DREAMERS

Design REsearch, implementation And Monitoring of Emerging technologies for a new generation of Resilient Steel buildings

Executive Structural Project

<u>Deliverable D2.2</u>

WP 2: Executive architectural and structural design of the building

<u>Task 2.2 – Executive structural design of the demonstration building</u>

<u>Coordinator:</u>

Vincenzo Piluso

<u>Authors:</u>

Vincenzo Piluso, Elide Nastri, Gianvittorio Rizzano

Massimo Latour, Sabatino Di Benedetto

University of Salerno (IT)

Date: 22/09/2023 (Revised)

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1. Introduction

This deliverable is aimed to provide a summary of the design choices concerning the executive structural project of C3 Building of the University Campus within the framework of RFCS DREAMERS project.
According to the Grant Agreement all the design documentation is due in Italian language, because it has to fulfil the Italian Code provisions and has to be delivered to the National local authorities to obtain the relevant authorizations. Therefore, in this report only a brief discussion is provided. However, the detailed design documentation is also listed at the end of this report and fully delivered (in Italian) on the project website (www.dreamersproject.eu).

2. Geotechnical report

2.1. Introduction

The present paragraph summarises the main geotechnical properties of the soil, useful for the calculation of foundations and support structures for the construction of Building C3, as part of the University Campus of Fisciano at the University of Salerno. Furthermore, the construction of Building C3 is part of the DREAMERS demonstrator project funded by the European Community under the RFCS 2020 call.
The geotechnical characterisation of the soils involved in the construction is based on the geotechnical survey and the corresponding Geological Report signed by Dr. Geologist Nicola Polzone.
The geotechnical survey consisted of two continuous coring geotechnical boreholes, both drilled to a depth of 30 meters from the ground level. The first borehole (S5 P2), referred to as the "pilot" borehole, was carried out to assess the stratigraphic arrangement and plan the subsequent in-hole tests conducted in the second borehole (S6 P2), including the retrieval of 3 soil samples and the execution of 3 Standard Penetration Tests (S.P.T.).
Additionally, for the geomechanical characterization of the subsurface, a Dynamic Penetration Test (D.P.S.H.) was performed, which was pushed to a refusal depth of 6.20 meters from the ground level. Finally, for seismic characterisation, a Multi-Channel Analysis of Surface Waves (MASW) seismic test (MASW No. 2) was conducted.
In particular, the conducted MASW test allowed for the classification of the foundation subsoil category based on shear wave propagation velocity. As reported in the Geological Report, the site's stratigraphy falls under category "B".
The reconstruction of the topographic profile revealed that the morphological layout is characterised by gentle slopes, with average values less than 7.5%, significantly lower than the 15° limit used in the NTC2018 for assessing seismic amplification effects related to morphology. Therefore, the examined area falls into topographic category T1, for which a topographic amplification coefficient (S_T) of 1.0 should be considered.
Based on reference data and field observations of geomorphological and hydrographic conditions, the Geological Report rules out the presence of a shallow water level within the first 30 meters from the ground level, which is an essential characteristic for excluding liquefaction verification of foundation soils.

2.2. Stratigraphic column

As also reported in the extended version of the geotechnical report, the conducted boreholes have revealed the following stratigraphy:

• Layer 1 (0.00-1.30 m)
Mixed organic soil with fill composed of loose sand – layer thickness: 1.30 m
• Layer 2 (1.30-5.20 m)
Moderately compacted sand with silt and scattered gravel – layer thickness: 3.90 m
• Layer 3 (5.20-30.00 m)
Loose coarse sand with polyhedral gravel alternating with lenses of pebbles and predominantly carbonate gravel – layer thickness: 24.8 m

Based on the results of in-situ and laboratory tests, the Geological Report suggests adopting the project stratigraphy, shown in Table 1, along with the corresponding values of the mechanical properties of the soils.

Table 1. Stratigraphy recommended in the geological report

Layer	Depth	Lithological Description	Unit Weight (kN/m^3)	Friction Angle (°)	Drained Cohesion (kPa)	Undrained Cohesion (kPa)	Edometric Modulus (MPa)
1	0,00 - 5.20 m	Silty sand	15.65	29	12.0	150	14.73
2	5.20 - 30.00 m	Sandy gravel with pebbles	14.50	35	0.0	-	-

The results suggested in Table 1 are consistent with the interpretation of the in-situ S.P.T. (Standard Penetration Test) results, which indicate for the first layer an estimated internal friction angle cautiously ranging from 27.4 (5th percentile) to 31.5 (16th percentile). For the second layer, it indicates an internal friction angle value ranging from 31.3 (5th percentile) to 35.5 (16th percentile) at a depth of 11.5 meters and ranging from 31.5 (5th percentile) to 35.5 (16th percentile) at a depth of 17.5 meters.

2.3. Calculation of the ultimate foundation load

The foundation structure consists of a grid of foundation beams. The depth of the foundation's base level assumes a minimum value of approximately 4.30 meters. Therefore, the foundation's base level is located near the end of the first layer of the stratigraphic column. However, since the width of the strip foundation is 1.10 meters, the stress bulb certainly extends into the second layer. The evaluation of the ultimate foundation load was carried out conservatively, assuming the geotechnical properties of the first layer, namely a unit weight of the soil of 15.65 kN/m^3 and an internal friction angle of 29°.
The calculation of the ultimate load is determined by the formula:

$$Q_{lim} = A_q \cdot N_q \cdot \gamma_1 \cdot D + A_c \cdot N_c \cdot c + A_\gamma \cdot N_\gamma \cdot \gamma_2 \cdot \frac{B}{2}$$

The formula used has a trinomial form in which each term is related to the angle of friction, cohesion, and the specific weight of the soil. A_q - A_c - A_g are correction coefficients that represent the product of the depth factor, shape factor, inclination factor, and eccentricity of the loads (Table 2). Various authors propose different formulas for these factors as well as for the coefficients N_q, N_c, and N_g. In particular, Meyerhof does not consider the inclination of the foundation and ground plane. For the sake of simplicity in statics, cohesion is neglected.

Table 2. Input data

γ_1=	1595	Kg/m^3	Specific weight of the soil above the foundation
ϕ =	29	°	Internal friction angle of the soil at the foundation level
γ_2=	1595	Kg/m^3	Specific weight of the soil below the foundation
c=	0	Kg/cm^2	Cohesion of the soil
D=	4,3	m	Height or Depth of the foundation
B=	1,1	m	Width of the foundation (shorter side)
L=	17.2	m	Length of the foundation (longer side)
δ =	0	°	Angle of inclination of the load relative to the vertical
ε =	0	°	Angle of inclination of the foundation base plane
η =	0	°	Angle of inclination of the ground plane
E_B =	0	m	Eccentricity of the load along the width B of the foundation
E_L =	0	m	Eccentricity of the load along the length L of the foundation

Foundation design dimensions: B = 1,10 m; L = 17,20 m.
The Hansen's theory has been applied.
Using the following formula:

$$Q_{lim} = A_q \cdot N_q \cdot \gamma_1 \cdot D + A_c \cdot N_c \cdot c + A_\gamma \cdot N_\gamma \cdot \gamma_2 \cdot \frac{B}{2}$$

with:

$$A_q = s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q$$

$$A_c = s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c$$

$$A_\gamma = s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot g_\gamma \cdot b_\gamma$$

Considering ϕ=0:

$$A_q = 1$$

$$A_c = 1 + s_c + d_c - i_c - g_c - b_c$$

where:
- s is the shape factor;
- d is the depth factor;
- i is the load inclination factor;
- g is the ground plane inclination factor;
- b is the foundation base plane inclination factor.

The values of the coefficients according to HANSEN's theory are (Table 3):
- coefficient of passive earth pressure $K_p = 2.882$;
- coefficients $N_q = 16.443$, $N_c = 27.860$, $N_\gamma = 12.84$.

Table 3. Corrective factors according to HANSEN's theory

	q	c	γ
Shape	1,035	1,038	0,974
Depth	1,020	1,027	1,000
Load inclination	1,000	1,000	1,000
Ground plane inclination	1,000	1,000	1,000
Foundation base plane inclination	1,000	1,000	1,000

The limit load is evaluated according to Hansen's theory.

CORRECTIVE COEFFICIENTS: A_q=1,056; A_c=1,066; A_γ=0,974

LIMIT LOAD: Q_{lim}= 13,01 Kg/cm^2 = 1276,28 kN/m^2

2.4. Estimation of the vertical reaction module of the soil

The estimation of the vertical reaction module of the soil (subgrade constant) is carried out using the method suggested by Bowles, which is the ratio between the ultimate bearing capacity of the foundation and the corresponding conventional settlement. Therefore, keeping in mind that this conventional settlement is equal to 1 inch, the result is:

$$k_s = \frac{q_{ult}}{\Delta H} = C\left(c \cdot N_c + \overline{q} \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma\right)$$

where C=40 m^{-1} corresponds to a settlement ΔH equal to 0.025 m (1 inch), c is the cohesion, $\overline{q}$ is the effective stress at the depth of the foundation, γ is the unit weight of the soil at the depth in question, B is the width in contact with the ground, and N_c, N_q, and N_γ are the bearing capacity factors calculated according to Hansen.

Therefore, since the ultimate load is equal to 13.01 kg/cm^2, the vertical reaction modulus of the soil can be estimated as:

$$k_s = \frac{q_{ult}}{\Delta H} = \frac{13.01}{2.5} = 5.20\ kg/cm^3$$

that corresponds to:

$$k_s = \frac{q_{ult}}{\Delta H} = \frac{1276.28}{0.025} = 51051\ kN/m^3$$

3. Structural report

3.1. Introduction

The need to create resilient societies requires the adoption of technologies capable of avoiding the impact of adverse events on people, such as those that occur in the event of intense earthquakes. The FREE from DAMage technology developed during the FREEDAM research project fits exactly this goal and, within the framework of the current DREAMERS project, will be implemented in a demonstration building providing a full-scale example in a relevant operational context.

The reasons that led to the design of this structure are based on UNISA's intention to further expand the services offered to the academic community through new offices, meeting and conference rooms, and by providing the Campus of Fisciano with a medical laboratory. The investment in this solution has provided the opportunity to make the construction the prototype for the application of the innovative FREEDAM steel beam-column connections (Figure 1), studied at the same University as part of the homonymous European research project.

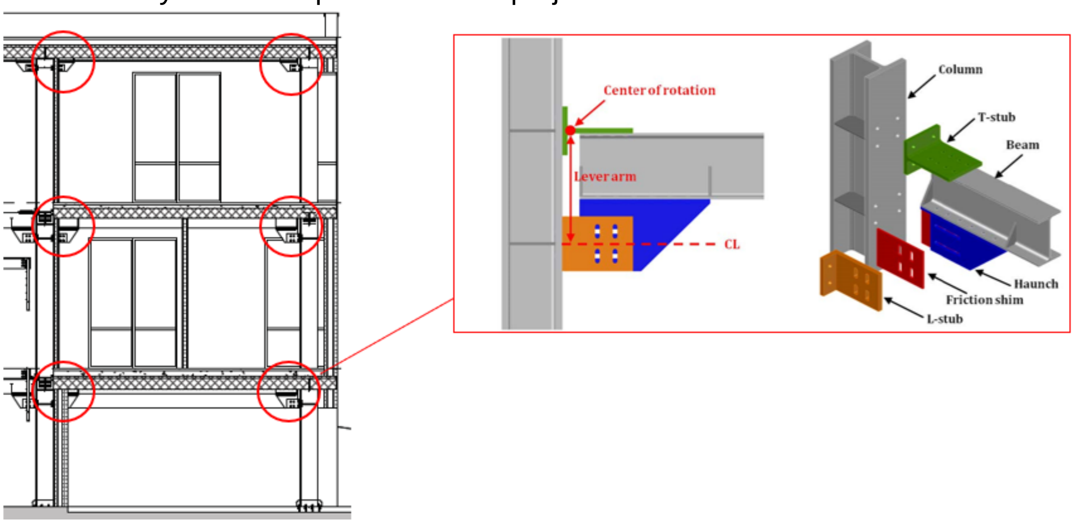

Figure 1 – FREEDAM solution

The building to be erected is developed on a ground floor, used in part as a covered outdoor area for parking spaces and as research premises, a first floor intended for analysis and research laboratories, and a second floor used as offices and the coverage. In detail, the first floor houses a room for sample preparation, one for data analysis, an analysis laboratory, a weighing room and toilets. The second floor, on the other hand, is mainly used as offices, meeting rooms and toilets.
The ground floor has a rectangular shape with dimensions of approximately 14.80m x 25.40m, equal to an area of 376 m^2. The building has a total height above ground of 12 m.
The structural design has been carried out complying with the Italian code NTC2018 (chapter 3, paragraphs 4.2, 4.3, 6.4, 7.1, 7.2, 7.3, 7.5, 7.6) and Eurocodes 1, 3 (parts 1.1 and 1.8), 4 (part 1.1) and 8 (part 1.1).
The vertical bearing structure of the building is characterised by:
- fifteen HE400B S355JR steel columns located at the intersections of the beams reported in the plan view of Figure 3;
- seismic-resistant frames (highlighted in red in Figure 2) equipped with FREEDAM joints belonging to IPE450 or IPE400 S355JR steel grade beams;
- pinned frames (highlighted in black in Figure 2) designed to support most of the gravitational loads and which do not contribute to bearing the horizontal actions, characterised by HE300B and HE240B beams with cut flanges, belonging to the Composite Slim Floor Beam (CoSFB) system conceived by Arcelor Mittal.

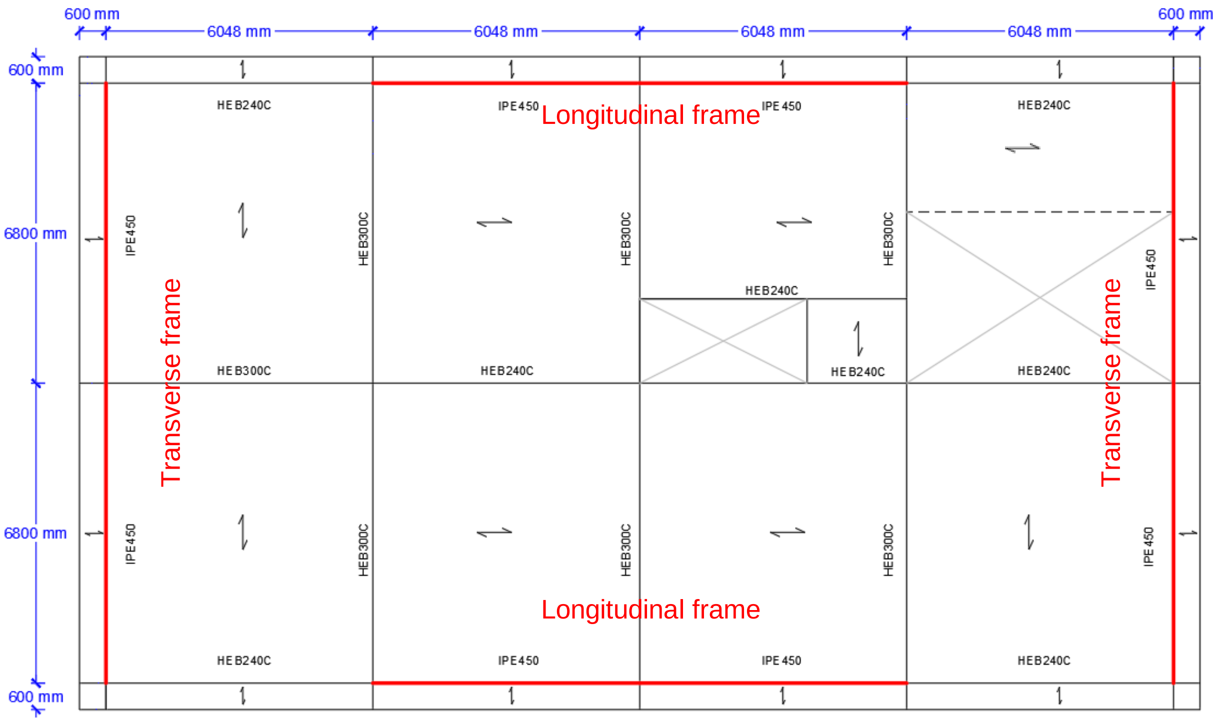

Figure 2 – Plan view of floor 1 for the identification of Moment Resisting Frames (MRFs)

3.2. Composite floor

The horizontal structure of the building is made up of Cofradal260 prefabricated steel-concrete composite floors (Figure 3), a solution proposed by Arcelor Mittal. The choice of this composite system has been dictated by its easy and rapid realisation, the excellent performance of acoustic and thermal insulation, and excellent fire resistance.

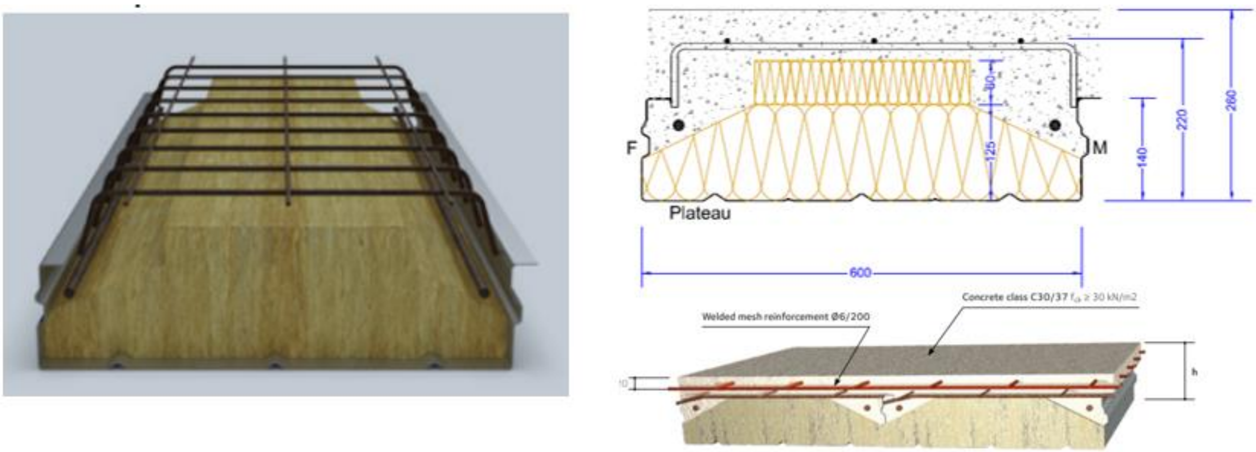

Figure 3 – Cofradal260 composite floor solution

The role of the steel decking is twofold. Initially, in the construction phase, it allows casting concrete directly on site (with a limited number of supports) and works as a formwork. Subsequently, after that concrete is completely cured, concrete and steel realise a monolithic cross-section, in which the connection between profiled steel sheeting and concrete is assured mainly by adhesion or friction. In this second phase, the steel sheeting is a tension reinforcement for the sagging bending moment. The only additional steel needed in practice is typically provided to take care of shrinkage, limit cracking for temperature effects, and, considering the continuity of the slabs, it has to be provided to resist hogging bending moments.

In both previous stages, for the analysis of a slab characterised by one-meter width and a length equal to 6.80 m, corresponding to the maximum bay span, the following checks have been fulfilled: i) Ultimate Limit State (ULS) check for bending (hogging or sagging); ii) ULS check for longitudinal shear; iii) ULS check for transverse shear; iv) ULS punching check; v) Serviceability Limit State (SLS) deflection check; vi) SLS stress limitation check.

Additional analyses have been devoted to the assessment of the frequency vibration of the floor. In fact, the Italian Code requires that, considering the load combination $G_k + 0.15Q_k$, the frequency of the deck is greater than 3 Hz for non-cyclic loads and 5 Hz in the presence of cyclic loads. However, it does not provide formulations for evaluating this frequency. Therefore, reference has been made to documents of proven validity developed in the context of research projects. In particular, reference was made to the research project "Human induced Vibrations of Steel Structures" (RFS2-CT-2007-00033), whose design and evaluation methods for floor vibrations are related to human-induced vibrations, mainly caused by walking in normal conditions. The analysis has highlighted that the frequency of the composite floor is about 6.90 Hz, the modal mass of two structure bays is about 11.5 tons, and the damping is 4%. As a result, the analysed floor falls into class D, which, concerning the intended use for offices, appears to be a performance requirement recommended by the research referred to.

3.3. Secondary beams

The Cofradal 260 slabs transfer the loads to secondary beams, designed according to a steel-concrete composite solution. These elements represent a solution proposed and patented by Arcelor Mittal and are marketed as CoSFB beams (Composite Slim Floor Beams). The peculiarity of the CoSFB beams is that they consist of composite steel-concrete beams with the steel profile embedded in the thickness of the floor; moreover, the double T steel section has the particularity of having the upper flange with a smaller width than the lower flange (for this reason the term cut-off is used; this detail is shown in Figure 4).

These beams are characterised by cut HE240B and HE300B profiles designed to belong to non-seismic-resistant frames and, for this reason, are schematised as beams simply supported at their ends. This behaviour is recreated through shear connections.
Again, the checks have been carried out controlling that the maximum bending moments and shear actions were lower than the capacity of the CoSFB beams and that the maximum deflections and the deflections induced by variable loads at SLS were lower than L/250 and L/300, respectively (where L represents the lengths of the beams).

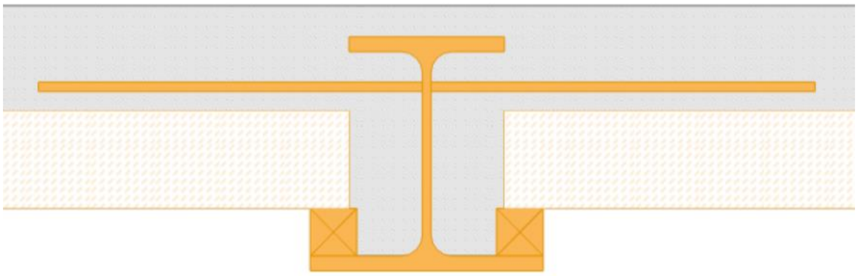

Figure 4 – CoSFB beam

It is important to point out that, in the seismic-resistant longitudinal (Figure 5) and transverse (Figure 6) bays, the deck slab is located on the top flange of the beam while in the gravity load resisting bays the steel profile is embedded in the thickness of the deck slab. This solution is aimed to realize a structural detail for the beam-to-column joints equipped with friction dampers as close as possible to the detail already subjected to experimental tests during the previous FREEDAM project. The goal is to prevent any collaboration of the concrete slab to the joint behaviour, assuring that the connection behaves like a bare steel connection.

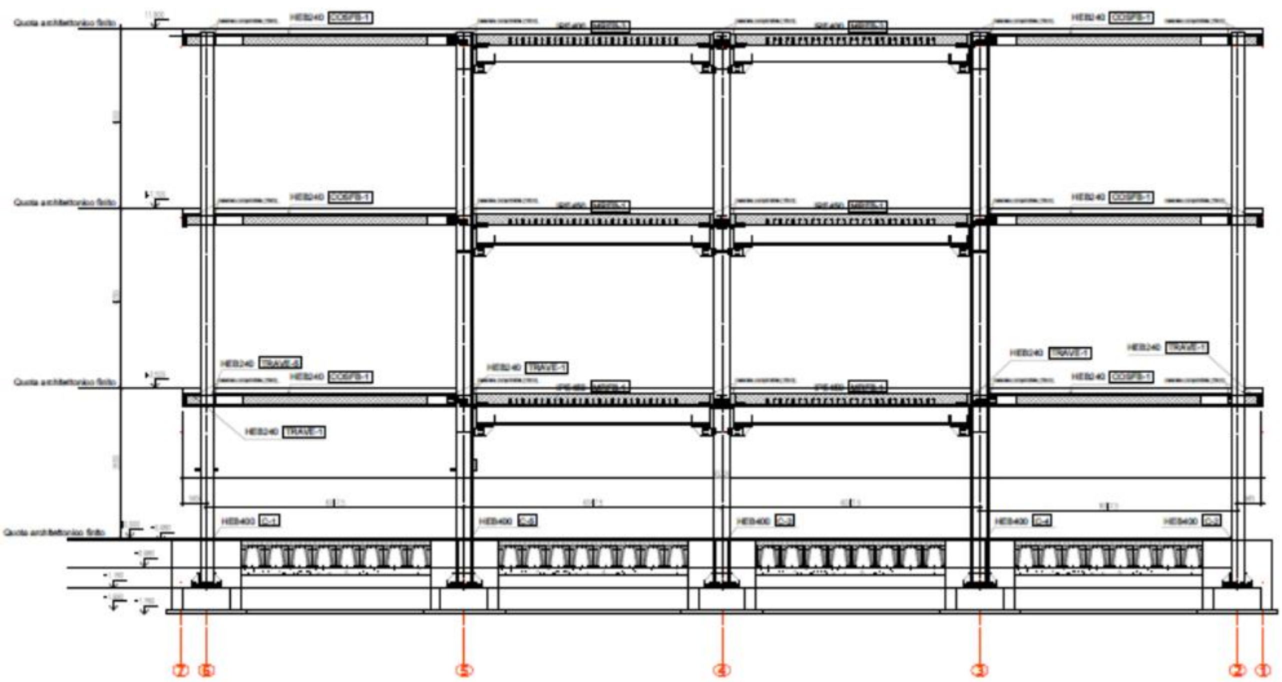

Figure 5 – Seismic load resisting system – Longitudinal direction

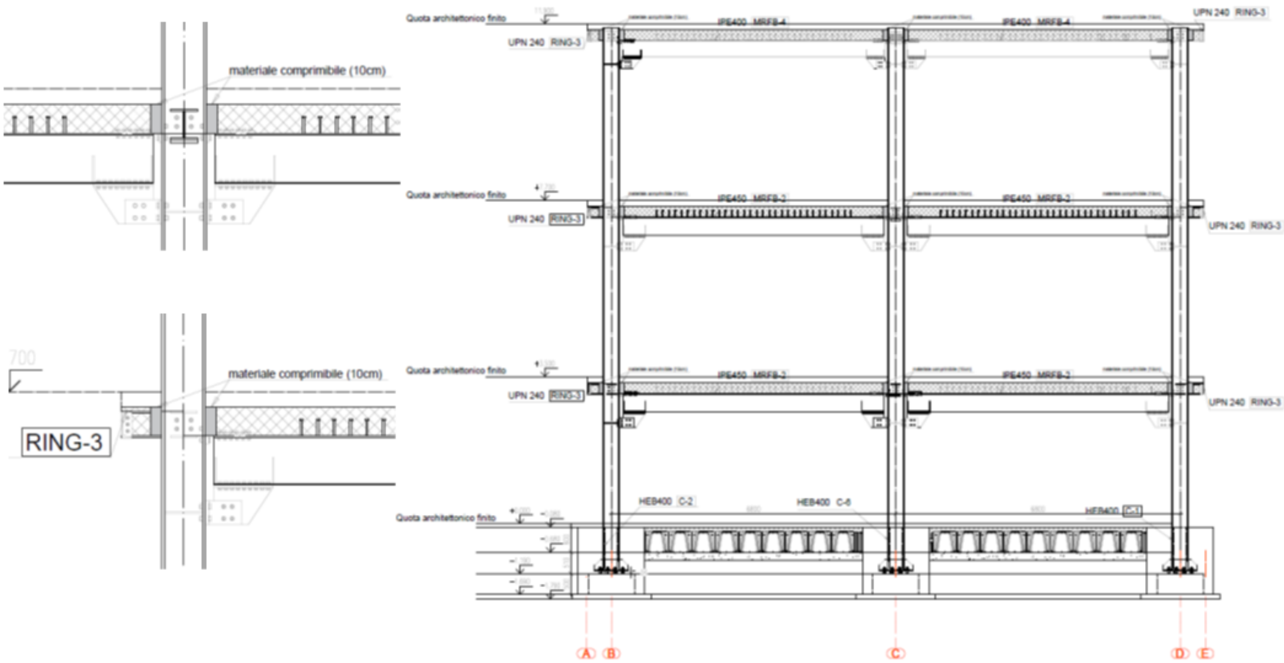

Figure 6 – Seismic load resisting system – Transversal direction

3.4. Seismic-resistant frames

The design of the MRFs has been carried out according to Italian Code NTC2018, Eurocode 8 provision and the Theory of Plastic Mechanism Control (TPMC) considering a seismic action defined referring to the construction site located in Fisciano, characterised by type-B soil and topography class T1.

In particular, TPCM is based on the kinematic theorem of plastic collapse and the concept of the equilibrium curve of the mechanism. The equilibrium curve of any possible collapse mechanism is obtained through a second-order rigid-plastic analysis in which the external work is calculated including the work of the second-order effects induced by the gravitational loads applied to the structure. The kinematic theorem of plastic collapse extended to the concept of the equilibrium curve of the mechanism ensures that, in a range of displacements compatible with the rotational capacity of the structural elements, the collapse mechanism developed is the one whose equilibrium curve is placed under those of all other possible mechanisms. Thus, it is possible to design the column sections at each level by making a design requirement that the mechanism equilibrium curve corresponding to the desired global mechanism is below the equilibrium curves of all unwanted mechanisms. The second-order effects are explicitly and rigorously considered through the equilibrium curve of the collapse mechanism.

In the case of seismic-resistant frames equipped with FREEDAM connections, the TPMC can be easily applied, provided that the internal work of the dissipative zones is suitably evaluated. For this purpose, the plastic moment of the beams has been replaced by the sliding resistance moment of the FREEDAM connections. The behaviour of beam-column connections equipped with friction dampers has been evaluated in the design process via a perfectly plastic rigid bonding of the dissipative zones. Furthermore, according to the second principle of capacity design, the overstrength associated with the variability of the coefficient of friction has also been considered.

In the first phase of the work, the columns have been preliminarily sized to support only the vertical loads according to the fundamental gravitational combination (ULS). With this in mind, IPE360 beams and HEB300 columns have been used only in the pre-sizing phase for vertical loads. Even though specific limits for the cross-section classes are not strictly required, to ensure a minimum

level of local ductility to the structural members, the profiles of beams and columns have been selected to be at least in class 2. However, according to the seismic design procedures currently implemented, it was necessary to modify the profiles. The final solution consisted of adopting HEB400 profiles for the columns, IPE450 beams for the first two levels, and IPE400 beams for the top floor. Resistance and stability checks of the columns and beams have been satisfied.
According to the first principle of capacity design, the sliding moments of FREEDAM connections have been defined considering the design actions of the relevant seismic combination (ULS). The available rotational capacity is demonstrated through experimental testing according to EC8 provisions and AISC 358-16 prequalification protocols.
Since FREEDAM connections are partial-strength beam-to-column joints, the beam-column hierarchy criterion has been properly modified as follows:

$$\sum M_{Rc} \geq 1.2 \sum M_{Rd,connections}$$

$\sum M_{Rc}$ and $\sum M_{Rd,connections}$ are the sum of the design values of the columns' resistances and the sum of the sliding moments of the connections framing the joint. In particular, $M_{Rd,connections}$ is assessed considering the maximum value of the static friction coefficient.
Table 4 summarises the main geometric and mechanical properties of the FREEDAM connections adopted in the pilot building.
The general performance requirements for the DREAMERS building are summarised in *Table 5*.

Table 4. Properties of FREEDAM connections

	First and second level (transverse frames)	First and second level (longitudinal frames)	Third level
MARK	FREEDAM – IPE 450/0.4	FREEDAM – IPE 450/0.3	FREEDAM – IPE 400/0.3
Name	D1	D1	D1
$F_{slip,Rd}$ [kN]	345.3	292.4	244.2
$M_{j,Rd}$ [kNm]	242	181	139
Bolts	M16 HV 10.9	M16 HV 10.9	M16 HV 10.9
Number of bolts, n_b	4	4	4
Number of surfaces, n_s	2	2	2
Preload force, $F_{p,d}$ [kN]	93.64	79.30	66.23

Table 5. Seismic requirements for the DREAMERS building

	Limit states			
Category	**IO (Immediate Occupancy)**	**DL (Damage Limitation) =SLS**	**LS (Life Safety) =ULS**	**CP (Collapse Prevention)**
Structural members and non-dissipative joints	Fully Elastic at IO. For elastic design: $q_{IO} = 1$ Resistance and stability checks for columns and beams. Resistance checks for non-dissipative joints. $drift \leq \frac{2}{3} 0{,}01$	Fully elastic at DL. For elastic design: $1 < q_{DL} \leq 1{,}5$ Resistance and stability checks for columns and beams. Resistance checks for non-dissipative joints. $drift \leq 0{,}01$	Fully elastic at LS. For elastic design: $1{,}5 < q_{LS} \leq min\left(5\frac{\alpha_u}{\alpha_1}; \frac{S_{e,LS}}{S_{e,DL}} q_{DL}\right)$ Resistance and stability checks for columns and beams also considering capacity design principles. Resistance checks for non-dissipative joints considering capacity design rules.	Slightly damaged at CP
Dissipative beam-to-column and column base joints	$M_{Ed}(q_{IO}) \leq M_{FREEDAM}$ Fully elastic friction pads for IO. $\vartheta_{device} = 0\, mrad$ $M_{Ed}(q_{IO}) \leq M_{cb}$	$M_{Ed}(q_{DL}) \leq M_{FREEDAM}$ Slight damage in the friction pads for DL $\vartheta_{device} \leq 10\, mrad$ $M_{Ed}(q_{DL}) \leq M_{cb}$	$M_{Ed}(q_{LS}) \leq M_{FREEDAM}$ Moderate damage in the friction pads for LS. $\vartheta_{device} \leq 25\, mrad$ $M_{Ed}(q_{LS}) \leq M_{cb}$	Significant damage in the friction pads for CP. $\vartheta_{device} \leq 40\, mrad$ $\vartheta_{cb} \leq 40\, mrad$

	Fully elastic friction pads for IO. $\vartheta_{cb} = 0\,mrad$	Slight damage in the friction pads for DL $\vartheta_{cb} \leq 10\,mrad$	Moderate damage in the friction pads for LS. $\vartheta_{cb} \leq 25\,mrad$	
Residual drifts	-	-	Residual interstorey drift $< 0{,}35\%$ [15]	Residual interstorey drift$< 0{,}5\%$ [15]
Partition walls, claddings	Elastic for $drift = \frac{2}{3}0{,}01$	Out-of-plane resistance and stability checks according to NTC18 with a behaviour factor equal to $q_a = 1$ Elastic for $drift = 0{,}01$	Out-of-plane resistance and stability checks according to NTC18 with a behaviour factor equal to $q_a = 2$ Slightly damaged for drift at least equal to $drift = 0{,}025$	Damaged without any significant loss of resistance (less than 10%) for drift at least equal to $drift = 0{,}04$
False ceiling	Elastic for $drift = \frac{2}{3}0{,}01$	Resistance and stability checks according to the NTC18 with a behaviour factor equal to $q_a = 1$ Elastic for $drift = 0{,}01$	Resistance and stability checks according to the NTC18 with a behaviour factor equal to $q_a = 2$ Slightly damaged for drift at least equal to $drift = 0{,}025$	Damaged without any significant loss of resistance (less than 10%) for drift at least equal to $drift = 0{,}04$
Plants	Specific detailing rules are needed to uncouple the deformation of the structure and the electric system, the water system, etc. for a $drift = \frac{2}{3}0{,}01$	Specific detailing rules are needed to uncouple the deformation of the structure and the electric system, the water system, etc. for a $drift = 0{,}01$	Specific detailing rules are needed to uncouple the deformation of the structure and the electric system, the water system, etc. for drift at least equal to $drift = 0{,}025$	Damaged without any significant loss of resistance (less than 10%) for drift at least equal to $drift = 0{,}04$

It is worth highlighting that the detail of the FREEDAM joint has been widely studied so that it is able to exhibit bare steel behaviour. For this reason, it has been chosen to consider the misalignment of primary and secondary beams and design proper gaps around the connections in order to disconnect the slab from the connections (*Figure 7*).

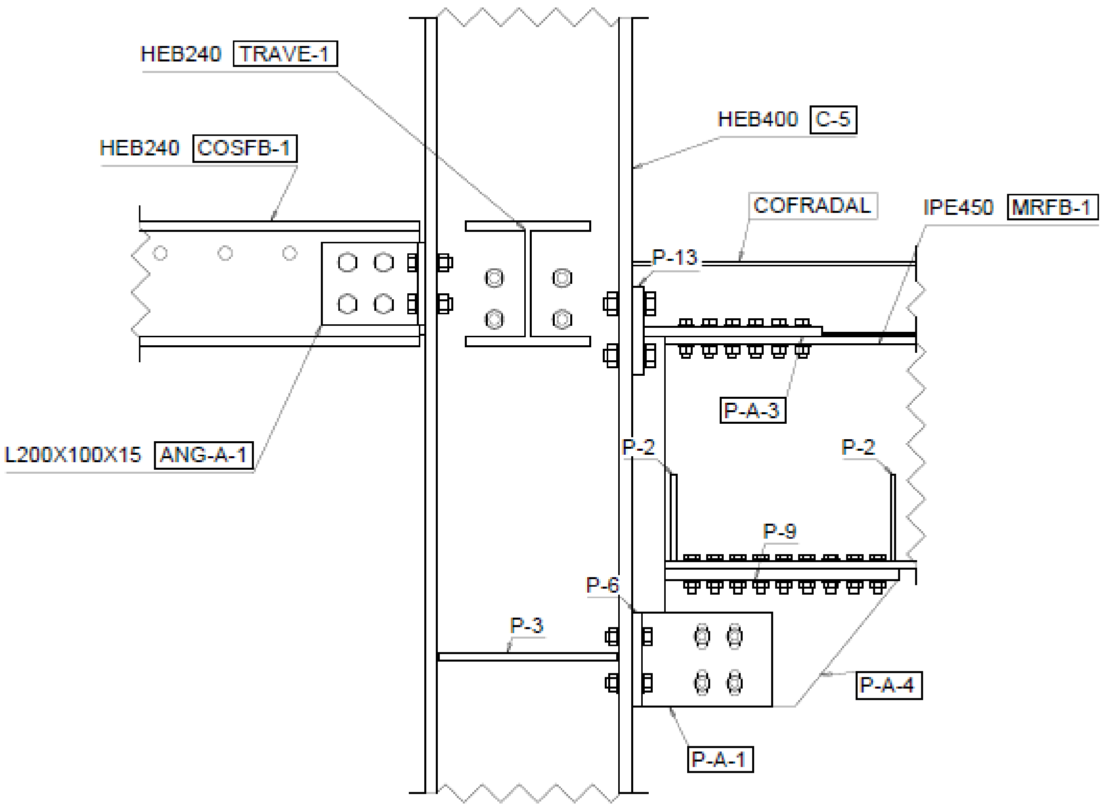

Figure 7 – Detail of a connection

Linear analysis methods have been used in the design phase, while the fulfilment of the performance objectives has been checked by performing linear and non-linear (pushover and time-history) analyses. In particular, the structure has been numerically modelled through SAP2000 (*Figure 8*), Advance Design and OpenSess software.

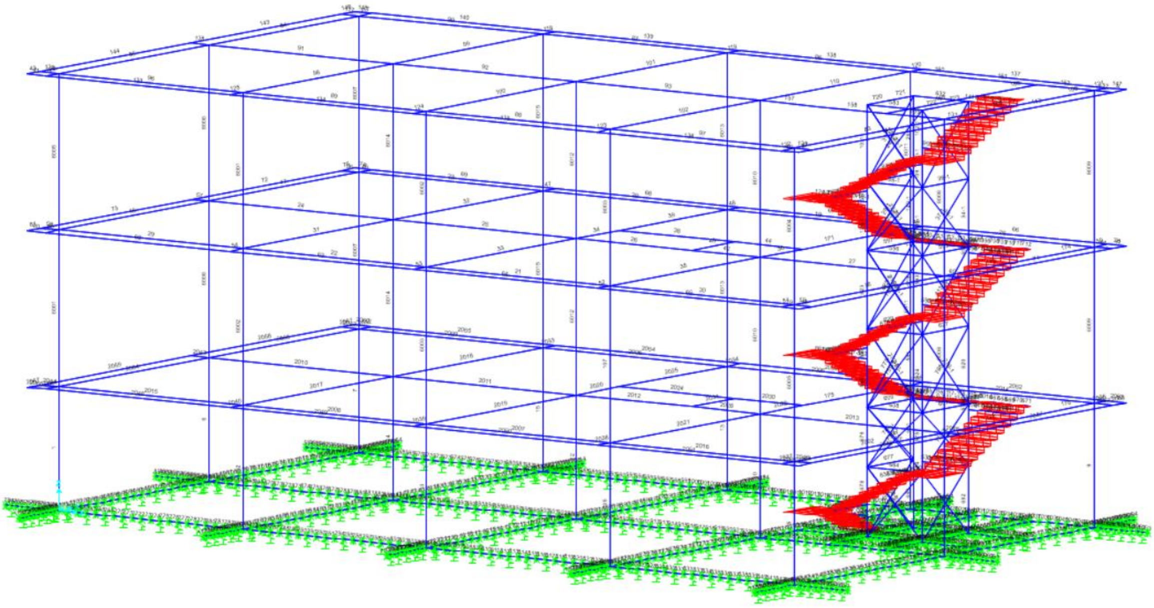

Figure 8 – SAP2000 model of the pilot building

3.4.1. Summary of calculation results and verifications

All the checks are satisfied. For the sake of clarity, in *Figure 9* an image of the working rate of the structural elements is shown, while in *Table 6* the maximum working rates are reported.

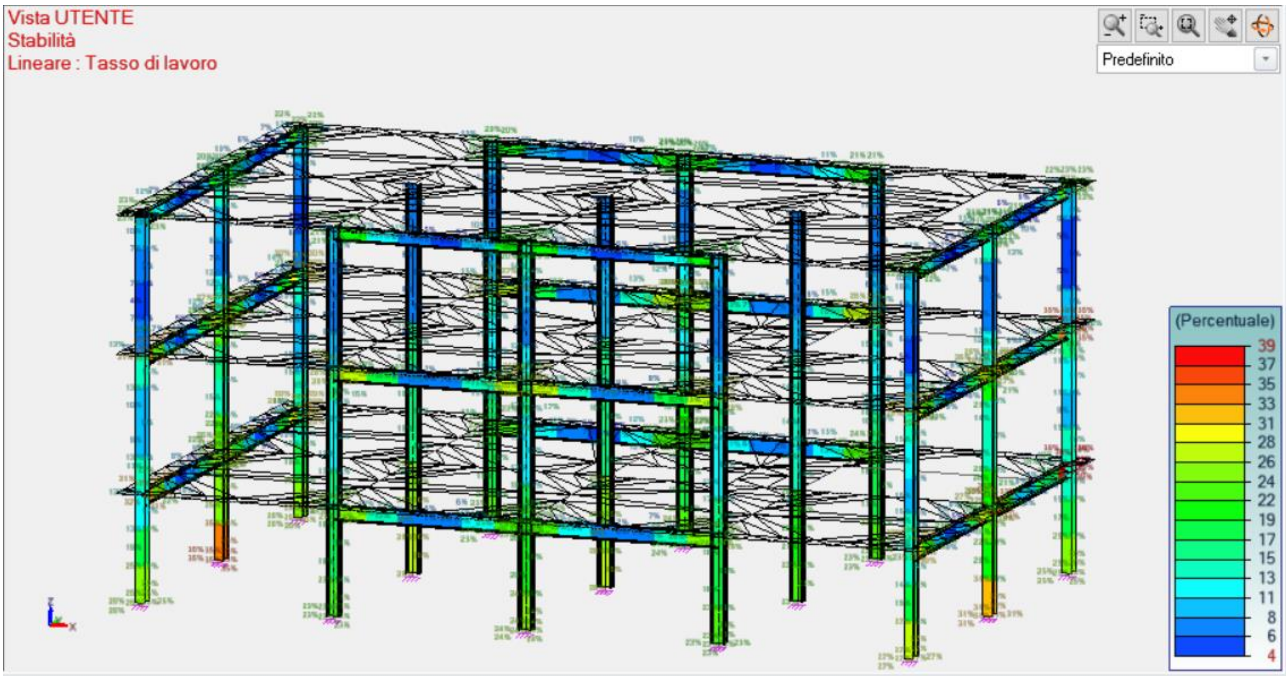

Figure 9 – Working rate of structural elements

Table 6. Maximum working rate - stability: per element

Name	***Section***	***Working rate (%)***
Beam	IPE450	39 21
Column	HEB400	35 28

Figure 10 shows the pushover analysis results, considering a vertical pattern distribution according to the first vibration mode of the structure. The outcomes are perfectly in line with the design requirements and expectations since it is evident that the building exhibits an elastic behaviour up to DLS, inducing the sliding of the FREEDAM connections at Ultimate and Collapse Limit States.

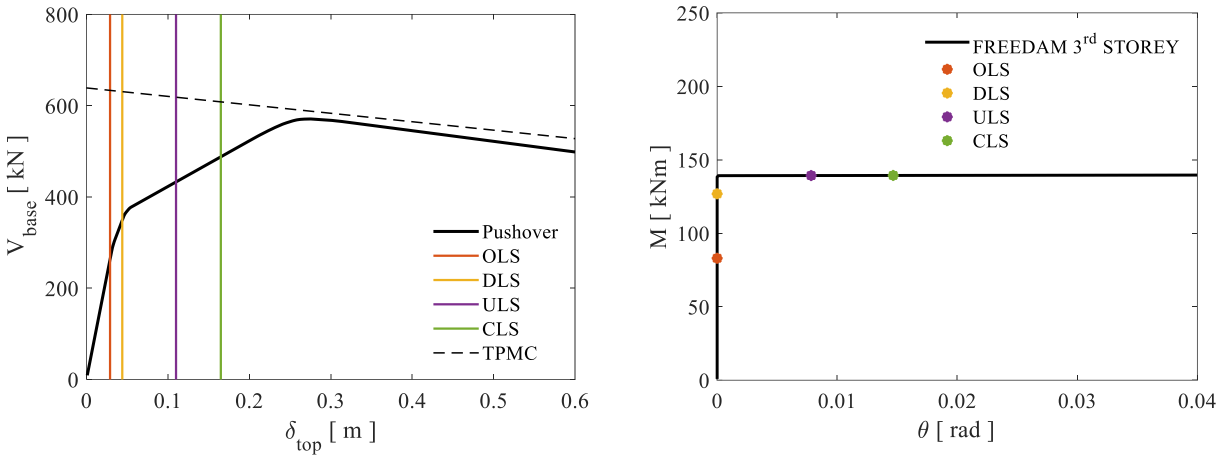

Figure 10 – Pushover analysis results (for one longitudinal MRF): Pushover curve (left); FREEDAM connection for the first storey (right)

Incremental Dynamic Analyses (IDAs) have been performed through OpenSees by considering a set of ground motion records scaled to increasing Intensity Measure values to cover the range from elastic to non-linear seismic response of the frame up to collapse. A set of 7 natural ground motions records has been selected from the Italian Database provided by Iervolino et al. (2010) with the following parameters: moment magnitude (M_w) ranging from 5 to 7, the epicentral distance R ≤ 30 km, site class B and spectrum-compatibility in the range of periods between $0.2T_1$ and $2T_1$.
Consistently with the pushover analysis, the results of IDAs (Figure 11 and Figure 12) have highlighted that connections activate for pseudo-accelerations higher than that referring to the DLS, and the maximum rotations experienced by the connections are compatible with the expected limits provided by the provisions. At DLS, the maximum interstorey drifts are below 1%, while at ULS, the residual interstorey drifts are lower than 0.5%, a limit conventionally associated with building reparability, as McCormick et al. (2008) suggested.
These outcomes demonstrate that the pilot building is expected to withstand severe seismic events without structural damage, except for the wearing of the friction pads.

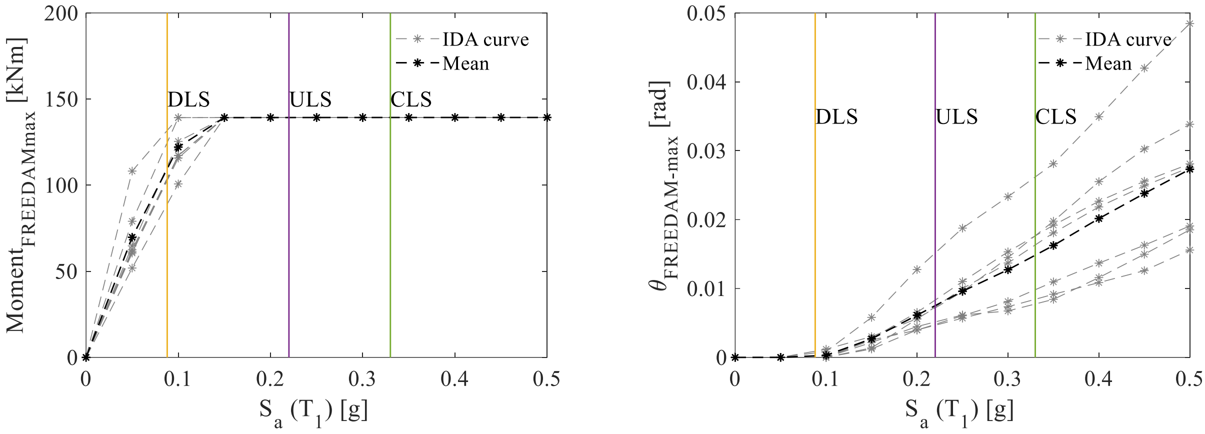

Figure 11 – IDA results in one FREEDAM connection (Third level): Maximum moment (left); Maximum rotation (right)

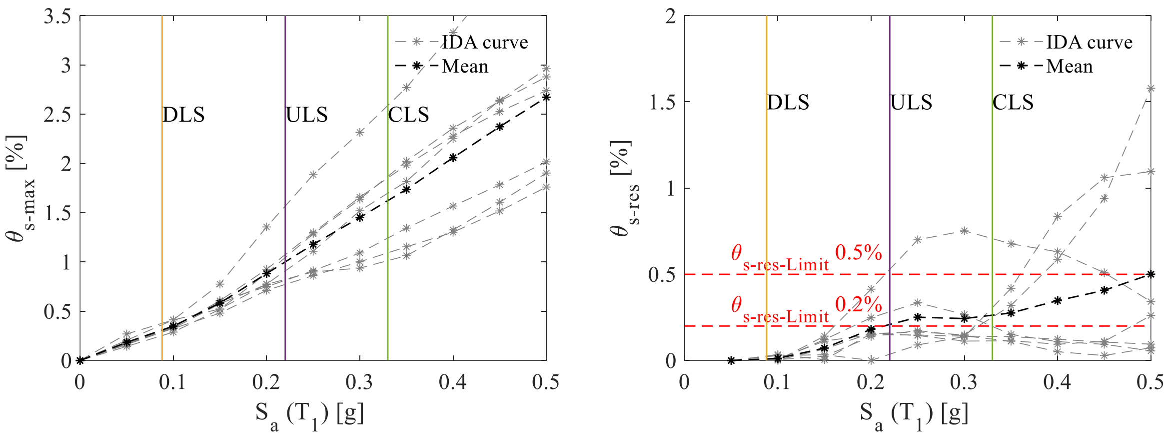

Figure 12 – IDA results (third floor): peak interstorey drifts (left); residual interstorey drifts (right)

The detailed calculations are provided in the “Extended design relation in Italian”.

3.5. Staircase-elevator body structures calculation report

3.5.1. Description of the structure

This calculation report concerns the steel structure of the stair-elevator body of the C3 building on the Fisciano University Campus.

The structure of the stair-elevator body is designed in such a way as to be structurally independent of the structure of the C3 building. In particular, it consists of a braced steel castle structure. The castle has four columns made up of a pair of IPE240 profiles arranged in a cross in a welded composition. The castle has six levels, three of which coincide with the levels of the decks of building C3. The beams are made of IPE240 profiles. The braces are made of CHS 76.1x3.2 round tubular profiles. All members are in S355 steel.

The flights of stairs and the landings are made using a reinforced concrete slab, folded according to the development of the steps, which rests on cantilever beams made of IPE240 profiles, connected to the castle (Figure 13).

The beam-to-column connections are bolted according to the flanged type. The connections of the bracing diagonals are made using a bolted system of the gusset and fork type. The foundation-column connections are made using a base plate with anchor bolts, embedded in the concrete casting for a length equal to the size of the webs of the foundation beams. Therefore, the column-foundation connection can be considered rigid.

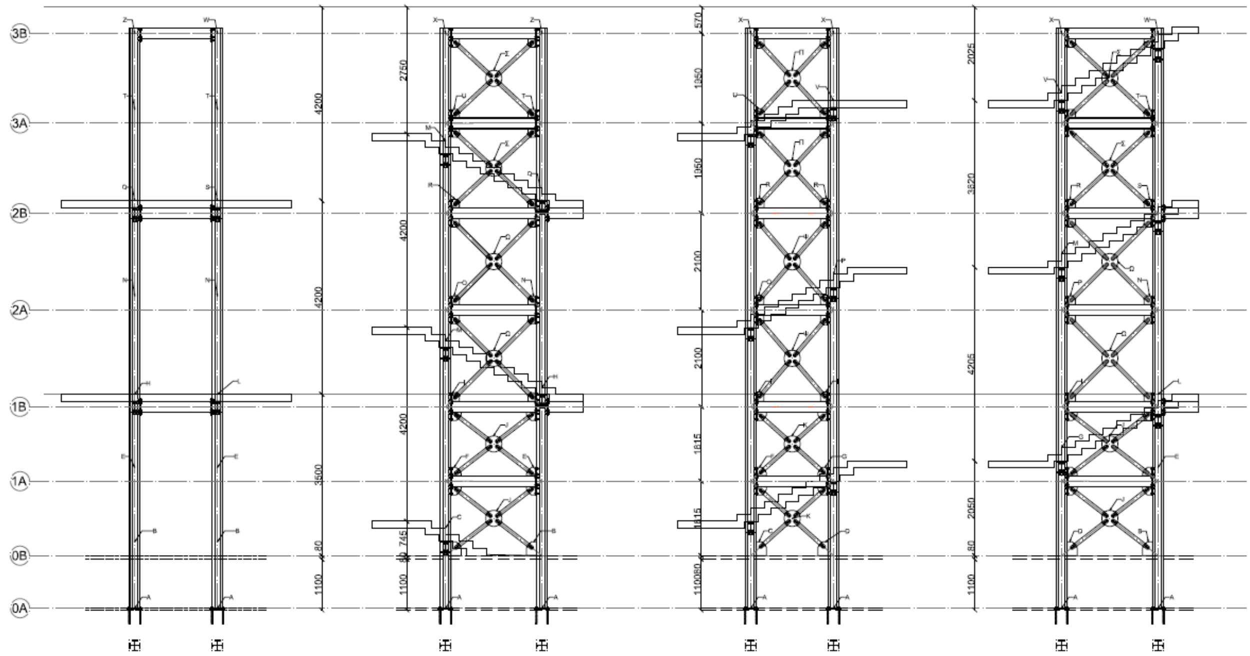

Figure 13 – Staircase-elevator body structures

Figure 14 shows the structural finite element model. The steel members are modelled using finite elements of the "beam-column" type. The slab is discretized using two-dimensional finite elements of the "plate" type.

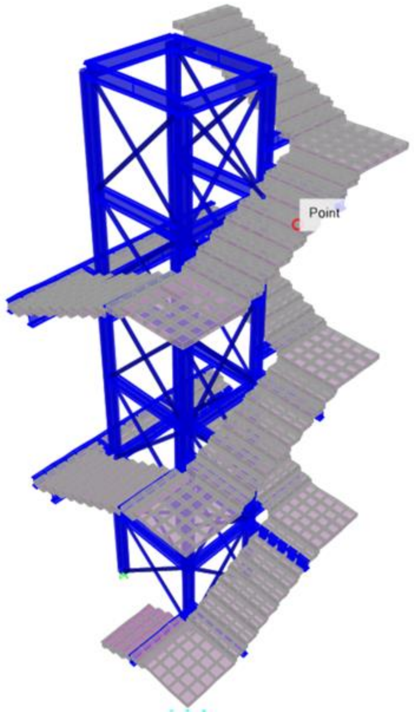

Figure 14 – Structural model of the stairs

3.5.2. Project actions

The actions considered for the structure design are:

- permanent structural loads G_{k1}, which in the case in question are constituted only by the weight of the steel members and by the weight of the concrete slab constituting the ramps and landings;
- non-structural permanent loads G_{k2}, which in the specific case are made up of the stairs' slab and the elevator car's weight.
- variable loads due to the intended use of the structure.
- seismic actions.

The structure of the stair-elevator body is not subject to wind actions, as it is entirely inside the body of the C3 building. Furthermore, the effects of thermal variations are negligible.

3.5.3. Structural permanent loads

The structural permanent loads consist of:

Table 7. Materials' list

Section	Total length (m)	Total Weight (kg)
2XIPE240	46,4	2694,51
IPE240	81	2485,87
TUBO-D76.1X3.2	100,3192	577,09

The slab of the landings and ramps weighs 25 kN/m^3.

3.5.4. Permanent non-structural loads

The non-structural permanent loads consist of the finishes of the stairs and the lift shaft for a total of 1.81 kN/m^2.

3.5.5. Variable loads

The variable loads, by the provisions of the legislation, are assumed to be equal to 4.0 kN/m^2.

3.5.6. Seismic actions

Concerning the town of Fisciano, the parameters for the determination of the design elastic spectra that define the seismic action for the various limit states envisaged by the legislation are shown in Table 8:

Table 8. Characteristics of the design elastic spectra

Stato Limite/ Limit State	T_R (years)	a_g (g)	F_0	T^*_c (s)
SLO/IO	45	0,053	2,361	0,313
SLD/DL	75	0,065	2,405	0,338
SLV/LS	712	0,148	2,527	0,431
SLC/CP	1462	0,182	2,591	0,448

The aforementioned parameters refer to Category of Use III for which C_u=1.50.
With these values of the parameters that define the seismic hazard of the site, the design elastic spectra for the reference site (rigid ground and horizontal countryside plane) are shown in Figure 15.

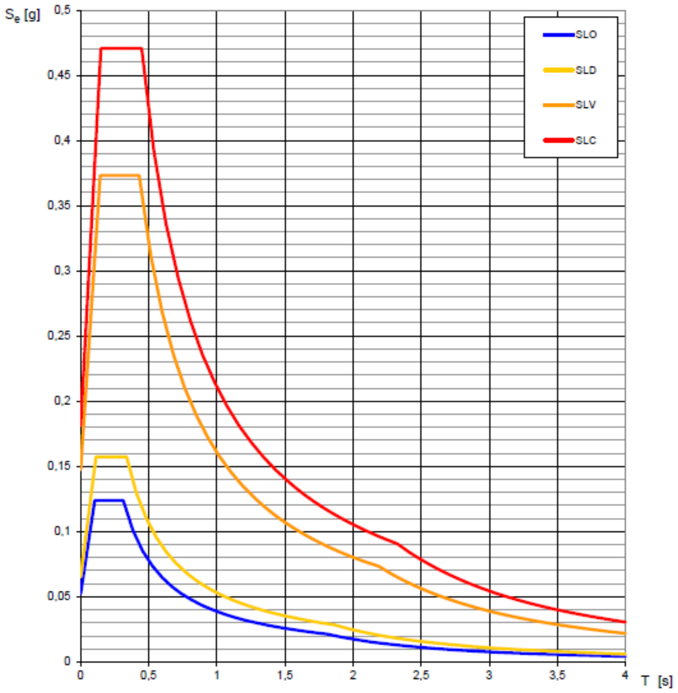

Figure 15 – Site seismicity

From the tests carried out on-site, as shown by the geological report, the stratigraphic column falls within the case of soil type B.

The seismic-resistant design of the structure was carried out to ensure that the structure remains in the elastic range for a value of the seismic action equal to the heaviest one defined as the maximum deriving from the SLO spectrum and the SLV spectrum. For the benefit of statics, the design spectrum at the limit state SLV was determined by considering the structure factor q=4. In particular, this choice of the structure factor is lower than the value set by the law for framed structures and equal to that set for structures with concentric X-bracings.

The design spectra for the SLO and SLV limit states, determined as specified, are given in Figure 16.

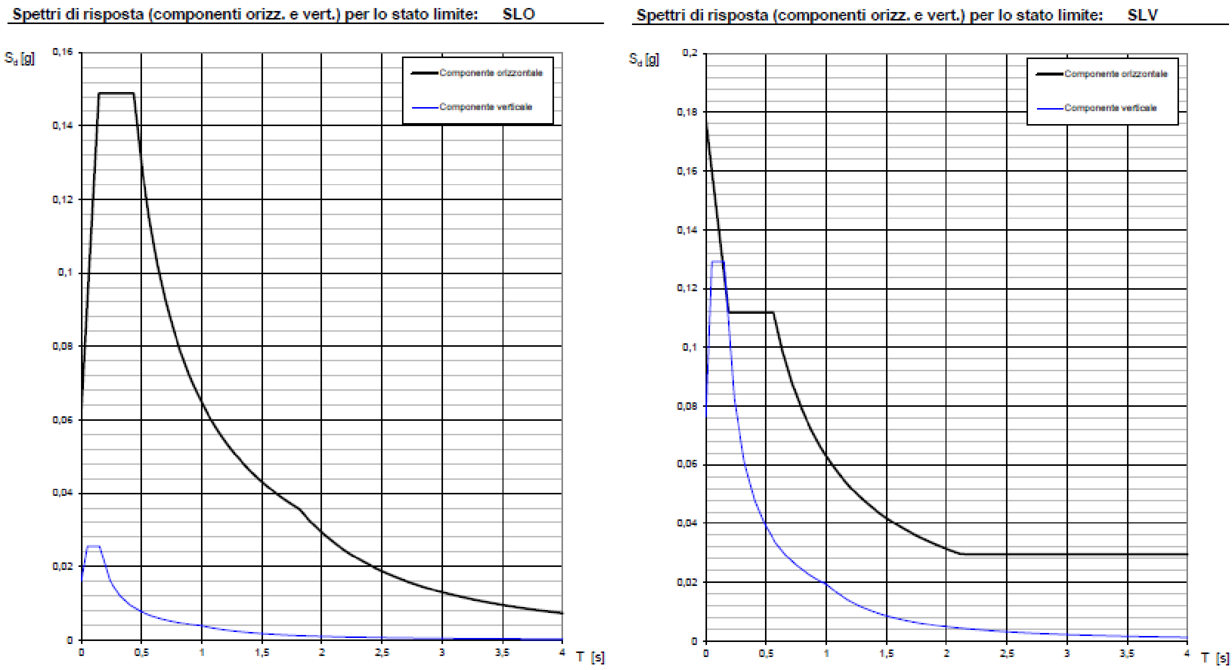

Figure 16 – Design spectra at SLO (left) and design spectra at SLV (right)

The seismic analysis of the structure was performed using the modal response spectrum analysis. The modal combination technique employed is the CQC.
The analysis was carried out considering the first 12 vibration modes which guarantee modal participation of the masses greater than 98% for both an earthquake in the x direction and for an earthquake in the y direction.
The periods of vibration of the 18 modes of vibration considered are shown in Table 9:

Table 9. Modal Participating Mass Ratios

Mode	Period (sec)	UX	UY	SumUX	SumUY
1	0,4981	0,4243	0,0065	0,4243	0,0065
2	0,3569	0,2585	0,0548	0,6828	0,0613
3	0,3088	0,0060	0,6177	0,6888	0,6790
4	0,1690	0,0965	0,0035	0,7853	0,6825
5	0,1470	0,0451	0,0001	0,8304	0,6827
6	0,1283	0,0046	0,0362	0,8350	0,7188
7	0,1248	0,0140	0,0538	0,8489	0,7726
8	0,1215	0,0091	0,0016	0,8581	0,7742
9	0,1184	0,0001	0,0015	0,8582	0,7757
10	0,1081	0,0019	0,0015	0,8601	0,7772
11	0,1077	0,0024	0,0000	0,8625	0,7772
12	0,1051	0,0003	0,0000	0,8629	0,7772
13	0,1013	0,0019	0,0346	0,8647	0,8118
14	0,0989	0,0061	0,0002	0,8708	0,8120
15	0,0984	0,0022	0,0060	0,8729	0,8180
16	0,0972	0,0005	0,0111	0,8734	0,8292
17	0,0952	0,0002	0,0050	0,8736	0,8342
18	0,0885	0,0197	0,0152	0,8933	0,8493

3.5.7. Partial safety factors and load combinations

As prescribed by the NTC 2018, the design load combinations have been determined taking into account the partial safety factors shown in Table 10.

Table 10. Partial safety factors

	Symbol	if favourable	if unfavourable
Permanent structural loads	G1	1.00	1.30
Permanent non structural loads	G2	0.80	1.50
Variable loads	Q	0	1.50
Seismic actions for SLO limit state	SISMA-SLO	0	1.00
Seismic actions for SLV limit state	SISMA-SLU	0	1.00

In particular, the variable load was placed on all ramps (load condition Q), only on the left ramp (load condition QS), only on the right ramp (load condition QD) or only on the ramp parallel to the arrival landing (load condition QB). In other words, the gravitational load conditions that maximize the flexure of the frame in the longitudinal direction or the transversal direction were also investigated.
Therefore the following load combinations have been considered:

1) 1.30 G1 + 1.50 G2
2) 1.30 G1 + 1.50 G2 + 1.50 Q
3) 1.30 G1 + 1.50 G2 + 1.50 QS
4) 1.30 G1 + 1.50 G2 + 1.50 QD
5) 1.30 G1 + 1.50 G2 + 1.50 QB
6) G1 + G2 + 0.60 Q + SISMA-SLO-U1 + 0.30 SISMA-SLO-U2
7) G1 + G2 + 0.60 Q + 0.30 SISMA-SLO-U1 + SISMA-SLO-U2
8) G1 + G2 + 0.60 Q + SISMA-SLU-U1 + 0.30 SISMA-SLU-U2
9) G1 + G2 + 0.60 Q + 0.30 SISMA-SLU-U1 + SISMA-SLU-U2

where U1 denotes the earthquake in the x direction and U2 is the earthquake in the y direction.

3.5.8. <u>Summary of calculation results and verifications</u>

From the calculation tables and the resistance and stability checks of the members, it appears that all the checks are satisfied.

The following Figure 17 shows the work rates of the members (i.e. the ratio between the design stress and the design strength). A value less than 1.0 indicates that the representative stress state point is within the strength domain of the member.

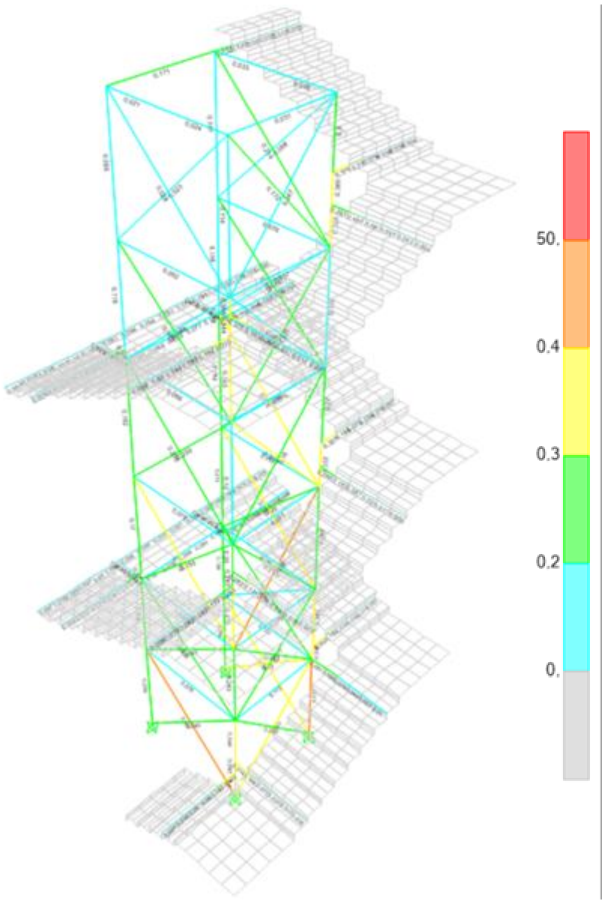

Figure 17 – Maximum member work rates

In particular, the maximum working rates of the structural members are as follows:

Columns: 42.7%

Diagonals: 40.8%

Castle beams: 23.9%

Cantilever beams: 34.2%

The checks concerning the connections are given in extended calculation documentation.

3.6. Foundations

The peculiarity of the foundation system consists in the design of embedded column base connections. In fact, such a solution represents a very effective strategy for designing rigid and full-strength column bases, representing details closer to the fixed supports commonly adopted to model the columns' restraints. From a mechanical point of view, the main characteristic of embedded column bases is that the bending moment and shear force are transmitted by the embedded steel column to the concrete of the plinth through a contact pattern of stress (Figure 18), while the base plate plays a role primarily in terms of axial strength. In this case, by considering literature research by Wald et al. (2000), Grilli and Kanvinde (2017), the AISC provision and the Japanese code about such a kind of connections, the embedment length has been fixed equal to 1.10 m.

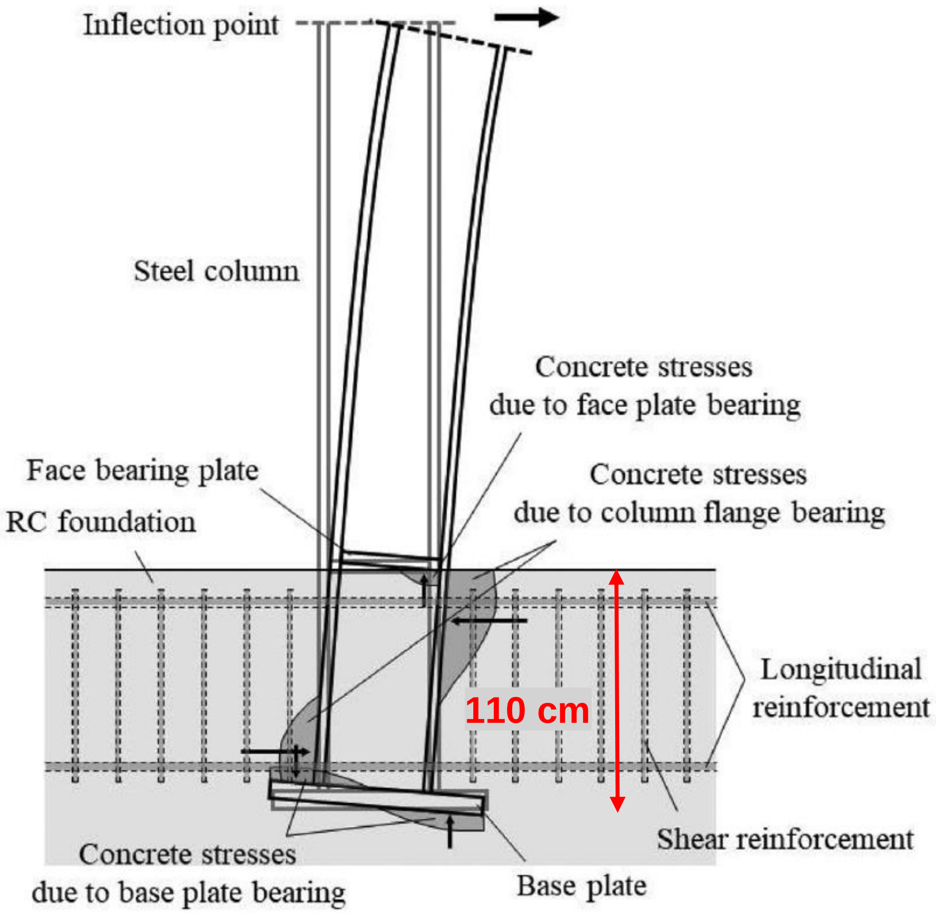

Figure 18 – Embedment length

The embedded column base connections transfer the actions deriving from the structure to the plinths, which are connected through the adoption of T-shaped beams (Figure 19) in order to create the structural foundation scheme reported in Figure 20.
Even though the drawings can deduce additional information, it is worth focusing on the detail of the reinforcement bars in the plinths: since the columns are embedded in the web of the T-shaped cross sections of the foundation beams, the reinforcement bars located at the upper side of the beams need to be shaped so that they pass next to the columns, as shown in Figure 21.

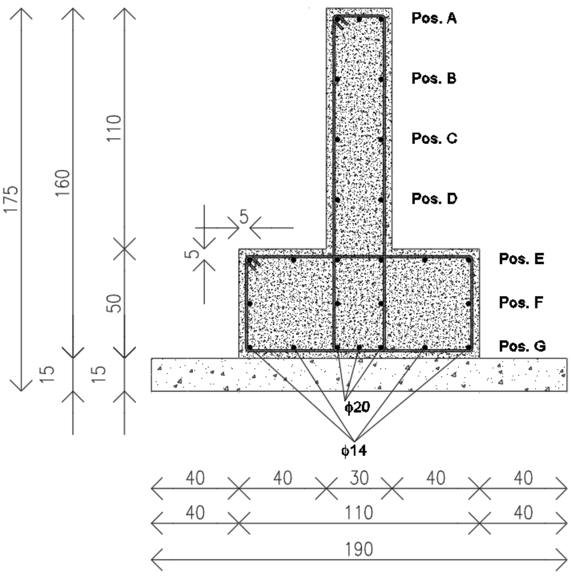

Figure 19 – Cross-section of the foundation beams

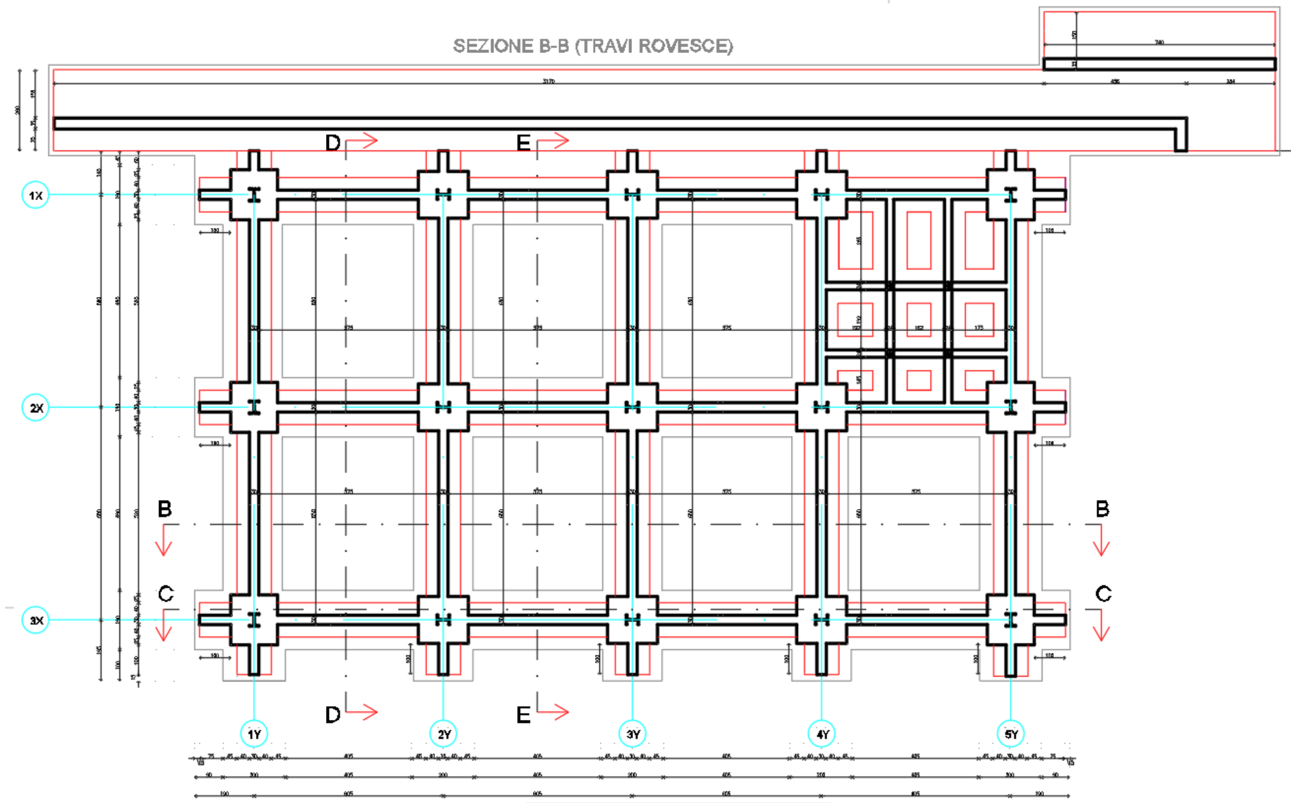

Figure 20 – Foundations

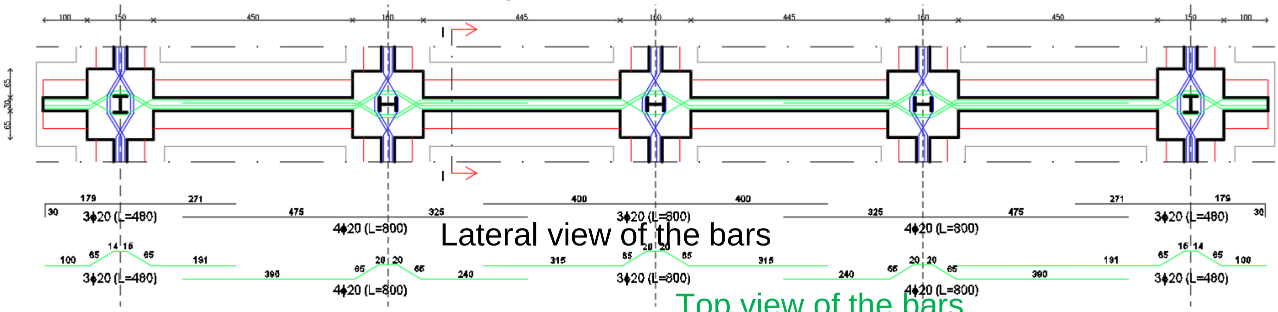

Figure 21 – Detail about reinforcement bars

4. List of complete design documentation

As already stated, according to the Grant Agreement all the design documentation is due in Italian language, because it has to fulfil the Italian Code provisions and has to be delivered to the National local authorities to obtain the relevant authorizations. The detailed design documentation is listed in the following Tables and fully delivered (in Italian) on the project website (www.dreamersproject.eu).

Executive Structural Design Reports

R 13	Relazione Geotecnica	Geotechnical Report
R 14	Relazione di Calcolo delle Fondazioni e delle opere di sostegno	Calculation Report of Foundations and Earth Retaining Walls
R 15.1	Relazione di Calcolo delle Strutture – Corpo Edificio	Structure Calculation Report – Main Building Body
R 15.2	Relazione di Calcolo delle Strutture – Corpo Scala	Structure Calculation Report – Staircase
R 15.3	Tabulati di Calcolo – Corpo Scala – Input	Calculation Tables – Staircase Body – Input
R 15.4	Tabulati di Calcolo – Corpo Scala – Output	Calculation Tables – Staircase Body – Output
R 16	Relazione sui materiali	Materials report
R 17	Relazione Geologica	Geological Report
R 18	Piano di Manutenzione opere strutturali	Structural works maintenance plan
AP 01	Analisi Prezzi strutture	Structural components' price analysis
EP 02	Elenco Prezzi strutture	Price list of structural components
CM 02	Computo Metrico Estimativo strutture	Estimated Metric Computation of Structures

Executive Structural Design Drawings

ST 01.1	Pianta fondazioni e opere di sostegno	Plan of foundations and earth retaining walls
ST 01.2	Fasi costruttive fondazioni - fase 1 (getto magrone)	Foundation construction phases - phase 1 (casting of non-structural slab)
ST 01.3	Fasi costruttive fondazioni - fase 2 (muro di sostegno e suole travi rovesce)	Foundation construction phases - phase 2 (earth retaining walls and flange of inverted T foundation girders)
ST 01.4	Fasi costruttive fondazioni fase 3 (posizionamento piastre di base e completamento travi di fondazione)	Foundation construction phases - phase 3 (positioning of base plates and completion of foundation girders)
ST 01.5	Armature travi di fondazione (1/3)	Foundation girders' reinforcements (1/3)
ST 01.6	Armature travi di fondazione (2/3)	Foundation girders' reinforcements (2/3)
ST 01.7	Armature travi di fondazione (3/3)	Foundation girders' reinforcements (3/3)
ST 01.8	Fondazione corpo scala	Staircase foundation
ST 01.9	Dettagli plinti e piastre di base con tirafondi	Details of plinths and base plates with anchor bolts
ST 01.10	Particolari del muro di sostegno	Details of the earth retaining walls
ST 01.11	Pianta fondazione con posizionamento dime	Foundation plan with positioning templates
ST 02	Scala - piante, prospetti e dettagli	Staircase - plans, elevations and details
ST 03	Scala - rampe in c.a.	Staircase - concrete ramps
ST 04.1	Carpenteria elementi metallici - livello 1	Carpentry of metal elements - level 1
ST 04.2	Carpenteria livello 1 - armature integrative	Carpentry level 1 - supplementary reinforcements
ST 05.1	Carpenteria elementi metallici - livello 2	Carpentry of metal elements - level 2
ST 05.2	Carpenteria livello 2 - armature integrative	Carpentry level 2 - supplementary reinforcements
ST 06.1	Carpenteria elementi metallici - livello 3	Carpentry of metal elements - level 3

ST 06.2	Carpenteria livello 3 - armature integrative	Carpentry level 3 - supplementary reinforcements
ST 07	Carpenteria telaio B-B	B-B frame carpentry
ST 08	Carpenteria telaio C-C	C-C frame carpentry
ST 09	Carpenteria telaio D-D	D-D frame carpentry
ST 10	Carpenteria telai 2-2 e 3-3	Carpentry of frames 2-2 and 3-3
ST 11	Carpenteria telai 4-4 e 5-5	Carpentry of frames 4-4 and 5-5
ST 12	Carpenteria telaio 6-6	Carpentry of frame 6-6
ST 13	Vista assonometrica elementi impalcato 1	Axonometric view of the deck elements: deck 1
ST 14	Vista assonometrica elementi impalcato 2	Axonometric view of the deck elements: deck 2
ST 15	Vista assonometrica elementi impalcato 3	Axonometric view of the deck elements: deck 3
ST 16	Particolari costruttivi – collegamenti – 1	Construction details – connections – 1
ST 17	Particolari costruttivi – collegamenti – 2	Construction details – connections – 2
ST 18	Particolari costruttivi – collegamenti – 3	Construction details – connections – 3
ST 19	Particolari costruttivi – collegamenti – 4	Construction details – connections – 4
ST 20	Elementi singoli - piastre	Single elements - plates

5. Reference standards

[1] L. 05.11.1971, n. 1086. Norme per la disciplina delle opere in conglomerato cementizio armato, normale e precompresso ed a struttura metallica
[2] D.M. LL.PP. del 14.02.1992. Norme Tecniche per l'esecuzione delle opere in cemento armato normale e precompresso e per le strutture metalliche.
[3] D.M. del 09.01.1996. Norme Tecniche per il calcolo, l'esecuzione ed il collaudo delle strutture in cemento armato, normale e precompresso e per le strutture metalliche.
[4] D.M. del 16.01.1996. Norme Tecniche relative ai "Criteri generali per la verifica di sicurezza delle costruzioni e dei carichi e sovraccarichi".
[5] D.M. del 16.01.1996. Norme Tecniche per le costruzioni in zone sismiche.
[6] Circolare Ministeriale del 04.07.1996 n. 156AA.GG./STC. Istruzioni per l'applicazione delle "Norme tecniche relative ai criteri generali per la verifica di sicurezza delle costruzioni e dei carichi e sovraccarichi" di cui al Decreto Ministeriale 16.01.1996.
[7] L. 02.02.1974, n. 64. Provvedimenti per costruzioni con particolari prescrizioni per zone sismiche.
[8] D.M. LL. PP. E INT. 19.06.1984. Norme Tecniche per le costruzioni in zone sismiche.
[9] D.M. LL. PP. 11.03.1988. Norme Tecniche riguardanti le indagini sui terreni e sulle rocce, la stabilità dei pendii naturali e delle scarpate, i criteri generali e le prescrizioni per la progettazione, l'esecuzione ed il collaudo delle opere di sostegno delle terre e delle opere di fondazione.
[10] Circolare Ministeriale del 24.07.1988, n. 30483/STC.
[11] Legge 2 Febbraio 1974 n. 64, art. 1 - D.M. 11 Marzo 1988. Norme Tecniche riguardanti le indagini sui terreni e sulle rocce, la stabilità dei pendii naturali e delle scarpate, i criteri generali e le prescrizioni per la progettazione, l'esecuzione ed il collaudo delle opere di sostegno delle terre e delle opere di fondazione.
[12] Circolare Ministeriale del 15.10.1996 N°252. Istruzioni per l'applicazione delle "Norme Tecniche per il calcolo, l'esecuzione ed il collaudo delle opere in cemento armato normale e precompresso e per le strutture metalliche" di cui al D.M.09.01.1996
[13] Circolare Ministeriale del 10.04.1997 N°65/AA.GG. Istruzioni per l'applicazione delle "Norme Tecniche per le costruzioni in zone sismiche" di cui al D.M.16.01.1996

[14] Ordinanza del Presidente del Consiglio dei Ministri N°3274 del 20.03.2003. Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica.
[15] Ordinanza del Presidente del Consiglio dei Ministri N°3431 del 10.05.2005. Ulteriori modifiche ed integrazioni all'ordinanza N°3274.
[16] Norme Tecniche per le Costruzioni - D.M. 14.09.2005 (TU 2005)
[17] Norme Tecniche per le Costruzioni - D.M. 14.01.2008 (NTC 2008)
[18] Norme Tecniche per le Costruzioni - D.M. 17.01.2018 (NTC 2018) - TECHNICAL STANDARDS FOR BUILDINGS as per D.M. 17/01/2018 - Official Journal 42 of 20 February 2018 (in Italian)
[19] Circular no. 7 of 21 January 2019 "Instructions for the application of the "Update of the "Technical standards for construction" referred to in the ministerial decree of 17 January 2018″ (in Italian)
[20] UNI EN 13670 - Execution of concrete structures
[21] UNI EN 1992-1-1: 2005 – Eurocode 2. Design of concrete structures Part 1-1: General rules and rules for buildings
[22] UNI EN 1993-1-1:2014 - Eurocode 3 - Design of steel structures - Part 1-1: General rules and rules for buildings
[23] UNI EN 10027-1:2016 - Steel designation systems - Part 1: Symbolic designation
[24] UNI EN 10025-1 - Hot rolled steel products for structural uses - Part 1: General technical supply conditions
[25] UNI EN 10025-2:2005 - Hot rolled steel products for structural uses - Part 2: Technical conditions for the supply of non-alloy steels for structural uses.