vulnerability of buildings is expressed in terms of the risk index *ζE* where *ζE*=CAPACITY/DOMANDA. If this value is > 1.00 (§ 8.4.3 NTC 2018), the building's capacity to withstand seismic action exceeds what is required by the regulations (demand). The lower the risk index is below this value, the more deficient the building's structures are in withstanding seismic action.

Numero PushOver	PgaSLO/Pga81%	PgaSLD/Pga63%	PgaSLV/Pga10%	PgaSLC/Pga5%
1	1.49	1.467	1.232	1.373
2	1.523	1.291	1.246	1.26
3	1.124	1.133	1.252	1.483
4	1.037	1.024	1.466	1.757
5	1.874	1.734	1.613	1.718
6	1.931	1.54	2.145	2.322
7	1.42	1.133	1.572	1.87
8	1.284	1.024	1.802	2.166
9	1.59	1.424	1.625	1.855
10	1.689	1.348	2.057	2.296
11	1.118	1.126	1.293	1.54
12	1.136	1.113	1.554	1.873
13	1.865	1.705	1.699	1.901
14	2.035	1.629	1.772	1.787
15	1.412	1.126	1.597	1.911
16	1.396	1.113	1.91	2.281
17	1.587	1.266	1.604	1.662
18	1.655	1.471	1.443	1.512
19	1.088	1.066	1.253	1.497
20	1.117	1.129	1.445	1.724
21	1.905	1.52	1.116	1.126
22	1.938	1.762	1.342	1.437
23	1.336	1.066	1.559	1.872
24	1.415	1.129	1.787	2.126
25	1.477	1.239	1.267	1.278
26	1.443	1.536	1.485	1.766
27	1.091	1.126	1.325	1.581
28	1.093	1.113	1.51	1.797
29	1.866	1.488	1.061	1.07
30	1.802	1.853	1.014	1.023
31	1.359	1.084	1.638	1.968
32	1.377	1.098	1.857	2.223
Min. PgaSL/Pga%	1.037	1.024	1.014	1.023

Analyzing the results of the 32 curves in the ADSR plane, it is evident that THE BUILDING IS SEISMICALLY ADEQUATE. The minimum risk indicator in reference to the life safety limit state, in terms of acceleration (PGA), is reported below:

Stato Limite	ζ_E (PGA_C/PGA_D)
SLV	1.014
SLD	1.024
SLO	1.037
SLC	1.023

26 CONCLUSION EX-POST

Through the necessary interventions to increase seismic safety outlined in the previous chapters, a risk coefficient higher than the minimum required by the NTC2018 has been achieved. Below are the reports detailing the results of the verifications of the Structural Unit comprising the building.

26.1 Report of verification results – US1

ANALISI STATICA LINEARE NON SISMICA		
PRESSOFLESSIONE	100 % elementi verificati	VERIFICA SODDISFATTA
TAGLIO	100 % elementi verificati	VERIFICA SODDISFATTA

ANALISI STATICA NON LINEARE		
SLV	ζ_E (PGA_C/PGA_D) = **1.014**	VERIFICA SODDISFATTA
SLD	ζ_E (PGA_C/PGA_D) = **1.024**	VERIFICA SODDISFATTA
SLO	ζ_E (PGA_C/PGA_D) = **1.037**	VERIFICA SODDISFATTA
SLC	ζ_E (PGA_C/PGA_D) = **1.023**	VERIFICA SODDISFATTA

Based on the results obtained, the Structural Unit as a whole, in accordance with §8.4.3 of the NTC2018, is deemed seismic compliant and suitable, therefore, to withstand the seismic actions anticipated for the building.

27 STRUCTURAL MODEL EX-POST

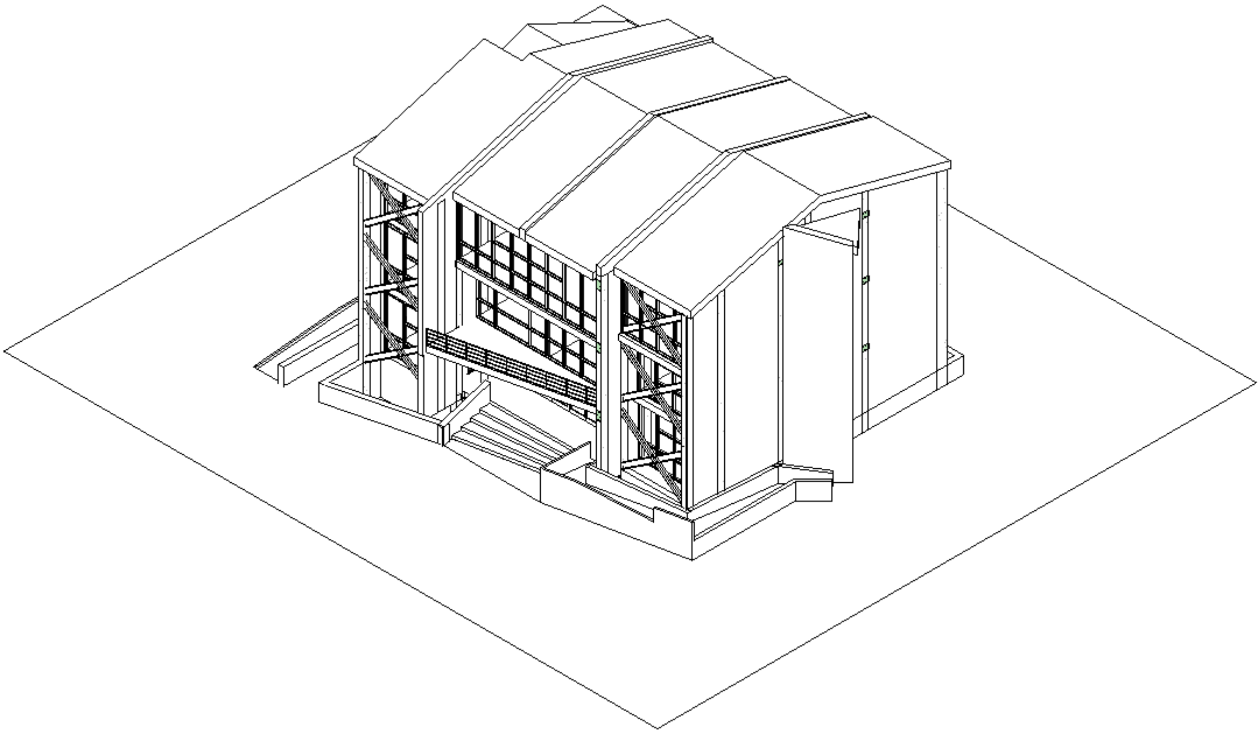


Figure 171:3d model of ex-post

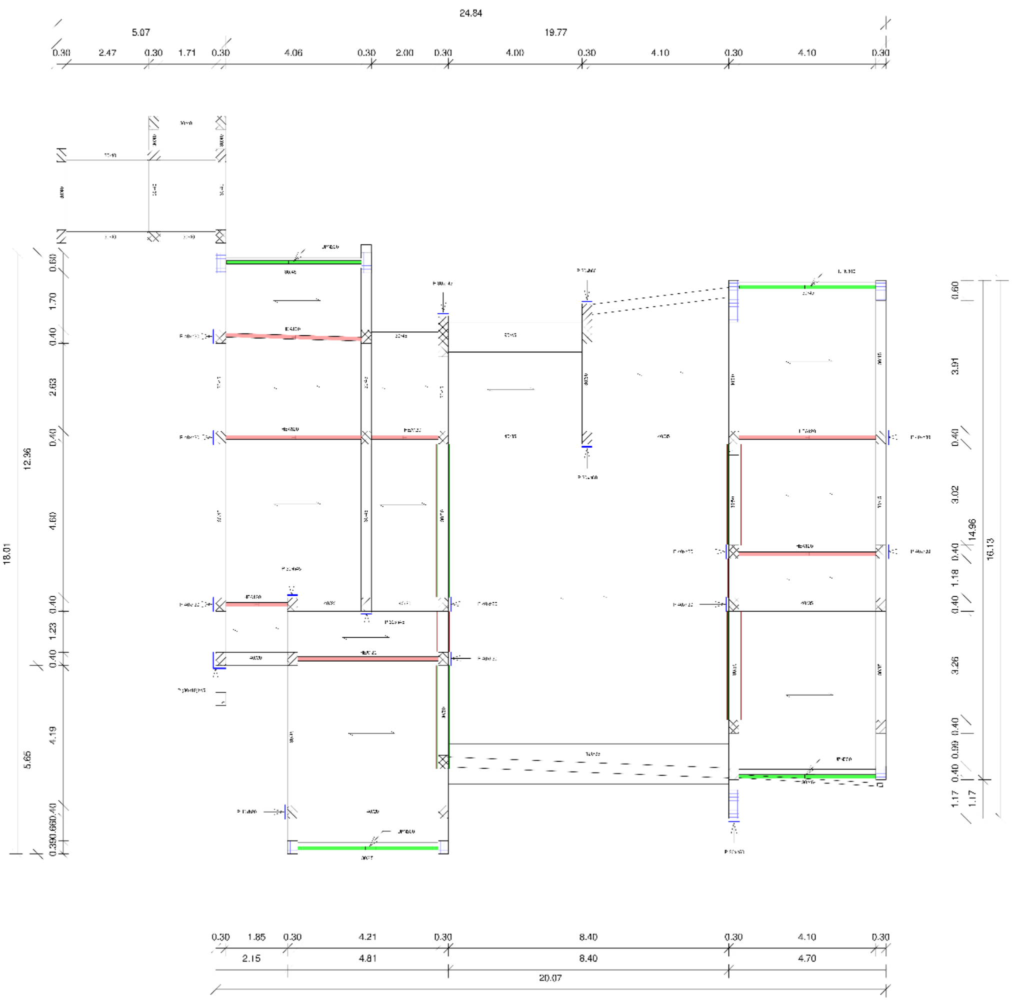


Figure 172– Structural plan–Slab of Underground floor

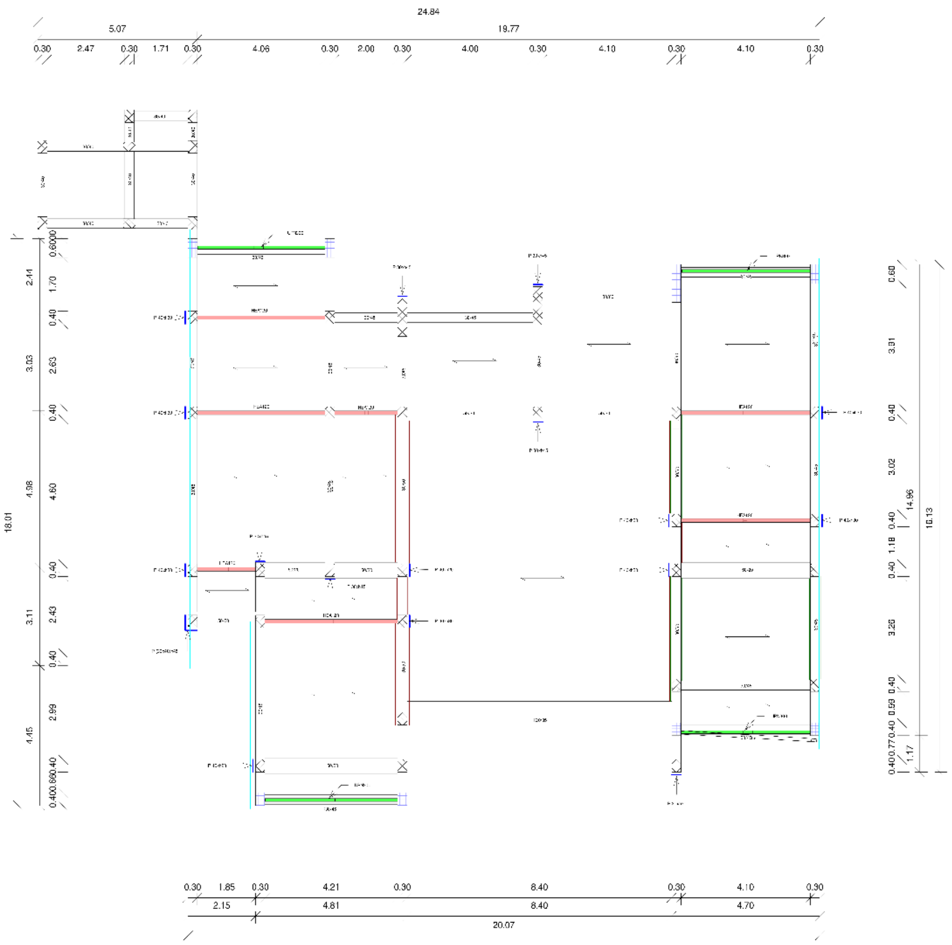


Figure 173– Structural plan–Slab of ground floor

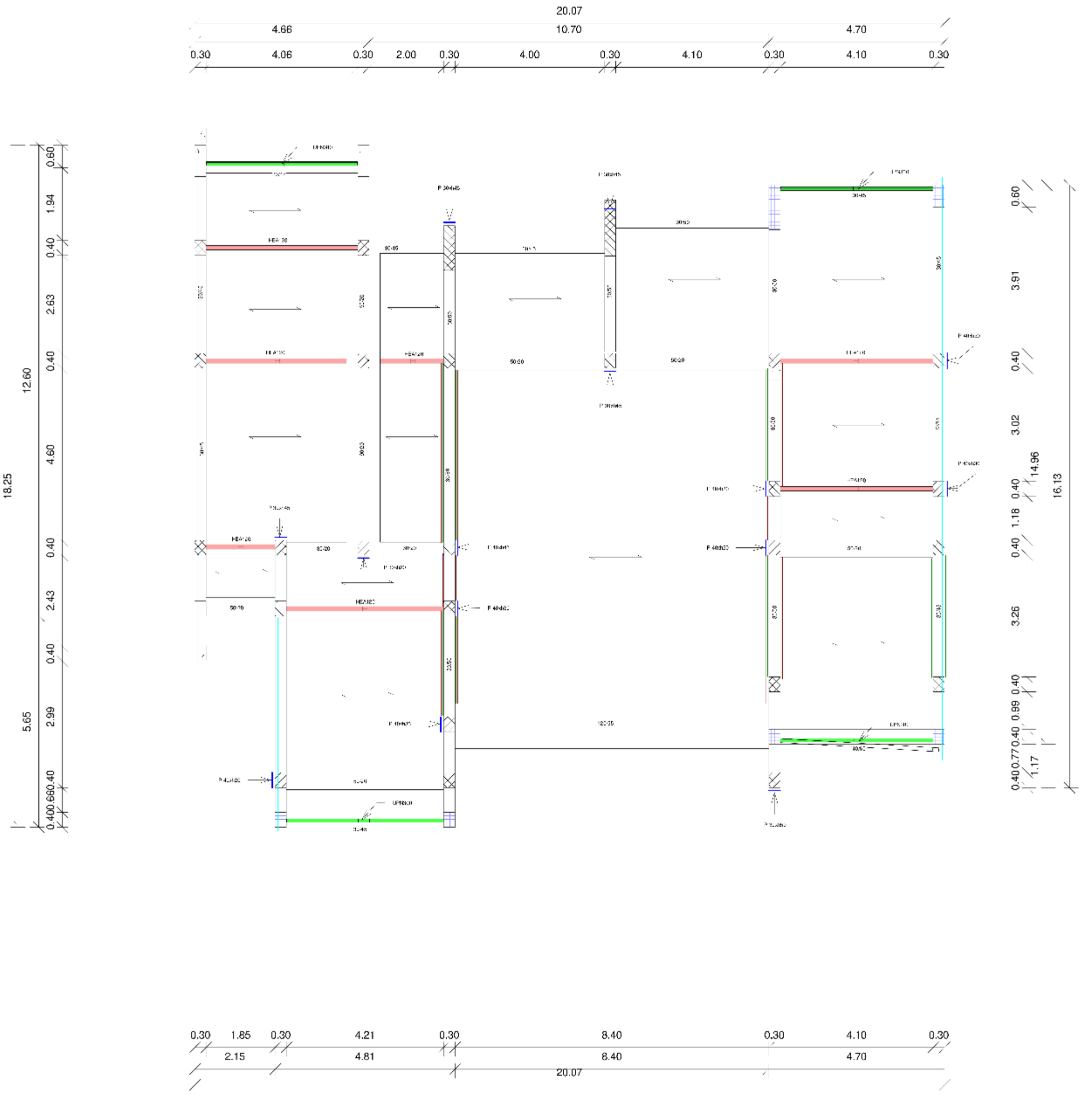


Figure 174– Structural plan–Slab of the first floor

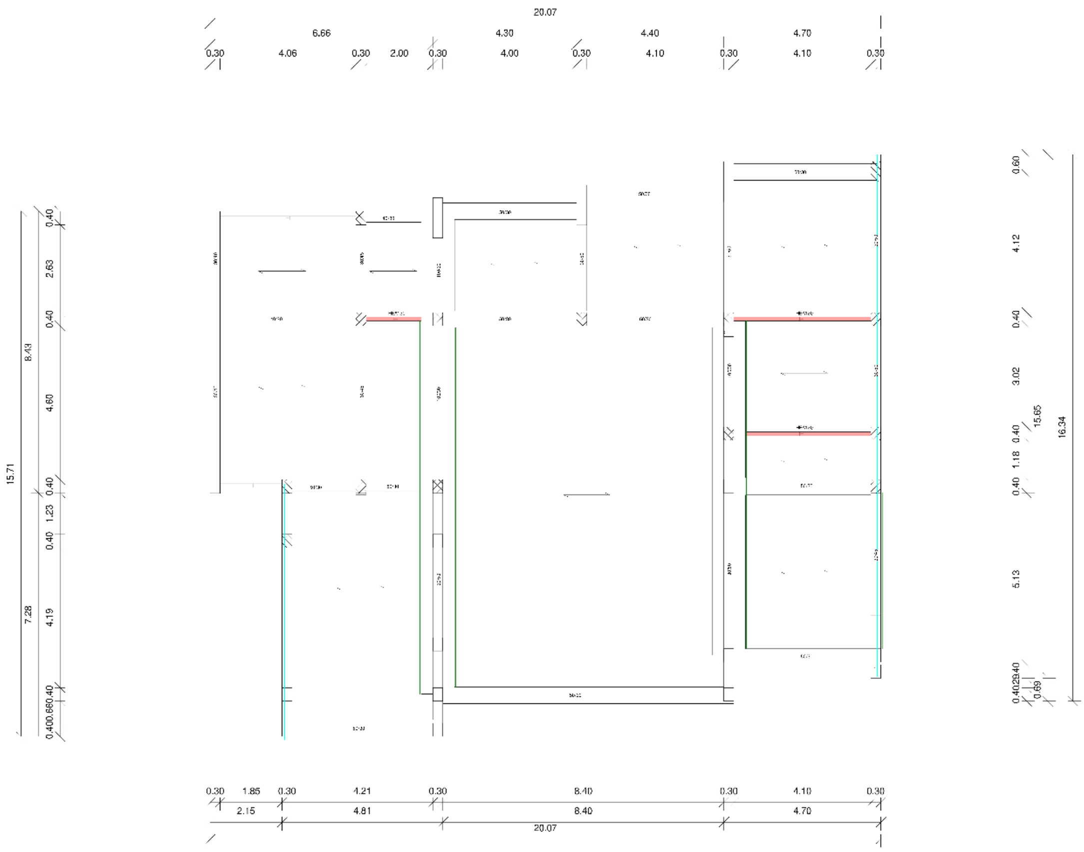


Figure 175– Structural plan–Slab of the Second floor

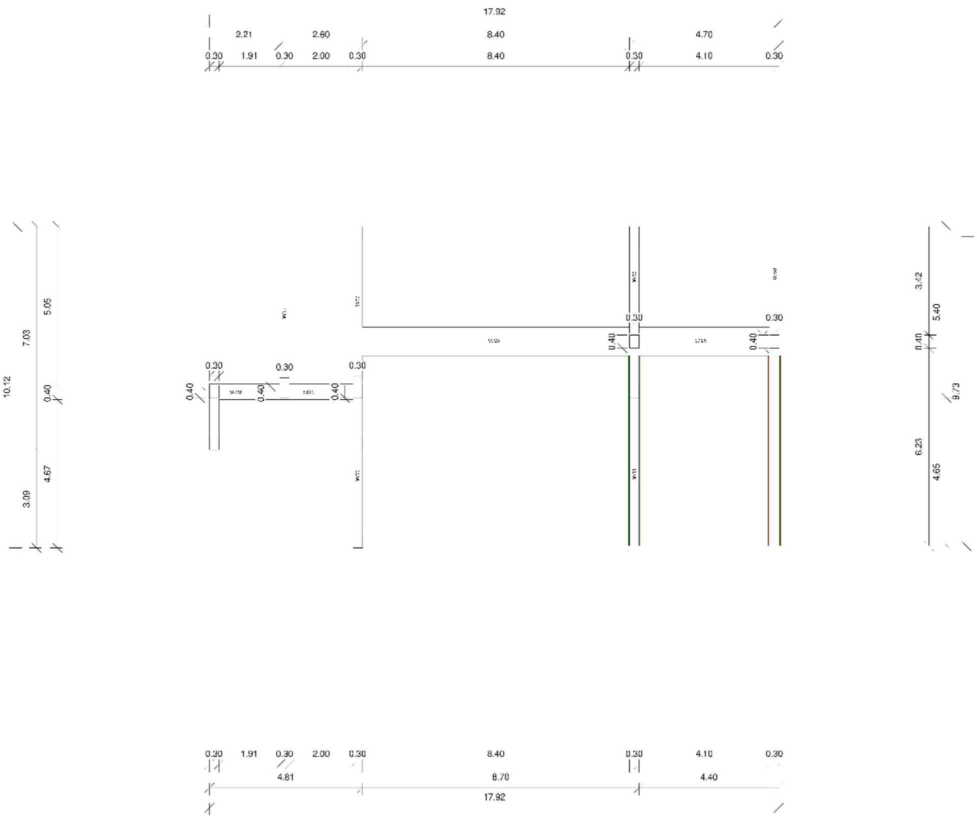
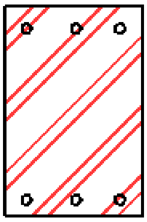


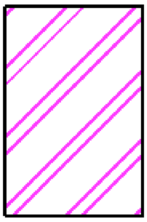
Figure 176– Structural plan–Slab of the Third Floor

STRUCTURAL INTERVENTIONS

Interventions aimed at achieving seismic improvement



Restoration of plaster for confinement plates of beam/column joints



Restoration of plaster for anti-overturning intervention

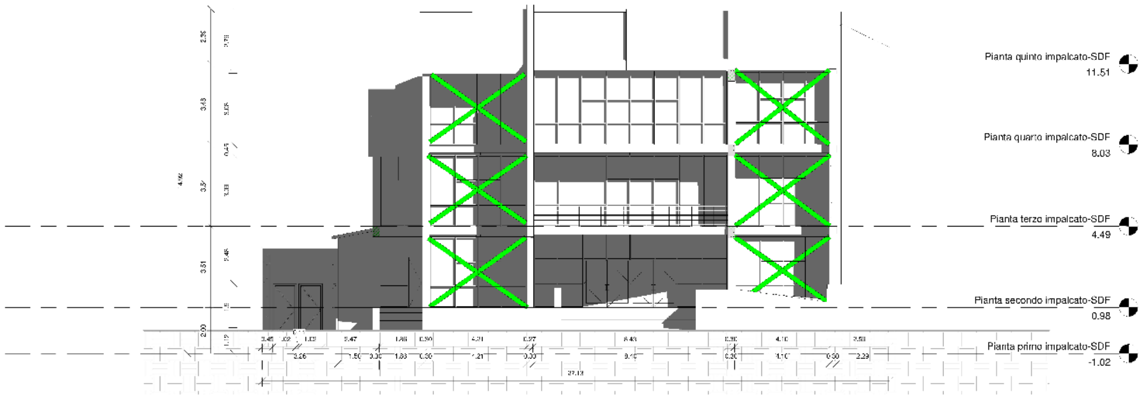


Figure 177– Northern view

Figure 178– southern view

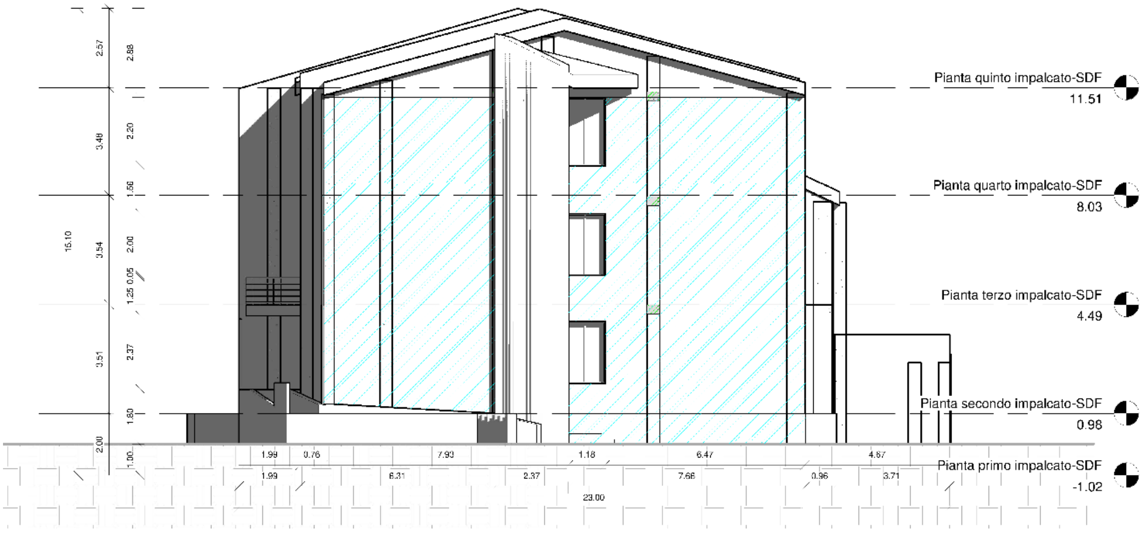


Figure 179– eastern view

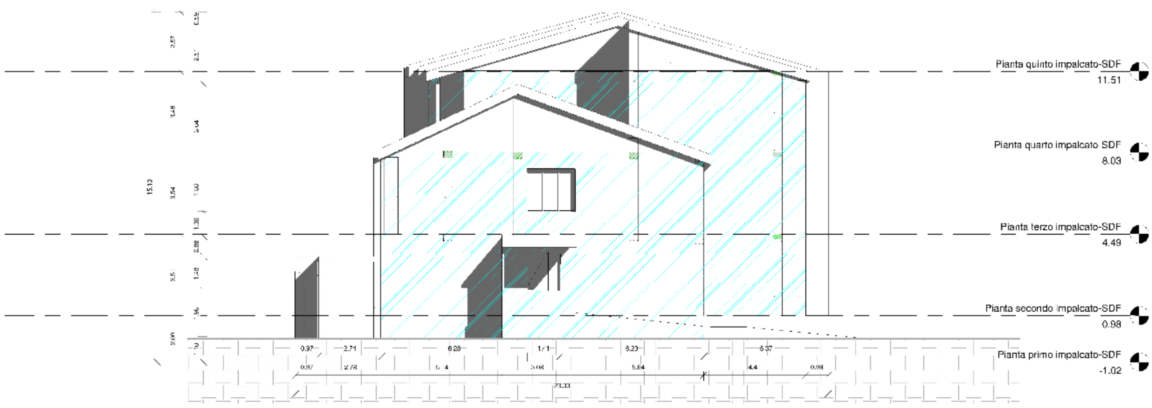


Figure 180– western view

28 MAIN REFERENCE REGULATIONS

The type of intervention falls under the provisions of DPR No. 380/2001 Article 3, as letter b), "extraordinary maintenance interventions," which include works and modifications necessary to renew and replace even structural parts of buildings, as well as to create and integrate sanitary and technological services, provided that they do not alter the overall volume of the buildings and do not entail urbanistically relevant changes in land use involving an increase in urban load. Extraordinary maintenance interventions also include those consisting of the fractionation or aggregation of real estate units with works that may involve changes in the surfaces of individual real estate units and urban load, provided that the overall volume of the buildings is not modified and the original intended use is maintained. Also included in extraordinary maintenance interventions are modifications to the facades of buildings legitimately made necessary to maintain or acquire the habitability of the building or access to it, provided that the intervention complies with current urban planning and building regulations and does not concern properties subject to protection under the Cultural Heritage Code and Landscape, Legislative Decree 22 January 2004, no. 42 (letter amended by art. 10, paragraph 1, letter b), of Law no. 120 of 2020).

For the execution of the works, the main reference legislation is generally as follows:

Public procurement regulations:

- Legislative Decree 50/2016 and subsequent amendments, as well as its corrective Legislative Decree 56/2017 and subsequent amendments;
- DPR 207/2010 and subsequent amendments;
- Ministerial Decree 145/2000 - General specification for public works contracts;
- DPR 34/2000 - Regulation of the qualification system for public works contractors;
- Compliance with the Minimum Environmental Criteria as per Ministerial Decree 24/12/2015.

Building regulations:

- DPR 06.06.2001, no. 380 and subsequent amendments, "Unified text of legislative and regulatory provisions on building matters";
- Current building regulations of the Municipality of Torre de' Passeri;
- Local hygiene regulations of the Municipality of Torre de' Passeri;
- Technical implementation regulations of the UCP.

Technical regulations on construction:

- Ministerial Decree 17/01/2018 - New technical regulations for constructions;
- Circular of the Ministry of Infrastructure and Transport 21.01.2019, no. 7 "Instructions for the application of the 'New technical regulations for constructions' pursuant to Ministerial Decree 17.01.2018";
- DM LL.PP. 11.03.1988 "Technical rules concerning investigations on soils and rocks, stability of natural slopes and embankments, general criteria and prescriptions for the design, execution, and testing of earth retaining structures and foundation works";
- Prime Minister's Ordinance 3274 of 20.03.2003, "Initial elements regarding general criteria for the seismic classification of the national territory and technical regulations for constructions in seismic areas" and subsequent amendments;

- DPCM 9 February 2011 "Assessment and reduction of seismic risk to the cultural heritage with reference to the technical regulations for constructions pursuant to Ministerial Decree 17 January 2018".

Environmental protection regulations:

- Legislative Decree 03.04.2006, no. 152 and subsequent amendments, "Environmental regulations."

Safety regulations on construction sites and workplaces:

- Legislative Decree 81/2008 - Implementation of Article 1 of Law 3 August 2007, no. 123, concerning the protection of health and safety in the workplace.