



Deliverable 7.3.4

MR-CH exposure monitoring Proof of Concept

Work Package 3, Task 7.3.2

Delivery Date: 15.01.2026

Lead: DIDA - UNIFI

Document information

Project Acronym: RETURN

Project Title: multi-Risk sciEnce for resilientT commUnities undeR a changiNg climate

Project duration: 2022-2025



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Return

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List of Acronyms

AdB – Arno River Basin Authority
ADRS – Acceleration-Displacement Response Spectrum
CH – Cultural heritage
CDP – Concrete Damage Plasticity
DA – Ancient Deposits
DDR – Disaster Risk Reduction
DEM – Digital Elevation Model (4.2.1)
DEM – Discrete Element Method (5.3.2)
DGPV – Deep-seated gravitational slope deformation
DMC – Degenerate Mohr–Coulomb
DS – Damage State (2.5.3)
DS – Distributed Scatterers (3.3.1)
DR – Recent Deposits
EC – European Commission
EFDD – Enhanced Frequency Domain Decomposition
EMA – Experimental Modal Analysis
EU – European Union
EWS – Early Warning System
FA – Amplification Factors
FAAb – Sands and sandy clays
FAAo – Val d’Orcia Clays
FAAt – Val Tuoma clays
FDD – Frequency Domain Decomposition
FEA – Finite Element Analysis
FEM – Finite Element Model
GIS – Geographic Information System
GCP – Ground Control Points
HEC-RAS – Hydrologic Engineering Centre’s River Analysis System
HRP – Flood Risk Plan
HVSr – Horizontal to Vertical Spectral Ratio
ICMS – Guidelines and Criteria for Seismic Microzonation
ICP – Iterative Closest Point
IDW – Inverse Distance Weighting
IM – Intensity Measure
INGV – Istituto Nazionale di Geofisica e Vulcanologia



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LiDAR – Light Detection and Ranging
LOS – Line-Of-Sight
LTSM – Limiting Tensile Strain Method
MOPS – Microzonazione Operativa per la Pianificazione Sismica
MR – Multi-Risk
ND – No Data
NRTK – Network Real-time Kinematic
OMA – Operational Modal Analysis
OPCM – Ordinance of the President of the Council of Ministers
PAI – Hydro-Geological Plan
PGA – Peak Ground Acceleration
PGRA – Flood Risk Management Plan
PLlc – Organic Limestone
PLIs – Pienza yellow sands and sandstones
PSI – Persistent Scatterer Interferometry
PUNDIT – Portable Ultrasonic Non-destructive Digital Indicating Tester
RC – Reinforced Concrete
SAR – Synthetic Aperture Radar
SB – Basin Synthem
SfM – Structure from Motion
SfM_IM – Structure from Motion, Image Matching
SILL – Sillano Formation
SIR – Hydrological Service of Tuscany
SITR – Sistema Informativo Territoriale Regionale
SLD – Damage Limit State
SLV – Life Safety Limit State
SM – Seismic Microzonation
SSI-cov – Covariance Driven Subspace Stochastic Algorithm
SU – Structural Unit
TLS – Terrestrial Laser Scanner
TS – Transversal Spoke
UAV – Unmanned Aerial Vehicle
UCS – Uniaxial Compressive Strength
UPV – Ultrasonic Pulse Velocity
WMO – World Meteorological Organization
WP – Work Package
XRF – X-ray fluorescence

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1 Introduction

The protection of Cultural Heritage (CH) assets requires innovative and multidisciplinary efforts capable of addressing the increasing threats posed by natural and anthropogenic hazards. In response to this growing need, the Transversal Spoke 7 is developing integrated frameworks for monitoring, assessing, and mitigating multiple risk sources affecting CH into unified multi-risk (MR) approaches. The presented research involves multiple spatial scales ranging from individual monuments to urban centres and regional networks.

This document illustrates a series of coordinated proof-of-concept (PoC) studies conducted by various research group from the University of Florence (UNIFI) and the University of Cagliari (UNICA), focusing on representative heritage contexts across Italy. Each case study demonstrates the implementation of advanced monitoring strategies, digital modelling techniques, and MR assessment methodologies tailored to the specific vulnerabilities of the selected CH sites.

2 CH monitoring at asset scale – Museo Marino Marini

2.1. The proof of concept (DIDA – UNIFI)

The proof of concept selected at asset scale by UNIFI research units is Marino Marini Museum in Florence. The PoC has been investigated considering two distinct hazards affecting the case study, i.e. the seismic risk and the flood risk. As the two natural sources are generally uncorrelated, the two risk analyses are treated separately. Concerning the monitoring activity, the focus has been on the updating of the seismic risk, carried out through the information coming from the results of a dynamic identification of a salient portion of the Museum.

The Marino Marini Museum is located in the historic center of Florence, just a short distance from landmark monuments such as the Basilica of Santa Maria Novella, the Cathedral of Santa Maria del Fiore and Palazzo Vecchio (Figure 1). Its proximity, approximately 200 meters, to the Arno River further emphasizes the need for an integrated multi-hazard assessment strategy. Due to its strategic location and the inherent vulnerability to both seismic and hydrological events, a comprehensive risk assessment is essential. The analysis must address not only the structural system, the museum building itself, but also the valuable contents it houses, namely the collection of sculptures and artworks by Marino Marini, the renowned 20th-century sculptor from whom the museum takes its name. Figure 2 presents an aerial overview of the historic center of Florence, highlighting the broader urban context in which the case study is embedded. A second, closer aerial view focuses specifically on the Marino Marini Museum, offering a clearer understanding of its spatial configuration and physical relationship with the surrounding architectural and infrastructural fabric.



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Figure 1. The historical city centre of Florence and the location of the Marino Marini Museum.



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a)



b)

Figure 2. Aerial views of the area of Florence where the Marino Marini Museum is located.

2.2. Survey (DIDA – UNIFI)

This section presents all the knowledge-acquisition activities that led to an improved understanding of the structure under investigation. The knowledge process was articulated into several complementary phases, aimed at reconstructing the historical, geometrical, structural, and geotechnical characteristics of the building.

In particular, the section begins with a historical analysis of the different construction and transformation phases of the structure, which is essential for identifying major structural discontinuities, construction techniques adopted over time, and modifications affecting the original building layout. Subsequently, the results of the seismic microzonation studies of the site are presented.

The knowledge framework is further developed through detailed geometric acquisition by means of laser scanner surveying, which constituted a reliable basis for subsequent modelling. This activity is complemented by a thermographic survey campaign, employed as a non-destructive diagnostic technique to detect constructional heterogeneities, voids, masonry discontinuities and potential structural anomalies that are not directly observable.

Finally, based on the comprehensive set of collected data, both two-dimensional and three-dimensional representations of the structure were developed. These representations are intended to provide an accurate geometric and structural restitution of the building and to support the development of the numerical models adopted in the subsequent analyses.

2.2.1 Historical development

Situated within a densely inhabited area of significant historical and architectural value, the Marino Marini Museum is housed within the pre-existing medieval Church of San Pancrazio, first documented in the early 9th century. Initially entrusted to the Benedictine nuns of Sant'Ilario, it subsequently passed to the Dominican friars and later to the Vallombrosan order, which retained it until 1808.

From the 14th century onwards, the church underwent significant renovations, culminating over a century later with the completion of the Rucellai Chapel in 1467. Commissioned by the noble Florentine family and designed by architect Leon Battista Alberti, the chapel houses the Tempietto del Santo Sepolcro, a masterpiece by Alberti himself that replicates the Holy Sepulchre of Jerusalem on a reduced scale. In the mid-18th century, architect Giuseppe Ruggeri introduced further modifications to the structure, including the removal of numerous tombs of illustrious families and citizens and the construction of a frescoed dome above the transept.

During the Napoleonic era, the church was deconsecrated in 1808 and it was repurposed as the seat of the Imperial Lottery of France. The structure was altered in accordance with the neoclassical aesthetic of architect Giuseppe Cacialli, who introduced two central columns flanked by two lion sculptures on the façade.

After serving as a court, in 1883 the Church of San Pancrazio was converted into the Royal Tobacco Factory, undergoing a radical transformation: the 18th-century central nave was partitioned into two levels through the installation of a metal mezzanine, whose riveted iron beams remain visible today. The building maintained this configuration until World War II and the immediate post-war period, when it was repurposed as a storage facility for the adjacent convent's military barracks.

Between the 1960s and the 1970s, following its decommissioning by the military administration, the building began to be recovered. In the 1982 the restoration of the San Pancrazio complex was entrusted to architects Bruno Sacchi and Lorenzo Papi. Their project aimed to preserve and enhance the diverse historical layers into a unique museal path and the result is the current configuration of the museum (Figure 3). The guiding principle of the restoration was to ensure the legibility of the monument's historical stratifications, from the medieval shell to the 18th-century internal framework and the 19th-century proto-industrial iron superstructures. The interventions by the two architects are characterized by a dynamic interplay of circulation paths, stairs and walkways clearly distinguished in form and colour from the ancient structures, and a deliberate contrast between the pre-existing masonry elements and the contemporary additions, which remain distinct in both materials and function. The entrance to the new museum, located behind the columned façade, features a transparent counter-façade, horizontally



interrupted by an exposed concrete beam that contrasts with the vertical continuity of the neoclassical pronaos. This transparent partition allows for a comprehensive visual perception of the museum's interior space, enabling visitors to grasp the full longitudinal development of the building.

The only part of the structure requiring a complete reconfiguration was the rear wall, corresponding to the former choir and apse, which has been emptied and enclosed with a transparent partition. Upon entering, a compact hall seamlessly transitions into the museum's ground floor, where Marino Marini's sculptures are arranged along the longitudinal axis, culminating in the collection's most significant piece, both in artistic value and scale, the monumental six-meter-high bronze Grande Cavaliere. The placement of this work, centrally positioned within the former presbytery under the cupola and illuminated by the transparent apse wall, influenced many of the project's architectural choices.

The full-height opening of this space is further accentuated by the preserved massive 19th-century riveted iron beams supporting the new mezzanine, creating an interplay of horizontality and verticality, solidity, and transparency. The two arms of the transept lead to different sections: the northern side provides access to the crypt and walkways, while the southern side houses former chapels now repurposed as sculptures rooms, service areas, offices, and additional staircases.

The concept of walkways is central to the project and is closely linked to the dynamic experience of Marini's work. These elevated paths extend across four levels, interconnected by staircases and skylights that establish visual continuity between different floors of the museum. The two main staircases, located in the transept, lead to the third level of the walkways, corresponding to the first floor. A spiral staircase provides access to the fourth and final level, a simple linear walkway positioned solely along the northern side.

The museum houses a vast collection of artworks displayed across all levels, comprising 183 pieces, including drawings, lithographs, paintings, and sculptures by the Pistoia-born artist Marino Marini, after whom the museum is named.



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Figure 3. Current configuration of the museum.

2.2.2 Seismic microzonation

Seismic risk represents one of the primary geological-environmental criticalities for vast areas of Italy, and particularly for Tuscany, which is characterised by Apennine-type seismicity, with earthquakes of moderate-to-strong intensity. The city of Florence, while not historically associated with destructive seismic events like other Italian areas, is located in a zone classified as having medium seismicity. Awareness of this condition makes a thorough analysis of seismic risk indispensable. This analysis should not be limited to mere hazard (the probability that a seismic event of a given intensity will occur) but must also consider the vulnerability of the built environment and infrastructure, and the exposure of people and assets. In this context, seismic microzonation (SM) emerges as a fundamental tool for risk mitigation, allowing for the identification of local ground responses to seismic input.

The basic seismic hazard of the Florentine territory is primarily defined according to the National Seismic Hazard Map (MPS04) developed by the Istituto Nazionale di Geofisica e Vulcanologia (INGV), which provides probabilistic estimates of ground motion parameters for different return periods. Florence is located in a region of moderate seismic hazard, with expected peak ground acceleration (PGA) values ranging between 0.15 g and 0.175 g for a 475-year return period, assuming reference conditions of rock outcrop.

From a seismotectonic perspective, the territory is influenced by two main seismogenic zones: Zone 915 (Mugello) and Zone 916 (Chianti), both of which have been historically active (Figure 4). While no major earthquakes have originated within the municipal boundaries, the area has repeatedly experienced moderate ground shaking as a result of regional events, such as the 1895 Impruneta earthquake and the 1919 Mugello earthquake.



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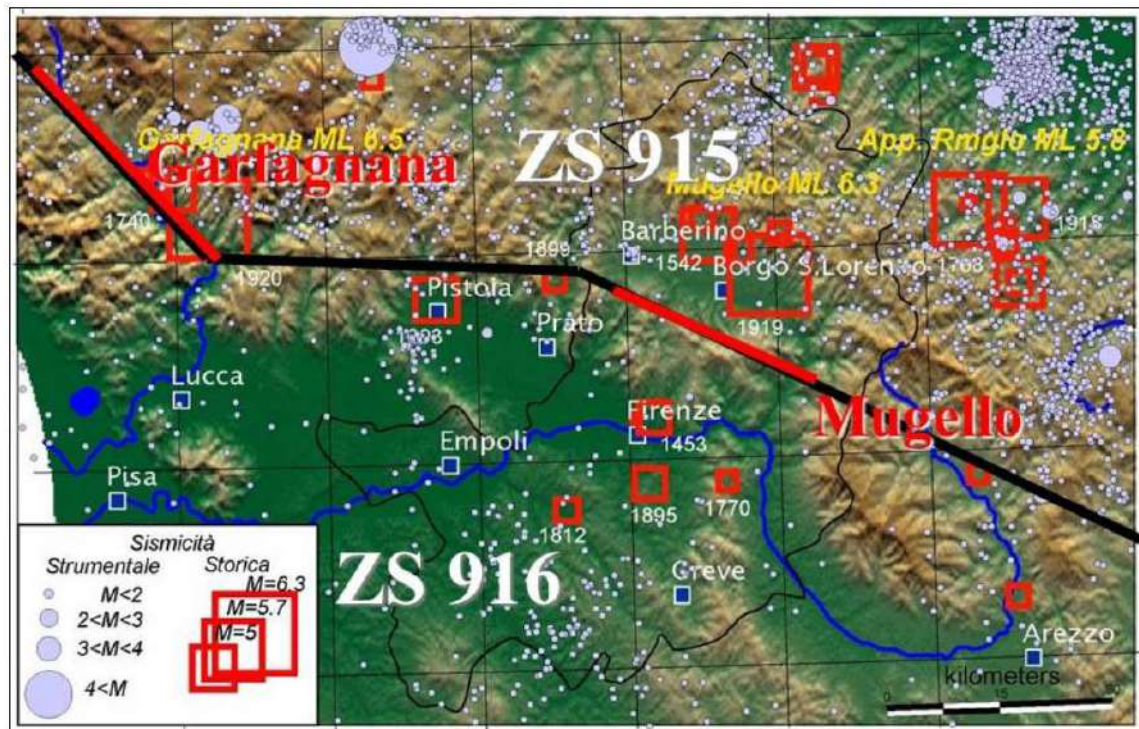


Figure 4. Seismogenic zones, image taken from [1].

The official seismic classification of the Municipality of Florence, established in accordance with national civil protection directives and regional planning instruments, places the entire territory within Seismic Zone 3. This classification reflects a level of seismic hazard in which structural design must account for horizontal ground accelerations and local amplification effects.

Furthermore, the microzonation study integrates this basic hazard assessment with site-specific investigations to evaluate how local lithostratigraphic and morphological conditions influence seismic response. This approach ensures a refined understanding of seismic risk beyond the regional-scale hazard model.



Seismic microzonation is the study of local seismic effects, meaning the modifications that seismic waves undergo due to the geological, geomorphological, and geotechnical characteristics of the subsoil. These effects can include:

- Seismic motion amplification: An increase in the amplitude and duration of ground motion, especially on soft soils or with particular topographical configurations.
- Permanent ground deformations: Phenomena such as liquefaction, seismically induced landslides, differential settlements, or surface fault ruptures.

In Tuscany, seismic microzonation has been promoted and implemented in various municipal administrations, also following the Ordinance of the President of the Council of Ministers (OPCM) No. 3274/2003 and, subsequently, with the "Guidelines and Criteria for Seismic Microzonation (ICMS)" [2] from the Department of Civil Protection. These documents have provided a normative and technical framework for conducting SM studies at various levels of detail (Level 1 - MOPS, Level 2 - SM, Level 3 - Advanced SM).

The Municipality of Florence is among the first in Italy to have carried out seismic microzonation studies across all three methodological levels established by the national guidelines [2]. These studies aim to refine the basic seismic hazard assessment by identifying zones with homogeneous seismic behaviour, based on geological, geotechnical, and seismological conditions. The project aimed to improve the understanding of local seismic hazards by integrating geological, geophysical, and geotechnical data, enabling informed urban planning and seismic risk reduction strategies. The initiative was realized in collaboration with research institutions and technical consultants, and it covers all three levels of seismic microzonation.

For the city of Florence, seismic microzonation studies have been conducted, although their extent and level of detail may vary depending on the areas and objectives. Such studies are crucial for:

- Urban planning: Identifying areas most suitable for new constructions or urban regeneration, considering the seismic response of the subsoil.

- Structural design: Providing designers with specific parameters for the seismic characterization of the site (e.g., Vs30 shear wave velocity, fundamental ground period of vibration), essential for accurate seismic calculation of buildings.
- Emergency management: Delimiting areas with greater potential for damage in the event of an earthquake, allowing for better planning of escape routes, assembly points, and rescue areas.

An example of the application of seismic microzonation for risk mitigation concerns the evaluation of amplification effects. The presence of sequences of incoherent and cohesive alluvial deposits, typical of the Florence Plain, makes these areas susceptible to resonance phenomena. SM studies allow for mapping these zones and implementing specific technical requirements for constructions.

The geomorphology of the Florentine territory reflects a complex interplay between tectonic processes and fluvial dynamics. The municipal area is predominantly situated within the Florence–Prato–Pistoia tectonic basin, a morphostructural depression bounded by the Apenninic reliefs. This basin is infilled with a thick sequence of Quaternary alluvial deposits, whose thickness locally exceeds 100 meters, particularly within the Arno River plain.

The central portion of the municipality is characterised by the wide alluvial plain formed by the Arno and its tributaries, which have shaped a flat, low-lying landscape. This plain hosts the historical urban core and is underlain by alternations of gravel, sand, silt, and clay layers, resulting in considerable heterogeneity in terms of lithological and geotechnical properties.

In contrast, the surrounding sectors, especially toward the north and south, include hilly morphologies composed of lithotypes such as sandstone (Macigno) and marl-limestone formations (Pietraforte), which are part of the Tuscan Nappe system. These elevated areas show well-defined hydrographic networks and higher morphodynamical activity, including slope instabilities and erosional features.

The morphological transition between the alluvial lowlands and the adjacent hills is abrupt in some sectors and gradational in others, producing a diversified topographic setting that significantly influences local seismic response. In particular, the marked lithostratigraphic contrast between soft alluvial deposits and

rigid bedrock contributes to seismic wave amplification phenomena, which are crucial in microzonation assessments.

The subsoil model of the Florence area is the result of an integrated analysis combining geological, geophysical, and geotechnical data as prescribed by the National Guidelines for Seismic Microzonation [2]. The model underpins all higher-level seismic microzonation assessments and plays a crucial role in evaluating local seismic response phenomena such as amplification and ground deformation.

The urban area of Florence extends predominantly over the alluvial plain of the Arno River, with altitudes ranging from 45 to 60 meters above sea level. The lithostratigraphic sequence beneath the plain comprises a thick package of Quaternary alluvial deposits overlying pre-Quaternary bedrock. These deposits are highly heterogeneous and include recent fluvial sediments, ancient alluvial units, and basin infill synthems.

According to the 1st-level microzonation, the territory was subdivided into 37 homogeneous seismic microzones (MOPS), each characterized by specific stratigraphic and geotechnical profiles (Figure 5). The lithological units identified include:

- **DR** (Recent Deposits): mainly silty and clayey layers associated with modern fluvial dynamics.
- **DA** (Ancient Deposits): more consolidated alluvial sediments, often including interbedded sands and gravels.
- **SB** (Basin Synthem): fine-grained, clay-rich layers representing the principal basin infill.
- **SILL** (Sillano Formation): the underlying bedrock unit, often composed of marl and sandstone.
- **Macigno** and **Pietraforte** formations: encountered in the hilly areas, these lithotypes form the structural highs of the Tuscan Nappe and represent the seismic bedrock.



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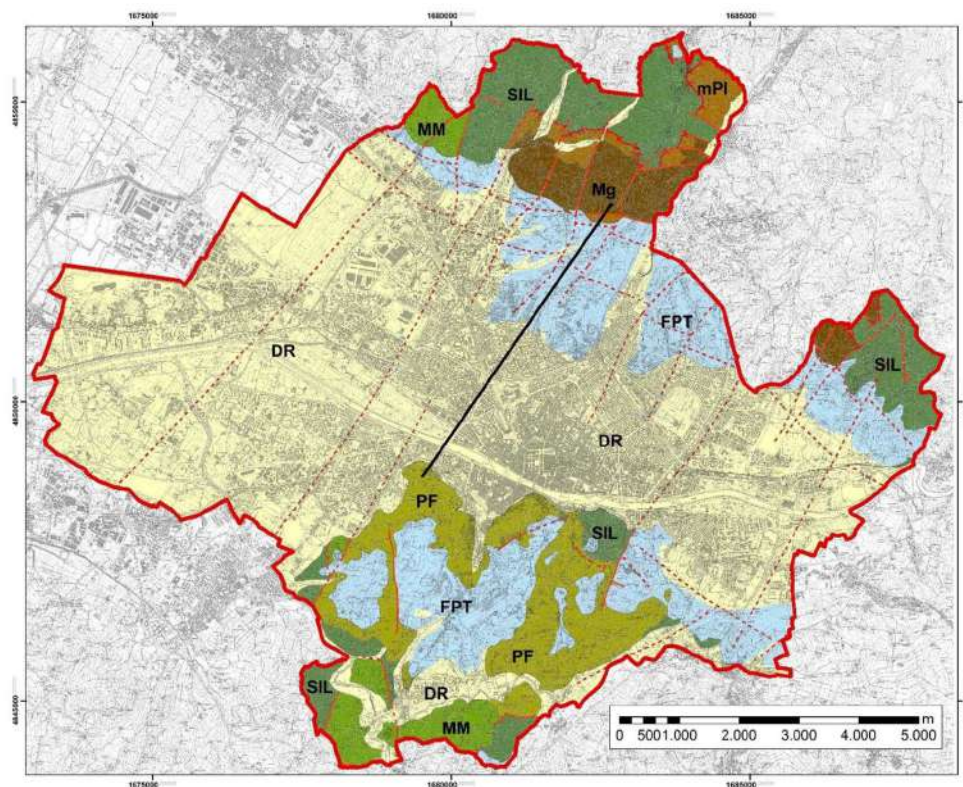


Figure 5. Simplified geological framework of the layout of the Florentine area [1].

From a geotechnical perspective, the subsoil model includes classification of soils according to V_{s30} (the average shear wave velocity in the top 30 meters), which is fundamental for estimating site amplification. These data were obtained through HVSr (Horizontal to Vertical Spectral Ratio) analyses and geophysical surveys.

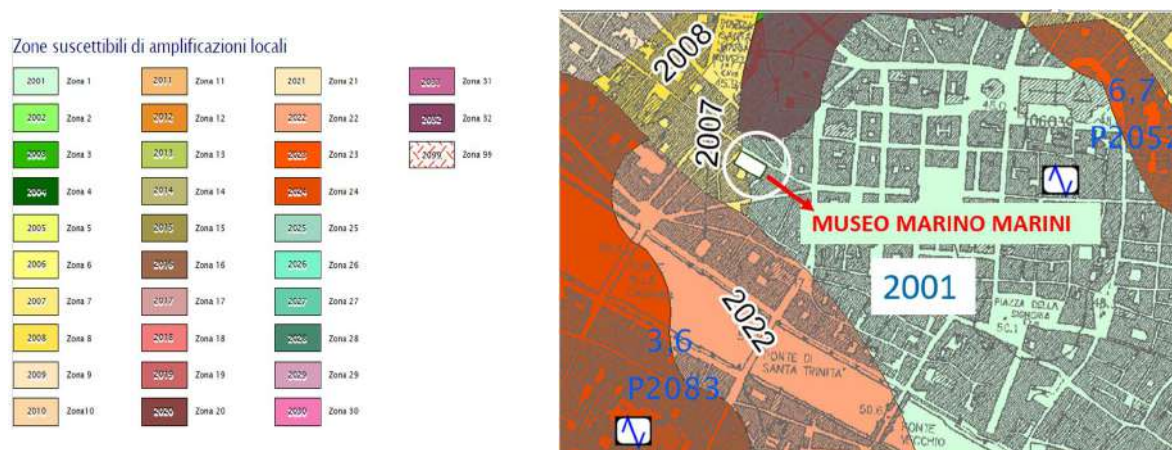
In the 2nd-level microzonation, the subsoil model is further refined using maps of fundamental frequency (f_0), V_s profiles, and stratigraphic amplification factors (FA). The resulting microzones are categorized by their susceptibility to local amplification phenomena.

The 3rd-level microzonation incorporates 1D and 2D numerical simulations, combining geological sections and seismic response analyses. These models provide response spectra ($T_r = 475$ and 50 years) for each subzone, helping define design spectra that account for local stratigraphy.

In conclusion, the subsoil model of Florence highlights a complex interplay between soft alluvial deposits and rigid substrata. This geological variability greatly influences seismic hazard at the local scale, making microzonation an indispensable tool for land-use planning and seismic design.

Seismic Microzonation of the Marino Marini Museum Area (Florence)

The Marino Marini Museum is located in Piazza San Pancrazio, in the historical center of Florence. This area was carefully analysed during the second and third levels of the municipal seismic microzonation studies. These analyses revealed that the museum is situated in a sector of the city characterised by significant local amplification effects, due to the presence of soft alluvial deposits beneath the surface. Specifically, according to the “Carta delle Microzone Omogenee in Prospettiva Sismica” (Figure 6), the museum lies between MOPS zone 1-2001 and MOPS zone 7-2007. These zones differ significantly in their geological composition and expected seismic response. However, a detailed observation of the spatial distribution indicates that the site is predominantly located within Zone 1-2001. This classification has important implications for seismic risk assessments and the selection of appropriate design spectra for the area.



amplification. Given that the Marino Marini Museum is mainly located in zone 1, it can be assumed that the seismic hazard at this site is comparatively lower than in adjacent sectors falling under zone 7. Nevertheless, the proximity to the boundary suggests that site-specific assessments could still be warranted for critical structures, especially in the context of historical preservation and seismic resilience.

According to the second-level microzonation results, the area surrounding the museum exhibits stratigraphic conditions that contribute to an amplification factor (FA) ranging between 1.7 and 1.8. This value already reflects a non-negligible modification of the seismic input due to local soil properties. Moreover, it is noteworthy that adjacent areas register even higher FA values, reaching up to 2.1–2.2, indicating that the broader urban context, particularly in the central basin zone, is subject to enhanced seismic amplification, Figure 7.



Figure 7. Image taken from “Carta della Microzonazione Sismica di II Livello”, Regione Toscana.

The third-level microzonation study provided a more detailed assessment of local seismic response through both one-dimensional and two-dimensional numerical simulations. According to the “Carta della Microzonazione Sismica di III Livello” the Marino Marini Museum was classified within Microzone 5, which is defined by thick sequences of fine-grained alluvial deposits with low to medium shear wave velocities. The site-specific modelling for this microzone yielded an FA value of 1.97, confirming the presence of significant amplification phenomena (Figure 8). This result can be used to calibrate design response spectra specific to the site, for return periods of both 475 and 50 years, as required by seismic design standards. The results of the simulations revealed a significant amplification of spectral response compared to the standard elastic spectra defined by the Italian Building Code (NTC 2008) for Seismic Zone

3. In particular, zones characterized by soft alluvial deposits showed increased spectral accelerations, especially in the frequency range between 0.5 and 1.5 Hz. These areas also exhibited a shift of the spectral peak toward more extended periods, reflecting the dynamic influence of low-stiffness, thick soil layers.

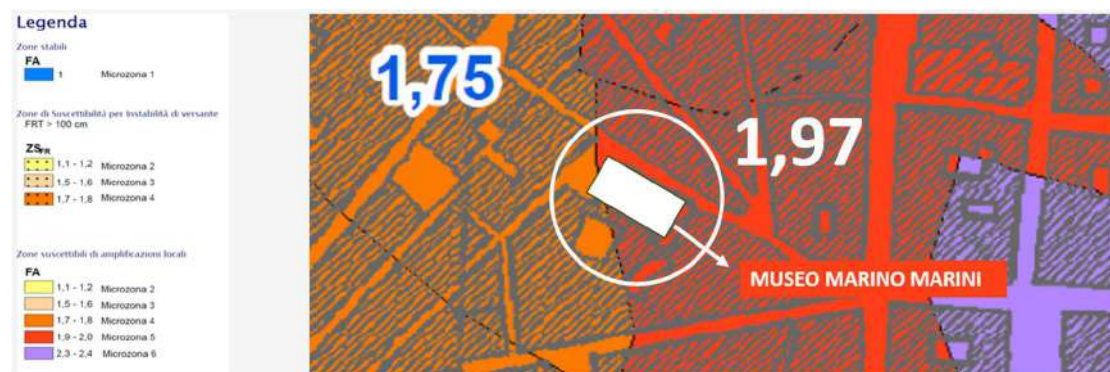


Figure 8. Image taken from “Carta della Microzonazione Sismica di III Livello”, Regione Toscana.

These site-specific spectra can be used to define design accelerations for each microzone, providing a more accurate basis for structural design. Their application is fundamental not only for the design of new buildings, but also for the seismic retrofitting of existing structures, especially in historically and architecturally sensitive areas. Furthermore, these data support informed urban planning strategies aimed at mitigating seismic risk across the municipal territory. By integrating local geological information with seismic response modelling, the third-level microzonation provides a reliable framework for enhancing the safety and resilience of Florence’s built environment.

Geotechnically, the subsoil beneath the museum consists of a complex alternation of silty and clayey materials deposited in a fluvial environment, overlying more consolidated sandy-gravelly layers. The HVSR measurements conducted in this area indicate low fundamental frequencies (f_0), which are consistent with the presence of thick soft layers and correspond to resonance periods that may interact unfavourably with the dynamic characteristics of buildings in the area.

In conclusion, the seismic microzonation results for the Marino Marini Museum highlight that local site effects should be included in detailed seismic vulnerability analyses.



2.2.3 Laser scanner acquisition

The available documentation concerning the structural configuration of the museum and its transformations over the centuries is limited, and the existing drawings contain significant qualitative and quantitative inaccuracies. These inconsistencies affect not only the general layout of the building but also key dimensional and geometric parameters essential for any reliable structural assessment. To address this issue, a comprehensive survey campaign was conducted using terrestrial laser scanner (TLS) technology.

A laser scanner survey consists of many scans that capture in a three-dimensional space the external and internal surface of a physical object by means of a TLS, an optical instrument that allows the generation in near real-time fine geometric representation of object by means of accurate dense cloud of 3D points. This method is particularly well-suited for heritage buildings, as it allows for the capture surfaces without physical contact or disruption to the structure.

In the present study, TLS was used to acquire a complete point cloud of the entire museum building, including the Rucellai Chapel. To achieve a comprehensive survey with minimal data gaps and sufficient overlapping scans, a total of 296 scans was conducted with a resolution ranging from 3 mm to 6 mm (Figure 9).

The acquired point cloud data (Figure 10) was processed using Autodesk ReCap Pro © software to generate 2D drawings for comparison with the existing design documentation.

The process involved data cleaning, segmentation, and vectorization. Despite the precision of TSL systems, point clouds often contain redundant or noisy data due to occlusions, surface reflectivity or the presence of temporary objects.

Following segmentation, horizontal sections at different strategic elevations, from the ground floor to the fourth level of walkways, were extracted, along with vertical sections and internal elevations of the museum. The resulting graphic outputs were subsequently refined in a CAD environment. From the 2D drawings, a solid 3D model of the building was realized.



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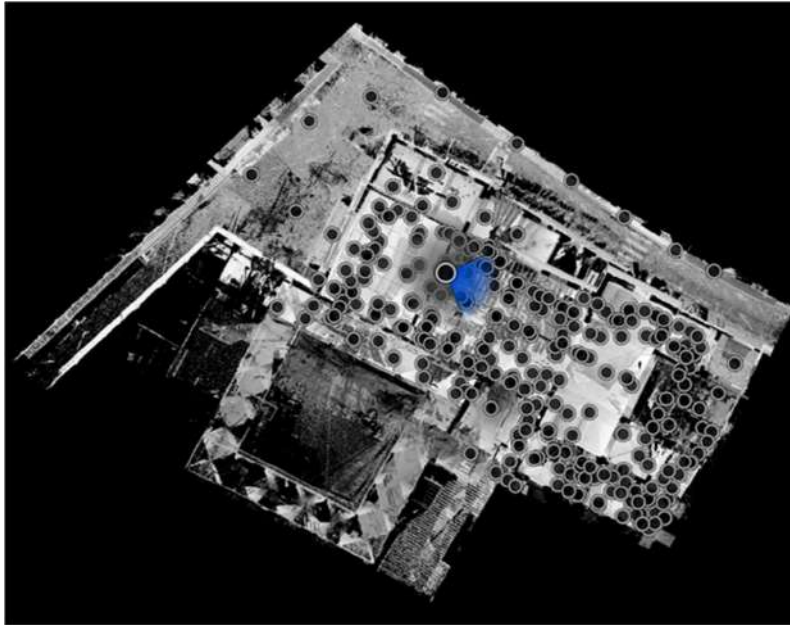


Figure 9. The total amount of scans and the laser scanner.

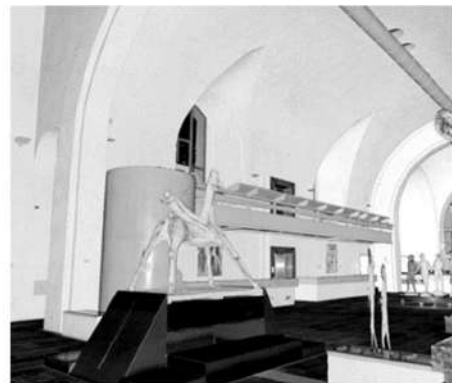


Figure 10. Scans inside the museum.

This approach made it possible to accurately measure each architectural element and compare the results with the existing drawings. For instance, the wall perpendicular to the main façade was found to be 50 cm thick, while the drawings we had indicated a thickness of 80 cm (Figure 11).



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It was clear from the outset that, in order to develop a finite element 3D model, a laser scanning survey campaign was essential to achieve the necessary level of accuracy and precision.

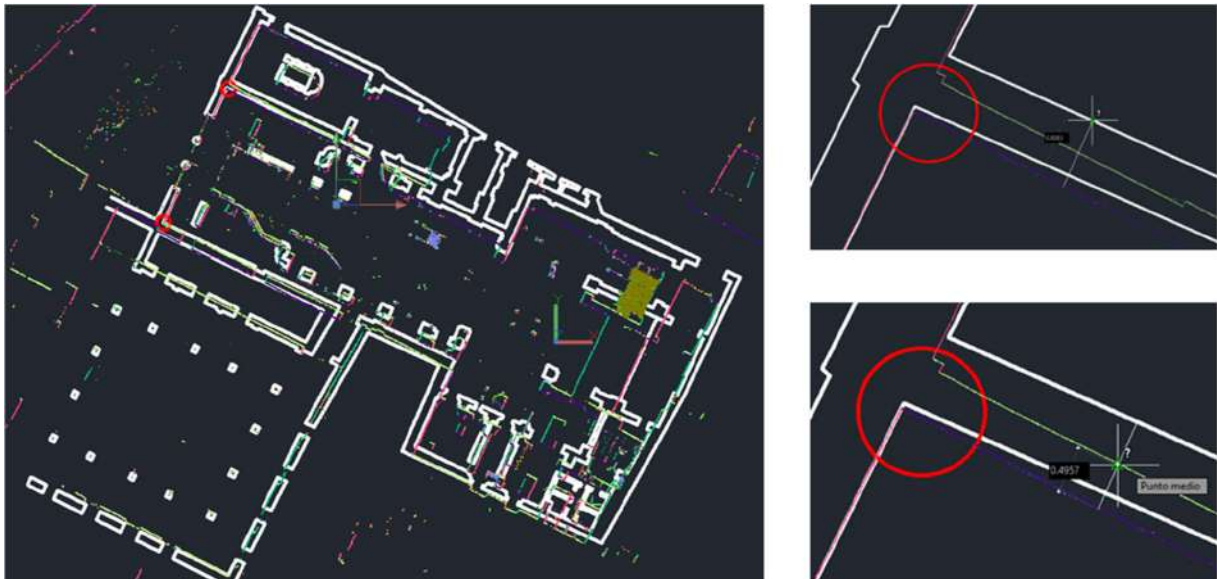


Figure 11. Drawings from the point cloud.

2.2.4 Thermography

Thermographic inspections use remote sensing tools to detect infrared radiation, producing colour maps based on temperature variations. In architecture, this technique helps assess structural integrity, often identifying electrical and hydraulic systems hidden beneath plaster. In the field of CH buildings, it is used to identify changes in the materials, discontinuities, and past alterations. It is especially suitable for historic structures, as it allows investigations without physical contact, making it ideal for uncovering structural details in both vertical and horizontal elements concealed by plaster.

To execute the thermography campaign, a FLIR T460 instrument with a thermal sensitivity lower than 0.03°C and a spectral range of $7.513\text{ }\mu\text{m}$ has been used. Due to environmental conditions in terms of temperature inside and outside the museum, the outcomes of the investigation did not provide clear results, however, the investigation from the inner side allowed showing the likely presence of a concrete



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ring beam over the façade, as on the right-side wall of the central nave. In other cases, the bearing walls in the elevation appeared more homogeneous, as these results are sensitive to the thickness of plaster layers. Figure 12 shows representative images of the investigation carried out.

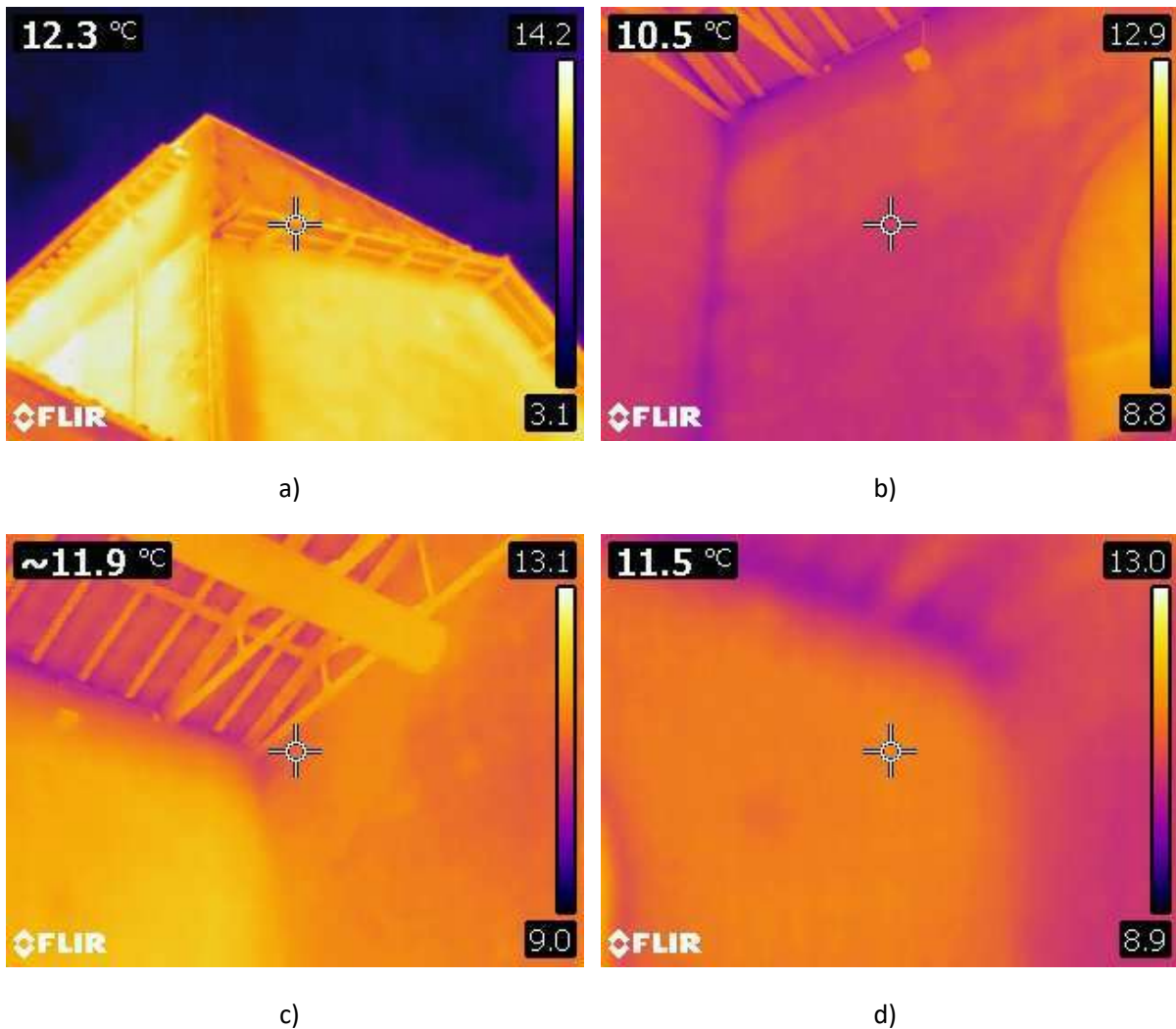


Figure 12. Thermography campaign of the Museum; a) view of the facade from San Pancrazio Square; b) view of the facade from the central nave; c) connection between facade and southern side wall; d) connection between the facade and the northern side wall.

2.2.5 CAD modelling

The complexity and irregularity of the museum structure made the modelling step a very challenging task. Based on the processed two-dimensional drawings, a solid three-dimensional model of the building was developed eliminating geometrical details not relevant to describe the mechanical behaviour of the structure. This model served as geometric reference for the subsequent phases of numerical modelling.

The 3D solid model was then imported into a finite element analysis (FEA) environment, divided into parts to facilitate both the import process and the subsequent phases (Figure 13). At this stage, several geometric simplifications and regularizations were required to ensure compatibility with the computational requirements of the software and to optimize mesh generation. These adjustments involved the removal of secondary details, the regularization of curved surfaces and the simplification of geometrically complex elements, while preserving the overall representativeness of the structure. In any case, the model faithfully reproduces the actual structure of the museum (Figure 14). The basement level of the crypt was also modelled, including the irregular cross vaults (Figure 15).

In parallel, the solid model was also used as the basis for the construction of a BIM model by the ARCHES research unit of Rome Tre University. This information model was aimed at achieving a complete and structured digital representation of the building, with the goal of supporting management, monitoring and conservation activities within a multidisciplinary framework.



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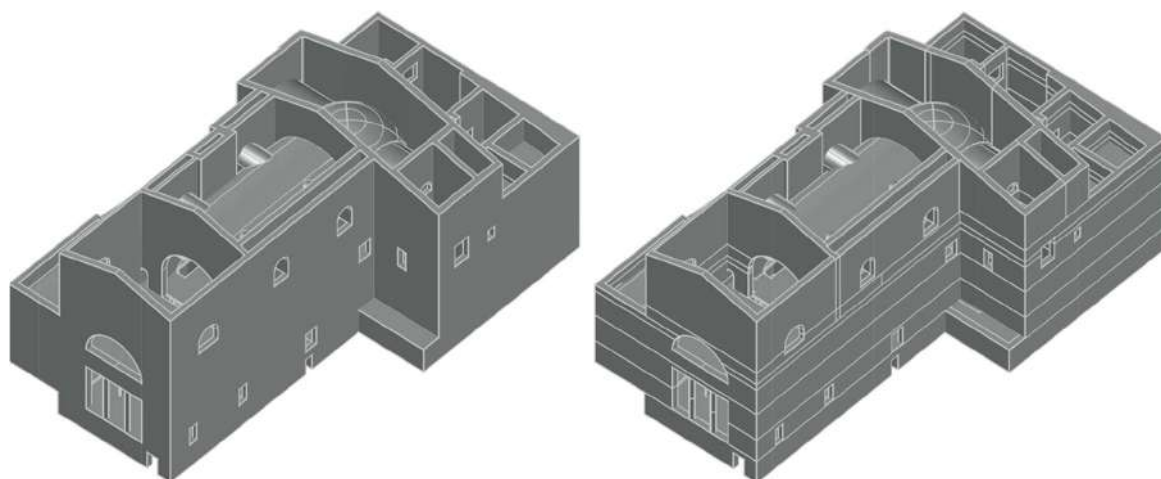


Figure 13. Solid 3D model in CAD environment. On the left, the model representing the building, on the right, the model partitioned for import into the finite element environment.

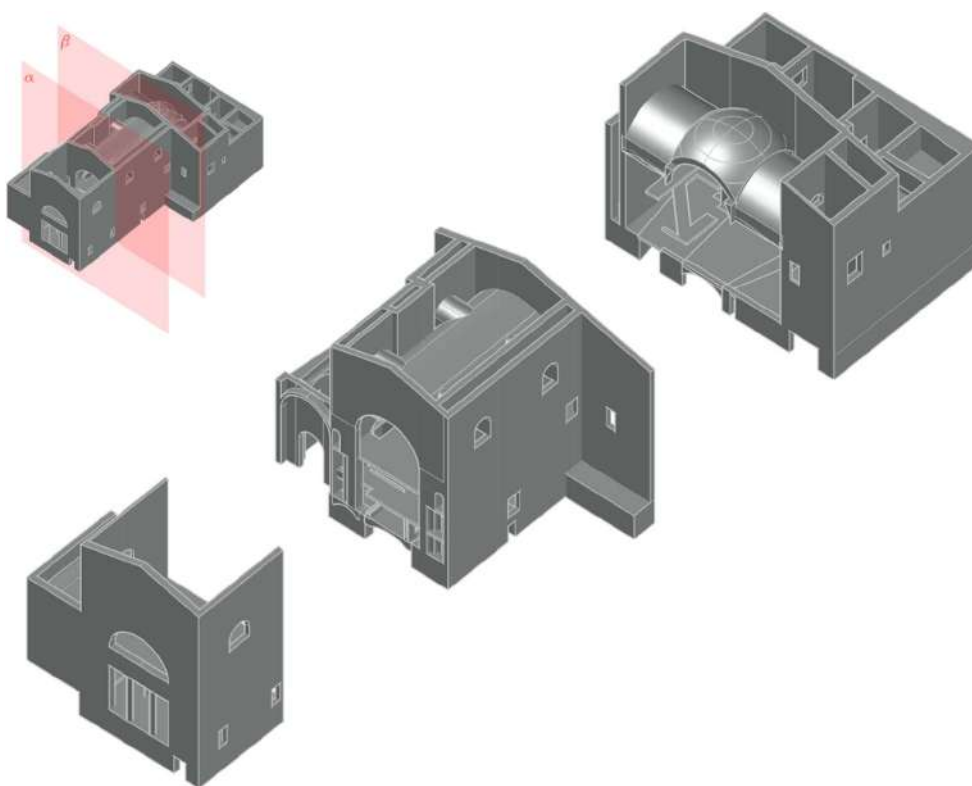


Figure 14. Exploded axonometric view of the museum.



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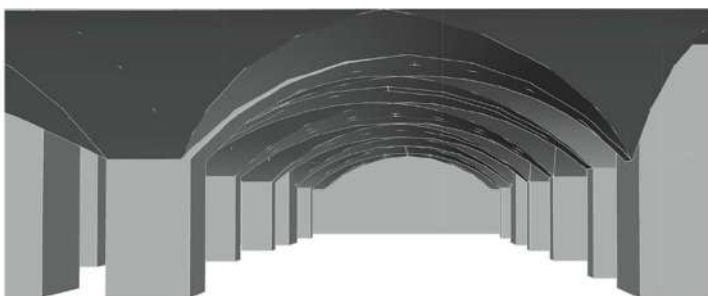


Figure 15. Basement level: on the left, the pictures of the crypt; on the right, the CAD model of the crypt.

2.3. Finite element model analyses (DIDA – UNIFI)

The section presents the various numerical models developed and the simulations performed throughout the study. In particular, three-dimensional FEMs were implemented to represent the main architectural and structural configuration that the building has undergone over time, referring both to the original phase as the Church of San Pancrazio (*church FE model*) and to its subsequent conversion into a museum (*museum FE model*). These models were developed on the basis of the preliminary information collected during the geometric and structural survey phase (see section 2.2 Survey (DIDA – UNIFI)), which provided preliminary information about the dynamic characteristics and seismic response of the structure.

Subsequently, following more in-depth investigative activities, an additional numerical model of the museum was developed (*prior detailed museum FE model*), in which specific structural elements not previously considered were explicitly included, such as the reinforced concrete ring beam at roof level



and the steel roof trusses. This modelling phase allowed for a more accurate representation of the global structural behaviour of the building.

Furthermore, on the basis of the results obtained from the dynamic identification campaign, the museum FEM was further updated through a calibration process of the stiffness properties. This procedure led to the development of the so-called *posterior detailed museum* FE model, which exhibits a structural response more consistent with the experimentally observed dynamic behaviour.

Finally, following a critical assessment of the results obtained from the analyses carried out on the global models, namely modal analyses and nonlinear dynamic analyses, a more refined numerical model of the main façade alone was developed. This decision was motivated by the fact that the façade was identified as the most vulnerable structural element of the entire complex. Nonlinear static (pushover) analyses were therefore performed on this structural sub-system, considering both the prior and posterior configurations, in order to further investigate its behaviour and assess the effectiveness of the considered interventions.

All numerical models and related analyses were grouped within a single section, with the aim of highlighting the relevance of this crucial phase of the study. This approach also avoids a purely chronological and fragmented presentation of the modelling activities across different sections, instead providing a coherent and comparative overview of the modelling evolution and the results obtained.

2.3.1 Numerical modelling

The numerical modelling was conducted using the FE environment Abaqus CAE © [3]. The three-dimensional CAD model was imported into the “Part” module using the IGES format and subsequently meshed in Abaqus for numerical simulations. The partition was based on the material (masonry, reinforced concrete, steel) of each structural component and its structural role (column, beam, slab) within the overall behaviour of the structure.

Three-dimensional solid elements were employed to simulate masonry walls, slabs, columns, and beams. A homogeneous macro-modelling approach was adopted, representing masonry as a continuum material.



In this strategy, units, mortar, and the unit–mortar interface are smeared out in a homogeneous continuum. This procedure is preferred for the analysis of large-scale masonry structures, while it is not suitable for the detailed stress analysis of a small masonry panel, due to the difficulty of capturing all its expected local failure modes.

Initially, two numerical models were developed to assess the influence of past structural modifications on the building's global behaviour. The first model (*church*) represents the structure as it existed before the 18th-century renovations, while the second (*museum*) incorporates all subsequent interventions up to the 1970s. These include the insertion of reinforced concrete slabs and walkways at the different levels, which reduce the nave's original free height, as well the steel beams and columns added during the tobacco factory phase and the 20th-century interventions carried out by architects Sacchi and Papi.

The first FE model consists of 244351 nodes, 132740 quadratic tetrahedral elements (C3D10), the second model comprises 646871 nodes, 375028 quadratic tetrahedral elements (C3D10).

Figure 16 presents a three-dimensional view of the mesh representation of both models, using different colours to distinguish materials.

In order to improve the reliability and the computational efficiency of finite element solutions, a mesh refinement around openings and vaults was made up to 0.3 and 0.15 m (Figure 17).

The crypt level was excluded from the finite element model. It was found that the presence of the basement level, either modelled as a solid volume or idealised as a set of springs with equivalent stiffness, had no significant influence on the overall structural response.



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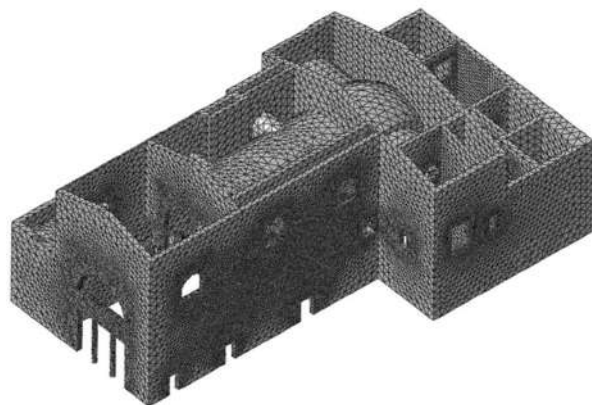
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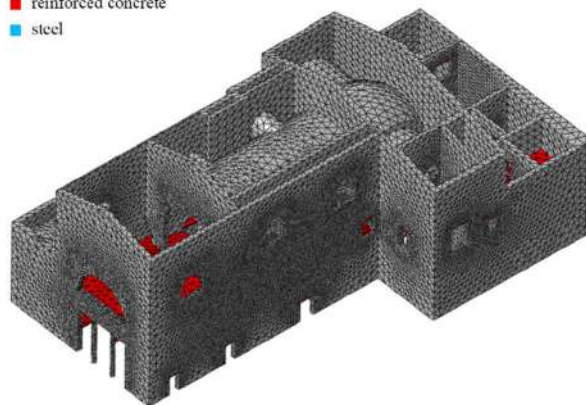
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■ masonry



a)

■ masonry
■ reinforced concrete
■ steel



b)

Figure 16. Mesh representation of both models: a) the "church" model, b) the "museum" model.

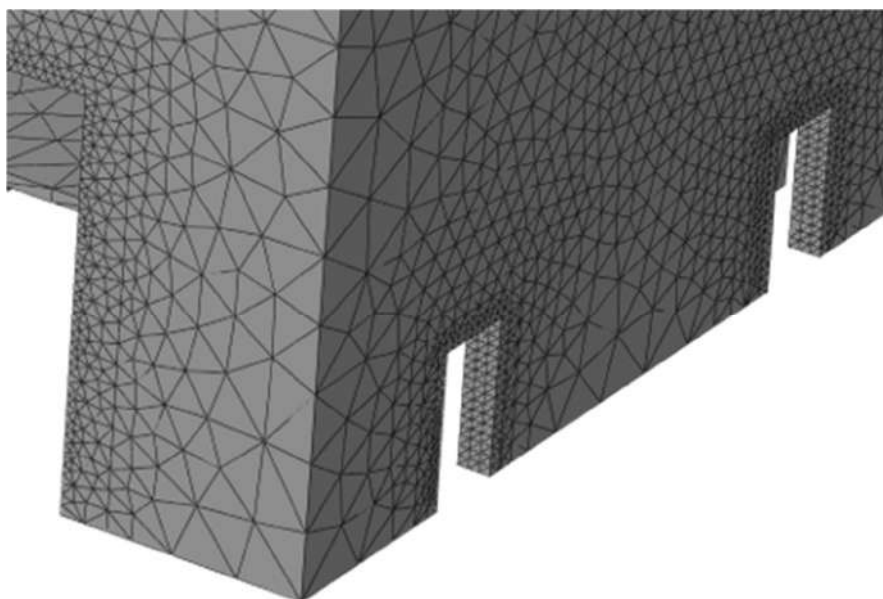


Figure 17. Mesh refinement around openings, vaults, and beams.

Due to the heritage significance of the museum, destructive or semi-destructive material testing methods, were not permitted. As a non-invasive alternative, a limited thermography campaign was conducted to



check on the presence of concrete ring beams at the roof level of central nave, as to get insights on discontinuities hidden by the plaster layer.

The thermographic investigation, however, did not reveal any conclusive evidence of reinforced concrete elements in the uppermost portion of the structure.

The existence of a concrete ring beam was confirmed exclusively through a direct visual inspection carried out above the vault at the top level of the building, an area accessible only to authorized museum personnel. This inspection revealed the presence of a continuous reinforced concrete ring beam, approximately 40 cm in height, extending along the entire perimeter of the ex San Pancrazio church.

In light of this finding, the second numerical model (*museum*), initially developed to represent the current structural configuration, was updated to include the newly identified structural elements. Specifically, the model was modified to incorporate both the reinforced concrete ring beam and the steel trusses, the latter represented using beam-type finite elements with appropriate mechanical properties.

The resulting finite element model, hereinafter referred to as the *prior detailed museum*, consists of 747606 nodes and 435679 ten-node quadratic tetrahedral elements (C3D10). This refined model enables a more accurate representation of the actual structural system and provides a robust basis for comparative analyses. The model is defined as *detailed* due to its ability to provide an accurate representation of the actual structural system. It is referred to as *prior* because it represents the reference numerical model that will be subsequently updated on the basis of the results of the dynamic monitoring campaign, as discussed in the following sections, with particular emphasis on the calibration of the masonry elastic modulus. This refined model provides a robust basis for comparative analyses.

Figure 18 presents a three-dimensional view of the models and their meshed representations.

The availability of three distinct numerical models allows for the execution of modal analyses aimed at evaluating the effects of the newly introduced structural components on the global dynamic response of the building. This comparative framework facilitates the assessment of changes in modal shapes, stiffness distribution and fundamental frequencies induced by the structural modifications.



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Figure 19 shows the axonometric sections of the *prior detailed museum FEM* at different elevations. It can be observed that, despite the necessary simplifications and regularizations, the model accurately reproduces all the structural elements: the floor slabs and cantilever walkways, the cross and barrel vaults, the round and pointed arches, the dome, and the I-beams.

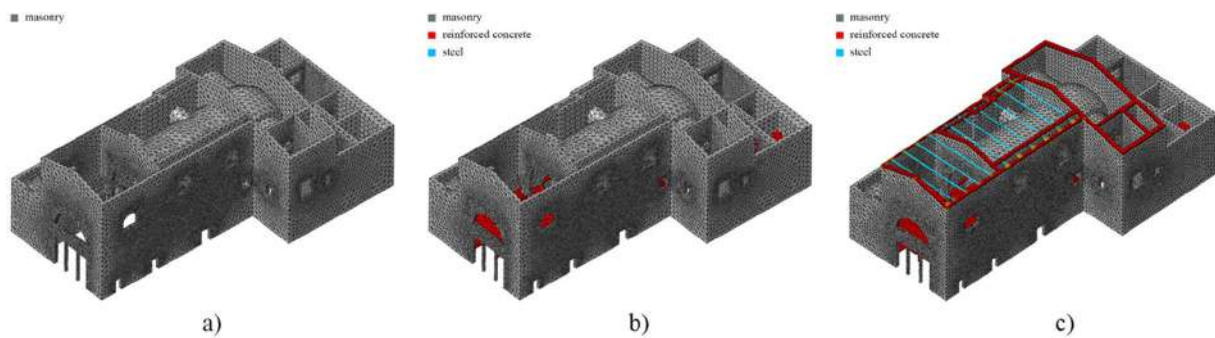


Figure 18. Mesh configuration of the three numerical models: a) church FE model, configuration before 18th- century interventions; b) museum FE model, layout of the museum prior to visual survey; c) prior detailed museum FE model, current configuration of the museum.



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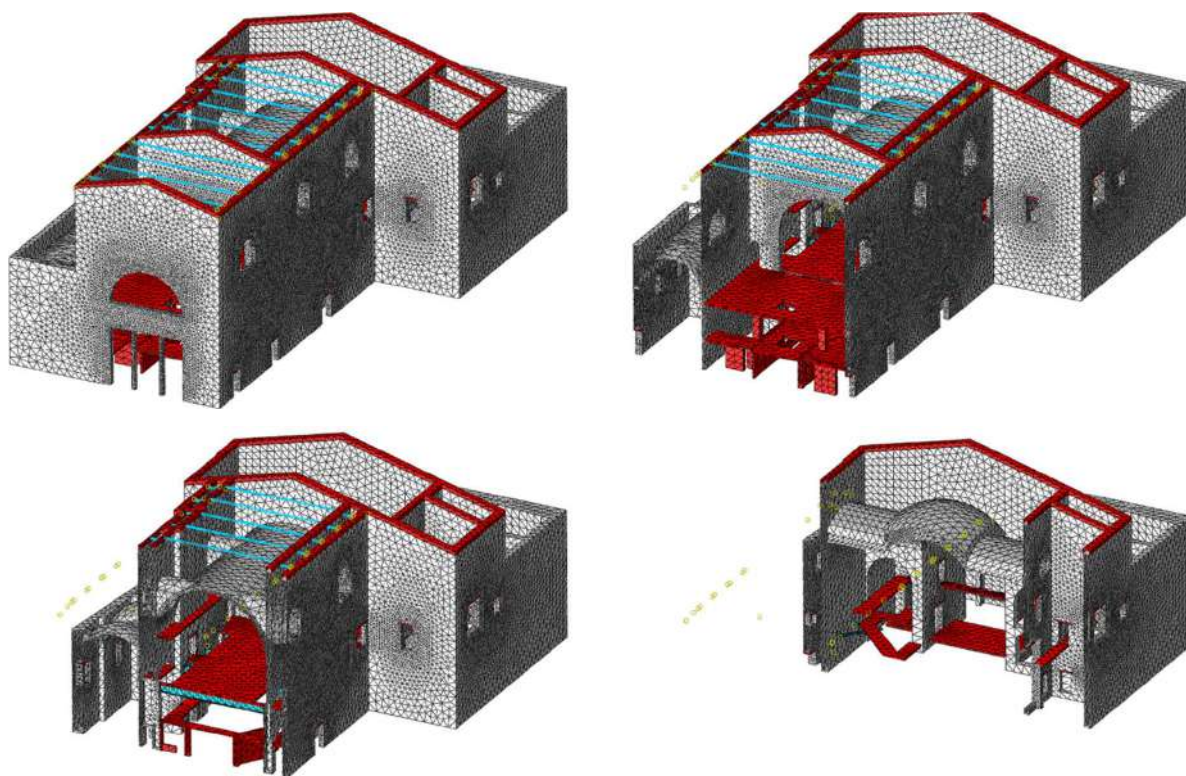


Figure 19. Axonometric sections of the prior detailed museum FEM.

2.3.2 Mechanical properties

Due to the heritage significance of the museum, destructive or semi-destructive material testing methods, were not permitted.

Hence the mechanical properties used for the analyses are chosen according to the Italian Recommendations (NTC 2018) and, for missing parameters, validated values from available scientific literature were used. The main mechanical properties adopted in this work are reported in Table 1.

In particular, for the masonry material, reference was made to Table C8.5.I of the NTC 2018, which provides reference values for strength, elastic modulus, and density for various types of existing masonry. In our case, the selected type was “masonry with roughly squared stones and irregular wall thickness”. The elastic modulus was assumed as the average value within the range provided by the code (Figure 20).



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The nonlinear behaviour of the masonry is outlined in the following section.

Table 1. Mechanical parameters of the materials involved.

	ρ	ν	E
	[kg/m ³]	[-]	[MPa]
Masonry	2000	0.15	1230
R.C.	2500	0.2	31500
Steel	7850	0.3	210000

Tipologia di muratura	f (N/mm ²)	τ_0 (N/mm ²)	f_{v0} (N/mm ²)	E (N/mm ²)	G (N/mm ²)	w (kN/m ³)
	min-max	min-max		min-max	min-max	
Muratura in pietrame disordinata (ciottoli, pietre erratiche e irregolari)	1,0-2,0	0,018-0,032	-	690-1050	230-350	19
Muratura a conci sbozzati, con paramenti di spessore disomogeneo (*)	2,0	0,035-0,051	-	1020-1440	340-480	20
Muratura in pietre a spacco con buona tessitura	2,6-3,8	0,056-0,074	-	1500-1980	500-660	21
Muratura irregolare di pietra tenera (tufo, calcarenite, ecc.,)	1,4-2,2	0,028-0,042	-	900-1260	300-420	13 ÷ 16(**)
Muratura a conci regolari di pietra tenera (tufo, calcarenite, ecc.,) (**)	2,0-3,2	0,04-0,08	0,10-0,19	1200-1620	400-500	
Muratura a blocchi lapidei squadriati	5,8-8,2	0,09-0,12	0,18-0,28	2400-3300	800-1100	22
Muratura in mattoni pieni e malta di calce (***)	2,6-4,3	0,05-0,13	0,13-0,27	1200-1800	400-600	18
Muratura in mattoni semipieni con malta cementizia (es., doppio UNI foratura ≤40%)	5,0-8,0	0,08-0,17	0,20-0,36	3500-5600	875-1400	15

Figure 20. Reference values of masonry mechanical parameters from Table C8.5.1 of NTC 2018.



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2.3.3 Pre-interventions (San Pancrazio church)

Modal analyses were conducted on the three models to: i) identify the main frequencies and modal shapes of the investigated building; ii) understand the effects of historical interventions on the dynamic response of the museum; iii) get insights on the most-likely damage mechanism of the building.

Regarding the *church* model, which is composed exclusively of masonry and lacks horizontal diaphragms, the first frequency involves the main façade overlooking Piazza San Pancrazio and its orthogonal walls toward an out-of-plane mode shape (Figure 21).

In Table 2 the main frequencies for the *church* FE model are reported, assuming the mechanical parameters reported in the Table 1.

Table 2. First ten natural frequencies of the church FE model.

Church FEM	
Natural frequencies [Hz]	
1	1.418
2	1.679
3	1.845
4	2.362
5	2.419
6	2.551
7	2.629
8	2.728
9	2.770
10	3.032



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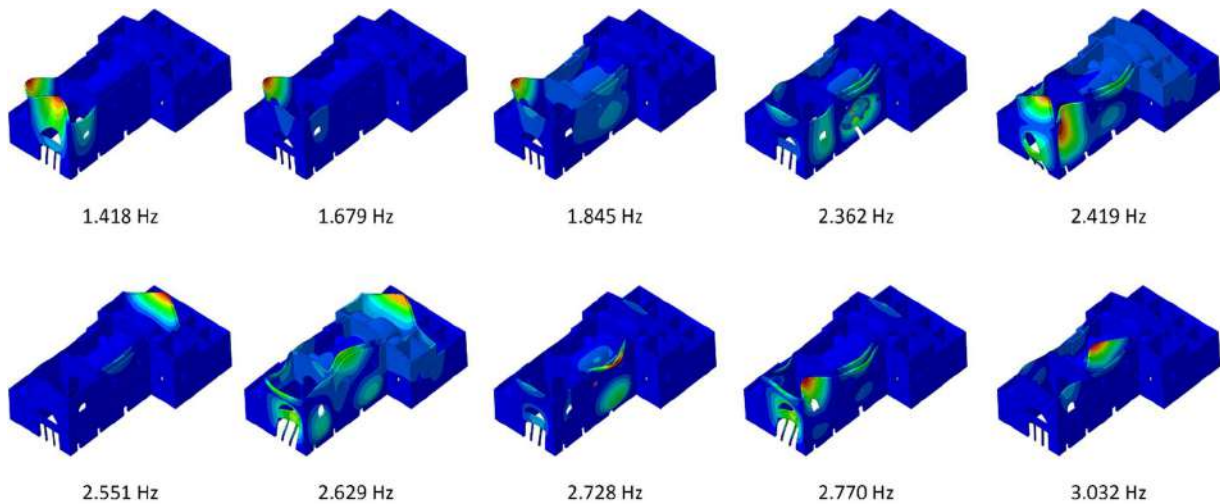


Figure 21. First ten mode shapes of the church model.

2.3.4 Post-interventions (Marino Marini Museum)

For the first two models, *church* and *museum*, the first frequency of the building affects the main facade and its orthogonal walls towards an out-of-plane mode shape. Further modes are given by the inner addition of slabs, altering the dynamic response of the longitudinal walls of the central nave (Figure 22). These insertions lead to a lack of correspondence between the mode shapes of the two models.

In Table 3 the main frequencies for the *museum* model are reported.



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Table 3. First ten natural frequencies of the museum FE model.

Museum FEM	
Natural frequencies [Hz]	
1	1.486
2	1.786
3	2.147
4	2.650
5	2.739
6	2.891
7	3.087
8	3.220
9	3.298
10	3.499



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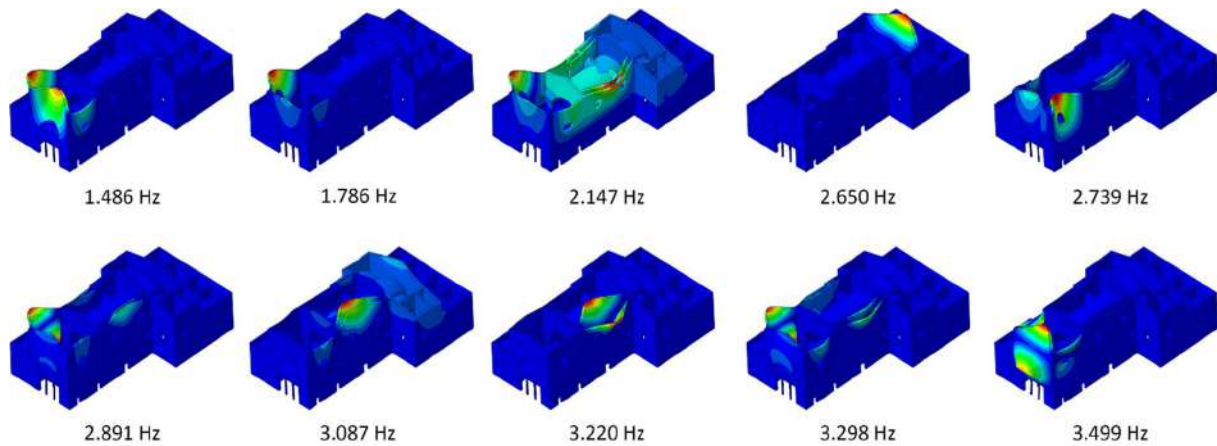


Figure 22. First ten mode shapes of the museum FE model.

An analysis of the *prior detailed museum* FEM, which includes the presence of the concrete ring beam and steel trusses, reveals a significant change in the structure's first mode of vibration. The introduction of these structural elements substantially alters the global dynamic behaviour of the museum, increasing its overall stiffness and modifying the primary deformation mechanism.

In particular, the out-of-plane displacement of the main façade is no longer the dominant mode shape. Instead, the longitudinal walls of the nave exhibit a more active contribution to the dynamic response (Figure 23). This redistribution of modal deformations results in a notable increase in the fundamental frequency, which rises from 1.486 Hz (associated with the previous configuration) to 2.292 Hz in the updated model. In Table 4 the main frequencies for the *prior detailed museum* FE model are reported.



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Table 4. First ten natural frequencies of the prior detailed museum FE model.

Prior detailed museum FEM	
Natural frequencies [Hz]	
1	2.292
2	2.547
3	2.908
4	3.214
5	3.315
6	3.848
7	4.212
8	4.498
9	4.587
10	4.596

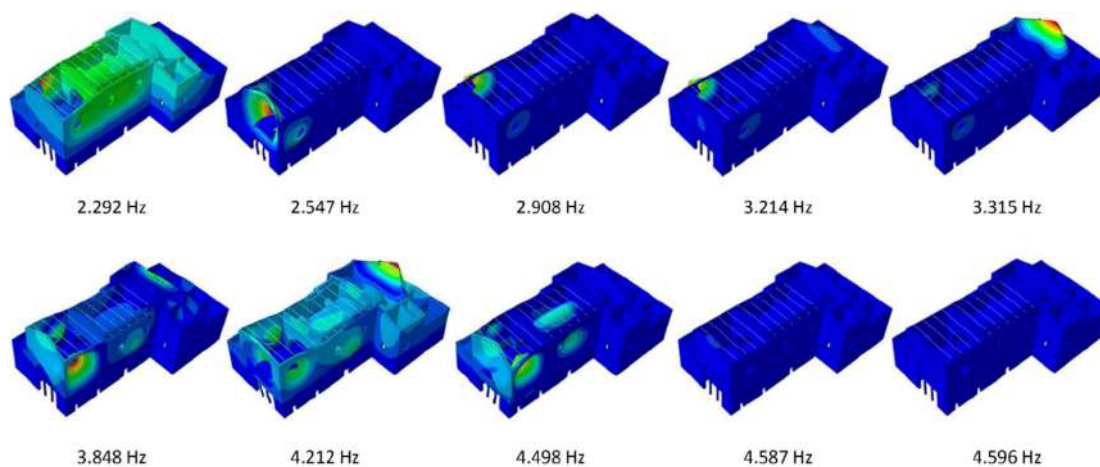


Figure 23. First ten mode shapes of the prior detailed museum FE model.



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The results of the modal analysis were crucial not only for understanding the influence of the newly introduced structural components, but also for informing the design of the subsequent experimental dynamic identification campaign. In particular, the information obtained from the mode shapes guided the configuration of the test setup, allowing for a rational placement of accelerometric sensors within the structure to effectively capture the expected modal deformations.

In Table 5 the main frequencies for the three models are reported.

Figure 24 presents a comparison of the first mode shape across the three numerical models.

Table 5. First ten natural frequencies of the three FE models.

	Church	Museum	Prior detailed museum
1	1.418	1.486	2.292
2	1.679	1.786	2.547
3	1.845	2.147	2.908
4	2.362	2.650	3.214
5	2.419	2.739	3.315
6	2.551	2.891	3.848
7	2.629	3.087	4.212
8	2.728	3.220	4.498
9	2.770	3.298	4.587
10	3.032	3.499	4.596



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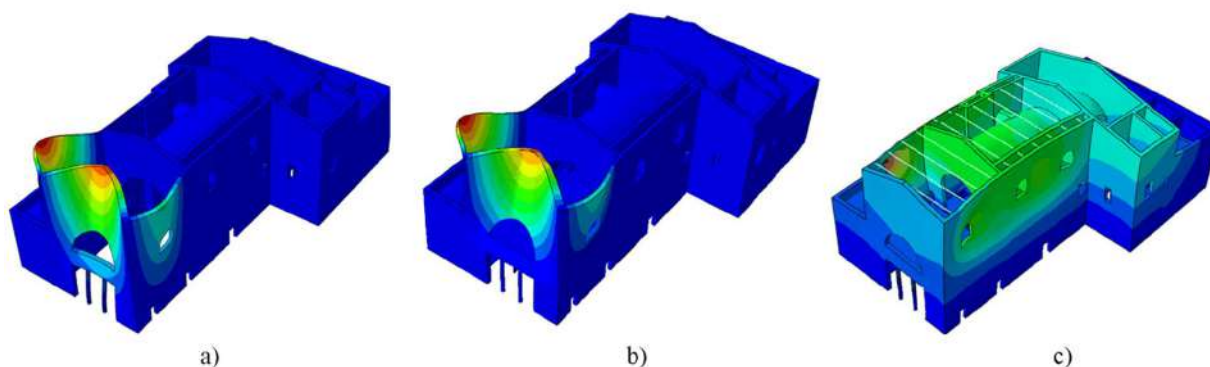


Figure 24. First modal shape of the three FE models: a) church FEM, b) museum FEM, c) prior detailed museum FEM.

2.3.5 Record selection

Figure 25 shows the two spectra considered in the analyses. For the definition of the demand, a return period of 50 years and an important class III were accounted, leading to a nominal period of 75 years. The associated return periods to the limit states are 75 years for SLD and 712 years for SLV. Given the microzoning information available in the detailed studies carried out for the municipality of Florence and treated in section 2.2.2, a very high value of S was determined. In this research, the results of the nonlinear time-history analyses will be used solely to gain insights into the possible collapse mechanisms of the structure. Therefore, a soil class B in a planar condition has been assumed corresponding to a common value of S (equal to 1.2).



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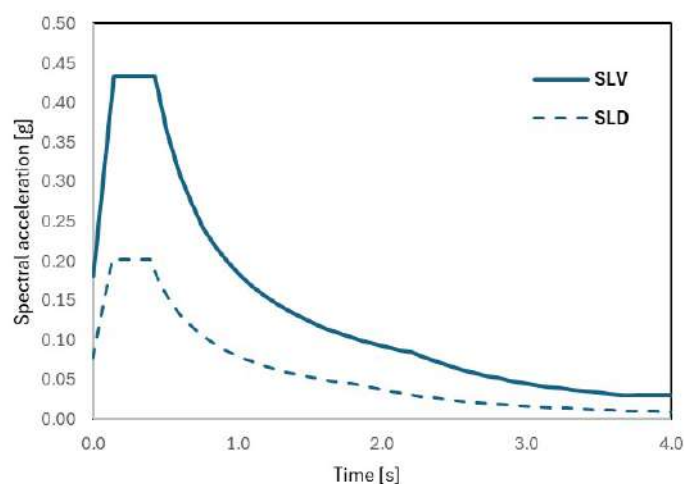


Figure 25. Demand spectra for the Marini Museum.

A series of accelerogram records for dynamic analyses have been selected with reference to the Marini Museum. Complexly, 14 distinct records coming from two distinct samples of 7 accelerograms have been identified (Figure 26). The first seven are provided by the Tuscany Region, which lists a sample of spectrum-compatible records for the various Tuscan municipalities. These accelerograms have been selected through a partnership with the University of Pavia (Department of Civil Engineering and Architecture) adopting the SCALCONA 3.0 software (SCALing of Compatible Natural Accelerograms). The final 7 records have been selected adopting the REXEL-web tool [4] matching the demand spectrum for the city of Florence.



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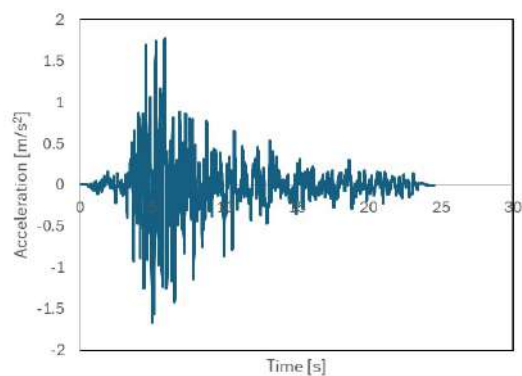
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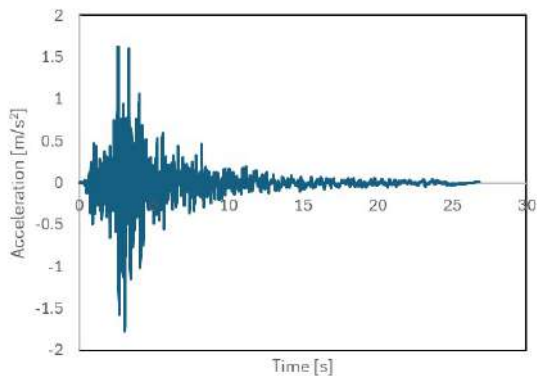
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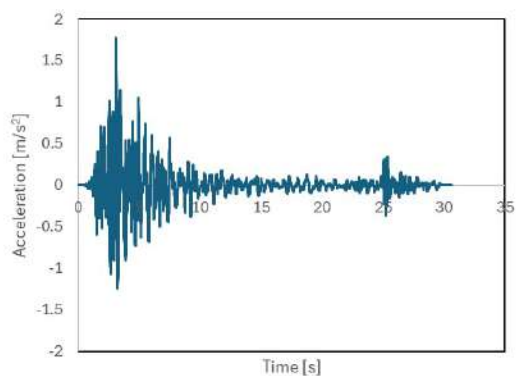
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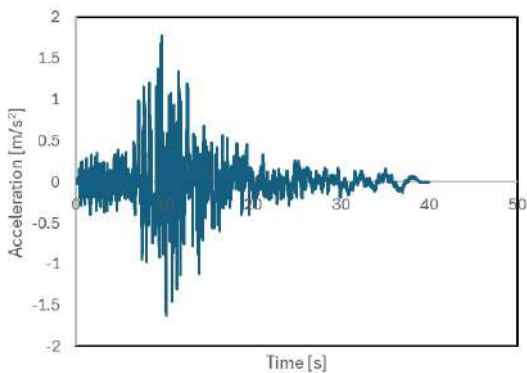
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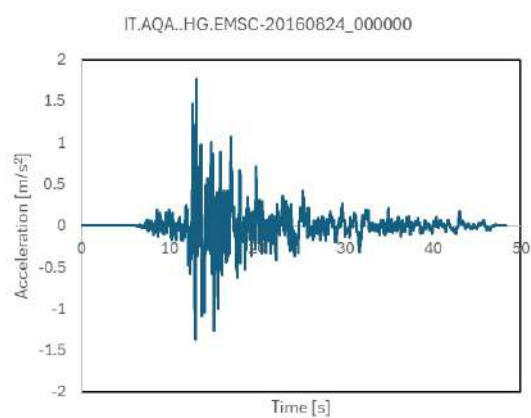
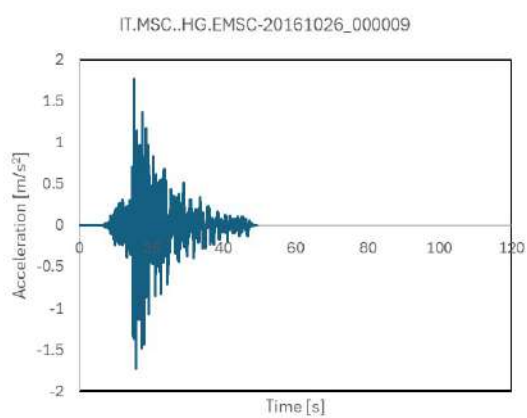
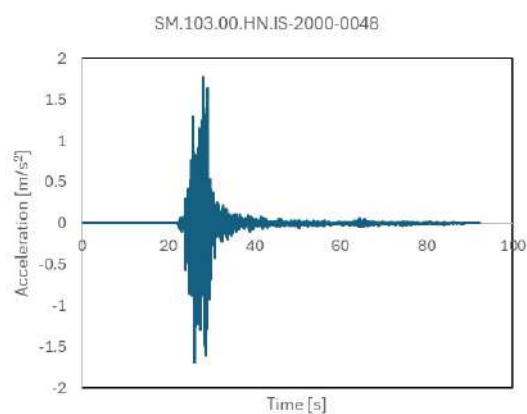
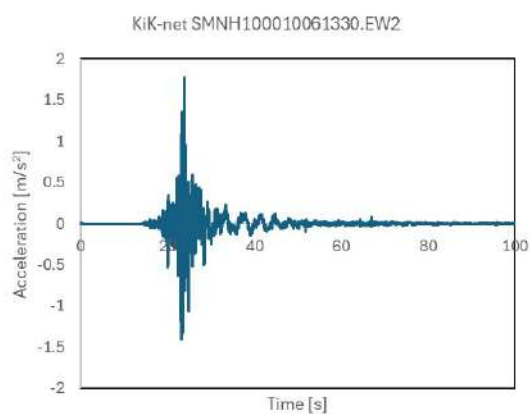
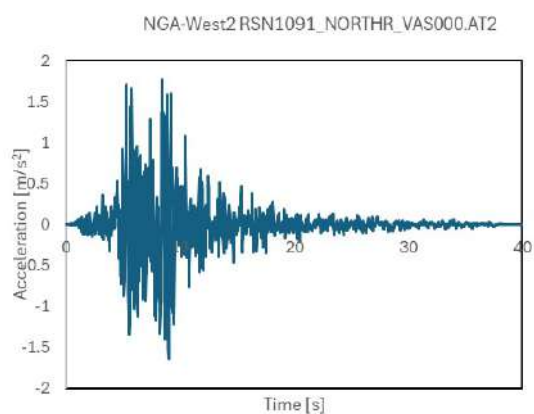
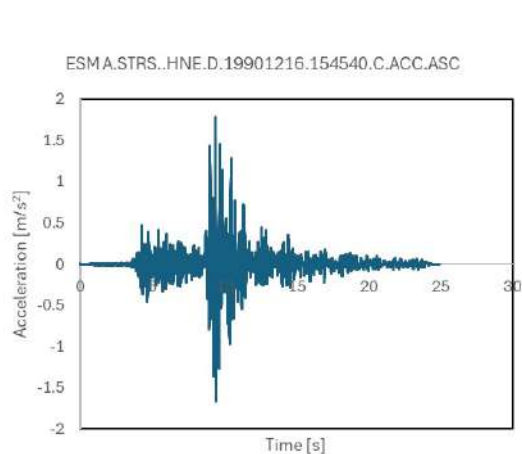
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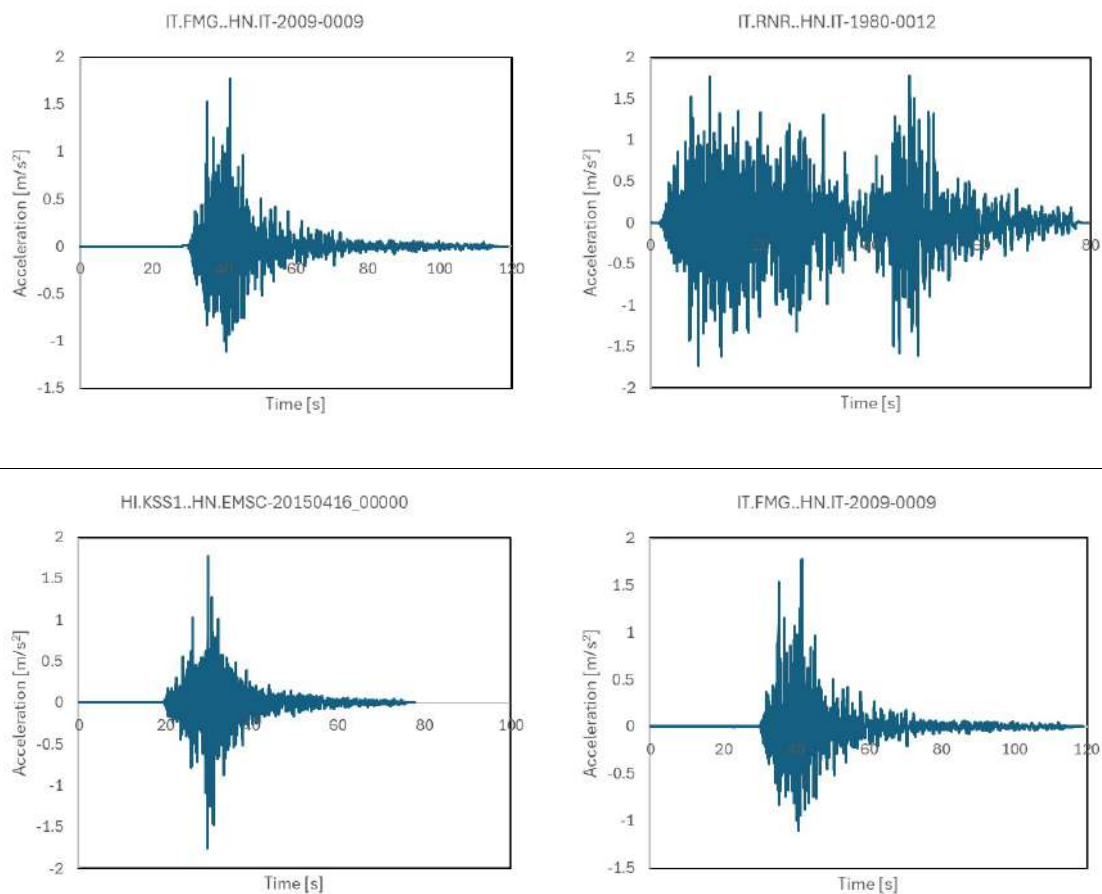


Figure 26. Accelerograms spectrum-compatible selected for the Museum according to the SCALCONA software and REXEL.



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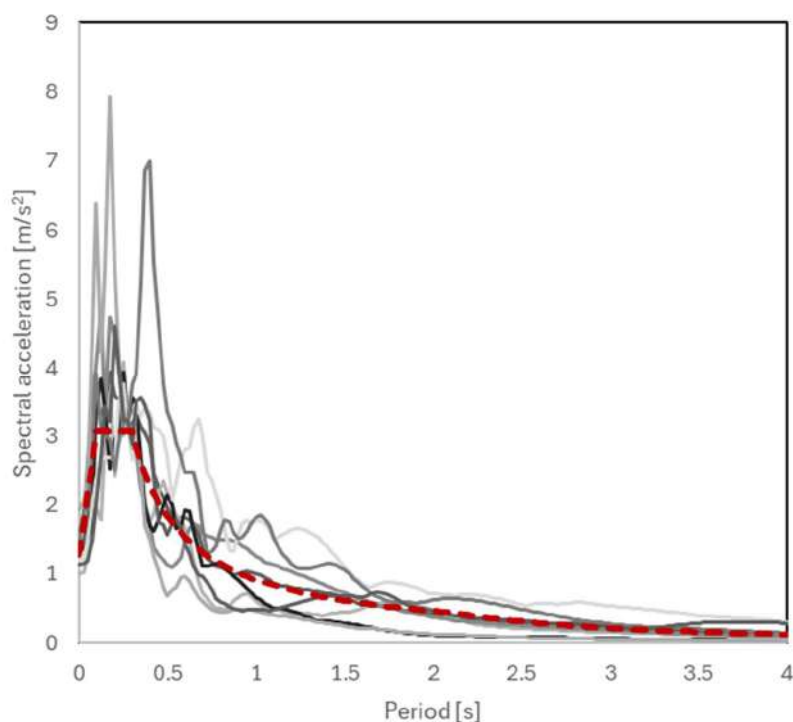


Figure 27. Acceleration response spectra at 5% viscous damping as a function of the structural period. Grey curves represent the response spectra obtained from the individual ground-motion records, while the solid black line denotes their mean spectrum. The red dashed line indicates the target response spectrum.

2.3.6 Nonlinear dynamic analyses: prior FEM

Nonlinear time-history analyses were executed on the FE model representing the current configuration of the museum (without ring beam and steel trusses). To this aim, a nonlinear behaviour was defined to characterize the masonry elements through the Concrete Damage Plasticity Model (CDP) [5]. Although originally defined to describe the behaviour of concrete elements, the latter has been widely used to investigate masonry structures [6] [7]. CDP is based on a plasticity-damage relation through two distinct tensile and compressive constitutive curves. The maximum compressive value was assumed equal to 2 MPa, according to the values available in MIT2019 [8] for the considered masonry typology (Rough block

masonry with non-homogeneous thickness of the external faces). For the tensile strength, the value was considered equal to 1/10 of the compressive one. The nonlinear curves describing the two constitutive laws (Figure 28) were defined according to [9]. In Table 6 the mechanical parameters adopted in the CDP model.

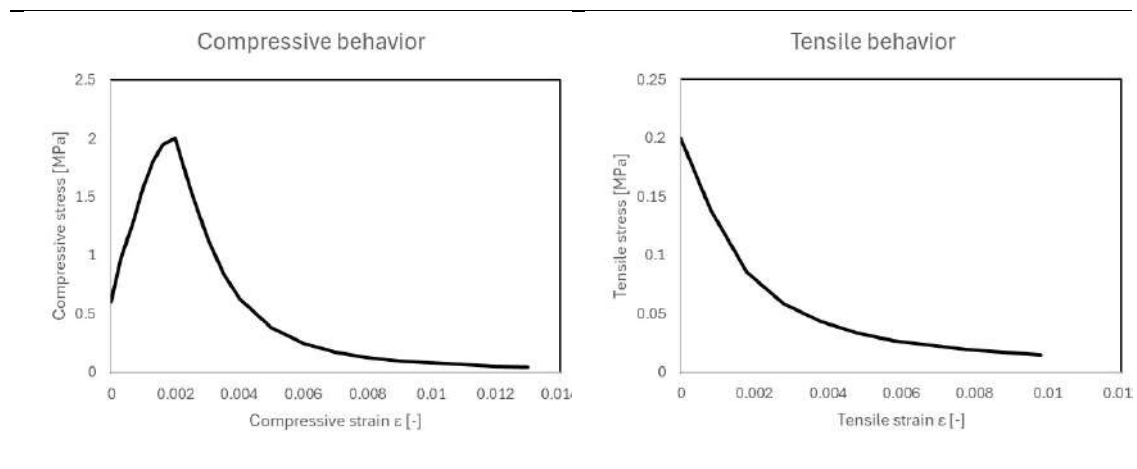


Figure 28. Concrete Damage Plasticity model: on the left compressive behaviour, on the right tensile behaviour.

Table 6. Mechanical properties of the CDP model used in the numerical procedure.

Dilation angle	Eccentricity	f_{b0}/f_{c0}	Kc	Viscosity parameter
37°	0.1	1.16	0.667	0.001

The load was given along the Y direction of the model (parallel to the main façade) applying the 30% of its amplitude along the X direction. Prior to the application of the seismic action, gravity loads were applied by a nonlinear incremental procedure. During the analysis, different control points of the building were considered.



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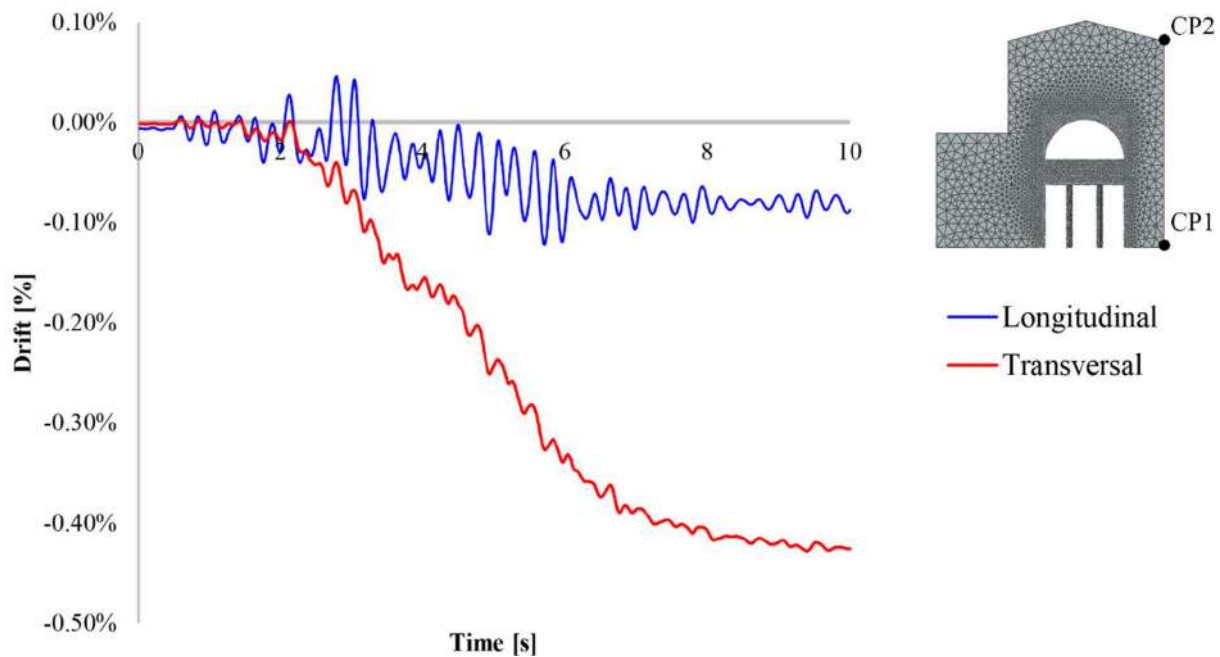


Figure 29. Non-dimensional drifts for the analysis. On the right, the two control points on the main façade.

The outcomes of the dynamic analyses highlight a relevant damage suffered by the building. Although the main seismic input was given along the longitudinal direction (Y axis), the building exhibited an out-of-plane failure of the façade. This outcome is coherent with the insights provided by modal analysis, and in line with the multiplier activating the mechanism calculated according to kinematic analysis. As a matter of fact, Figure 29 shows the results of the nonlinear dynamic simulation with the accelerogram selected from REXEL web tool in terms of non-dimensional drift, considering both the out-of-plane direction (X) and the in-plane (Y) directions of the building façade. The results refer to the two control points, CP1 and CP2, located at the base and the top of the main façade of the museum, respectively. The drift in the X direction (transversal) is significantly higher than the one in the Y direction (longitudinal). The maximum drift recorded in the transversal direction is 0.43%, whereas in the longitudinal direction it reaches a peak value of 0.12%.



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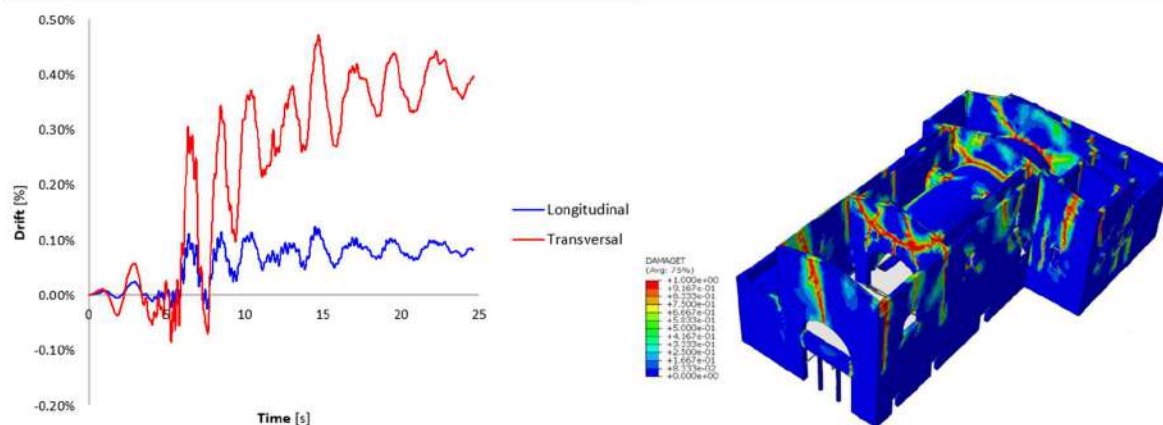
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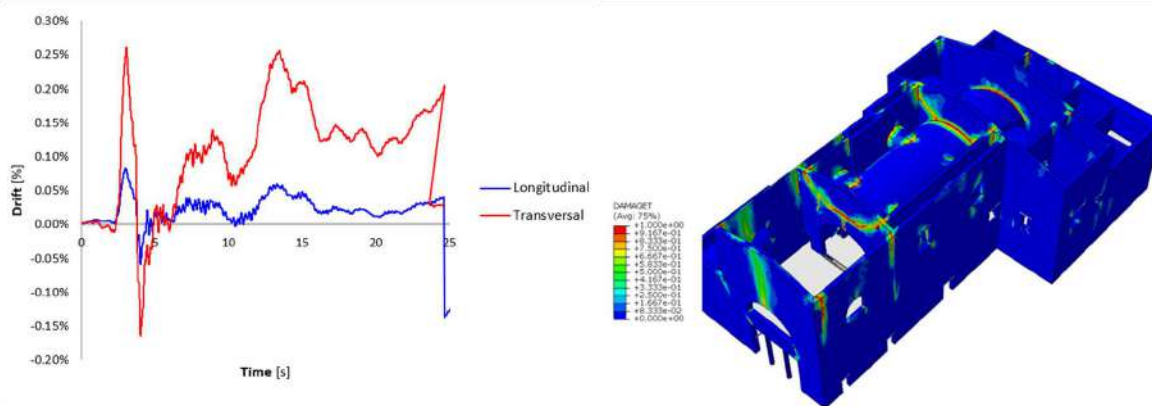
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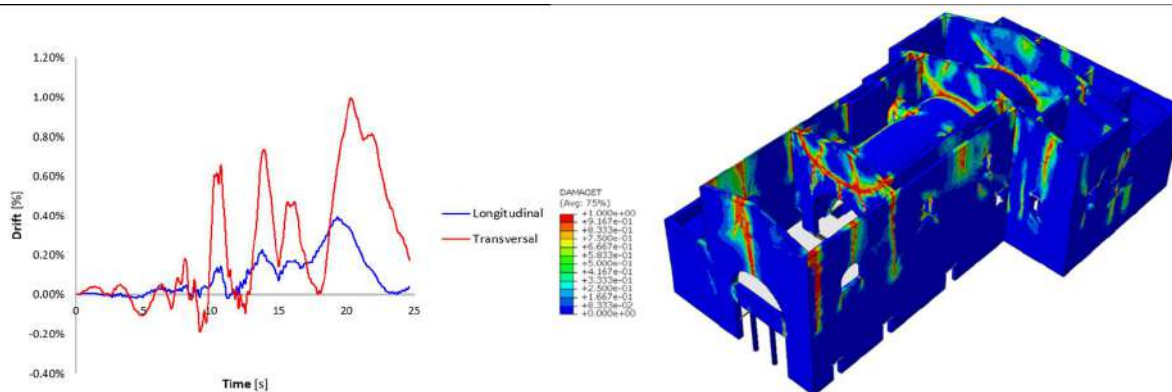


Figure 30. Results from nonlinear dynamic analyses.

In Figure 30, the results of the nonlinear dynamic analyses performed with the three accelerograms selected from the SCALCONA web tool are presented. The left side of the figure reports the structural response in terms of non-dimensional drift, while the right side illustrates the corresponding damage scenarios.

Despite the main seismic input applied along the longitudinal direction, the building exhibited an out-of-plane failure of the façade.

With reference to the damage evolution, the maximum transversal drift reached 0.47% at 14.69 s for the first accelerogram, 0.26% at 13.37 s for the second and 0.98% at 21.30 s for the third.

In all cases, the cracking pattern clearly shows the onset of a fracture that would lead to the detachment of the main façade. The tensile damage distribution is comparable across the three simulations, although in the second case the cracks appear less widespread. Overall, the most affected components are the main façade, the vault, the dome, and the triumphal arch.

2.3.7 Updated FEM (posterior detailed museum FE model)

The final updating of the numerical model was performed on the basis of the results obtained from the dynamic experimental tests. For the sake of clarity, the main outcomes of the dynamic tests are

summarized in the following, while a detailed description of the experimental setup, measurement procedures, and data processing is provided in the following section, although the chronological order of the research activities would require the experimental tests to be discussed first.

The experimental measurements have highlighted the following outcomes:

- The FE model was able to capture the expected behaviour of the building, exhibited in terms of mode shapes, confirming the good reliability of assumptions made during the modelling phase;
- The Elastic modulus of the masonry walls, preliminary assumed according to the listed values reported in [10], proved to be underestimated, as the real structures exhibited higher stiffness than the numerical model.

Based on these results, a probabilistic model updating procedure was undertaken to develop a more reliable numerical model for the subsequent seismic analyses. In particular, the elastic modulus of the masonry was recalibrated on the basis of the experimental evidence derived from in situ dynamic identification tests.

The *prior detailed museum* FE model was characterized by a fundamental frequency equal to 2.29 Hz, which was significantly lower than the experimentally observed value. The on-site dynamic identification campaign determined a fundamental natural frequency of 2.90 Hz, clearly indicating the need for a stiffness recalibration, with specific attention to the elastic modulus of the masonry.

The initial value of the elastic modulus, set equal to 1230 MPa, had been assumed as an intermediate reference value in accordance with current technical standards. A relatively poor masonry typology was considered, namely “masonry with roughly squared stones and irregular wall thicknesses”. However, in order to achieve a better agreement between numerical predictions and experimental observed dynamic behaviour, the elastic modulus was subsequently increased to 2050 MPa.

With this updated value, the recalibrated numerical model (*posterior detailed museum* FE model) reproduced a fundamental frequency of 2.90 Hz, showing excellent agreement with the experimental results (Figure 31). Table 7 reports the main frequencies of the updated model, obtained by adopting the mechanical parameters listed in Table 1.

The updating of this mechanical property significantly improved the reliability of the model, making it more representative of the actual dynamic response of the structure.

Figure 32 presents a comparison of the first mode shape across the four numerical models.

Table 7. First ten fundamental frequencies of the posterior detailed museum FE model.

Posterior detailed museum FEM	
1	2.8998
2	3.1039
3	3.6961
4	3.958
5	4.0927
6	4.7156
7	4.8088
8	5.3031
9	5.5235
10	5.5437



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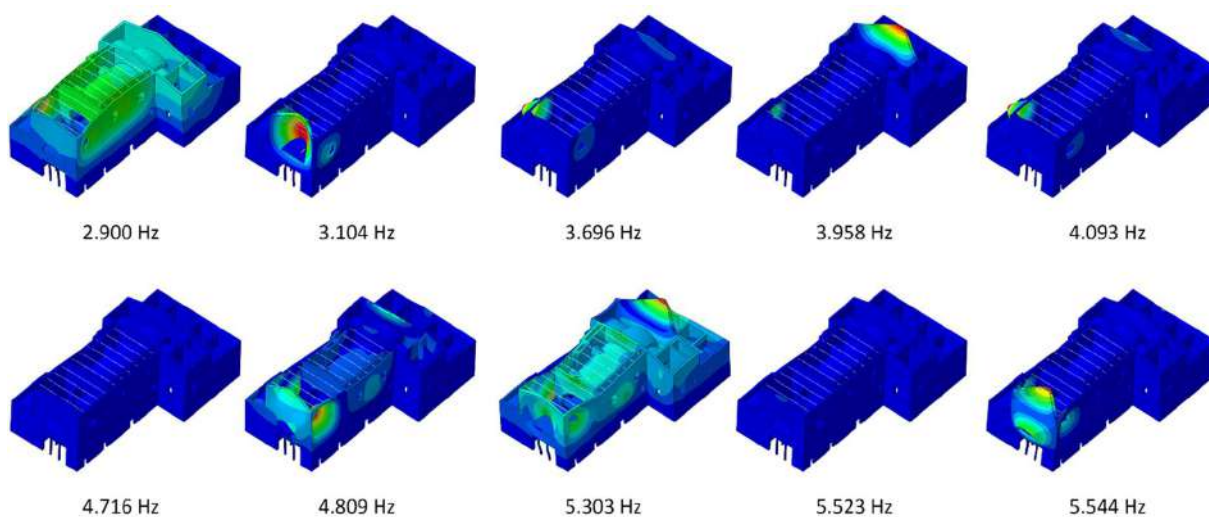


Figure 31. First ten mode shapes of the posterior detailed museum FE model after dynamic identification.

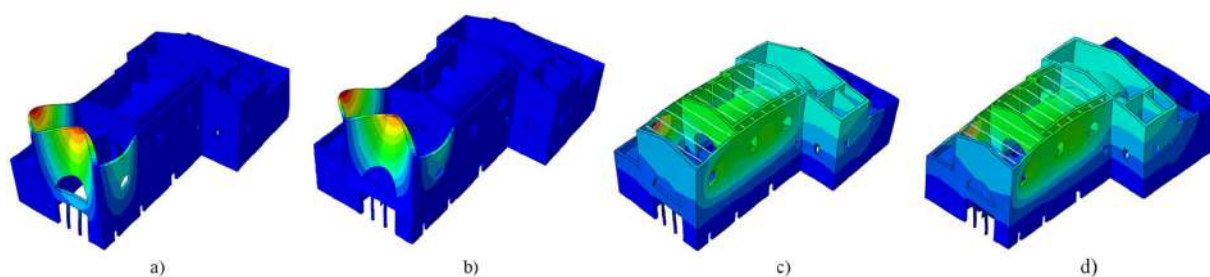


Figure 32. Comparison of the first mode shape of the four models: a) church FE model; b) museum FE model; c) prior detailed museum FE model; d) posterior detailed museum FE model.

2.3.8 Nonlinear static analysis: façade prior and posterior FEM

Modal analyses on the entire museum complex highlighted that the most vulnerable structural component is the main façade.

This result is primarily attributable to its geometric configuration: above the top floor slab, located below the large arch opening, no intermediate diaphragms are present up to the level of the trussed roof, for a total height of 9.60 m. This unrestrained vertical portion, lacking transverse restraints, leads to a markedly flexible response, exposing the façade to a high risk of out-of-plane instability.

As previously discussed, the expected failure mechanism is analogous to those typically observed in historical ecclesiastical buildings, where tall and slender masonry walls develop out-of-plane collapse mechanisms in the absence of efficient transverse connections and continuous floor diaphragms.

To gain deeper insight into this response, a dedicated finite element model was developed, comprising exclusively the main façade facing Piazza San Pancrazio (Figure 33). The model faithfully reproduces the primary geometric and construction features of the structural component, namely: the arch opening (7.70 m × 3.80 m), the entrance portico consisting of two relatively slender masonry columns (diameter 0.55 m, free buckling height 6 m), and the masonry portion corresponding to the adjacent chapel.

The basement level was not included in the numerical model, in accordance with the considerations discussed in section 2.3, according to which it may be regarded as effectively confined by the surrounding soil and therefore not significant for the global structural response under investigation.



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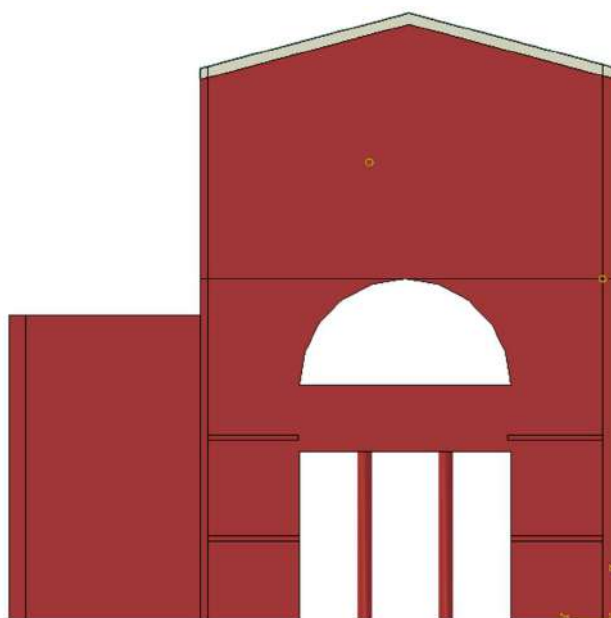


Figure 33. Façade model.

The masonry was modelled using a nonlinear constitutive law based on CDP, in order to reproduce the progressive degradation of stiffness and strength associated with the initiation and evolution of cracking and crushing mechanisms.

A reinforced-concrete ring beam, 40 cm thick, was modelled at the top of the masonry, as identified through visual inspection (section 2.3). Longitudinal and transverse steel reinforcement bars were included within the beam (Figure 34) and modelled using an elasto-plastic constitutive law, to account for their contribution to lateral stiffness and deformation capacity.

The rebars were modelled as truss elements with a cross-sectional area equal to 113 mm^2 , arranged as three top and three bottom bars, with 3 cm concrete cover. The stirrups were assigned a cross-sectional area equal to 50 mm^2 .



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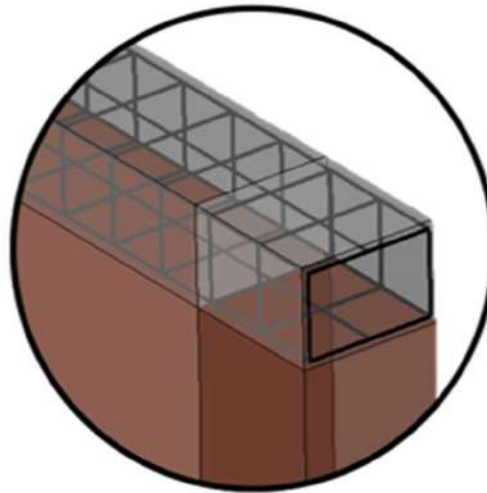


Figure 34. Detail of the concrete ring with rebars and stirrups.

A parametric variation of the masonry elastic modulus was carried out on the same model, in order to compare the structural response before and after the information obtained by the dynamic identification campaign. For clarity, in the following the model with elastic modulus initially assumed equal to 1230 MPa will be referred to as the *prior*, whereas the model with calibrated elastic modulus equal to 2050 MPa will be referred to as the *posterior*.

A nonlinear static pushover analysis was performed on the numerical model of the façade by applying a uniform lateral force distribution along the height (rectangular load pattern). The intensity of the horizontal load was incrementally increased up to a value equivalent to the gravitational acceleration g .

The base of the façade was modelled using fully fixed boundary conditions, in order to simulate the effect of full continuity with the foundations and with the basement level confined by the surrounding soil.

At the locations of transverse diaphragms (floors and walkways) and along the top reinforced-concrete ring beam, external translational constraints were introduced to prevent horizontal displacements in the x direction, i.e., the direction of the applied lateral load. These constraints reproduce the restraining effect



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of the transverse structural elements which, although not explicitly modelled, provide partial confinement against out-of-plane displacement mechanisms (Figure 35).

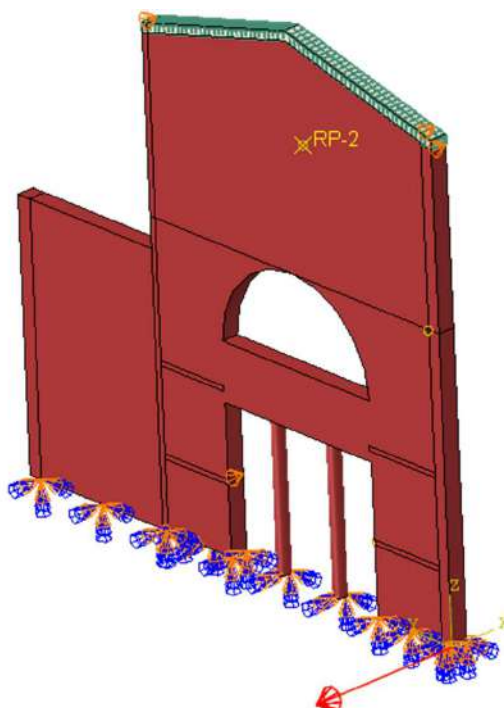


Figure 35. Application of the seismic action for pushover analysis.

In the *prior* model, the pushover analysis terminates due to numerical instability when the applied horizontal load reaches 20% of g . This value may be roughly interpreted as an upper bound of the façade capacity when adopting the preliminary mechanical parameters derived from the Italian technical code NTC 2018.

The distribution of horizontal displacements along the x -axis, i.e., the direction of application of the lateral force, shows that the central portion of the façade, between the arch and the roof ridge, reaches a peak displacement of 2.76 cm (Figure 36).

To improve visualization of the deformed configuration, a magnification factor equal to 30 was applied.



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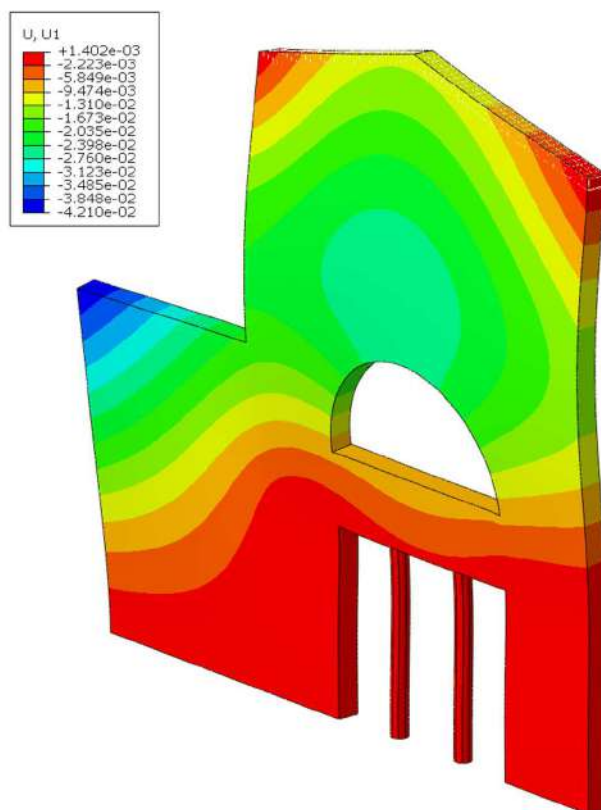


Figure 36. Results for the prior model.

For the *posterior* model, characterized by the elastic modulus calibrated following the experimental campaign, the pushover analysis terminates when the applied horizontal load reaches 27% of g , indicating an increase in capacity of 7% compared to the *prior* model. In this case as well, the model was analysed in isolation, without using equivalent springs to represent the stiffness of the orthogonal walls.

The maximum displacement in the central portion reaches 2.80 cm, corresponding to an increase of 0.04 cm compared to the uncalibrated model (Figure 37).

Similarly, to facilitate qualitative visualization, a deformation factor of 30 was applied.



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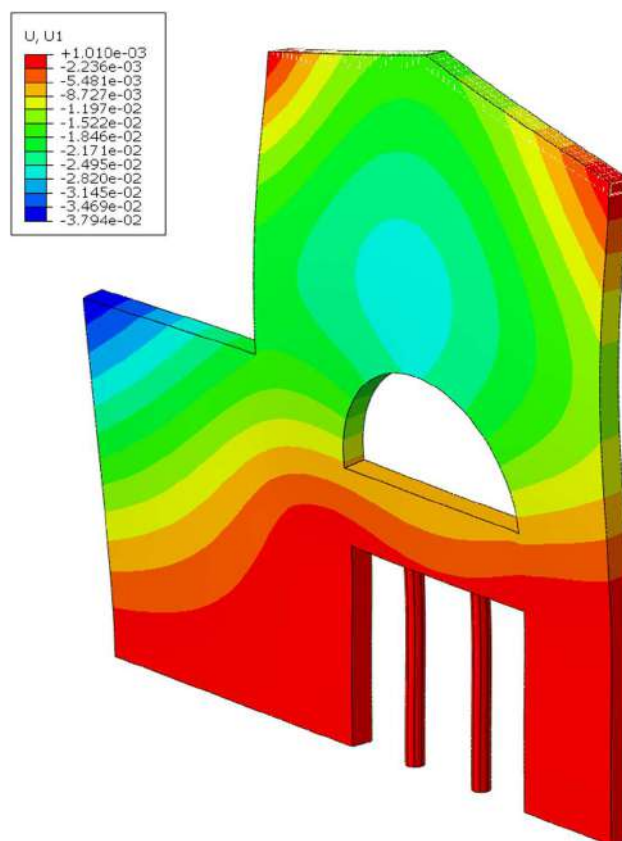


Figure 37. Results for the posterior model.

2.4. Monitoring strategy (DIDA – UNIFI)

2.4.1 Dynamic monitoring

In this work, the pursued monitoring activity concerns the dynamic identification of the Museum by means of in-situ measurements through a wired accelerometer network. This measurement system allows to define the dynamic characteristics of the structures (frequencies, mode shapes and damping values). This investigation method presents different pros, especially related to the existing buildings and the monumental CH structures. In fact, it is easily achievable, non-invasive and it does not interfere with the normal use of the buildings, nor does it introduce additional loads.



Accelerometers measure accelerations induced by ambient vibrations or by external forcing. Through the measurements obtained from the accelerometers it is possible to identify the main dynamic characteristics of the monitored structure [11, 12, 13, 14].

2.4.2 Instrumentations, setup, and data acquisition

Preliminary to the definition of the setup of the experimental monitoring, FE models of the investigated asset should be realized based on the architectural and structural evidence from the previous surveys. This phase is important to execute preliminary modal analysis to analyse the expected behaviour of the structure. In this way it will be possible to point out the portions subjected to major displacements as the ones significant to install properly the instruments for a correct dynamic identification. In this phase, different hypotheses on the mechanical parameters involved as on the boundary conditions should be considered, in order to understand whether these features could lead to different global trends of the structure, thus arranging an experimental setup able to catch most of the likely configurations [15].

When a limited number of sensors is available, it is possible to alternate their position by maintaining some fixed instrument. The sensor positions should be planned according to the results of the preliminary FE modal analysis, considering the architectural morphology of the building as the capacity of the horizontal diaphragms to behave as rigid membranes [15].

Acquisitions during ambient vibration tests should be performed for time windows of at least 30 minutes, with a minimum sampling rate of 50-100 Hz.

2.4.3 Data analysis and interpretation

Given the measurement signals classical post-processing methods based on filtering and detrend procedures can be employed to clean the external noise. To extract the dynamic parameters of the structure considered, Operational Modal Analysis (OMA) is suggested as non-invasive technique instead of Experimental Modal Analysis (EMA) techniques which require the application of external load to the



building. The most used technique is the OMA: Frequency Domain Decomposition (FDD), Enhanced Frequency Domain Decomposition (EFDD) and Covariance Driven Subspace Stochastic Algorithm (SSI-cov).

A dynamic identification campaign was carried out at the Marino Marini Museum over a three-day period (8th-10th July), aimed at experimentally characterizing the dynamic behaviour of the historical structure.

Measurements were performed using a wired accelerometer network made of 16 uniaxial accelerometers (8 PCB 393B31, 4 PCB 393B12 and 4 PCB 393 B32) connected to 4 4-channel ICP® sensors signal conditioner leading to 2 DAQ models QuantumX MX840B by HBM ©. The recordings were taken through Catman easy software (HBM ©).

This activity involved the determination of the main dynamic parameters of the building, namely natural frequencies, mode shapes and damping ratios. Based on the outcomes of the preliminary modal analysis, the FE model indicated that the upper level of the first aisle facing the façade, was the structural part to in-depth investigate with the in-situ measurements. In fact, due to the lack of intermediate elements, this area appears to be vulnerable to potential out-of-plane mechanisms involving the three walls of the naïve, the façade and the two side walls.

The schemes of the measurement setup executed during the campaign are shown in Figure 38. Unfortunately, due to bureaucratic reasons, it was not possible to conduct the measurements installing the instruments on the external side of the façade through the usage of cherry pickers, therefore the setup dispositions were designed considering a 6-meter scaffolding to be installed at the last floor of the Museum. In addition, due to indentations in the slab of this level, it was not possible to measure the central points of the three walls.

On the first day (8th July), set-up 1 was implemented. Most sensors were installed at the top floor of the structure at two different elevations (+10.60 and +12.60), while two additional accelerometers were placed at the intermediate mezzanine level (+4.90), in order to investigate the influence of the reinforced concrete slab on the overall structural behaviour.

On the second day (9th July), set-up 2 was configured. One of the two sensors previously located at the mezzanine level was repositioned at the location of the other, but with its sensing axis oriented



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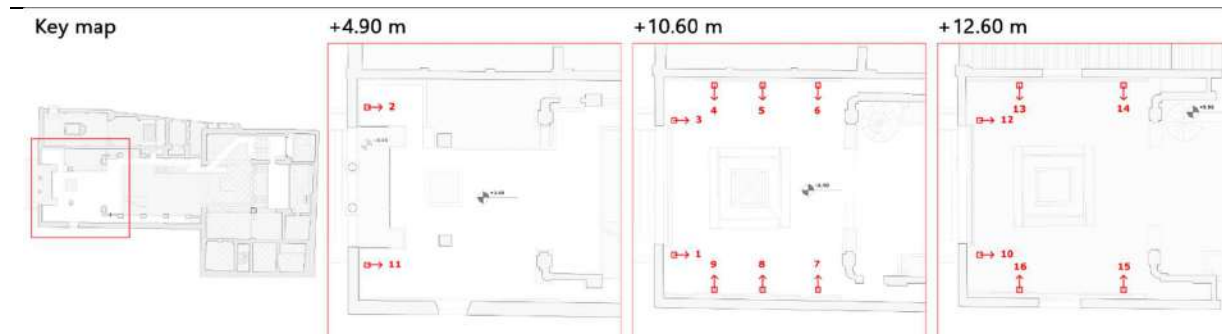
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orthogonally. This change aimed to improve the reconstruction of modal components in the transverse direction.

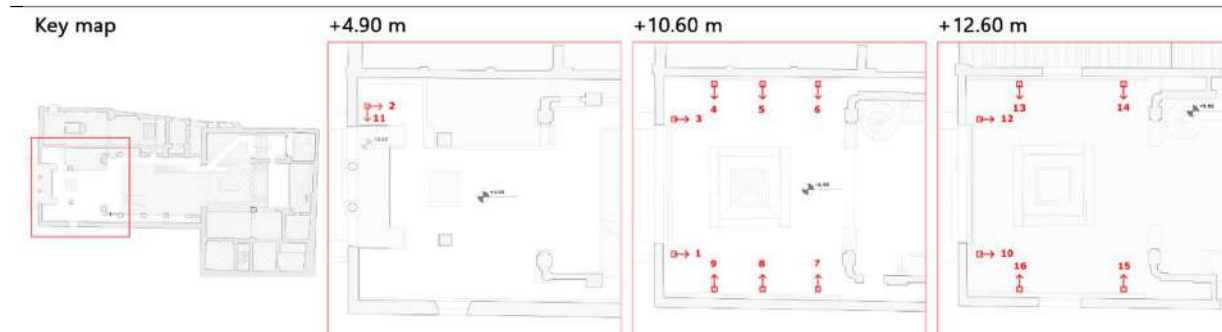
On the second day (10th July), set-up 3 was also implemented. One of the mezzanine sensors was relocated to the level corresponding to the triumphal arch on the top floor (+16.10 m), while two accelerometers originally positioned near the central area of the upper level were moved to the longitudinal sides of the church hall (+11.10). This arrangement aimed to obtain more detailed information on the bending behaviour along the longitudinal axis of the nave.

In Figure 39 the view of the installation of the accelerometer inside the hall.

Setup 1



Setup 2





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Setup 2

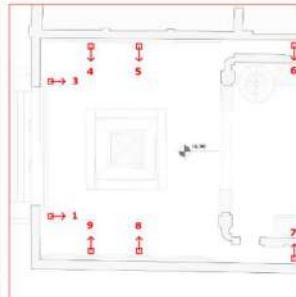
Key map



+4.90 m



+10.60 m



+12.60 m

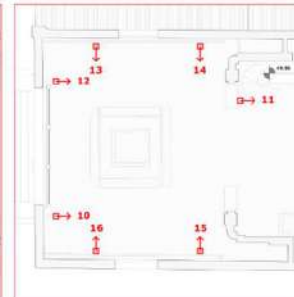


Figure 38. View of the experimental setup for in-situ measurements.



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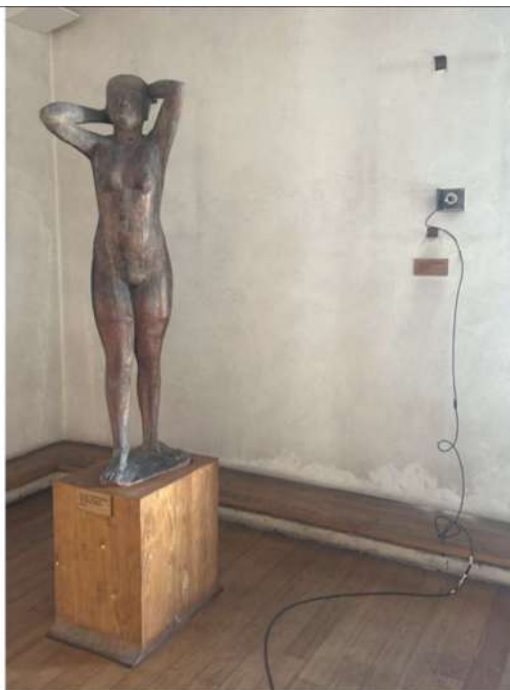


Figure 39. View of the installation of the accelerometer inside the hall.

Recordings were carried out adopting a sampling frequency of 1200 Hz and variable time-windows of around 30 minutes. The neighbourhood of the Museum is located in the historical center with limitations to vehicular traffic and the measurements were executed during flat days not particularly windy, thus limiting the ambient vibrations exciting the structure. Traffic excitations were induced driving the minivan of Laboratorio Ufficiale Prove Materiali e Strutture over bumpers disposed in various ways in front of the main façade of the Museum and along the northern street, via della Spada permitting the excitations of the different sensors.

Measurements were analysed through ad-hoc Matlab codes and through Artemis Modal Pro software (version 8.0). The three setups have been analysed individually maintaining points in common within the different configurations.

Given the complexity of the geometry investigated and the modes coming from FE modelling, the purpose of the dynamic identification was to identify the first modes, namely the first according to each direction and the first following ones.

In general, the first two modes according to each direction are found based on every setup investigated. In Table 8 the results considering the SSI-UPCX method are shown in terms of frequencies and damping. Then, in Figure 40 the mode shapes for the first four modes highlighted in Table 8 are reported.

Table 8. Dynamic identification of Marino Marini Museum: frequencies according to SSI-UPCX method.

Setup 1	Setup 2	Setup 3
Freq [Hz]	Freq [Hz]	Freq [Hz]
2.923	2.925	2.886
3.234		3.398
3.395	3.421	3.42
3.435	3.529	3.478
3.515		4.781
5.059	5.056	5.088
6.114		



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6.145		
6.231		
6.939		
6.943		6.951
7.068	7.031	
7.229		
7.44		
8.947	8.971	8.969
9.181		9.811
9.226		10.644
11.451	11.538	11.391
12.199		11.437
13.957		
14.053		
14.12	14.221	
14.897		
14.984		
15.827		
15.837		
16.136		16.211
16.139		
16.187		16.339

The results of the dynamic identification indicate mode shapes in line with the ones coming from the preliminary FE modelling. Besides the frequencies of the real structure are slightly higher than the one coming from the numerical mode, the first two mode shapes highlighted through measurements are



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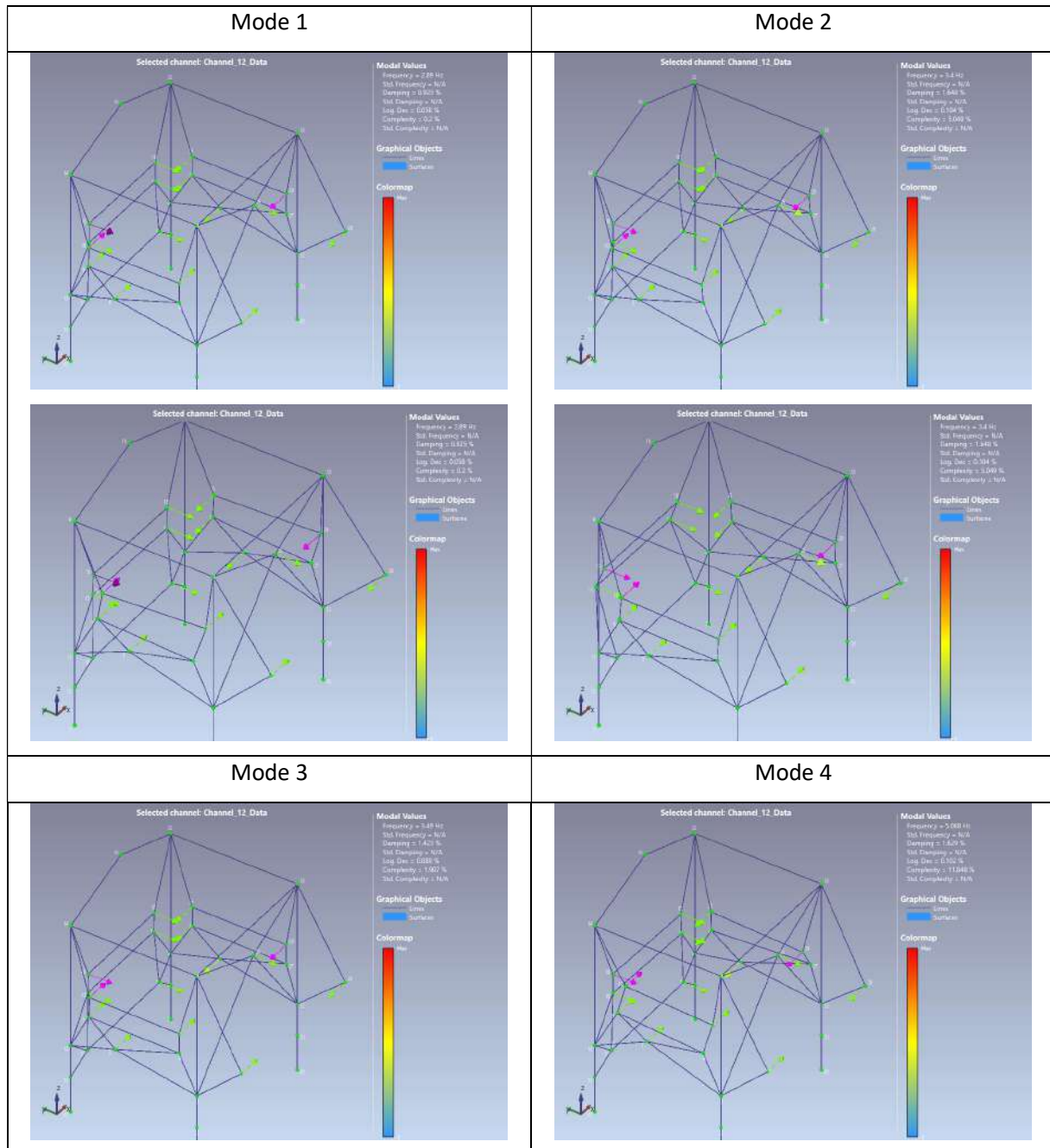
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consistent with the ones of the model, with a first mode transversal to the main façade and exciting globally the naïve of the museum, and a second mode involving the main façade and the two side walls.





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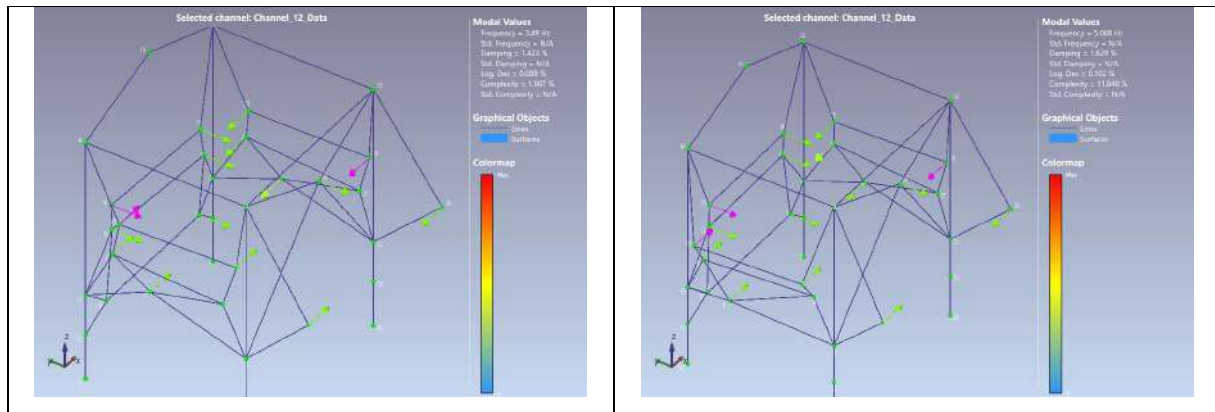


Figure 40. Deformed shapes for the first four identified modes.

2.5. Seismic risk analysis (DIDA – UNIFI)

2.5.1 Seismic hazard

The hazard curve for the city of Florence has been plotted according to the Italian Technical Code [10]. The latter expresses the average annual frequency of exceedance of the local seismic intensity defined in terms of peak ground acceleration (PGA). The hazard curve is obtained starting from the 9 PGA values associated with the different return periods reported in [10] (from 30 years to 2475 years) considering a soil class A. In Table 9 the values of the elastic spectra associated with the different return periods for the city of Florence are shown, while in Figure 41 the corresponding hazard curve is presented.



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Table 9. Values of the elastic spectra associated with the different return period for the site acceleration (a_g), the maximum value of the amplification factor of the spectrum in horizontal acceleration (F_o), starting period of the section with constant velocity of the spectrum in horizontal acceleration (T_c^*).

Tr [years]	a_g [g]	F_o [-]	T_c^* [s]
30	0.047	2.551	0.253
50	0.056	2.586	0.268
72	0.064	2.594	0.276
101	0.072	2.591	0.282
140	0.080	2.601	0.287
201	0.094	2.524	0.294
475	0.131	2.413	0.302
975	0.167	2.388	0.311
2475	0.221	2.414	0.319



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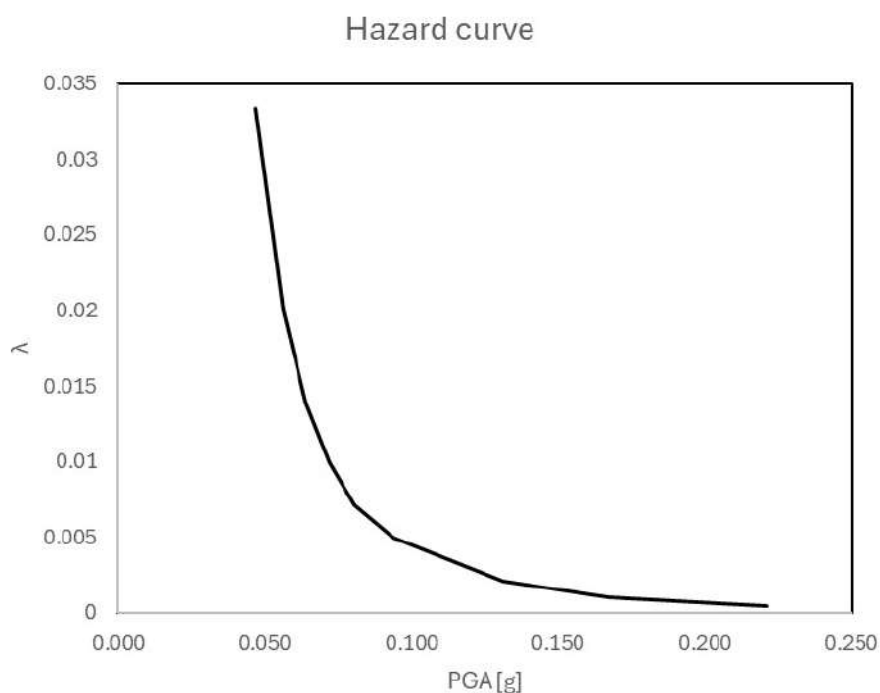


Figure 41. Hazard curve for the city of Florence.

2.5.2 Vulnerability

The vulnerability of the Museum has been investigated by means of nonlinear static (pushover) analysis. Pushover analyses consist of the application of an incremental monotonic horizontal load to the building. Hence, the plot of the relationship between the horizontal displacement of a selected control point and the total base shear of the structure leads to the definition of the pushover curve for the performance evaluation.

The analyses were carried out along the direction orthogonal to the main façade, adopting a mass-proportional load pattern, as described in Section 2.3.8. Due to the presence of the uppermost level characterized by a significant height and the absence of intermediate floors capable of effectively

distributing the seismic forces, multiple control points were selected at the top level of the structure in order to better capture the deformation patterns.

For the *prior* model, the pushover analysis terminates as a result of numerical instability when the applied horizontal acceleration reaches approximately 20% of gravity (0.20 g). In contrast, the *posterior* model exhibits a higher seismic capacity, with the analysis terminating at an applied horizontal acceleration of 27% of gravity (0.27 g), corresponding to an increase in capacity of about 35% with respect to the prior configuration.

The seismic verification was performed in accordance with the Italian code provisions, adopting the N2 method [16]. Pushover curves were converted into capacity curves and subsequently transformed into bilinear curves according to the bilinearization procedure reported in [10]. The performance points for capacity associated to SLD and SLV have been determined according to [17] relating SLD to the yielding point and the SLV capacity to the ultimate point of the curve. The resulting bilinear curves were then expressed in the ADRS (Acceleration-Displacement Response Spectrum) format and compared with the seismic demand spectra to evaluate the structural performance.

2.5.3 Fragility curves

Once the capacity curves were obtained for the investigated case study, the seismic fragility of the structure was evaluated. The procedure proposed in the Hazus method [18] was adopted. Fragility curves were modelled as cumulative lognormal distributions; for a given a value of the intensity measure (IM), the fragility function expresses the probability of reaching or exceeding a specific damage state (DS):

$$P [DS|IM] = \Phi \left[\frac{1}{\beta_{DS}} \ln \left(\frac{IM}{IM_c} \right) \right] \quad (1)$$

where: Φ is the standard normal cumulative distribution function; IM is defined in terms of PGA, so IM_c is the capacity PGA; β_{DS} is the standard deviation of the lognormal distribution, defined, for each damage state DS . For $DS4$ it can be described according to the following relationships:

$$\beta_{DS4} = 0.1 + 0.4 \ln(\mu) \quad (2)$$

μ is defined as a ductility index and is accounted as the ratio between the ultimate and the yielding displacements computed on the capacity curve.

Figure 42 the fragility curves obtained for both the prior and posterior models. The fragility curve of the prior model is reported in blue and is characterized by a reference PGA equal to 0.20 g, corresponding to a 50% probability of exceeding the considered damage state. The fragility curve of the posterior model, shown in red, exhibits a higher reference PGA equal to 0.27 g.

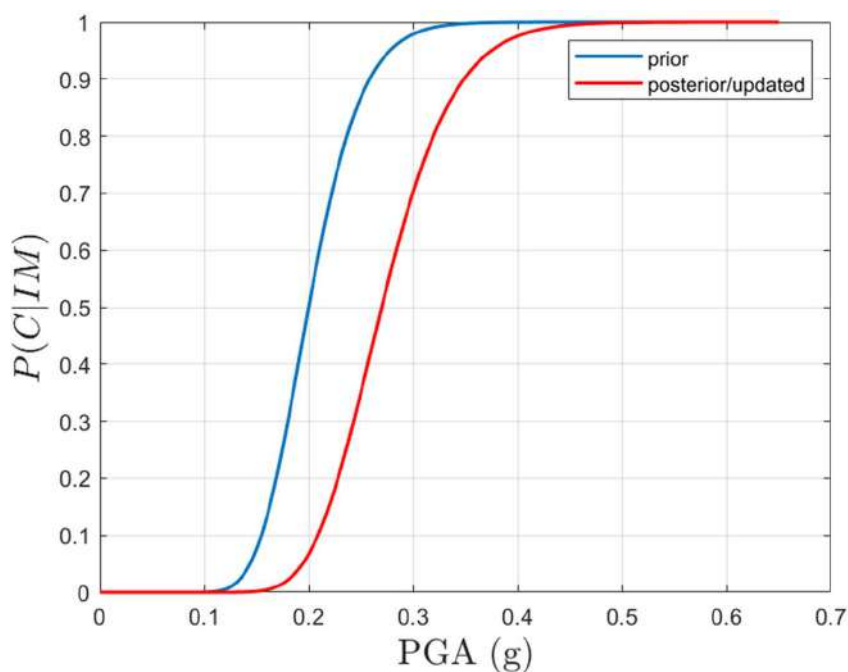


Figure 42. Fragility curves of the considered damage state: comparison between the prior model (blue curve) and the posterior model (red curve).

2.5.4 Seismic risk

The seismic collapse risk is quantified in terms of the mean annual frequency of collapse, obtained by convolving the site-specific hazard curve with the collapse fragility function. Specifically, the risk R was defined as:

$$R = \int P(C|IM) \cdot \frac{d\lambda(IM)}{dIM} dIM$$



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Based on the dynamic experimental tests carried out, the posterior model exhibits a significant reduction in seismic risk: a risk value equal to $4.8 \cdot 10^{-5}$ compared to $3.7 \cdot 10^{-4}$ of the prior model. This reduction reflects the higher seismic capacity captured by the posterior/updated fragility curve.

2.5.5 Bayesian model updating

The model updating carried out deterministically using the results of the ambient vibration tests could be improved by adopting a Bayesian approach, in which measurement errors and modelling uncertainties can be considered. For the specific case study considered in this work, the elastic modulus can be defined in terms of an updated (or posterior) probability density function through Bayes' theorem:

$$P_{\bar{T}}(E) = P(E|\bar{T}) = \frac{p(\bar{T}|E)p_0(E)}{\int p(\bar{T}|E)p_0(E)dE}$$

Where:

- $P_{\bar{T}}(E) = P(E|\bar{T})$: the updated probability density function of the elastic modulus E , given the measured value of the period $\bar{T}$;
- $p(\bar{T} | E)$: the probability of measuring a period equal to $\bar{T}$ when the structure has a value of the elastic modulus equal to a given E . This function represents a curve with E on the horizontal axis. Therefore, for each value of E , there will be a value of $p(\bar{T} | E)$ between 0 and 1;
- $p_0(E)$: the *prior* probability density function, defined on the basis of expert judgment and any reference values provided by the literature or regulations.

The following table reports some indications about the parameters of the lognormal distribution for different masonry typologies.



Tipologia di muratura		f_m [N/cm ²]	τ_0 [N/cm ²]	E [MPa]	G [MPa]
Muratura in pietrame disordinata	μ	140	2.6	870	290
	σ_{ln}	0.29	0.24	0.21	0.21
Muratura a conci sbozzati, con paramento di limitato spessore e nucleo interno	μ	250	4.3	1230	410
	σ_{ln}	0.20	0.19	0.17	0.17
Muratura in pietra a spacco con buona tessitura	μ	320	6.5	1740	580
	σ_{ln}	0.19	0.14	0.14	0.14
Muratura a conci di pietra tenera	μ	190	3.5	1080	360
	σ_{ln}	0.27	0.20	0.17	0.17
Muratura a blocchi lapidei squadriati	μ	700	10.5	2800	860
	σ_{ln}	0.14	0.14	0.14	0.09
Muratura in mattoni pieni e malta di calce	μ	320	7.6	1500	500
	σ_{ln}	0.26	0.21	0.20	0.20
Muratura in mattoni semipieni con malta cementizia (es. doppio UNI foratura < 40%)	μ	650	28.0	4550	1138
	σ_{ln}	0.24	0.14	0.24	0.24
Muratura in blocchi laterizi semipieni (perc. foratura < 45%)	μ	500	35.0	4500	1350
	σ_{ln}	0.20	0.14	0.20	0.20
Muratura in blocchi semipieni con giunti verticali a secco (perc. foratura < 45%)	μ	350	11.5	3150	945
	σ_{ln}	0.14	0.13	0.14	0.14
Muratura in blocchi di calcestruzzo o argilla espansa (perc. foratura tra 45% e 65%)	μ	175	11.0	1400	350
	σ_{ln}	0.14	0.14	0.14	0.14
Muratura in blocchi di calcestruzzo semipieni (perc. foratura < 45%)	μ	370	21.0	2960	740
	σ_{ln}	0.19	0.14	0.19	0.19
f_m : resistenza a compressione della muratura; τ_0 : resistenza a taglio della muratura; E: modulo di elasticità normale; G: modulo di elasticità tangenziale					

Table 10. Parameters of the lognormal distribution for different masonry typologies [19].

The term denoted by $p(\bar{T}|E)$ is called likelihood function, which accounts for the uncertainties present in the new information. They can be subdivided into two main classes: error measurements and modelling uncertainties. Thus, the likelihood function can be estimated decoupling the two sources of uncertainty through [20]:

$$p(\bar{T}|E) = \int p(\bar{T}|E, T) \cdot P(T|E) dT$$

in which $p(\bar{T}|E, T)$ is the term accounting for the measurement errors. In fact, it represents the conditional PDF of having the measured natural period on a structure characterized by a certain elastic modulus and natural period. The second term in the integral, denoted by $P(T|E)$, represents the modelling uncertainties being the conditional probability of the natural period for a structure characterized by a certain elastic modulus.

2.6. Seismic evaluation of inner artworks (DIDA – UNIFI)

The on-going research on the museal building has become the pretext to pursue some detailed evaluation of the art goods, starting from a pilot case. To this aim, the statue “The Juggler” made by Marino Marini in 1954 was adopted as representative case study.

2.6.1 Geometrical acquisition

“The Juggler” is an artwork of 166 x 58.5 x 38.4 cm and it is located at the last floor of the Museum (Figure 43a). The statue is made of bronze and characterized by welded parts within them. The Juggler is represented by a stylize man, standing with the face oriented towards the right side (Figure 43b). Although the figure has the bust right oriented, the legs present a misalignment configuring a stabler position.



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(a)



(b)

Figure 43. a) View of the upper level of the Museum; b) The Juggler statue, by Marino Marini.

Acquisitions have been carried out through photogrammetric survey conducted by Prof. Giorgio Verdiani (Department of Architecture, UNIFI). The campaign was conducted using a digital medium format camera, the Fujifilm GFX-50s capable of taking shots at the resolution of 50 Mp on a sensor of 43.8×32.9 mm, mounting a Fujinon Zoom lens with a focal length of 32-64 mm and a maximal aperture of F4.

- *Photogrammetry*

The options for having a successful SfM_IM (Structure from Motion, Image Matching) photogrammetry mostly depends on the quality of the shots (free from shaking blur, with minimal digital noise and well positioned so to cover all the corners of the subject, reducing the occlusion spaces). In the case of the statue “the Juggler” one possible additional issue was the material in itself, the bronze surface may turn to be too much reflective and interfering with the proper productions of the various steps of the digital model generation. The lens was setup at 64 mm focal length (equivalent to a 51 mm length on a full frame



sensor) and stop-down at F8/F11. The quite intense natural light on the statue allowed a sequence of images well flooded by light, with a proper depth of field on the statue and with a minimal amount of reflections. The intensity of natural light made preferable to operate with the camera on a tripod and then with minimal issues about shaking blur effects, the ISO setting of the sensor was placed at 400, with no significant digital noise production. In all the parts where the reflections started to appear, the shooting was intensified with a denser coverage and with sensible variations in the position of the camera, so to gradually reduce the influence of these strong lights on the modelling process. The overall number of shots was 2057, taken moving around the statue and taking images in circle and then moving up the camera to an upper level and creating a new circle of images. In this way, when the position of the camera reached the height of the statue, a final series of shots were taken placing the camera on an extensible pole, so to cover completely all the top parts of the sculpture. The photogrammetry work at the museum was then finished taking some direct measurement on the base of the statue for further scaling of the resulting model. The following processing was brought on using EG Reality Capture software, with the classic process starting from the alignment and then passing to the polygonal mesh generation (creating a 108,9 million triangles model), the optimization/simplification of the mesh (creating a surface reduced to four million triangles) and finishing with the application of the texture and the extraction of high resolution orthophotos.

- *3D modelling*

The resulting model was then exported in its simplified version, at four million triangles, using the OBJ format. This version was used to upload the model into Sketchfab.com allowing an easy streaming of the result to any user of that community. The model can be viewed and inspected at the URL: <https://skfb.ly/p9sNY> on the Sketchfab platform. The model was then imported into Autodesk 3D Studio Max 2025, where the surface was processed to add a thickness of about one centimetre and then to receive a specific and very practical modifier, the “multires”, it allows to calculate a multi-resolution value for the polygonal surface and then regulate the number of triangles at any reduced value. This solution speeds up the creation of a version compliant with the following needs for FR modelling and seismic analysis allowing multiple versions to be created from the same base model and then tested in the final part of the process. After the correct number of triangles was found the model was processed in Instant



Mesh, a freeware/BSD software dedicated to polygonal simplification and retopology, the optimized and simplified version of the Juggler was then ready to be moved to the following steps.

2.6.2 FE modelling and seismic analysis

The geometric model has been converted in the .iges format and imported into a FE environment. The imported geometry consisted of 8142 faces configuring the statue's shape. Strand7 © was used to realize the mesh of the statue, while Abaqus CAE © has been adopted to execute numerical analyses. The model consists of 43124 nodes and 38504 elements divided between 8271 linear quadrilateral elements of type S4R and 11328 quadratic triangular elements of type STRI65 (Statue) and 18905 linear tetrahedral elements of type C3D4 (Basement) (Figure 44). A linear elastic material was assumed, with a density of 8500 kg/m^3 and an elastic modulus of 113000 MPa. Based on a rough estimation of the weight of the statue, the thickness of the shell elements was assigned as 5 mm, leading to a weight of 70 kg.

The statue was modelled as rested on a pedestal. In this first evaluation, the pedestal has been modelled in a simplified manner, considering it as a rigid body fixed to the ground. To simulate the interaction between the artwork and the pedestal, a contact between the surfaces was modelled through a normal behaviour (defined in Abaqus through a "hard contact") and a tangential behaviour (defined through a friction coefficient).



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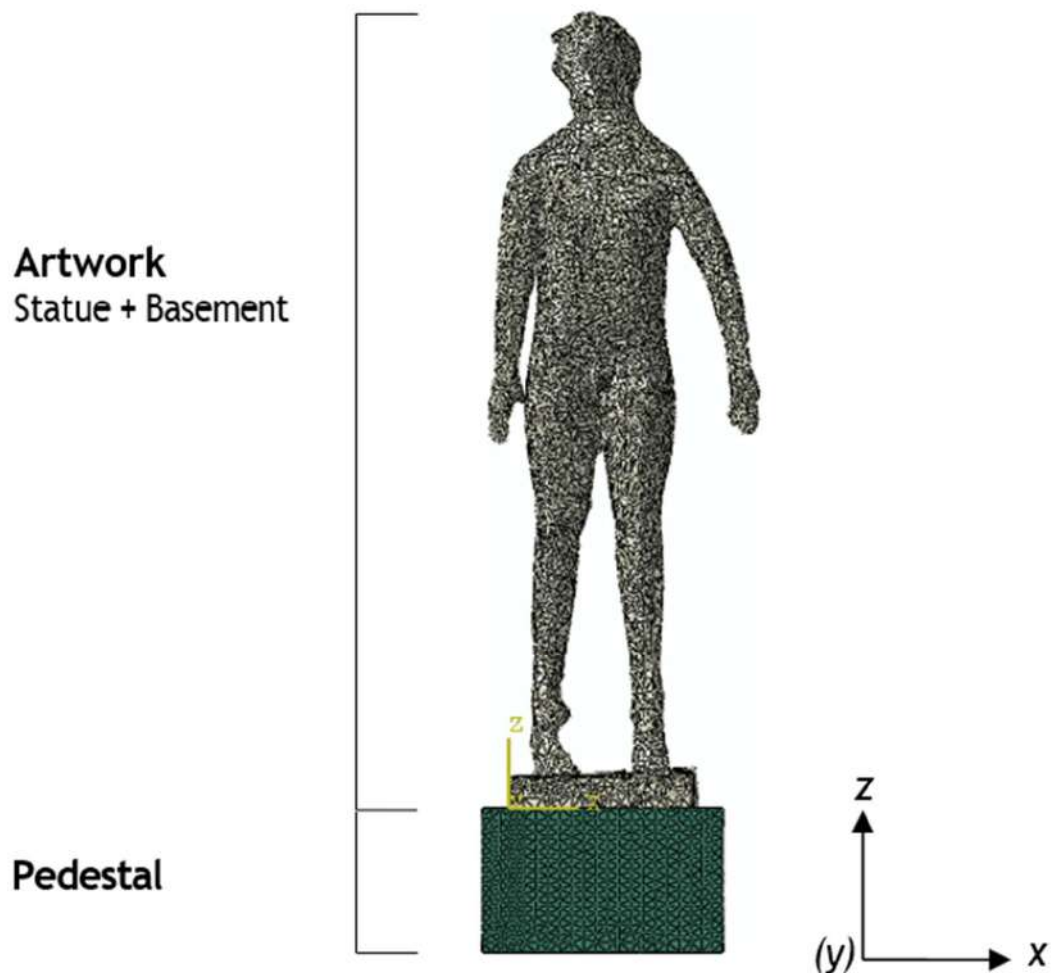


Figure 44. Meshed model of The Juggler with its pedestal.

Preliminary to the seismic assessment, gravity analysis and modal analysis were executed. In terms of gravity loads, the mass of the statue leads to the maximum tensions at the feet and ankles of the model, with maximum stress values in the order of 2.8 MPa.

According to the modal analysis, a first mode of 12.06 Hz was found, with a slight diagonal mode shape given by the geometrical position of the legs. The other principal modes correspond to 43.25 Hz (translational) and 72.92 Hz (torsional).



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2.6.3 Seismic assessment

The seismic input has been defined in accordance with the Italian Building Code adopting parts of the accelerograms reported in Section 2.3.4 (Figure 45). The demand spectrum was considered assuming a nominal life of 50 years and an important factor of 1.5, leading to a reference life of 75 years.

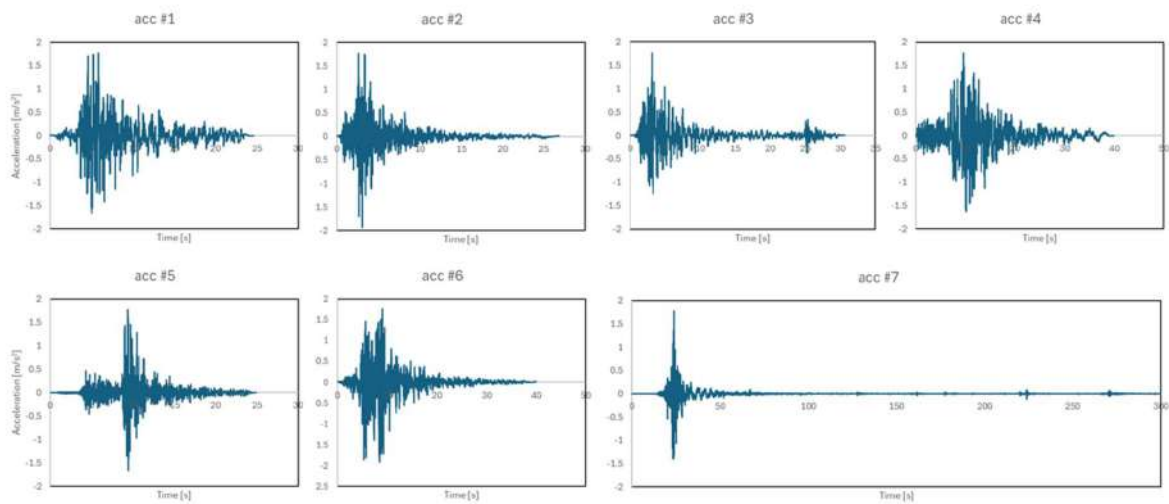


Figure 45. Sample of accelerograms used for the dynamic analyses

A simplified scheme based on the mass of the statue and its center of mass to assess the possibility of sliding was considered. The center of mass is at 0.705 m (z direction), and at 0.20 m from the edges of the pedestal (x, y directions), thus obtaining a horizontal multiplier for activating the overturning equal to 0.283. The sliding condition occurs when the applied force is greater than the static friction force. Considering the surface of the Statue (bronze) and the one of the Pedestal (steel), the available literature indicates a static friction coefficient of 0.16 which brings to an expected sliding mechanism of the statue before overturning.

The numerical analyses have been conducted considering two steps: firstly, the gravity load has been applied, secondly, the accelerogram was assigned to the pedestal. The excitation was given according to the y direction, with a scaled action of 30% in the x direction.



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In Table 11 the main results from the dynamic analyses in terms of differential displacement and maximum stress are reported, while in Figure 46 the time-history graphs of each run are shown.

In absence of vertical component, excluded in this study, the artwork tends to rotate over its vertical axis, pointing out reduced displacements from the initial position. The scatter between points B and C during the time-history reaches limited increments, with residual scatters at the end of the analyses. In terms of maximum stress, the values coming from FE analysis are within the range of the material.

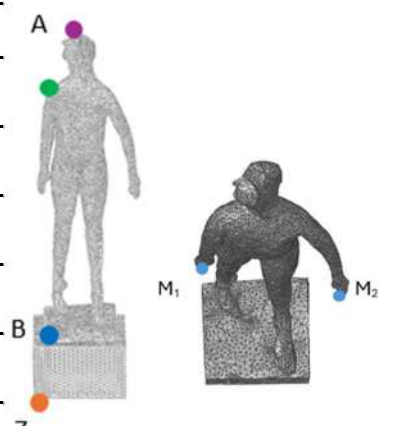
Table 11. Results from the dynamic analyses

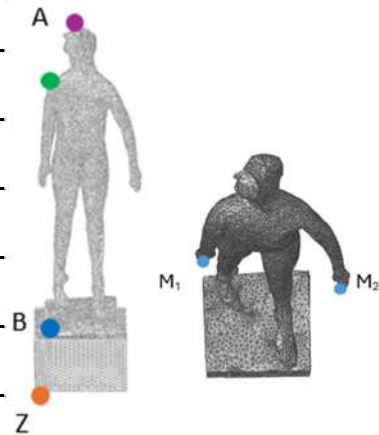
	base disp.		top-base disp.			
	(B-Z)		(A-B)		rotation	max stress
ACC	[m]		[m]		[rad]	[MPa]
	x	y	x	y		
1	1.12E-03	3.58E-03	6.12E-04	7.00E-04	0.000882	13.91
2	6.68E-04	1.50E-03	8.63E-04	7.62E-04	0.001617	11.87
3	3.91E-04	1.41E-03	5.75E-04	5.73E-04	0.000594	12.26
4	1.15E-03	3.93E-03	5.43E-04	5.59E-04	0.000663	12.38
5	7.97E-04	1.18E-03	8.50E-04	7.37E-04	0.001595	14.10
6	7.52E-04	2.14E-03	1.38E-03	9.15E-04	0.003495	12.26
7	2.11E-04	5.56E-04	5.66E-04	5.17E-04	0.000653	12.67

A

B

Z







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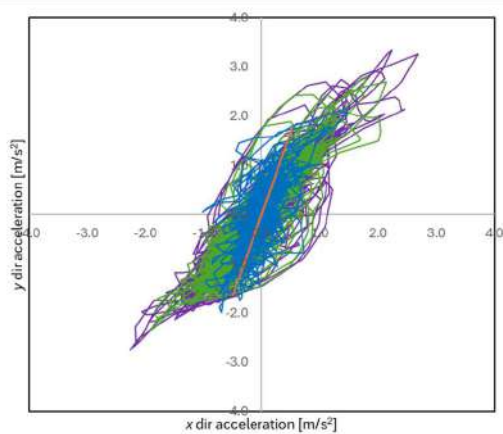
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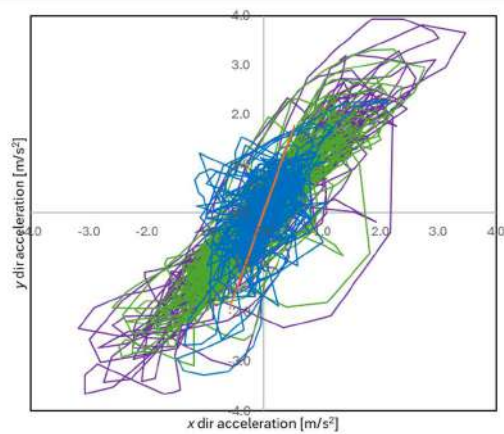
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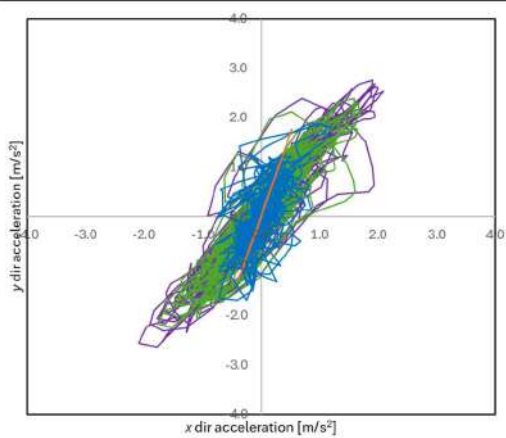
Acc 1



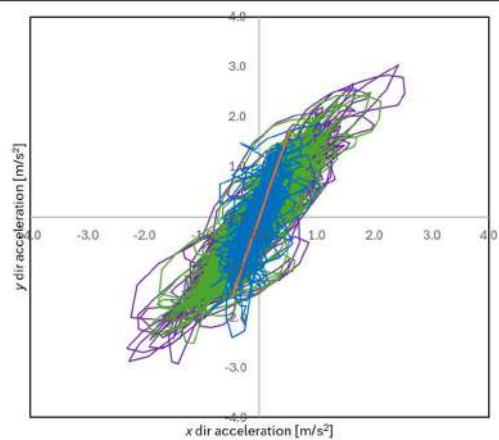
Acc 2



Acc 3



Acc 4





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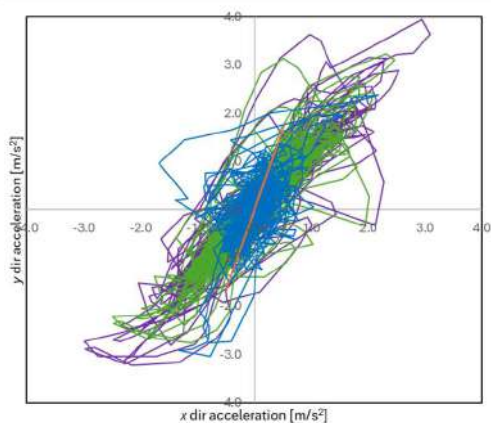
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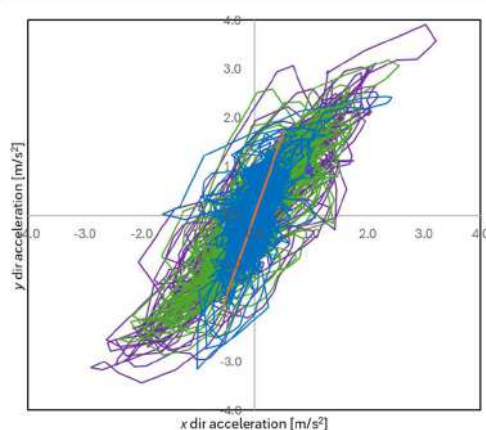
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Acc 5



Acc 6



Acc 7

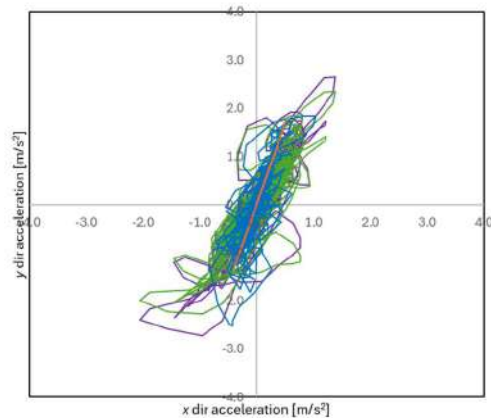


Figure 46. Acceleration time-history for different monitored points for the different dynamic analyses

The outcomes of this study provide partial results, offering initial insights into the seismic assessment of the statue. Further analyses are required to comprehensively evaluate its behaviour under ground motions. Future steps of the work will include analyses considering additional accelerogram records, the

account of vertical component, as additional interactions involving the possible sliding of the pedestal on the pavement.

2.7. Flood risk analysis (DICEA – UNIFI)

2.7.1 Flood hazard in the context of the city of Florence

Florence, built along the Arno River, has a notable historical and ongoing flood risk, rooted in its geography, climate, and urban development [21].

The catastrophic flood that struck Florence in November 1966 stands as the most devastating event in the city's history in terms of damage to cultural heritage and economic activity. It resulted in the deaths of 38 people across the city and its surrounding province, and caused the widespread destruction of invaluable artworks, early manuscripts, and archaeological artefacts. This event remains a defining moment in Florence's modern history. The Arno River basin, located in Tuscany within the mountainous terrain of the Northern Apennines, spans a total catchment area of 8,238 km² and has an average elevation of 353 meters above sea level. It is divided into six main sub-basins:

- Casentino (upper valley), where the Arno initially flows southeast;
- Val di Chiana (left tributary);
- Upper Valdarno, where the Arno shifts direction northwest;
- Sieve (a right tributary joining the Arno 18 km upstream of Florence);
- Middle Valdarno, which includes the Florence plain;
- Lower Valdarno, encompassing the Pisa plain and several tributaries.

In Florence itself—situated within a drainage area of approximately 4,300 km²—the Arno flows through a confined, straight channel with no natural floodplain. As it passes through the city, it encounters four weirs (locally known as *pescaie*) and eight bridges that connect the historic center on the right bank to the left bank. Within the 2-kilometre section of the river that runs through the city's historic core—formerly enclosed by Florence's outer medieval walls—the Arno is flanked by two oblique weirs: San Niccolò and



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Santa Rosa. Four key bridges span this reach: Ponte alle Grazie, Ponte Vecchio, Ponte Santa Trinita, and Ponte alla Carraia. These structures play a significant role in influencing the river's flow dynamics, as they constrain its conveyance capacity and affect both hydrodynamics and sediment transport.

Flooding has been an integral part of Florence's history since at least the 12th century. Among the most significant flood events were those of 1333, 1547, 1557, 1589, 1740, 1758, 1844, and 1966. The extreme water levels reached during the most severe floods (notably those of 1333, 1557, 1844, and 1966) are still memorialized by stone plaques on buildings throughout the city and by historical maps detailing the areas affected.

On November 3 and 4, 1966, multiple rain gauges across the Arno River basin recorded over 200 mm of rainfall within 48 hours. This intense precipitation was compounded by prior rainfall events in October 1966, which had already saturated the soil throughout the basin. As a result, Florence experienced widespread flooding on the morning of November 4.

Initially, the Arno overflowed its banks upstream of the city, but soon floodwaters breached embankments along the Lungarni—the riverfront avenues—where bank failures occurred at multiple points. The inundation caused significant infrastructure damage, including the destruction of many hydrometric stations and their recorded data. Consequently, the flood hydrographs had to be reconstructed retroactively using high-water marks combined with hydraulic modelling of the flood's propagation.

The peak discharge just upstream of Florence was estimated at 4,100 m³/s, corresponding to an approximate return period of 200 years. However, what made the event especially catastrophic was the flood volume: the river's discharge exceeded its carrying capacity—then estimated at around 2,500 m³/s—for over 20 continuous hours. This sustained overflow resulted in an estimated 100 million cubic meters (mm³) of water inundating the city.

Maximum flood depths reached 5 to 6 meters in parts of Florence, particularly within the historic center and the Santa Croce district. The floodwaters carried significant amounts of mud—from suspended sediments in the Arno—and naphtha, a fuel used at the time for heating and commonly stored in building

basements. This toxic mix proved particularly damaging to artworks and cultural materials, many of which required specialized and complex restoration techniques in the aftermath.

In the immediate aftermath of the 1966 Florence flood, the Italian government established the De Marchi Commission—named after its chairman, Professor Giulio De Marchi—to assess national flood risk and formulate strategies for hydraulic and land management (Commissione Interministeriale per lo studio della sistemazione idraulica e della difesa del suolo). Over the course of two decades, the Commission's work laid the groundwork for a national land protection program and culminated in the passage of the landmark Law 183/1989. This legislation marked a major shift toward an integrated approach to soil and water management in Italy and led to the creation of River Basin Authorities.

Among its many contributions, the De Marchi Commission initiated several flood risk-reduction projects for the Arno River Basin. However, these efforts were never fully implemented. In fact, between 1966 and 1996, little concrete action was taken to significantly reduce flood risk in the Arno Basin. While various local flood protection measures were proposed—and a few executed—these initiatives lacked coordination and were not part of a comprehensive flood management plan. The only major intervention during that period was carried out in 1980, when the aprons beneath the Ponte Vecchio and Ponte Santa Trinita were lowered. This modification increased the river's conveyance capacity in Florence's urban section from approximately 2,500 m³/s (in 1966) to 3,100 m³/s—or 3,400 m³/s without any safety buffer.

Following the enactment of Law 183, the Arno River Basin Authority (AdB) was established and, in 1996, it released the Flood Risk Plan (HRP) for the basin. However, the HRP did not specifically address the protection of Florence's cultural heritage, despite the severe damage suffered during the 1966 flood. The HRP acknowledged that Florence would still be at high risk if a flood comparable to that of 1966 were to occur again. It identified the following key factors contributing to flood hazard:

- Inadequate hydraulic infrastructure and a lack of floodwater storage capacity along the river;
- Urban expansion into former agricultural zones, increasing impermeable surfaces;
- Neglect of forestry and agricultural lands in the surrounding hills, leading to faster runoff and reduced absorption;



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- Insufficient maintenance of river channels and flood defense structures.

To address these issues, the HRP proposed a flood management strategy focused on:

- Increasing floodwater storage capacity within the basin;
- Expanding the conveyance capacity of the Arno River;
- Raising and reinforcing the levee system;
- Enhancing monitoring, control, and emergency response capabilities.

Despite the clarity of these objectives, implementation was limited. The most significant project realized was the construction of the Bilancino Dam on the Sieve River, completed around 2000. This reservoir has since reduced peak flood flows into Florence by an estimated 100–200 m³/s, providing some degree of protection, though not enough to eliminate the risk posed by a flood similar in magnitude to that of 1966.

In 2005, after a modification of the relevant legal framework in Italy, the HRP was replaced by the Hydro-Geological Plan (PAI). The interventions planned by the PAI essentially were the same that were foreseen by the HRP. The PAI devoted some attention to the funding required to implement the measures.

The most recent planning initiative aimed at addressing flood risk in Florence is the Flood Risk Management Plan (PGRA), which was approved in 2015 in accordance with EU Directive 2007/60/EC—commonly known as the EU Flood Directive. The PGRA was developed by the Arno River Basin Authority (AdB) in collaboration with the National Civil Protection Authority. It replaces the previous PAI (Hydrogeological Structure Plan) and now serves as the primary framework for guiding future flood risk mitigation efforts in the region.

In line with the Flood Directive, the PGRA classifies mitigation actions into four categories, and introduces a new, forward-looking approach to flood risk: shifting from a traditional “culture of safety” toward a broader “culture of risk management.” This paradigm emphasizes cost-benefit analysis, sustainability, and the acceptance of residual risk—recognizing that total protection is neither always possible nor economically justifiable.



Importantly, for the first time, cultural heritage is explicitly identified as a key asset to be preserved in the context of flood risk management. Unlike previous strategies that prioritized large-scale engineering solutions, the PGRA favors low-impact, sustainable interventions. It de-emphasizes high-impact infrastructure such as new dams, and instead promotes targeted, flexible solutions that operate only during extreme flood events and avoid altering the river's normal flow regime.

The core interventions planned under the PGRA include:

- Creation of flood retention areas—modestly sized and designed to temporarily store excess water during major flood events;
- Raising the height of existing dams to enhance their flood retention capacity;
- Dredging and removal of sediment deposits from existing reservoirs to restore lost storage volume.

Together, these measures reflect a modern, risk-informed strategy for protecting Florence—one that balances safety, environmental impact, cost-efficiency, and the preservation of its invaluable cultural heritage.

2.7.2 Flood hazard simulation of the city of Florence

The simulation of flood hazard adopts the HEC-RAS software. HEC-RAS (Hydrologic Engineering Center's River Analysis System) is a hydraulic modelling software developed by the U.S. Army Corps of Engineers. It is widely used for simulating one-dimensional and two-dimensional hydraulic behaviour of natural and engineered river systems. The software is freely available and extensively applied in flood risk assessment, river engineering, urban drainage design, and environmental flow studies.

HEC-RAS is based primarily on solving the Saint-Venant equations—a set of partial differential equations that describe unsteady, gradually varied open-channel flow:



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- Continuity Equation (mass conservation)
- Momentum Equation (Newton's 2nd law applied to fluid flow)

Depending on the modelling setup, HEC-RAS can solve these equations in either:

- Steady-state flow mode (energy-based calculations)
- Unsteady flow mode (fully dynamic wave modelling)

HEC-RAS offers different numerical solvers depending on the model type:

- 1D steady/unsteady flow: Implicit finite difference methods (Preissmann scheme)
- 2D unsteady flow: Finite volume methods with conservative mass balance (options include diffusion wave, full dynamic wave, and hybrid solvers)

Time-step and grid resolution choices are essential for stability and accuracy.

The geometric representation of the physical system is described by:

- Cross-sections (1D): Extracted from terrain data (e.g., LiDAR), bathymetric surveys
- 2D mesh (HEC-RAS 2D): Created using terrain rasters with flexible grid sizes (rectangular or irregular cells)
- Bridges, culverts, weirs, gates, levees: Represented with empirical/structure-specific equations

In unsteady simulations, HEC-RAS requires boundary conditions (i.e., hydrologic input) such as:

- Inflows (hydrographs or rainfall-runoff inputs)
- Downstream stage/discharge conditions
- Lateral inflows, rainfall, and infiltration (in 2D)

The flood simulation for the urban area of Florence is performed using a coupled 1D–2D hydraulic model implemented within the HEC-RAS platform. River flow is modeled using a one-dimensional approach, with



cross-sectional geometry derived from a high-resolution topographic survey that includes detailed bathymetric data and the location of key hydraulic structures such as bridges and weirs.

The river is hydraulically connected to the surrounding floodplain through lateral weirs, allowing overflow into a two-dimensional computational domain. This floodplain is modelled using the diffusive wave approximation, which captures the dynamic behaviour of overland flow during flood events.

The 2D mesh is generated from a LiDAR-based digital terrain model with a spatial resolution of 1 meter, and buildings are represented as impermeable obstacles to flow. Boundary conditions include a rating curve at the downstream end and inflow hydrographs at the upstream boundary, reflecting different flood scenarios with return periods of 30, 100, 200, and 500 years and different hydrograph durations (18 to 36 hours).

Model validation is conducted at the urban scale, using recorded flood depth data from the most recent historical flood event in the study area. Simulation outputs are exported at intervals ranging from 15 to 30 minutes, depending on the total duration of the simulation.

The simulation involves 12 km of Arno river reach, with approximately 60 cross sections and all the hydraulic infrastructures. Downstream rating curve is built with an ad-hoc 1D-quasi 2D model of the flood plain between Pontassieve and Signa. The following figure shows the whole computational area and a detail close to the historical city centre.

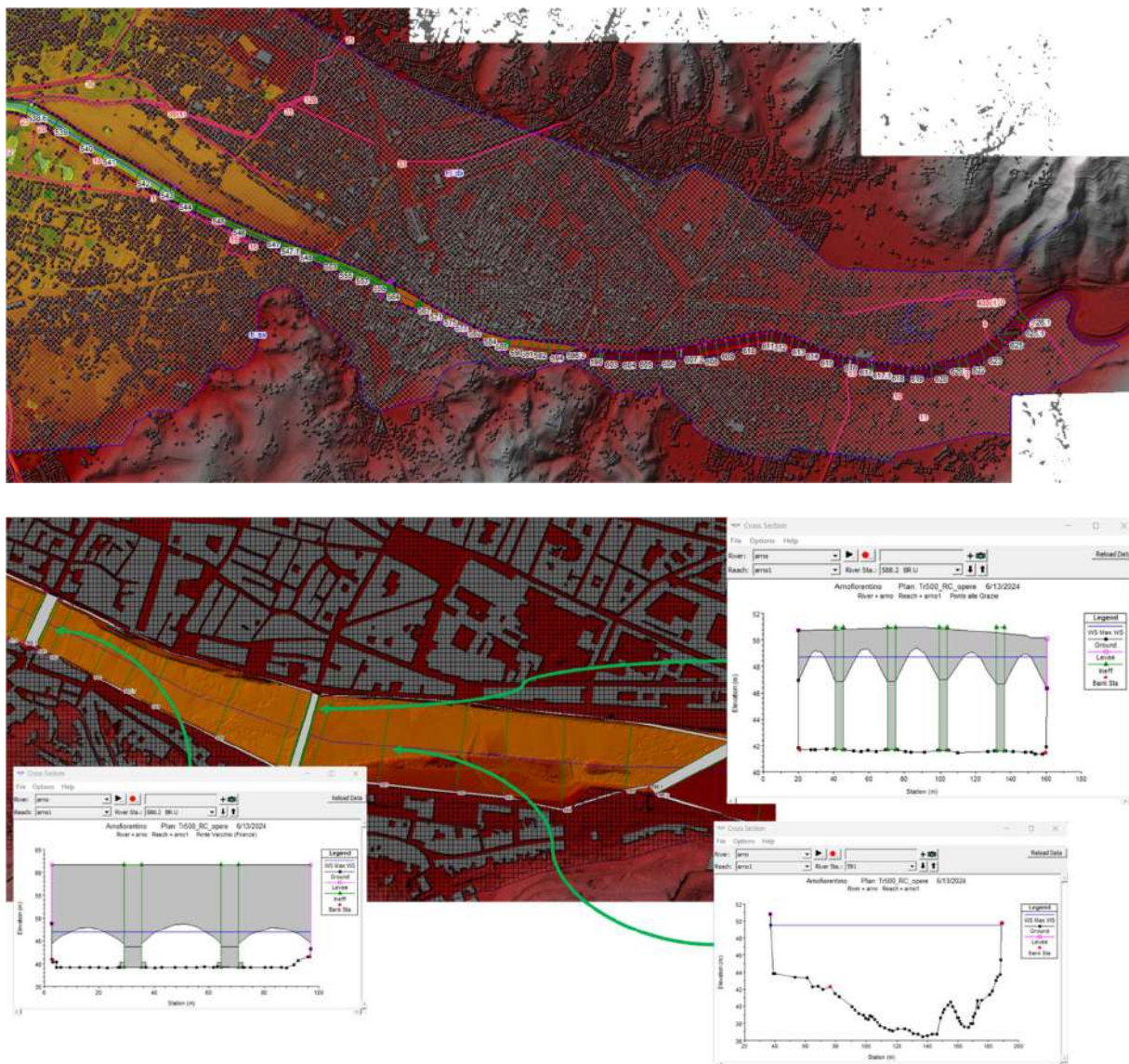


Figure 47. Computational domain for the 1D-2D flood model set-up (top), detail of the city centre with bridge geometries and cross sections (bottom).



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Return

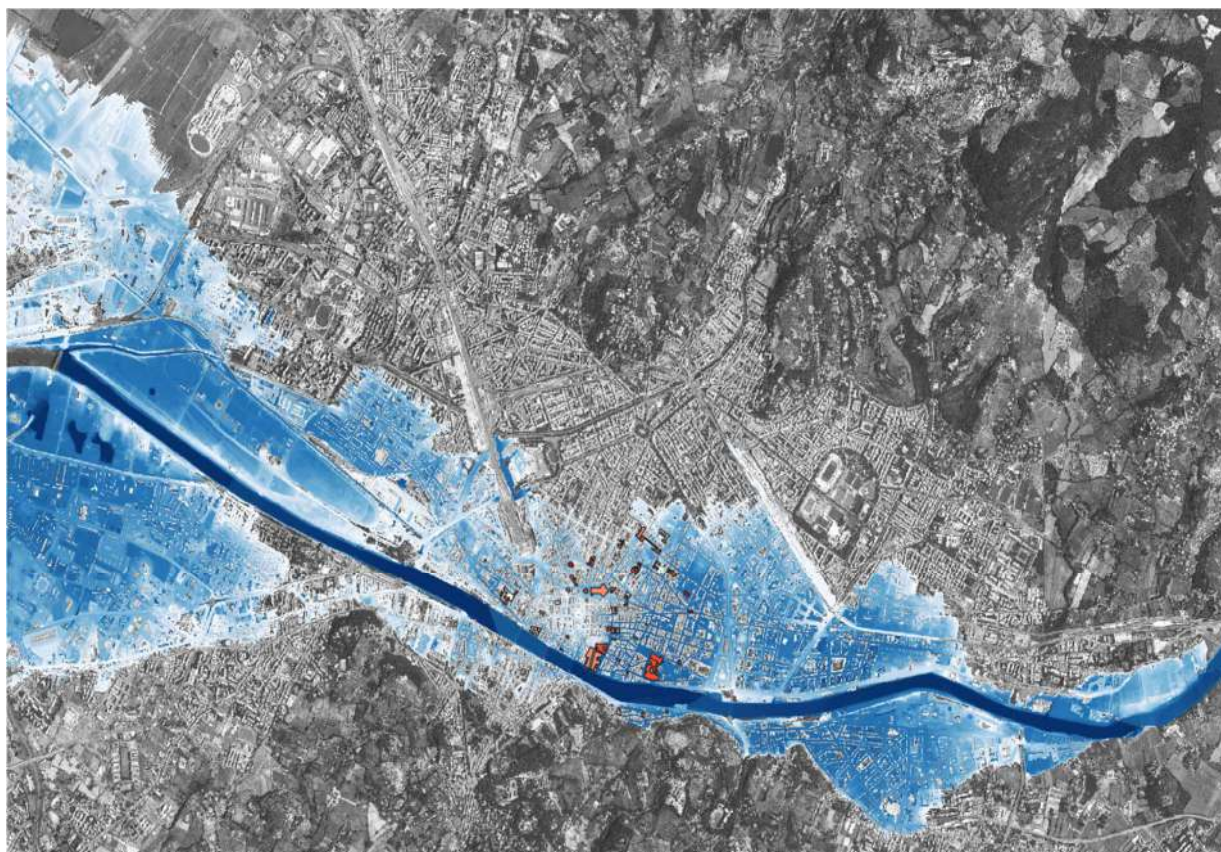


Figure 48. Urban-scale inundation map for the historical 1966 flood event (Return period of about 500 years).

2.7.3 Flood hazard simulation of the Marini Museum

The simulation of the inundation inside the Museum requires two main steps:

- An on-site inspection of the perimeter of the building and floor elevation to identify the potential entry points of floodwater
- A new set-up of a 2D inundation model for the building that downscales the urban inundation model to the building.

The outcome of the inspection identifies the following characteristics relevant to the flood exposure:



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- The entrance of the museum is elevated through stairs up to 1.16 m above the road pavement
- The exposed part of the building is the basement, i.e., the former crypt, for flood scenarios equal or rarer than 200 years return period
- There are three points from which the flood water can enter the building (two of them on the façade are shown in Figure 49)
- The basement floor typically hosts temporary exhibitions of different nature.

The **Marino Marini Museum** is exposed only for its basement floor (former crypt)

- Flood depth at the entrance (Tr 200): 0,06 m
- Elevation of the entrance (above road pavement): +1,16 m
- Elevation of the exit from the former crypt: -0.63 m



Figure 49. Points of possible ingress of floodwater and position of the main entrance above the road pavement.

For the setup of the building inundation model, two main phases are required: the mesh creation and the boundary condition set-up.

For the mesh creation the activity involves the following steps:

1. Floor Plan Digitization in QGIS

The process begins by importing the floor plan image into QGIS. Two polygon shapefiles are created: one for the outer boundary (excluding internal walls) and another for internal walls. This is done by selecting



“Create New Shapefile Layer”, choosing “Polygon” as the geometry type, and digitizing the floor plan accordingly.

2. Digital Terrain Model (DTM) Preparation

A temporary DTM is created by clipping the existing terrain model to the extent of the floor plan and modifying it using elevation data from site surveys. The Serval plugin is used to edit elevation values directly within QGIS.

3. Boundary Conditions Mapping

Two-line shapefiles representing boundary conditions are drawn in QGIS. This involves creating a new shapefile with “LineString” geometry and digitizing the appropriate lines.

4. Importing Data into RasMapper

The shapefile of the outer building perimeter (excluding internal walls) is imported into RasMapper under Geometries → 2D Flow Areas → Perimeters → Import Features from Shapefile. The temporary DTM is also imported via “Create New RasTerrain.”

5. Defining Breaklines

To represent elevation changes like steps, breaklines are added in the 2D Flow Area. After editing the breaklines, the “Near Repeat” property is set to 2 for accurate elevation interpolation.

6. Computational Mesh Generation

A computational mesh is generated by setting the point spacing (DX) to 0.5 meters under Edit → Edit 2D Area Properties, followed by creating computation points using all breaklines. The resulting mesh contains 5,298 computational cells at 0.5 m resolution.

7. Incorporating Internal Walls

To simulate internal obstructions, the modified DTM is cloned as a virtual terrain. A new modification layer is created where internal walls are drawn with an elevated height (representing ceiling level). The



wall shapefile from QGIS is imported into RasMapper and traced. A temporary geometry is created and later deleted.

8. Refinement Regions

Refinement regions are defined around internal walls and columns to increase mesh resolution. This is done under Geometries → 2D Flow Area → Refinement Region, setting cell sizes to 0.5 m or 0.2 m depending on feature scale.

For the boundary condition setup, the following steps are needed:

9. Importing Boundary Conditions

Boundary condition lines digitized in QGIS are imported into RasMapper under Boundary Condition Lines → Edit → Import Features from Shapefile.

10. Hydrograph Extraction

Hydrographs are extracted by opening the urban-scale model and using RasMapper to define profile lines. These are created under Features → Profile Lines → Edit → Import Features from Shapefile, using a shapefile that includes boundary conditions.

For Via della Scala, two segments are defined—one parallel and one perpendicular to the street—to facilitate hydrograph extraction. Using “Plot Time Series → Flow,” hydrographs are generated for scenarios where floodwaters reach the building openings observed during on-site inspections.

11. Flow Adjustment and Scenario Testing

Different flow reduction coefficients are applied to simulate mitigation measures such as flood gates or sandbags. For both Tr200 and Tr500 events (when available), 10% of the extracted hydrograph is assigned to the boundary condition.

Additionally, for the Tr200 scenario, two null boundary conditions are set in Piazza S. Pancrazio. The Via della Scala hydrograph is reused, and the simulation duration is set to eight hours. For this scenario the flood depth is about 20 cm in the basement.



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The result for the 500 years return period is shown in Figure 50. For this inundation scenario all the basement is filled-up with flood waters up to the ceiling.

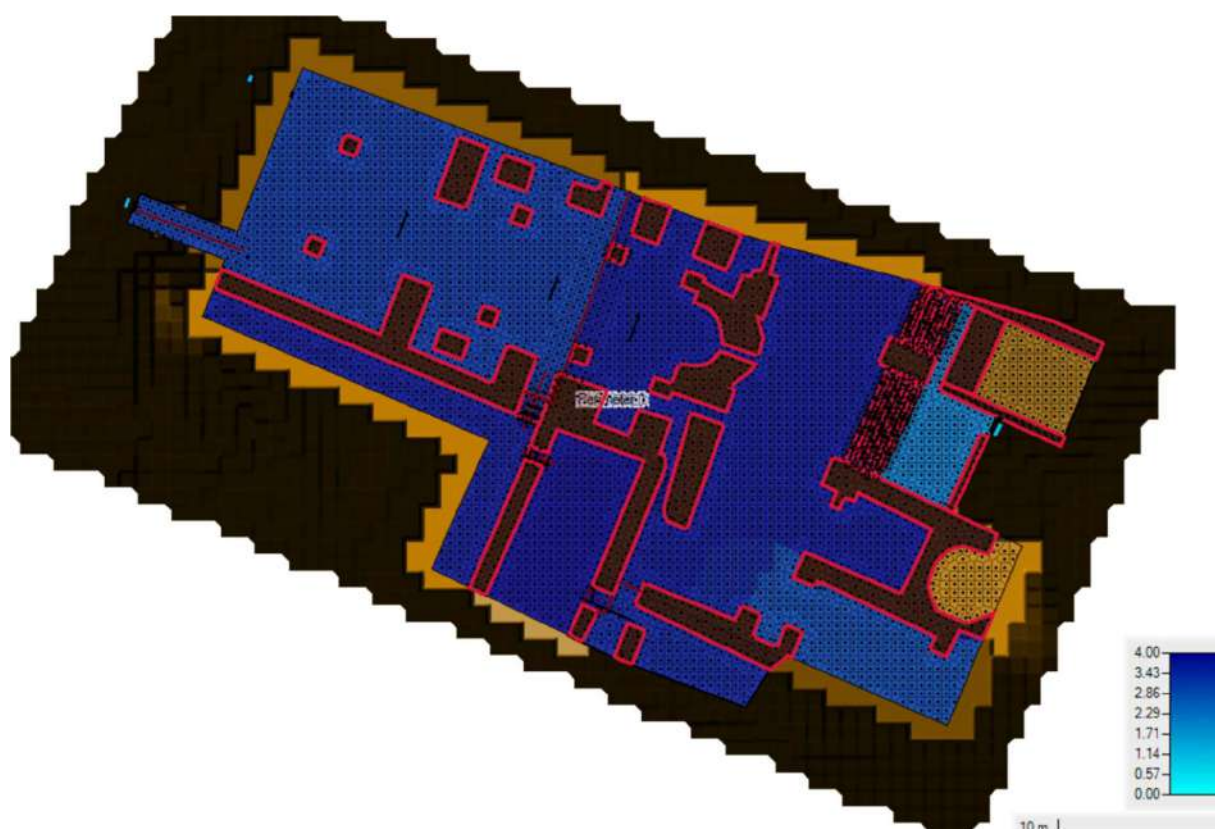


Figure 50. Flood depth inside the former crypt of the Marini Museum for a 500 years recurrence interval (historical 1966 flood scenario).

2.7.4 Flood exposure and vulnerability monitoring

Besides the building envelope which is exposed continuously in case of a flood, the presence of artworks or art installations depends on the temporary exhibition hosted by the museum. The museum hosts in the former crypt 2 artworks by M. Marini that are all sculptures in stone with an overall low vulnerability to water. Therefore, a flood-damage function for the structure can be created easily, while flood-damage functions for the contents can be only produced based on assumptions on the type of artworks material.



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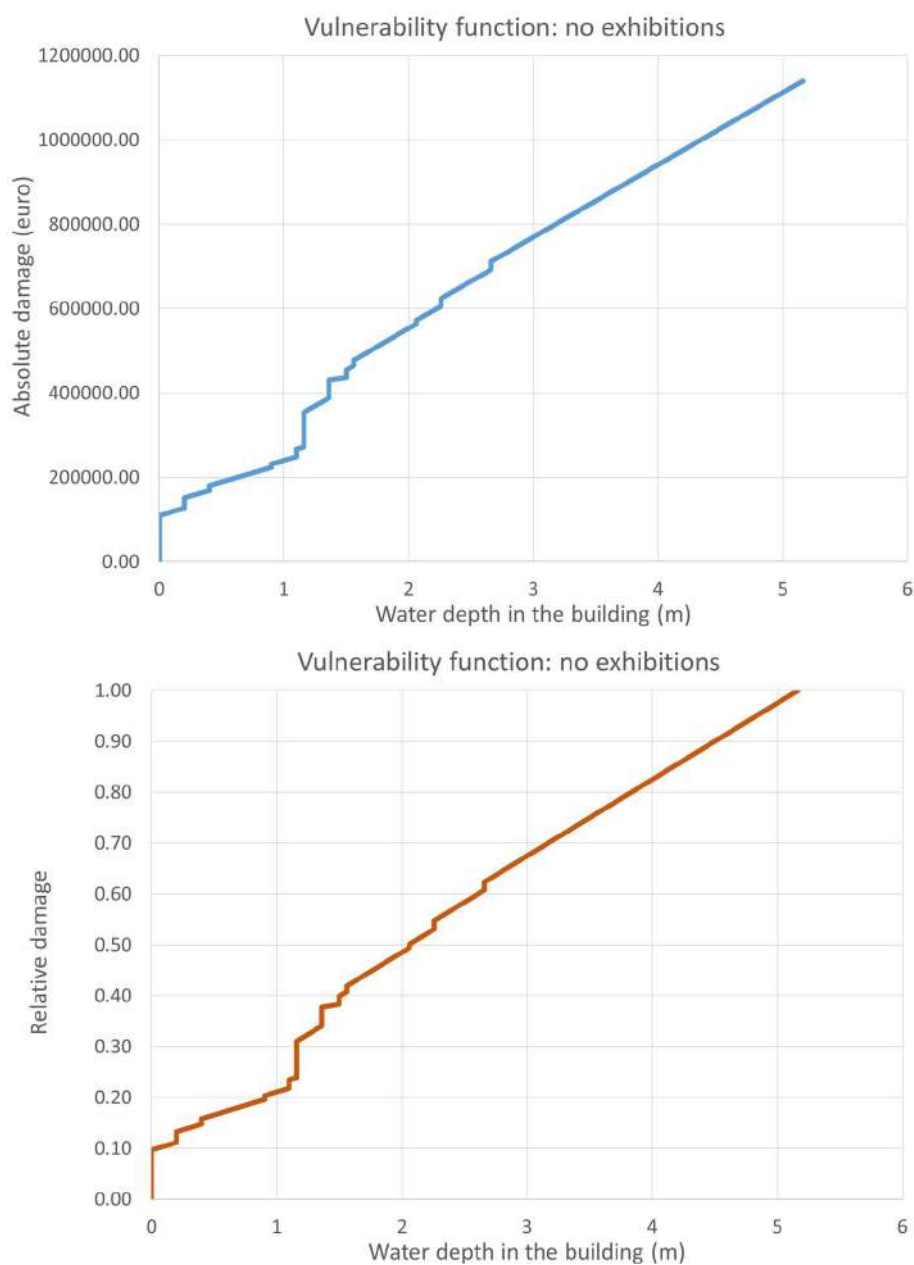


Figure 51. Flood damage function in absolute (top) and relative (bottom) terms.



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For a flood similar to the 1966 event, all the crypt is filled with floodwaters, the estimated impact is therefore close to 100% i.e., ca. 1 million euros only for the structure. The impact considers non-structural impacts, such as deep renovation of wall plastering, finishing, etc.

The damage function for artworks considers the hypothetical type of material possibly exhibited.

Starting from the assumption of about 20 paintings of average size, placed at 1 m from the floor (lower margin) a vulnerability function can be drawn as follows, considering that once the canvas is in contact with water at the lower margin the whole painting must be restored.

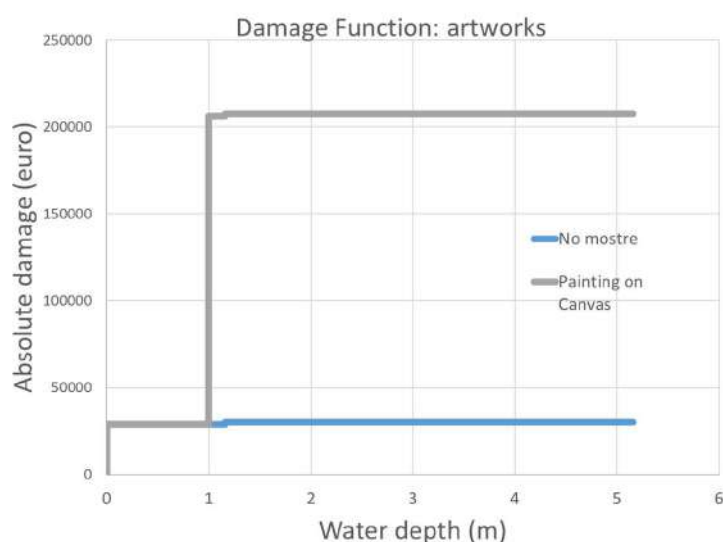


Figure 52. damage function for contents in case of painting exhibition.

In order to better understand and monitor the exposure and vulnerability of the artworks that are not permanently exhibited, research on the type and duration of temporary exhibition in the Marini Museum has been carried out. This allows understanding the dynamic part of exposure and vulnerability to be accounted for.

The following table lists the exhibitions of the past ten years (based on the available data), their start and ending date, duration in months, number of artworks exhibited and a classification of vulnerability of



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artworks based on the materials. Four classes of vulnerability are used from low (e.g., stone or metal artworks) to very high (paper, photographic material, or paintings).

Table 12. Temporary exhibition in the Marini Museum in the last ten years.

Beginning Date	End Date	Name of Exhibition	Exhibition duration (months)	Exposure (n artworks) in crypt	Potential flood Impact to artwork	Potential flood Impact to artworks	Types of artworks (materials)
12/9/2024	7/1/2025	Il Tesoro di Terrasanta al Museo Marino Marini	4	100	very high	4	manuscripts, textiles, metalwork, paintings on wood
5/10/2023	8/1/2024	Andature III Helene Appel / Eva Marisaldi	3		very high	4	paintings, sculptures, videos, drawings
9/6/2023	18/09/2023	Rachel Feinstein in Florence	3	15	very high	4	paintings, sculptures (painted clay)
31/03/2023	29/5/2023	Guardare e aspettare: Enrique Martínez Celaya Sculpture Selezionate 2005-2023	2		high	3	sculptures (Bronze, concrete, wax, wood)
17/12/2022	20/03/2023	Jesper Just – SEMINARIUM	3	6	high	3	multi-screen
24/09/2022	24/10/2022	ANDATURE, II edizione – Elisabetta Di Maggio / Sophie Ko	1		very high	4	mosaic made with paper, clay, soap and vegetable elements. Paintings
23/07/2022	19/09/2022	Dino Ignani. Dark Portraits – Florence/Rome 1982-1985	2	400	very high	4	photos
24/4/2022	6/6/2022	Fons vitae	1.5		high	3	paintings, sculptures, light installation



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11/3/2022	30/05/2022	Pas de deux. Marino Marini Igor Stravinskij	2	50	very high	4	paintings artprints
21/10/2021	17/01/2022	Simbiosi – Una mostra fotografica di Fabio Gambina	3	25	very high	4	photos
3/10/2022	30/10/2021	Andature, opere di Chiara Bettazzi e Daniela De Lorenzo	1	2	high	3	videoinstallation
24/7/2021	18/10/2021	Lumen, fotografie di Clara Melchiorre	3	24	very high	4	photos
1/7/2020	30/09/2020	Di Squali e di Balene Michele	3	2	medium	2	bones, XIX century
18/01/2020	2/3/2020	Ciacciofera, The library of encoded time	2		medium	2	clay, brics, sculptures
7/12/2019	16/12/2019	“Mystic India”. Fotografie di Parul Sharma	0.5	40	very high	4	photos
2/11/2019	2/12/2019	DERNO RICCI – RITRATTI	1		very high	4	photos
5/10/2019	30/12/2019	Nephilim. Una moltitudine di maschere sonore, di Yuval Avital	2	60	high	3	musical sculpture
13/09/2019	27/09/2019	Nancy Burson al Museo Marino Marini per “Firenze Suona Contemporanea”	0.5	5	high	3	glass sculpture, video installation, lighting sculpture
8/7/2019	8/9/2019	AION. Opere di Louise Manzon	2	10	medium	2	metal sculptures
21/01/2019	24/02/2019	EFFETTO MUSEO. Intrusioni istantanee nei luoghi dell’arte. Fotografie di Massimo Pacifico a cura di Claudio Di Benedetto.	1		very high	4	photos
2/6/2017	1/10/2017	YTALIA Energia Pensiero Bellezza. Tutto è connesso.	3	25	high	3	sculptures in clay



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27/02/2016	27/05/2016	TOP HUNDRED.					
		MAURIZIO	3	100	very high	4	books, vynil,
		NANNUCCI					videos

The following figure shows the dynamics of flood exposure and vulnerability based on the previous table. In the absence of exhibition, the vulnerability is low because there are only two statues in stone material from the permanent collection.

The following figures show the trend of the potential flood impact to artworks in temporary exhibitions and their average duration. It can be noticed that as the Museum acquires new value for the community the duration as well as the vulnerability of the exposed artworks is increasing. Typically, the exhibitions take place in autumn-winter and early spring, that are the seasons where the floods in Florence occur.

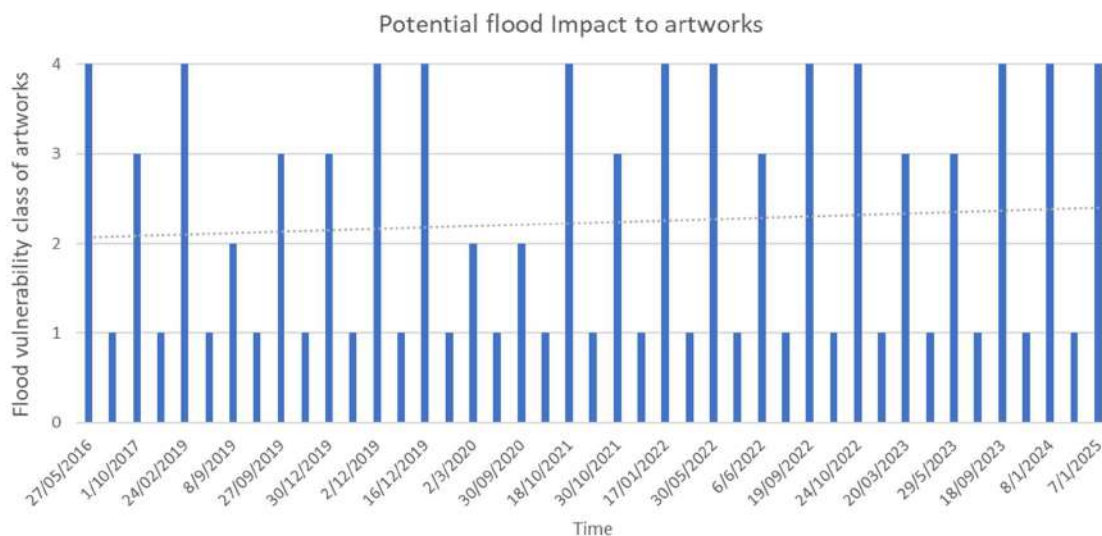


Figure 53. Vulnerability class of the exhibitions of the past ten years in the Museum (1 low to 4 very high).



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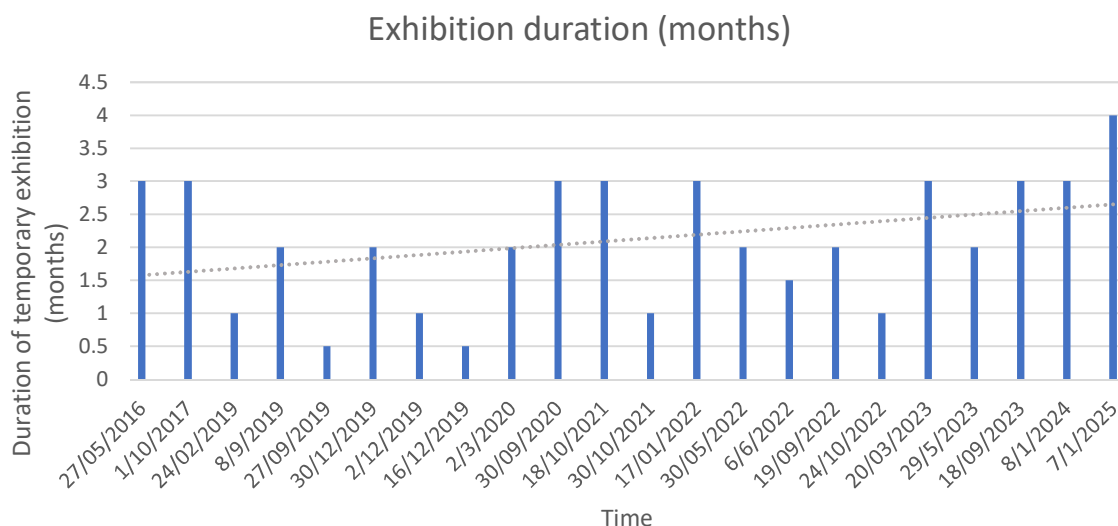


Figure 54. Duration of temporary exhibitions (months)

2.7.5 Flood risk

By considering the flood hazard context in the city of Florence, the building envelope and the permanent collection exposed to floods in the Marini Museum the following impacts are obtained:

- For return periods lower than 200 years the museum is not exposed to floods, because the city center is not flooded.
- For the 200 years return period, the floodwater enters only from via della Spada (emergency exit), the flood depth in the basement is about 0.2 m and the estimated loss is about 200000 euros.
- For the historical 1966 scenario, the floodwater enters from via della Spada and from two openings on the main facade, the flood depth in the basement is about 4.5 m and the estimated loss is about 1.1 million euros
- Overall, the annual average loss for the sole building envelope is ca. 5000 euros per year (equal to 500 full entry tickets).

- The overall risk can be considerably higher in case a flood occurs during a temporary exhibition when the exposed values and vulnerability are much higher (in relation to tangible and intangible values of artworks and specific vulnerability of the material to flood).

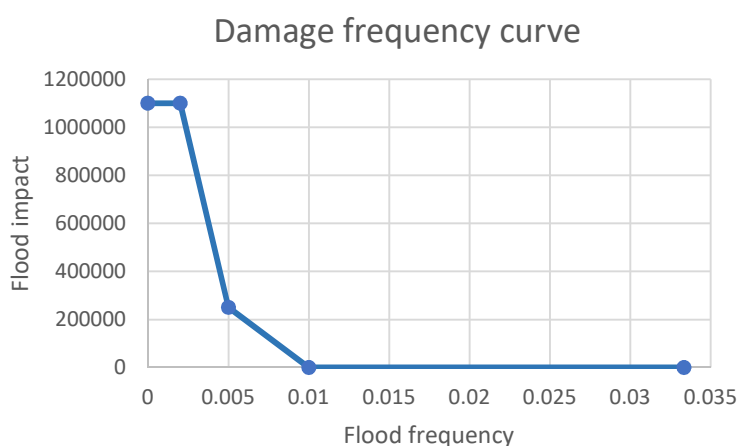


Figure 55. Damage-frequency curve for the envelope of the Museum.

The situation of the Marini Museum in Florence deserves specific attention for the following reasons emerged during the analysis of flood risk:

- The building is located in an area at medium flood hazard (P2) with expected low water depths with respect to other portions of the city centre, however the presence of the basement significantly aggravates the exposure to flooding.
- The basements host temporary exhibitions with insured values up to 50 million euros (as in the case of the last exhibition) and in the last decade the duration of the exhibitions and the vulnerability of exhibited artworks tend to increase, based on the collected data.
- The Museum manager has limited awareness of flood risk in the area and limited confidence with the early warning procedure. Moreover, there isn't a contingency plan for the temporary exhibitions.



The possible flood risk mitigation and adaptation measures are:

- Increase flood risk awareness among employees and managers and prepare a contingency plan for floods, including training activities for emergency; the increase understanding of flood hazard can be supported by 3D visualization, augmented reality and HBIM.
- As the ingress points of flood waters cannot be sealed, a dry waterproofing is not possible, therefore mitigation should involve mainly the reduction of exposure and vulnerability components
- Re-design exhibitions in a risk informed way and limit the presence of fragile materials in the basements.



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3 CH monitoring at urban scale – Pienza historical center

3.1. The proof of concept (DIDA – UNIFI)

The historical center of the Town of Pienza, located in the Province of Siena in Tuscany (IT) has been selected as proof of concept for MR analysis (Figure 56). Recognized as a UNESCO World Heritage Site since 1996, the historic center is notable for the group of Renaissance buildings surrounding the main square, as well as for the overall integrity of the surrounding urban fabric. According to the UNESCO World Heritage website, the historical center meets the following criteria:

Criterion (i): The application of the principle of the Renaissance “ideal city” in Pienza, and in particular in the group of buildings around the central square, resulted in a masterpiece of human creative genius.

Criterion (ii): The Historic Centre of Pienza, as the first application of the Renaissance humanist concept of urban design, was to play a significant role in subsequent urban development in Italy and beyond.

Criterion (iv): The buildings surrounding Pienza’s central square are an outstanding example of Humanist Renaissance design.



a)



b)

Figure 56. a) View of the historical center of Pienza; b) the central square with the Dome and Piccolomini Palace.

In addition to its universal value, the Historic Centre of Pienza retains a high degree of authenticity in its urban fabric, architectural elements, and spatial organization, reflecting the original Renaissance vision.



The integrity of the site is ensured by the preservation of its essential features and the harmonious relationship between the central square and the surrounding urban structure. The protection and management of the property are governed by established legal frameworks and planning instruments, which safeguard its cultural values and ensure the long-term conservation of its historical, architectural, and social significance.

In the MR analysis of the selected case study, the research considered two distinct risks, i.e. the landslide risk and the seismic risk. Considering the interrelations between these two natural sources, differently from other hazards which are strictly independent, three distinct cases, from two alternative approaches can be considered. In the first approach, each risk is treated separately and independently: the MR is obtained combining the two different contributions, e.g. in terms of physical damage or economic losses. In the second approach, the risks are assumed in consequential steps, where the effects of the first source alter the outcomes of the second. Two distinct scenarios come from ordering landslides and earthquakes to one another. A landslide can be caused by the sudden ground motion given due by an earthquake, inducing the collapse of a certain mass of earth or rock from a slope (accelerated landslide). These evaluations consider the so-called cascading triggering. Differently, the slow movement at the building' foundation induced by subsidence of the ground can induce structural damage to the buildings, increasing their vulnerability towards seismic actions. The procedure carry out to account on these combined effects will be further discussed. Dealing with the MR analysis, the selected monitoring activity conducted at the urban scale level regards the landslide risk, and it is based on the usage of satellite interferometry measurements on the built environment as detailed in the following sections.

3.2. Survey (DIDA/DST – UNIFI)

3.2.1 Historical evolution of the urban center

The center of Pienza has ancient origins. During the Renaissance it was known as Corsignano, and it constituted a simple village in the countryside on the southern territory of Siena. The historical evolution



of the city changed with the birth of Enea Silvio Piccolomini in 1405, that 53 years later became Pope Pio II. During his pontificate, he decided to build a new ideal city on top of the ancient village, entrusting the renovation project to the architect Bernardo Rossellino. Starting in 1459, the work lasted about four years, leading to the creation of a harmonious town aspiring to the ideal city. With the premature death of Pope Pio II the renovations activities stopped, and the new city was named after him (Pienza means “city of Pio”). After these events, the town has undergone few changes maintaining his authentic characteristics.

3.2.2 Definition of structural units and realization of a GIS database

The analysis of the historical center has been carried out through the usage of Geographic Information System environment (GIS), able to recollect the different information coming from hazard/susceptibility maps, monitoring, urban stock etc.

Pienza historical center is mainly composed of aggregated masonry structures. Houses are mostly realized through continua urban clusters, where structures interact influencing reciprocally their behaviour for gravitational and seismic actions (Figure 58). Referring to the current definitions reported in buildings codes, a structural unit (SU) is identified as a homogenous construction with a uniform behaviour from the ground to the roof [22]. The US can be delimited by open spaces or structural joints, otherwise, it can be in the middle of adjacent structures.

As the target of the analysis are the single SUs, a specific layer with the geometrical information of the different units and their localization is required. The database has been made starting from the spatial information available in the regional website Geoscopio Regione Toscana (<https://www502.regione.toscana.it/geoscopio/cartoteca.html>), which provides planar shapefiles of the urban environment considering anthropomorphic and natural entities. The documentation referred to constructions is not specifically addressed to structural units, but it follows the geometrical shapes of roofs, subdividing the geometries in a higher number of polygons. As a result, a data cleaning effort was required, along with the creation of the specific geometries of interest. In this research, the special buildings such the monumental Palazzo Piccolomini or the Dome have not been considered, as they



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represent exceptional constructions that would require specific and detailed analyses beyond the purposes of this project. On the contrary, aiming to conduct an urban scale research, the historical center of Pienza has been studied adopting empirical vulnerability models specifically calibrated for standard constructions. In panel a) from Figure 57 the aerial view of historical center of Pienza is shown. Although the presence of more recent expansions on the northern and western side, the old village is clearly recognizable in terms of shapes, configurations, and dimensions. In panel b), the geometrical shapes realized for the different structural units considered in the research are shown. This distinction has been carried out through external visual inspections, observing the detachments between units and in absence of drawings or cadastral information on the constructions.



Figure 57. a) aerial view of the historical center of Pienza; b) realization of the geometrical shapes for each SU considered in the research.



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Figure 58. View of the historical center of Pienza with the aggregated masonry buildings composing the urban cluster.

3.2.3 Geological and geomorphological background

The historic centre of Pienza lies on a gently sloping plateau at around 500 metres above sea level. This elevated area, mainly made up of Pliocene marine sediments, forms a natural ridge between San Quirico d'Orcia and Montefollonico, separating two large tectonic basins: the Siena Basin to the north and the Radicofani Basin to the south. The ridge is built on an older geological structure, where sediment thickness is reduced compared to the deeper surrounding basins.

The tectonic setting is defined by a system of asymmetric faults trending NNW–SSE, with major displacements on the eastern side (e.g., Mt. Cetona Fault) and gentler steps on the western side. Within the urban core, two major inferred faults—known as the Pienza Fault (WNW–ESE trending, south-dipping)

and the Fontanelle Fault (NW–SE trending, north-dipping)—delimit zones of differential stability. These fault structures have played a crucial role in controlling both sedimentation and the ongoing deformation processes that affect the stability of the historic settlement.

The stratigraphic succession beneath Pienza reflects a marine regression sequence, transitioning from deep to shallow marine and eventually to more continental conditions:

Recent deposits:

- Val d'Orcia Clays (FAAo): compact clays from deep marine settings.
- Pienza yellow sands and sandstones (PLIs) rich in marine fossils.

Bedrock:

- Organic limestone (PLIc) with Lithothamnion fossil remains, located beneath the historic centre.
- Sands and sandy clays (FAAb) with brachiopods, indicating shifting marine conditions.
- Val Tuoma clays (FAAt): soft clays prone to landslides, particularly in northern parts of the town.

The terrain gently slopes northward, while the southern side is steeper and more affected by ground instability. This southern escarpment shows various signs of gravitational movement, such as deep-seated slope deformations, rotational landslides, and slow clay flows, especially in areas with weak soils or near faults. From a hydrogeological point of view, the central part of the ridge is relatively stable due to the low permeability of the clays, which limits water infiltration. The outer edges, where sands and gravels are more common, are more sensitive to water-related processes.



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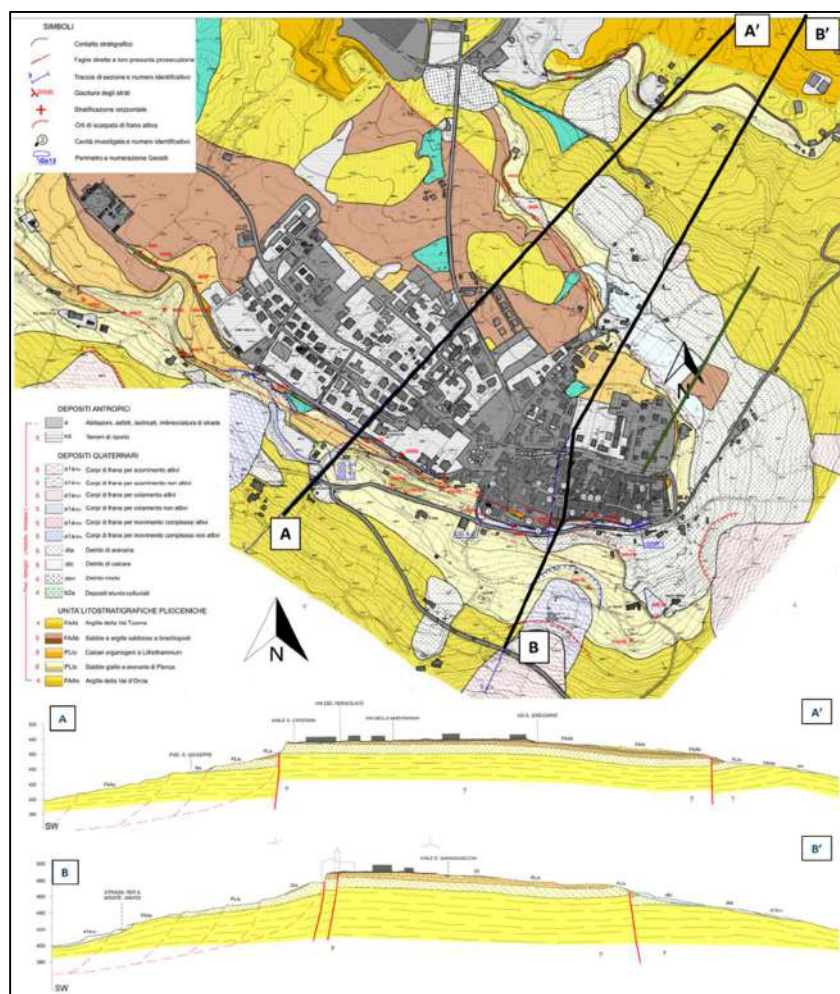


Figure 59. Geolithological map, legend, and symbols: geological map and cross-sections (from the Structural Plan of the Municipality of Pienza).

3.2.4 Seismic microzonation

This seismic microzonation (MS) study forms an integral part of the geological investigations supporting the Municipal Operational Plan (Piano Operativo) of Pienza, in accordance with the regional regulation DPGR 53/R (October 25, 2011). The Level 1 MS study follows national and regional guidelines, using



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standardised procedures for the collection and analysis of geological, geophysical, and geotechnical data to identify seismically homogeneous microzones within the urban areas of Pienza and Monticchiello.

Pienza is located on a gently sloping plateau at approximately 500 meters above sea level, resting on Pliocene marine-coastal sediments. This geomorphological surface is part of a structural ridge extending from San Quirico d'Orcia to Montefollonico, separating the T. Tuoma and T. Tresa streams. The area of Pienza, from the year 1000 to 2017, was affected by numerous earthquakes with peaks of 7-8 maximum epicentral intensity (equivalent magnitude 5.3) recorded in damaging events in 1545 and 1940. The most significant tectonic feature is the “Pienza Fault”, a set of ONO–ESE trending normal faults responsible for structural damages observed in the historic town center and in the Cathedral of Pienza.

Based on historical deformation data and ground monitoring (precision levelling from 1983–1992), the Pienza Fault is classified as potentially active and capable, given the detected vertical displacements and geomorphological evidence of slope deformation.

The main fault crossing the town of Pienza is detectable through surface mapping and structural damage in buildings along its trace. It is clearly visible along Via Santa Caterina, where a fault scarp shows a vertical offset of about 12 meters, decreasing to 4 meters west and 3.5 meters east of the Cathedral. The fault has a WNW–ESE orientation, dips southward, and is nearly vertical.

The geological substratum consists of stratified Pliocene deposits underlain by Mesozoic formations. Key lithostratigraphic units include:

- Val d'Orcia Clays (FAAo)
- Pienza Yellow Sands and Arenites (PLIs)
- Calcareous Lithothamnium Limestone (PLIc)
- Brachiopod-Bearing Sands and Clays (FAAb)
- Val Tuoma Clays (FAAt)

These formations display a range of mechanical behaviours, with lithified units (PLIs, PLIc) forming favorable foundation conditions in the central sector of the town, while clayey units (FAAo, FAAt) are prone to instability, especially in the northern and southern slopes.



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From a structural standpoint, two main fault systems characterize the area. The most prominent is the Pienza Fault, which shows evidence of ongoing, albeit slow, displacement and has been linked to damage in the historic center. The second is the Fontanelle Fault, a secondary structure with a NE–SW orientation and limited vertical offset, located to the north of the urban core. These structural features divide the territory into three morpho-structural sectors. The southern sector is the most unstable, exhibiting signs of active gravitational deformation and landslide processes. In contrast, the central sector, which includes most of the historic center, is relatively stable and underlain by competent lithified Pliocene formations, Figure 60. The northern sector, meanwhile, is more vulnerable due to the prevalence of clay-rich formations and evidence of solifluction and minor slope instability.

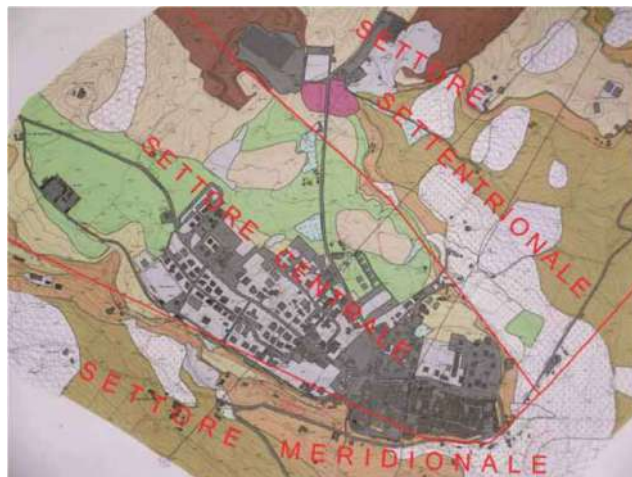


Figure 60. The three sectors, with different morphostructural characteristics, in the Pienza area [23].

From a geomorphological perspective, the area shows widespread gravitational slope deformation. This includes both rotational and translational landslides, especially in areas where sandy and clayey units interface, as well as zones of solifluction, typically found in the northern clayey sectors. Moreover, deep-seated gravitational slope deformations (DGPVs) have been identified, particularly along the southern slopes of the town near Porta al Giglio and Podere La Chiusa.

The subsoil model of Pienza was developed using an integrated approach that included historical borehole data, new geophysical investigations (such as MASW and HVSr surveys), and detailed geological mapping. This analysis allowed the identification of four primary stratigraphic zones. The historic center sits on a

foundation of well-cemented Pliocene formations, such as lithothamnium limestone and yellow arenaceous sands, which provide favorable geomechanical properties despite the presence of faults. In contrast, the surrounding slopes show a progressive transition to softer and more unstable clayey formations that are prone to landslides.

As part of the Level 1 seismic microzonation study conducted for the Municipality of Pienza, the urban territory was subdivided into homogeneous seismic response units, commonly referred to as MOPS (Microzonazione Operativa per la Pianificazione Sismica). These zones are defined based on consistent geological, geomorphological, and geotechnical characteristics that determine their expected seismic behaviour. The study identifies several microzones with distinct seismic potential, primarily influenced by the local lithostratigraphy and the presence of structural discontinuities such as faults and landslide-prone areas. The following categories of seismic response units were implicitly delineated:

- Stable zones: Found mainly in the central area of Pienza's historic center, underlain by well-cemented Pliocene formations (e.g., Lithothamnium limestone and yellow sands), with low amplification potential.
- Potentially unstable zones: Located along the northern and southern flanks, characterized by clayey deposits prone to solifluction and landslide processes, especially where slope gradients increase.
- Zones affected by active or capable faults: These include sectors near the Pienza Fault, particularly the southern margin of the town, where fault activity and deep-seated gravitational slope deformation (DGPV) are evident.
- Anthropized areas: Including regions affected by landfills, cut-and-fill slopes, and former or active clay quarries, where ground heterogeneity may contribute to localized seismic amplification or differential settlement.



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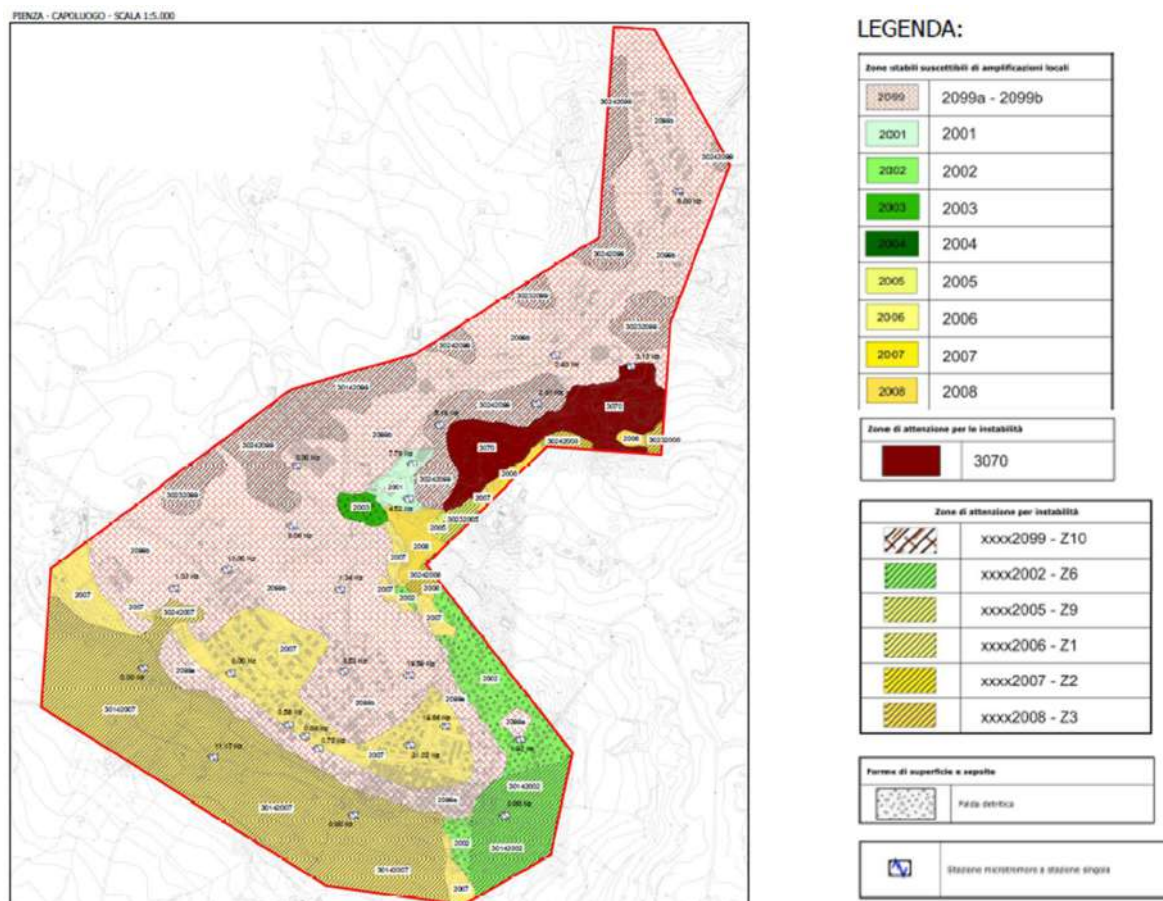


Figure 61. Image taken from "Carta delle MOPS", Comune di Pienza, Regione Toscana.

These MOPS delineations provide the basis for assigning different levels of seismic hazard to urban planning zones and guiding risk-informed urban development, particularly in areas where site amplification or ground failure may occur during seismic events, Figure 61.

The results of this study, supported by geological, geophysical, and geomorphological evidence, have led to the delineation of microzones with distinct seismic responses. The identification of potentially active faults and deep-seated slope deformations underscores the necessity for careful planning, particularly in the southern and eastern sectors of the town. These findings provide a crucial scientific foundation for



seismic risk mitigation, informed land-use planning, and the design of structural reinforcement measures within the municipality of Pienza.

3.3. Monitoring strategy (DST – UNIFI)

3.3.1 Ground motion analysis through satellite SAR Interferometry

- *Methodological procedure*

The ground motion analysis follows a methodological framework based on multi-temporal satellite radar interferometry, specifically Persistent Scatterer Interferometry (PSI) technique. This method allows the detection and quantification of slow deformations of the ground and the built environment with millimetric accuracy, by using non-invasive remote sensing techniques. It supports systematic and scalable monitoring of Cultural Heritage at the urban scale, fully aligned with the principles of non-invasiveness, repeatability, and continuity essential for the management and preservation of historic sites.

PSI exploits the phase information from long temporal series of Synthetic Aperture Radar (SAR) images to measure displacements along the Line-Of-Sight (LOS) of the satellite. It identifies Persistent Scatterers (PS) — typically architectural structures or natural features (e.g. buildings, walls, rock outcrops) — that maintain stable backscattering properties over time and provide reliable targets for measuring slow ground movements. This technique ensures accurate deformation measurements especially in urbanised areas where many radar benchmarks can be retrieved. To extend coverage in less urbanised or vegetated sectors where PS density is lower, the analysis employs the SqueeSAR® processing algorithm. This approach integrates Distributed Scatterers (DS): radar targets corresponding to homogeneous surface areas with moderate but stable radar backscatter, such as debris, bare soil, or low vegetation. The combined use of PS and DS enables detailed mapping of deformation patterns across the entire study area.

For the Pienza case study, COSMO-SkyMed (CSK) X-band SAR imagery acquired in Stripmap mode was processed. The high spatial resolution (3×3 m) provided dense measurement point distributions, exceeding 7,000 PS/km². Data from both ascending and descending orbits in the spanning time 2015–2023 were analysed, with an average revisit time of 16 days. The dual-geometry configuration enabled the decomposition of LOS velocities into vertical and east–west horizontal components, enhancing the interpretability of displacement patterns in relation to topography and structural features. Data processing produced:

- Maps of mean LOS velocity (mm/year).
 - Maps of vertical and horizontal components of the mean velocity derived from combined ascending and descending datasets.
 - Ivmean index maps, representing the average absolute LOS velocity of radar targets associated with each building, classified into stability classes to support exposure assessment at the urban scale.
 - Cumulative displacement time series for individual PS and DS, used to analyse deformation trends and assess potential correlations with rainfall data.
 - Spatial gradient maps of deformation velocity obtained from the interpolated vertical component of the mean velocity, used to identify zones of differential motion that may pose structural risks.
- *Mean LOS velocity maps (ascending and descending orbits)*

The PSI analysis of CSK X-band data, covering the period 2015–2023, provided mean annual velocity maps for both ascending and descending orbits. The spatial distribution of deformation shows that most of the historic center of Pienza is stable, with LOS velocities within the threshold of ± 1.5 mm/year, corresponding to the expected stability range for X-band data. Localised areas of significant deformation were identified mainly along the southern slope of the urban plateau, near the Cathedral, where velocities reached up to



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-3.1 mm/year in the LOS direction (indicating downward movement). The ascending and descending datasets both confirmed these patterns, with strong agreement between the two acquisition geometries.

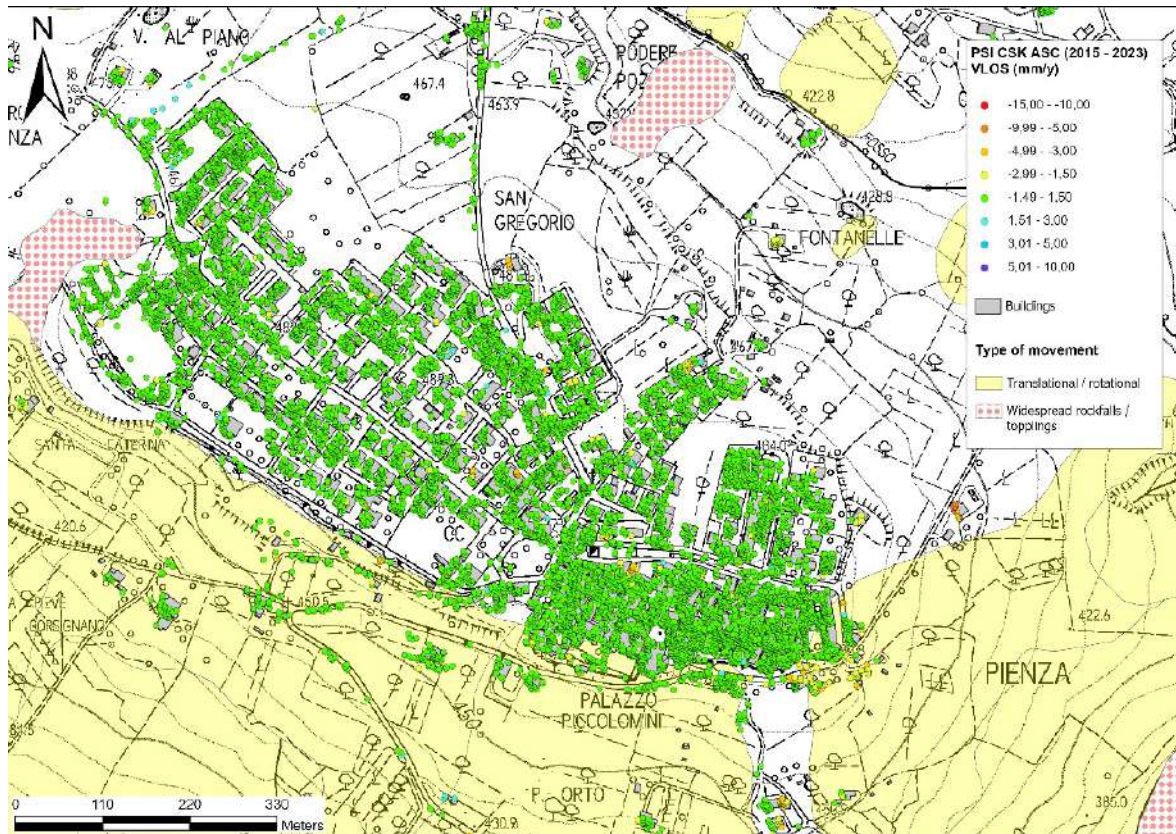


Figure 62. COSMO-SkyMed ascending data (2015–2023) on CTR base map (1:10000).



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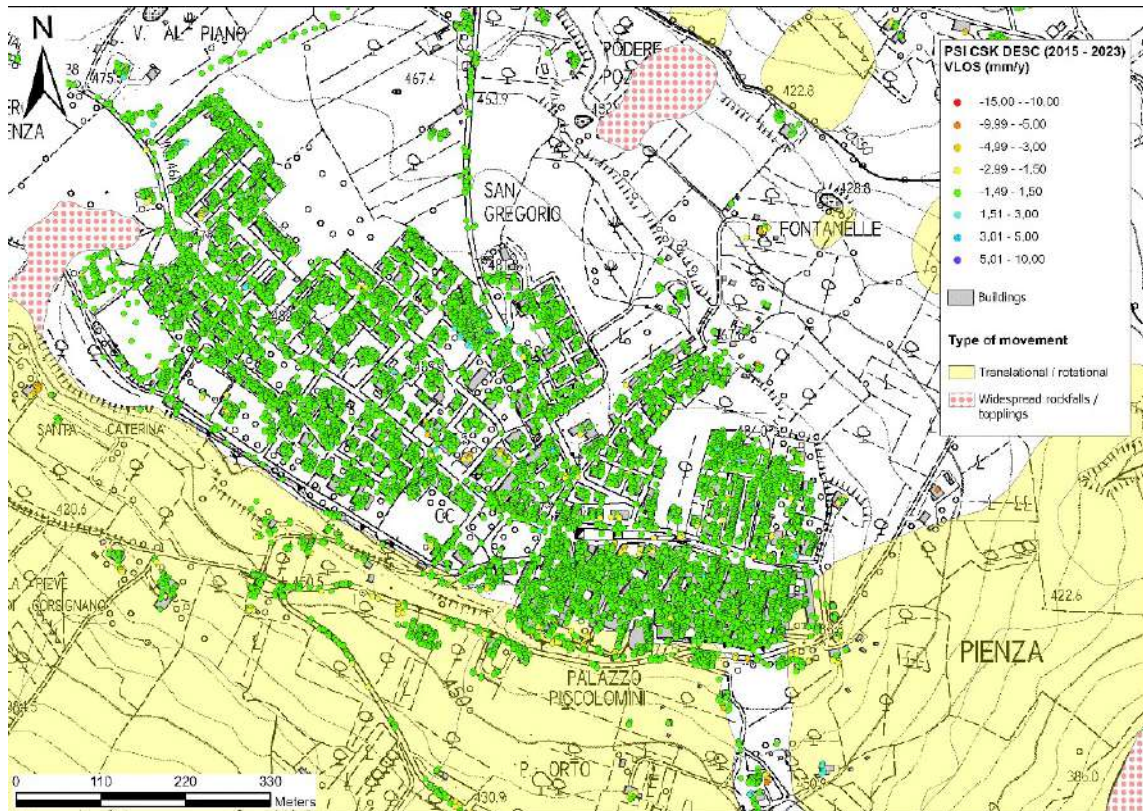


Figure 63. COSMO-SkyMed descending data (2015–2023) on CTR base map (1:10000).

- *Mean velocity maps (vertical and horizontal components)*

The decomposition of displacement into vertical and horizontal (E–W) components allows for a rapid characterisation of the kinematics of ground movement. The horizontal component is predominantly eastward at the south-eastern edge of the town, consistent with the local morphology. Moreover, the southern area shows evidence of downward movement occurring exclusively in the vertical direction.

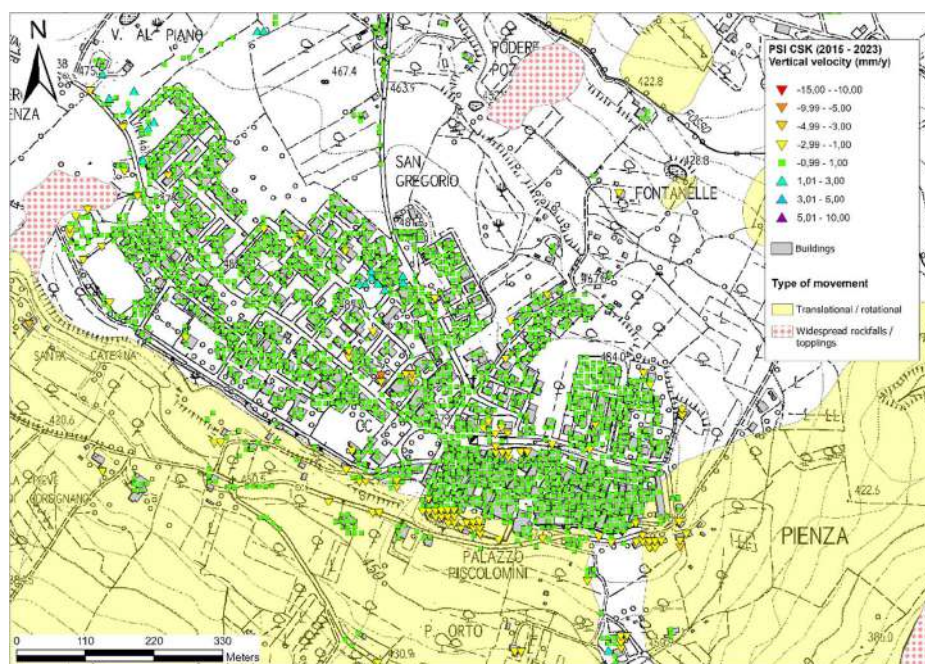


Figure 64. Synthetic CSK PSI data showing the vertical component of the mean annual velocity on CTR base map (1:10000).

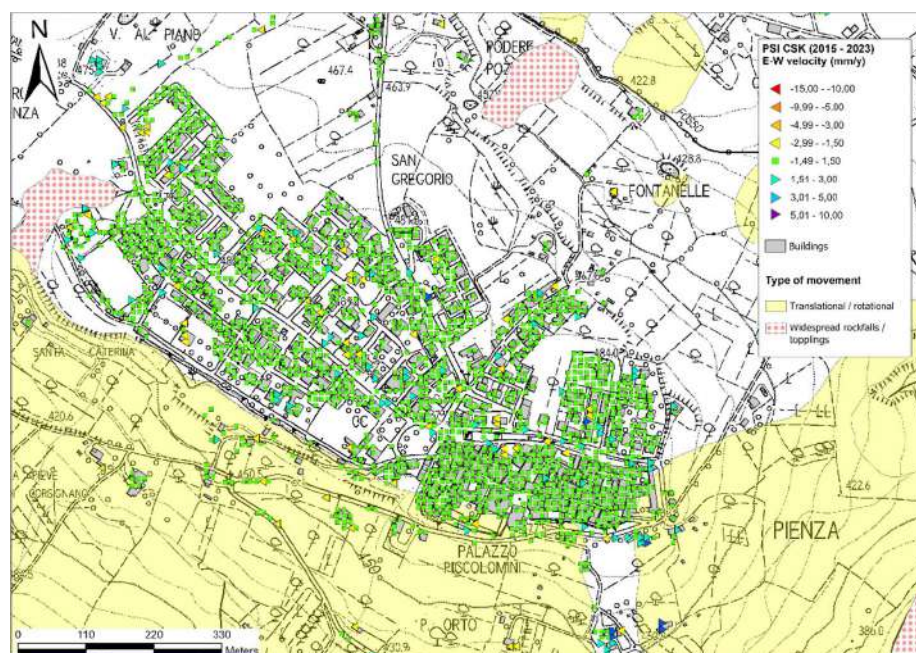


Figure 65. Synthetic CSK PSI data showing the horizontal component of the mean annual velocity on CTR base map (1:10000).

- *Mean velocity index (lvmean) map*

In urban environments, when CSK high-resolution PSI data reveal displacements affecting buildings, a classification procedure is applied to assess their stability, focusing on the computation of the mean velocity index (lvmean). lvmean represents the average absolute LOS velocity (in mm/year) of PS located within a 3-meter buffer around each building. This buffer, calibrated to the spatial resolution of CSK StripMap imagery, accounts for possible geolocation mismatches and ensures that relevant PS data—potentially located just outside the building footprint—are included in the analysis. To ensure the reliability of the lvmean calculation, buildings must include at least one radar target (PS) within the buffer zone. Buildings without any PS are excluded from classification and are classified as No Data (ND). lvmean values are grouped into five velocity classes, ranging from Class E (stable, <1.5 mm/year) to Class A (high deformation, >10 mm/year). The results show that the historic centre is largely stable:

- 324 buildings (approximately 77%) fall into Class E (stable condition, velocities within ± 1.5 mm/year).
- 41 buildings are classified in Class D, indicating minor deformations.
- 4 buildings fall into Class C, representing moderate deformation levels.
- No buildings were assigned to the more critical Class B or Class A categories.
- 52 buildings (mainly in peripheral areas) had no PSI data coverage and are marked accordingly.



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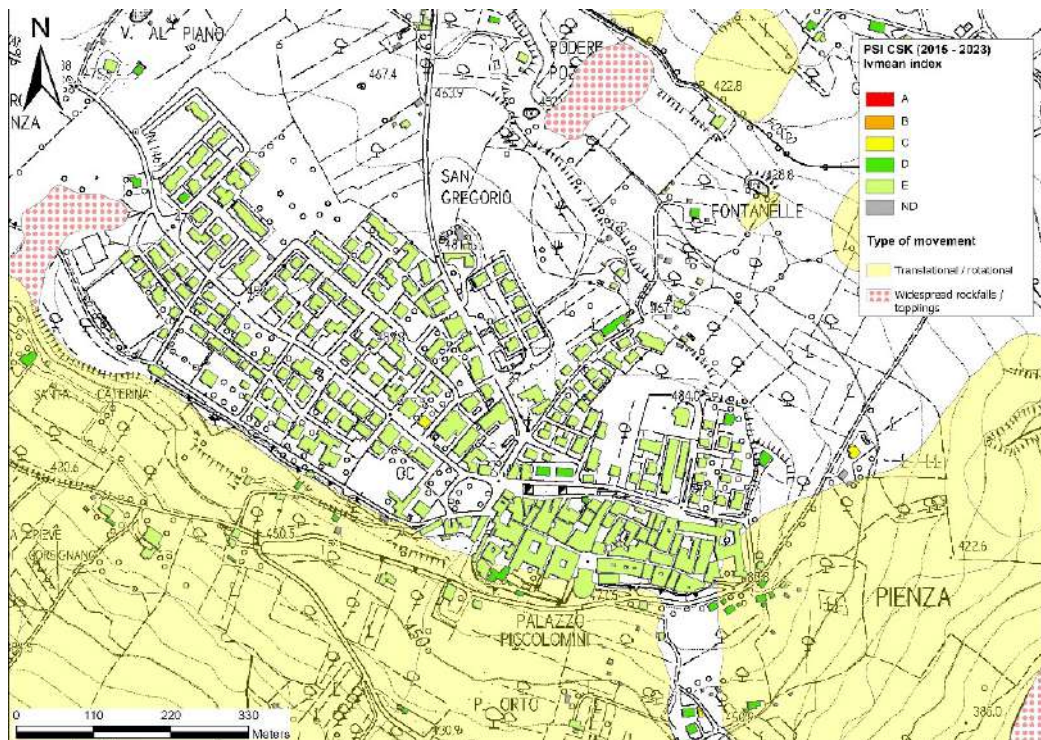


Figure 66. Map of building velocity index (lvmean) on CTR base map (1:10000).

- *Ground motion time series and rainfall correlation*

As part of the multi-temporal analysis, ground deformation time series derived from CSK PSI data were examined to explore possible correlations with local precipitation patterns. This step aimed to assess whether rainfall could be a triggering or contributing factor to the detected ground displacements within the historic centre and its surroundings.

Rainfall data were obtained from the Monticchiello weather station, managed by the Regional Hydrological Service of Tuscany (SIR). This station, located approximately 5 km southeast of Pienza, provides validated daily and monthly precipitation records through a dense and reliable monitoring network. The correlation analysis focused on PSI points classified as unstable, that is, those with mean deformation velocities greater than 1.5 mm/year or lower than -1.5 mm/year. Both Pearson's correlation

coefficient (r), to test linear relationships, and Spearman's rank correlation coefficient (ρ), to assess monotonic trends, were computed between cumulative displacement and precipitation time series. Results indicated very low correlation values across all datasets (ascending, descending, vertical components), showing no statistically significant association between precipitation events and deformation trends. A cross-correlation analysis was also performed, considering time lags of up to 30 days to identify potential delayed deformation responses to rainfall. This analysis did not reveal any consistent patterns, as correlation values remained low and oscillatory, with no clear peak indicating delayed effects.

These results support the interpretation that ground deformations in Pienza are predominantly driven by geological and structural conditions (e.g. lithological heterogeneity, fault systems), rather than by direct rainfall-induced processes within the temporal scales investigated.

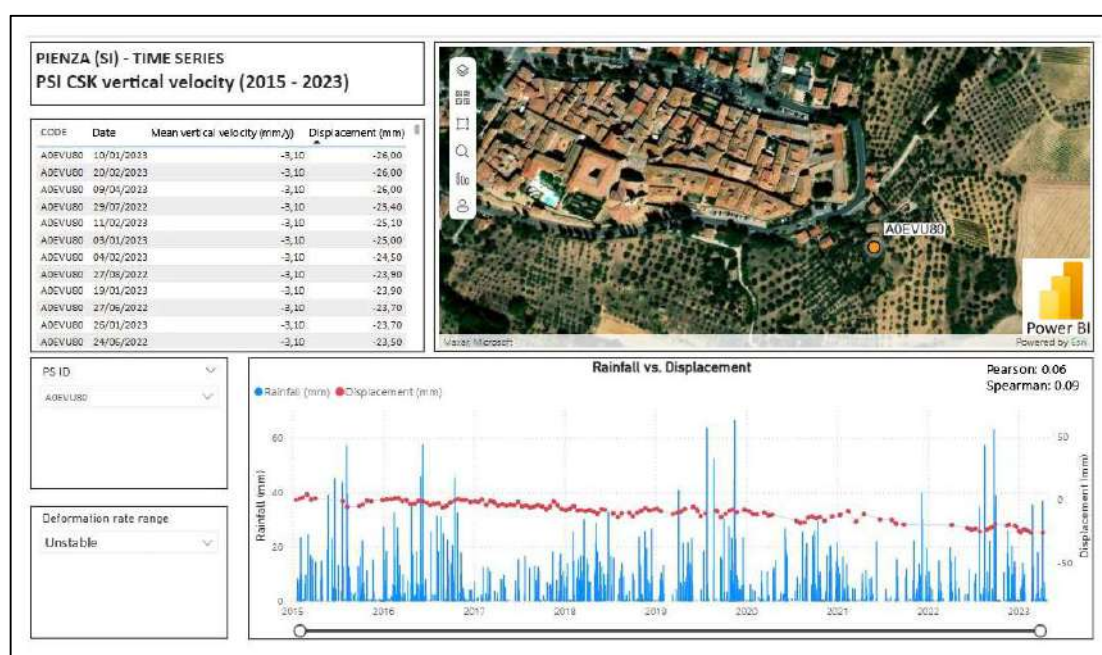


Figure 67. Vertical component of the mean annual velocity vs. rainfall correlation analysis on Power BI Report.



- *Spatial gradient analysis of deformation velocity*

To better characterise the spatial variability of ground motion and to identify areas of differential displacement that may pose a hazard to structures, spatial gradients of deformation velocity were computed using the vertical component of the mean annual velocity derived from CSK PSI data, following the methodologies proposed by Shahbazi et al. [24] (2024) and Barra et al. [25].

To enhance the analysis, the vertical velocity data were resampled to a 5×5 m grid, improving spatial resolution and interpretability, and increasing precision in identifying deformation gradients. The dataset was interpolated using the Inverse Distance Weighting (IDW) method to generate continuous velocity surfaces, from which the slope (percent rise) was calculated and converted into physical units (mm/year/m). The resulting spatial gradient maps highlight zones of significant differential motion, where variations in ground velocity between adjacent points may indicate structural stress or subsurface heterogeneities. As a result, several areas exhibiting pronounced spatial gradients were identified in the southeastern sector of the historic center.

A detailed comparison was conducted between the gradient maps and in situ monitoring data, particularly in the area of the Cathedral. Field measurements obtained from crack gauges, inclinometers, and laser distance sensors confirmed the presence of ongoing deformations, especially along the southern apse and



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transept. Long-term crack monitoring (2004–2021) recorded progressive opening trends that are consistent with the spatial gradients derived from PSI data, thereby validating the satellite-based analysis.

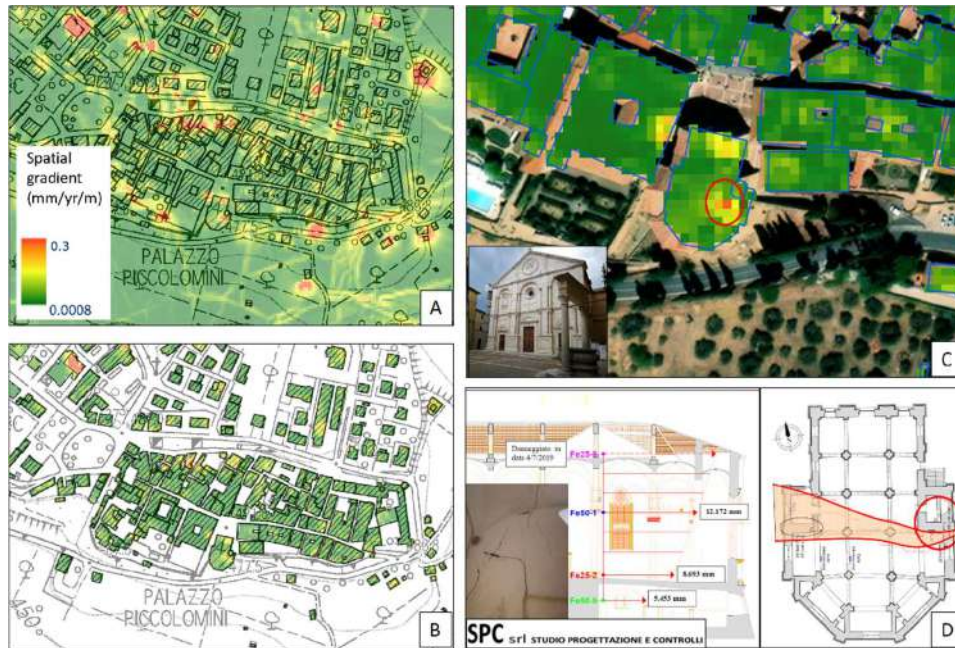


Figure 68. (A) Spatial gradient map (mm/yr/m) derived from CSK PSI data, highlighting zones of differential ground motion. (B) Same map as in (A), masked to show only affected buildings. (C) Zoom on the southern transept of the Cathedral, where the highest gradients are concentrated (red circle). Inset: frontal view of the cathedral. (D) Structural monitoring data (2004–2021): crack gauges confirm active deformation consistent with PSI-derived gradients.

3.4. Landslide risk analysis (DST – UNIFI)

The landslide risk assessment conducted in this work follows a GIS-based procedure proposed by Bianchini et al. [26], integrating PSI-derived ground deformation data with hazard zoning and building-scale exposure information. In this framework, risk is defined as Specific Risk, i.e. the product of hazard and vulnerability, where vulnerability is a function of both the exposure (based on building typology) and the intensity of ground movement, represented here by the I_{vmean} index derived from CSK PSI data.



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PSI velocity measurements are a valuable source of spatially distributed, high-precision ground motion data that can be directly used to support the intensity zonation, a key component for estimating vulnerability in landslide-prone areas. Unlike traditional approaches that rely only on geomorphological proxies or qualitative inventories, the use of measured ground motion velocities enables a more dynamic and data-driven evaluation of risk, especially in urban contexts where even low-magnitude, slow-moving processes may lead to significant cumulative effects on structural integrity.

The spatial hazard component corresponds to the landslide susceptibility classes defined by the PAI (Piano di Assetto Idrogeologico), which provides a territorial zoning into hazard levels based on geomorphological and historical landslide data.

The vulnerability was derived by combining two elements:

- The exposure, based on the functional typology of each building (e.g., residential, agricultural, religious, etc.), reflecting the potential consequences of damage depending on use;
- The intensity, represented by the I_{vmean} index that provides a proxy for the structural response to ongoing slow ground deformation.

The cross-comparison of hazard classes and Vulnerability classes was performed by means of a contingency matrix shown in Table 13.

Table 13

		Vulnerability class				
		V0	V1	V2	V3	V4
		Vulnerability class				
Hazard class	H0	R1 V0	R1 V1	R1 V2	R2 V3	R2 V4
	H1	R1	R2	R2	R2	R4
	H0	R1	R1	R1	R2	R2
	H2					
	H1	R1	R2	R3	R3	R4
Hazard class	H3					
	H2	R1	R3	R3	R4	R4
	H4	R2	R3	R4	R4	R5
		H3				
		H4	R2	R4	R4	R5



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Table 13. Contingency matrix for deriving landslide specific risk.

By integrating these components within a GIS-based environment, each building within the study area was assigned a qualitative risk class, enabling the generation of the Risk map of the Pienza Urban heritage and thus the identification of critical structures potentially affected by landslide-related deformation.

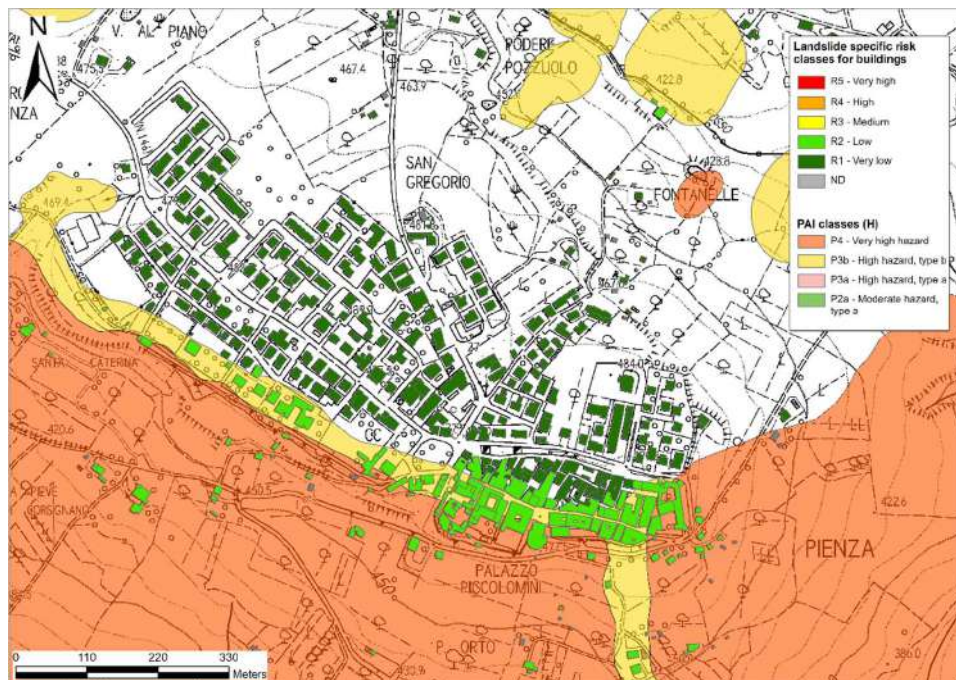


Figure 69. Landslide specific risk classification of buildings in the historic center of Pienza on 1:10000 base map (CTR).

The use of official hazard maps (PAI) as the hazard component ensures consistency with existing planning frameworks, while the classification of buildings by use and deformation rate enables the identification of priority areas for risk mitigation and monitoring. The approach is modular, reproducible, and adaptable to different SAR datasets, making it suitable for application in other heritage-rich or geologically sensitive urban settings



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3.5. Prior seismic risk analysis (DIDA – UNIFI)

Seismic risk analyses are conducted according to two distinct configurations: a prior condition, in absence of considerations on the structural damage given by the vertical settlements, and an updated one, where the outcomes of the monitoring activity are accounted for determining the damage grade of buildings according to differential ground movements. The prior risk analysis investigates Pienza's urban stock in standard conditions, consistent with other urban scale seismic risk applications carried out in many different contexts.

3.5.1 Seismic hazard

The hazard curve for the city of Pienza has been plotted according to the Italian Technical Code [10]. The latter expresses the average annual frequency of exceedance of the local seismic intensity defined in terms of peak ground acceleration (PGA). The hazard curve is obtained starting from the 9 PGA values associated with the different return periods reported in [10] (from 30 years to 2475 years) considering a soil class A. In Table 14 the values of the elastic spectra associated with the different return periods for the city of Pienza are shown, while in Figure 70 the corresponding hazard curve is presented.

Table 14. Values of the elastic spectra associated with the different return period for the site acceleration (a_g), the maximum value of the amplification factor of the spectrum in horizontal acceleration (F_0), starting period of the section with constant velocity of the spectrum in horizontal acceleration (T_c^*).

Tr [years]	a_g [g]	F_0 [-]	T_c^* [s]
30	0.047	2.514	0.247
50	0.058	2.537	0.256
72	0.068	2.508	0.262
101	0.078	2.510	0.265
140	0.089	2.483	0.270
201	0.102	2.476	0.271
475	0.139	2.481	0.278



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975	0.174	2.514	0.283
2475	0.225	2.564	0.293

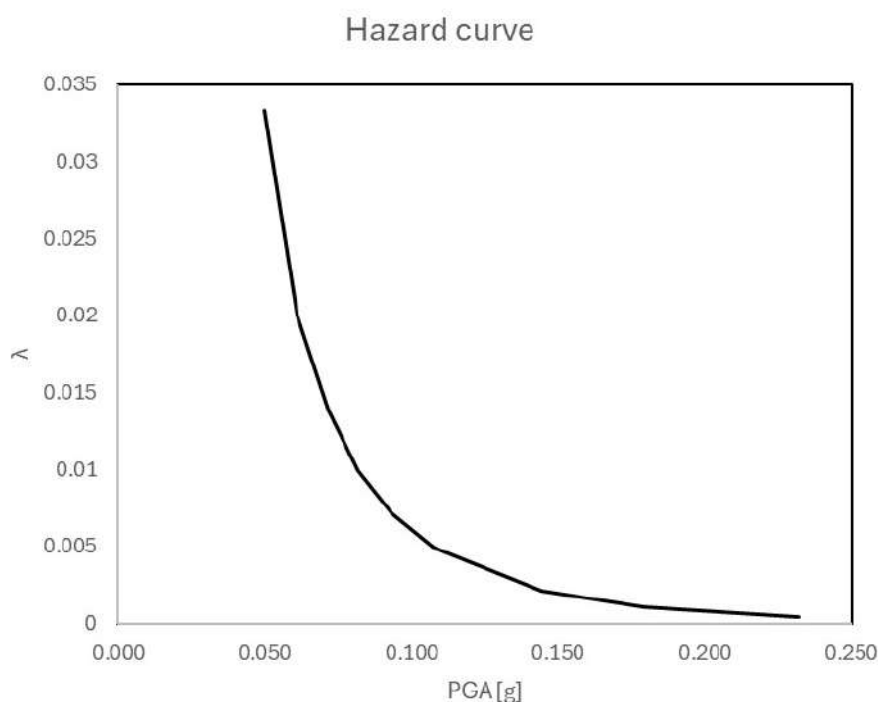


Figure 70. Hazard curve for the city of Pienza.

3.5.2 Seismic vulnerability assessment

To execute a rapid seismic vulnerability assessment of masonry buildings, a macroseismic approach based on the EMS-98 scale has been adopted [27]. The latter correlates building taxonomy to vulnerability classes through the Typological Vulnerability index V_I^* . Then, for each structural unit, the structural modifiers ΔV_m reported in [28] are considered. In this work, all the structures considered have been related to the same structural typology, i.e. a modified M1 masonry type (rubble stone) considering also the vulnerability factor of M6 (Unreinforced Masonry with r.c. floors). This approach is consistent with



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other previous research [29, 30] which consider ancient masonry structures accounting on previous reinforcements at the slab levels. The V_I^* index for the masonry typology has been assumed to be equal to 0.74, considering a weighted value between the V values for M1 and M6. The final Vulnerability index V is given by:

$$V = V_I^* + \Delta V_m \quad (3)$$

In Figure 71 a map with the distribution of the different vulnerability indexes along the historical center is shown. As previously specified, the special monuments such as the Dome, Palazzo Piccolomini or the defensive building on the eastern side of the center have been excluded from this application.



Figure 71. Seismic vulnerability indexes obtained for the different structural units of the historical centre.

The values range between 0.66 and 0.90, with a mean value equal to 0.76. Given the empirical evaluation, the highest values are driven by irregular configurations and head or corner positions in the aggregates. On the other hand, the presence of low-rise masonry buildings with regular dispositions built in the middle

of urban aggregates lead to the lowest vulnerability values. This condition represents the initial one, which does not account for eventual pre-existing damage based on slow-moving landslides and differential settlements on the structures.

3.5.3 Prior probability of failure

A vulnerability curve defining the relationship between macroseismic intensity (I) and mean damage grade (μ_D) is defined. In case the buildings are constituted by masonry structures, the relationships provided by [29] has been used:

$$\mu_D = \begin{cases} 2.5 \left[1 + \tanh \left(\frac{I+3.45V-11.7}{0.9+2.8V} \right) \right] & \text{for } V \geq 0.32 \\ 2.5 \left[1 + \tanh \left(\frac{I+3.52V-12.6}{1.8} \right) \right] & \text{for } V < 0.32 \end{cases} \quad (4)$$

Once the vulnerability curve is obtained, the associated probability of attaining a given damage grade D_k ($k=0, 1, 2, \dots, 5$ with D0 No Damage, D1 Light Damage, D2 Moderate Damage, D3 Heavy Damage, D4–D5 Very heavy damage/Collapse) is expressed through a binomial distribution as a function of the mean damage grade:

$$p_k = \frac{5!}{k!(5-k)!} \left(\frac{\mu_D}{5} \right)^k \left(1 - \frac{\mu_D}{5} \right)^{5-k} \quad (5)$$

Numerical fragility curves can be then derived by adopting:

$$p_{V,k}(I) = P(DM \geq D_K | I) = \sum_{i=k}^5 p_i \quad (6)$$

Figure 72 presents the fragility curves for the historical center of Pienza, considering the average V value within the range between the minimum and maximum observed values. The plotted curves are derived according to the Damage grade 4, corresponding to the Life Safety Limit State (SLV).



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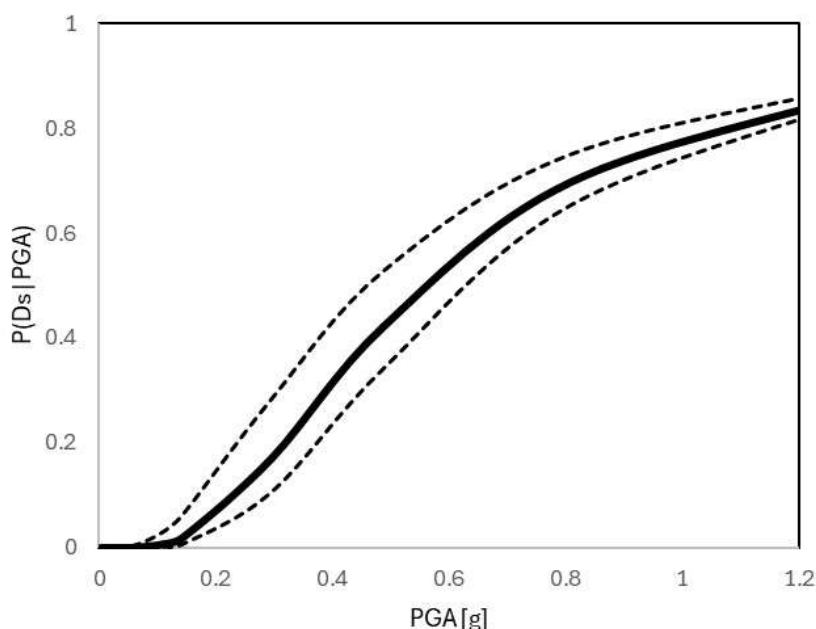


Figure 72. Fragility curves for the city of Pienza. In black, the mean curve, dashed the curves for the maximum and minimum V values.

3.6. Updated seismic risk analysis (DIDA – UNIFI)

3.6.1 Procedure for seismic vulnerability analysis including differential settlements

The purpose of this section is to develop a seismic risk assessment of urban areas including the effects of vertical settlements on constructions. In the analysis soil-structure-interaction issues are neglected, as the idea is to measure directly on the buildings the occurred differential settlement through monitoring. The research aims to carry out a rapid approach based on geometrical parameters, coherent with large-scale applications.

The procedure is constituted by the following steps:

- i) The built environment is divided in terms of structural units, assumed as independent structural systems target of the risk analysis;

- ii) A preliminary empirical seismic vulnerability assessment of masonry buildings is conducted, based on the identification of the structural units and their structural features;
- iii) With respect to the measured settlements, the buildings are evaluated in terms of limiting tensile strain method (LTSM); to consider reliable E/G values for each construction, calibrations through linear FE models are executed;
- iv) Numerical models are tested to combined actions accounting on nonlinear constitutive laws. The outcomes of the analyses are used to compute the increased seismic vulnerability of buildings depending on the damage given by vertical settlements;
- v) Urban MR analyses are carried out and the outcomes of the research are determined in terms of damage scenarios, risk analysis and loss.

3.6.2 Calibration of the Limiting tensile strain method to evaluate the effects of vertical settlements

The limiting strain method relies on the assumption that building constructions can be assessed as equivalent beams laying on the ground [31, 32]. When subjected to a settlement, the beam will deform, leading to bending and shear strain. The formulations are based on the Timoshenko's beam theory. Given a building with length L and height H, the strain given by bending (ε_b) and shear (ε_d) actions can be estimated through:

$$\frac{\Delta}{L} = \left(\frac{L}{12} + \frac{3EI}{2tLHG} \right) \varepsilon_b \quad (7)$$

$$\frac{\Delta}{L} = \left(1 + \frac{HL^2G}{18EI} \right) \varepsilon_d \quad (8)$$

Where Δ/L represents the deflection ratio, E the elastic modulus and G the shear modulus, I the second moment of area of the equivalent beam and t the distance from neutral axis to the edge of the beam



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(assumed equal to $H/2$). These relationships are influenced by the E/G ratio; although this is typically expressed through the formula $G = E/2(1+\nu)$, the relationship for structures with openings may change.

Calibrations of E/G ratio for different building facades have been provided by [33]. Starting from their work, which considered clay brick masonry structures, in this paper a similar calibration has been made oriented at the vernacular building typology previously described (Figure 73).

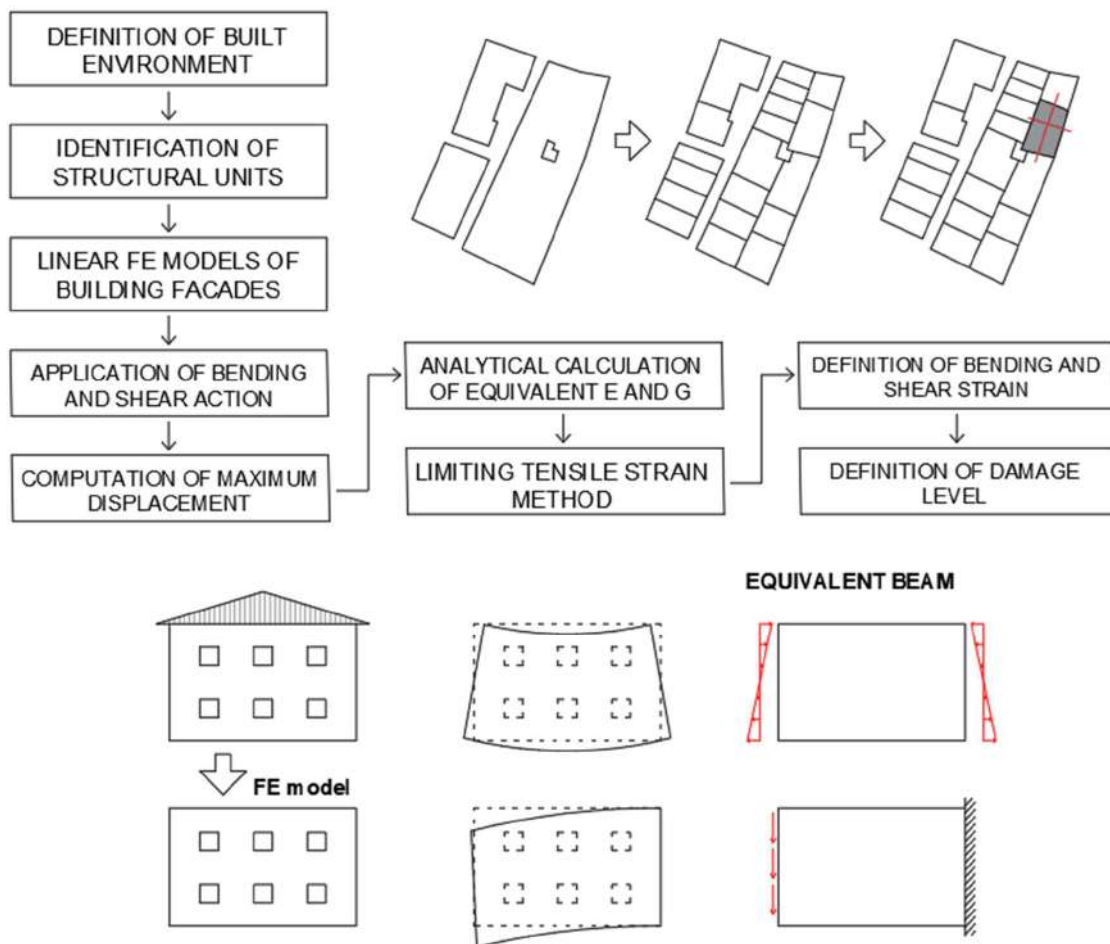


Figure 73. Adopted methodology and E/G calibration based on FE models.

FE models have been realized in Abaqus © environment (Simulia), considering a linear isotropic behaviour of masonry. Although the acknowledged anisotropy of masonry material, this assumption is coherent with irregular rubble stone masonry walls in absence of horizontal mortar beds. In the approach, FE models of each facade are realized, and they are tested to bending and shear actions. The maximum displacement obtained through the numerical models is used as input to analytically calculate the equivalent values of E and G for a solid beam (without openings) characterized by the same length and height of the facade. The adopted curve is reported in red in Figure 75, overlapped to the results from [34]. The black curves were obtained by the Authors considering a discrete modelling of masonry facades, involving the joint normal stiffness K_n and the joint shear stiffness K_s . Considering the results from this research, these values are in line with the one obtained assuming $K_n=K_s$, in agreement with irregular wall textures.

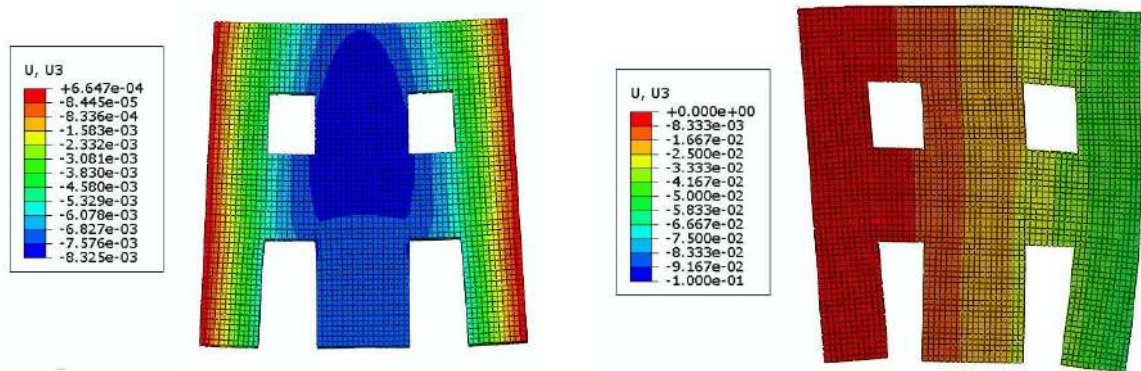


Figure 74. View of the considered linear FE models for the calibration of the curve reported in Figure 75. Comparison of the E/G ratios from (black curves) with the ones obtained in this research from FE modelling (red curve).

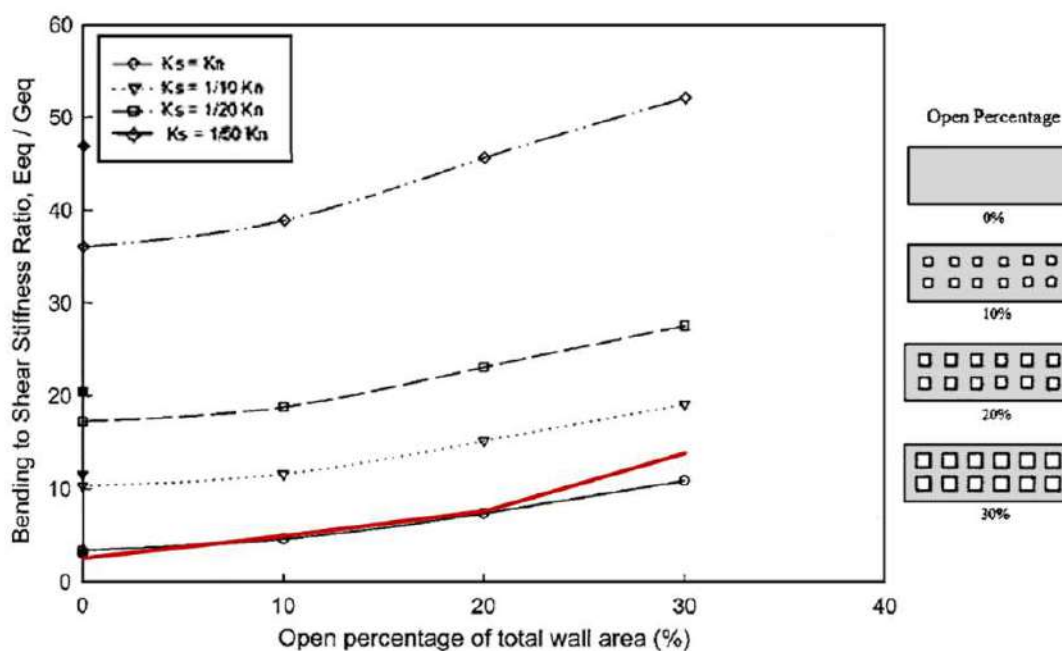


Figure 75. Comparison of the E/G ratios from [33] (black curves) with the ones obtained in this research from FE modelling (red curve).

Once E/G ratio is calculated for every facade, the LTSM can be calculated considering the specific features of each construction. The expected structural damage is defined by means of strain thresholds; values available from the literature are reported in Table 15 [32].

Table 15. Classification of damage according to the limiting tensile strain.

Category of damage	Degree of severity	Limiting tensile strain [%]
0	Negligible	0-0.05
1	Very Slight	0.05-0.075
2	Slight	0.075-0.15
3	Moderate	0.15-0.3



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4 to 5

Severe to Very Severe

>0.3

3.6.3 Updated probability of failure

To account for the possible effects of slow-moving landslides producing vertical settlements leading to structural damage, numerical models were adopted. In this work, the equivalent frame (EF) modelling approach was used through the Tremuri code [35, 36]. EF methods discretize masonry walls through a mesh of piers, spandrels, and rigid nodes. This approach is based on the evidence of the post-earthquake damage in masonry structures. Hence, the piers and the spandrels rule the nonlinear response of the materials.

In this work, 12 different models have been developed, representative of a respective number of facades. These facades have been modelled to have 0, 10, 20, and 30% of openings, while they vary from one to three storeys (Figure 76).

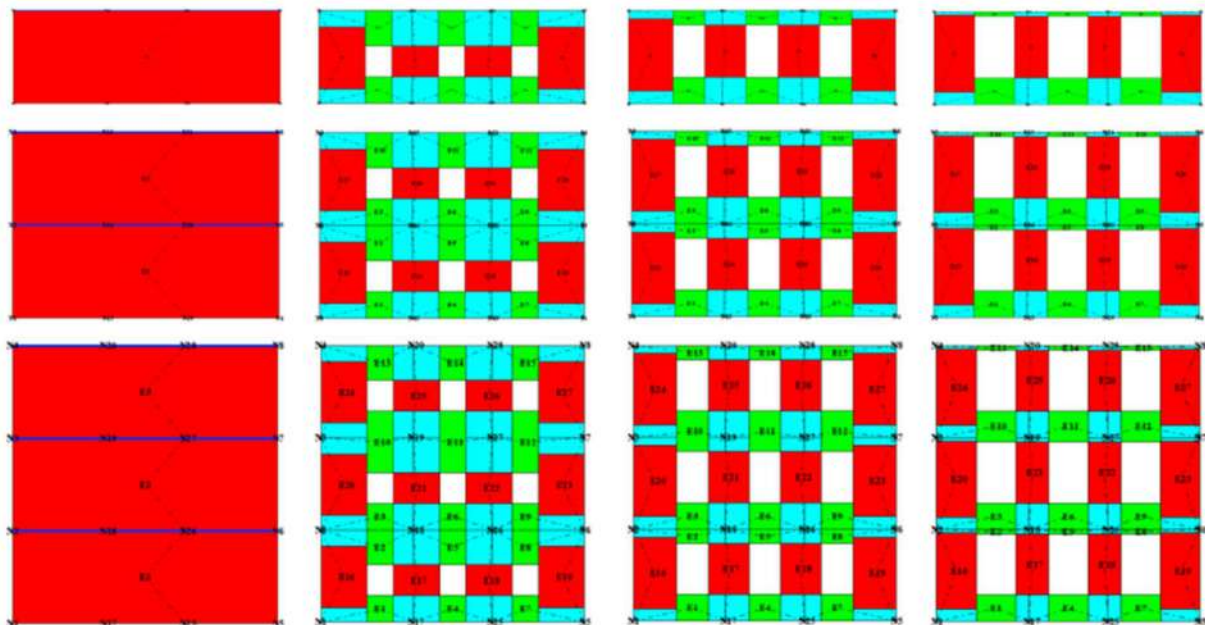


Figure 76. Schemes of the facades analysed in the research.



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Each model has been first subjected to gravity loads, then, a settlement following a Gaussian profile has been applied to one side of the facade. The intensity of the settlements has been computed for each case through Burland's formulations to obtain different damage grades, from 1 to 3. Finally, the damaged models have been subjected to pushover analyses adopting an inverse triangular load pattern. Higher damage intensities caused by the settlements have not been considered, as it was considered unreasonable to attempt to define a seismic risk for buildings suffering such damage in static conditions. All models have considered the same building characteristics in terms of inter-story height, wall thickness and loads coming from the slabs. For the masonry characteristics, they have been assumed on the basis of [10] for a clay brick masonry typology and a level of knowledge LC1.

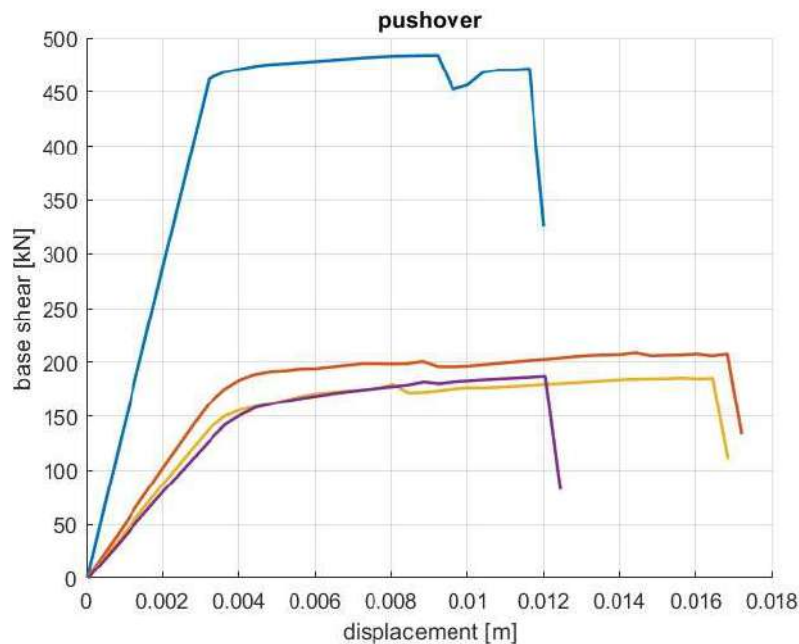


Figure 77. Capacity curves from the four facades in absence of settlements. In blue, the 0% facade, in red 10%-opening facade, in yellow 20%-opening facade, in purple 30%-opening facade.

Once the capacity curves were obtained for the investigated case study, fragility can be defined. The procedure reported in the Hazus method [18] has been adopted. Fragility curves are modelled as

cumulative lognormal distributions; given a value of the intensity measure (IM) the fragility function expresses the probability of reaching or exceeding a damage state (DS):

$$P [DS|IM] = \Phi \left[\frac{1}{\beta_{DS}} \ln \left(\frac{IM}{IM_c} \right) \right] \quad (9)$$

where: Φ is the standard normal cumulative distribution function; IM is defined in terms of PGA, so IM_c is the capacity PGA; β_{DS} is the standard deviation of the lognormal distribution, defined, for each damage state DS . For $DS4$ it can be described according to the following relationships:

$$\beta_{DS4} = 0.1 + 0.4 \ln(\mu) \quad (10)$$

μ is defined as a ductility index and is accounted as the ratio between the ultimate and the yielding displacements computed on the capacity curve.

In Figure 78 the results from the 10%-opening facade in terms of fragility curves are reported. The graphs highlight the influence of the settlements in anticipating the attainment of the limit state threshold at lower PGA levels. These outcomes, obtained for the different representative facades previously presented, allow obtaining a general overview of the effects of settlements in the historical center of Pienza and updating its seismic risk.



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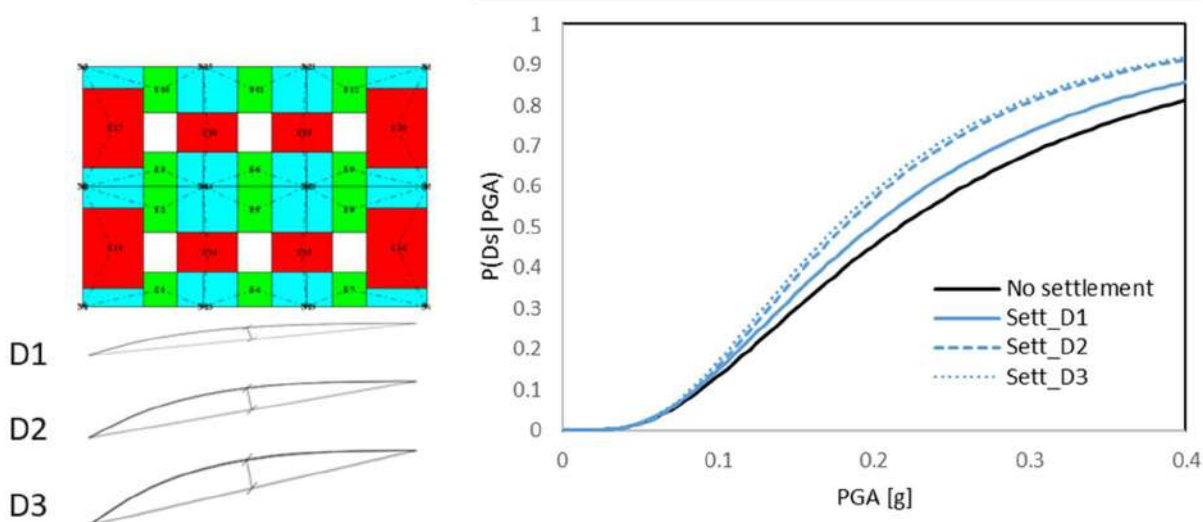


Figure 78. 10%-opening case: fragility curves accounting on different settlement intensities

3.6.4 Urban scale assessment including the effects of differential settlements (DIDA-DST)

The analysis at the urban scale for the historical centre of Pienza has included the previous steps in comprehensive evaluation. The cumulative displacement maps of the ground where the buildings insist and provided by the Department of Earth's Science have been overlayed to the structural units of the built environment. With respect to the monitoring period (January 15, 2015, to April 9, 2023) three different dates have been considered:

1. January 31, 2015 = T1, first survey update (Figure 79);
2. January 23, 2019 = T2, halfway through the period considered (Figure 80);
3. April 9, 2023 = T3, last survey update (Figure 81).

The maps were processed by the Department of Earth's Science from the synthetic PS dataset relating to the purely vertical component of the motion. From this point data, an interpolation was performed using the IDW (Inverse Distance Weighting) method, a spatial interpolation method commonly used for this type of data to estimate values at unsampled points based on data collected at nearby points; it is based



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on the principle that points closer to a given point have a greater influence on the estimated value than those further away. The size of the interpolation cell was set at 3 x 3 m. This resulted in maps (raster) corresponding to the three periods indicated above. The displacement values recorded range from -39 to 20 mm, with negative values representing downward movement and positive values representing upward movement. For easier reading of the data, the displacement values were classified according to a colour scale from red (downward displacement) to blue (upward displacement); areas with cumulative displacements between -2 and 2 mm, considered stable, are indicated in green.



Figure 79. Interpolation of CSK synthetic PS data showing vertical displacement as of January 31, 2015, based on CTR 10k for the Region of Tuscany



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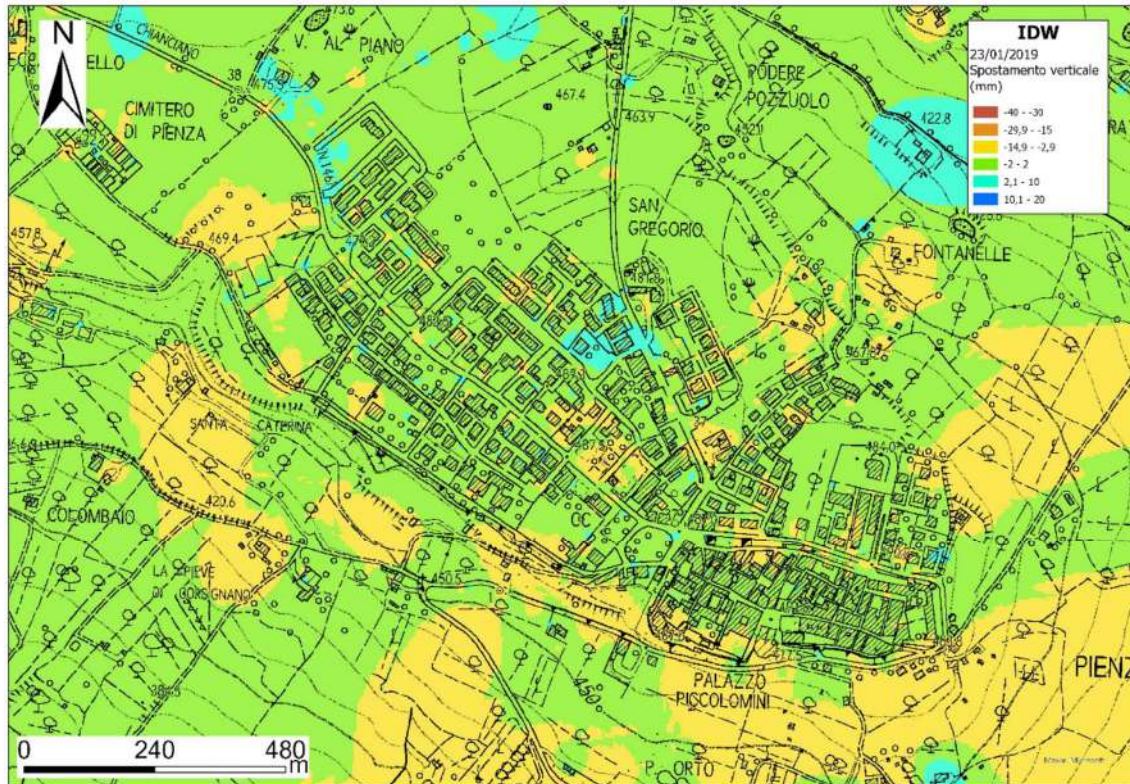


Figure 80. Interpolation of CSK synthetic PS data showing vertical displacement on 23/01/2019, based on CTR 10k for the Tuscany region



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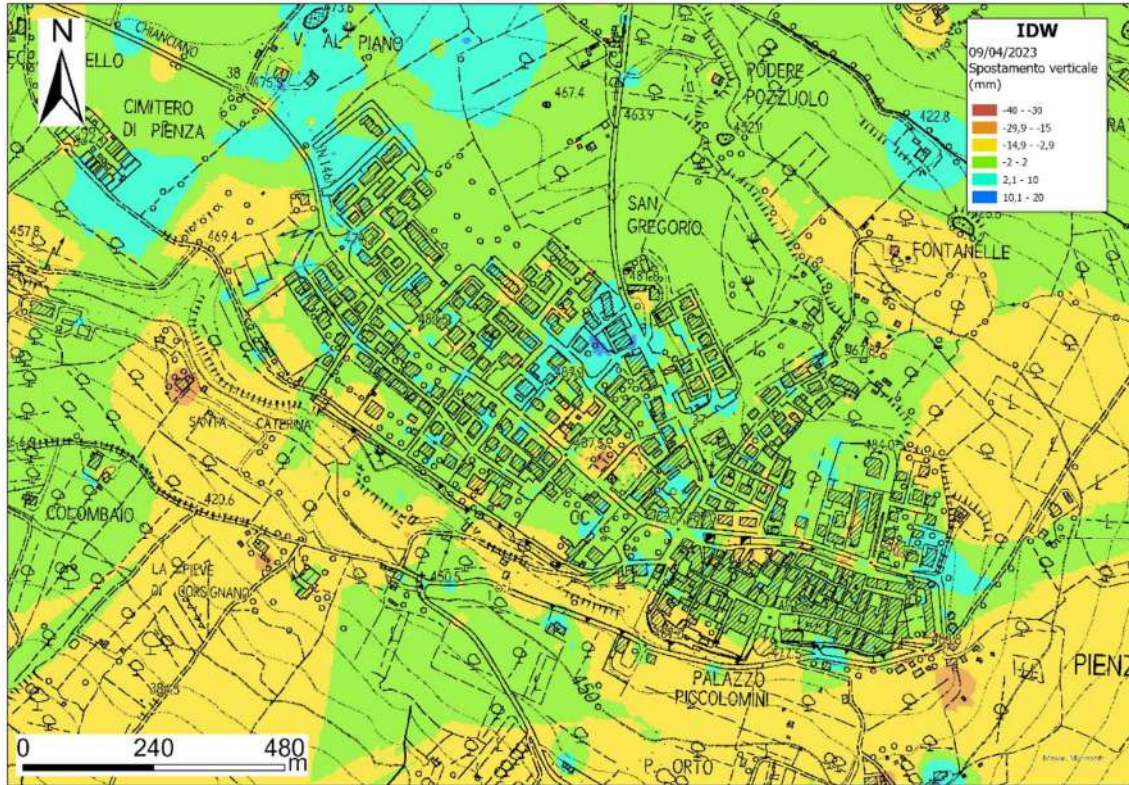


Figure 81. Interpolation of CSK synthetic PS data showing vertical displacement on 09/04/2023, based on CTR 10k for the Region of Tuscany

Once the three maps have been overlayed to the structural units, the differential settlement for each building according to the ground movement was computed in accordance with the formulations reported in section 3.6.2. A graphic visualization of the axes adopted to obtain the differential settlement is shown in Figure 82. Based on the geometry of the different buildings, each unit has been related to the various façade schemes reported in Figure 76. Finally, based on the damage level of each building according to the three maps, the increase of seismic risk given the occurrence of the vertical settlement can be computed. The final risk is given in terms of:

$$\lambda(DS) = \int P(DS | IM) \cdot d\lambda(IM)$$

which combines the probability of exceeding the damage state with the annual exceedance rate of ground-motion intensity. The final results indicate that, starting from a configuration without including the effects of the satellite-measured settlements, the risk computation for the three considered dates leads to a non-significant increase for the 2015 scenario, with a risk increase of +1.43% for the year 2019 and a +5.67% increase for the year 2023. These results point to the effect of including monitoring data into multi-hazard risk analyses for more reliable outcomes, providing a methodological approach that can be replicated in other contexts of interest.



Figure 82. Visualization of the urban environment with the distinction of the structural units and the inner axis used to compute the differential settlements

4 CH monitoring at Territorial scale – Sardinian Region

4.1. The proof of concept

4.1.1 Historical development

The monitoring conducted at the territorial scale, on the Sardinian Region, considers peculiar cultural heritage built in Sardinia between the Neolithic and the Early Iron Age during the so-called pre-Nuragic (V – III millennium b. C.) and Nuragic (XVIII - VI centuries b. C.) periods.

During the pre-Nuragic period, Sardinia was inhabited by communities whose settlements span from the Middle Neolithic to the Copper Age, reaching the dawn of the Bronze Age (between the 5th and the end of the 3rd millennium BCE). This marks the final phase of Late Prehistory, when the island became permanently occupied by sedentary populations engaged in the domestication and breeding of animals, agriculture, and other subsistence activities.

The variety of structural typologies considered aims to offer a comprehensive and scientifically grounded perspective on the archaeological remains from this period, focusing on those with specific functions—civil, funerary, ritual, and/or magical-religious—or marked iconographic features. Special attention is given to two major cultural phenomena also observed across the Mediterranean coasts and in continental Europe: Hypogeism and Megalithism. Both are attested in Sardinia during this timeframe, with hypogeism appearing slightly earlier, in the first half of the 5th millennium BCE.

Although present in other parts of the Mediterranean and occasionally in continental Europe, the earliest evidence of hypogeism has been found in Sardinia. The island is particularly notable for its *domus de Janas*—rock-cut chamber tombs—widely considered to be of local origin and distributed throughout the region. This hypogeal tradition continued into the Early Bronze Age (c. 1800–1600 BCE) with the excavation of new tombs, rather than the reuse of older ones. These later hypogea often feature façades that emulate the stelae of the Tombs of the Giants, a type of sepulchre that would become characteristic of the subsequent Nuragic Age. This evolution reflects both continuity and transformation, as burial practices shifted from subterranean to monumental above-ground structures. Forms of settlement hypogeism, including caves and natural shelters used as dwellings, are also documented.

By contrast, Megalithism in Sardinia is part of a broader phenomenon specific to Neolithic communities in the western Mediterranean, which flourished between 5000 and 2000 BCE. Sardinian megalithism is generally divided into four phases:

- Proto-megalithism: circular burial structures (Middle Neolithic II)
- Monolithic megalithism: standing stones or menhirs (Late Neolithic–Copper Age)
- Trilithic megalithism: portal tombs or dolmens (Late Neolithic–Copper Age)

The age of "megalithic walls": large, fortified enclosures (Copper Age, Monte Claro culture)

Both hypogeism and megalithism gave rise to a diverse array of monuments across the island, constructed using innovative techniques such as rock-cutting and trilithic assembly, alongside the enduring tradition of dry-stone construction, which continues to define Sardinian architecture to this day.

The two phenomena display distinct patterns of distribution: domus de janas (Figure 83), emblematic of hypogeism, are concentrated in central-northern Sardinia and, to a lesser extent, in the central-western part of the island. Megalithic walls, on the other hand, are primarily found in the central-northern region. Other megalithic structures—including menhirs, stone circles, dolmens, and allées couvertes—are more frequent in inland areas, particularly in the province of Nuoro, along the eastern side of the island. In some inland zones, there is an overlap between hypogeic and megalithic funerary structures.



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Figure 83. Domus de janas in Montessu (Villaperuccio, SU).

The density and distribution of these prehistoric sites were influenced by multiple factors. In addition to the inherent randomness associated with archaeological discovery, natural processes such as vegetation overgrowth and sediment accumulation have significantly impacted the visibility and detectability of sites in the landscape.

It is known as one of the most remarkable cultures of European prehistory, characterized by the construction of monumental structures of great significance [37, 38]. Although this culture evolved through several distinct phases, it chronologically spans from the Middle Bronze Age to the Late Bronze Age (circa 1750–1200 BCE), retaining some of its features into the Early Iron Age (up to the 6th century BCE). Despite these changes, the culture maintained a high degree of internal consistency, to the point that it is recognized as a unified and coherent historical entity over time [39, 40, 41, 42].



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The most iconic symbol of this civilization is the nuraghe, a megalithic dry-stone structure, sometimes built on multiple levels. Nuraghi fall into two main categories: isolated or single-tower structures, and more complex multi-tower (polylobate) constructions with interconnected towers. These buildings served a variety of functions and often formed the central core of community organization.

Most nuraghi were integrated within settlement contexts, with villages distributed in close proximity and often developing around the main tower, which played a key role in overseeing and controlling the surrounding territory.

The Bronze Age archaeological landscape of Sardinia is therefore marked by a notable density of construction: during the Middle and Recent Bronze Ages, there was an intense period of nuraghe and tomb building, followed by the reuse of these structures in later periods [43], as well as the emergence of buildings associated with cult practices—a phenomenon that peaked during the Iron Age, when sanctuaries began to contain prestige goods from across the Mediterranean [44].

These monumental structures, although multifunctional, were primarily intended to ensure strategic and visual control over the island's territory [39, 40, 41, 42].

Over the past two decades, the increasing systematization of research activities across Sardinia, combined with a growing number of archaeological investigations into settlements of various types, has led to a substantial increase in scientific knowledge. The introduction of increasingly advanced methodologies and more precise excavation techniques has played a crucial role in redefining interpretations of the functions of Nuragic monuments and, more broadly, the role of Nuragic civilization within the protohistoric Mediterranean context.

The analysis of material culture, as well as the study of external relations maintained by the island with other areas of the Mediterranean basin, has made it possible to more precisely define the chronological framework within which this civilization developed. In particular, two major evolutionary phases have been identified: the first, spanning the Middle to Recent Bronze Age (18th – 1150 BCE), and the second, covering the Final Bronze Age and Early Iron Age (1150 – 500 BCE) [45].

Taking into consideration the nuraghe, from a constructional perspective, two main categories of nuraghi can be distinguished: the protonuraghe, dating from 1700 to 1350 BCE, and the classical nuraghe or tholos-type, which was widespread between 1350 and 1150 BCE.

During the early phase of the Nuragic civilization, a type of structure known as the protonuraghe or archaic nuraghe (sometimes referred to as “corridor nuraghe”) became widespread. These buildings, often with elliptical or subcircular floor plans, are characterized by the absence of true ground-floor chambers and by the presence of corridors covered with large stone slabs, from which upper terraces could be accessed via internal staircases.

Over time, the buildings became more complex: the masonry became more regular, the internal chambers took on elliptical or ogival-truncated shapes, and were arranged across two levels. Some protonuraghi include central courtyards, additional towers (two or three), and in some cases, external defensive walls, giving them a fortified appearance.

From the later phase of the Middle Bronze Age (ca. 1350 BCE), a new architectural type emerged, in some cases transforming the earlier structures: the classical nuraghe, or tholos-type nuraghe. These are circular-plan towers with a truncated conical shape, featuring superimposed chambers constructed with corbelled rows of stones that form ogival (tholos) vaults. An entrance corridor, wall niches, and spiral staircases carved within the masonry allowed access to upper levels or to a rooftop terrace. The towers could exceed 20 meters in height and contain up to three stacked chambers.

Although various planimetric layouts exist, no single, linear evolutionary path has been identified [46, 47, 38, 41]. The classical nuraghe can be simple, consisting of a single tower (mastio), or complex, incorporating lateral towers connected by walls that form bilobate, trilobate, tetralobate, or pentalobate bastions (Figure 84). Some central towers reached heights of 23 meters, while lateral ones could be up to 18 meters tall, arranged over multiple levels.



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Figure 84. Nuraghe Santu Antine (from Sardegna Turismo).

The most developed complex nuraghi were protected by outer bastioned walls with smaller towers, and in some cases by a third enclosure wall that surrounded the associated village. In certain sites, such as Serri (Santa Vittoria), Mogoro (Cucurada), and Villanovafranca (Su Mulinu), earlier protonuraghi were incorporated into the newer complexes [48, 49].

Classical nuraghi also display expansions (bastions, courtyards, additional towers), showing great typological variety [50, 47, 38, 41]. The upper part of the tower could include terraces supported by large corbels, as seen at Villanovaforru, Serri, and Barumini. Externally, large outer fortifications are found, for example at S'Urachi (San Vero Milis) and Su Mulinu (Villanovafranca). Although they were once interpreted primarily as fortresses [51], the current prevailing view sees them as structures for territorial control, without excluding a defensive function [52, 53].

Many villages developed in close connection with nuraghi, but there were also independent settlements, sometimes associated with places of worship. In the early phases of the Middle Bronze Age, huts were generally rectangular or quadrangular, often with apsidal ends and roofs made of plant materials supported by stone walls.



From the later Middle Bronze Age and into the Recent Bronze Age, huts began to adopt circular or elliptical floor plans, with stone bases and conical roofs, similar to the modern *pinnettas* found in Sardinian pastoral tradition.

During the Final Bronze Age and the Early Iron Age, a greater degree of urban complexity is observed, with huts arranged in *insulae* (blocks) around shared courtyards. This period saw the appearance of “round huts with basins”—circular buildings with benches and a central basin, likely for ceremonial use—and “meeting huts”, large structures with seating, intended for communal activities.

Between the Middle and Recent Bronze Ages, there was a transition from sub-rectangular huts with stone foundations and wooden upper structures to buildings made entirely of local stone, often featuring niches and wooden roofs. Buildings made from perishable materials have also been documented, such as those at Sa Osa (Cabras) and Monte Zara (Monastir). In the Final Bronze Age, many *nuraghi* were abandoned (e.g., Santa Vittoria di Serri), and hut architecture underwent transformation: isolated groups with a central courtyard began to appear—a layout that continued into the Iron Age, though with smaller rooms not always arranged around a central court [54, 55] (Figure 85).

As concerns sanctuaries, these appear primarily in the final phases of the Bronze Age and the Early Iron Age, with a central role attributed to water, as evidenced by the well temples, which feature vestibules with benches, access stairways, and underground chambers covered by tholos-style roofs. Some of these structures included advanced hydraulic systems for collecting groundwater or spring water.

The blocks of these temples, finely worked and fitted together with precision, were sometimes secured with lead poured from local deposits. Megaron temples, less widespread, are found in strategic mountainous areas, often along transhumance routes or near springs. Some complexes also included civil structures, domestic spaces, and large communal buildings, such as amphitheaters capable of hosting hundreds of people.



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Figure 85. The Sanctuary Village of Santa Vittoria di Serri - (From acropolinuragica.it – photo from the archives of the Municipality of Serri).



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These sanctuaries served not only as religious centers but also as economic and political hubs, as indicated by the votive offerings, ritual gifts, and furnishings found there: small tables, models of nuraghi, basins, and tanks [56, 57] [58].

The most representative Nuragic burials are the Giants' Tombs (Figure 86), large collective funerary monuments that could reach up to 30 meters in length and 4 meters in height, often grouped together and intended for multiple burials. The façade is characterized by a concave exedra bordered by vertically inserted slabs or arranged in rows, while the interior consists of a long, straight funerary chamber. These structures evolved from the older Eneolithic corridor tombs (*allée couverte*) and from Early Bronze Age sepulchers [59]. The oldest tombs, contemporary with the protonuraghi, feature a dolmenic chamber covered by horizontal slabs and a lintelled stele with a lower opening for access. Later burials employ blocks arranged in projecting rows that form a truncated ogival or ogival vault, without the presence of a stele.



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Figure 86. Giants tomb of Is Concias, Quartucciu (CA).

Some monuments are built using isodomic masonry and show a toothed block on the façade, possibly intended to hold miniature betyls. The exedra was the site of funerary ceremonies and features stone seats for participants. The betyls placed near the tombs, often conical, are interpreted as divine symbols linked to ancestor worship [60, 61, 62, 63].

Relationship Between Nuragic Monuments and the Landscape

Nuragic monuments represent an essential component of the contemporary cultural landscape of Sardinia, attesting to the widespread presence of Nuragic communities throughout the territory, regardless of the geomorphological characteristics of the different areas.

The distribution of sites reveals a strong interaction between settlement and natural environment: large Nuragic complexes, often located within extensive productive contexts, are accompanied by villages and other structures, forming true settlement networks on a territorial scale. These complexes are generally



situated in elevated and strategic points, capable of offering wide views for territorial control, while at the same time designed to be visible from afar, thus creating a reciprocal visual relationship with the surrounding landscape [64, 65].

Funerary and cultic monuments, such as giants' tombs and temples, arise in locations of strong landscape significance—often in elevated positions—which confer upon them a sacredness linked also to their placement and connection with prominent natural and geomorphological elements [56].

A significant correspondence can be recognized between still well-defined landscape units—such as plateaus, valleys, and river courses—and the distribution of Nuragic settlements, often organized within large, multifunctional areas. Nuragic communities favoured areas rich in natural resources, such as fertile soils, water, forests, and minerals, which were used and transformed according to settlement and economic needs: agricultural areas, grazing lands, and spaces for hunting and gathering were developed [39, 65].

4.1.2 Pre-Nuragic and Nuragic structures: typologies and materials

All the monuments dated back from this period, falling under the category 5. *Monuments, remains, and archaeological areas*, have been specifically investigated in accordance with the period of construction and their dimensions, intended as height/width ratio (Equidimensional – B; Wide – E; Subterranean – D; Small – S; Tall – A).

Each typology of monument has a sheet of analysis, where are specified the key characteristics. This has been significant for the project in order to define different vulnerabilities and then to choose a category as proof of concept for validating the methodology.

In summary, we have 12 typologies of assets:

5.1 "Corridor" nuraghe

5.2 Mono-tower nuraghe

5.3 Complex nuraghe

5.4 Giant's tomb

5.5 Sacred wells

5.6 Domus de janas

5.7 Cave

5.8 Sherter

5.9 Menhir

5.10 Dolmen

5.11 Megalithic circle

5.12 Sanctuary

5.13 Prehistoric Villages

In the following records, each asset is presented following the chronological order of construction.



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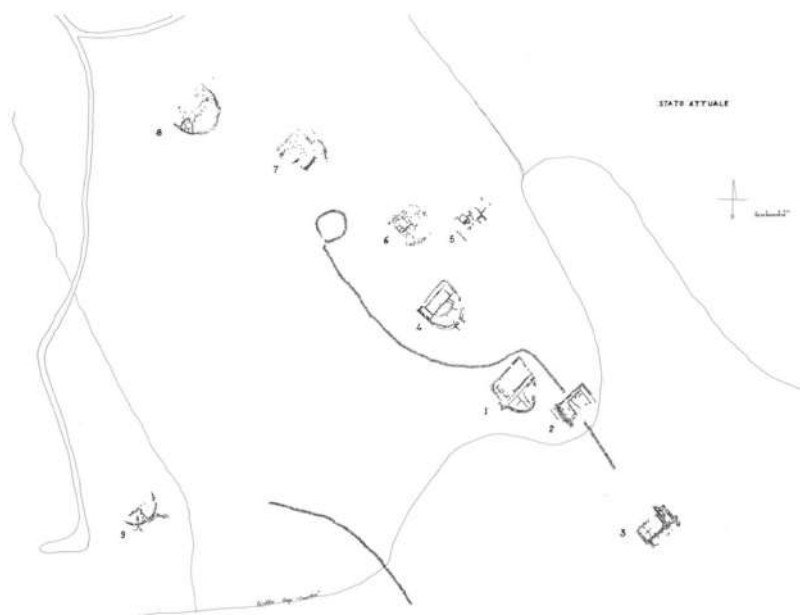


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5.13 Prehistoric Villages

Neolithic | V millennium b.C.



Village of Serra Linta (Sedilo)

Dimension

Wide – E

Description

An open-air village, located in dry and often flat or sloping areas. The settlement framework is heterogeneous: evidence of these villages is concentrated in central southern Sardinia. The lack of data in northern Sardinia stands out even more if one considers that occupation must have been capillary, given the considerable concentration of domus de janas burials and necropolises, even large ones.



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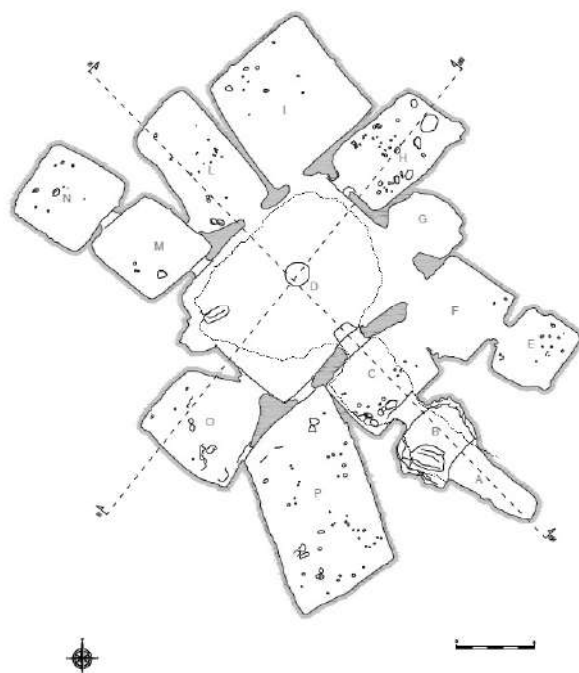
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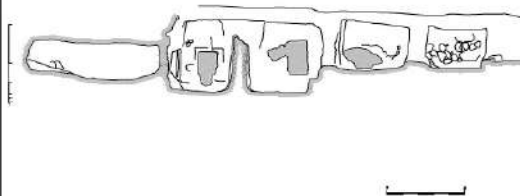
5.6 (Necropolis of) Domus de janas

1800-1600 B.C / Middle Neolithic

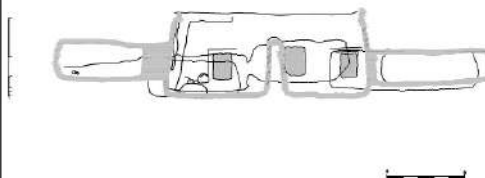
Pianta



Sezione A-A'



Sezione B-B'



Planimetry of Tomb VIII of Su Crucifissu Mannu



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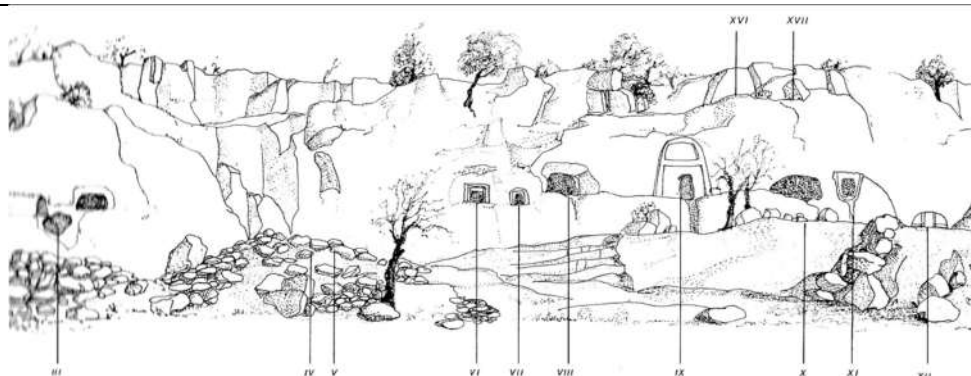


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Planimetry of the necropolis of Sos Furrighesos, Anela

Dimension

Subterranean - D

Description

Collective chamber tomb excavated in the rock.

The hypogea, which were accessed via well, dromos or stepped entrances, sometimes develop in a complex manner up to 20 rooms. The complexity does not respond to a pre-established design but is the result of renovations and additions over the long period of time in which the hypogea were used.

It is believed that other burial or passage chambers were added to an original nucleus out of necessity or for ideological reasons, without a pre-established pattern. It is not always possible to establish, for each hypogeum, at



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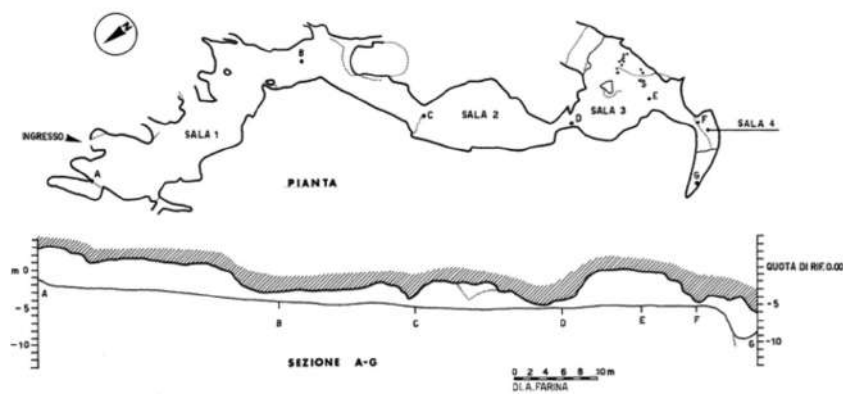
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what time and in what order these interventions were carried out.

5.7 Cave

from the 5th to the end of the 3rd millennium B.C.



Corbeddu Cave, Oliena

Dimension

Subterranean - D

Description

Caves are extensive underground cavities used for civil purposes that sometimes also feature painted or engraved decorations



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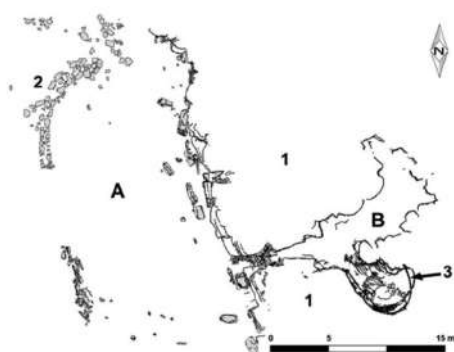


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5.8 Shelter

5th to the end of the 3rd millennium B.C.



Riparo di Su Forru de Is Sinzurreddus - planimetria generale del sito. B: settore E con l'area di faglia nel duomo riolitico (1) con posizionamento della grotta A (3)

Dimension

Small – C

Description

Shelters are small natural cavities used by prehistoric man, generally used for habitation purposes.



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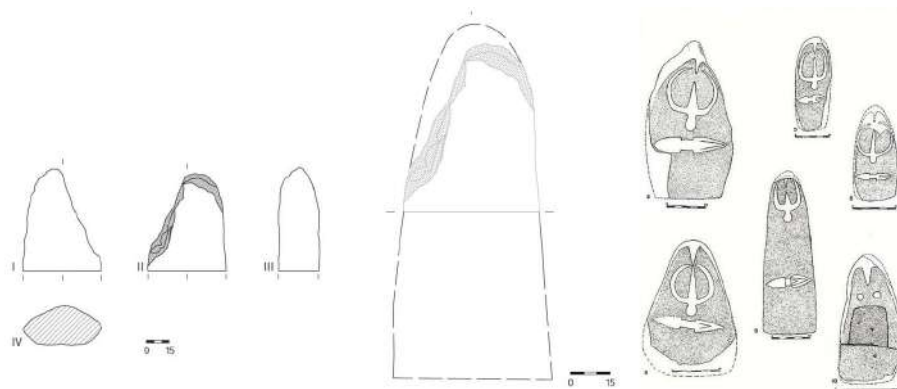


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5.9 Menhir e statue menhir

From the Neolithic to the Copper Age



Serri. Menhir »Sa Porta«: graphic reconstruction; Laconi, Statue Menhir

Dimension

Tall - A

Description

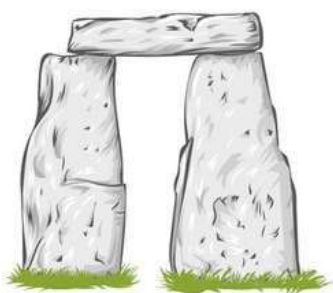
A menhir is a large stone monolith, set vertically into the ground, dating back to prehistory. Usually found standing alone or arranged in groups, the menhir likely served ritual, commemorative, or symbolic purposes.

In the Copper Age, central Sardinia is known for so-called statue-menhirs, a specific type of menhir that, in addition to symbolic and ritual functions, displays a more complex figurative evolution. In this variant, the stone block is no longer simply an upright stone but is sculpted to assume anthropomorphic features, often with carved details such as:

- Stylized human faces (eyes, nose, sometimes mouth)
- Weapons (daggers, axes)
- Sexual symbols (indicating gender, male or female)

5.10 Dolmen

From the Neolithic to the Copper Age



Dimension

Tall - A



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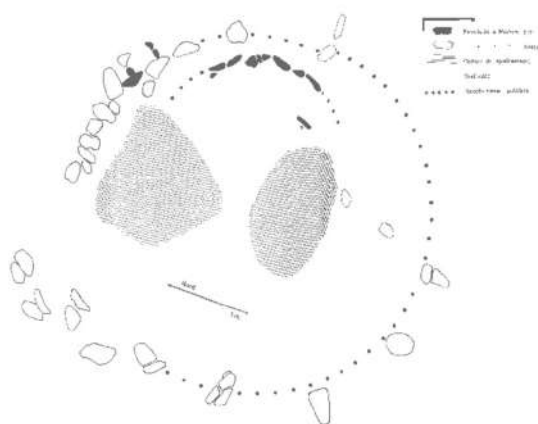
Description

They are trilithic structures, composed of vertical stones driven into the ground and a large covering slab to form a more or less regular rectangular funerary room.

more or less regular rectangular plan. Usually, dolmens occur in isolation. In a few cases they are grouped together in necropolises. In 13 specimens, engraved figures and cup-marks were observed, in two cases associated with gullies that would have a ritual function.

5.11 Megalithic Circles

From the Neolithic to the Copper Age



Circle of Monte Dominigu - Uri

Dimension

Tall - A

Description

They are made up of one or two rows of stones, sometimes dense, arranged in a circle with a central lithic cyst, sometimes



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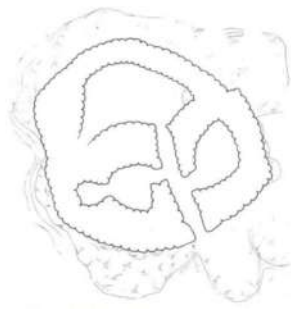
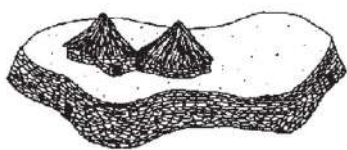
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associated with menhirs. They were presumably covered by a mound.

5.1 Protonuraghe or "Corridor" nuraghe

Bronzo medio 1600-1350 a.C.



Dimension

Equidimensional - B

Description

Morphology: They have sub-circular or elliptical floor plans; sometimes they have no real rooms on the ground floor but are crossed by a corridor with a truncated cone section



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truncated-ogival section with two entrances, sometimes straight, sometimes curved, 'Y'-shaped or 'L'-shaped, passing through covered with flat slabs, from which a staircase leads to the terraced top. In some cases one or more niches open along the sides of the corridor.

The most complex ones have two or three added towers and a central courtyard, with larger elliptical chambers. Building technique: Archaic nuraghi have an outer wall face of large polygonal boulders (cyclopean technique) superimposed in a wasp's nest or of rough ashlar in the shape of an irregular parallelepiped and arranged in rows.

The protonuraghe is distinguished by its rudimentary construction, featuring cyclopean and polygonal masonry, heights of up to 10 meters, variable layouts (circular, elliptical, trapezoidal), and multiple entrances. Inside, corridors, niches, and staircases leading to the terrace are commonly found. Studies [66, 67] have classified protonuraghi according to an evolutionary progression of floor plans, from massive forms to more harmonious structures with ship-like interior spaces.



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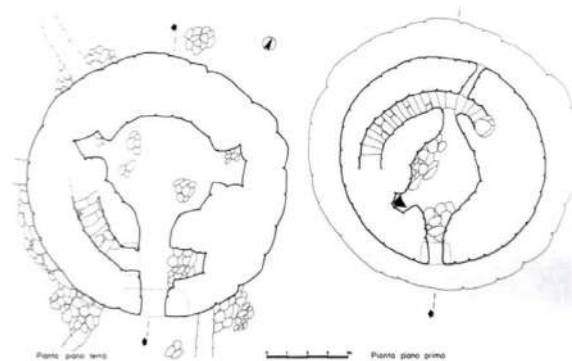
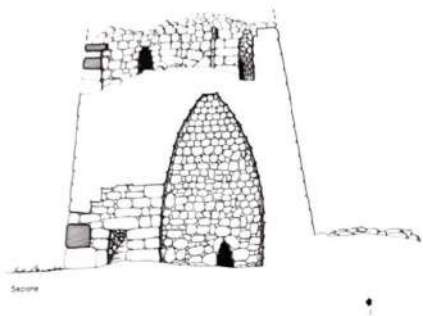


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5.2 Mono-tower nuraghe

1350-1150 b.C.



Nuraghe Ponte (Dualchi), from [68]

Dimension

Equidimensional - B



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Description

Morphology: a structure consisting of a truncated cone-shaped outer masonry, an inner chamber with a circular cross-section covered by a projecting dome (Tholos), and a granular infill between the inner and outer masonry. The inner chambers, also arranged on two and three storeys, are built with the progressive upward projection of rows of stones to form the internal ogival vault.

The systematic use of high ogival vaults in the chambers and in the stone staircases that ascend in a helicoidal path linking the different orders of chambers up to the terraces of the curtains and towers supported by corbels.

Construction technique: use of large boulders offered by the nature of the soil; the gaps between the stones of the wall faces are filled with wedges of smaller stones bound by a very tenacious clay mortar.



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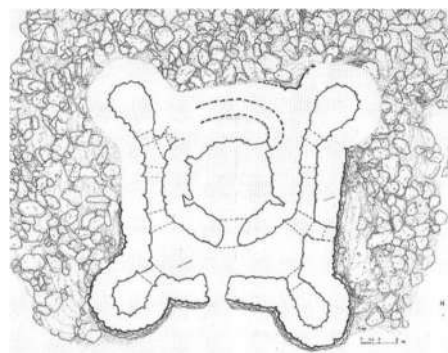
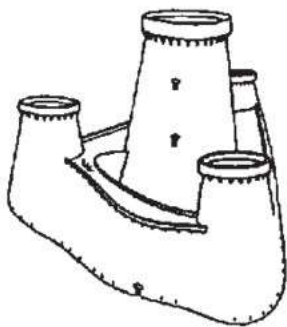
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5.3 Complex nuraghe

a) with an ogival vault - with a central tower with the addition of secondary towers (complex type)

Polylobate



Nuraghe Appiu (Villanova Monteleone), [69]



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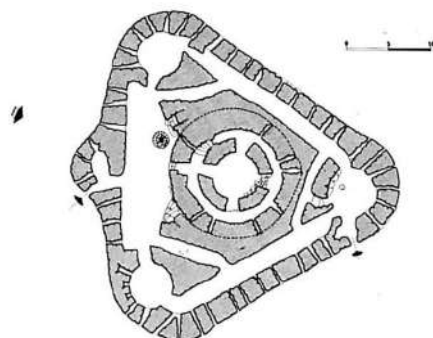
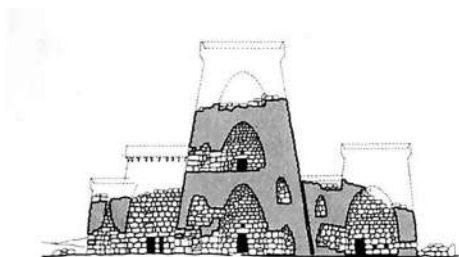


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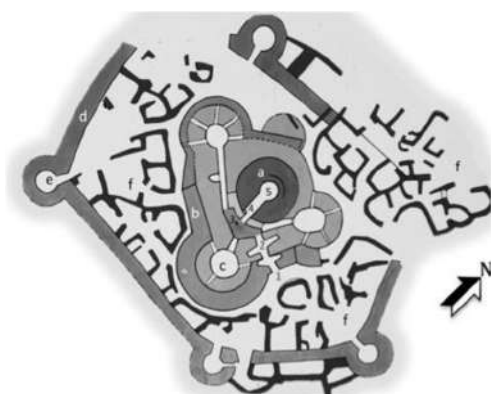
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Nuraghe Santu Antine (Torralba), [70]

b) with antemural enclosure





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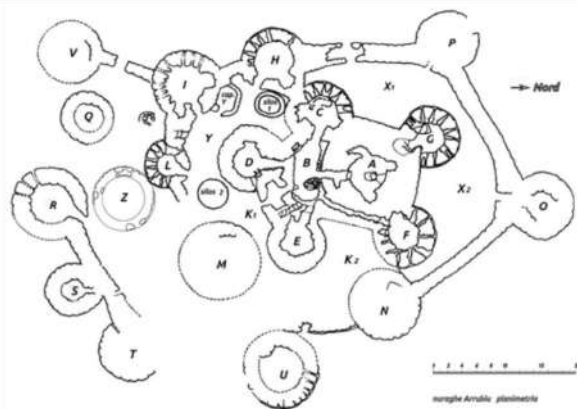
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Nuraghe Genna Maria (Villanovaforru), from E.
Casula <http://www.arsetfuror.com/r12Antico56>.



Nuraghe Arrubiu (Orroli), Survey by A. Pitzalis, R. Pitzalis,

Dimension

Equidimensional – B

Description

The complex nuraghe is configured in two variants:

- a | polylobate: one consists of a central tower with the addition of several secondary towers, connected by a curtain wall that flanks or encircles the central tower (keep) that stands taller than the lateral ones. This is formed by three chambers, one on top of the other. The side towers with their curtains form the polylobate bastions (bi-tri-tetra-penta).

All tower chambers are characterised by their circular layout and characteristic ogival vaulting.

Height of central towers (23-25 m approx.)

Height of side towers (approx. 18 m)

These usually have several overlapping chambers, up to a maximum of three, covered with ogival vaults, connected by a helical staircase cut into the masonry.



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b | with wall doors

(in addition to the polylobate structure)

5.4 Giant's tomb

Bronze and Iron Ages



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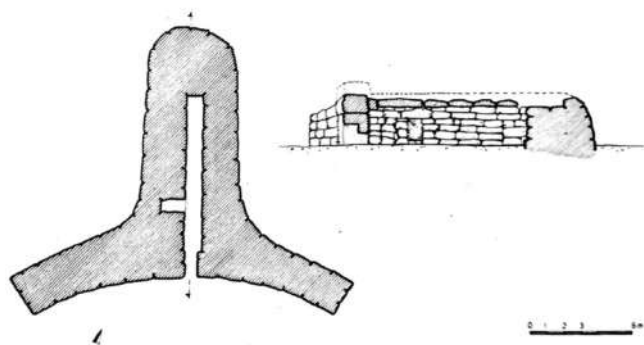


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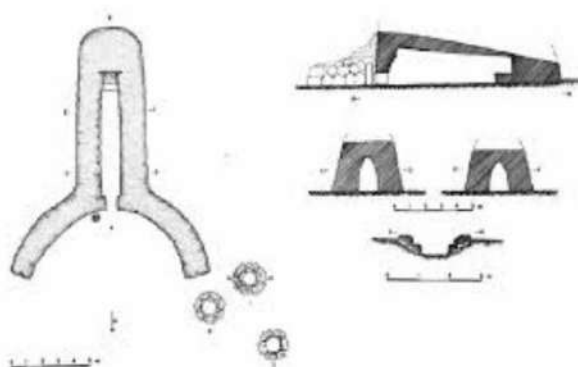


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Giant's Tomb of Domu 'e s'Orku (Siddi), from [71]



Giant's Tomb of Is Concias (Quartucciu), from [72]

Dimension

Wide – E

Description

Monumental collective tombs.

Morphology:

The tombs of giants have a concave front (exedra) composed of



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vertically embedded stone slabs or boulders arranged in rows, set to delimit a semicircular open space, and a corridor-like inner burial chamber.

The burial chamber is built with vertical slabs covered by horizontal slabs. In the centre of the exedra was a stele with a curved top, known as a 'centred stele', at the base of which opens the small access to the burial chamber; the entire monument was originally covered by a mound of earth and small stones.

Dimensions: 30 m long and over 4 m high.

Construction technique:

Isodomic work and at the top of the façade some have the so-called 'notched ashlar', a block with three cavities was probably intended to house three miniature betili.



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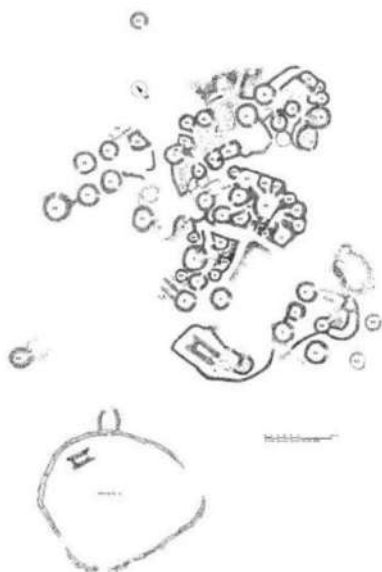


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5.13 Nuragic Village

1350 - 950 bC



Villaggio nuragico di Serra Orrios (Dorgali), [68]

Dimension

Wide – E

Description

Village around a nuraghe or single

Middle Bronze Age: rectangular or quadrangular huts with short apsidal and elliptical sides. The roof consisted of plant elements resting on stone masonry.

Late Middle Bronze Age and Recent Bronze Age: huts with a circular and elliptical ground plan, and still showing a stone



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base, while the roofing is assumed to be a conical hut with rays of poles and branches converging at the top.

Late Final Bronze and Early Iron Age phase: greater urban complexity, with the huts grouped in blocks and arranged around a central courtyard. Occasionally, villages of this phase contain so-called 'round huts with basins', small circular structures characterised by a bench-seat running along the inner side and a central basin.

The simplicity and small size of their single room emphasise their single-family purpose.

Also dating from this period are the meeting huts that appear as community buildings in Nuragic villages between 1600 and 900 B.C. They were circular in shape, built of stone, often larger than normal dwellings, with seats arranged along the inside walls. In the centre could be a hearth or symbolic altars.

They were used for council meetings, religious assemblies or public ceremonies. The presence of model nuraghi and votive objects also suggests a sacred function. These structures indicate the existence of an organised society, with forms of collective participation and structured social life.



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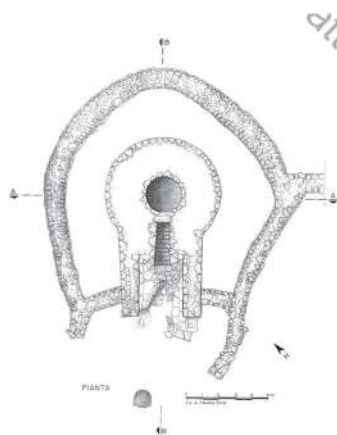
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5.5. Wells and holy temples

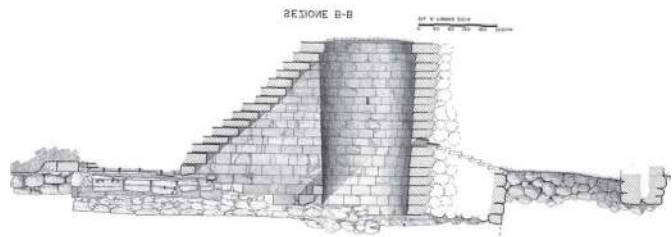
Bronze Age and Early Iron Age

a) Holy well and springs



Dimensione

Descrizione



*Sacred well of the Nuragic sanctuary of Santa Vittoria di Serri –
from [73]*

Subterranean - D

Cult buildings, in which water plays a central role.

Morphology: water temples (wells or fountains) consisted of a vestibule with side benches, a staircase that ran deep until it reached the spring



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vein, included within an underground chamber covered with a tholos.

Construction technique: perfectly worked blocks with a wedge and 'T' shape set in place and held in perfect adherence with rods and cast lead from the large Sardinian deposits. These structures can be found individually or within sanctuaries.



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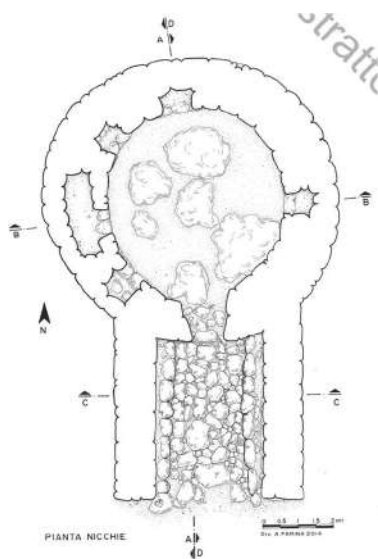


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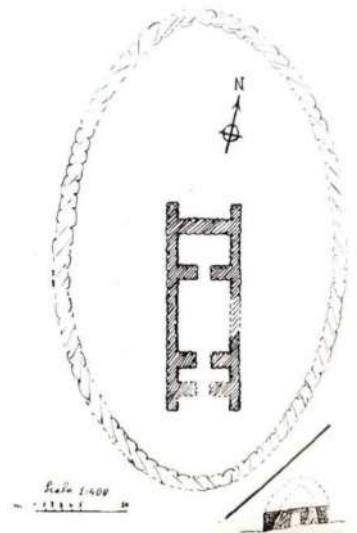


b) in antis temple

Bronze Age and Early Iron Age



Tempio in Antis, Area archeologica di Santa Vittoria. Da [73]



Domu de Orgia (Esterzili), [74]

Dimensione

Wide – E

Description

Nuragic in antis temples represent one of the most refined expressions of the religious architecture of the Nuragic civilization, which developed in Sardinia between the 2nd and 1st millennia BCE. The term "in antis" comes from the vocabulary of ancient Greek architecture and refers to buildings whose entrance is flanked by two projecting pillars or walls, known as "antae". This style, reinterpreted in a Nuragic context,



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was applied to certain temple structures with religious and ceremonial functions.

Temples in antis in Nuragic Sardinia generally have a rectangular or circular plan, with an entrance preceded by a small vestibule and bordered by two masonry doors. The inner cella constituted the sacred heart of the building, probably covered by a wooden or tholos roof in the case of a circular-plan chamber or a double-sloping roof if rectangular. Functionally, these temples were places of worship, often connected to rituals. It is not uncommon to find them near sacred wells or natural springs, a sign of the importance of the water element in Nuragic religious practices.

From a constructional point of view, these buildings are made of local stone, with thick, solid walls, according to the masonry techniques typical of the Nuragic period. They were sometimes part of larger complexes, delimited by cult enclosures or sacred courtyards.



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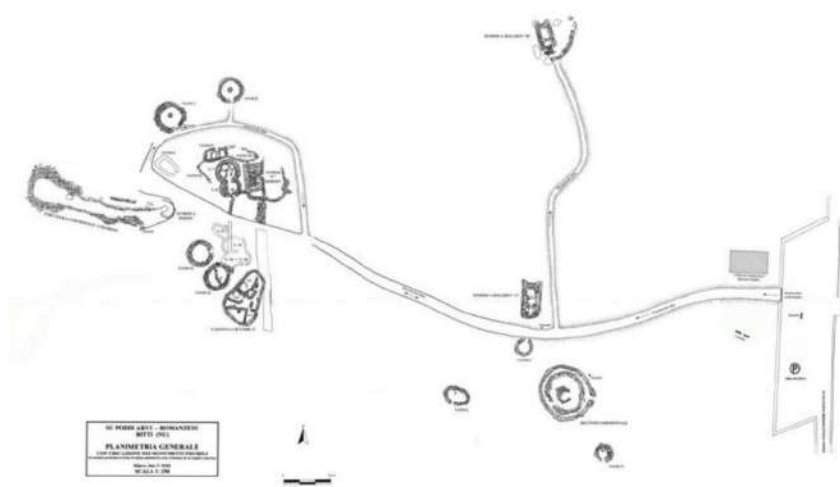
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13. Sanctuary



Nuragic Sanctuary of Romanzesu (Bitti), [75]





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Santuario nuragico di Santa Vittoria di Serri – da [73]

Dimension

Wide - E

Description

The sanctuaries consist of temples of different morphologies and according to different spatial configurations. They are set in large areas that also included structures of a civil nature such as domestic rooms and community buildings; there are large terraced rooms ('amphitheatres'), capable of holding hundreds of people.

These sacred places were not simply buildings dedicated to worship, but real ceremonial complexes, made up of different elements. At the heart of many sanctuaries were sacred wells or water sources, around which temples, ceremonial enclosures, assembly spaces and, in some cases, temporary accommodation for pilgrims were built, such as the festival enclosure of the Nuragic Sanctuary of Santa Vittoria di Serri.

But sanctuaries were not only places of individual worship. They were also centres of social aggregation, where different communities met to celebrate rites, exchange gifts, and form alliances.

Nuragic sanctuaries were spaces where the sacred and the everyday met. Places where the forces of nature were worshipped, protection was sought for the harvest or for the community, and rites linked to the cycles of life and time were celebrated. They were also centres of exchange and encounter, on an island that, although

geographically isolated, was fully integrated into the cultural and commercial routes of the Mediterranean.

4.1.3 Vulnerability definition

Vulnerability represents the damage suffered by a structure as a result of the occurrence of a natural phenomenon of given intensity I [76]. In general, it is expressed on a scale from 0 (no loss) to 1 (total loss) and is a function of the intensity of the phenomenon I and the type of exposed element.

Vulnerability can be quantitatively assessed as the probability that a structure will suffer a certain level of damage following the occurrence of a catastrophic event of given intensity I .

The archaeological assets under investigation are composed of blocks of stone material, stacked and placed side by side according to typical geometries, which interact mechanically through mutual contact. In particular, the research focuses on archaeological heritage dating from the Neolithic to the Iron Age, referred to as prehistoric and Nuragic. This cultural heritage, unevenly distributed across Sardinia, was chosen for its uniqueness and for its symbolic role in strengthening the resilience of the communities that inhabit the region.

The archaeological system has been systematically mapped and typologically defined through the study of morphology, construction techniques, and materials. These thematic studies formed the starting point for assessing the vulnerability to various hazards (floods, landslides, earthquakes, wildfires) that may compromise their state of preservation and transmission to future generations.

In particular, the study of the material component is crucial. Indeed, their deterioration, compromise, or loss—closely linked to the nature of the lithotypes—defines the potential for recovery and long-term preservation, even after events that lead to the structural loss of integrity.

The assessment of vulnerability is therefore articulated along two axes: on one hand, the evaluation of the structural system's capacity to resist hazards as a whole; on the other, the resistance capacity of the individual building elements.



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In other words, since the structures are composed of multiple individual stone blocks arranged in more or less complex geometries that ensure their overall stability, the vulnerability of the asset can be attributed to two distinct mechanisms: the first considers the stability of the structure, depending on the overall geometry (size and shape of the blocks, structural geometry); the second refers to the resistance capacity of the material constituting the individual blocks when exposed to hazards. The latter mechanism appears more critical, as the loss of block integrity not only leads to collapse but also makes reconstruction impossible after events that dismantle the structure. While the first mechanism, although it may cause structural collapse, allows for recovery through reconstruction (by reassembling the original blocks), the second implies irreversible destruction, preventing recovery. In this perspective, the petrography of the blocks introduces a specific vulnerability distinct from that which describes the structure's capacity to withstand hazards through mechanisms leading to disassembly into separate, disconnected blocks.

In this context, geometric vulnerability is associated with the position of the structure relative to the topographic surface and its geometry, identified through the height-to-width ratio: equidimensional (h/w ratio approximately equal to 1); wide (h/w ratio < 1); tall (h/w ratio > 1); small; subterranean (hypogeal structures).

Based on the above geometric criterion, the cultural heritage assets under study are classified as follows:

- Equidimensional: Corridor nuraghi, single tower nuraghi, complex nuraghe
- Wide: giants' tombs; sanctuaries; villages
- Subterranean: Sacred wells; Domus de Janas; Caves
- Small: Rock shelters
- Tall: Menhir structures: Dolmens: Megalithic circles

Instead, the resistance capacity of the blocks is described through two distinct parameters: mechanical resistance, meaning the ability of the blocks to remain intact under external actions (hazards) without cracking or breaking into smaller pieces; and chemical resistance, defined as the ability of the constituent material not to degrade under exposure to chemical weathering, leading to disintegration at the mineralogical level. The vulnerability scale of the construction materials used — granitoids, basalts,



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trachytes “auctorum”, metamorphic rocks, and carbonate rocks – will thus be described with reference to both mechanisms. The approach to assessing material-related vulnerability depends on the nature of the hazard. In the case of fires, the analysis is conducted directly on the extent of damage suffered by individual elements in contact with fire, which, because of their physical degradation, can compromise the stability of the entire structure. In contrast, earthquakes, landslides, and floods exert horizontal and/or vertical forces on the structures, and the severity of the damage is primarily influenced by the overall geometry of the structure. However, the nature of the construction material remains a key factor in determining the resilience of the blocks following such events.

Against this background, the vulnerability of prehistoric and Nuragic structures—understood as the propensity of a structure to suffer damage due to a hazardous event—has been assigned a value on a scale from 1 to 5. Vulnerability is assessed with reference to the integrity of the structure, the blocks, and the material composing the blocks. A damage level from 1 to 5 is considered reached if at least one of the conditions related to the structure, the blocks, or the material integrity occurs. Damage level 5 is reached when either mechanical disintegration of the blocks or chemical-physical alteration of the constituent minerals occurs.

Definition of damage:

- 1: The structure is unaffected by the event and remains in good preservation condition. The blocks do not undergo relative movement, and there are no cracks or mineralogical-petrographic alterations in the constituent material.
- 2: The blocks undergo relative displacements that do not compromise the stability of the overall structure. No fracturing occurs, and no chemical or physical alterations are observed in the materials.
- 3: The blocks undergo significant relative displacements that compromise global stability but not the structural geometry (the structure does not collapse). New, non-through cracks appear in the blocks, and superficial mineralogical-petrographic alterations of the materials occur.
- 4: The blocks undergo major relative displacements that compromise the geometric configuration (the structure collapses). The blocks show new through-going cracks that do not lead to fragmentation.



Mineralogical-petrographic alterations are more widespread but do not compromise the blocks' structural integrity.

5: The blocks undergo major displacements compromising the structure's geometry (the structure collapses). Original blocks present through-going cracks that result in fragmentation. Material alteration is such that it causes the disintegration of the blocks and their loss of integrity. The structure cannot be recovered.

4.1.4 Historical evolution of the Nuraghe Regional Network

The primary objective of this study was to create a unified database of the cultural heritage sites in the Sardinia region, specifically those from the pre-Nuragic period (5th to 3rd millennium B.C.) and the Nuragic period (18th to 6th centuries B.C.) [77]. In particular, the final database was designed for GIS use and formatted as a vector dataset in shapefile format. To achieve this, two publicly available inventories were utilized. The first was obtained from “SardegnaArcheologica,” an initiative focused on cataloguing and georeferencing Sardinia's archaeological sites, with a particular emphasis on Nuraghi (<https://www.sardegnaarcheologica.it/>). This open-access database is available for both online browsing and downloading and includes a significant number of monuments, such as Nuraghi and Giants' tombs, reflecting the extensive distribution of these structures across Sardinia. The second data source, “Nurnet – La rete dei Nuraghi,” stems from a participatory initiative aimed at documenting and promoting Sardinia's Nuragic and pre-Nuragic cultural heritage (<https://nurnet.crs4.it/nurnetgeo/>). This inventory is accessible through a dedicated Geoportal, a web-based platform that allows interactive access to data. Therefore, expert and non-expert users - such as citizens, tourists, associations, and institutions—can visualize and browse site information and also directly contribute by adding or updating data, including textual descriptions and photographs. Because of this, the data in this inventory is continually updated and undergoes a reviewing phase by experts in the field, which should be taken into account when using it.

For this study, we focused on different building typologies covered by both databases. To avoid duplication, we first extracted the chosen typologies from the SardegnArcheologica inventory and used the Nurnet database to complete the categories that were not present in the first source, as shown in Table 16.

Table 16. Building typologies retrieved from the two regional public inventories of archaeological monuments.

SardegnArcheologica	Nurnet
Nuraghi	Domus de Janas
Giants' tombs	Menhir
Springs	Dolmen
Sacred wells	Caves and shelters
Sanctuaries	Megalithic circles
Villages	

After downloading the two datasets in shapefile format, we first selected the chosen building typologies from the data sources, ensuring a common format was applied and essential information was simplified. The combination of these two data sources resulted in a total of 7,335 monuments located throughout Sardinia. Table 17 shows the percentage of coverage of various types of structures, while Figure 87 displays the Sardinian map with the completed regional database and the associated pie chart.

Table 17. Percentage of coverage of the various types of structures.

Megalithic circles	2.1%
Dolmen	1.3%
Domus de Janas	11.0%
Springs and sacred wells	4.2%
Cave	0.4%
Menhir	1.8%
Nuraghe - corridor	6.2%
Nuraghe - complex	15.1%

Nuraghe - complex mixed	0.9%
Nuraghe - singletower	35.0%
Sanctuary	0.4%
Giants' tomb	17.0%
Nuragic Village	4.4%
Shelter	0.2%

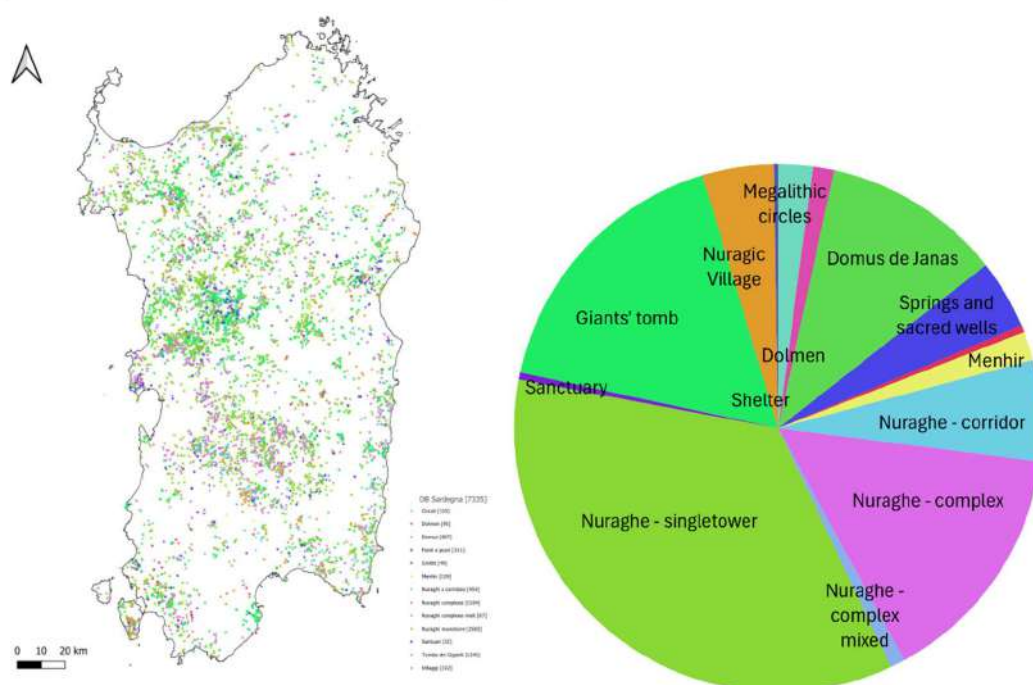


Figure 87. Completed regional database with the building typologies; pie chart showing typology percentages.



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4.1.5 Definition of a GIS database for the Network monitoring

The building typologies were characterized with specific attributes based on their geometry and dimensions, considering the overall proportions of each type of structure (tall, equidimensional, small, subterranean, wide). For the sake of simplicity, a numerical ID was assigned to each typology, as listed in Table 18, which also provides information regarding historical periods.

Table 18. Classification of the selected building typologies in terms of geometry and historical period.

ID	Building Typology	Geometry	Period
5.1	Nuraghe – corridor	Equidimensional (B)	1700-1350 A.C., Middle Bronze Age
5.2	Nuraghe – singletower	Equidimensional (B)	1500-1350 B.C.
5.3	Nuraghe - complex	Equidimensional (B)	
5.4	Giants' tomb	Wide (E)	Bronze Age
5.5	Sacred weels	Subterranean (D)	Bronze Age, early Iron Age
5.6	Domus de Janas	Subterranean (D)	1800-1600 B.C., Middle Neolithic
5.7	Cave	Subterranean (D)	From 5th to the end of the 3rd millennium A.C.
5.8	Shelter	Small (C)	From 5th to the end of the 3rd millennium A.C.
5.9	Menhir	Tall (A)	From Neolithic to Copper Age
5.10	Dolmen	Tall (A)	From Neolithic to Copper Age
5.11	Megalithic circles	Tall (A)	Middle Neolithic
5.12	Sanctuary	Wide (E)	
5.13	Nuragic Village	Wide (E)	1350 B.C. – 950 B.C.



In addition to the dimensional characteristics of each structure, we focused on the construction materials, which are crucial for assessing the monuments' vulnerability to various hazards. However, this information is not available for most of the mapped sites, with only a few significant ones from cultural and touristic perspectives providing material details. Consequently, some simplifying assumptions were necessary during this step of the analysis. Given the complexity of the territory and the significant topographical variations -even among nearby areas- it seemed unlikely that non-local materials would be used for construction. Therefore, we deduced the construction materials from the in-situ lithology. This assumption is also supported by real cases that confirm the use of local materials for constructions, especially considering the weight of the megalithic blocks. Notable examples include the famous Su Nuraxi of Barumini, Santu Antine, and Losa, which suggest that transporting massive stones over long distances would be impractical, leading to a preference for locally abundant stones.

Despite the supporting reasons for using lithological maps, this approach has some limitations. Indeed, while these maps can provide general geological information, they may not accurately reflect localized material selections. In certain cases, short-range transportation of materials might have been practical or necessary due to specific properties or symbolic significance. Furthermore, some changes that occurred after deposition could lead to discrepancies between the original materials and the current litho-logical context. Future steps will aim to validate this analysis through field surveys, archaeological reports, or the integration of other data sources to enhance reliability.

The construction material for each monument was determined through a spatial analysis conducted using QGIS software (<https://qgis.org/>), based on the official regional lithological map shown in Figure 88 (Sardegna GeoNetwork). For clarity, the map's legend in Figure 88 presents the classifications with associated alphanumeric labels corresponding to the different lithologies; additional information can be obtained by consulting the original data source.

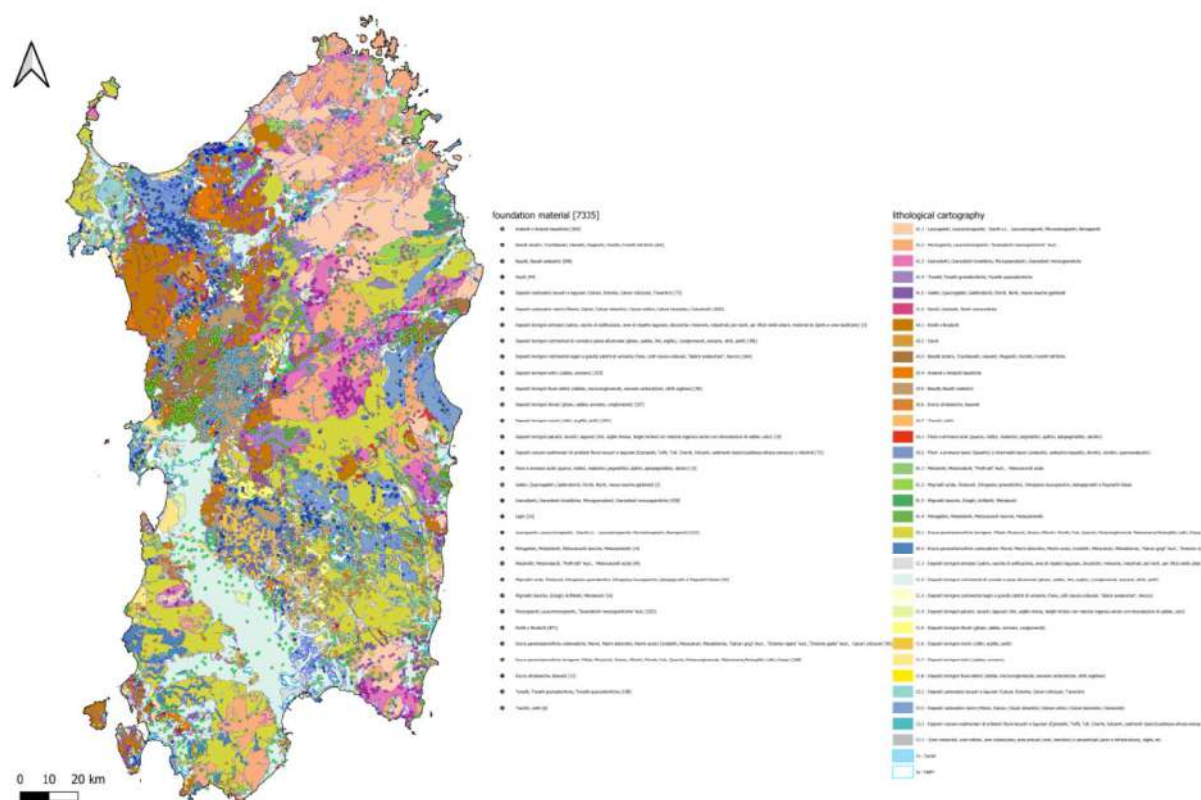


Figure 88. Official lithologic map of Sardinia (Sardegna GeoNetwork).

The official classification of lithologic materials includes 34 detailed categories, covering a variety of rock types as well as other classifications such as lakes and anthropogenic artefacts. Based on the above-mentioned assumptions, it was necessary to exclude categories that do not represent viable construction materials, specifically sedimentary rocks, lakes, and anthropogenic artefacts.

After sampling the lithologic map at the locations of the monuments and assigning the corresponding classes to each, a procedure using GIS and Python was implemented to address cases related to the excluded classes. For monuments located within polygons associated with the excluded categories, the algorithm searches the neighbouring polygons to identify the most probable lithology based on a detailed proximity principle. In particular, the analysis focused on the following parameters:



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- $dist\_b$: Shortest distance between the monument and the polygon's boundary
- $area$: Area of the polygon
- $dist\_c$: Distance between the monument and the polygon's centroid.

For each neighbouring polygon i , different weights were assigned as follows:

$$w_{i,dist\_b} = \frac{1}{dist\_b_i}$$

$$w_{i,area} = \frac{area_i}{\sum_i^n area_i}$$

$$w_{i,dist\_c} = \frac{1}{dist\_c_i^2}$$

Where n represents the total number of neighbouring polygons.

The final score was calculated as:

$$S_i = w_{i,dist\_b}^\alpha \cdot w_{i,area}^\beta \cdot w_{i,dist\_c}^\gamma$$

where α and β were set equal to 1, while γ was assigned a value of 2. The distances from the polygon's boundary and the area of the polygon were considered linearly, ensuring that polygons closer to the monument were preferred and larger polygons had a proportional impact without dominating the score. The distance from the polygon's centroid was given a quadratic weight, making it the most significant factor, and prioritizing it over the other two factors. Indeed, given the complexity and irregular shapes of the lithology polygons, a closer centroid suggests that most of the polygon is distributed near or around the monument. This weighting methodology helps to filter out polygons with misleading thin extensions or elongated parts that might otherwise appear relevant due to proximity based on boundary distance. Such polygons reaching close to the monument, having a small boundary distance, could falsely suggest a strong relevance. Moreover, the resulting ranking is more robust since the score decreases more sharply as the centroid distance increases compared to the other parameters (α and β). This ensures that only polygons with most of their mass near the monument remain highly ranked. By using this metric of

proximity, we achieved a better understanding of the polygon’s actual location, and we were able to identify the most likely construction material, corresponding to the lithology with the highest score.

Moreover, to enhance the selection of construction materials, an additional prioritization criterion was implemented. After scoring the neighbouring polygons, the algorithm selects candidates, favouring the one with the best construction material when multiple options exist. This conditional assignment ensures that when the highest quality materials are present, they are preferentially selected. This approach combines a rigorous metric-based selection process with informed lithological prioritization, thereby enhancing the accuracy of material attribution.

Next, the appropriate valid lithologic class was assigned to each monument, grouping them into macro-categories as shown in Table 19 to simplify the analysis. This clustering not only simplifies the classification process but also supports the underlying hypothesis of the spatial analysis used to derive the construction materials, as it averages the initial detailed categories.

The spatial analysis results for defining the construction material are illustrated in Figure 89.

Table 19. Grouped materials categories.

Official lithologic class	New class
A1.1, A1.2, A1.3, A1.4, A1.5, A1.6	Granites
A2.1, A2.2, A2.4	Trachytes auctorum
A2.3, A.2.5, A.2.6, A2.7	Alkaline vulcalites (basalts)
B1.1, B1.2, B1.3, B1.4, B2.1	Metamorphites
B2.2, C2.1, C2.2	Carbonates



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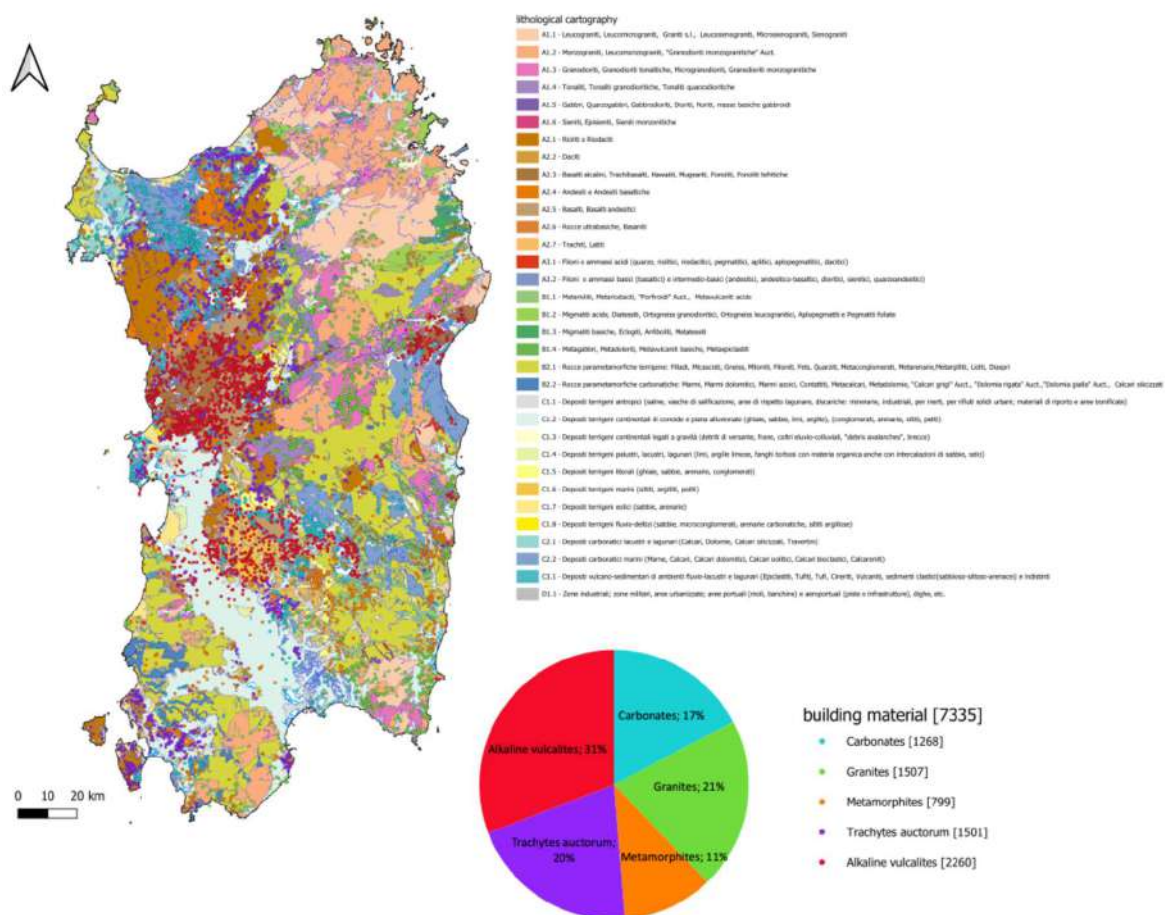


Figure 89. Completed regional database with the construction materials; pie chart depicting materials percentages.

4.2. Fire risk analysis

4.2.1 Wildfire hazard

The Geoportal of the Sardinia region was used to download the wildfire hazard map, given its specificity for the Sardinia region, which particularly suffers from this issue [78, 79] (Figure 90). The map combines several key parameters: ignitability, slope, exposure, altitude, road network, and urban settlements. Ignitability indicates how combustible a plant is and is analysed using the 2008 geographical Land Use Database published by the Sistema Informativo Territoriale Regionale (SITR). Slope significantly impacts the speed at which fire spreads, as fire propagates much more quickly on steep slopes compared to flat surfaces. This parameter is assessed using the 10-meter Digital Elevation Model (DEM) provided by the Sardinia Region Cartographic Service. Exposure refers to the terrain's orientation, which affects the amount of solar radiation locally received, influencing local humidity and air and soil's temperatures. Exposure is also processed using the same 10-meter DEM. Altitude is another important topographical factor that affects fire propagation, determined using the 10-meter DEM. The presence of a road network increases the likelihood of fire in nearby areas due to enhanced accessibility. This factor is assessed by identifying different areas around the road networks, as outlined in the SITR. Similarly, urban centers exert significant anthropogenic pressure, raising the fire risk in their vicinity. This parameter is determined by mapping areas surrounding the perimeters of urban centers and scattered settlements, using data retrieved from the SITR. To create this information layer, the regional territory was divided into a regular grid of 100 meters, and these areas were classified into four hazard levels: very low, low, medium, and high (<https://www.sardegnageoportale.it/>).

As a preliminary step, the official hazard map was resampled on the monuments' positions, generating an additional field that contained the corresponding official hazard class. Subsequently, the official categories were simplified, providing a numeric and unified classification divided into classes with equivalent meanings. The adopted approach is detailed in Table 20, which outlines the official hazard classes alongside their corresponding values for our analysis. For the wildfire hazard, the two classifications are equivalent since the official maps already employed a numeric classification. Following this step, a new field value, Hw was given to each monument.



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Table 20. Adopted classification on the hazard classes, starting from the official categories.

Wildfires w	
official	new
Very low - 1	1
Low - 2	2
Medium - 3	3
High - 4	4

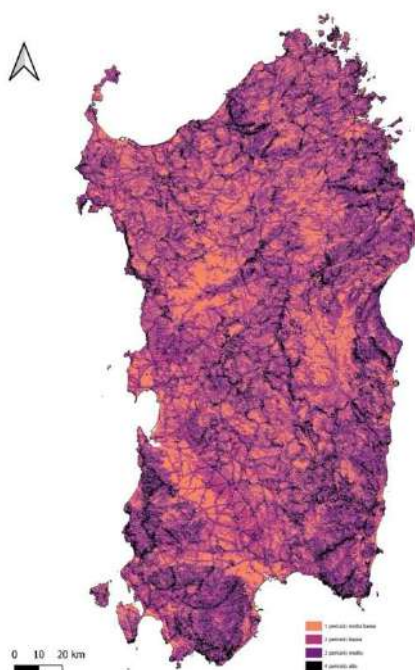


Figure 90. Wildfire Hazard Map from Geoportal of the Sardinia Region (<https://www.sardegna.geoportale.it/>).



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4.2.2 Procedure for fire vulnerability analysis

- *Geometric vulnerability*

Regarding geometric aspects, the vulnerability of structures to fire is related to their location (underground or on the surface), their height and depth (in the case of underground structures), their dimensions, and the number of blocks they are made of.

The geometric vulnerability to fire is determined by the structural characteristics that influence the extent to which the blocks are exposed to fire and high temperatures. Fire does not directly impact the structure as a whole (it does not cause displacement or exert force); instead, the effect is indirect: the structural stability is modified as a consequence of the fire's effect on the materials that make up the blocks (fracturing and/or disintegration – material fusion). As noted, the geometry of the structure determines the degree of exposure of the material to fire, thus affecting the overall vulnerability of the asset. In this context, while the exposure of the blocks (and thus the material) to fire is related to the geometry of the structure, the effect of fire on the material is represented by the material vulnerability indicator.

For the assignment of geometric vulnerability to each category of archaeological asset, based on the definition of the damage level, the following criteria were used.

Underground positioning protects structures from the development of high temperatures caused by fires and effectively prevents the effects of fires on the structures themselves; therefore, structures located underground are minimally or not at all vulnerable to fire.

The height of the structures is generally a factor of greater vulnerability due to two reasons:

- Greater thermal gradients, and thus thermal stresses, between the base and the top of the structure;
- The greater fall height of blocks in the event of structural disassembly into its elementary components (stone blocks). This greater height increases the likelihood of block breakage due to impact from the fall.



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The size of the blocks is also a vulnerability factor due to the greater impact of thermal expansion and resulting thermal stress on the fracturing of the blocks. In this regard, the case of menhirs and dolmens is significant, as both are composed of large monolithic stone elements. In these, thermal gradients can cause the formation of fractures that may split the original elements into multiple parts. On the other hand, structures made up of many small-sized blocks are less vulnerable to high temperatures due to smaller thermal gradients within the block volume: in this case, the discontinuities between blocks function as expansion joints.

Table 21. Wildfire hazard: geometric vulnerability classification of the Pre-Nuragic and Nuragic structures.

Geometric type	Archeological type	Damage Level after Fire (damage level specification)	Geometric vulnerability
Subterranean	Sacred weels Domus de Janas Cave	1 (the block material is not exposed: the structure is not changed)	None (1)
Wide Small	Giants' tomb Sanctuary Nuragic Village Shelter	2 (the block material is exposed – the blocks are small and suffer limited damage)	Low (2)
Equidimensional	Corridor Nuraghe Single tower Nuraghe Complex Nuraghe	3 (the block material is exposed – the blocks are large and suffer significant damage – the structure does not lose geometric integrity)	Medium (3)
Tall	Menhir Dolmen Megalithic circles	4 (the block material is exposed – the structures are monolithic; the blocks	High (4)



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		are cracked and fractured into smaller pieces, and the structure loses geometric integrity)	
-		5 (the block material is exposed – the blocks are cracked and fractured into smaller pieces, the structure loses geometric integrity – the material of the blocks is disintegrated)	Very High (5)

- *Material vulnerability*

Although rocks are not combustible materials, their structural stability can be compromised following exposure to high thermal stresses, especially if accompanied by thermal shocks or sudden temperature changes [80, 81]. The different types of lithic materials used for the construction of pre - nuragic and nuragic buildings exhibit specific behaviour in case of fire. This paragraph assesses the fire hazard vulnerability of the main lithologies employed, based on their mineral composition and the thermo-mechanical behaviour of the minerals composing them. Vulnerability is therefore quantified according to the presence of critical minerals, particularly quartz and calcite, which significantly influence the thermal and mechanical response of the rocks and determine the susceptibility of the materials to degradation [80, 81].

Granitoids

Granitoids are acidic intrusive rocks containing a variable percentage of quartz between 5% and 35% (3). At temperatures above 600°C, the crystalline phase of quartz transforms from alpha quartz to beta quartz, causing thermal expansion that can lead to the formation of microfractures (microcracking) and the detachment of surface fragments (spalling) [82, 83]. These effects are mainly superficial and do not significantly compromise the macrostructure of the rock blocks.



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Basalts

Basalts are basic effusive rocks free of quartz and other thermally critical minerals [82]. Their high thermal resistance makes them extremely stable even under fire conditions, with a very low susceptibility to structural deterioration.

Trachytes “auctorum”

Trachytes “auctorum” are acidic effusive rocks with a composition similar to granitoids, also containing quartz in percentages around 25-30% [84]. They show behaviours similar to granitoids in case of fire, with microcracking and spalling phenomena starting at approximately 600°C.

Metamorphic rocks

The metamorphic rocks involved in nuragic constructions include lithologies such as gneisses and mica schists, containing quartz in amounts comparable to granitoids (5-35%) [82]. Here too, the presence of quartz represents the main vulnerability factor, with effects comparable to those observed in granitoids and rhyolites.

Carbonate rocks (limestones and marls)

Carbonate rocks, composed mainly of calcite (90-100%) [82], are the most vulnerable to fire. Calcite begins to decompose by calcination at around 750°C, causing a significant loss of cohesion and structural integrity of the rock matrix. This process results in severe deterioration, even at a deep structural level, which can evolve into the pulverization of the lithic matrix [85].

Table 22. Wildfire hazard: vulnerability classification of the materials used for the construction of Pre-Nuragic and Nuragic structures.

Material Type	Damage Level after Fire (damage level specification)	Material Vulnerability
	1 (lithotype undergoes color changes but no form of degradation)	None (1)



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Basalts	2 (lithotype undergoes color changes and degradation pathologies such as slight fissuring (microcracking))	Low (2)
Granitoids Metamorphic rocks Trachytes “auctorum”	3 (lithotype undergoes color changes and degradation pathologies such as fissuring and detachment of surface fragments)	Medium (3)
	4 (lithotype with severe compromise of material consistency/composition)	High (4)
Carbonate rocks	5 (lithotype loses even material consistency, becoming totally inefficient and unrecoverable)	Very High (5)

4.2.3 Risk assessment

Having completed the monuments' database with the associated classes of vulnerabilities and hazard enabled the assessment of the wildfire risk R_w , computed as:

$$R_w = H_w \cdot V_{geom,w} \cdot V_{mat,w}$$

Subsequently, a normalization was applied to adjust the final values to the five-level scale, considering the minimum and maximum possible risk values $R_{w,min}$ and $R_{w,max}$:

$$R_{w,min} = 0$$

$$R_{w,max} = H_{w,max} \cdot V_{geom,w,max} \cdot V_{mat,w,max} = 4 \cdot 5 \cdot 5 = 100$$

At this point, the risk value was reclassified according to the criteria outlined in Table 23, which divides values from 1 to 100 into 5 distinct risk classes.

Table 23. Risk classification according to the 5-level scale.

0-20	20-40	40-60	60-80	80-100
1	2	3	4	5
VL	L	M	H	VH

The map reporting wildfire risk values associated with each considered monument is shown in Figure 91.

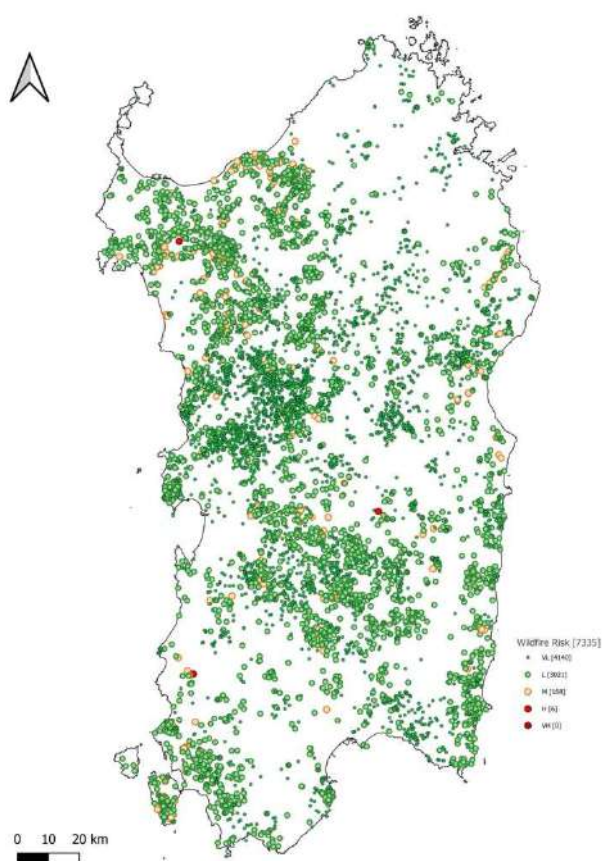


Figure 91. Wildfire risk map.

4.3. Landslide Risk analysis

4.3.1 Landslide hazard

The hazard analysis was based on the official maps sourced from the PAI [86]. In particular, the "hydrogeological" map was specifically used to indicate landslide risk exposure (Figure 92). As a preliminary step, the official hazard map was resampled on the monuments' positions, generating an additional field that contained the corresponding official hazard class. Subsequently, the official categories were simplified, providing a numeric and unified classification divided into five classes with equivalent meanings. The adopted approach is detailed in Table 24, which outlines the official hazard classes alongside their corresponding values for our analysis. Following this step, a new field value, HI was given to each monument.

Table 24. Adopted classification on the hazard classes, starting from the official categories.

Landslides /	
official	new
Hg0	0
Hg1	1
Hg2	2
Hg3	3
Hg4	4
No value	0



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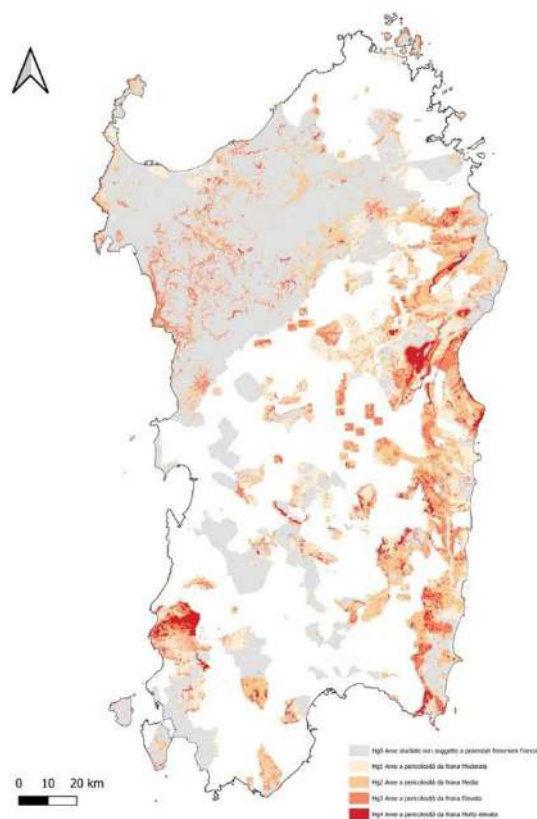


Figure 92. PAI official map for landslides hazard.

4.3.2 Procedure for landslide vulnerability analysis

- *Geometric vulnerability*

The action of a landslide can involve lateral pressure from the moving debris or rock blocks, instability, or removal of the foundation soil, and partial or total burial of the structure. Unlike fires, the effects of landslides act directly on the structure as a whole and can alter its static equilibrium, causing sliding, rotation, overturning, or collapse of the blocks.

With regard to geometric aspects, the vulnerability of structures to landslides is strongly influenced by their underground or surface location, the height-to-width ratio (h/w) of the structure, and the size and



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weight of the elements from which they are made. To determine the geometric vulnerability of each category of archaeological asset—based on the definition of damage levels—the following criteria were applied:

- Subterranean locations represent a protective factor: in the event of a landslide, the structures may at most be filled by debris flows. In such cases, debris can exert pressure causing minor displacement of structural elements, but this does not severely damage the structure.
- Extended and horizontal structures, such as Giants' Tombs and sanctuaries, with a low height-to-width ratio, offer greater resistance to lateral pressure and tend to maintain stability, although they may experience relative displacements.
- As the height-to-width ratio increases (in equidimensional and especially tall structures), structural instability increases following the impact of landslide material. The fall height of displaced blocks also increases, and they may be destroyed upon impact with the ground.
- Small structures, such as rock shelters, offer little resistance to landslide pressure and can be easily deformed or overturned.

Table 25. Landslide hazard: geometric vulnerability classification of the Pre-Nuragic and Nuragic structures.

Geometric type	Archeological type	Damage Level after Landslide (damage level specification)	Geometric vulnerability
		1 (the structure is not affected by the event and is intact, in a good state of preservation; the blocks do not undergo relative movement)	None (1)
Subterranean Wide	Sacred weels Domus de Janas Cave Giants' tomb Sanctuary Nuragic Village	2 (structural elements may undergo relative displacements that do not compromise overall stability)	Low (2)



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Equidimensional	Corridor Nuraghe Single tower Nuraghe Complex Nuraghe	3 (structural elements can undergo displacements that challenge the overall stability without leading to the collapse of the structure)	Medium (3)
Small	Rock shelter	4 (the structure undergoes major relative displacements that challenge its geometric structure)	High
Tall	Menhir Dolmen Megalithic circles	5 (the structure undergoes major relative displacements that challenge its geometric structure, and the original blocks have newly formed through-going fractures that generate fragmentation)	Very High

- *Material vulnerability*

Due to risks associated with landslides, floods, and earthquakes, incidental scenarios may occur that lead to the collapse or displacement of the blocks composing archaeological structures. In such situations, the blocks may undergo degradation phenomena resulting from impacts with the ground or with other moving blocks. The vulnerability of the blocks is therefore closely related to the mechanical strength of the materials from which they are made.

In the literature, several indices have been proposed to assess the tendency of rocks to degrade due to impacts and falls. However, for the materials considered in this study — particularly rocks outcropping in Sardinia— there is no established or standardized classification for the values assumed by such indices.

For this reason, the parameter chosen to represent the vulnerability of the materials is the Uniaxial Compressive Strength (UCS). This parameter, well documented in the literature and widely applied across various fields, shows good correlation with the tendency of rocks to fracture under impact and mechanical stress, both in natural contexts and during extraction and processing activities [87, 88, 89, 90, 91, 92, 93, 94, 95, 96].

Once the UCS values for the examined materials were determined, they were related to the Impact Strength Index (ISI), an index that quantifies the ability of a rock sample to retain its original dimensions after undergoing a standardized impact test. The correlation between UCS and ISI values was defined according to the equation provided by [93] (Figure 93). The classification of material vulnerability, based on ISI values, was organized into classes by discretizing the range 66–94%, corresponding to the validity interval of the UCS–ISI relationship used.

Material Vulnerability Classes:

- 1: lithotype that undergoes no degradation (intact) ($ISI \geq 89\%$).
- 2: lithotype that does not fracture but may exhibit minor surface degradation due to abrasion ($ISI: 82\text{--}88\%$).
- 3: lithotype that shows degradation phenomena such as fissuring and detachment of surface fragments ($ISI: 75\text{--}81\%$).
- 4: lithotype affected by the development of numerous through-going fractures ($ISI: 68\text{--}74\%$).
- 5: lithotype that loses material consistency, becoming totally inefficient and irrecoverable ($ISI \leq 67\%$).

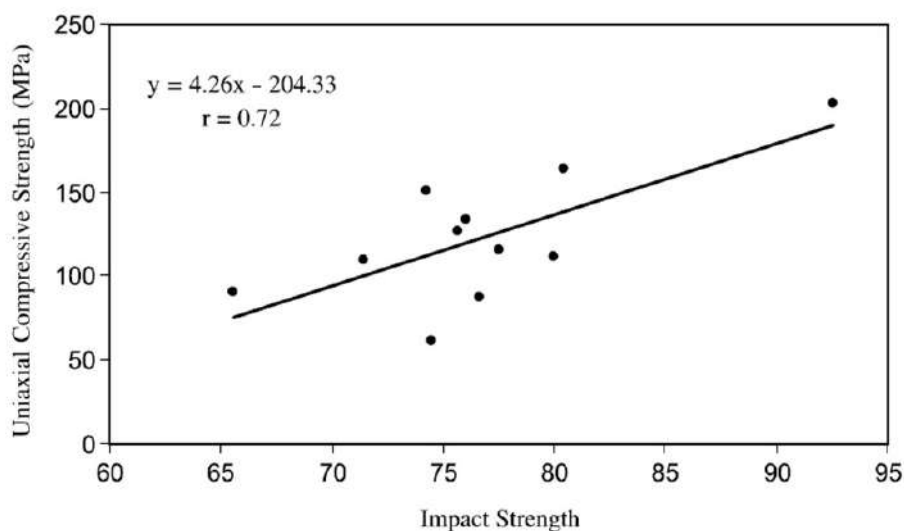


Figure 93. UCI – ISI correlation, from [93].

Uniaxial Compressive Strength of the materials:

- Granitoids: Palmström [97] reports values between 95 and 230 MPa for granites, and between 75 and 135 MPa for granodiorites, based on data previously published Hansens [98] by Hoek and Brown [99] and Beniaowski [100]. Specifically for Sardinian granites, the literature provides values ranging from 60 to 250 MPa [101].
- Basalts: Palmström [97] indicates values between 100 and 355 MPa, based on data previously published by Hansens [98], Hoek and Brown [99], and Beniaowski [100]. Specifically for Sardinian basalts, the literature reports values ranging from 30 to 200 MPa [101].
- “Auctorum” Trachytes: The term trachite auctorum historically referred to a broad range of volcanic rocks (rhyolites, rhyodacites, dacites, and andesites). For rhyolites, Palmström reports an average uniaxial compressive strength of 85 MPa, based on a study by Lama and Vutukuri [102]. Sardinian trachytes have compressive strength values ranging from 15 to 100 MPa [103, 101].
- Metamorphic Rocks: Palmström [97] reported values between 80 and 185 MPa for gneisses and between 35 and 105 MPa for schists, based on results obtained by Hansens [98] and Hoek and Brown [99]. Specifically for schists in Sardinia, the literature provides values between 100 and 120



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MPa [101]. The presence of weakness planes and foliation favours the division of the material into thin layers and blocks.

- Carbonate Rocks (limestones and marls): Palmström [97] indicates values between 50 and 180 MPa for limestones and between 60 and 300 MPa for dolomites. The data for limestones come from analyses conducted by the Technical University of Norway (NTH), Institute for Rock Mechanics, while the values for dolomites are from Lama and Vutukuri [102]. Specifically, for carbonate rocks in Sardinia, the literature reports values between 20 and 180 MPa for both limestones and marls [101].

Table 26. Relationship between the UCS values of the lithotypes present in Sardinia, their ISI value and their vulnerability to landslide, flood and earthquake hazards.

	Granitoids	Basalts	"Auctorum" Trachytes	Metamorphic rocks	Carbonate Rocks
UCS [MPa] (Sardinian mean value)	155	115	58	110	100
ISI [%]	84	75	61	74	71
Material vulnerability	Low (2)	Medium (3)	Very High (5)	High (4)	High (4)

4.3.3 Risk assessment

Having completed the monuments' database with the associated classes of vulnerabilities and hazard enabled the assessment of the landslides risk R_l , computed as:

$$R_l = H_l \cdot V_{geom,l} \cdot V_{mat,l}$$

Subsequently, a normalization was applied to adjust the final values to the five-level scale, considering the minimum and maximum possible risk values $R_{l,min}$ and $R_{l,max}$:

$$R_{l,min} = 0$$

$$R_{l,max} = H_{l,max} \cdot V_{geom,l,max} \cdot V_{mat,l,max} = 4 \cdot 5 \cdot 5 = 100$$

At this point, the risk value was reclassified according to the criteria outlined in Table 27, which divides values from 1 to 100 into 5 distinct risk classes.

Table 27. Risk classification according to the 5-level scale.

0-20	20-40	40-60	60-80	80-100
1	2	3	4	5
VL	L	M	H	VH

The map reporting landslides risk values associated with each considered monument is shown in Figure 94.



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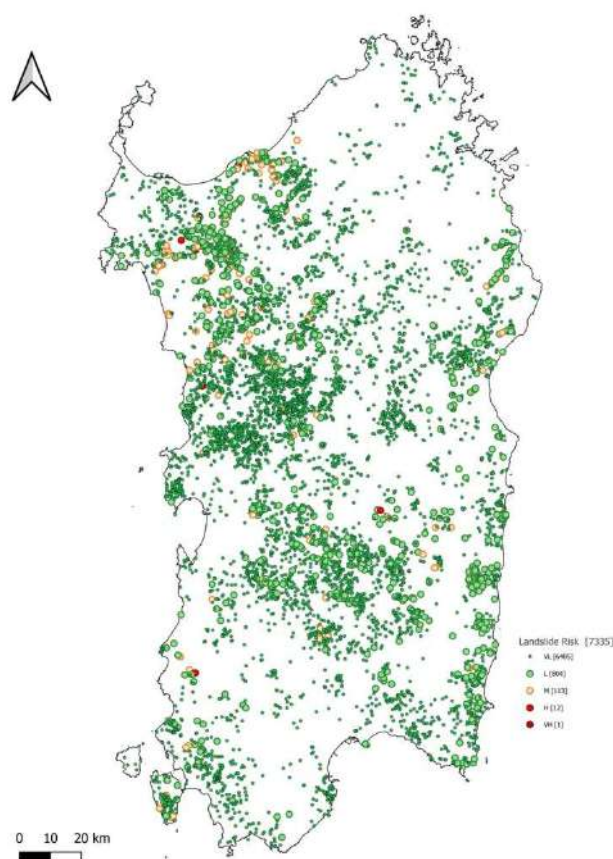


Figure 94. Landslides risk map.

4.4. Seismic risk analysis

4.4.1 Seismic hazard

Since Sardinia is known to be particularly stable from a seismic point of view [104, 105, 106], no seismic hazard map was employed, and a constant hazard value, H_s , was applied. A value $H_s = 1$ was assigned to the seismic hazard to indicate the low seismicity in Sardinia while preventing a total underestimation of the issue (without using class 0).

4.4.2 Procedure for seismic vulnerability analysis

- *Geometric vulnerability*

Earthquakes generate dynamic forces that can cause instability, collapse, or disassembly of structures. In this context, with regard to geometric aspects, the vulnerability of structures is related to their underground or surface location, their height and depth (in the case of underground placement), their dimensions, and the number of blocks (and consequently, discontinuities) that compose them.

In particular, underground structures—especially those at significant depths—tend to experience lower horizontal accelerations and generally exhibit low seismic vulnerability.

Among surface structures, those that are horizontally extended usually have good resistance due to their low center of gravity and wide base, which increase static stability. However, if they are made up of poorly connected blocks, they may undergo local dislocations. Conversely, tall structures are the least stable in the presence of seismic oscillations, as their high height-to-width ratio (h/w) results in a higher likelihood of overturning of the structure or of its components (due to a high overturning moment). Furthermore, the greater falling height of blocks in the event of structural disaggregation into elementary components (e.g., stone blocks) increases the probability of block breakage due to impact upon falling.

Equidimensional structures show an intermediate level of vulnerability between that of tall structures and extended structures.

Small structures are subjected to relatively low forces during earthquakes (as seismic forces are proportional to the mass of the structure) and are therefore classified as having intermediate vulnerability.

Table 28. Seismic hazard: geometric vulnerability classification of the Pre-Nuragic and Nuragic structures.

Geometric type	Archeological type	Damage Level after Landslide (damage level specification)	Geometric vulnerability
		1	None (1)



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		(the structure is not affected by the event and is intact, in a good state of preservation; the blocks do not undergo relative movement)	
Subterranean	Sacred weels Domus de Janas Cave	2 (structural elements may undergo relative displacements that do not compromise overall stability)	Low (2)
Wide Small	Giants' tomb Sanctuary Nuragic Village Rock shelter	3 (structural elements can undergo displacements that challenge the overall stability without leading to the collapse of the structure)	Medium (3)
Equidimensional	Corridor Nuraghe Single tower Nuraghe Complex Nuraghe	4 (the structure undergoes major relative displacements that challenge its geometric structure)	High
Tall	Menhir Dolmen Megalithic circles	5 (the structure undergoes major relative displacements that challenge its geometric structure, and the original blocks have newly formed through-going fractures that generate fragmentation)	Very High

- *Material vulnerability*

Due to risks associated with landslides, floods, and earthquakes, incidental scenarios may occur that lead to the collapse or displacement of the blocks composing archaeological structures. In such situations, the blocks may undergo degradation phenomena resulting from impacts with the ground or with other moving blocks. The vulnerability of the blocks is therefore closely related to the mechanical strength of

the materials from which they are made. On the basis of the above, the assessment of the vulnerability of materials to earthquakes was carried out in line with what has already been described for landslides (4.3.2.2).

4.4.3 Risk assessment

Having completed the monuments' database with the associated classes of vulnerabilities and hazard enabled the assessment of the seismic risk R_s , computed as:

$$R_s = H_s \cdot V_{geom,s} \cdot V_{mat,s}$$

Subsequently, a normalization was applied to adjust the final values to the five-level scale, considering the minimum and maximum possible risk values $R_{s,min}$ and $R_{s,max}$:

$$R_{s,min} = 0$$

$$R_{s,max} = H_{s,max} \cdot V_{geom,s,max} \cdot V_{mat,s,max} = 4 \cdot 5 \cdot 5 = 100$$

At this point, the risk value was reclassified according to the criteria outlined in Table 29, which divides values from 1 to 100 into 5 distinct risk classes.

Table 29. Risk classification according to the 5-level scale.

0-20	20-40	40-60	60-80	80-100
1	2	3	4	5
VL	L	M	H	VH

The map reporting seismic risk values associated with each considered monument is shown in Figure 95.



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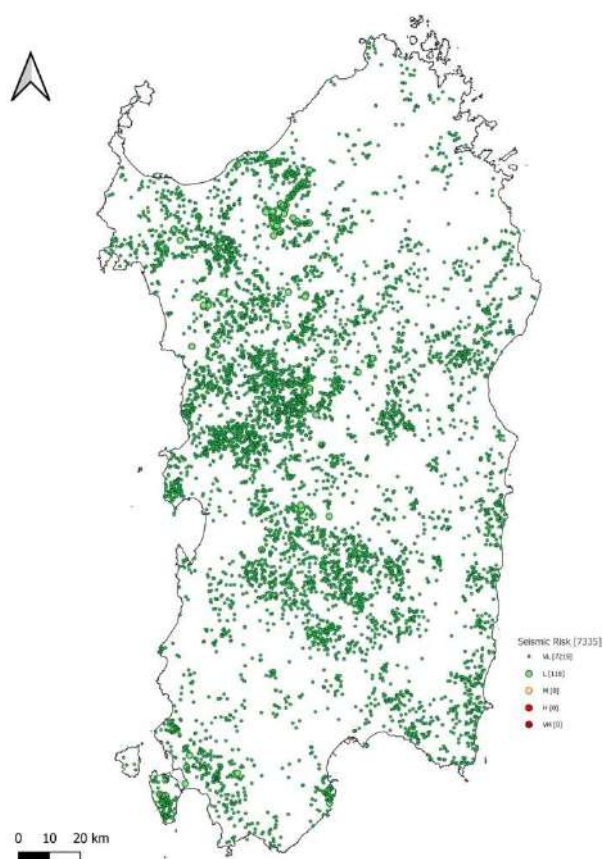


Figure 95. Seismic risk map.

4.5. Hydraulic risk analysis

4.5.1 Hydraulic hazard

The hazard analysis was based on the official maps sourced from the PAI (Piano Stralcio per l'Assetto Idrogeologico) [86] (Figure 96). As a preliminary step, the official hazard map was resampled on the monuments' positions, generating an additional field that contained the corresponding official hazard class. Subsequently, the official categories were simplified, providing a numeric and unified classification divided into five classes with equivalent meanings. The adopted approach is detailed in Table 30, which outlines the official hazard classes alongside their corresponding values for our analysis. Following this step, a new field value, H_h was given to each monument.

Table 30. Adopted classification on the hazard classes, starting from the official categories.

Hydraulic h	
official	new
Hi0	0
Hi1	1
Hi2	2
Hi3	3
Hi4	4
No value	0



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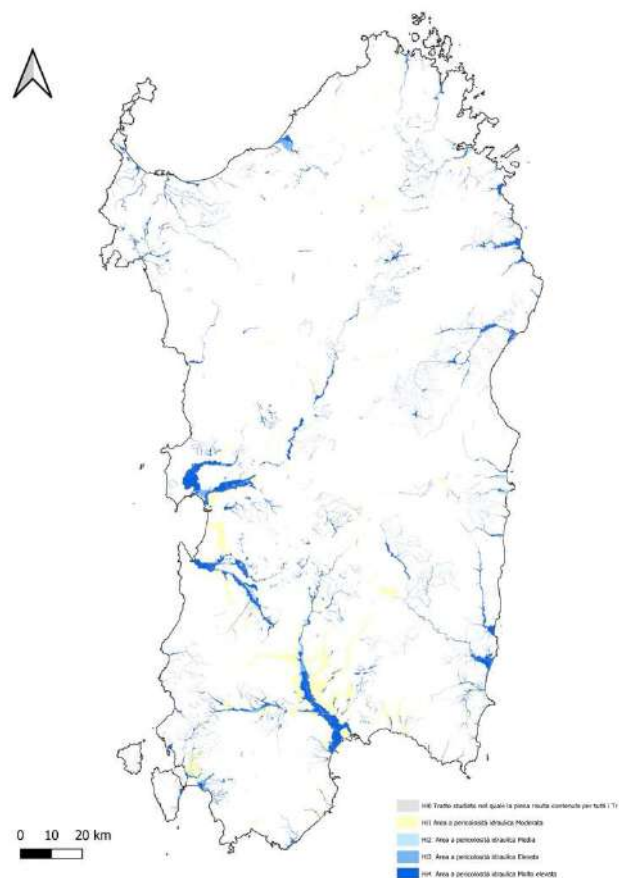


Figure 96. PAI official map for hydraulic hazard.

4.5.2 Procedure for hydraulic vulnerability analysis

- *Geometric vulnerability*

Floods exert primarily horizontal forces on structures, mainly due to hydrostatic pressure, as well as erosive and/or scouring actions at the base, and flooding. The geometric vulnerability of structures to floods is defined based on their underground or surface location, the height-to-width ratio (w/l) of the structure, and the size and weight of the elements from which they are made.



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In particular:

- Underground structures and small-sized structures can be completely flooded or submerged, suffering significant damage, though typically not to the point of becoming irrecoverable.
- Extended structures (such as sanctuaries or Giants' Tombs) risk being submerged for a significant portion of their height. However, the very high weight of their constituent elements provides strong resistance to water pressure and ensures that the damage sustained by individual elements does not compromise the overall stability of the structure.
- Equidimensional structures (e.g., nuraghi) and tall structures (e.g., menhirs, dolmens) come into contact with water only along a limited portion of their geometry. They may experience minor displacements of the base blocks—very large, heavy, and loaded by the weight of the overlying structure—which do not compromise global stability nor cause the blocks to be displaced from their original positions.

Table 31. Hydraulic hazard: geometric vulnerability classification of the Pre-Nuragic and Nuragic structures.

Geometric type	Archeological type	Damage Level after Landslide (damage level specification)	Geometric vulnerability
		1 (the structure is not affected by the event and is intact, in a good state of preservation; the blocks do not undergo relative movement)	None (1)
Equidimensional Tall	Corridor Nuraghe Single tower Nuraghe Complex Nuraghe Menhir Dolmen Megalithic circles	2 (structural elements may undergo relative displacements that do not compromise overall stability)	Low (2)
Wide	Giants' tomb Sanctuary	3	Medium (3)



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	Nuragic Village	(structural elements can undergo displacements that challenge the overall stability without leading to the collapse of the structure)	
Subterranean Small	Sacred weels Domus de Janas Cave Rock shelter	4 (the structure undergoes major relative displacements that challenge its geometric structure)	High
		5 (the structure undergoes major relative displacements that challenge its geometric structure, and the original blocks have newly formed through-going fractures that generate fragmentation)	Very High

- *Material vulnerability*

Due to risks associated with landslides, floods, and earthquakes, incidental scenarios may occur that lead to the collapse or displacement of the blocks composing archaeological structures. In such situations, the blocks may undergo degradation phenomena resulting from impacts with the ground or with other moving blocks. The vulnerability of the blocks is therefore closely related to the mechanical strength of the materials from which they are made. On the basis of the above, the assessment of the vulnerability of materials to earthquakes was carried out in line with what has already been described for landslides (Errore. L'origine riferimento non è stata trovata.).

4.5.3 Risk assessment

Having completed the monuments' database with the associated classes of vulnerabilities and hazard enabled the assessment of the hydraulic risk R_h , computed as:

$$R_h = H_h \cdot V_{geom,h} \cdot V_{mat,h}$$

Subsequently, a normalization was applied to adjust the final values to the five-level scale, considering the minimum and maximum possible risk values $R_{h,min}$ and $R_{h,max}$:

$$R_{h,min} = 0$$

$$R_{h,max} = H_{h,max} \cdot V_{geom,h,max} \cdot V_{mat,h,max} = 4 \cdot 5 \cdot 5 = 100$$

At this point, the risk value was reclassified according to the criteria outlined in Table 32, which divides values from 1 to 100 into 5 distinct risk classes.

Table 32. Risk classification according to the 5-level scale.

0-20	20-40	40-60	60-80	80-100
1	2	3	4	5
VL	L	M	H	VH

The map reporting hydraulic risk values associated with each considered monument is shown in Figure 97.

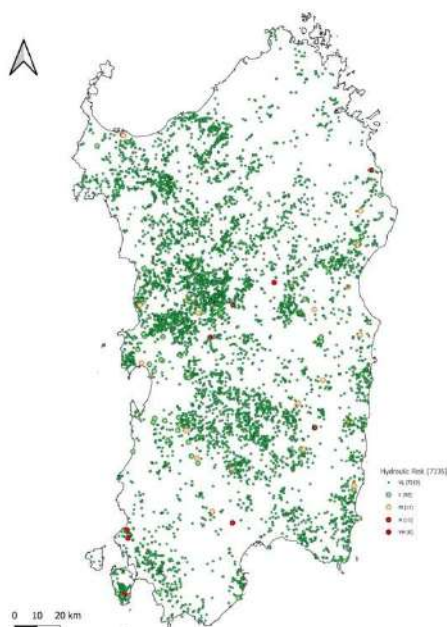


Figure 97. Hydraulic risk map.

4.6. Multi-risk analysis

4.6.1 Monuments

For each monument, we previously calculated the risk values associated with each individual hazard and their normalizations according to the 5-levels scale.

To assign the multi-risk value to each monument, we employed two different methods, referred to as the National and Regional Methods.

- *National Method*

This method is based on the normalized individual risks. For each monument, the four classified risks (R_h, R_l, R_w, R_s) each varying from 1 to 5, are summed:

$$sum R_i = R_h + R_l + R_w + R_s$$

The total value can range from 0 to a maximum of 20 (5 multiplied by 4) and is then reclassified into customized a 5-level scale, with non-uniform bin sizes, as outlined in Table 33, obtaining the multirisk value *MR*.

Table 33. Multirisk classification according to the National Method.

0-5	6-8	9-10	11-14	15-20
1	2	3	4	5
VL	L	M	H	VH

The results of this method present a regional perspective based on a national framework. This national approach is specifically applied to a subset of cultural heritage, including monuments from the pre-Nuragic and Nuragic periods, and focuses on a smaller spatial scale limited to the Sardinia region. Overall, the multi-risk values, shown in Figure 98, suggest that Sardinia is less vulnerable than other regions, partly due to its low hazard values.



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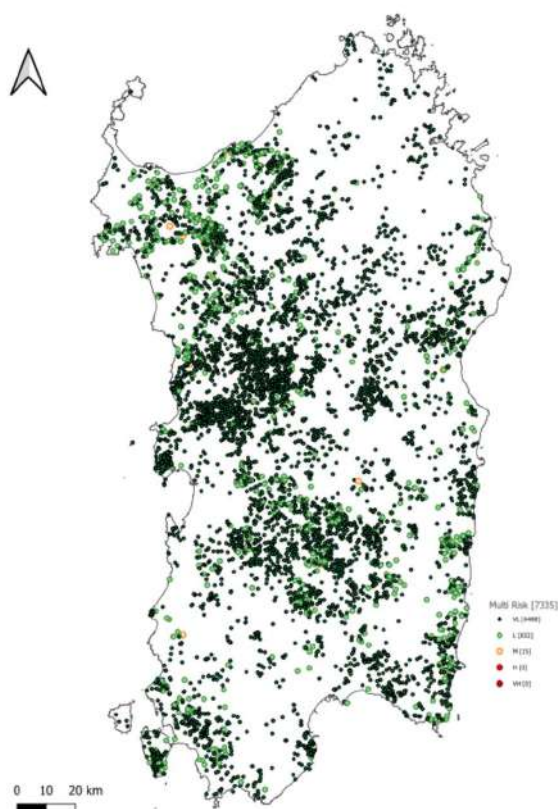


Figure 98. Multirisk map – National Method.

- *Regional Method*

To obtain more useful information on a regional level, we proposed a classification normalized specifically on a regional basis, considering the maximum risks present in our situation.

In this approach, we consider for each monument the continuous values (before the 5-level classification) of the four individual risks, each ranging from 0 to 100. These values, R_i , are then normalized by dividing each risk by the maximum value among the four risks, ensuring that the impact of each hazard is balanced. Therefore, for each monument and for risk type i , we have:

$$R_{N_i} = \frac{R_i}{\max(R_h, R_l, R_w, R_s)}$$

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The four normalized values are then summed to produce a final score ranging from 0 to 4, where 0 indicates no risks and 4 represents maximum exposure to every risk.

$$sum\ R_{N_i} = R_{N_h} + R_{N_l} + R_{N_w} + R_{N_s}$$

The final multirisk for each monument value is determined by applying a 5-level scale with uniform bin sizes to the range of 0 to 4, as illustrated in Table 34.

Table 34. Multirisk classification according to the Regional Method.

0.0-0.8	0.8-1.6	1.6-2.4	2.4-3.2	3.2-4.0
1	2	3	4	5
VL	L	M	H	VH

Figure 99 displays the multi-risk map according to the Regional method for all the selected monuments along with a pie chart illustrating the percentage of each multirisk class.



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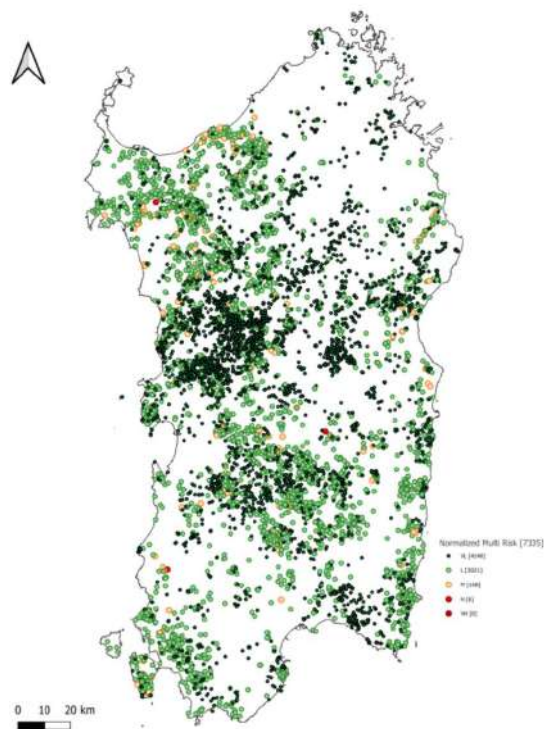


Figure 99. Multirisk map – Regional Method.

This map illustrates the prevalence of very low and low multi-risk classes, with only six monuments categorized as high risk and none classified as very high risk. Upon examining these cases more closely, we found that they are associated with tall structures, specifically five megalithic circles and one menhir. These structures are primarily made of carbonatic rocks and are located in areas with a high risk of landslides.

4.6.2 Municipalities

To evaluate the multirisk values at the municipality level, a spatial analysis was conducted to link each monument with its respective municipality (Figure 100).



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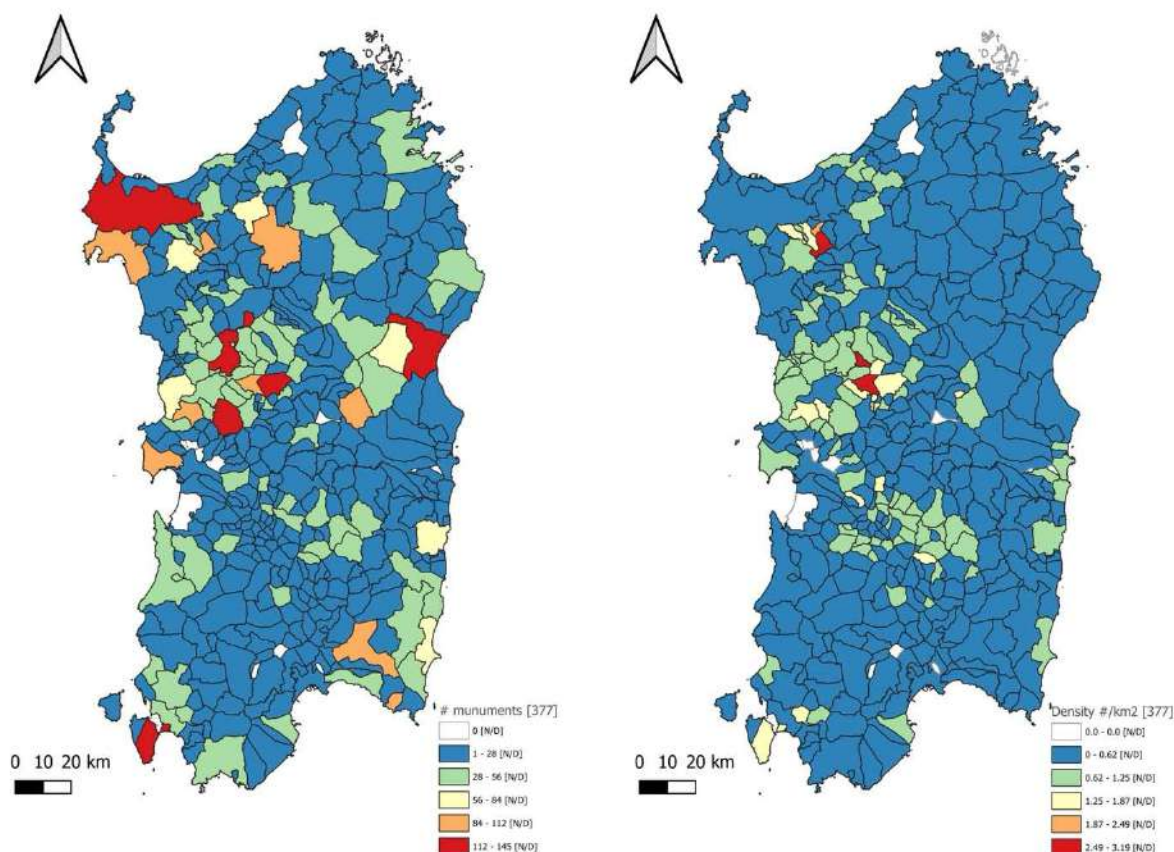


Figure 100. Spatial analysis on the municipalities: number of monuments (left), and density (right).

The multirisk assessment method for municipalities starts by analysing individual risks instead of relying on previously calculated multirisk values for each monument. First, all the monuments are aggregated at the municipal level, and their risk values are summed within each municipality. This total is then divided by the maximum risk value among all municipalities, resulting in four normalized risk values that range from 0 to 1 for each municipality. The multirisk is calculated by summing these four normalized values, which can result in a range from 0 (if the risk associated with each hazard is zero) to 4 (if all risks are at their maximum of 4). Finally, this continuous value is classified according to a discrete five-level scale, as outlined in Table 35.

Table 35. Multirisk classification of municipalities.

0.0-0.8	0.8-1.6	1.6-2.4	2.4-3.2	3.2-4.0
1	2	3	4	5
VL	L	M	H	VH

Figure 101 displays the multi-risk map at municipality level according to the National Method, along with a pie chart illustrating the percentage of each multirisk class.

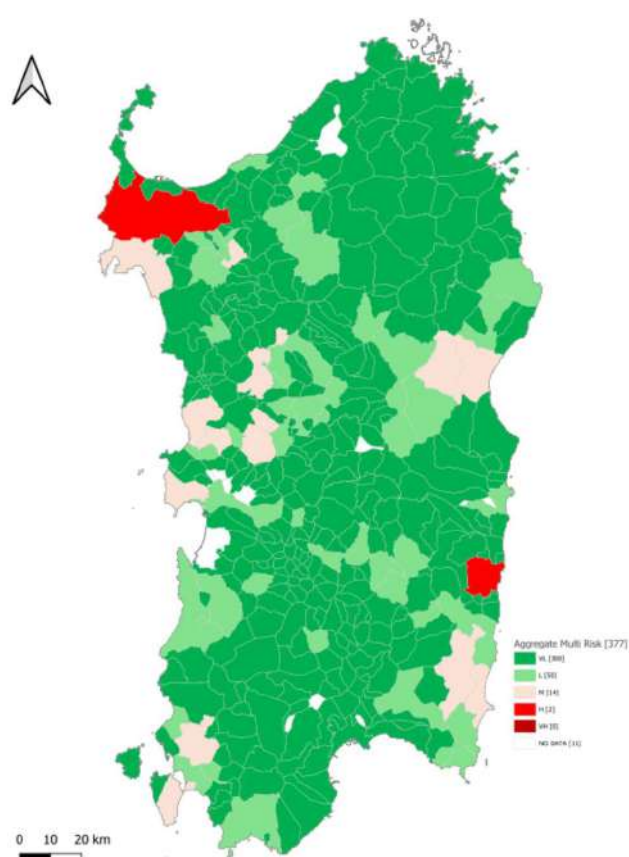


Figure 101. Municipalities multirisk map.

5 CH monitoring at asset scale – Mono-tower Nuraghe

5.1. The proof of concept: The mono-tower Nuraghe

The present paragraph has been extracted from a work currently under review by an international journal [107].

The Nuragic civilization developed in Sardinia over a period extending from the Early Bronze Age to the Early Iron Age (approximately 1800–700 B.C.) [37, 108, 109, 50] [110] [111]. A key distinguishing feature of this prehistoric culture is the widespread construction of stone-built architectural structures, typically circular in shape and with varying degrees of complexity [57, 112], known as Nuraghe [113, 114].

The Nuraghe constructions have undergone a distinct architectural evolution, from the first archaic form, known as the corridor Nuraghe [115, 116], to the following mono-tower nuraghi, between the Middle and the Recent Bronze Ages (from the 16th to the 12th centuries BC, a period which scholars still debate), characterized by a large central chamber, sometimes with multiple floors, which afterwards developed into more complex structures made of multiple towers, walls and internal courtyards [37, 108, 117] [111].

The mono-tower nuraghe is characterized by a single, vertically developed structure known as the Nuragic Tower, that constitute the elementary architectural nucleus of Nuragic complexes.

The Nuragic Tower (mono-tower nuraghe) consists of the following structural elements: a truncated cone-shaped external wall (external tower), an internal chamber sheltered by a corbelled dome (or corbel vault), also reported by local bibliography with the Greek term tholos, and a filling granular material between the internal and external walls [51, 109, 50].

The Nuragic Tower is sometimes integrated with a corridor that allows access to the internal chamber, with niches contained in the thickness of the internal wall, and one or more staircases between the tholos and the external tower [51, 109]. From a structural point of view, one fundamental aspect regarding the Nuragic construction is the absence of binding materials [51, 108, 47]; in fact, both the internal and external wall are dry-built structures made of overlapping rock blocks, while the space in between is filled

with granular loose material [118, 117, 119], meaning that only compressive and shear stresses are transmitted through the joints, while tensile stresses may occur solely within the blocks.

Over time the Nuragic structures have been the object of many studies and archaeological analyses aimed at defining the historical, social and cultural issues concerning the development of the Nuragic civilization, also within the broader context of the Mediterranean societies [120] [121], as well as their function and symbolic meaning [110] [120] [122]. Those studies are still ongoing, and up to the present time, there is not an univocal interpretation of what has been observed [123].

While the geometric properties of the structural visible elements can be easily acquired, not much is known about the internal characteristics of the Nuragic Towers. However, it is possible to perform indicative estimates from direct observation of the exposed parts of partially destroyed Nuraghes. The relevant parameters of the tower's 3D model, described in the next paragraph, have been derived from the available scientific literature and through direct observation, in order to define the ranges of values which describe a typical Nuragic Tower. For each data category, the consistent literature sources are also reported.

5.1.1 Geometry and materials

- *Nuragic Tower geometry*

The visible parts of an intact Nuraghe are the truncated cone-shaped external tower, from outside, and the ogival chamber, from inside. Data on the geometry of both structures are reported in many documents [37, 108, 47, 50, 57, 124, 125], from which the ranges of dimensional values hereby reported have been acquired.

The diameter at the base of the external wall is between 10 and 15 m, the height between 7 and 16 m and the vertical inclination between 8° and 15° [47, 124].

The diameter at the base of the internal chamber (tholos) is between 3.5 and 6 m, while the chamber height varies between 4 and 11 m. Some authors have processed the geometrical data of the internal chamber, finding a correlation with a rotational paraboloid [126, 127, 128, 119, 124, 125]. This hypothesis

is hereby assumed, with specific reference to Cappai [127]; Cappai and Pulina [128] and Campus and Cappai [119]. For a vertical section containing the paraboloid axis, the profile of the internal wall is described in Figure 103, with the parabola vertex at the origin of a Cartesian system with downwards y-axis.

Equation (1) represents the parabola in Figure 102:

$$y = ax^2 \quad (1)$$

With $F(0; 1/4a)$ and $y = -1/4a$ the parabola focusses and the directrix equation, respectively. If h_t is the chamber height and r_t the radius at the base, $h_t = ar_t^2$. This implies that given a base radius, the height expected for the internal chamber uniquely defines its shape.

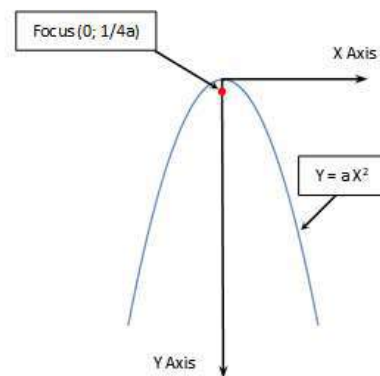


Figure 102. Graphic representation of the tholos internal wall.

- *Blocks geometry*

Depending on the workability of the rock material and in order to increase the overall stability of the Nuragic Tower, the geometry of the blocks changes from trapezoidal prisms to V or T shaped ashlar [126, 129, 47, 117, 124, 125], while their arrangement goes from polygonal to isodomic masonry [128, 50]. To

define the 3D model, a perfectly isodomic structure has been assumed to be representative of both the internal and external wall.

As regards the dimension of the blocks some useful indications are reported by Contu [47]. The blocks at the base of the walls are generally a meter or more, corresponding to boulders of several tens of kN. It has been observed that the dimensions of the blocks decrease with the height of the tower, with the blocks of the internal chamber smaller than those of the external wall at the same height.

To the best of the authors' knowledge, there is no indication in the scientific literature of a function which correlates the position of the block (i.e. height) with its size. This relationship was therefore estimated by integrating the indications of the scientific literature with the additional data acquired on-site by direct observation of the Nuragic Towers having dimensions consistent with the model (the size of blocks may, in fact, differ for bigger nuraghes). As mentioned above, the depth of the blocks can be acquired from partially collapsed Nuraghes or from the top of the structures, when the blocks are exposed.

The acquired data was elaborated to estimate the mean dimensions of the blocks as a function of the wall's height. For the external and internal wall (tholos), respectively, Figure 103 e Figure 104 show the variation of the blocks size (height and depth) with the tower height. The blocks mean dimensions are reported as blue dots, the standard deviation as red error bars and the best fit curves as blue dotted lines.

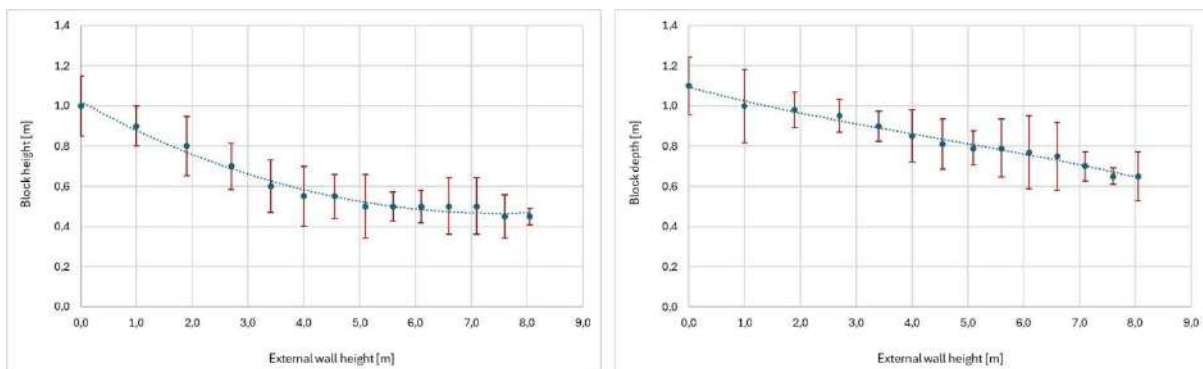


Figure 103. External wall: (a) Heights of the Blocks Depending on the Height of the Wall; (b) Depth of the Blocks Depending on the Height of the Wall.



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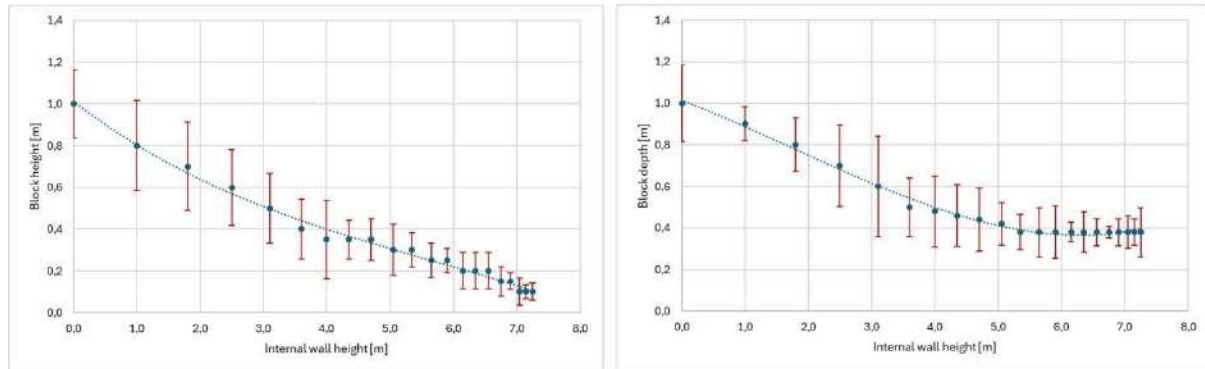


Figure 104. Tholos (left) Heights of the Blocks Depending on the Height of the Wall; (b) Depth of the Blocks Depending on the Height of the Wall.

- *Blocks material*

The Nuragic structures are spread throughout the entire Sardinian territory [50] and the rocks used for their construction can be clearly correlated to the specific geological zones they belong to. The rocks mainly used are basalts, rhyolites, granitoids, limestones, marly limestones, and shales [37, 108, 47, 130, 128, 125].

In order to perform the stability analysis, the fundamental geo-mechanical parameters hereby considered are the rock density, between 29 kN/m^3 (basalt) and 21 kN/m^3 (marly limestones) [131, 132], and the friction angle between the blocks, ranging from slightly above zero to values above the unit, depending on both the block material and shape. Other geo-mechanical properties, such as the compressive, tensile and shear resistance, as well as the deformation coefficients, are not taken into account, as they are considered irrelevant due to significant values of the loads under consideration [130]. This means that the rock blocks are assumed to be infinitely resistant and rigid.

- *Backfill material*

The type of filling material placed in the space between the internal and external walls varies considerably from one structure to another. A variety of materials are reported in the literature, from shapeless stones of variable sizes incorporated into the soil [117], to rubble [50, 127, 128, 119, 124, 125] or sub-rectangular packed blocks with cavities filled with small-sized stones or wedges [126, 118, 133].



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All these types of backfill materials can generate horizontal forces, which specifically depend on the internal friction angle [134]. Apart from the density, in the range between 15 and 20 kN/m³, the filling material is characterized by the coefficient $k = \sigma_0 / \sigma_v$, which represents the ability to generate a horizontal load σ_0 when subjected to a vertical load σ_v . Consequently, the horizontal forces can be amplified by the compaction operations presumably carried out during construction. In this work, various hypotheses are developed about the characteristics of the backfill material and the effect deriving from its compaction.

The photos in Figure 105 show two examples of backfill materials in Nuraghe Sa Domu e' S'Orcu, Domusnovas (5a), and Nuraghe Mereu, Orgosolo (5b).



Figure 105. Backfill Material: (a) Soil and Stones of Various Sizes, in Nuraghe Sa Domu e' S'Orcu, Domusnovas (SU); (b) Stones, in Nuraghe Mereu, Orgosolo (NU).

5.1.2 Case of study: The Nuraghe Longu in the Mielogu Sardinian area

The historic region of Meilogu is an area inhabited since the earliest times, characterized by a high density of monuments. This richness is also due to favorable environmental conditions: the presence of fertile alluvial plains, watercourses, and abundant building materials such as trachyte and tuff favored settlement and the monumentalization of the territory. The orographic features also ensured effective

territorial control. The Nuraghe Longu is located on the plain crossed by the Rio Puddiu, at about 300 meters above sea level.

This nuraghe, well preserved and never previously subjected to in-depth studies, has been the object of a detailed geometric and material survey. The results of these analyses are presented in the following sections.

- *Geometric characterization*
 - Terrestrial Laser Scanner (TLS)

The Terrestrial Laser Scanner (TLS) survey was conducted using a Faro Focus Premium Laser Scanner (Figure 106), a high-precision 3D scanning device designed to capture detailed and accurate spatial data of complex environments. It features an integrated high-resolution RGB camera that enhances scans with true-colour imagery, making it ideal for applications in construction and architecture.



Figure 106. Faro Focus Premium. (Left image source: <https://www.faro.com/>).

The TLS survey of Nuraghe Longu covered both the interior and exterior of the structure (see Figure 107 and Figure 108), involving a total of 23 scans divided as follows: 10 scans for the exterior, 1 scan connecting the interior and exterior, 2 scans inside the atrium, 1 central scan within the tholos along with 3 additional scans—each targeting a niche; and 6 scans of the stairs, with approximately one scan taken every 1.3 to 1.5 meters.



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Figure 107. Exterior TLS survey of the Nuraghe.



Figure 108. Interior TLS survey of the Nuraghe.

Given the complexity of the structure, different parameters were used during acquisition to thoroughly capture all parts of the Nuraghe. For the exterior, scans were taken with 4x quality and a resolution of 260



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1/5, achieving a point density of about 1 point every 7 mm at 10 meters. Inside, a resolution of 1/10 with 4x quality (1 point every 15.3 mm at 10 meters) was chosen due to the limited space around the instrument. For the exterior, eight targets were placed around the structure to facilitate both the alignment of the scans and the georeferencing process (see Figure 109), as they were also surveyed using a GNSS receiver (detailed in the next paragraph).

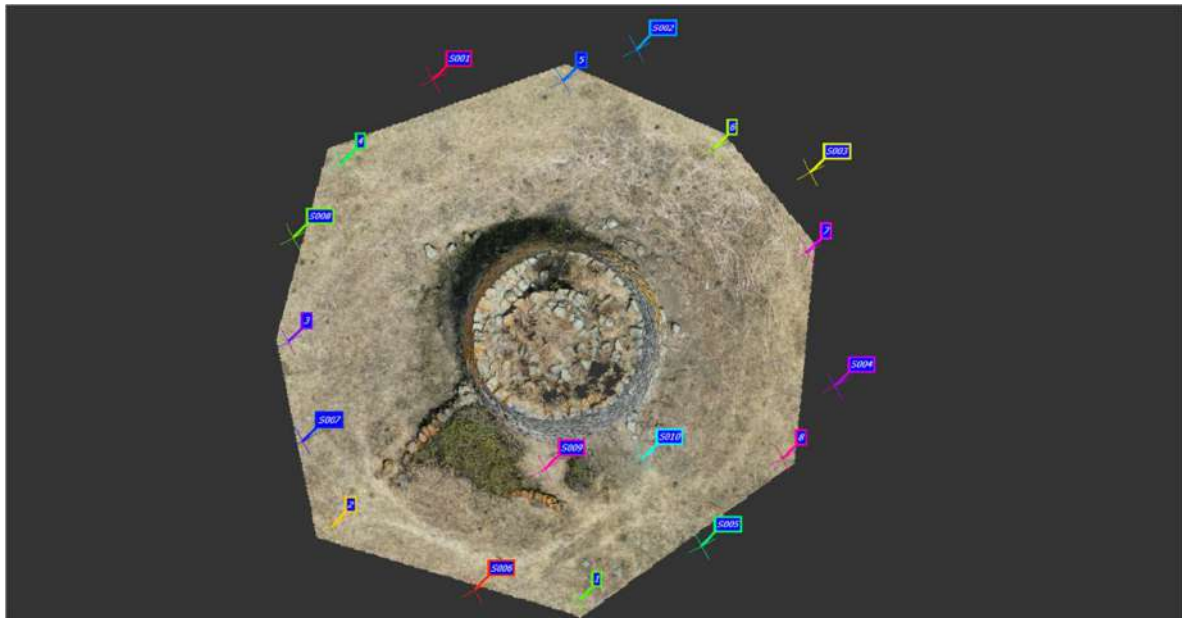


Figure 109. Exterior of the Nuraghe: scans positions and targets positions.

Regarding the interior, we needed a different approach due to the almost complete lack of natural light in most areas, as well as the inability to use GNSS-acquired points. To address this, we utilized several polystyrene spheres, which were easily recognizable by the Terrestrial Laser Scanner (TLS), to facilitate the alignment of the various scans (Figure 110).



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Figure 110. 360-degree views from the TLS, showing the arrangement of the polystyrene spheres.

To address the issue of darkness inside the Nuraghe, which affected the colours in the point cloud, we used LED lights arranged on the scanner (Figure 108). These lights moved in conjunction with the scanner to create more uniform illumination within the interior. However, the narrow spaces made it challenging to achieve consistent results and not all scans of the interior displayed realistic lighting. Nevertheless, the outcomes were certainly better than having no light or relying solely on directional lighting.

The TLS scans were processed using Reconstructor software version 4.4.2. First, the various scans were registered together by applying the Iterative Closest Point (ICP) algorithm [135]. Next, the GNSS coordinates of the targets were utilized and the resulting model was aligned to the RDN2008 / UTM zone 32N (N-E) reference system using the GNSS coordinates. The point clouds from different scans were

aligned, resulting in a final average residual of 1 mm, while the mean georeferencing error was about 1.6 cm.

The following images depict different views of the TLS point cloud.

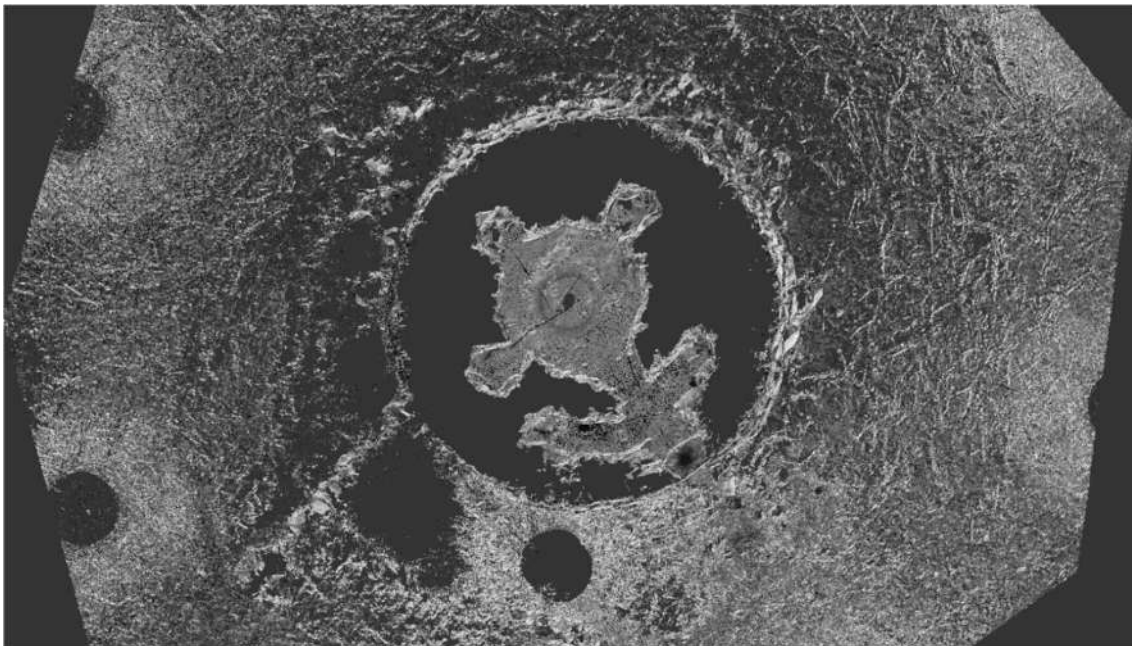


Figure 111. Horizontal section of the point cloud.



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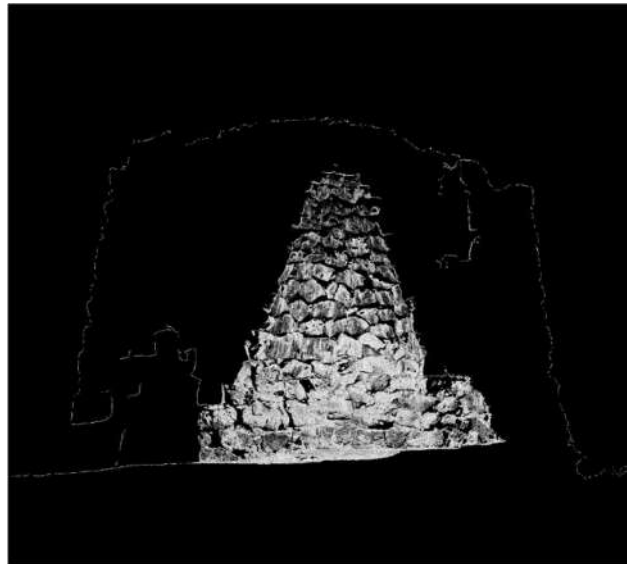


Figure 112. Interior view of the tholos from the point cloud.

○ Global Navigation Satellite Systems (GNSS)

Surveying the external area allowed the use of targets for the georeferencing process (Figure 113). A total of 8 points were surveyed using a Ublox multi-constellation receiver operating in NRTK (Network Real-time Kinematic) mode, which received corrections from the Sardinian SARNET Network (www.sarnet.it/servizi.html). The positions of these targets are shown in Figure 114, and their coordinates have been aligned to the ETRS89-ETRF2000 (2008.0) / UTM zone 32N (N-E) reference system (EPSG:6707). The ConveRgo software (<https://www.cisis.it/>) was used to obtain the orthometric heights of the Ground Control Points (GCPs) by utilizing the ITALGEO05 geoid height grid (GK2 format) [136, 137].



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Figure 113. A Leica GZT21 Scanning Target (left – Source: <https://q2survey.com/>), the GNSS survey of the targets (right).



Figure 114. Location of the GNSS targets (Source: Qgis software).

○ Unmanned Aerial Vehicle (UAV)

A photogrammetric dataset was collected using a Mini 3-DJI Pro equipped with an RGB sensor. A total of 825 images, both nadiral and oblique, were captured while flying at a maximum relative altitude of approximately 10 meters (Figure 115).

Table 36. Characteristics of the UAV camera.

Camera Model	Resolution	Focal Length	Pixel Size	Precalibrated
FC3582 (6.72mm)	4032 x 3024	6.72 mm	2.4 x 2.4 μm	No

The drone maneuvered around the Nuraghe to capture its entire view. The flight was manually planned to ensure a forward overlap of 80% and a side overlap of 70%. For the georeferencing process, the same ground control points (GCPs) of the TLS survey were used (Figure 109, Figure 114). These GCPs were evenly distributed around the Nuraghe also to minimize deformation effects in the resulting model [138, 139, 140]. As mentioned, the coordinates of the GCPs were aligned with the official reference system in Sardinia (EPSG: 6707) using a Ublox GNSS receiver. The UAV survey successfully captured the roof of the Nuraghe, which was not included in the TLS survey.



Figure 115. Example of an image taken by a UAV.



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The UAV dataset was processed using the Structure from Motion (SfM) [141] workflow integrated into Metashape software. After aligning all 825 captured images with the “Highest” accuracy setting, the dense point cloud was generated by selecting “High” quality and “Mild” filtering parameters. The final point cloud was georeferenced using the coordinates of the markers achieving a total error of 1 cm on the Control points and 3 centimetres on the Check point (Table 37).

Table 37. RMSE values on the Control points and on the Check point (X - Easting, Y - Northing, Z – Altitude).

Type	Count	X error (cm)	Y error (cm)	Z error (cm)	Total (cm)	Image (pix)
Control point	7	0.360743	0.432435	0.841254	1.01235	1.573
Check point	1	1.44709	2.47104	0.917984	3.00712	2.329

Additionally, the UAV dataset allowed the creation of the georeferenced Digital Elevation Model (DEM) and orthophoto (Figure 116).

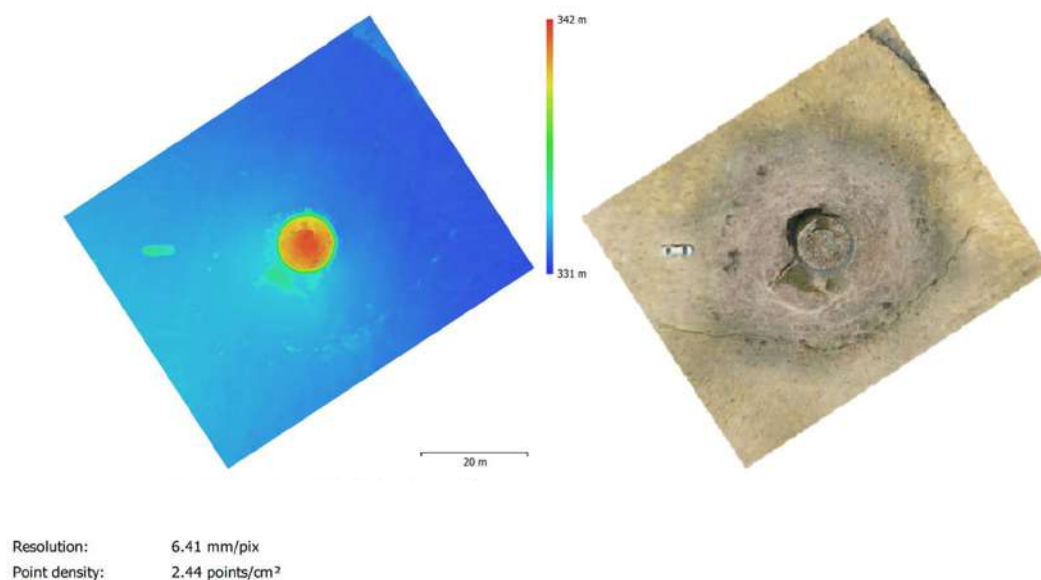


Figure 116. Digital Elevation Model (DEM) (left) and orthophoto (right) of the area surveyed by the UAV (orthometric heights above the sea level).



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○ 3D modelling

The primary products of the TLS and UAV surveys were utilized for the 3D modelling and reconstruction of both the interior and exterior of the Nuraghe. This work serves as a crucial source of information for conducting further analysis [138, 142, 143].

The following images depict various views of the UAV point cloud: North-East, South-East, South-West, and North-West.

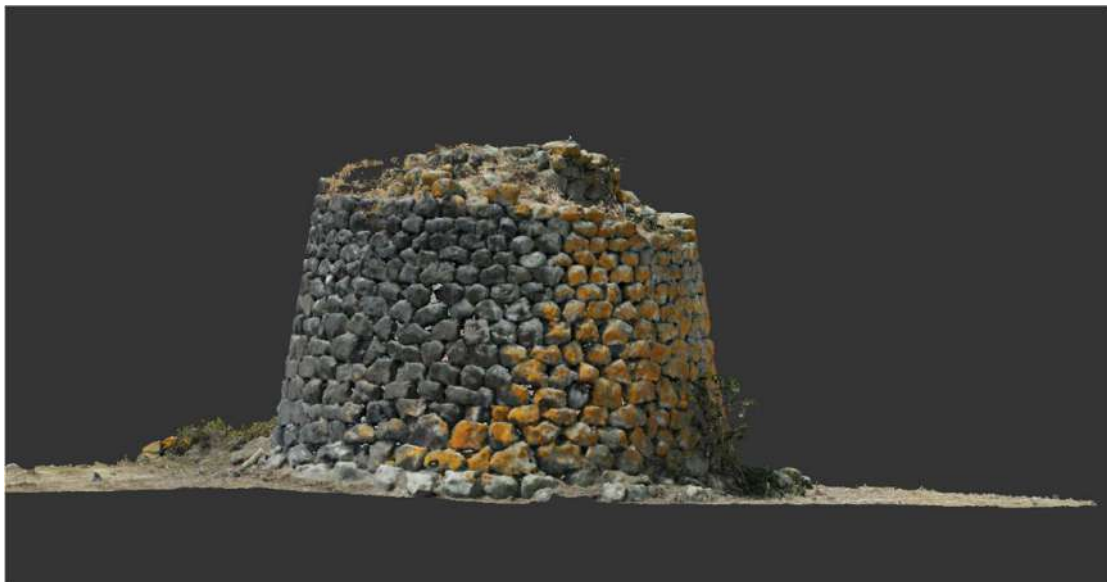


Figure 117. View of the UAV point cloud from North-East



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Figure 118. View of the UAV point cloud from South-East.



Figure 119. View of the UAV point cloud from South-West.



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Figure 120. View of the UAV point cloud from North-West.

- *Morphologic and material descriptions*

- Geometry

The Longu Nuraghe shows an external diameter of 11.6 m at the base, while the interior tholos-vaulted room measures 4.3 m. The latter is characterised by three niches (1,30 m long – 1,15/30 large) arranged in a cross with respect to the entrance (with an average height of 1.5 meters). The truncated cone structure narrows towards the other and at the height of 95 cm h outer diameter of 10.6 m, inner about 3 m. The stairwell, on the other hand, is 80 cm wide (Figure 121).



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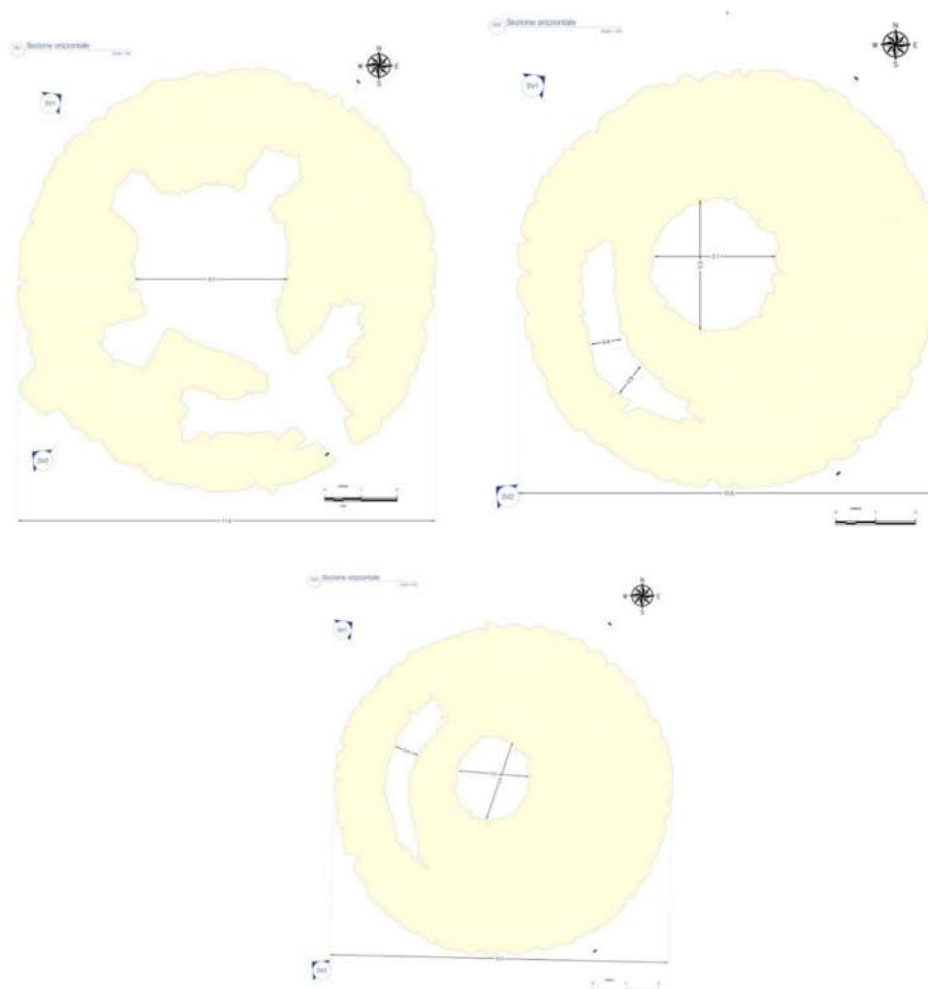


Figure 121. Horizontal sections (height 0,95 m, 3,95 m e 5,45 m).

The monument is externally tall around 8 m while the tholos is tall around 7 m (Figure 122).



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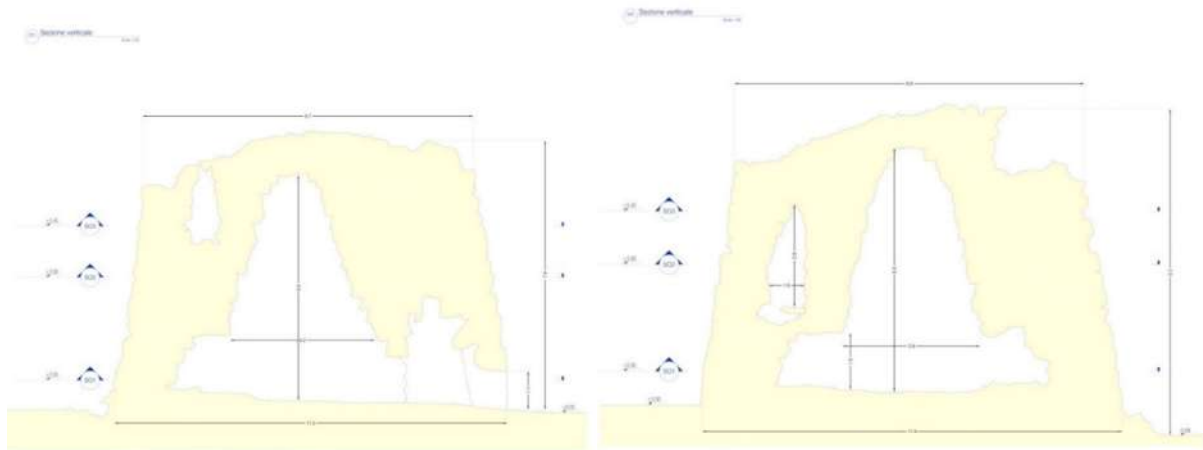


Figure 122. Vertical sections.

Masonry technique

The masonry techniques, which is double and about 1m thick, is made of polygonal dry stonework consisting of basalt blocks of decreasing size from the bottom to the top.

Given the analysis of the 12 rows of overlapping blocks, it was possible to highlight how the height of the blocks varies from 66 cm (lintel) up to 38 cm, while their length, also decreasing as one goes upwards, ranges from approximately 1.53 - 1 m in the first 4 rows to between 1 m and 60 cm (Figure 123).

The gradual decrease in the size of the blocks towards the top, a technical and structural choice typical of cyclopean architecture, contributes to stability and mechanical strength, allowing efficient distribution of loads and minimizing the risk of instability or slippage.



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Figure 123. Detail of the masonry techniques.

○ Materials

The materials used, as is normally the case for historical assets, are local ones; this makes the artefact take on not only a historical-architectural value, but also an environmental one, representing the fusion between the man-made landscape and the natural one.

In this context, it is useful to recall Acocella's statement that “stone material communicates a sense of topological belonging with immediacy, and in the encounter between the naturalness of rough stone and the work of man, architecture finds a strong bond with the individual and the community” (Acocella, 2004). By reading the history of materials and the infinite possibilities of their applications, one can understand the close link that has always existed between stone and architecture.

The geological landscape in which Nuraghe Longu is located is mainly characterised by basaltic, detrital and limestone-arenaceous lithologies [144] (Foglio 193) (Figure 124). In particular, the nuraghe is located on land characterised by q3l, indicating recent alluvial fills of the main valleys with Nuragic and Eneolithic



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archaeological layers. Nearby there are outcrops of β_2 , augitic-olivine basalts, β_1 , dark and black boiled and vesicular basalts (compatible with those used in the construction of Nuraghe Longu).



Figure 124. a) Extract of ISPRA sheet 193 - Bonorva scale 1:100,000. Localisation of the Nuraghe Longu; b) Basalt surface.

The study involved the characterisation of natural stones and mortar samples, taken at various points in the internal tholos of the nuraghe, useful for understanding the different types of basalts and their state of conservation (Figure 125). The analytical investigation was carried out using instrumental techniques, such as transmitted light optical microscopy and X-ray diffraction, in accordance with the UNI - Normal ICR and CNR Recommendations [145].

Specifically, the analyses were carried out at the LabMast (Mediterranean Laboratory for Materials and Historical-Traditional Architecture) and the DICAAR laboratories. The mineral-petrographic study was carried out on X-ray diffractometry analyses, on powder samples, with a Rigaku diffractometer, Ulrima IV, in parallel optics, with Cu-Kalfa radiation, 30kV and 30mA, and 5°-65° pattern.

Through reasoned sampling, 2 samples of bedding mortars and 3 of basaltic stone were selected.



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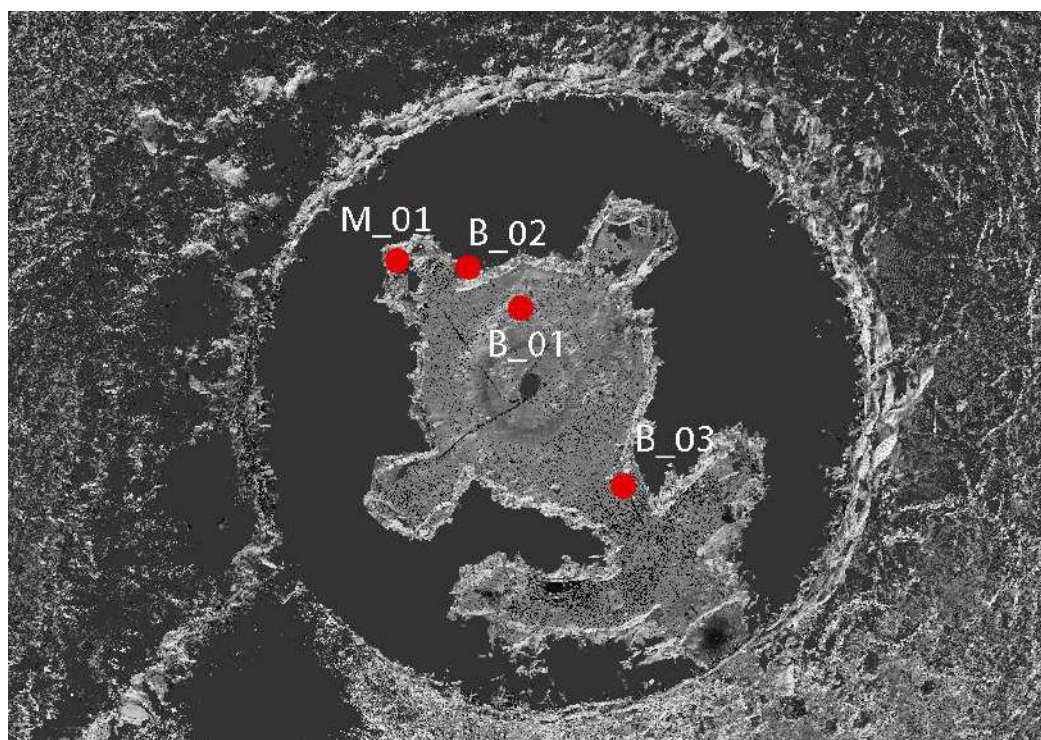


Figure 125. Localisation of material samples.

As concerns basalts, the three samples collected: two came from the ground inside the tholos (B_01 and b_03) and one (B_02) from the interstitial space between two blocks. These samples show how the basalts can have different textures, moving from the most compact basalt (B_03) to vesicular (B_01 e 02). Compact basalt is widely recognized for its exceptional durability, exhibiting high compressive strength and elastic modulus, often exceeding 300 MPa and 100 GPa, respectively. In contrast, vesicular basalt tends to have lower mechanical performance due to the presence of macroscopic voids — in some cases reaching several centimetres — formed by the expansion of gas bubbles within the magma during solidification. Conversely, before the development of Iron Age technology, prehistoric humans were capable of mass-producing only the vesicular variety, as the available tools—initially made of stone, and later of copper and bronze—were not suitable for working the more compact and harder types [146]. For that reason, all the nuraghe is made by vesicular basalts with some exceptional cases of small pebbles of compact basalt (B_03). This samples of basalts, supported by consolidated mineralogical studies by optical



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microscope essentially reveals that these rocks have a porphyritic texture characterized by phenocrysts of calcic plagioclase and olivine. The phenocrysts come in a fine crystalline groundmass, with intersertal texture, in which natural volcanic glass occupies the wedge between plagioclase laths. Plagioclase occurs in two generations as larger phenocrysts and as small tabular to elongated microlites in the groundmass (Figure 127).



Figure 126. Samples of Basalt.

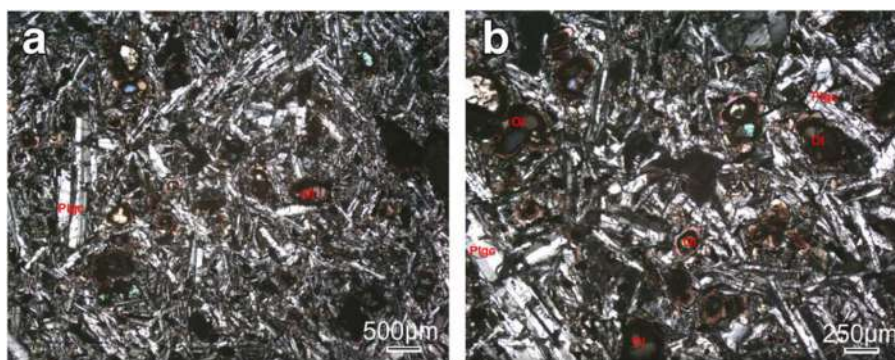


Figure 127. Microphotographs under crossed polars of basalts. Porphyritic basalt with phenocrysts of twin plagioclase (grey white), olivine (pink, blue) showing alteration rim of brown iddingsite (black) set in a finely groundmass with intersertal structure of plagioclase and olivine. In the left is the presence of small-voids (analysis conducted by M.S. Grillo).



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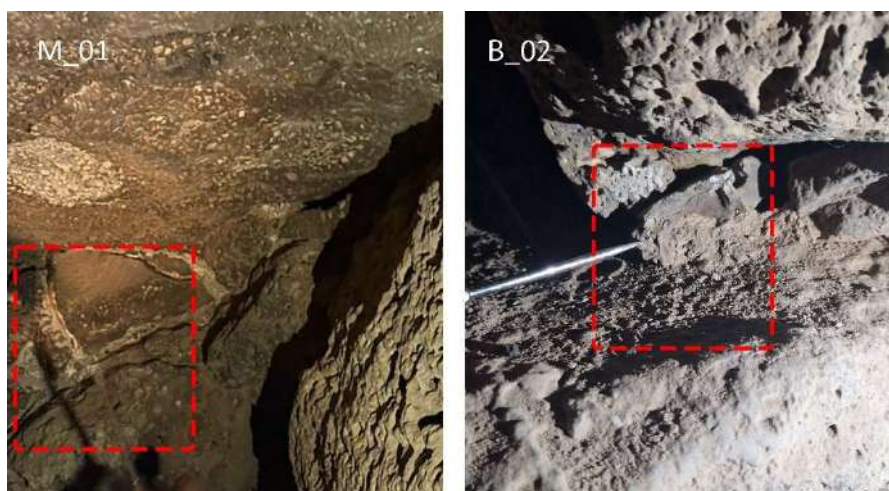


Figure 128. Mortar sampling points: a) sample M_01; b) sample B_02.

As concerns mortars, these were chosen because of the different macroscopic nature. In fact, as known from the study of the building techniques, while bedding mortars were totally absent between basalt rocks, this was anyway present in the middle of the masonry and in the addition of small pebbles of basalt to fix the biggest blocks. Thus, two samples were chosen: the first one M_01, collected inside a recess of the main room, seemed to be a recent addition to the original masonry as it is added as consolidation intervention never acknowledged; the second one instead, B_02, is a sample of hypothetical original hearten mortar, collected deep into the masonry (Figure 128). In order to understand the mineralogical composition of the two samples, an X-ray diffraction was conducted.

As it can be seen in Figure 129, M_01 shows a dominant intensity peak at about $29.5^\circ 2\theta$, associated with the calcitic phase, compatible with the presence of calcium carbonate (CaCO_3). Also visible are numerous low- and medium-intensity peaks attributable to the anorthitic phase, i.e. $\text{CaAl}_2\text{Si}_2\text{O}_8$, a plagioclase calcium-aluminosilicate feldspar.

In summary, calcite is the predominant mineralogical phase, confirming the use of a limestone binder, probably carbonated as a result of the setting and hardening process of a lime-based mortar. The anorthite phase ($\text{CaAl}_2\text{Si}_2\text{O}_8$) is fully compatible with an aggregate of basaltic origin, the same material used in the construction of the artefact. The XRD analysis of sample M_01 shows a mineralogical

composition compatible with that of a historical lime-based mortar, which was certainly laid during the reuse phases of the nuraghe.

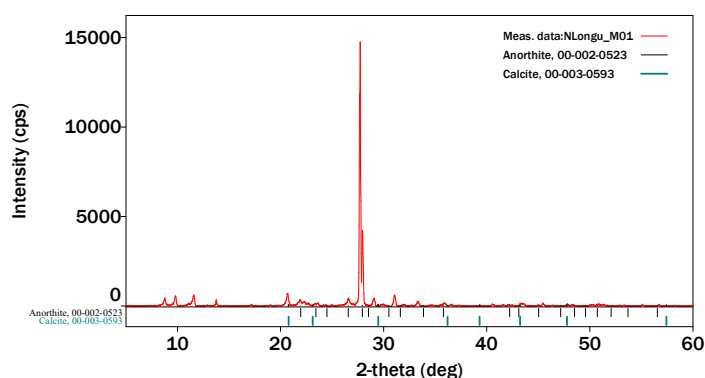


Figure 129. X-ray diffraction of Sample M_01.

As concerns for B_02, the diffractogram (Figure 130) shows peaks of low and medium intensity, indicating low overall crystallinity, consistent with the presence of a clayey earthy matrix. The phases identified are: Illite, a clay mineral typical of loamy and sandy soils, with discrete cohesion; kaolin/montmorillonite: indicative of expansive or plastic clays, compatible with raw earth mortars or stabilised soils; disordered sodic anorthite: sodium plagioclase feldspar, probably of detrital origin or from altered feldspathic sediments. Therefore, this is a natural earth rich in clay fraction, lacking in limestone binder. The combined presence of illite and montmorillonite suggests a plastic and binding behaviour that is useful in the manufacture of raw mortars. The identification of the feldspathic phase (sodic anorthite) indicates the presence of lithic granules or feldspathic sands in the sediment used.



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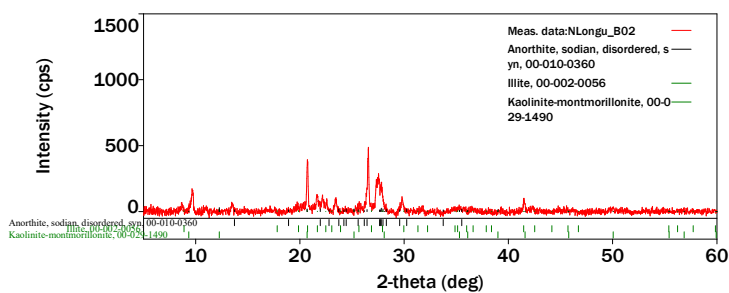


Figure 130. X-ray diffraction of Sample B_02.

From the comparison of the two samples (Figure 131), it is clear that these are two mortar mixtures not only of a different nature but also dating from two different historical periods.

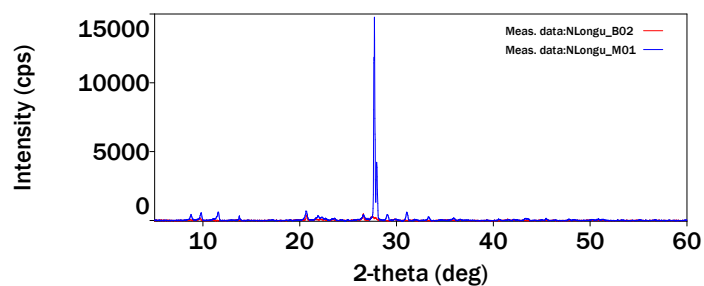


Figure 131. Compared diffraction pattern X-ray of the two basalts.

5.2. Monitoring strategy

5.2.1 Theoretical background

The overall vulnerability of the mono-tower nuraghe is described through a geometric vulnerability index and a block-material vulnerability index.

The geometric vulnerability is examined in the next paragraph which highlights that the stability of the mono-tower essentially depends on the block interconnection level and on the stress between the blocks.

The vulnerability of the blocks is described by parameters that characterize their strength, integrity (presence and severity of cracking), and the state of deterioration of the constituent material.

- *Geometric vulnerability*

The key aspects necessary for assessing the geometric vulnerability of the *mono-tower* nuraghe have been identified through the analysis of the static behaviour of its primary structural components: the **tholos** and the **external wall**.

The objective of the analysis was to identify the principles underlying the stability of the Nuragic Tower, investigate the structure's collapse mechanisms under the action of its weight and define the level of static safety of the structure, by describing the contribution of each structural element.

The stability analysis was carried out separately for the two principal elements (tholos and external wall) in order to identify their respective collapse mechanisms and to understand how these are influenced by the original construction techniques.

The analysis was carried out by numerical simulation on a three-dimensional model generated in the 3DEC environment.

In the following, before presenting the analyses carried out and the results obtained, the conceptual reference model on which the numerical model was based is described.

- *Conceptual reference model*

As anticipated above, the Nuragic Tower (Mono-tower nuraghe) represents the elementary unit of more complex Nuragic developments; its conceptual model is composed of 3 structural elements: two concentric dry-built isodomic walls with circular horizontal sections, which respectively define the internal chamber and the external tower, and the cavity between them. The conceptual model does not include the entrance opening, the internal niches and the stairs that often characterize Nuragic constructions.

The model geometry is derived from the analysis of existing structures presented in the previous section (paragraph 5.1.1 **Errore. L'origine riferimento non è stata trovata.**). The internal tholos is described by a rotation paraboloid (Equation 1), whose vertical median section and main dimensions are represented in



Figure 102; the blocks that compose the tholos have a trapezoidal shape and are arranged to form overlapping and staggered horizontal rings; their dimensions decrease from bottom to top, in accordance to the block dimensional curves in Figure 104.

The external tower is concentric to the tholos, with a radius 5 m bigger than the tholos internal radius; its height exceeds that of the tholos by about 1 m, and the upper volume is filled with material similar to the filling material between the two walls. The blocks of the external tower are also arranged to form an isodomic wall, with decreasing dimensions from bottom to top, and larger than the blocks of the tholos at the same height (block dimensional curves in Figure 103).

The cavity between the two concentric walls, approximately 3 m thick at the base, is filled with granular loose material.

The external forces acting on the structure are represented exclusively by the weight of its components. Each horizontal ring of blocks is subject to the weight of the rings above, transmitted through the horizontal contact surfaces: the vertical load acting on the blocks therefore increases from the highest to the lowest ring.

Similarly, in the cavity filled with loose material, the gravitational forces generate a vertical tension (σ_v) which is proportional to the specific weight of the material (γ) and linearly increases from top to bottom: $\sigma_v = \gamma \cdot z$. Due to its nature (loose) and because of the presence of the two confining walls, a horizontal tension is created that is proportional to the vertical tension: $\sigma_h = k \cdot \sigma_v$, with k depending both on the internal friction angle of the material and the way it has been arranged during construction.

○ Numerical model

The numerical model translates the conceptual reference model discussed in the paragraph above.

The following main assumptions were made:

- the profile of the tholos is described by a defined mathematical function;
- both the tholos and the external tower are isodomic structures;



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- the blocks constituting the tholos and the external tower are infinitely resistant and rigid (density 2800 kg/m^3), they have a regular shape and their dimensions decrease with the height;
- the mechanical behaviour of the structure only depends on the stiffness and shear resistance of the joints between the blocks;
- the filling material between the tholos and the external tower applies a system of forces to the confining walls;
- the forces exerted by the filling material upon the blocks of the two confining walls F_r are fixed and invariant with the deformation, and linearly increase from the top to the bottom of the structure.

The blocks constituting the walls are in contact through interfaces (joints) characterized by zero tensile strength and shear strength expressed by Coulomb's formulation ($\tau < \sigma_n \cdot \tan\phi$). The following constitutive laws describe their mechanical behaviour:

- $\sigma_n > 0$ i.e. only normal compressive stresses are transferred through the joint;
- $\sigma_n = k_n \cdot \delta_n$, per $\sigma_n < \sigma_c$, where σ_c represents the uniaxial compressive strength of the rock at the fracture surface, k_n is the joint normal stiffness coefficient and δ_n is the joint normal displacement;
- $\tau = k_s \cdot \delta_s$, per $\tau < \sigma_n \cdot \tan\phi$, where τ is the shear stress transferred through the joint, k_s and δ_s the joint shear stiffness coefficient and the joint shear displacement respectively, ϕ is the joint friction angle.

The main geometric features of the tholos model are:

- Internal diameter = 5 m;
- Internal height = 7.4 m;
- Block dimensional curves as a function of the tower height (Figure 104).

The main geometric features of the External tower numerical model are:

- External diameter = 15 m;
- External height = 8.1 m;
- Inclination of the external wall face = 10° ;

- Block dimensional curves as a function of the tower height (Figure 103).

○ Stability Analysis of the Tholos

The collapse of the tholos has been analysed in two different conditions: $K = 0$, i.e. null horizontal pressure generated by the filling material and $K=0.33$, i.e. horizontal stress linearly increasing from top to bottom ($\sigma_h = K \cdot \gamma \cdot z$) assuming a backfill material density of 1500 kg/m^3 .

The limit condition is reached by reducing the shear resistance of the joints, starting from the equilibrium of the structure ($\phi = 35^\circ$). For the case of null horizontal pressure ($K = 0$) the collapse condition of the tholos is reached when $\phi = \phi_c = 6^\circ$ which corresponds to a safety factor $S\phi = 6.7$.

The collapse develops with the radial expulsion of the blocks of one of the median rings (the 5th ring in the analysed configuration), followed by the loss of contact between the upper blocks and, finally, the disconnection of the entire structure (Figure 132).

The analysis was then reiterated for the construction phase, simulated as a ring overlapping process from the base (ring A0) to the top (ring A21). The simulations were carried out for the case $\phi = \phi_c = 6^\circ$ (i.e. achievement of the limit condition), seeking the condition of static equilibrium which follows the overlapping of a new ring. Operating in this way, the collapse occurs when the construction reaches the A15 level, with the expulsion of the blocks of ring A4 (Figure 133).



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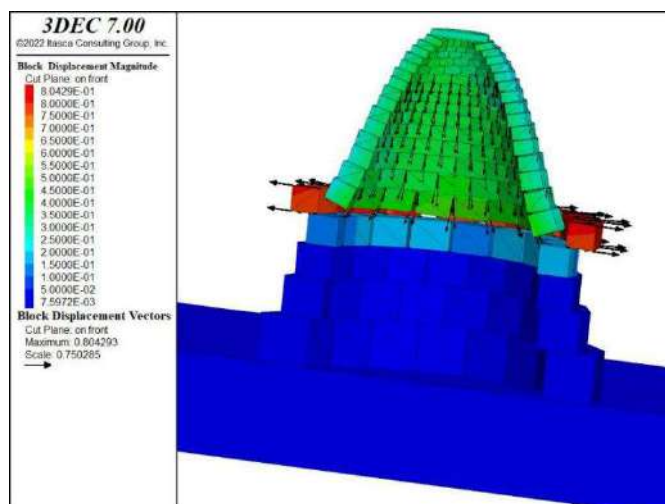


Figure 132. Collapse Mechanism of the Tholos.

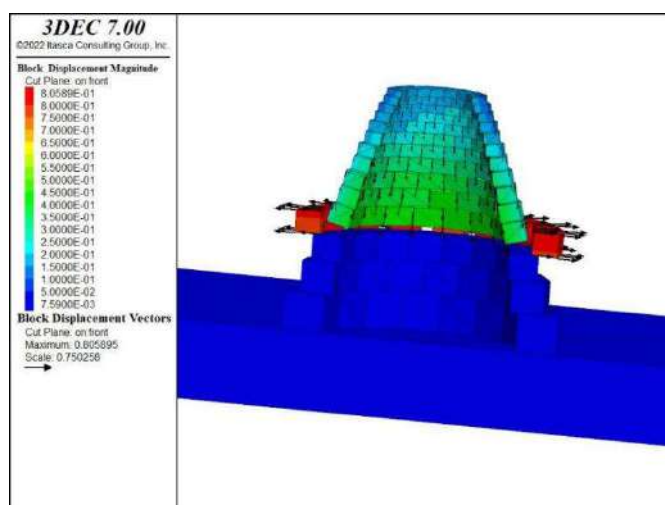


Figure 133. Development of the Tholos Collapse During Construction.

Figure 132 and Figure 133 show that the collapse of the tholos is triggered by the inward rotation of the blocks of the upper rings, followed by the expulsion of the blocks of one of the intermediate rings. The schemes in Figure 134 summarize the described kinematics.



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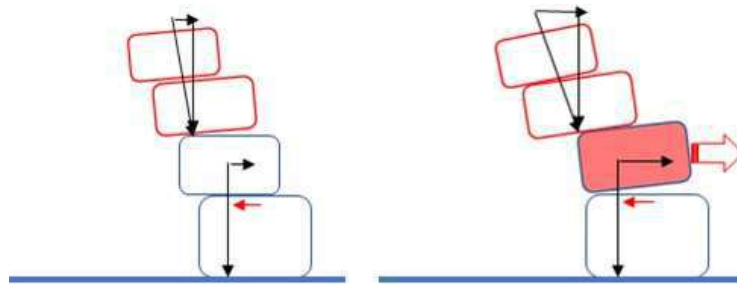


Figure 134. Kinematics of the Tholos Collapse Mechanism: The Increasing Rotation of the Upper Blocks Induces the Radial Expulsion of the Lower Blocks.

When considering the horizontal pressure applied by the filling material with $K = 0.33$, the collapse is never reached, even when $\phi = 0$; the stability of the tholos no longer depends on the joint friction angle and the structure remains inherently stable.

Kinematics

The collapse of the tholos develops according to two linked kinematics: it is triggered by the rotation of one of the rings towards the inside of the chamber, followed by the radial translation of the blocks in the lower rings. The rotation of the blocks is favoured by the high cantilever rate of the rings (defined by the s/l ratio) and it is contrasted by the weight force P of the filling material acting on the external surface of the blocks (block A in Figure 135) and by the continuity of the lateral contact between the blocks of the same ring; the subsequent expulsion (translation) of the blocks in the lower rings is opposed by the horizontal action exerted by the filling material (F_r) and the shear resistance of the horizontal joints (T) (block B in Figure 135).

Hence, the filling material in the cavity between the walls plays a role in the static equilibrium of the tholos, both through vertical and horizontal actions (P and F_r in Figure 135). Furthermore, it counteracts the rotation of the blocks by increasing their degree of connection, according to the mechanism illustrated in Figure 136.



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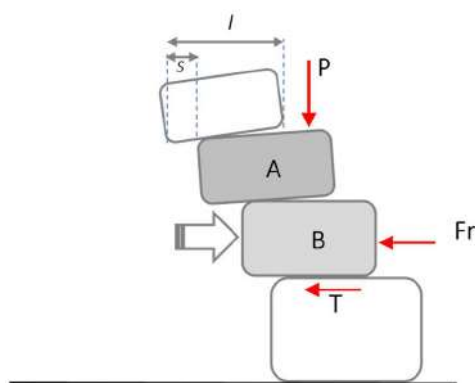


Figure 135. Schematic Representation of the System of Forces Acting on the Blocks of the Tholos.

For each ring of blocks, it is possible to identify the following geometric elements (represented in Figure 136) the lower and upper base of the ring Σ_i and Σ_s respectively); the internal circumference C_i of the lower base Σ_i ; the internal circumference C_s of the upper base Σ_s .



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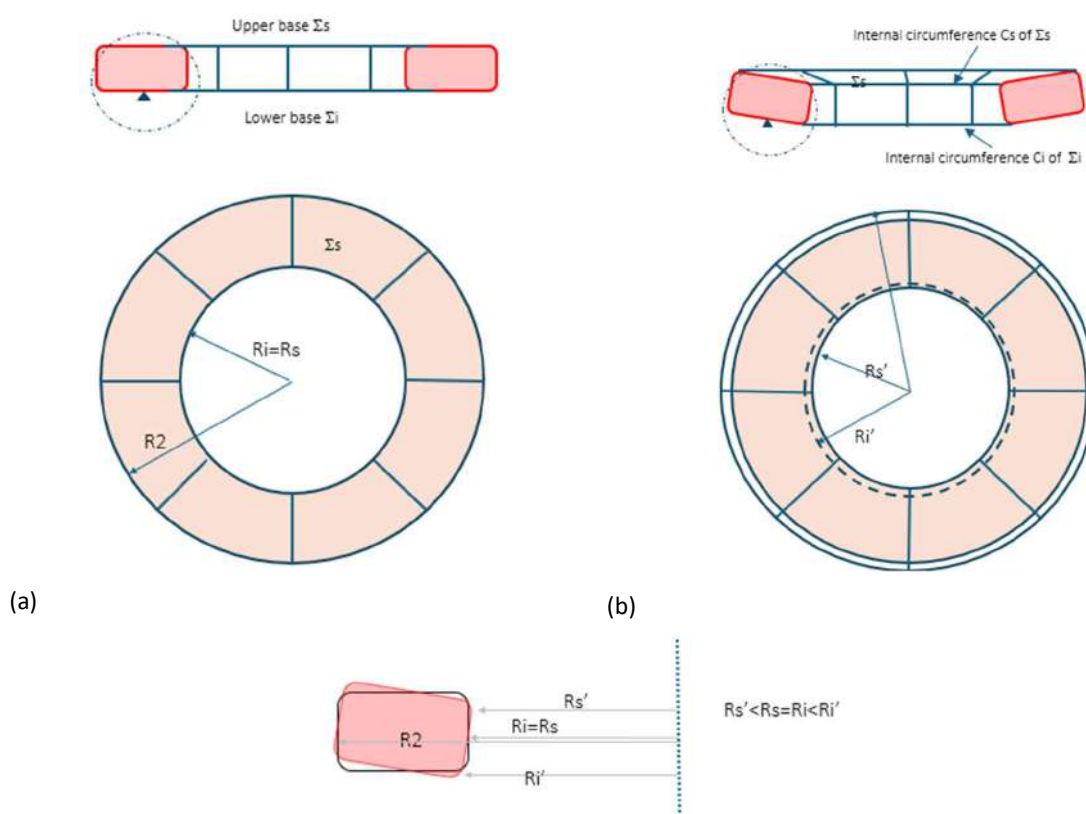


Figure 136. Ring's Geometric Elements and Dimensional Variation with the Rotation of Blocks: a) Undeformed; b) Deformed.

In the undeformed configuration (a), the radius of the circumference C_i (R_i) and that of the circumference C_s (R_s) are equal. The virtual rotation (b) of the blocks of the ring determines the reduction of the radius of C_s , from R_s to R'_s , and the increase of the radius of C_i , from R_i to R'_i . The reduction of the radius R_s and, consequently, that of the circumference C_s , would require the interpenetration of the upper edges of the blocks or their breakage. If this condition is excluded (hypothesis of non-interpenetration and integrity of the material), the rotation can only occur with a radial displacement Δ_r towards the outside of the blocks, as to keep the radius R_s unchanged.

Figure 137 describes the above discussed kinematics: R_i and R_s represents the lower and upper circumference radius in the undeformed configuration, R'_i and R'_s are the radius in the final deformed configuration, after the rotation and the radial translation of the blocks. The horizontal action F_r (Figure



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138) exerted by the loose material in the cavity counteracts the radial displacement of the blocks and thus their rotation towards the inside (trigger phase), inhibiting the development of the collapse mechanism.

In other words, the radial horizontal force F_r exerts a *circling* action on the ring of blocks by increasing the tangential contact force between them (F_t in Figure 138) and bringing the discontinuous circular structure made of separate blocks closer to that of a continuous ring (without joints).

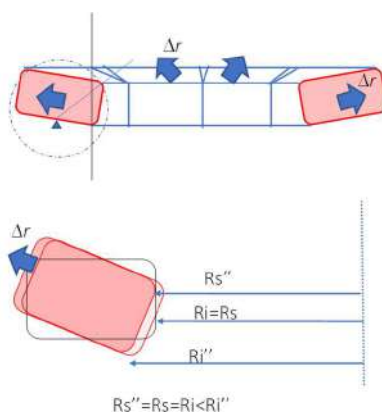


Figure 137. Relationship between the Rotation of the Blocks and their Radial Displacement Δr .

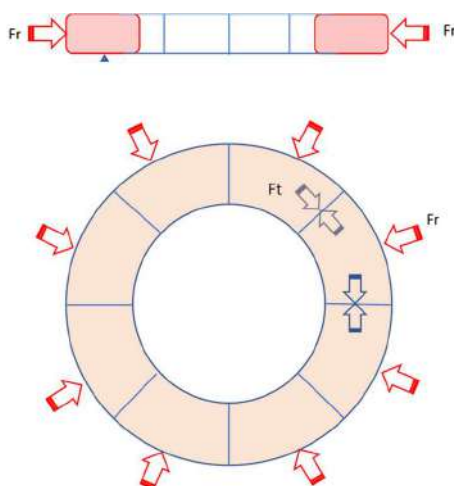


Figure 138. Effect of the Force Exerted by the Filling Material in the Cavity on the Ring Structure.



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○ Stability Analysis of the External Tower

The analysis of the collapse mechanism for the external tower is developed considering the horizontal pressure applied by the filling material, $\sigma_h = K \cdot \gamma \cdot z$ with $K = 0.33$. The limit condition ($\phi = \phi_{lim} = 25^\circ$) is reached by reducing the shear resistance of the joints, starting from the equilibrium of the structure. The safety factor is 1.5. The collapse develops with the radial translation of the blocks followed by the disconnection of the entire structure (Figure 139).

The construction process was simulated by overlapping the horizontal rings, from A0 to A14: the collapse occurred when the construction reached level A13.

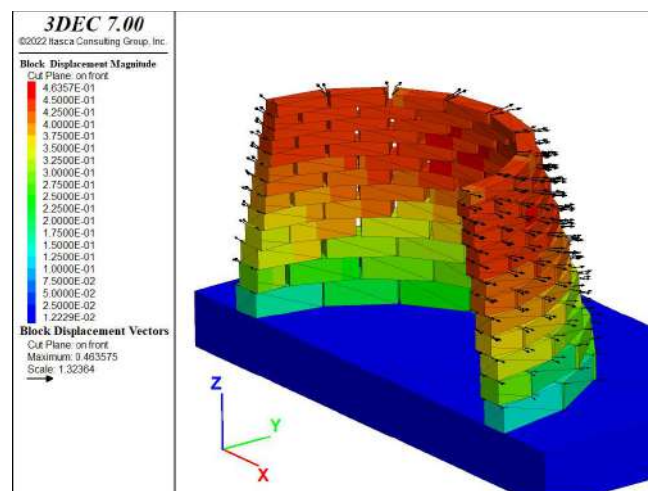


Figure 139. Collapse Mechanism of the External Tower.

Kinematics

The collapse of the external tower is triggered by the translation of the blocks under the thrust of the force F_r , which therefore assumes a destabilizing role for the external wall. This is opposed by the shear resistance of the horizontal joints ($T = G \cdot \tan \phi$ in Figure 140), the entity of which depends on the characteristics of the contact surfaces between the blocks (synthesized by the friction angle ϕ) and on the normal force G which, given the geometry of the structure, is represented by the weight of the blocks.



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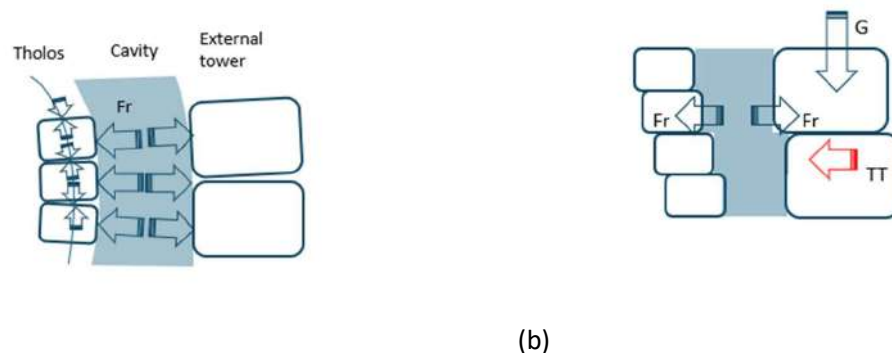
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Therefore, while the filling material contributes significantly to the stability of the tholos, it stresses the external tower, which in fact constitutes its containment structure. For this reason, the external tower is normally made of blocks of metric dimensions (cyclopean boulders), capable of opposing with their weight the horizontal actions exerted by the filling material, whereas the blocks of the tholos have decimetric dimensions. Figure 140 illustrates the effect of the force F_r on the two masonry structures (the tholos and the external tower) and the reactions that contribute to maintain their equilibrium.



(a)

(b)

Figure 140. Diagram of the Actions Exerted by the Filling Material on the Wall Structures; a) Horizontal View; b) Vertical Cross Section.

o Formation of the Horizontal Force F_r

The force F_r acting on the tholos and the external Tower arises from the horizontal component of the stress (σ_h) acting in the volume of the loose material in the cavity. Its value is expressed as a function of the vertical stress component σ_v through a factor K ($\sigma_h = K \cdot \sigma_v$).

The value of K , in turn, depends on the geotechnical characteristics of the material and the methods with which the layers of filling material are constituted. If the material is simply poured into the cavity, K assumes a value lower than the unity and around 0.3–0.5; if the material is compacted, K can reach values greater than 1, depending on the level of compaction. Correspondingly, given the value of the vertical stress $\sigma_v = \gamma \cdot z$, the horizontal component σ_h , which acts on the blocks' surface, can assume different values depending on the degree of compaction. In this regard, it is interesting to observe that during construction, the level of stability of the tholos can be increased by compacting the filling material in the cavity; correspondingly, to cope with the increased centrifugal forces, the dimensions of the blocks of the



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external tower need to be increased. Figure 141 illustrates the relationship between the theorized construction method and the actions acting on the structure.

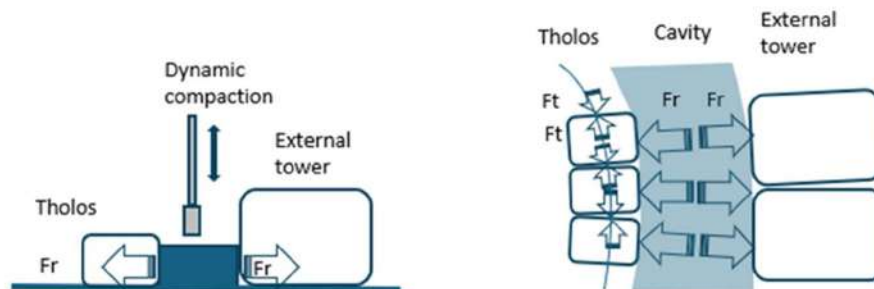


Figure 141. Diagram of the Actions Exerted by the Filling Material on the Wall Structures; a) Horizontal View; b) Vertical Cross Section.

Since the filling material plays a decisive role in the stability of the structure during the construction process, it could only have been arranged at the same time as the construction of the tholos and the external tower (i.e. from bottom to top, in succeeding horizontal layers). As a matter of fact, it is unrealistic to imagine a construction that first involves the construction of the entire tholos, then the covering with the loose material and finally the construction of the external tower; similarly, a construction method that involves the construction of the tholos followed by that of the external tower and then the filling of the cavity with loose material seems highly improbable.

The only plausible hypothesis remains the construction by superposition of horizontal rings, formed by an internal circle of blocks (tholos), an external circle of blocks (tower) and an intermediate circle of loose material (inside the cavity). The latter was most likely placed between the two rings of blocks and compacted in such a way as to confer horizontal thrusts and stability to the blocks of the tholos; towards the outside, the thrusts were contrasted by the cyclopean blocks of the external tower.

○ Conclusion and geometric vulnerability monitoring

The results proved that the filling material plays a fundamental role in the static equilibrium of the Tholos, under the hypothesis of compaction and progressive construction of the Nuragic Tower by succeeding horizontal rings, which create the discussed circling mechanism. On the other hand, the filling material



assumes a destabilizing effect on the external tower, which is made of cyclopean boulders, capable of opposing with their weight the horizontal actions exerted by the compacted filling material.

Geometric vulnerability is therefore characterized through the value of the tangential stress exchanged between the blocks of the horizontal rings composing the structure.

- *Materials monitoring*

Monitoring the conservation status is a particularly important moment to pass its tangible and intangible values to future generations. The proposed key factors were identified with the aim of constructing a rigorous, objective, effective and updatable tool capable of recording the health conditions of monuments, assessing the trend of degradation processes, but also verifying the timeliness and effectiveness of the maintenance and restoration actions that will be implemented over time. As a matter of fact, monitoring seeks to identify threats or impacts that may undermine the universal values of the property, with particular attention to the aspects of authenticity and integrity of the materials.

The starting point for monitoring the conservation status of each asset is the condition, that serves as “ground zero” for monitoring. Although there are several cases when monitoring certain factors, especially environmental and structural ones, has already been prompted it is believed that only through the simultaneous application of a common protocol to the entire set of assets and the systemization of the findings of the surveys carried out on individual sites, can we reach a full awareness of the processes of alteration and degradation and, as a result, feed a virtuous process of constant care of the sites and the planning and implementation of truly effective conservation and promotional measures. This applies both to the understanding of risk factors and the conditions that can accelerate or slow down degenerative processes, and to have operation implemented in the past on individual monuments stand the “test of time”, in order to assess the appropriateness - depending on the outcomes - of re-using or rejecting the use of methods and solutions already tested in similar type and environmental conditions.



It is useful to remind that the monuments are the material testimonies to civilizations that would otherwise be unknown. Therefore, monitoring the conservation of authentic materials should be a priority in management strategies.

With this in mind, monitoring will provide valuable information for the asset manager, who will be able to verify through this process the existence of potential or actual problems, the effectiveness of the protection, conservation and management actions implemented with respect to the actual achievement of the set objectives, or the possible need for corrective actions. The indicators are dedicated to monitoring the most important universal value, such as the monument integrity, through the control of the most vulnerable elements: slope stability and safety; structural damage, lesions, cracks and collapses; mechanical aggression by tree species, the presence of weeds, the covering of surfaces by compact biological patinas, especially in the case of hypogea; erosions and detachments triggered by environmental, geological and/or intrinsic factors to the lithological nature of the artefacts.

It is believed that the monitoring plan, thus set up, can provide a structured corpus of consistent and verifiable information, which is also indispensable for periodic reports, to be produced every six years. Due to the nature of the monuments, this timeframe - possibly divided into two or three data collection sub-periods - is a time reference that is also consistent with the time required to check the methods and speed of evolution of the processes of alteration and degradation affecting the types of artefacts included in the Site, but also not too long in order to ensure good timeliness in the introduction of corrective measures to management strategies and the implementation of restoration work, where required. The monitoring intervals of the indicators were calibrated according to the robustness or fragility of the attributes and their susceptibility to change. In order to make the interpretation of the magnitude of the phenomena and their evolution as objective as possible, numerical, or multiple-choice vocabularies, were chosen whenever possible. For many indicators, moreover, some recommendations have been made to make the measurement repeatable and the data comparable. We refer to, including but not limited to, the attention to be paid when using the same frame in photographic campaigns, or to carry out environmental measurements always at the same point and possibly using the same tools.

5.2.2 Tools, setup, and data acquisition

- *Geometric vulnerability*

- Introduction

This protocol outlines a non-invasive methodology aimed at assessing the structural stability of mono-Tower nuraghes through the application of geophysical techniques. Ultrasonic testing is a widely used non-destructive technique that exploits the sensitivity of longitudinal wave (P-wave) propagation to microcracks, schistosity, bedding structures, and mineral decomposition in rocks. It is used to assess material properties, evaluate the degree of weathering, and detect internal defects or crack depth in stone monuments. By measuring the P-wave velocity in the laboratory, you can more confidently interpret P-wave velocity data in situ conditions. In the context of historic stone monuments, such as the Nuraghi of Sardinia, ultrasonic testing allows for the assessment of the elasto-mechanical condition of rock materials without causing any physical damage.

Since the relationships between the static behaviour of these structures and relevant geophysical parameters have not yet been investigated, an experimental research plan has been developed. The specific objective of this plan is to establish empirical correlations between the propagation velocity of seismic waves within the structures and the horizontal radial forces exerted on the outer surfaces of the Tholos by the granular material that fills the space between the Tholos and the external masonry.

These correlations will be derived from experimental testing conducted at the DICAAR (Department of Civil and Environmental Engineering and Architecture, University of Cagliari) laboratories, using various configurations of stone blocks. The study will focus on examining the relationship between the horizontal radial forces applied by the filling material and the seismic wave velocities within the Tholos masonry, with particular attention to two main parameters:

- the lithotype of the stone blocks,
- the characteristics of the discontinuity surfaces between adjoining blocks.



The identification of these correlations at the laboratory scale will, in the future, enable the transfer of results to real-world applications. Ultimately, this will allow for in-situ quantification of the stabilizing forces exerted by the filling material on the outer surfaces of the Tholos using geophysical measurements.

o Experimental program

To implement the experimental work plan, prismatic stone blocks with trapezoidal bases have been procured. The blocks have the following dimensions:

- Height: 20 cm
- Length: 22.5 cm
- Larger base: 21 cm
- Smaller base: 14 cm

The study involves four distinct lithotypes:

- White Marble from Orosei (a specific type of limestone)
- Trachyte “Auctorum”
- Sardinian Grey Granite
- Basalt

Geophysical characterization has been carried out on individual blocks (varying by lithotype) and is currently being extended to different configurations made of consecutive blocks. The final phase of the laboratory work will focus on configurations assembled into a semicircular arrangement with fixed ends, representing a hypothetical ring of a mono-tower nuraghe under symmetry assumptions. This semicircular prototype, illustrated in Figure 142, will have a radius of 75 cm.

The experimental program makes use of the following instruments and equipment:

- Pundit Lab (+) ultrasonic testing device
- PUNDIT PD8050 ultrasonic tomography system
- A load testing kit comprising an electric hydraulic pump and 11 hydraulic jacks
- Load cells for force measurement



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- A set of strain gauges for deformation monitoring
- A real-time data acquisition system
- EWJ-200 3015 water jet cutting machine

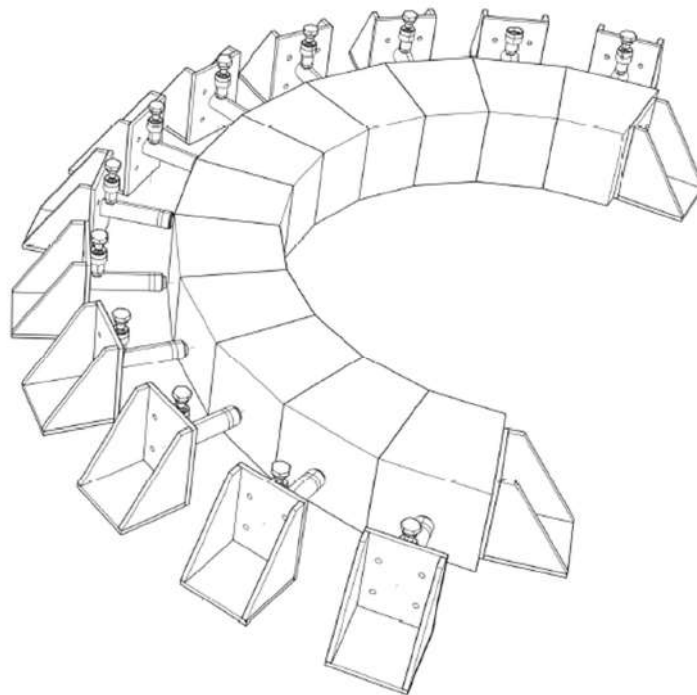


Figure 142. Experimental prototype.

The experimental program is structured in the following phases:

Phase 1: Characterization of Individual Blocks

Ultrasonic tomography was performed on intact samples of each lithotype (marble, trachyte, granite, and basalt), along with baseline measurements of P-wave velocities (V_p) under unstressed conditions.

The experimental setup for ultrasonic measurements utilized a pair of piezoelectric transducers (transmitter and receiver) with central frequency of 82 kHz placed on opposite faces of the sample

(transmission mode) frequency was selected as a practical compromise between resolving power and acceptable signal attenuation, based on results from previous laboratory experiments [147] [148] [149]. To ensure measurement accuracy, the prismatic sample faces were carefully prepared to be flat, parallel, and almost free of morphological irregularities, in accordance with ASTM (American Society for Testing and Materials) guidelines. In fact, sample surface preparation plays a critical role in ensuring reproducibility and accuracy of ultrasonic measurements. P-wave velocity in rock samples was determined under ambient condition using a Portable Ultrasonic Non-destructive Digital Indicating Tester (PUNDIT Lab plus - Proceq, Switzerland) following the guidelines established by the International Society for Rock Mechanics [150] for ultrasonic pulse velocity testing. The tester is capable of detecting propagation times within the range 0.1-9999 μs , meaning it can handle both very short (0.1 μs) and relatively long (9999 μs) pulse travel times. The device can measure with an accuracy of 0.1 microseconds, making it highly precise for testing.

Phase 2: Analysis of the Effect of Discontinuity Quantity

Tests have been conducted on horizontal stacks of 2 to 4 blocks for each lithotype, using "undisturbed" contact conditions, meaning full contact between adjacent faces. Progressive loads ranging from approximately 500 to 2000 kg have been applied, while variations in seismic wave velocities have been monitored. The goal is to assess the effect of multiple discontinuities on wave propagation.

P-wave velocity in the block stacks was determined under ambient condition using a PUNDIT Lab plus.

Phase 3: Analysis of Discontinuity Surface Characteristics

Artificial surface roughness will be introduced on the contact faces using high-pressure water jet cutting to simulate different types of discontinuity conditions. Tests will again be carried out on 2 to 4 block stacks for each lithotype and each roughness configuration. The same progressive loading will be applied (500–



2000 kg), with wave propagation monitored to evaluate how reduced contact surfaces affect seismic response.

Phase 4: Semicircular Prototype Testing

A semicircular prototype will be assembled and equipped with load cells and strain gauges. Radial compression will be applied through an electrically operated hydraulic pump.

This phase will evaluate the seismic response of the semicircular configuration under varying load conditions, different lithotypes, and various discontinuity geometries. The system will continuously record load and deformation data for detailed analysis.

Phase 5: Data Analysis and Development of Reliable Correlations between Stress and Seismic Velocity

Empirical relationships will be developed based on:

- The specific responses of different lithotypes
- The influence of interface roughness on wave attenuation
- Anomalies in ultrasonic pulse velocity (UPV) associated with varying stress levels
- Statistical treatment of missing or inconsistent data
- Confidence intervals to support real-world application of the results

Phase 6: Field Application Protocol

The field implementation phase will involve UPV (Ultrasonic Pulse Velocity) surveys on accessible sections of the Tholos. These data will be compared with laboratory results to qualitatively determine the hoop stress exerted by the granular infill material.

- *Material vulnerability*

Based on detailed surveys, this phase of monitoring seeks to analyse the building according to protocols deriving from the archaeological sciences, called archaeometry. These investigations represent the cooperation of different scientific methods that involve physical and biological sciences with archaeology and art history. Their helpful combination and the use of dating methods, detailed studies of the artefacts, remote sensing techniques, and studies of state of conservation offer an accurate investigation of materials, building techniques, technologies, and chronologies that are fundamental for the identification of tangible values such as those defined by functional and documental aspects, as well as intangible values with associated features linked to historical practices and relationships.

In the present investigation, the archaeological approach applied to mono-tower nuraghi focused on characterizing the fundamental properties of the stone blocks, including compressive strength, mineralogical composition, and surface roughness. This characterization was complemented by an assessment of the mechanical alteration state—evaluated through the density, length, and depth of fractures—and by an analysis of the mineralogical – chemical alteration of the materials.

In particular, material aspects will be investigated through a detailed diagnostic protocol due to the poor consideration that materials often have in the analysis of historical contexts. The issues relating to the poor study of materials, especially mortars used during phases of restoration and reuse in different periods. They are potential elements of knowledge of their state of conservation but are frequently neglected due to their lack of use and consequent insufficient consideration. Thus, the protocol also considers nuraghi as a powerful historical links between humans and their territory through materials, territorial signs of a place [151]. In fact, materials can also clearly represent topography and the interaction between nature and builders' skills and abilities [152].

In this light, the present step of monitoring illustrates the importance of the characterization, the identification of historical materials, both stones and mortars, and the analysis of their state of conservation. For various reasons, mortars have been considered an important material to be analysed for their structural role and their use during recent phases of reuse. First, they are precious records that permit us to understand the building's materials and the evolution of traditional techniques,



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understanding the composition of historical products in relation to different workforces. Second, they are valuable bases to identify new repair layers designed with chemical, physical and visually compatibility with the existing structure [153]. Moreover, the analysis of old mortars is fundamental in determining the origin of decay or failure of old mortars, evaluating the stability of the structures and proposing appropriate and compatible new mixtures (if needed), whether for conservation or for re-use. Finally, they can be used as dating elements to support the study of chronologies especially when historical archives do not provide documentary or photographic evidence.

In these cases, analysis reveals great information about the samplings chosen; factors such as site conditions, position, time, and method of application can change the results. For this reason, after a careful sampling of natural and artificial stones, the protocol for material investigation sought to define the mineralogical-petrographic nature of stones and mortars, identifying their binder and aggregate.

Overall, the study of alterations or decay defines the material state of conservation. Macro assessment allows us a first general identification of the composition and the structure of mortars whereas archaeometry analysis such as microscopic analyses on thin sections, diffractometric and fluorescence X-ray, microscopy, and micro analyses, permit a complete understanding and characterization of building materials, defining not only the mineral-petrographic characteristics and state of conservation but also the monument's chrono-typologies [151].

In addition to the autopsy investigation, the mineralogical and petrographic characterization of samples of basalt stone and mortar in the masonry was carried out on a reasoned sampling in order to define their composition, degradation and the techniques used by the local craftsmen.

○ Tools

The monitoring will be carried out using both laboratory-based instruments and portable devices suitable for in-situ applications. Non-destructive tests will be performed directly on in-place materials, while laboratory analyses will be conducted on collapsed material with characteristics similar to those of the stone blocks still present in the structures.

In situ, representative stone blocks will be examined to measure fracture density, length, and depth using a structured-light 3D optical handheld scanner (Go!SCAN, Creaform) for detailed surface profiling. Surface roughness—an essential parameter, particularly for the contact surfaces between adjacent blocks—will be determined using traditional tools, such as a surface roughness comb, and complemented by high-resolution 3D surveys performed with the same Go!SCAN scanner.

The compressive strength of the rock blocks will be assessed using a Schmidt hammer, a non-destructive device that measures surface hardness by recording the rebound of a spring-driven mass impacting the stone surface, providing an indirect estimate of compressive strength. In particular, the GEOSTONE mechanical rock Schmidt hammer will be used.

The compositional characteristics and the presence of mineralogical alterations will be monitored in situ using the portable X-ray fluorescence (XRF) spectrometer SPECTRO xSORT XHH04: an elemental analyzer that's designed for high-throughput analysis of metals and alloys on the spot.



Figure 143. 3D optical handheld scanner (Go!SCAN, Creaform).



During the in-situ surveys, a sampling phase is also carried out for subsequent laboratory analysis using optical microscopy and X-ray diffraction. Optical microscope observation in transmitted and reflected light is an indispensable tool for the minero-petrographic characterisation of stone materials. This technique allows the identification of mineralogical constituents, the evaluation of texture, grain size and paragenetic relationships, providing essential information for the interpretation of the material's origin, alteration processes and state of preservation. In the case of artificial stone, such as mortars and bricks, microscopic analysis also makes it possible to distinguish between the binding matrix and aggregates, identifying their nature and any pozzolanic reactions. This methodology is consolidated in the tradition of petrographic studies applied to cultural heritage [151], which highlights its effectiveness in the diagnosis of degradation processes and the definition of criteria for conservation intervention.

Rigaku diffractometer | X-ray diffractometry analyses, on powder samples, with a Rigaku diffractometer, Ulrima IV, in parallel optics, with Cu-Kalfa radiation, 30kV and 30mA, and 5°-65° pattern (Figure 144). This is one of the most important non-destructive tools to analyze all kinds of amorphous, powder and crystalline materials. From research to production and engineering, XRD is an indispensable method for materials characterization and quality control.

In particular, the combination of microscope and Xray diffraction are used to define the miner petrographic characterisation of natural and artificial stones.



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Figure 144. Rigaku diffractometer.

5.2.3 Data analysis and interpretation

- *Geometric vulnerability*

The ultimate purpose of this monitoring is to identify an indicator of the geometric vulnerability of the structure as a function of the propagation velocity of seismic waves through the masonry of the Tholos. The development of this indicator must first be preceded by the definition of relationships describing the correlation between P-wave velocity and the horizontal radial forces exerted on the outer surfaces of the Tholos by the granular material filling the space between the Tholos and the external masonry.

The work carried out so far for the definition of these relationships is described below. At the current stage of the research, the characterization of individual blocks has been completed (phase 1 in in paragraph 5.2.2), along with an initial assessment of the effect of the number of discontinuities (phase 2 in paragraph 5.2.2 **Errore. L'origine riferimento non è stata trovata.**).

During the tests, a pulse generator emitted high-frequency acoustic signals (82 kHz), and the first arrival time (T) of the longitudinal transmitted signal was recorded using an ultrasonic device interface. The distance between transducers (Δs) was precisely measured to calculate the longitudinal (P-wave) velocity (V_p). The V_p is calculated based on the **distance (Δs)** between the transducers and the **transit time (T)** recorded between the emission and reception of the ultrasonic pulse. Multiple measurements were taken

in different orientations to evaluate anisotropy coefficient (AC). The AC measures the degree of directional dependence of wave velocity. Higher values indicate more anisotropy, possibly due to fabric orientation or microfractures, grains, and pores, as well as stratification and lamination structures. One of the key petrophysical properties analysed in this study is bulk density (ρ), which is estimated from ultrasonic longitudinal velocity values using empirical relationships. Bulk density considers the total volume of the rock specimen, encompassing both the solid mineral framework and the pore spaces. It can generally be stated that P-wave velocity (V_p) and bulk density exhibit a linear relationship across various rock types.

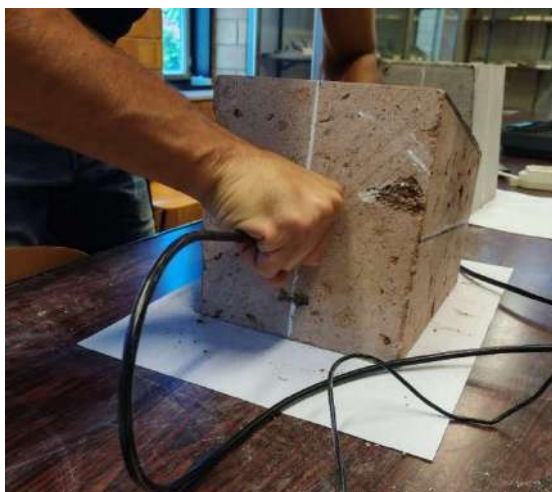


Figure 145. Testing of a Trachite "Aucoturm" sample.

Table 38 summarizes the results of ultrasonic testing conducted on the investigated rock samples. In the investigated rocks, bulk density ranges from 1.85 to 2.69 g/cm³, and P-wave velocity (V_p) ranges from 2682 to 6259 m/s. Variations in longitudinal velocity observed in the investigated rocks are caused by differences in petrographic characteristics, especially texture, as well as variations in their elastic and mechanical properties. In general, low P-wave velocity (V_p), as observed in the trachyte, is associated with reduced rock quality, particularly in terms of mechanical strength and durability. In fact, lower velocities typically correspond to rocks with higher porosity, lower density, fractures or microcracks, poor cementation or weathering. A classification of the ultrasonic velocity for stone materials is reported in



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Table 39. These data also highlight that, for the other materials, high (basalt and granite) or very high (marble) wave velocity values were obtained.

The analysis of ultrasonic waveforms obtained from each measurement on the investigated rock samples provides valuable insights into their intrinsic properties. Well-defined, high-amplitude waveforms are characteristic of competent, hard rocks such as marble, granite, and basalt. In contrast, porous or fractured rocks, such as the trachyte analysed in this study, typically produce low amplitude waveforms and delayed arrivals. As an example, Figure 146 shows typical waveforms recorded for a basalt sample and a trachyte sample.

Table 38. Ultrasonic tests on representative lithologies used to construct the Mono-Tower Nuraghes). Δs (m) distance between transmitter and receiver transducers; T (s) transit time of elastic longitudinal wave (P-wave); V_p (m/s) velocity of elastic longitudinal wave and $V_p(\text{mean})$ mean value; δ_{dyn} = Dynamic bulk density and $\delta_{\text{dyn}}(\text{mean})$ mean value; AC = Anisotropy Coefficient.

Lithologies	Faces	Δs (m)	T (s)	V_p (m/s)	$V_p(\text{mean})$ (m/s)	δ_{dyn} (g/cm ³)	$\delta_{\text{dyn}}(\text{mean})$ (g/cm ³)	AC
Marble	A-A'	0.0000308	0.195	6331	6259	2.70	2.69	0.03
	B-B'	0.0000293	0.180	6143		2.68		
	C-C'	0.0000365	0.230	6301		2.69		
Granite	A-A'	0.0000444	0.200	4505	4491	2.44	2.45	0.02
	B-B'	0.0000397	0.180	4534		2.45		
	C-C'	0.0000496	0.220	4435		2.45		
Trachyte	A-A'	0.0000699	0.197	2818	2682	1.90	1.85	0.12
	B-B'	0.0000662	0.180	2719		1.86		
	C-C'	0.0000905	0.227	2508		1.78		
Basalt	A-A'	0.0000395	0.200	5063	5000	2.60	2.57	0.02
	B-B'	0.0000363	0.180	4959		2.57		



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	C-C'	0.0000462	0.230	4978		2.53		

Table 39. Classification of the ultrasonic velocities.

Class	Velocity Vp(m/s)	Definition
1<	2500	Very low velocity
2	2500-3500	Low velocity
3	3500-4000	Middle velocity
4	4000-5000	High velocity
5>	5000	Very high velocity

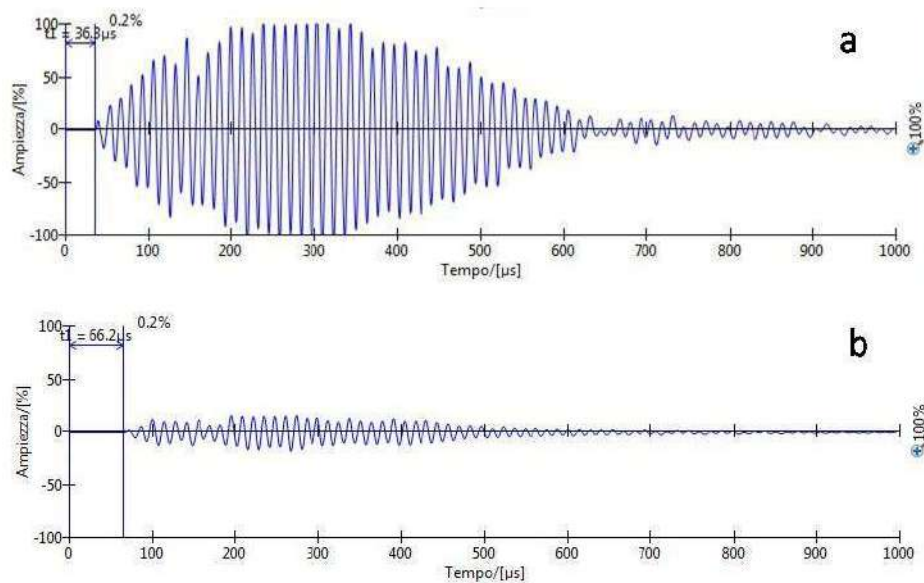


Figure 146. Waveform recorded during the tests on the basalt sample (top) and the trachyte sample (bottom).

Once the characterization of the individual blocks was completed, the analysis of the effect of discontinuities was carried out. Tests were conducted on basalt specimens assembled in a horizontal stack composed of four blocks with flat, parallel faces, subjected to varying loads using a hydraulic pump Figure 147. The results clearly showed a significant decrease in P-wave velocity due to the presence of



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discontinuities between the blocks. As the load applied to the system increased, the P-wave velocity also increased, as a result of the progressive closure of cracks and discontinuities (Table 40).



Figure 147. Ultrasonic tests using a laboratory setup compound by four basalt blocks by applying different pressures.

Table 40. Ultrasonic tests using a laboratory setup compound by four basalt blocks by applying different pressures. V_p (m/s) velocity of elastic longitudinal wave.

Lithology	Pressure (Kg/cm ²)	V_p (m/s)
Basalt	50	1125
	100	1383
	150	1560
	200	1613

- Material vulnerability

The data analysis strategy will focus on the development of composite indices designed to represent the material vulnerability of the investigated materials. These indices will integrate information derived from the mechanical, physical, and compositional characterization of the rock blocks.

The first component of the index will focus on the mechanical performance of the materials, combining parameters related to:



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- The intrinsic mechanical properties of the intact stone (e.g., UCS as measured with the Schmidt hammer).
- The degree of fracturing will be expressed through a fracture index, which is derived by integrating different measurable parameters, including fracture density (number of fractures per unit area), average fracture length, and average fracture depth.
- The roughness of contact surfaces, which directly influences the interlocking capacity between blocks and the overall structural stability.

The second component of the index will reflect the minero-petrographic characteristics and their state of alteration, based on:

- The lithological type of the rock blocks, as defined through optical microscopy and X-ray diffraction analyses.
- The presence and extent of mineralogical alteration, identified in situ with XRF spectrometry and confirmed by laboratory observations.

By integrating these two complementary perspectives—mechanical integrity and compositional condition—the resulting indices will provide a concise yet robust representation of the material vulnerability. The vulnerability of the stone blocks will then be classified on a scale from 1 to 5, where 1 corresponds to intact blocks that are free of fractures, exhibit very low surface roughness, high uniaxial compressive strength (UCS), and no mineralogical-petrographic alterations; and 5 corresponds to highly fractured blocks with very high surface roughness, very low compressive strength, and severe chemical alterations.

These indices are intended to support comparative assessments among different structures and to serve as a basis for defining priorities in monitoring and conservation interventions.

5.3. Ground instability Risk analysis

5.3.1 Ground instability hazard

Ground instability refers to a range of geotechnical hazards associated with the movement or displacement of the Earth's surface, which can significantly threaten the stability and safety of built and natural environments. These movements may occur abruptly, as in the case of landslides or rockfalls, or gradually, such as through subsidence or ground heave. They result from a combination of natural processes—including tectonic activity, soil saturation, and erosion—and human-induced actions such as excavation, mining, groundwater extraction, or poor land-use practices.

5.3.2 Procedure for ground instability vulnerability analysis including differential settlements

The geometric vulnerability of mono-tower nuraghi was assessed in relation to ground instability hazards, specifically focusing on the effects of localized settlements at the foundation level. The analysis aimed to understand how differential settlement impacts the structural integrity and long-term stability of these ancient dry-stone constructions.

To simulate these effects, numerical modelling approaches were employed, utilizing both the Discrete Element Method (DEM) and Finite Element (FE) analysis. The DEM analysis was performed using 3DEC (Three-Dimensional Distinct Element Code), developed by Itasca Consulting Group, which is particularly effective for modelling the discontinuous and jointed nature of dry-stone masonry. In parallel, FE analysis was conducted using ABAQUS (originally developed by Hibbitt, Karlsson & Sorensen, Inc. and now maintained by Dassault Systèmes under its SIMULIA suite) a robust finite element platform that enables the assessment of stress distribution, deformation, and potential failure under various subsidence scenarios.

5.3.3 DEM Analysis

- *Introduction of DEM*

3DEC is a numerical program based on method proposed by Peter Cundall in 1971 for the analysis of rock slopes by means of rigid block or circular particle models. Cundall named his approach the 'distinct element method', but the designation 'discrete element method' is nowadays more common, encompassing not only the codes derived from Cundall 'work, such as UDEC [154] and 3DEC [155], but also many other formulations and codes [156].

In discrete element method (DEM), the particulate material is modelled as an assembly of individual particles, which interact with each other or any other solid body such as the walls or moving blades of mixing equipment. The macroscopic behaviour of the assembly of particles is determined by microscopic interactions amongst particles and between particles and boundaries. The path and velocity of each particle is computed in discrete time steps. By representing materials as a collection of distinct blocks or particles, DEM captures the interactions and movements between these elements under various loading conditions. This method allows for the analysis of complex phenomena, including fracture propagation, stability of slopes, and the effects of jointing and faulting.

The applicability of DEM to study the stability, the failure mechanisms and the seismic vulnerability of masonry structures has been discussed by various authors [156, 157, 158, 159, 160], who have emphasized its suitability in capturing the unique mechanical behaviour of masonry characterized by discontinuities and interactions between individual units. DEM methods have been especially applied to study the historical structures' seismic response, such as adobe churches, traditional pagodas, and multilevel towers [161, 162, 163].

A 2D DEM model of the Nuraghe has been developed with the software UDEC (Universal Distinct Element Code) by Mirabella Roberti and Spina [164] to assess the structure stability and understand the structural behaviour.

- *Essential features in 3DEC*

Developed in the early 1980s, UDEC was the first commercial implementation of the Cundall DEM formulation. It allowed 2D analysis of systems of rigid bodies in mechanical interaction, either polygonal blocks or circular particles. The intended application was the analysis of slope stability in jointed rock masses, with specific attention to the potential separation and movement of individual rock blocks during the failure process.

3DEC was developed in 1988 and implements the same core Cundall formulation, but in three dimensions, with polyhedral blocks instead of polygons. The handling of the contact problem in 3D, using the common-plane concept [165], was the main innovation. Particular care was devoted to implement efficient contact detection and update techniques, which are critical for large 3D systems.

In 3DEC blocks can be modelled as either rigid (i.e., infinitely stiff) or deformable [166].

The software requires that all polyhedral blocks be convex, primarily to simplify contact detection and interaction algorithms. However, non-convex geometries can be obtained by “joining” or “gluing” together the faces of multiple convex sub-blocks, which are then constrained to move as a single unit without relative displacement. Deformable blocks in 3DEC are internally meshed using tetrahedral elements. Additionally, higher-order elements are available to provide improved accuracy in modelling bending behaviour in shells and elasto-plastic deformation in materials.

Deformable blocks in 3DEC can be modeled as either elastic or exhibiting nonlinear behaviour. **Elastic blocks** deform under load but return to their original shape upon unloading, following a linear stress–strain relationship. In contrast, **nonlinear blocks** display a more complex, non-proportional relationship between stress and strain, often involving permanent deformations. One of the most widely used constitutive models is the **Mohr-Coulomb elasto-plastic model**, which enables the simulation of both tensile and shear failure within the block material. Furthermore, 3DEC allows the implementation of **user-defined constitutive models**, significantly enhancing the flexibility and applicability of the software to a wide range of geomechanical problems.



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In 3DEC, each block interacts with adjacent blocks through **point contacts**, meaning that mechanical interaction forces are computed and applied at a discrete set of contact points rather than over continuous surfaces. These contact forces are functions of the **relative displacements** between the interacting blocks at each contact point.

However, each contact point is assigned an area, and all the areas add up to the total contact surface (for face-to-face contacts). Therefore, joint stresses can be evaluated at each point contact, and the standard joint constitutive models, relating normal and shear stresses with joints displacements, can be employed.

In 3DEC, the contact area is defined using the concept (Cundall 1988) of a common-plane between 2 convex polyhedral, defined as the plane that maximizes the gap, or minimizes the overlap, between the blocks. The common-plane defines the contact normal. In 3D, 2 types of elementary point contacts (also termed sub-contacts) may exist, vertex-to-face and edge-to-edge. These may represent true pointwise interactions (in which the vertex-to-face includes the particular cases of vertex-to-edge and vertex-tovertex), or be associated to simulate a face-to-face interaction. The sum of the areas of the point contacts equals the total contact area. For true point interactions, a minimum contact area is defined. This minimum area multiplied by the joint stiffness gives the stiffness of the point contact, and should not be too low, in order to avoid excessive overlap between blocks.

In 3DEC, joints between blocks are modeled as **zero-thickness interfaces** located at the boundaries between adjacent blocks. Along these interfaces, **contacts** are defined at block edges or corners, where mechanical interaction is represented by a pair of conceptual springs: one in the **normal** and one in the **shear** direction. These springs enable the transfer of normal and shear forces across the joint. The **normal and shear stiffness** parameters control the **elastic deformability** of the contact, while **strength parameters** (e.g., cohesion, friction angle, tensile strength) govern the **nonlinear behaviour**.

A coulomb-slip joint model is the basic constitutive joint model employed in 3DEC, and all the sub-contacts between the blocks follow the Coulomb friction rules; with shear failure, tensile failure, and joint dilation all being modelled. A full Mohr-Coulomb model, with tensile and shear strength, is also available. As for block constitutive models, 3DEC allow user-defined joint models. In particular, softening laws adequate for masonry joints may be introduced by modifying the coding examples provided for built-in models.

In 3DEC, the equations of motion for the block system are solved using a **time-stepping explicit algorithm**. Unlike the stiffness matrix approach typically employed in finite element (FE) methods, the explicit algorithm is particularly well suited to problems involving **large displacements, discontinuous deformation, and rapidly changing contact conditions**, such as those observed during collapse mechanisms or when blocks become unconnected. Although fundamentally a **dynamic solution method**, the explicit time integration scheme in 3DEC is also commonly applied to **static or quasi-static problems**. In such cases, the algorithm proceeds iteratively until the system reaches **static equilibrium** [156]. To facilitate convergence in static analyses, **damping** is introduced—either as **numerical damping** or **artificial viscous damping**—which helps to dissipate kinetic energy and guide the system toward equilibrium or a stable failure mode.

- *Nuragic Tower model*

The geometric vulnerability assessment, related to ground instability and seismic hazards (topic of the next chapter), was carried out using a complete model of the Nuragic Tower, which incorporates the two primary structural elements (tholos and external wall) and the filling material.

The geometries and mechanical properties of the masonry structures (i.e., blocks of tholos and external wall and joints) are the same as those previously described for the models of the tholos and the external wall. (paragraph 5.2.1.2):

- the blocks are infinitely resistant and rigid (density 2800 kg/m^3), they have a regular shape and their dimensions decrease with the height;
- shear resistance of the contacts between the blocks expressed by Coulomb law ($\tau < \sigma_n \cdot \tan\phi$), with friction angle $\phi = 30^\circ$.

The filling material was modeled as a single deformable block characterized by an initial stress state. The stress linearly increases from top to bottom, with vertical component defined as $\sigma_v = \gamma \cdot z$ and horizontal components defined as $\sigma_h = K \cdot \gamma \cdot z$. The density of the filling material is set to 1500 kg/m^3 .

As reported in paragraph 5.2.3.5 the value of K depends on the geotechnical characteristics of the material and the methods with which the layers of filling material are constituted. Similarly, to K , the stiffness of

the backfill material, expressed by the Young's modulus E , is also correlated to its degree of compaction. These two parameters ultimately determine the load transferred by the filling material to the retaining walls. In order to assess their influence, various combinations of the parameters were simulated, as described in more detail below.

A Poisson's ratio of 0.25 was assumed for all cases. The analysis was initially performed within the elastic range, followed by the plastic range, applying the Mohr-Coulomb failure criterion to the filling material. The parameters assumed for this criterion were a cohesion of 0 and a friction angle of 30° .

- *Simulation of the initial static equilibrium state of the Nuragic Tower*

The complete model of the Nuragic Tower was subjected only to gravitational forces in order to obtain the initial equilibrium condition, which serves as the starting configuration for subsequent simulations (Figure 148). Four hypotheses regarding the condition of the filling material were considered:

- Case 1 $E = 5 \cdot 10^5 \text{ kg/m}^2$ and $k=0.33$;
- Case 2 $E = 5 \cdot 10^6 \text{ kg/m}^2$ and $k=0.33$;
- Case 3 $E = 5 \cdot 10^6 \text{ kg/m}^2$ and $k=1$;
- Case 4 $E = 5 \cdot 10^6 \text{ kg/m}^2$ and $k=3$.

The maximum radial block displacements for the simulated cases are reported below:

- Case 1: $d_{\text{max\_extwall}} = 2.2 \text{ cm}$;
- Case 2: $d_{\text{max\_extwall}} = 0.2 \text{ cm}$;
- Case 3: $d_{\text{max\_extwall}} = 2.5 \text{ cm}$;
- Case 4: A parameter combination that does not result in a stable equilibrium condition.



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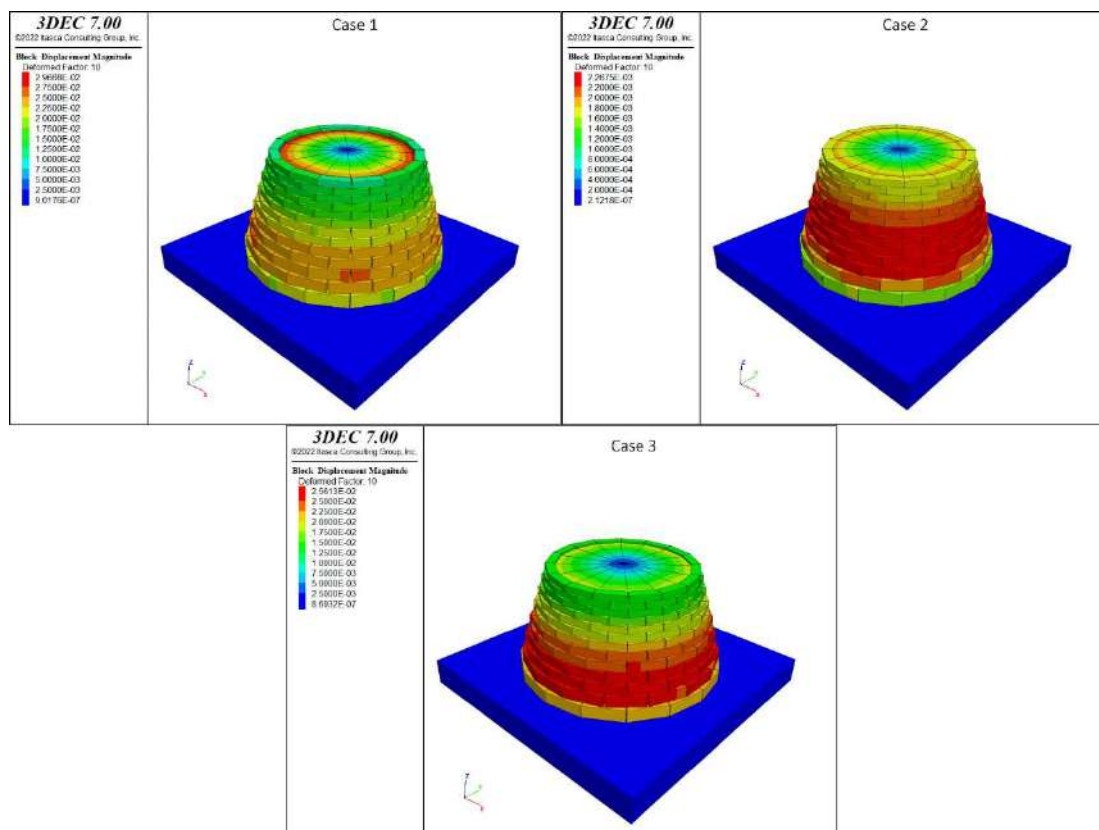


Figure 148. Initial static equilibrium configuration of the Nuragic tower for the simulated cases.

- *Comparison between the results obtained from the initial models (separated models of tholos and external tower) and the final model (complete model)*

The evaluation of the performance of the Nuragic Tower model was conducted by replicating the stability analyses previously carried out separately for the tholos and the external wall—outlined in 5.2.1 — on the entire Nuragic Tower, and by comparing the resulting data with the previously obtained outcomes.

For each case, the limit condition is reached by reducing the shear resistance of the joints, starting from the equilibrium of the structure ($\phi_{\text{joint}}=30^\circ$). The collapse of the Nuragic Tower develops with the radial translation of the external tower blocks; the Tholos remains stable and exhibits negligible displacements.

For the simulated cases, the limit conditions are reached at the following friction angle values:

- Case 1 $\phi_{lim} = 20^\circ$;
- Case 2 $\phi_{lim} = 15^\circ$;
- Case 3: $\phi_{lim} = 12^\circ$.

These values of limiting friction angles are lower than those previously determined for the external wall ($\phi_{lim} = 25^\circ$ per $k = 0.33$ paragraph 5.2.1). This difference arises from the fact that in the previous simulation, the effect of the filling material was modelled as a horizontal stress applied to the external wall independently of the radial displacements of the blocks. In the simulation of the complete structure, however, this stress is applied as an initial condition but is redefined during the calculation based on the deformability of the filling material, following the radial displacements of the blocks. Consequently, it decreases progressively as the radial displacements develop. This model appears to be more realistic than the initial one, where the two masonry structures were analysed separately and the effect of the filling material was simulated by applying a constant stress throughout the simulation.

Considering the operational construction methods of the Nuragic tower, the most realistic compaction level of the filling material corresponds to Case 3, where the coefficient K is equal to 1. Therefore, the subsequent vulnerability analysis of the Nuragic tower with respect to differential settlements is conducted with reference to this case.

- *Differential settlements Vulnerability analysis*

The analysis was conducted in order to establish the relationship between the differential settlement of the nuraghe foundation and the resulting damage caused to the structure.

The consequences of the following differential settlement values were analysed: 2, 4, 15, 25, 35, 50 e 65 cm, with the last value representing the threshold at which structural collapse occurs.

The simulation of the settlement was conducted by preliminarily dividing the nuraghe foundation base along the vertical YZ plane passing through its center, and by imposing a prescribed vertical displacement value on one of the two parts.



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The simulations demonstrate that differential settlement leads the following mechanisms:

- the blocks of the external wall and the tholos resting on the portion of the foundation affected by settlement undergo vertical displacement, which gradually decreases from the bottom towards the top;
- the blocks of the external wall in direct contact with the foundation, as well as those immediately above, primarily undergo vertical displacements, while the blocks higher up experience greater rotations and more pronounced horizontal displacements (Figure 149);
- the blocks of the tholos undergo smaller settlements compared to the external wall at the same height (see Figure 150) and begin to exhibit noticeable rotations starting from the level in contact with the foundation;
- The filling material experiences a maximum vertical displacement equal to that of the foundation (Figure 151).

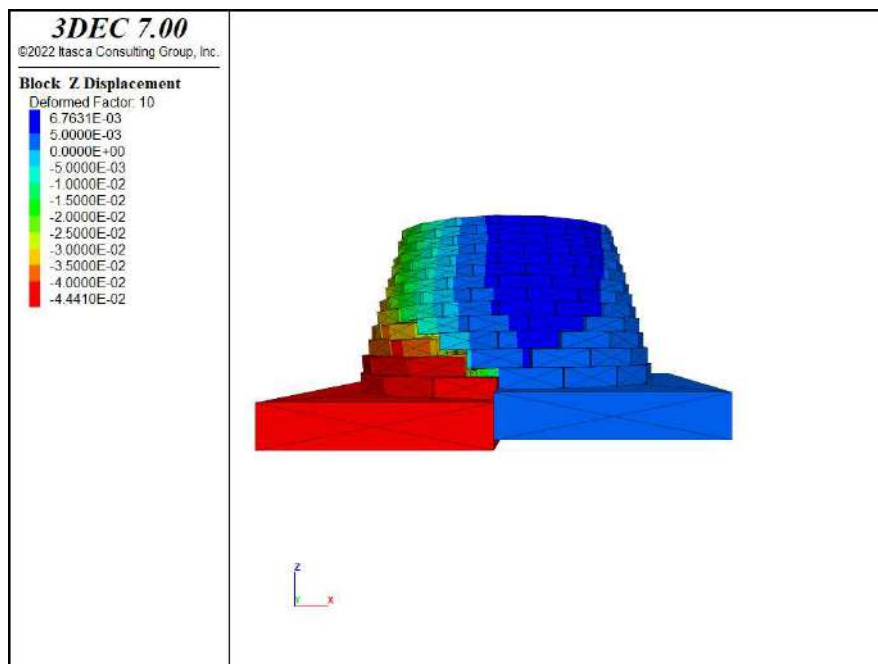


Figure 149. z-displacement of the external wall due to settlement of the basement (4 cm) – Front view.



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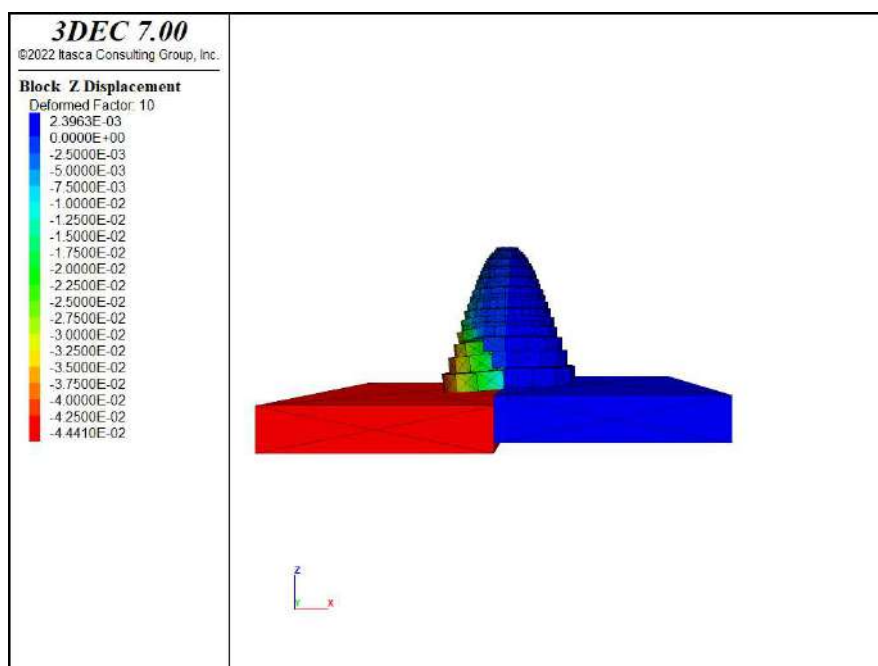


Figure 150. z-displacement of the tholos due to settlement of the basement (4 cm)– Front view.



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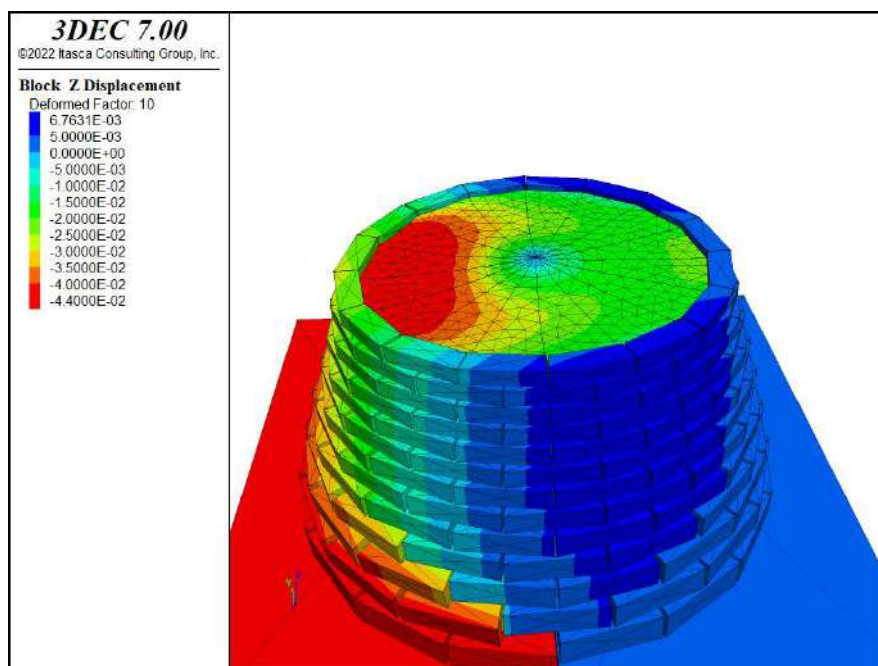


Figure 151. z-displacement of the Nuragic Tower due to settlement of the basement (4 cm)– 3D view.

The collapse of the structure occurs with the disconnection of the external wall, measurable through the displacement of the blocks relative to their initial positions. This displacement has been adopted as the reference parameter for evaluating the damage magnitude.

- *Conclusions and Vulnerability Assessment*

Based on the results presented in the previous sections and the damage definition provided in Chapter 4.1.3, this section quantitatively assesses the geometric vulnerability of the mono-tower Nuraghe with respect to the ground instability hazard factor.

To quantitatively assess the geometric vulnerability of the structure, numerical displacement values are first assigned to the displacement ranges characterizing each damage class, which were previously described in qualitative terms in the general discussion on vulnerability. In the following analysis, an upper displacement threshold is therefore introduced for each damage class (Table 41).



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The collapse threshold value of 800 mm was identified as a result of the analysis.

Table 41. Upper displacement threshold related to damage classes – Ground instability Hazard.

Class	Definition of the damage	Maximum displacement
1	The structure is unaffected by the event and remains in good preservation condition. The blocks do not undergo relative movement, and there are no cracks or mineralogical-petrographic alterations in the constituent material	0 -1 mm
2	The blocks undergo relative displacements that do not compromise the stability of the overall structure. No fracturing occurs, and no chemical or physical alterations are observed in the materials	1 – 10 mm
3	The blocks undergo significant relative displacements that compromise global stability but not the structural geometry (the structure does not collapse). New, non-through cracks appear in the blocks, and superficial mineralogical-petrographic alterations of the materials occur.	10 – 800 mm
4	The blocks undergo major relative displacements that compromise the geometric configuration (the structure collapses). The blocks show new through-going cracks that do not lead to fragmentation. Mineralogical-petrographic alterations are more widespread but do not compromise the blocks' structural integrity	>800 mm

The diagram in Figure 152 illustrates the relationship between the hazard intensity (expressed as the value of differential settlement) and the damage sustained by the structure, identified through the displacement of the blocks relative to their original position. This diagram therefore provides a quantitative description of the structure's vulnerability to the specific hazard.



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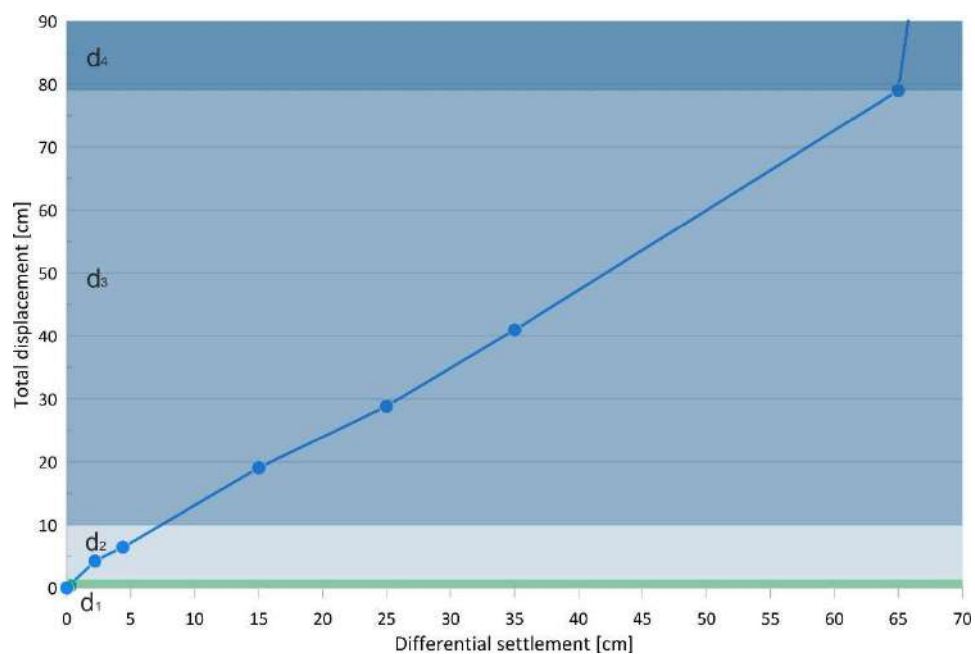


Figure 152. Damage classes defined as a function of displacement threshold levels.

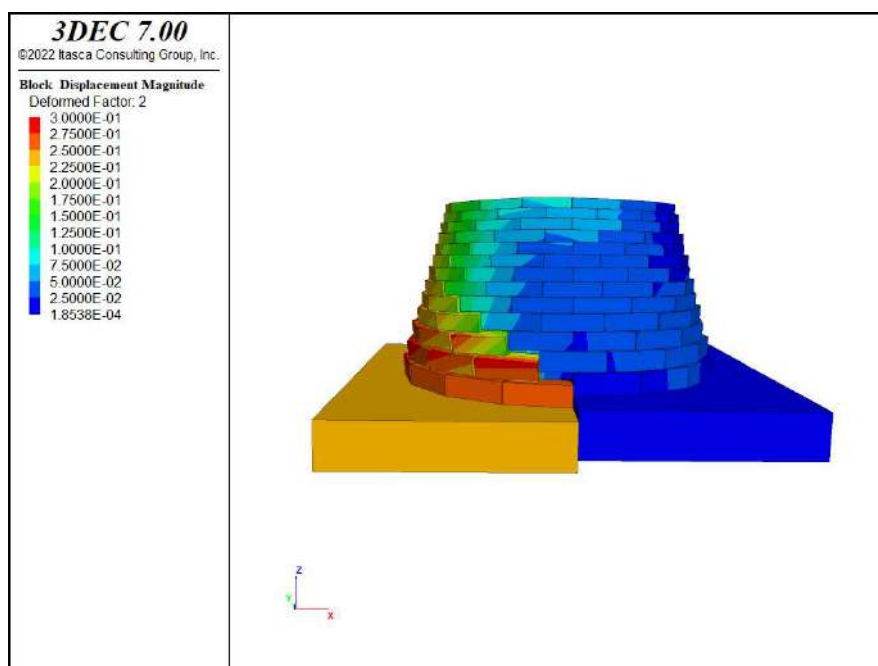


Figure 153. Total displacement of the Nuragic Tower due to settlement of the basement (30 cm)– 3D view.

5.3.4 FEM Analysis

- *The model of Nuraghe Pontes*

Nuraghe Pontes, located in the Marghine area near to the village of Dualchi, serves as a key case study (Figure 154). It is one of the best-preserved examples mono-tower nuraghe, distinguished by its intact tholos, which is encircled by a spiral staircase leading to a roof terrace (Figure 155). Its structure exemplifies the typological and structural characteristics typical of the Nuragic tradition [167, 168], as well as the common construction and decay mechanisms observed at many Nuragic sites. Owing to its exceptional state of preservation and representativeness, Nuraghe Pontes is one of the 32 sites selected by “La Sardegna Verso UNESCO” association for UNESCO World Heritage nomination.

A series of site inspections was carried out to reconstruct the precise geometry, document material degradation, and assess the structural condition of the monument [167]. A high-resolution LiDAR (Light Detection and Ranging) survey was commissioned by the “La Sardegna Verso L'Unesco” association and executed by Gianni Alvito and his company “Teravista” as part of the framework agreement with the Dept. of Civil and Environmental Engineering, and Architecture (DICAAR) of the University of Cagliari. The LiDAR point cloud enabled the acquisition of accurate geometric data, which formed the basis for a detailed structural model of both the tholos and the entire Nuraghe Pontes.

From the processed point cloud, a cleaned mesh was created and used to develop a detailed 3D model of the Nuraghe Pontes using Rhinoceros software [170]. A separate model of the tholos alone was created in AutoCAD (Autodesk, 2023), based on real internal curvature derived from the Rhino model. The model also incorporates variation in the tholos wall thickness, obtained through a statistical analysis carried out by the DICAAR research team [107], as illustrated in Figure 156. The actual geometry of the Nuraghe Pontes has been used to develop a Finite Element model.

The structural vulnerability of Nuraghe Pontes, considered a paradigmatic example of a mono-tower nuraghe with a complete tholos, was assessed through numerical analysis. In the present study, two principal hazards were examined: ground instability, modelled via differential settlements applied at the base of the tower, and seismic action, simulated through tilting analyses. The investigation focused both on the tholos, as the primary structural element, and on the tower as a whole. The geometry of the



structure was reconstructed using advanced digital modelling techniques, supported by point cloud data and field observations.

A 3D continuum model, based on the Finite Element Method (FEM), was used to conduct a numerical analysis of the tholos, accurately reproducing its geometry and mechanical behaviour. This model incorporates a discretised representation of the tholos featuring a simplified idealisation of its dry-jointed stone configuration. First, the geometry was reconstructed; then, to enable efficient and accurate nonlinear simulations, it was manually meshed with hexahedral and tetrahedral elements (Figure 157a). The meshed model illustrates the regular discretisation adopted for the finite element simulation. This mesh ensures the capability to capture both local stress redistribution and global collapse mechanisms. However, the complete nuraghe model was automatically meshed using tetrahedral elements (Figure 156b).

The model has been implemented in ABAQUS FEA [3].

From a structural perspective, the design and construction of corbelled domes — such as those found in the tholos of nuraghes — rely heavily on the principles of dry-stone masonry and the strategic horizontal projection of successive stone layers. These so-called *false domes* are constructed without mortar, using horizontally laid stones that progressively cantilever inward until they converge at the apex, forming a dome-like shape. Their stability is achieved primarily through gravity and friction between the stone elements [171]. A refined theoretical interpretation describes corbelled domes as three-dimensional extensions of the false arch concept [172]. Although these structures may appear intuitive or vernacular in nature, their construction reflects a deep body of constructive knowledge passed down through generations — particularly evident in monumental examples such as the Mycenaean Tholoi [173].

When evaluating the mechanical behaviour of nuraghes and other dry-stone structures, it is important to acknowledge the discrete and nonlinear nature of masonry materials. Classical Finite Element approaches often fail to capture the unique interactions between unbonded stones. For this reason, discrete element models have been proposed for masonry structures [156]. Specifically, for nuraghes, relatively few studies have focused on modelling their mechanical behaviour. Discrete element analyses of a Sardinian nuraghe have been conducted [164, 107] [174] to simulate inter-stone contact and investigate load-bearing



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performance under various conditions. In the broader context of vaulted masonry structures — where geometry and equilibrium are more critical than material strength [175] — Limit Analysis based on both the lower [176] and upper [177, 178] bound theorems is widely adopted. Nevertheless, Finite Element models can still be suitable, provided they accurately represent the mechanical behaviour of masonry. A detailed review of these modelling strategies is offered by D'Altri et al. [179]. A specific finite element model for masonry is adopted here, based on a perfectly plastic material formulation that enables the application of limit analysis [180].

In the absence of detailed material testing, the simulations are based on the typical properties of the local basalt and volcanic stone commonly used in Nuragic construction. The dry-joint behaviour is modelled conservatively to capture potential sliding or detachment under tilting conditions. The mechanical properties adopted for masonry are defined in Table 42 according to the most recent Italian technical standards [10] as updated by Circolare Esplicativa [22]. Average values were adopted for the masonry typology that is most representative of Nuraghe construction: *muratura in pietrame disordinata*, which refers to irregular, uncut stone masonry that is laid without mortar or with a weak binder.

Table 42. Elastic mechanical properties adopted for the Nuraghe Pontes

Elastic Modulus E [Mpa]	Density γ [kN/ m ³]
3200	19

Ad hoc designed parameter for plastic behaviour of masonry material are adopted, in particular a modified Mohr-Coulomb for no-tension material, whose mechanical parameter are reported in Table 43.

The Degenerate Mohr–Coulomb (DMC) model aligns with Heyman’s idealised assumptions for masonry structures: (i) zero tensile strength, (ii) infinite compressive strength (i.e., no crushing), and (iii) no sliding along joints [175]. This modelling approach captures the essential mechanics of dry-jointed masonry while remaining computationally efficient. Therefore, masonry is modelled as a frictional continuum governed by shear behaviour, with a friction angle in the range of $89^\circ < \phi < 90^\circ$ and a low value of cohesion $c > 0$

kPa. These properties adapt the mechanical parameters of the Mohr-Coulomb criterion to the no-tension material idealised by Heyman for masonry.

Table 43. Plastic mechanical properties adopted for the Nuraghe Pontes

Friction angle φ [°]	Cohesion c [Pa]
89	0.5



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Figure 154. The Nuraghe Pontes, Dualchi: aerial view.



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Figure 155. The Nuraghe Pontes, Dualchi: vertical section (with idealized backfill).



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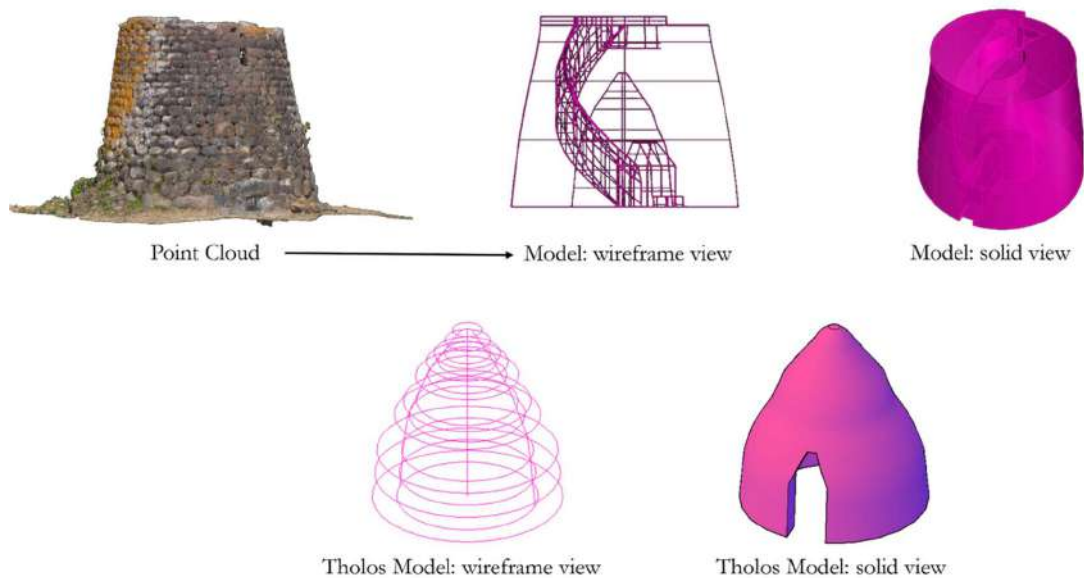


Figure 156. The Nuraghe Pontes: from point cloud to the geometrical model; full geometry (above) and the tholos alone (below).

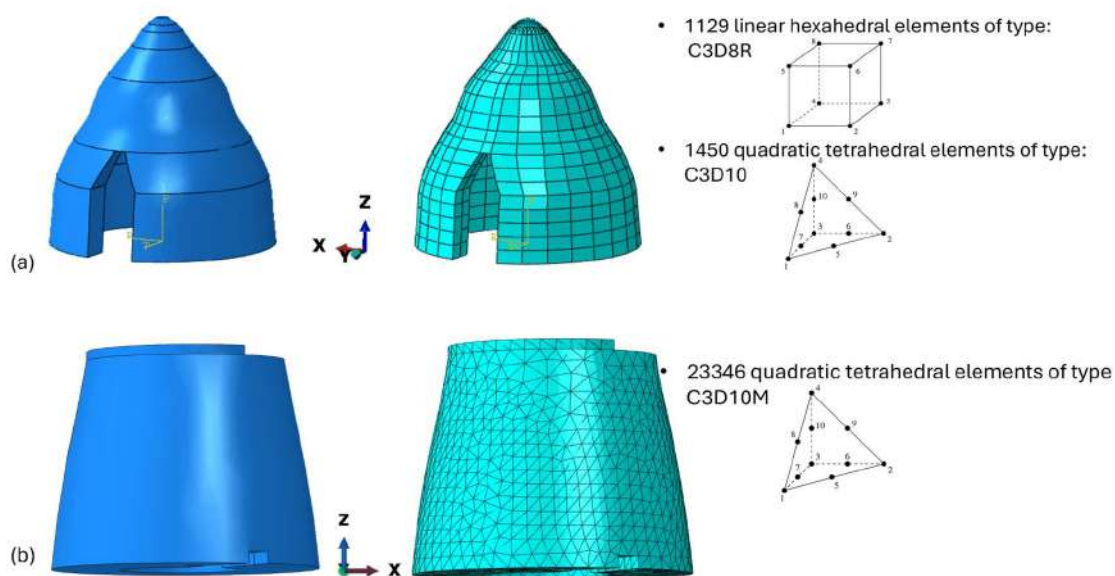


Figure 157. Assembly and mesh properties of the isolated tholos (a) and the Nuraghe Pontes (b).

- *Differential settlements*

This study investigates the impact of differential settlement on the structural performance of dry-jointed masonry in the Nuraghe Pontes, focusing on both the complete architectural complex and the isolated tholos. By analysing these two configurations under deformation induced by settlement, the objective is to assess the influence of foundation irregularities on structural stability and to compare their respective failure mechanisms.

The boundary condition simulates differential settlement, a common source of structural distress in historic masonry. This is implemented by applying an increasing uniform vertical displacement to one half of the base of both the tholos and the entire *nuraghe*, thereby inducing a tilt and generating non-uniform support reactions. The opposite half of the base remains fixed, creating a contrasting constraint that replicates the effects of uneven ground deformation.

This setup enables three-dimensional nonlinear analysis of deformation patterns, fracture evolution, and ultimate collapse mechanisms. The objective is to determine whether such imposed base distortion initiates localized failure (e.g., block separation or rotation) or triggers global instability. This approach builds upon previous studies investigating settlement-induced damage in dry-masonry systems and heritage structures [181, 182].

The structure is idealised using a Degenerate Mohr–Coulomb (DMC) material model [180], which effectively captures the key mechanical behaviour of dry-jointed masonry subjected to both gravitational loading and differential displacements. The model is based on Heyman's [175] classical assumptions for masonry structures: zero tensile strength, infinite compressive capacity, and no sliding along joints. The masonry is modelled as a frictional continuum, characterised by shear resistance governed by a very high value of friction angle (in the range $89^\circ < \phi < 90^\circ$) and low value of cohesion $c > 0$ kPa.

The analysis was conducted within a dynamic explicit framework. Each simulation begins with the gradual application of gravity loading using a smooth amplitude ramp, thereby minimising artificial dynamic effects. In the second step, a linearly increasing vertical settlement profile is imposed on a portion of the base to simulate controlled differential displacement (see Figure 158). As illustrated in the corresponding



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graph (Figure 159), the imposed displacement increases progressively over time, enabling detailed observation of the initiation and evolution of structural failure mechanisms.

This analysis was performed on both the complete Nuraghe Pontes structure and the isolated tholos to assess how structural context, geometric complexity, and load-path redundancy influence the system's capacity to accommodate differential settlement. The comparative results offer valuable insight into the resilience of ancient construction techniques when subjected to realistic ground deformation scenarios.

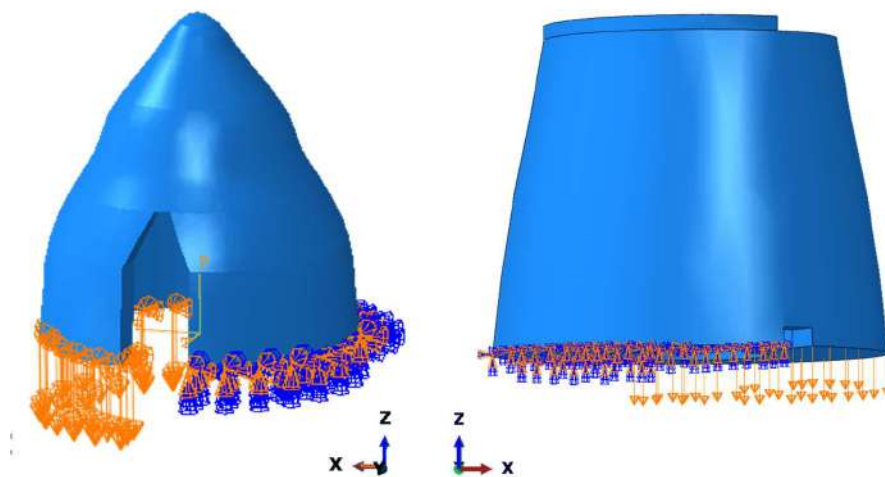


Figure 158. Applied vertical settlement and boundary conditions for the isolated tholos (a) and the Nuraghe Pontes (b).



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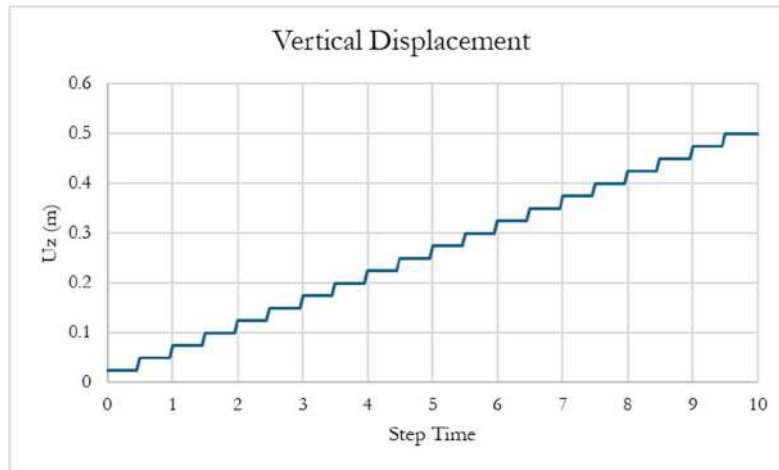


Figure 159. Graph representing the ramped applied values of the settlement.

- *Differential settlements of the tholos of the nuraghe Pontes*

The evolution of internal and external work throughout the simulation is illustrated in Figure 160 e Figure 161. Both quantities exhibit a progressive increase, reaching a peak followed by a subsequent decline, which marks the transition associated with structural collapse. The recurring pattern of peaks and plateaus corresponds to the stepped application of vertical settlement. As displacement is introduced incrementally, the structure undergoes successive cycles of adjustment, each characterised by an increase in energy followed by a temporary stabilisation phase.

Similarly, the strain and total energy graphs (Figure 162 and Figure 163) reach their respective maxima at the same point in time, signalling the onset of structural destabilisation. This critical condition, consistently observed across all monitored parameters, occurs at approximately step time = 5. According to the displacement graph, this corresponds to a vertical settlement of approximately 27.5 cm.



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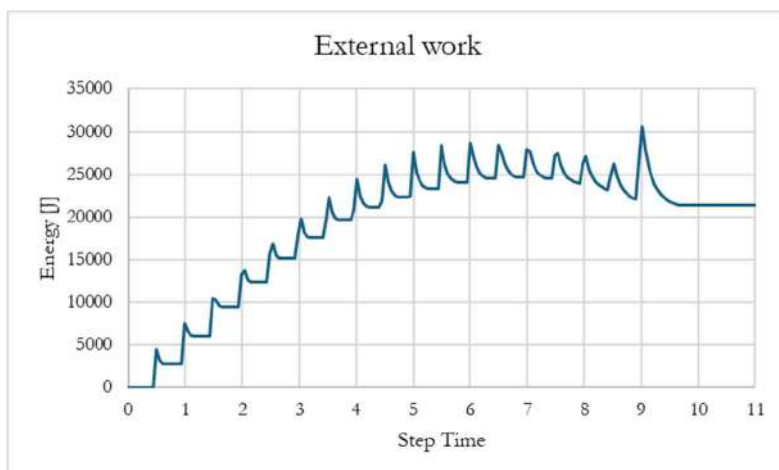


Figure 160. Graph of external work during differential settlement of the isolated tholos.

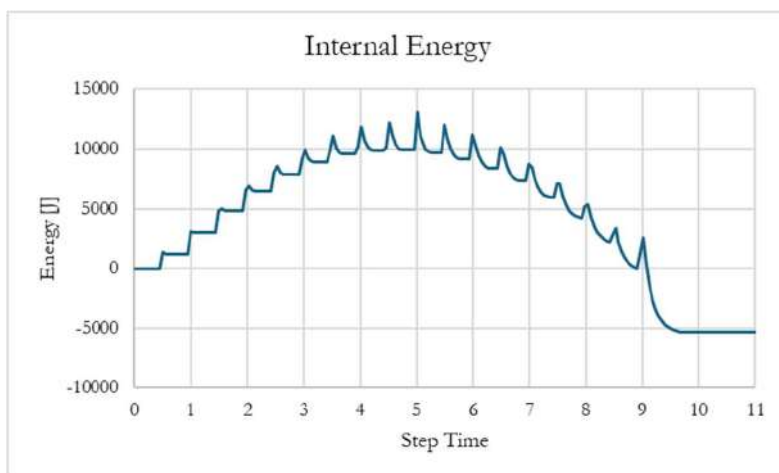


Figure 161. Graph of internal energy during differential settlement of the isolated tholos.



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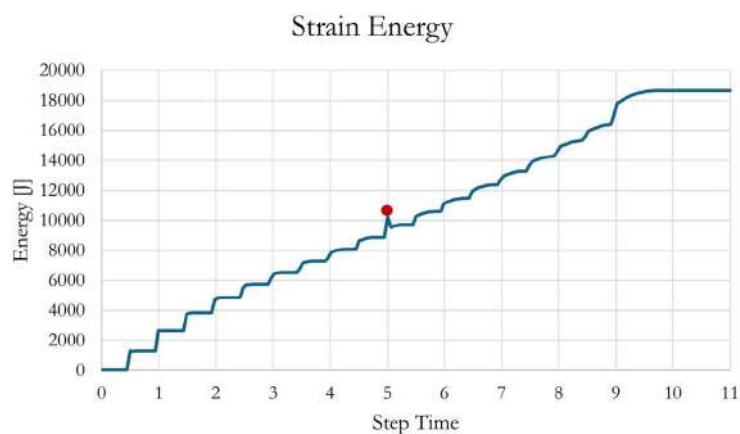


Figure 162. Strain energy evolution for the isolated tholos under differential settlement. The red marker highlights a significant change in the energy growth trend, marking the onset of instability.

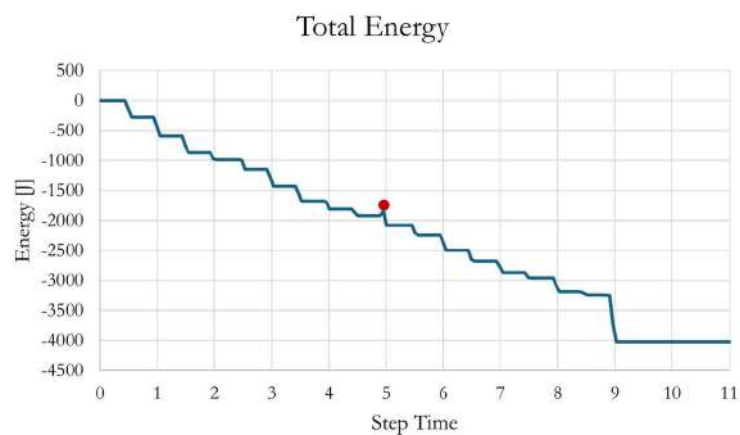


Figure 163. Total energy evolution for the isolated tholos under differential settlement. The red marker highlights a significant change in the energy growth trend, marking the onset of instability.

Figure 164 illustrates the distribution of maximum plastic strain in the tholos subjected to differential settlement. Plastic deformation is primarily concentrated near the base, especially around the doorway, where uneven support movement induces pronounced stress concentrations. Additional strain

accumulation at the top of the structure reflects global deformation patterns. The asymmetry in the strain distribution confirms the presence of non-uniform settlement, while the plastic strain zones indicate areas of irreversible damage and the onset of failure mechanisms.

Of particular note are the light blue vertical bands extending from the base toward the top in the central portion of the structure (Figure 164b), which correspond to regions undergoing tensile plastic strain. These zones mark the development of fractures resulting from internal force redistribution under differential settlement. Interestingly, the visualisation module identifies the initiation of collapse at $t = 6$ s, corresponding to a vertical displacement of $u = 33$ cm — approximately 20% higher than the displacement value inferred from the energy graphs.

This discrepancy between the collapse prediction obtained from the DMC model, the graphically identified multiplier, and the hinge formation observed in the visualisation is typical within the nonlinear regime. Rigid body models generally predict failure at significantly higher load levels. Indeed, as noted by [183], the peak collapse loads predicted by such models can exceed those obtained from refined continuum-based approaches by as much as 25–30%.



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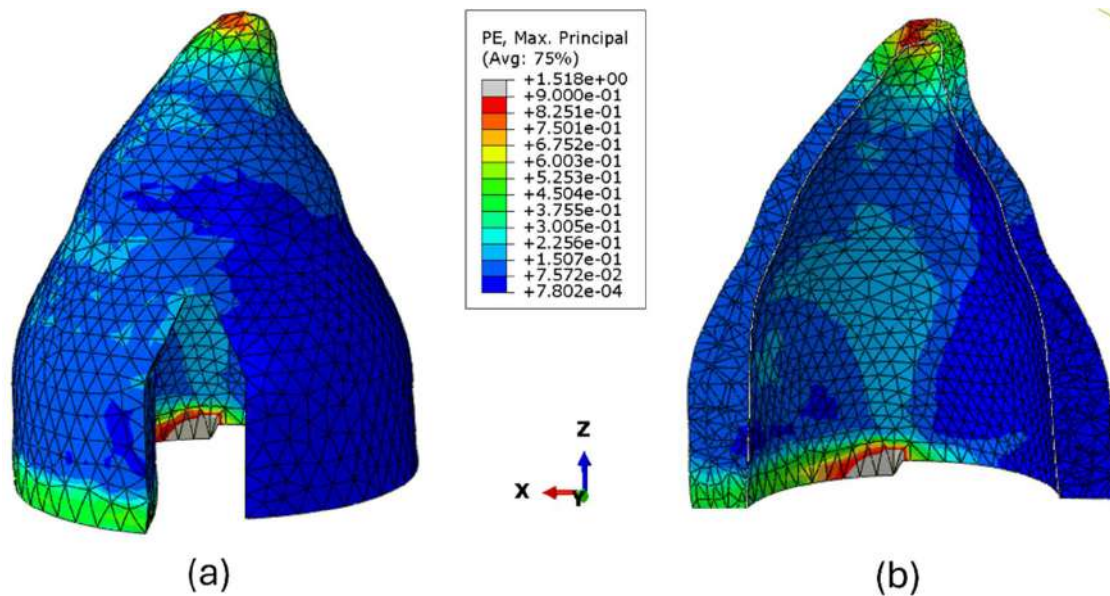


Figure 164. Plastic strain results of the differential settlement analysis of the tholos

- *Differential settlements of the nuraghe Pontes*

The following analysis evaluates the response of the full Nuraghe model to vertical settlement. As shown in Figure 165 and Figure 166, which depict the evolution of internal and external work, neither graph exhibits a distinct peak or subsequent decline. Instead, both quantities increase steadily throughout the simulation, with no evident energy release or transitional behaviour that would typically signal the onset of collapse. This continuous upward trend suggests that the structure remains globally stable under the applied settlement, sustaining deformations up to an imposed displacement of 1 m.

However, the absence of a clear peak in the energy graphs (Figure 167 and Figure 168), combined with the early development of cracks in the strain field, implies that structural failure may occur beyond the 1 m displacement limit used in the present analysis. These observations point to an impending collapse mechanism that is not yet fully activated within the simulated range, highlighting the structure's capacity to absorb substantial differential settlement before reaching a critical state.



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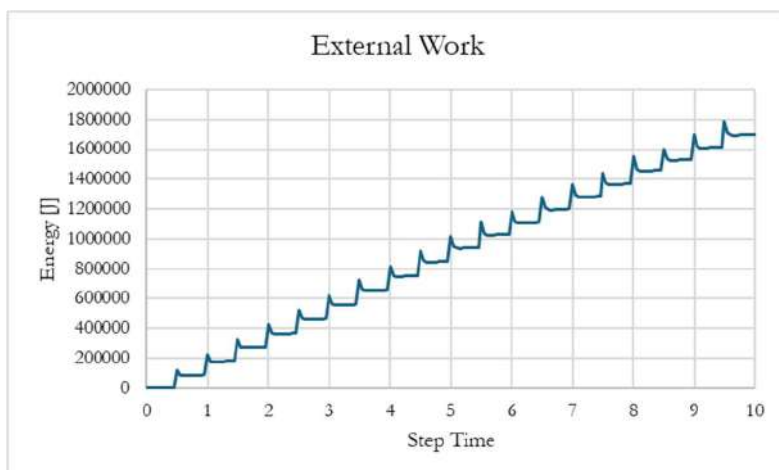


Figure 165. Graph of external work during differential settlement of nuraghe Pontes.

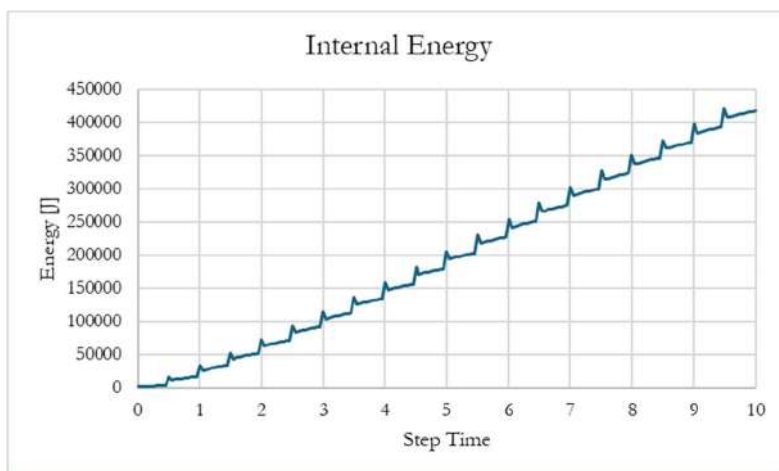


Figure 166. Graph of internal energy during differential settlement of nuraghe Pontes.



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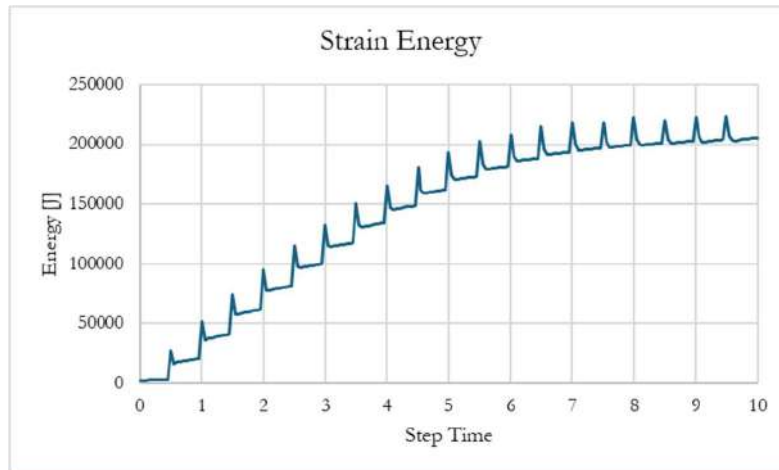


Figure 167. Strain energy evolution for the Nuraghe Pontes under differential settlement.

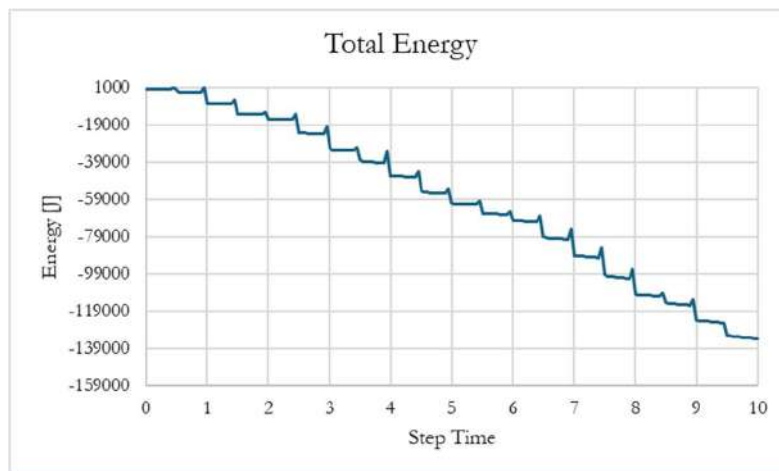


Figure 168. Total energy evolution for the Nuraghe Pontes under differential settlements.

Although the energy-based indicators remain inconclusive in identifying structural failure, the distribution of plastic strain offers a more revealing perspective. As illustrated in Figure 169, the maximum plastic strain at the end of the simulation highlights the development of a vertical crack line originating at the base and propagating upward. This localised damage pattern suggests the initiation of failure mechanisms that are still in the early stages and have not yet evolved into a full collapse within the current simulation range.



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These results imply that, while the structure remains globally stable, it is approaching a critical state characterised by accumulating internal damage. The presence of this incipient cracking indicates that further imposed settlement would likely exacerbate the deformation, making the point of destabilisation more pronounced and enabling the full collapse mechanism to be captured.

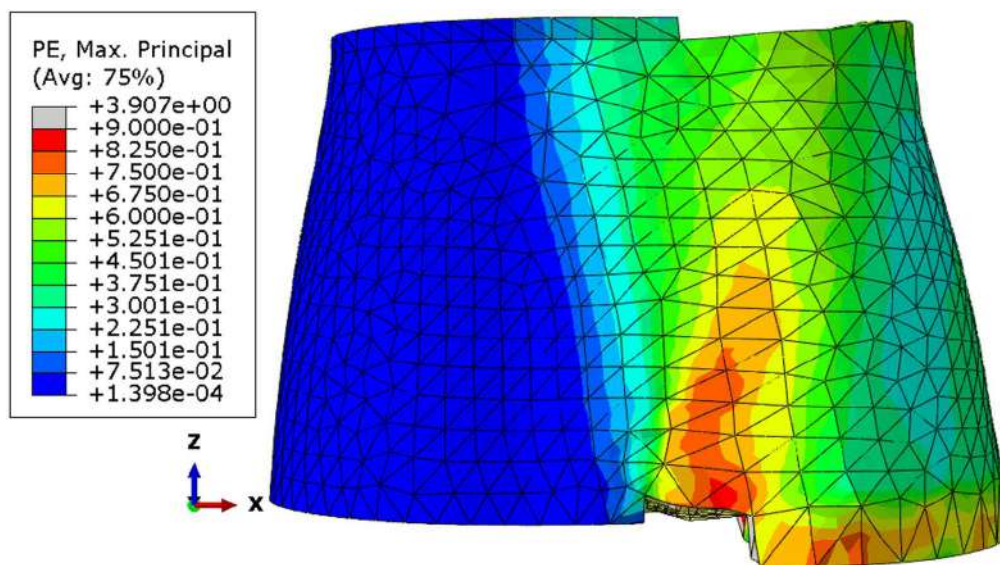


Figure 169. Plastic strain results of the differential settlement analysis of Nuraghe Ponte.

- *Ground instability vulnerability of nuraghe Pontes*

This research examines the structural response and vulnerability of dry-jointed masonry in the Nuraghe Pontes by analysing both the isolated tholos and the complete architectural complex under two destabilising conditions: tilting induced by inclined supports and differential settlement. The analysis employs a numerical framework based on the Degenerate Mohr–Coulomb (DMC) model within a dynamic explicit simulation environment, enabling a detailed representation of the non-linear behaviour and failure mechanisms typical of unreinforced dry-stone masonry structures.

Under differential settlement, the isolated tholos experiences progressive collapse, marked by a steady increase in internal energy, strain, and plastic dissipation. Collapse is observed at $t = 6$ s, when the



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imposed vertical displacement reaches approximately 33 cm, resulting in a continuous vertical fracture extending from the base to the top. What begins as localised deformation near the doorway and base evolves into a dominant axial failure mode, indicative of a brittle, global collapse mechanism. The lack of lateral confinement and structural integration—features provided by the surrounding elements in the complete Nuraghe—limits the tholos's ability to redistribute stresses under uneven support conditions. While the isolated tholos demonstrates some capacity to withstand moderate differential settlement, its integration within the full Nuraghe configuration significantly improves overall resilience. In the complete structure, no signs of global collapse were observed up to a settlement of 1 m, indicating a markedly higher tolerance to ground deformation. Nevertheless, the distribution and concentration of plastic strain near the base suggest the initiation of cracking, with substantial structural damage likely to emerge beyond this displacement threshold.

The comparative analysis clearly demonstrates that the structural stability of the tholos is greatly enhanced when it is incorporated into a larger architectural system. This improvement results not only from the additional mass and lateral support provided by the surrounding masonry but also from the inherent effectiveness of the tholos's corbelled dome—a self-supporting form in which dry-jointed stones are gradually cantilevered inward to construct a false vault.

5.4. Seismic risk analysis

5.4.1 Seismic hazard

Seismic hazard represents the probability that, at a given site and within a specified time interval, an earthquake will occur with characteristics capable of producing significant ground shaking. This hazard is generally expressed in terms of peak ground acceleration (PGA), ground displacement, or spectral intensity.

5.4.2 Seismic vulnerability analysis

The seismic vulnerability of mono-tower nuraghi has been investigated through pseudostatic (pushover) analyses and tilt tests. The former was conducted using the Discrete Element Method (DEM), while the latter were performed through Finite Element Modelling (FEM).

Pushover analysis is a widely used simplified method to estimate the seismic response of structures. Unlike linear static methods (pseudostatic method), it is a nonlinear static analysis technique that applies incrementally increasing lateral loads to a structural model in order to simulate the gradual accumulation of damage and inelastic behaviour under seismic action. This approach allows to evaluate the structure's capacity to withstand seismic forces beyond the elastic range, providing critical insights into its ultimate strength, ductility, and failure mechanisms.

Tilt tests represent a valuable method for evaluating the seismic vulnerability of masonry structures, particularly those composed of dry-jointed or irregular stone blocks. By gradually tilting a structure or a physical model until instability occurs, these tests allow for the identification of critical angles of overturning and the onset of failure mechanisms. When interpreted in relation to seismic hazard, tilt test results can provide an estimate of the horizontal acceleration levels that would lead to collapse, serving as a proxy for the structure's seismic capacity.

5.4.3 DEM analysis

- *Pushover analyses*

The geometric vulnerability assessment related to seismic hazards was carried out using the geometric model of the Nuragic Tower described previously (paragraph 5.3.3 **Errore. L'origine riferimento non è stata trovata.**).

Pushover analyses were performed by applying a gradually increasing horizontal acceleration to the Nuragic Tower; in this way, each block experiences a force proportional to its mass. This type of loading simulates the inertia-induced forces generated during a seismic event—although only in a pseudostatic manner.

First, gravity is applied and then an increasing horizontal acceleration is applied in successive steps; for each step, the analysis is performed until an equilibrium configuration is attained. Acceleration increments of 0.1 m/s^2 were applied until reaching 1.0 m/s^2 ; beyond this point, the increment was increased to 0.5 m/s^2 .

Although the most realistic model is that described in Case 3 ($E = 5 \cdot 10^6 \text{ kg/m}^2$ and $K = 1$) the analysis was also carried out with reference to Case 1 ($E = 5 \cdot 10^5 \text{ kg/m}^2$ and $K = 0.33$) in order to highlight the effect of the compaction degree of the filling material on the structural vulnerability.

- *Results*

In Case 1, the simulations demonstrate the onset of the mechanisms detailed hereunder.

The application of acceleration in the x-direction induces displacements in the same direction for both the masonry blocks and the filling material. This displacement reaches its maximum for the blocks of the external tower located along the generatrix defined by the intersection of the structural surface with the vertical xz-plane (line BB in Figure 170 and Figure 171). The highest displacement values are observed for the external tower blocks positioned in the direction of acceleration propagation (line BB). The difference in horizontal displacement between the blocks located along the outgoing generatrix (line BB) and those along the incoming generatrix (line AA in Figure 170 and Figure 171) results in a horizontal expansion of the structure in the horizontal direction, which can reach several tens of centimetres in the pre-collapse condition.

This difference is greatest for the blocks located along the two aforementioned lines (AA and BB) at an elevation of $z = 3 \text{ m}$ (identified as position A and position B in Figure 170). The graph in Figure 172 shows the x-direction displacements of these blocks as a function of the applied x-acceleration.



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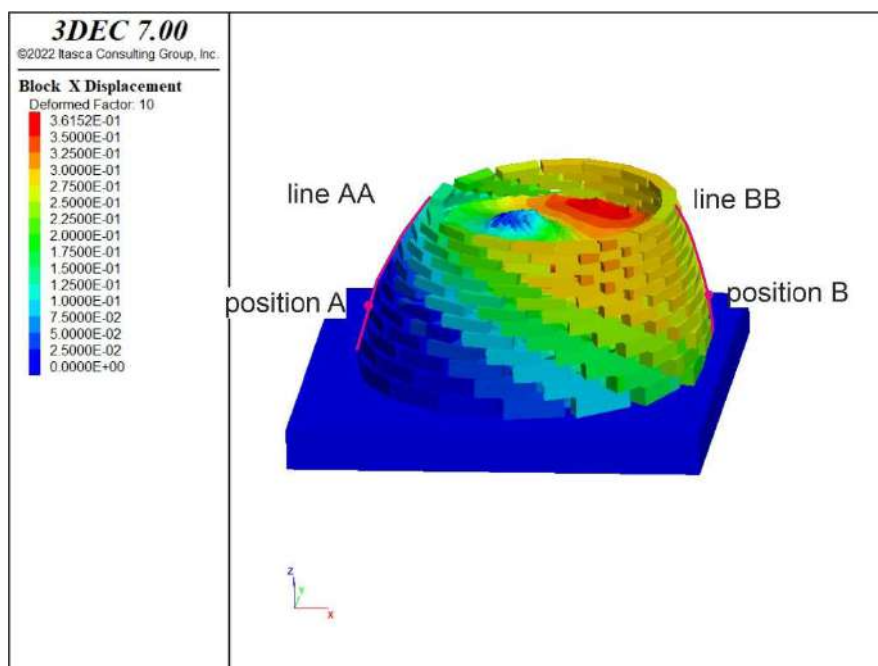


Figure 170. X-displacements of the deformed 3D model ($ax = 2.5 \text{ m/s}^2$) and indication of the reference positions (line AA and line BB)" – 3D view.



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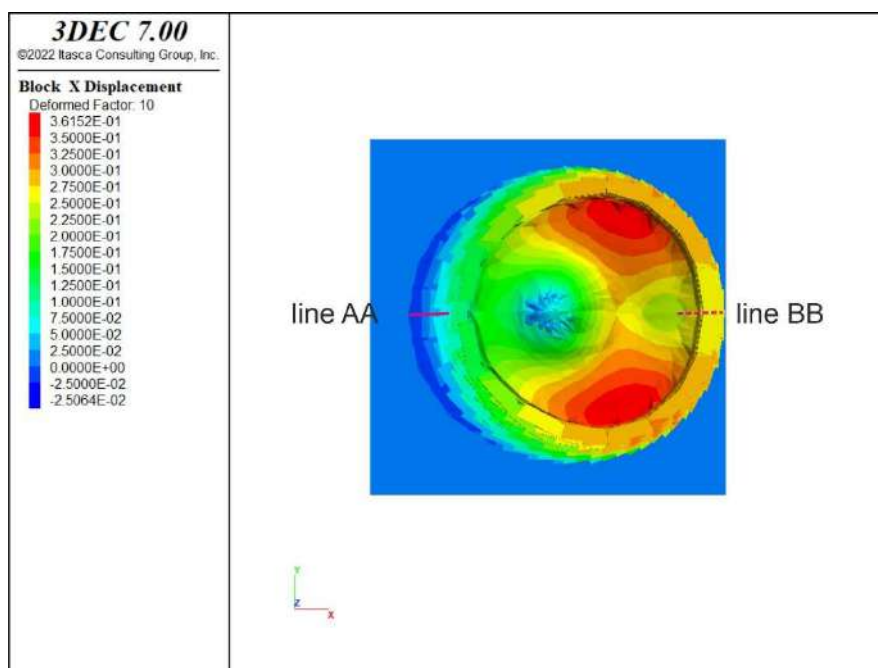


Figure 171. X-displacements of the deformed 3D model ($a_x = 2.5 \text{ m/s}^2$) and indication of the reference positions (line AA and line BB)" – TOP view.

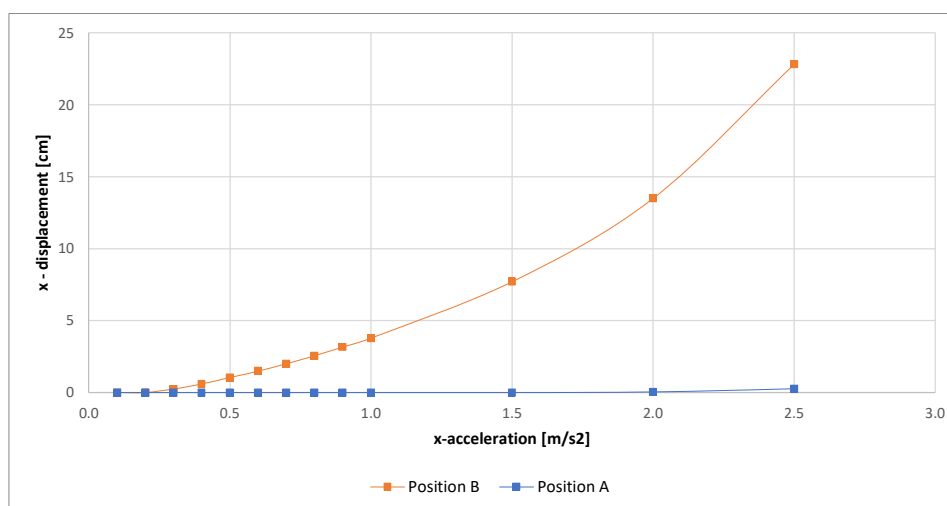


Figure 172. x displacements of the external wall blocks located at positions A and B – Case 1.



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The blocks of the tholos experience significantly smaller horizontal displacements compared to those of the external tower, as shown in Figure 173 and evidenced by the comparison between the x-displacement values of points C and D of the tholos (location shown in Figure 174) shown in the diagram in Figure 175, and those of the points on the external tower.

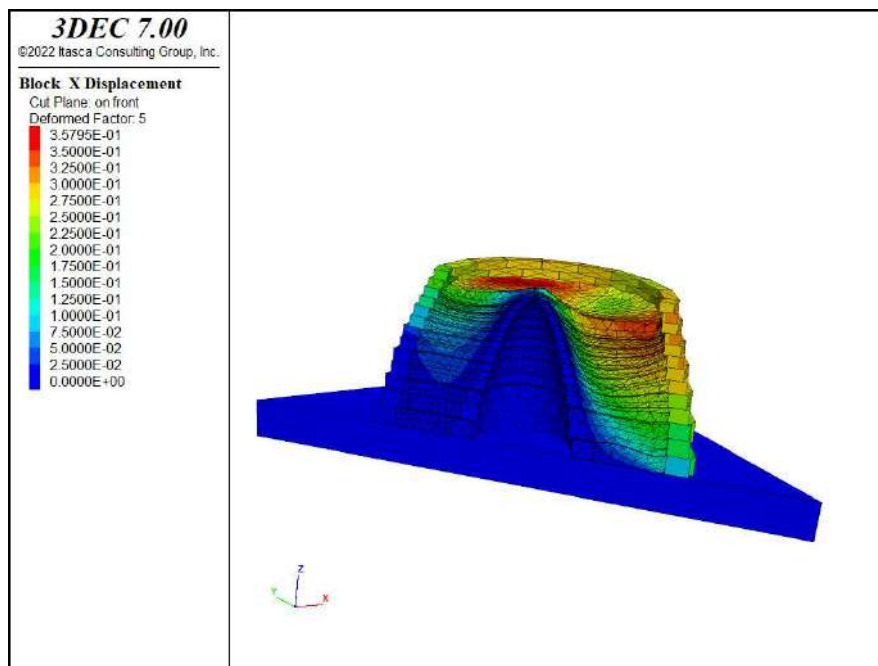


Figure 173. X-displacements of the deformed 3D model ($ax = 2.5 \text{ m/s}^2$) and indication of the reference positions (line AA and line BB)" – Section view – Case 1.



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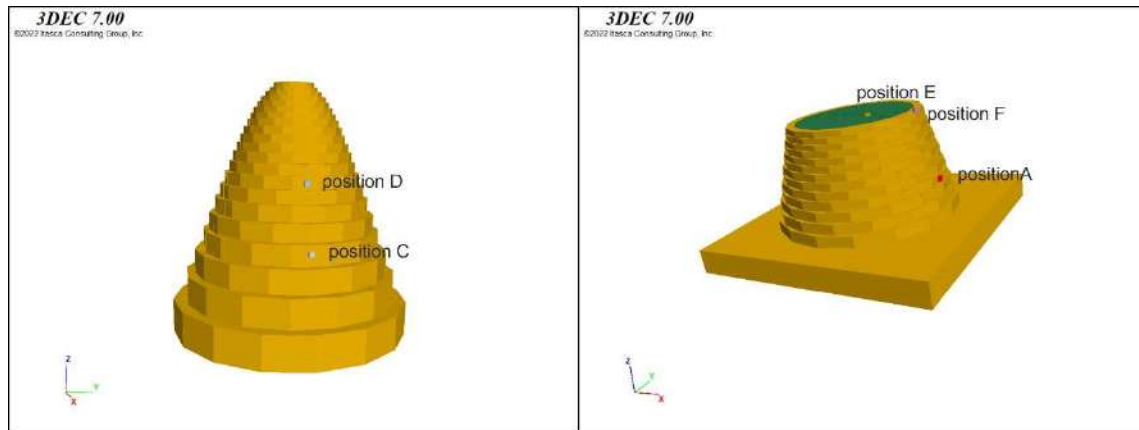


Figure 174. Locations of reference points.

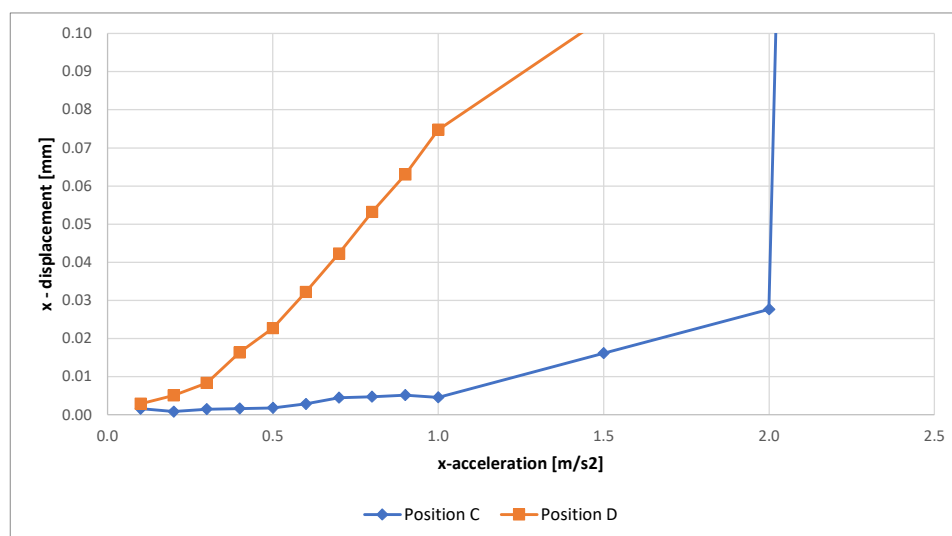


Figure 175. x-displacements of the tholos blocks located at positions C and D – Case 1.

As a result of the horizontal expansion of the structure , the filling material exhibits dilation in the direction of acceleration propagation (positive x-direction), accompanied by a significant reduction in vertical dimension, as evidenced by the collapse of the upper surface of the tower, shown in Figure 176 and Figure 170.



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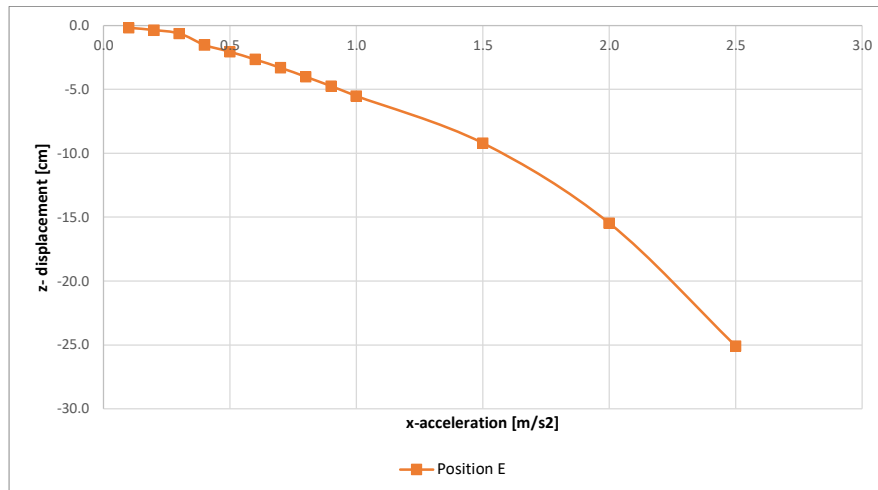


Figure 176. z-displacements of the filling material at positions E – Case 1.

The collapse of the structure occurs at horizontal acceleration values exceeding 2.5 m/s^2 , as a result of the failure of the external tower.

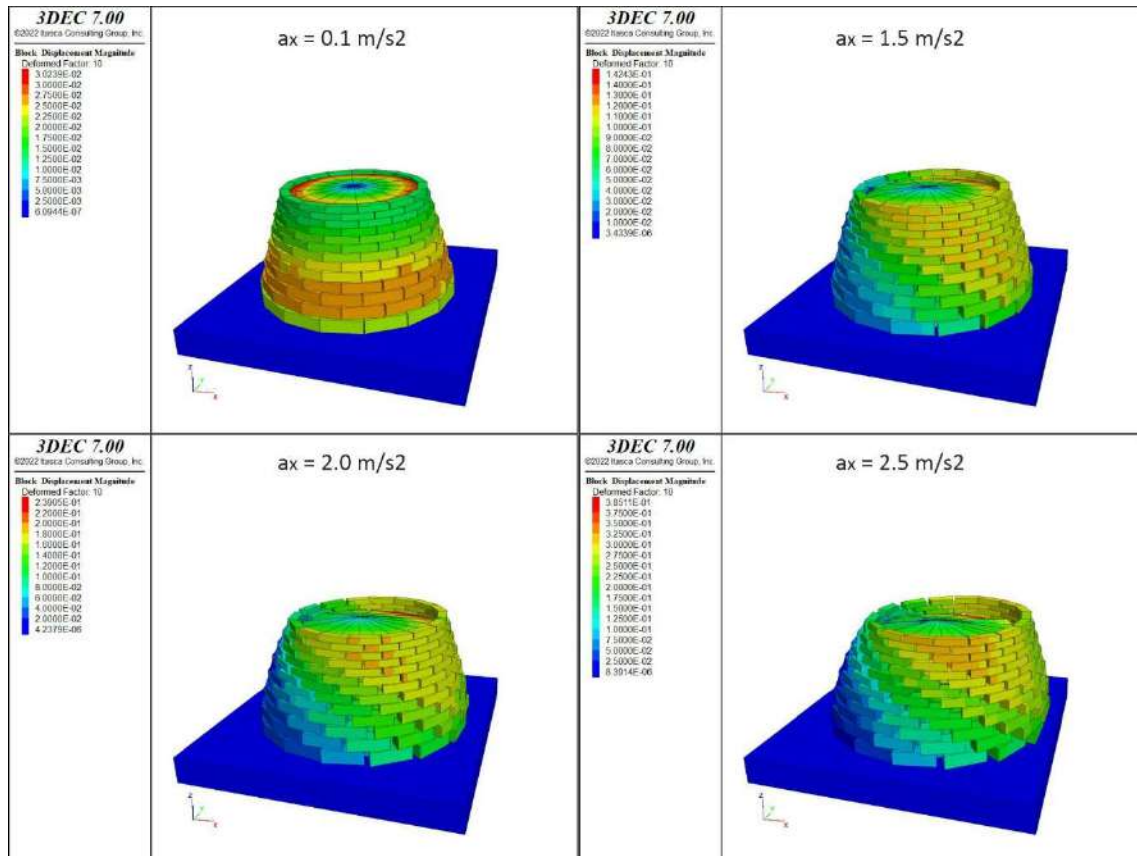


Figure 177. Displacements under varying levels of incremental horizontal loading (magnification factor equal to 10) -Case 1

With reference to Case 3, the simulations show the occurrence of failure mechanisms qualitatively similar to those described for Case 1. In this case, the higher stiffness of the infill material leads to:

- Lower horizontal displacements of the external tower blocks for the same value of horizontal acceleration;
- Maximum horizontal displacement of the external wall blocks at elevation $z = 8.1$ m along line AA (position F in Figure 174);
- Reduced vertical displacement of the infill material;
- Collapse occurring at an acceleration value of 5 m/s^2 , highlighting the stabilizing effect of the infill material's stiffness and the horizontal stress it induces.



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These results are illustrated in the following figures.

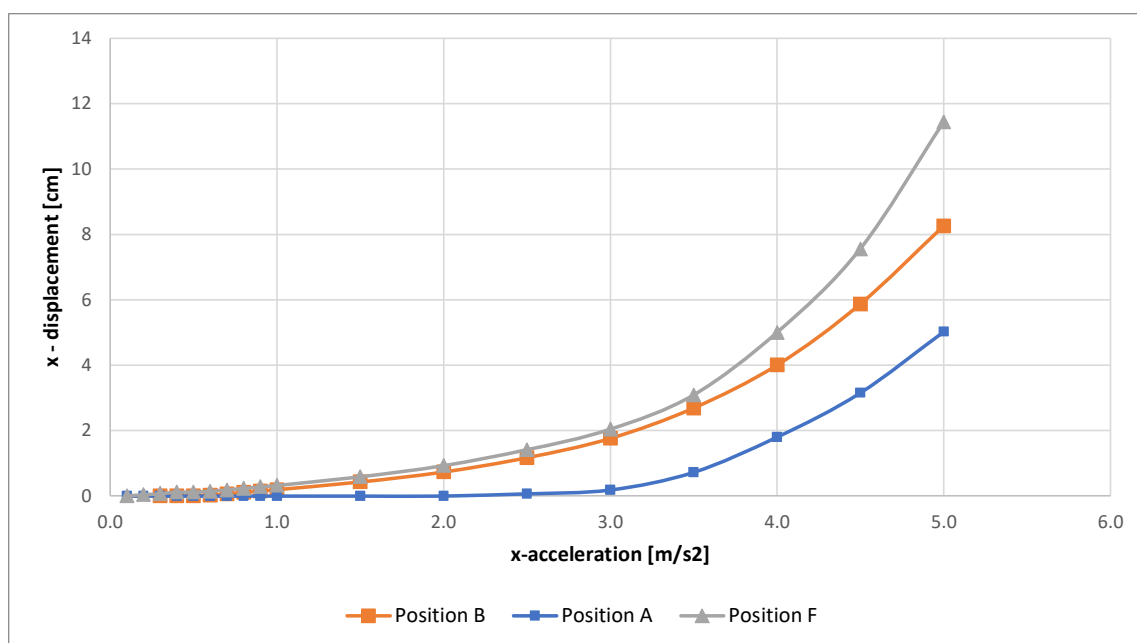


Figure 178. *x* displacements of the external wall blocks located at positions A, B and F – Case 3.

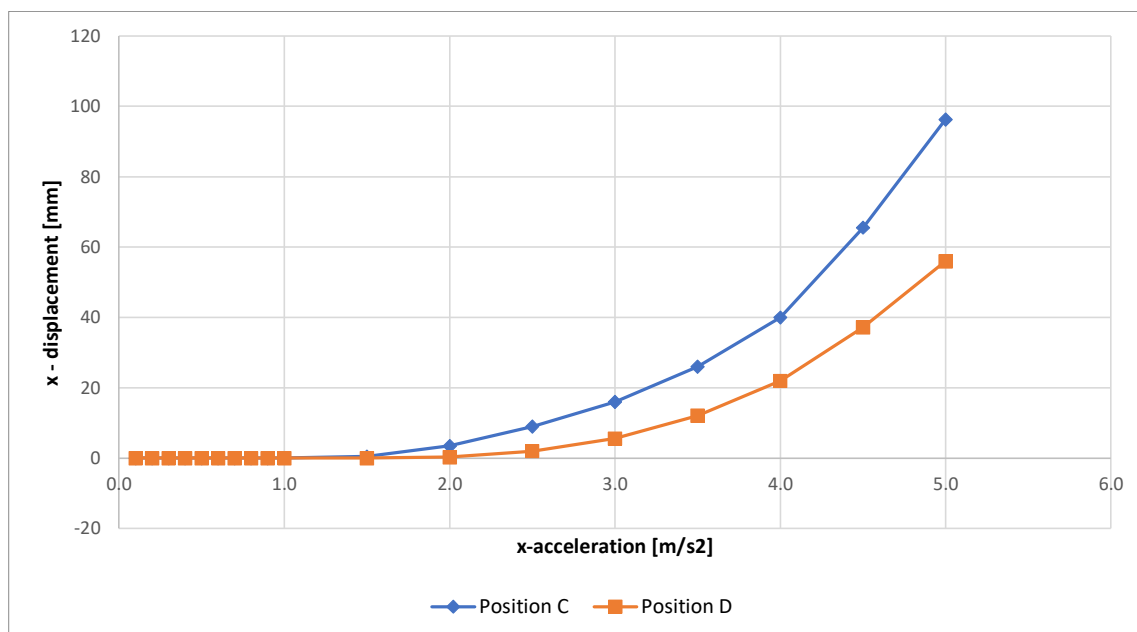


Figure 179. x-displacements of the tholos blocks located at positions C and D – Case 3.

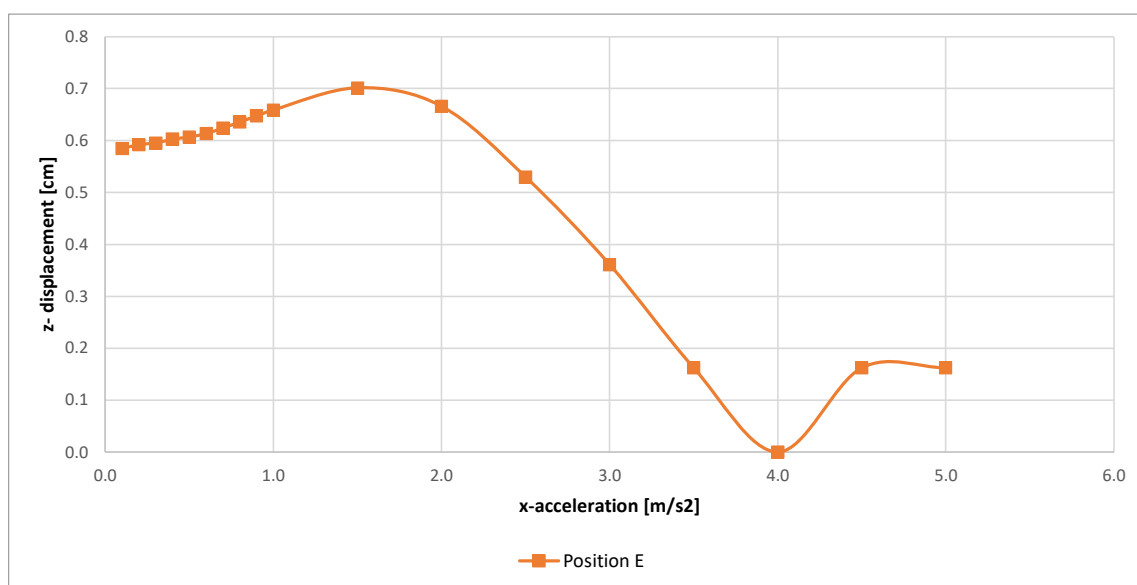


Figure 180. z-displacements of the filling material at positions E – Case 1.



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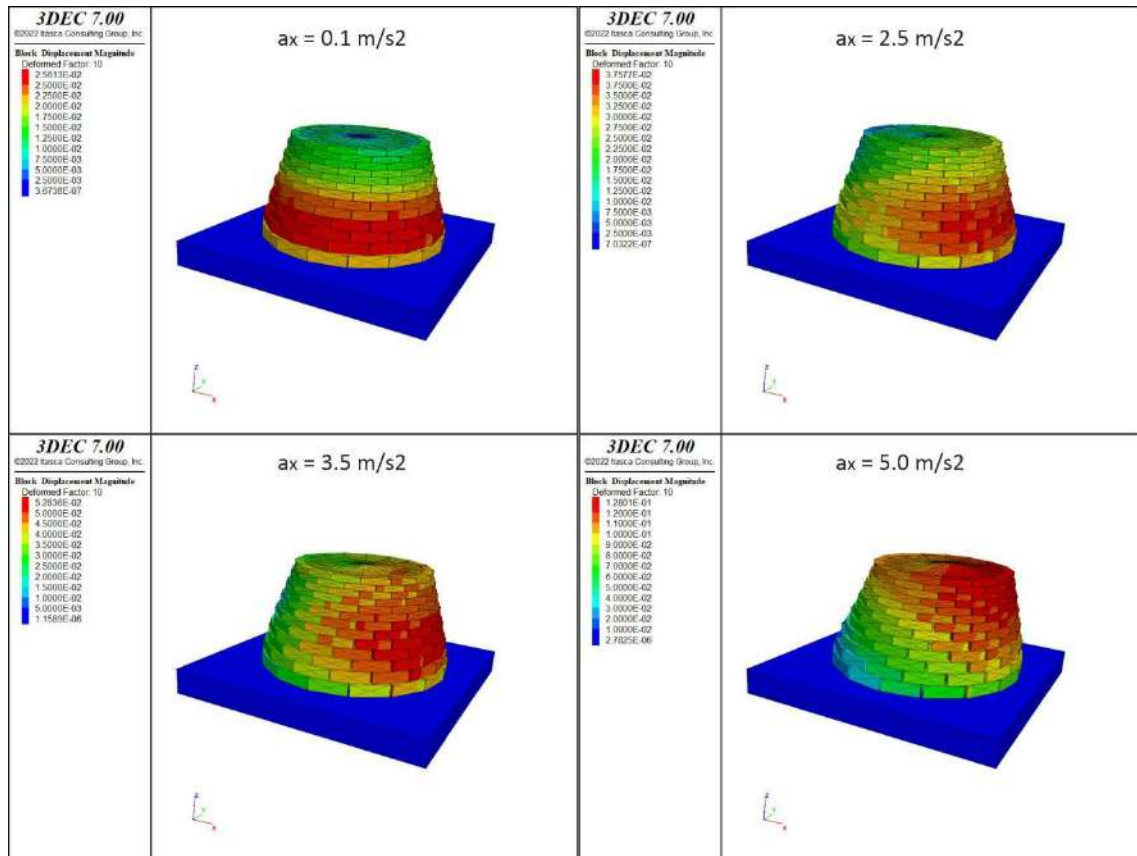


Figure 181. Displacements under varying levels of incremental horizontal loading (magnification factor equal to 10) -Case 3

- Conclusions and Vulnerability Assessment

The quantitative assessment of the geometric vulnerability of the mono-tower Nuraghe with respect to the seismic hazard was carried out using the same methodology described for the ground instability hazard (paragraph 5.3.3 **Errore. L'origine riferimento non è stata trovata.**).

Case 1, in which the filling material is placed within the cavity without significant compaction efforts, is adopted as the baseline for the analysis. For this case, the parameters K and E are assumed to be 0.33 and $5 \times 10^5 \text{ Pa}$, respectively. Subsequently, considering the influence of the filling material's compaction level—

as emphasized in the analysis of Case 3—the variation in vulnerability with respect to this parameter is discussed.

Table 44 reports the upper displacement threshold defined for each damage class, with values derived from the results of the analyses described in the previous paragraph.

Table 44. Upper displacement threshold related to damage levels – Seismic Hazard.

Class	Definition of the damage	Maximum displacement
1	The structure is unaffected by the event and remains in good preservation condition. The blocks do not undergo relative movement, and there are no cracks or mineralogical-petrographic alterations in the constituent material	0 -1 mm
2	The blocks undergo relative displacements that do not compromise the stability of the overall structure. No fracturing occurs, and no chemical or physical alterations are observed in the materials	1 – 10 mm
3	The blocks undergo significant relative displacements that compromise global stability but not the structural geometry (the structure does not collapse). New, non-through cracks appear in the blocks, and superficial mineralogical-petrographic alterations of the materials occur.	10 – 220 mm
4	The blocks undergo major relative displacements that compromise the geometric configuration (the structure collapses). The blocks show new through-going cracks that do not lead to fragmentation. Mineralogical-petrographic alterations are more widespread but do not compromise the blocks' structural integrity	>220 mm

Figure 182 summarizes the results of the analysis carried out for Case 1 and highlights the horizontal acceleration values corresponding to the threshold values of the damage classes.

This figure provides a graphical representation of the geometric vulnerability of the mono-tower Nuraghe with respect to seismic action, expressed through horizontal acceleration.



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The graph shows that damage of Class 1 occurs for horizontal accelerations below 0.25 m/s²; Class 2 damage corresponds to accelerations between 0.25 and 0.50 m/s²; Class 3 damage occurs for accelerations between 0.50 and 2.5 m/s²; finally, Class 4 damage, which leads to structural collapse, occurs for accelerations exceeding 2.5 m/s².

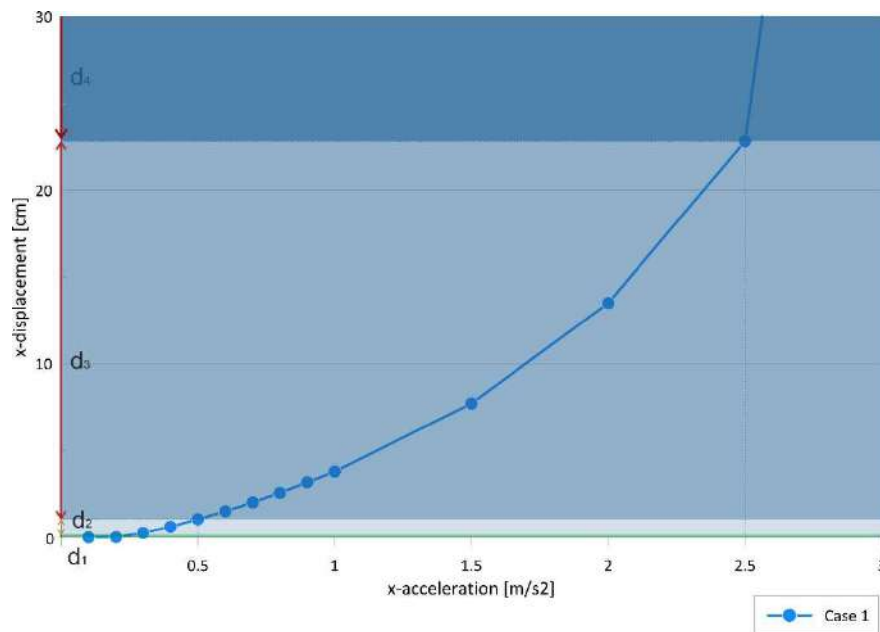


Figure 182. Damage classes defined as a function of displacement threshold levels – Case 1

As highlighted in the previous chapters, the damage sustained by the mono-tower structure following exposure to a seismic event of given intensity depends particularly on the compaction level of the filling material (expressed by the parameters K and E).

The following figure, with fixed displacement thresholds for the damage classes, shows that in Case 3 the structure experiences the indicated damage classes at higher levels of hazard intensity (acceleration). Conversely, for a fixed seismic event intensity (acceleration), the diagram demonstrates that in Case 3 the Nuragic Tower sustains significantly lower damage than in Case 1. For example, the earthquake that causes collapse in Case 1, with an intensity of 2.5 m/s², results in only a level 2 damage in Case 3.

The figure also highlights that the collapse of the structure occurs at displacement values that depend on the compaction degree of the infill material. Therefore, the numerical value of the upper limit of damage Class 3 depends on the specific compaction level.

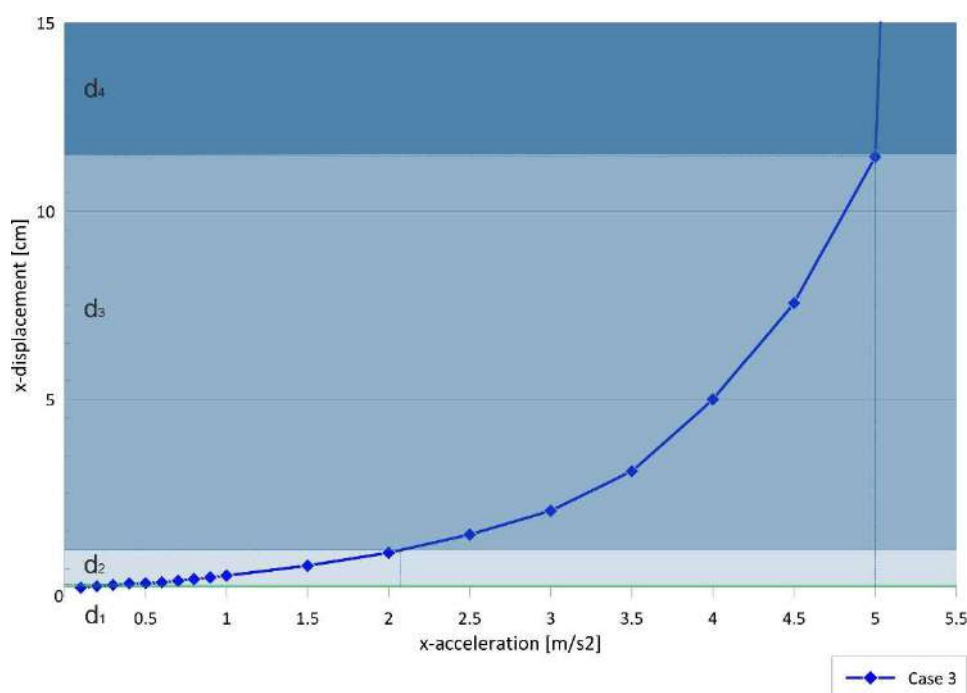


Figure 183. Damage classes defined as a function of displacement threshold levels – Case 3.

5.4.4 FEM Analysis

- *Tilting Analysis*

This study investigates the structural vulnerability of Nuraghe Pontes in relation to seismic hazards, with a focus on both the individual tholos and the entire architectural complex. By analysing these two configurations, the objective is to compare their responses to horizontal forces and to understand how the overall structural system influences collapse mechanisms.



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The primary vulnerability investigated is the destabilisation of the dry-jointed tholos under inclined base conditions. This scenario is simulated by introducing an increasing horizontal component to the gravity vector, allowing the evaluation of stability with respect to horizontal actions and effectively approximating the effects of pseudo-seismic forces. This approach enables a three-dimensional, non-linear analysis of fracture propagation and collapse behaviour in dry-stone masonry. The objective is to determine whether such disturbances trigger failure through the formation of kinematic hinges or through the progressive dislocation of individual blocks. The methodology builds on previous analytical and numerical studies of historical structures subjected to tilting or base distortion [184, 185].

This approach builds on previous structural assessments of corbelled stone forms [186], which demonstrated that tholos structures exhibit efficient load distribution but are particularly sensitive to asymmetric displacement and rotation. To simulate this behaviour, a Degenerate Mohr–Coulomb (DMC) model was adopted as the primary numerical strategy [180]. As previously reported, The DMC model aligns with Heyman’s idealised assumptions for masonry structures: (i) zero tensile strength, (ii) infinite compressive strength (i.e., no crushing), and (iii) no sliding along joints [175]. This modelling approach captures the essential mechanics of dry-jointed masonry while remaining computationally efficient.

In this study, masonry is modelled as a frictional continuum governed by shear behaviour, with a very high level of friction (assuming an angle in the range of $89^\circ < \phi < 90^\circ$) and a very low value of cohesion ($c > 0$ kPa). As previously reported, these properties adapt the mechanical parameters of the Mohr–Coulomb criterion to the no-tension material idealised by Heyman for masonry. The chosen parameters are consistent with findings from previous studies on unreinforced dry-stone constructions — particularly *pinnettas* and *nuraghes* — which employ irregular basalt or volcanic blocks assembled without mortar [187].

In these analyses, both the single tholos and the complete nuraghe are connected to an underlying rigid body via a surface-to-surface interaction defined by a specified contact property. In the normal direction, contact is formulated using the penalty method, which permits limited penetration between surfaces based on a defined contact stiffness, while still ensuring effective enforcement of contact constraints. In

the tangential direction, hard contact behaviour is implemented to prevent separation under compression, and frictional resistance is modelled using a coefficient of friction of 2.0, representing substantial resistance to sliding. The rigid body is associated with a reference point, to which a clamped boundary condition is applied — fully constraining all translational and rotational degrees of freedom — thus effectively fixing the rigid body in space. This configuration enables the realistic transfer of both normal and tangential forces between the deformable structure and the rigid base, allowing accurate simulation of contact pressures, frictional effects, and structural responses under loading.

Structural simulations were carried out using a Dynamic Explicit Analysis framework, with the process divided into sequential steps. The first step, common to all analyses, involved the progressive application of the structure's self-weight. To avoid introducing artificial dynamic effects, gravitational acceleration g was applied smoothly rather than instantaneously, using a time-dependent function to control its amplitude. In the second step, a horizontal multiplier λ was applied, subjecting the structure to a progressively increasing horizontal acceleration profile expressed as $\lambda \cdot g$ (see Figure 184). This horizontal loading was ramped in magnitude over time (see Figure 185). By comparing the response of the isolated tholos with that of the complete Nuraghe Pontes structure, the study offers an insight into how geometric configuration and architectural integration influence the stability and failure mechanisms of historic dry-stone masonry structures.



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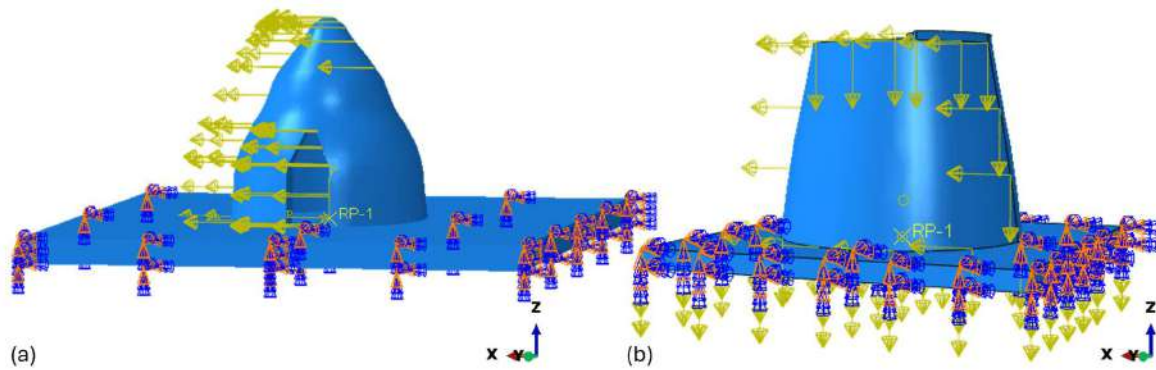


Figure 184. Applied horizontal load distribution and corresponding boundary conditions: the isolated tholos (a), the complete Nuraghe Pontes (b).

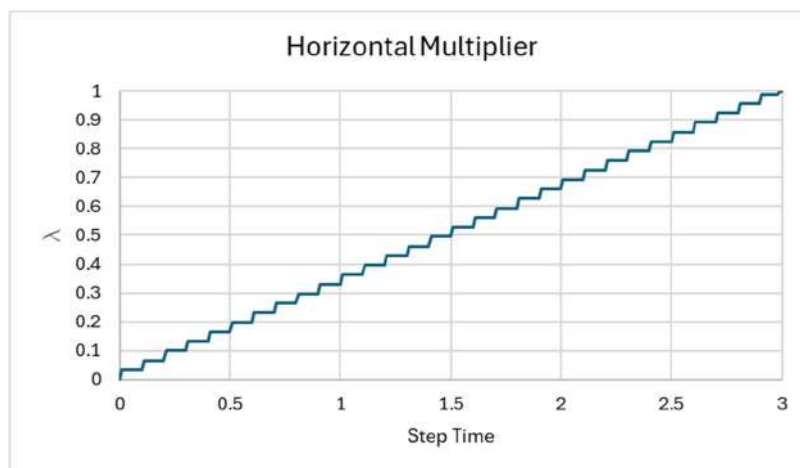


Figure 185. Graph representing the ramped applied values of the horizontal multiplier λ .

- *Tilting analysis of the tholos of nuraghe Pontes*

An examination of the internal energy, total energy, and plastic dissipation graphs (from Figure 186 to Figure 188) indicates that the onset of non-linear behaviour and collapse dynamics occurs shortly before and around $t = 1.5$ s. This moment is marked by a pronounced increase in collapse-related parameters,



including strain, internal variables, and energy. These changes signal the initiation of structural failure and correspond to a horizontal load multiplier of $\lambda = 0.495$, as predicted by the DMC model (see Figure 185).

However, full structural collapse occurs at a higher load multiplier, calculated as $\lambda = 0.495 / 0.75 \approx 0.67$, corresponding to $t = 2$ s. This discrepancy is clearly illustrated in the visualisation module, where the formation of a four-hinge failure mechanism is observed. By analysing the distribution of maximum principal stress—setting the upper bound to zero—tensile stresses appear in grey and compressive stresses in red (see Figure 189). The four hinge locations are distinctly visible in Figure 189, with a green thrust line, drawn by the authors, passing through these points, and illustrating the development of the collapse mechanism.

Discrepancies between the predicted collapse multiplier from the DMC model, the energy-based observations, and the hinge formation seen in the visualisation are characteristic of non-linear regimes. Rigid-body models, including those grounded in limit analysis, are known to overestimate collapse loads, frequently predicting failure at significantly higher thresholds compared to more refined continuum-based numerical simulations. In some cases, such deviations may reach 25–30% [183].

It is also worth noting that the peak ground acceleration (PGA) typically used for seismic design in Sardinia is approximately 0.05 g, reflecting the island's relatively low seismic hazard due to its geodynamic stability compared to other Italian regions [188]. Given that the collapse in this study occurs at an equivalent horizontal acceleration of approximately 0.67 g, the tholos of the Nuraghe Pontes can be considered structurally safe under expected Sardinian seismic conditions.



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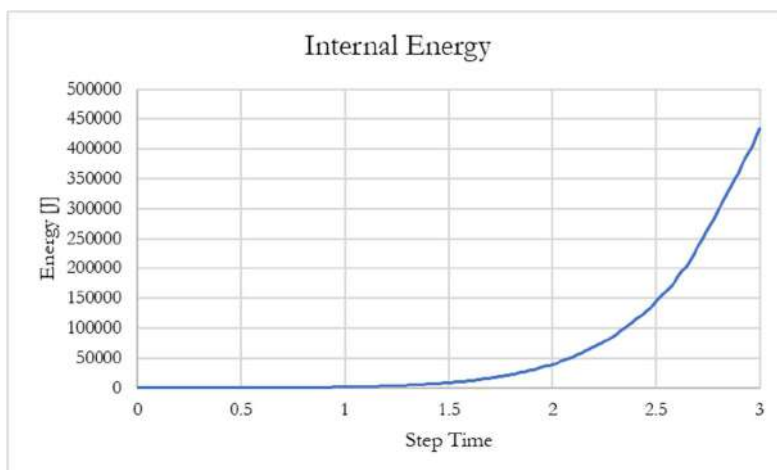


Figure 186. Graph of internal energy evolution during the tilting analysis of the isolated tholos.

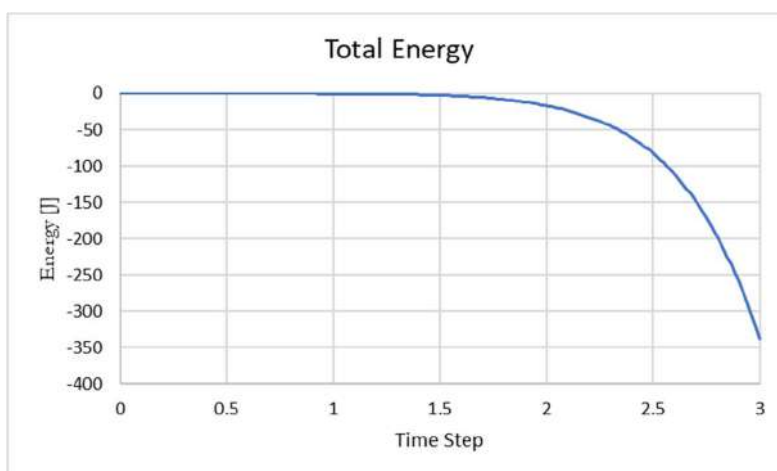


Figure 187. Graph of total energy evolution during the tilting analysis of the isolated tholos.



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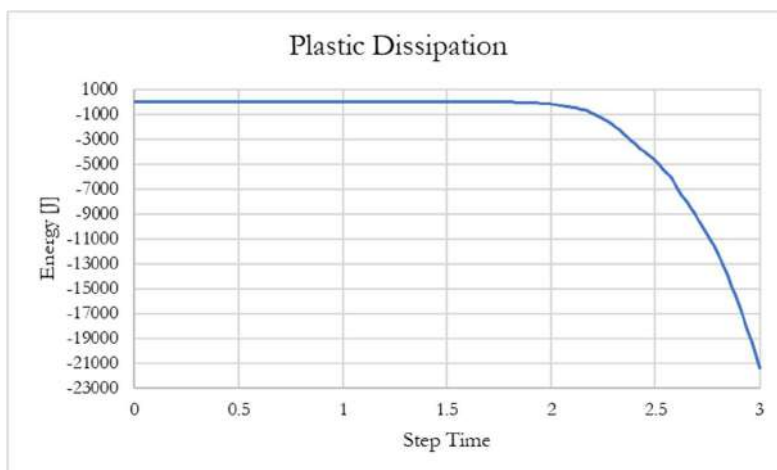


Figure 188. Graph of plastic dissipation evolution during the tilting analysis of isolated tholos.

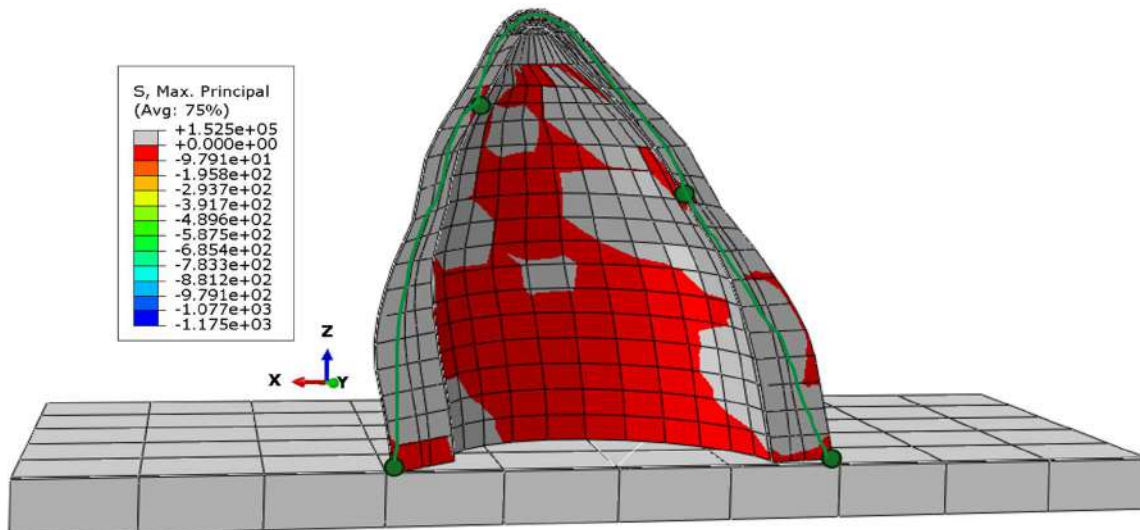


Figure 189. Maximum stress distribution during the tilting of the isolated tholos, with tensile stresses shown in grey and compressive stresses in red. Hinge locations are highlighted with green circles, and the thrust line, traced by the authors, is shown in green.

- *Tilting analysis of the nuraghe Pontes*

An analysis of the graphs depicting internal energy, total energy, and plastic dissipation (from Figure 190 to Figure 192) reveals that the onset of non-linear behaviour and collapse dynamics occurs just before $t = 2$ s. At this point, a marked increase is observed in collapse-related variables such as strain, internal variables, and energy, signalling the initiation of structural failure. According to the DMC model, this corresponds to a horizontal load multiplier of $\lambda = 0.67$ (see Figure 191).

However, full structural collapse occurs at a higher load multiplier, estimated as $\lambda = 0.67 / 0.75 \approx 0.89$, corresponding to $t = 2.5$ s. This is clearly illustrated in the visualisation module, where the development of a failure mechanism is shown in Figure 193. The distribution of maximum principal strain effectively highlights this progression: tensile zones appear in blue, while compressive regions are rendered in warmer tones. A distinct fracture line emerges, traversing the nuraghe laterally, as evidenced by the continuous propagation of plastic strain—highlighted in purple. This observation confirms both the onset and evolution of structural failure, in line with the anticipated formation of a global collapse mechanism.

As previously discussed, discrepancies between collapse predictions from the DMC model, the multipliers identified in energy plots, and the hinge formations observed in visual outputs are typical of analyses within the nonlinear regime [183].

Finally, it is worth emphasising that the peak ground acceleration (PGA) typically used for seismic design in Sardinia is approximately 0.05 g . This relatively low seismic demand reflects the island's geodynamic stability compared to other Italian regions [188]. Since collapse in the present analysis occurs at an equivalent horizontal acceleration of approximately 0.89 g , the structure may be considered robust under Sardinia's expected seismic loading. This significant safety margin underscores the inherent stability of Nuraghe Pontes, despite its dry-jointed construction and lack of modern reinforcement.



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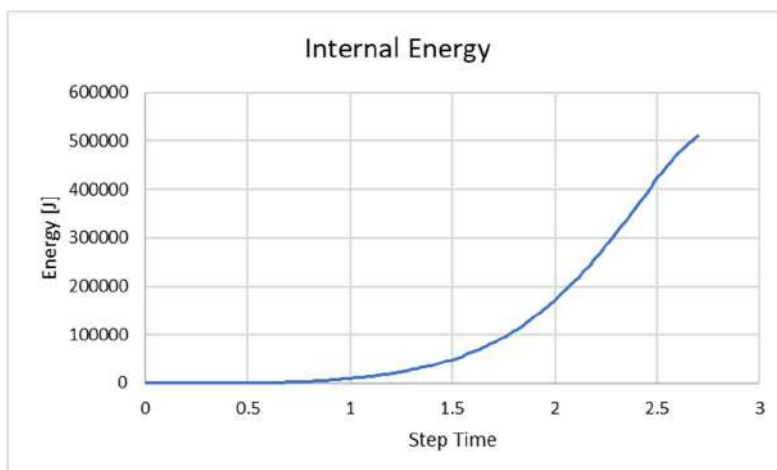


Figure 190. Graph of internal energy evolution during the tilting analysis of the Nuraghe Pontes.

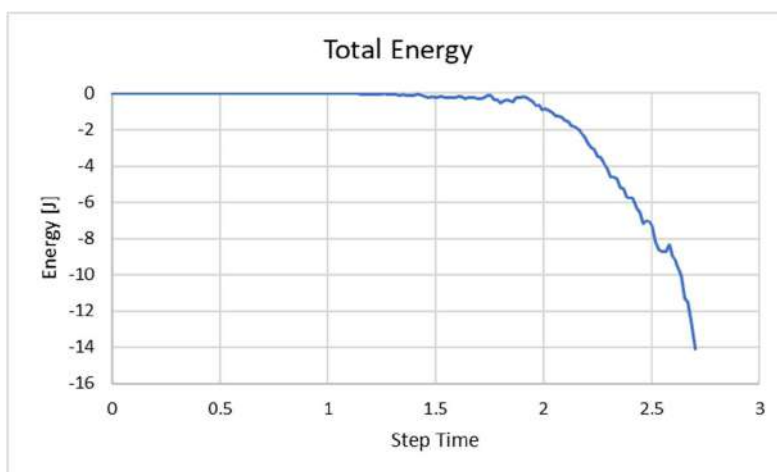


Figure 191. Graph of total energy evolution during the tilting analysis of the Nuraghe Pontes.



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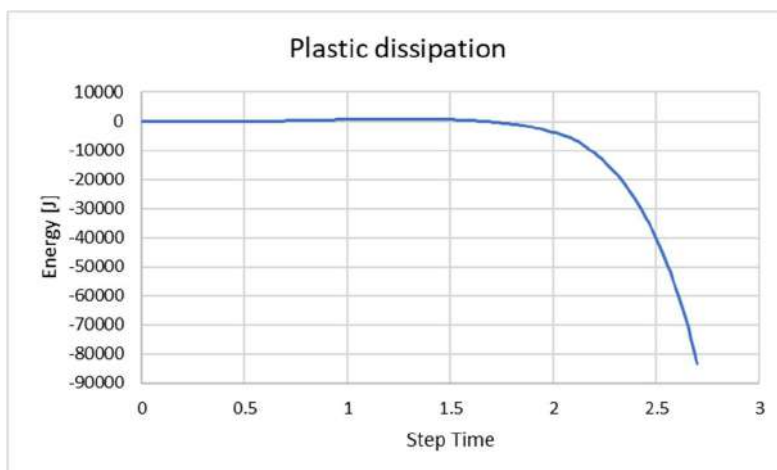


Figure 192. Graph of plastic dissipation evolution during the tilting analysis of the Nuraghe Pontes.

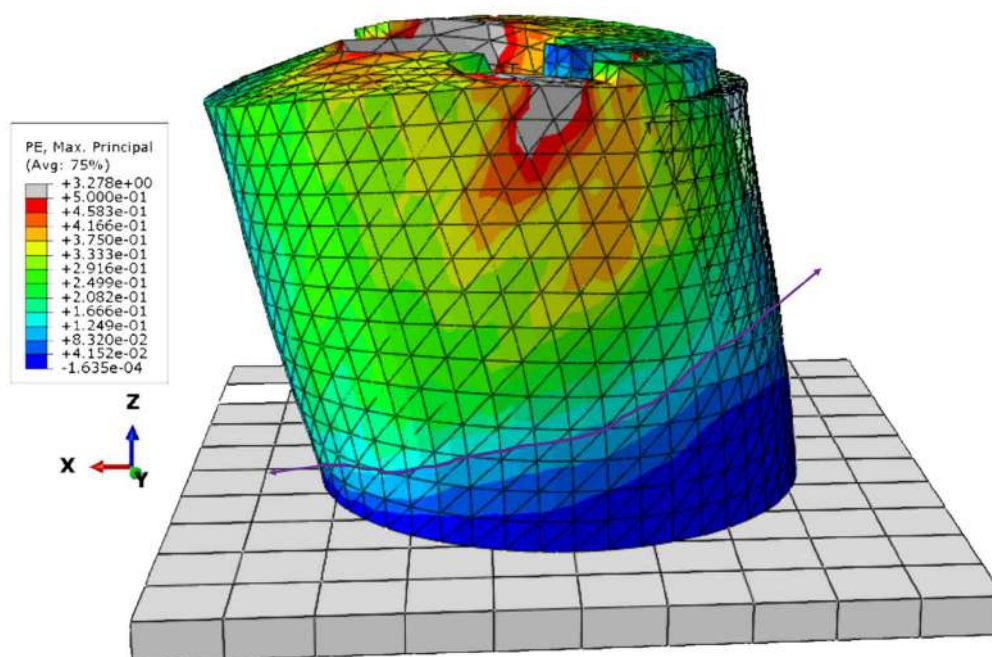


Figure 193. Plastic strain results for the Nuraghe Pontes under tilting analysis for $\lambda \approx 0.89$, corresponding to $t = 2.5$ s



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- *Seismic vulnerability of nuraghe Pontes*

This study investigates the structural behaviour and vulnerability of dry-jointed masonry in Nuraghe Pontes by analysing both the isolated tholos and the complete architectural complex under two destabilising scenarios: tilting induced by inclined support conditions and differential settlement. The numerical framework, based on the DMC model within a dynamic explicit analysis environment, effectively captures the non-linear response and failure mechanisms typical of unreinforced dry-stone construction.

Both the isolated tholos and the complete Nuraghe Pontes exhibit significant sensitivity to asymmetric base disturbances under tilting conditions. In the tholos, collapse occurs through the formation of a classic four-hinge kinematic failure mechanism. In contrast, the full Nuraghe Pontes develops a distinctive diagonal fracture line crossing the structure, indicative of a more complex failure mode influenced by its geometric complexity. The isolated tholos fails at a lower equivalent horizontal multiplier ($\lambda \approx 0.67$ g) compared to the complete nuraghe ($\lambda \approx 0.89$ g), highlighting the stabilising effects of architectural context and structural redundancy. The presence of additional structural elements in the full Nuraghe Pontes configuration delays the onset of instability

Importantly, both configurations only fail under load levels well above Sardinia's typical seismic demand, where the expected peak ground acceleration (PGA) is approximately 0.05 g. This finding underscores the inherent strength of Nuraghe structures and suggests that, despite their lack of mortar or reinforcement, their geometry and traditional construction techniques provide a level of safety well suited to the region's low seismic hazard.

Overall, the comparative analysis confirms that embedding the tholos within a broader architectural framework significantly enhances its structural stability. This improvement stems not only from the additional mass and confinement provided by the surrounding masonry but also from the inherent efficiency of the tholos's corbelled dome construction—a self-supporting system where dry-jointed stones are progressively cantilevered inward to form a false vault. These findings provide valuable insight into the engineering acumen of ancient builders, illustrating how geometry, construction techniques, and spatial context collectively contribute to the resilience of these prehistoric structures. This study offers an



important reference for the seismic assessment and conservation of dry-stone heritage architecture, underscoring that both form and context are critical to the performance and durability of such historic systems.

6 CH monitoring at Landscape scale – The Valle dei Nuraghi

6.1. The proof of concept

6.1.1 Nuragic Architecture and Contemporary Landscape. Risk Analysis and Mitigation in the Valley of the Nuraghi

The protection of architectural heritage and the assessment of the risks to which it is exposed, together with the surrounding landscapes, have long been the subject of interdisciplinary studies [189].

Archaeological assets, exemplary evidence of the transformations that have shaped the territory over time, must be analysed through a relational lens that considers archaeological remains not merely as fragments of a lost whole, but as integral components of a coherent environmental scenography [190] [191].

The team of architectural historians and designers from the DICAAR research unit at the University of Cagliari has identified the Valle dei Nuraghi and the Nuraghe Longu as a case study useful for assessing the vulnerabilities and criticalities that emerge from the analysis of its specific landscape unit. This territorial entity—centred on the monumental presence of Nuraghe Santu Antine, representing the apex of early settlement development—has been identified as a relevant study area for understanding the risks affecting the contemporary Nuragic landscape.

Due to the exceptionally high concentration of historical and proto-historical architectures, the research presents itself as an integrated reflection on risk factors within a broad and coherent territorial framework. The continuous use of the area up to the present day, along with the persistence of environmental invariants that have endured since the Nuragic period, defines a stratified rural landscape rich in meaning, one that may be regarded as representative of many landscape contexts across the Sardinian region.

Indeed, within this vast valley, interpretable as a “monumental region”, numerous expressions of monumentality have succeeded one another throughout history, including cultural phases subsequent to the Nuragic architectures and of significant historical relevance [192] [193].

Such a constellation of monumental elements forms the palimpsest whose vulnerabilities have been assessed in light of the threats brought by contemporary development, especially whenever new road and urban infrastructures are proposed, alongside the creation of new production centres and agricultural or industrial activities—or more recently, large-scale energy production installations associated with unplanned ecological transition programmes.

6.1.2 Historical and Landscape Configuration of the Valley of the Nuraghi

The *Valle dei Nuraghi* constitutes a portion of the *Meilogu*, a historical region in northern Sardinia characterised by an alternation of plateaus, hills, extinct volcanic craters, and valleys [194]. The fortunate combination of a favourable topography and abundant water resources has facilitated human settlement since antiquity, with numerous villages attesting to a territory capable of offering excellent opportunities for settlement thanks to its rich natural resources.

The presence of numerous archaeological features, stratified over time and dating from prehistoric, protohistoric, ancient, and medieval periods, allows this portion of territory to be interpreted as representative of a coherent settlement strategy and as exemplary of a «profound connection between individual monumental elements and the landscape, reflecting a strategy of territorial occupation based more on the exploitation of natural resources than on immediate defensive needs» [193].

Whereas later urban civilisations developed within walled enclosures, the inhabitants of the nuragic phase demonstrated a broad territorial control, erecting architectures still today deeply integrated with the landscape, following a proto-urban settlement scheme [192].

Within the *Valle dei Nuraghi*, the system of monumental architectures gravitating around the hydrographic basin of the *rio Cabu Abbas* represents evidence of continuous territorial occupation across the different historical phases. The toponym *Cabu Abbas*, which in Sardinian literally means “head of the waters”, with the connotation of “origin” or “starting point”, refers to an area of notable naturalistic and historical interest, characterised by eponymous natural features, a spring, a watercourse and a plain, so as architectural ones, a *nuraghe*, a Romanesque church and other significant elements.

Cabu Abbas's Spring, among the most important water resources in the *Meilogu* region, ensured the area's water supply for millennia, enabling the development of rural settlements and sustaining agriculture and livestock farming through works of water control and channelling. Today, the spring, partly intercepted, still feeds some local aqueducts. The spatial proximity between the spring, the *nuraghe Cabu Abbas*, and other nuragic and pre-nuragic monuments — including *nuraghe Longu*, located between the *rio Cabu Abbas* and the *rio Pudidu* — suggests that nuragic communities were acutely aware of the strategic importance of the water resource.

The closeness of these elements demonstrates a deliberate settlement choice in which the presence of water played a key role in territorial organisation and in the construction of nuragic structures, representing a strategic, sophisticated, and effective model capable of sustaining itself and guiding transformations throughout ancient and medieval times.

The continuity in signs and toponyms, evident in historical documentations, persists into modern times and suggests a symbiosis between architectural monumentalities and the water resource. Another exemplary case is the late-medieval rural church of *Spirito Santo*, which stands adjacent to the eponymous *nuraghe*, and forms one of the nodes of a network of rural paths, evolved over time and linking the constellation of minor *nuraghi*.

Traces and signs of medieval occupation are also present — according to preliminary surveys by the *Soprintendenza*, currently under study — in the areas facing the *nuraghe Santu Antine*, to the east, where recent discoveries have revealed a Byzantine necropolis.

From an infrastructural point of view, the area of *Cabu Abbas* holds a position of strategic importance within the historical landscape of the *Valle dei Nuraghi*, lying in proximity to the ancient Roman road axis that connected two of the island's major centres: *Turris Libisonis* (Porto Torres) and *Karales* (Cagliari). This was one of the main transit corridors in ancient Sardinia, vital for the movement of goods, people, and information between the two coastal poles.



Supporting this centrality, several Roman milestones (*miliaria*) dating to the 1st and 3rd centuries AD have been found near the site, confirming the proximity to the Roman road and suggesting that *Cabu Abbas* may already have been a significant transit node during the Imperial age.

According to historical reconstructions and field surveys, the route of the ancient Roman road appears to largely coincide with that of the old *Strada Reale Carlo Felice*, designed in 1822, along a path that crosses the areas of *Prunaiola*, *Cabu Abbas*, and *Murighenti* [195] [196].

Evidence of this importance can also be found in the design drawings for the *Strada Reale*, a major infrastructural project linking the north and south of the island [197] [198].

In addition to layering a new stratum upon part of the ancient Roman road's trajectory, the project drawings include, in the segment studied here, a specific annotation referencing the *Chiesa di Nostra Signora di Cabu Abbas*, thereby recognising it as a known landscape landmark.

6.1.3 Recent Infrastructural Criticalities

Considering the continuity described thus far, we will examine the extent to which recent infrastructural transformations—primarily related to road networks—have altered the existing configurations, hindering the interpretation and recognition of that very web of routes and architectural markers that is closely interwoven with the natural morphology consolidated over centuries.

This condition is further complicated by the fact that the management of the portion of the valley under examination is currently shared among several municipalities—including the towns of Torralba, Giave, Cheremule, Borutta, Bonorva, and Bonnanaro—whose lack of consistent coordination has hindered the development of a shared, long-term planning strategy capable of approaching the territory as a unified historical-landscape system.

The erosion of this system lies at the heart of the critical issues currently affecting the entire valley and adds to the specific vulnerabilities of each individual heritage asset.



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These criticalities will be further detailed in the following section, which aims to demonstrate how individual risk prevention and mitigation strategies—tailored to the specific characteristics of each site—must be informed by a broader understanding of systemic issues, to devise effective and context-sensitive measures.

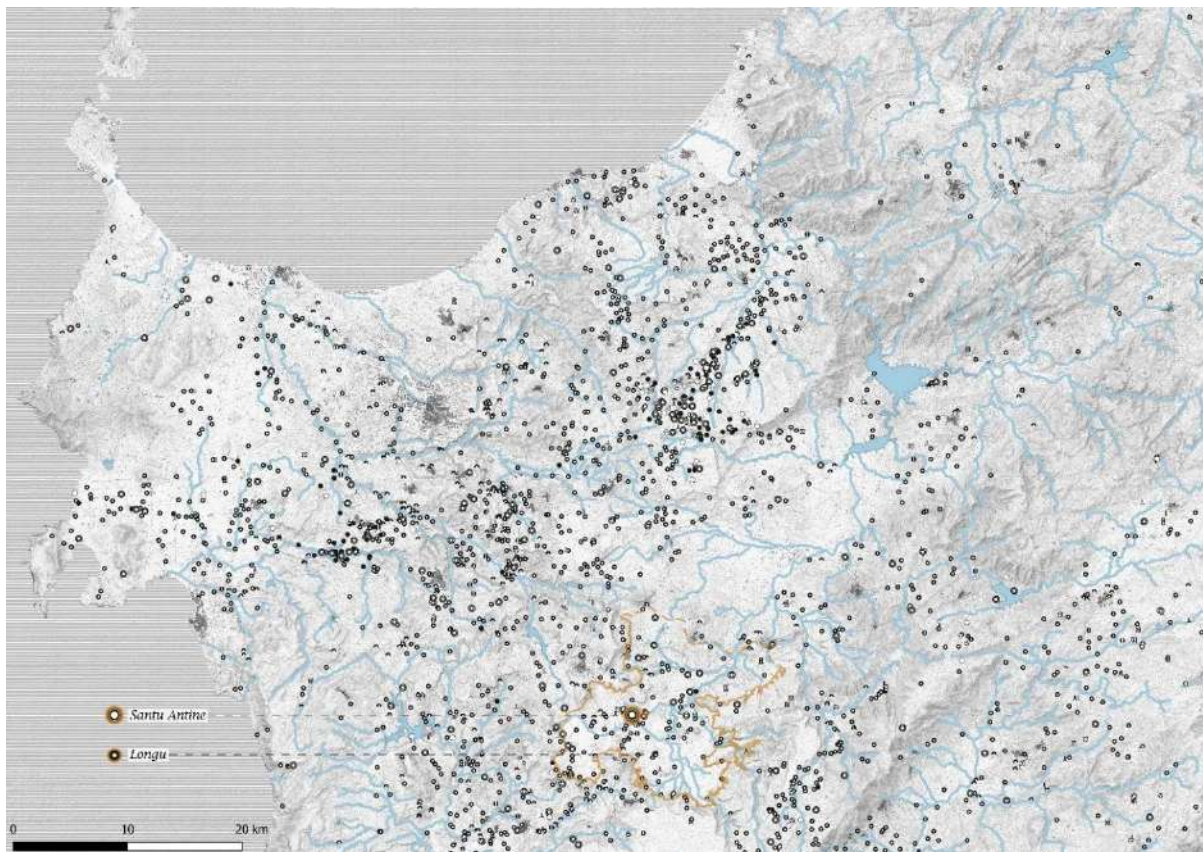


Figure 194. General overview of cultural heritage in northwestern Sardinia. The main monuments are indicated based on GIS Sardegna Archeologica sources (graphic processing by Silvia Orione, in CADINU, ORIONE, 2024).



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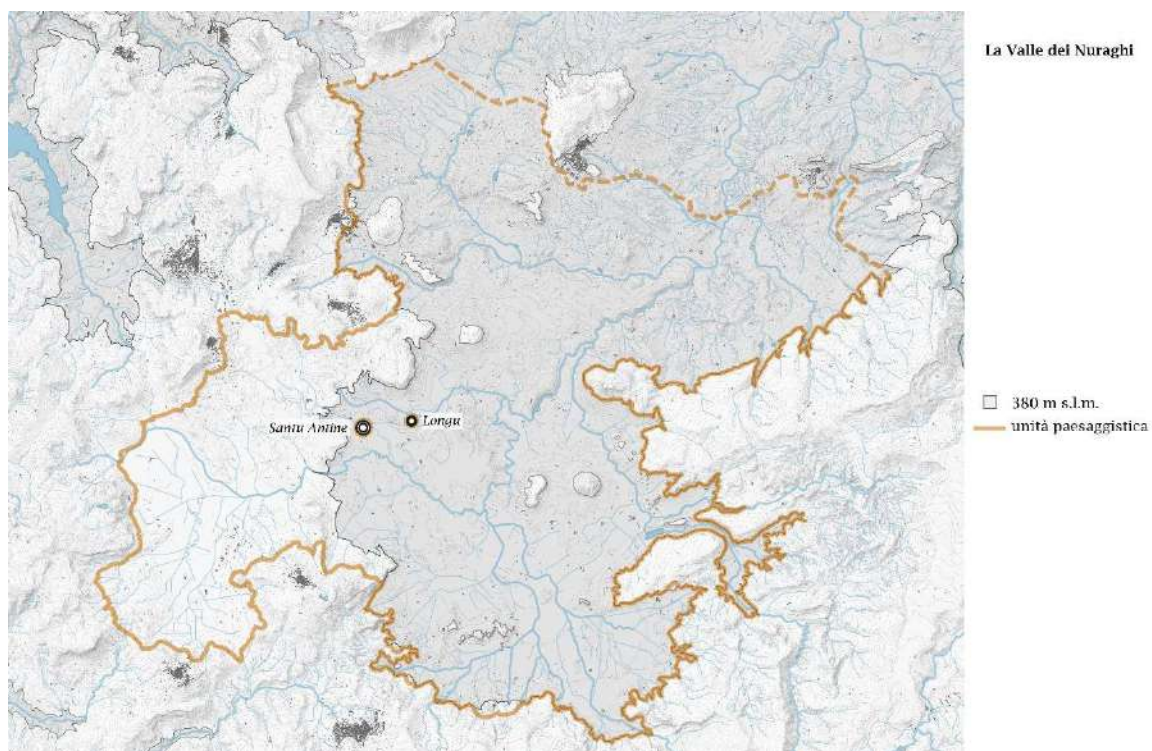


Figure 195. Landscape unit of the valley floor in relation to the Santu Antine nuraghe complex and the Longu single-tower nuraghe, encompassing the Valle dei Nuraghi within an extended perimeter. The continuation of the valley towards the northern sectors is limited here by the dotted line adjacent to the towns of Mores and Ittireddu (graphic processing by Silvia Orione 2025).



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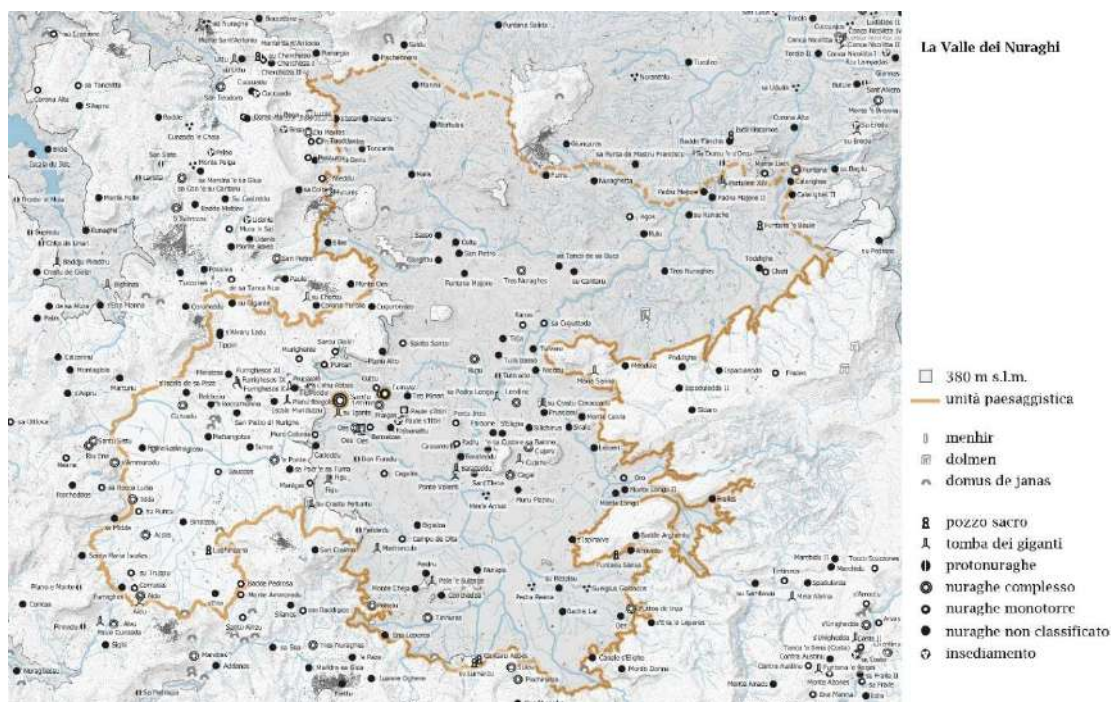


Figure 196. Map of archaeological heritage in the Valle dei Nuraghi (graphic elaboration by Silvia Orione 2025).



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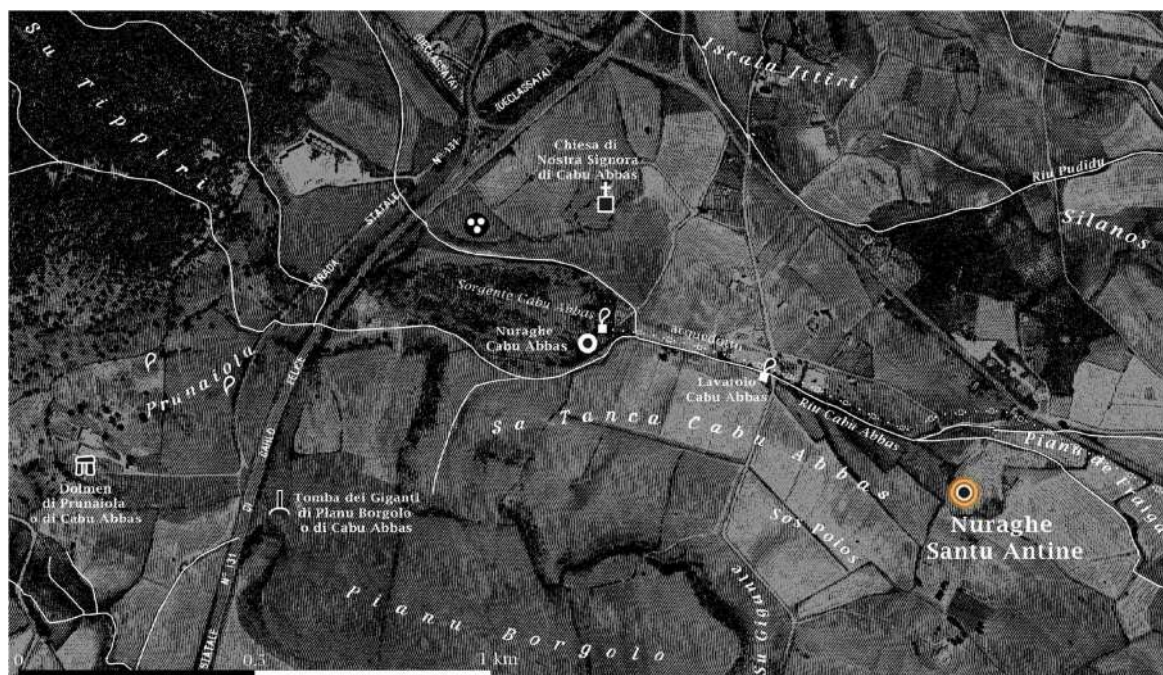


Figure 197. Territorial area between Santu Antine and Cabu Abbas. Valley floor landscape unit related to the Nuraghe of Santu Antine (UNESCO candidate) encompassing the Valle dei Nuraghi within a large perimeter. The toponyms shown are taken from the IGM cartography (graphic processing by Silvia Orione, in CADINU, ORIONE, 2024).



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Figure 198. Portion of the Valle dei Nuraghi around Nuraghe Longu (graphic elaboration from Google Earth by Silvia Orione 2025).



Figure 199. In the foreground, Nuraghe Longu and the portion of Valle in its immediate vicinity seen from the west (photo by Silvia Orione).



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Figure 200. Nuraghe Longu within its context (photo by Alessandro Meloni).



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*Figure 201. Relations between Nuraghe Longu and the landscape. In the background, a large agricultural building can be seen
(photo by Alessandro Meloni).*



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Figure 202. Part of the Valle dei Nuraghi near Nuraghe Longu (photo by Alessandro Meloni).



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Figure 203. View of the Nuraghi Valley from the top of Nuraghe Longu (photo by Silvia Orione).



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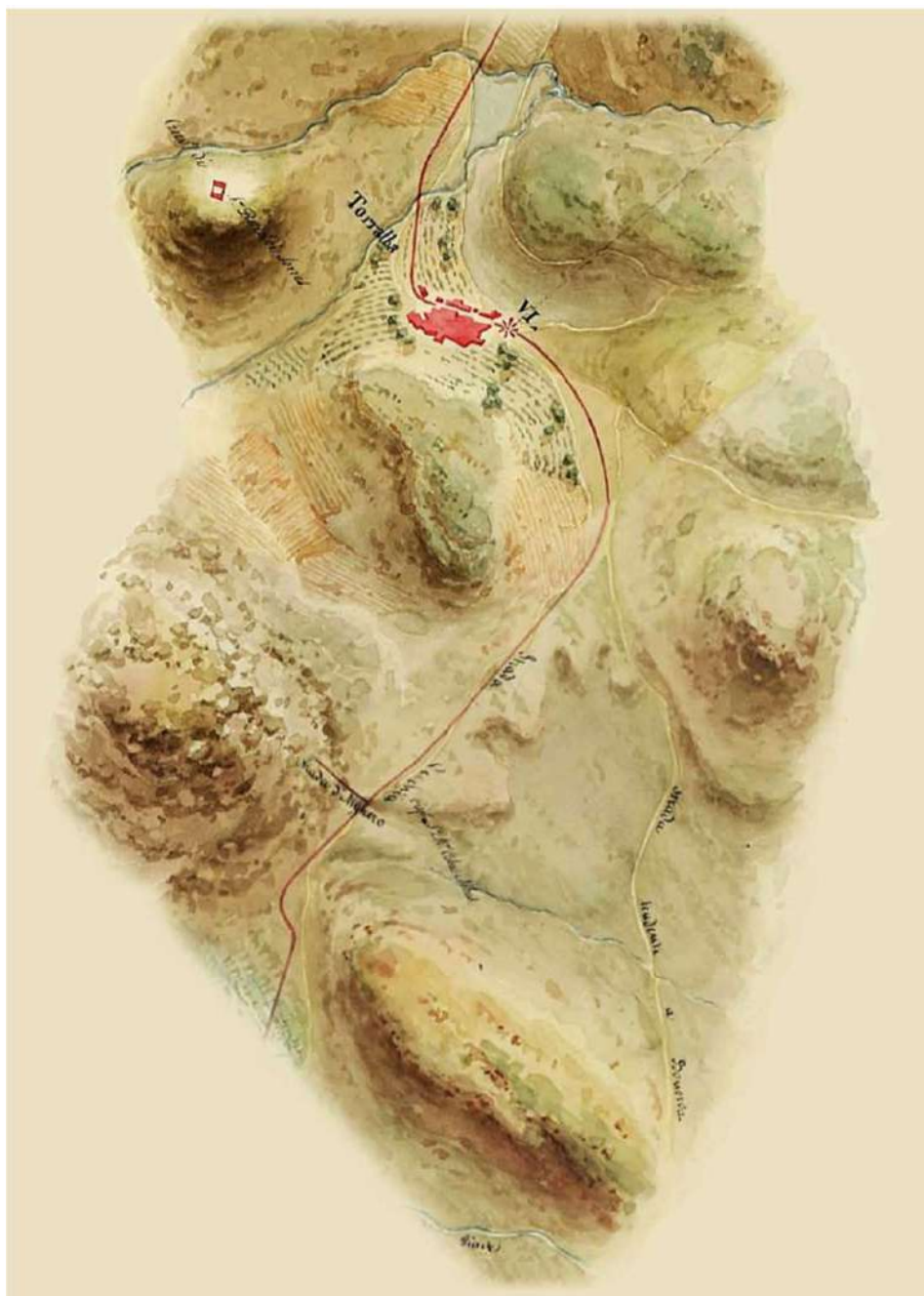


Figure 204. Detail taken from the drawings for the construction of the Strada Reale (Royal Road) in 1822 (graphic processing by Silvia Orione, in CADINU, ORIONE, 2024).



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6.1.4 Vulnerability of the Nuragic Landscape

Starting from the historical analysis of the landscape unit of the Valle dei Nuraghi, the research has primarily examined the topographical, infrastructural, and settlement-related characteristics of the area, followed by the consequent natural and anthropogenic risk and pressure variables. The study then progressed with the collection and analysis of further data, with the aim of identifying risk factors.

Specifically, the following categories of risk have emerged and are analysed here: infrastructural risk, hydrogeological risk, fire risk, and other vulnerabilities.

Considering the categories identified, it should be noted that fire risk appears to be substantially consistent with the natural morphology of the area and, therefore, does not require investigation. By contrast, regarding infrastructure and hydrogeological risks, a significant difficulty has emerged in aligning the quantitative terms set out by planning instruments with the actual impact of interventions on the landscape.

In the first case, it is worth highlighting the intersections between historical settlement patterns and the abrupt acceleration of transformation processes affecting the *Valle dei Nuraghi* since the post-war period. An assessment of this hazard cannot be derived directly from available data but was instead conducted by superimposing infrastructure layouts from various historical periods onto the current configuration. This allowed for a verification of how certain distinctive features have not been preserved and, indeed, have been compromised.

In the second case, there is a notable absence of specific data concerning the Rio Cabu Abbas system. The water regulation processes that have altered the river's flow have also diminished its functional and perceptual significance within the landscape.

Based on both regional and local planning instruments, several critical issues have nevertheless been identified—some of which are linked to the high number of tributaries flowing into the river. This is evidenced by the high Strahler number assigned to the section between Santu Antine and the Nuraghe Longu currently under examination.

Furthermore, in terms of risk perception, the construction of production and tourist accommodation facilities near the riverbed is indicative. These issues, related to land consumption, constitute key elements to be assessed for a proper evaluation of risks, especially considering the prospective economic and touristic development of the area.

In this regard, it also appeared appropriate to frame the vulnerability assessment of specific monuments as exemplary of the broader contextual issues observed. As the archaeologist Andrea Carandini recently noted «Contexts withstand hair-splitting analysis only when followed immediately by powerful synthesis; what they cannot withstand [...] is the plundering of what stands out to the detriment of what serves as background and setting, because in them everything is interconnected» [199].

Within the RETURN project, in which the approach is markedly interdisciplinary, such a suggestion may prove valuable for achieving the necessary synthesis between diverse fields of knowledge, aimed at defining more effective strategies for the protection and enhancement of cultural heritage.

6.1.5 Infrastructural risk

The *Valle dei Nuraghi* reaches us today in a form that differs from its *nuragic* landscape dimension, particularly as a result of infrastructural developments that have disrupted the vast landscape unity and its original interrelations.

The most significant infrastructural transformations concern the 19th-century construction of the railway and the *Strada Reale Cagliari – Porto Torres* (Figure 204), later renamed *Strada Statale Carlo Felice 131*. In the 20th century, the road was expanded and replaced by a modern high-speed roadway. The introduction of the railway and the development of secondary roads have also contributed—albeit to a lesser extent—to the fragmentation of the *Valle's* systemic unity.

One example of such profound transformation is the construction of *Strada Provinciale 21*, clearly visible in the aerial photograph in Figure 208. By linking the route of the former *Strada Reale* with rural mobility networks, the SP 21 passes within just a few dozen metres of *Santu Antine*, interrupting the continuity of



ancient routes and separating the great monument from the minor *nuraghi* that encircle it, including the single-tower *nuraghe Longu*, located 1.20 km away in a straight line.

The rupture in the relationship between the monument and its immediate context, brought about by infrastructural development, has constituted a largely unacknowledged threat, revealing a pronounced vulnerability in the system of paths and rural connections.

A similar reading applies to the transformation resulting from the expansion and replacement of the old *Strada Statale 131* with the new high-speed road (Figure 210). Its construction has caused the most evident rupture between the system of hills adjacent to the primary forest of *Tippiri* and the *Valle dei Nuraghi*, physically separating the historical routes that once connected the valley's various sectors. This intervention has generated an artificial geographical configuration, one already historically fragmented by the presence of multiple municipal boundaries.

The extent of the disruption caused by infrastructure within the system is particularly evident when studying visual cones. Figure 211 and Figure 212 present simulations from the *nuraghi Longu* and *Santu Antine*. In the case of *Santu Antine* (Figure 213 and Figure 214), the 131 constitutes a significantly impactful element, not only as a linear rupture in the plan of existing paths.

The intervention—characterised by the construction of massive embankments and overpasses—has permanently altered the soil topography, introducing a new type of obstacle that is also perceptible in three dimensions. As such, it represents a substantial breach in the visual relationships between the valley and the surrounding hills.



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Figure 205. Location of Santu Antine and Cabu Abbas areas in the maps drawn up by the Royal Corps of General Staff, 1851. In relation to the rural road connecting Torralba and Bonorva, the sign represented by the Strada Reale (Royal Road) stands out.



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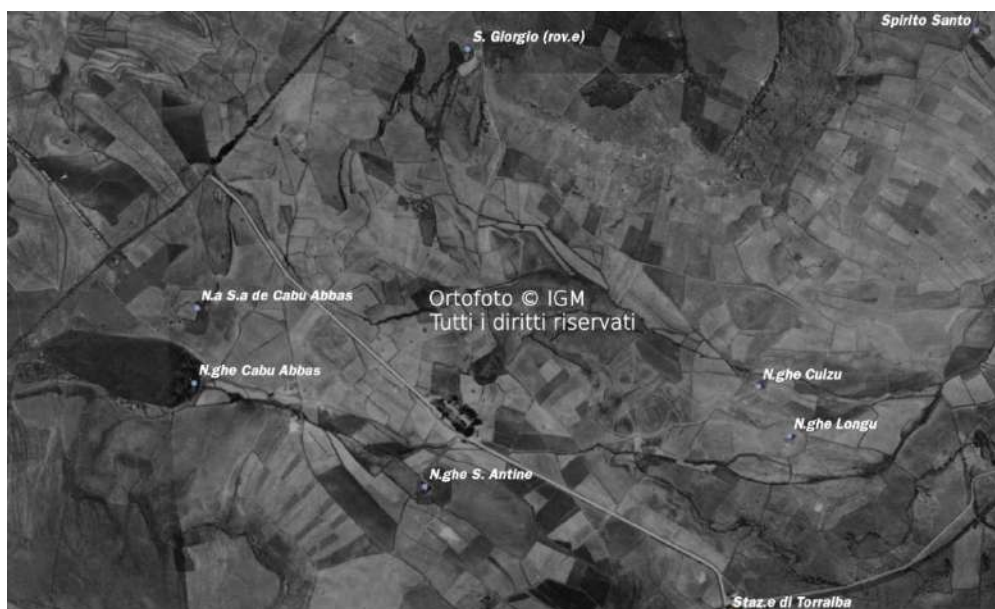


Figure 206. Aerial photo of the Santu Antine and Cabu Abbas area, 1954-1955. The new SP 21, connecting the former Strada Reale and the secondary roads, alters the existing road and river continuity by passing close to the areas in front of Santu Antine.



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Figure 207. Aerial photo of the Santu Antine and Cabu Abbas area, 2022. The new SS 131 and the system of junctions and connections that characterize it stand out for their size.



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Figure 208. Aerial photo of the current state clearly showing the disruption caused by the SS 131 highway between Su Tippi Primary Forest system and the portion of the valley beyond the road (graphic elaboration from Google Earth by Silvia Orione 2025).



Figure 209. Su Tippi Primary Forest system, view from Cheremule (photo of Silvia Orione).



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Figure 210. In the foreground, Su Tippi Primary Forest seen from Cheremule towards the Valle dei Nuraghi. The Nuraghe Santu Antine is highlighted. It should be noted how the SS 131 road interrupts the vegetation with a sharp break, which then resumes further ahead at the foot of the hills in the background (photo by Silvia Orione).



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Figure 211. Simulation of visual cones with a viewpoint set at human height (1.60 m) near the Nuraghe Longu. The obstructed portion of the view is shown in dark.



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Figure 212. Enlargement of Figure 211 with the highlighting of the caesura represented by 131.



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Figure 213. Simulation of visual cones with a viewpoint set at human height (1.60 m) near the Nuraghe Santu Antine. The obstructed portion of the view is shown in dark.



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Figure 214. Enlargement of Figure 213 with the highlighting of the caesura represented by 131.

6.1.6 Hydraulic risk

A close relationship has emerged between the hydrographic basin of the *rio Cabu Abbas* and the *nuraghe* constructions located downstream from the spring, and interpretative hypotheses have been advanced regarding the relationship between *nuragic* settlement and water, the dominant element of the valley. Given the imposing nature and technical precision of the *nuraghe* of *Santu Antine*, it is plausible to suppose that it stood at the centre of a deliberately designed landscape intervention, within the wide channel—or riverbed—originating from the spring and possibly associated with an artificial basin. This basin is hypothesised not only as a water reserve system but also as the focal point of a particularly evocative setting, within a deforested area dominated by high-quality agricultural cultivation. The resulting context is one in which the large catchment basin of the valley was once traversed by water resources far more abundant than those currently available. From a historical perspective, it may be



assumed that, following the decline of *nuragic* civilisation, the loss of control over the hydrographic network led to the flooding and waterlogging of part of the plain, thereby reducing agricultural potential and favouring its transformation into a predominantly wooded area [192].

The position of the *nuraghe Longu* (Figure 217) is also noteworthy, located approximately equidistant between the *rio Pudidu* to the north and the *rio Cabu Abbas* to the south. Once again, this underscores the evident relational link between *nuragic* constructions and water resources on the plain. The diversion of the natural river branches originating from *Cabu Abbas*, aimed at supplying modern aqueducts, has had significant repercussions on the hydrographic configuration of the valley. The redistribution of water has led to a reduction in the flow and, consequently, a progressive loss of meaning of one of the valley's primary resources—alongside the *rio Mannu*—which underpins the entire territorial system.

The manipulation of water distribution therefore raises a crucial issue regarding hydraulic risk, linked to the emerging landscape configurations. These effects are currently being observed, also in light of the hydrogeological risk assessments commissioned for some time by the municipality of *Torralba*, which are intended to integrate with regional planning strategies. Specifically, documents from local urban planning show that various building processes—entailing land consumption—have taken place within 150 metres of the *rio Cabu Abbas* (Figure 216).

This situation, made possible by the progressive reduction of the river's flow, may still constitute a condition of risk, especially in view of potential tourist development in the areas facing *Santu Antine*. Additional land consumption processes in direct proximity to the river, even when intended to strengthen economic activity, must consider the hazards posed by an imprudent relationship with the water system. As illustrated in documentation provided by the Region regarding watercourse flow (classified according to the Strahler method in Figure 218), the *rio Cabu Abbas*—particularly in the section from *Santu Antine* toward the northern quadrant of the valley—has been assigned a value of “5”. This figure implies a high number of tributaries and indicates the potential to receive significant volumes of water, with possible flooding phenomena, especially in the event of extreme weather conditions. In light of these considerations, the water system emerges as a critical risk factor in the contemporary



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transformation processes affecting the landscape of the *Valle dei Nuraghi*. In this particular territory, safeguarding the heritage embodied in a historically and geographically consolidated configuration appears inseparable from a renewed, contemporary reflection on the relationship with local hydrography. In this sense, the study of archaic settlement strategies seems to offer an effective reference point for a conscious and adaptive relationship with water resources.



Figure 215. In the center in blue, Rio Cabu Abbas in the Maps of the Real Corpo di Stato Maggiore, 1854.

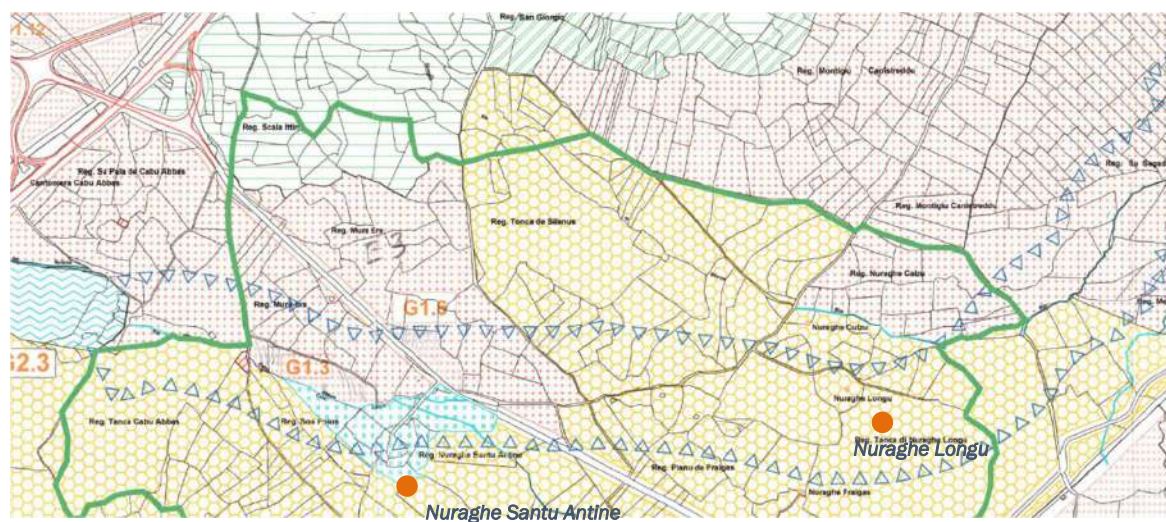


Figure 216. Buffer area of the 150 m distance from the Rio Cabu Abbas, excerpt from the extra-urban zoning table, Municipality of Torralba.



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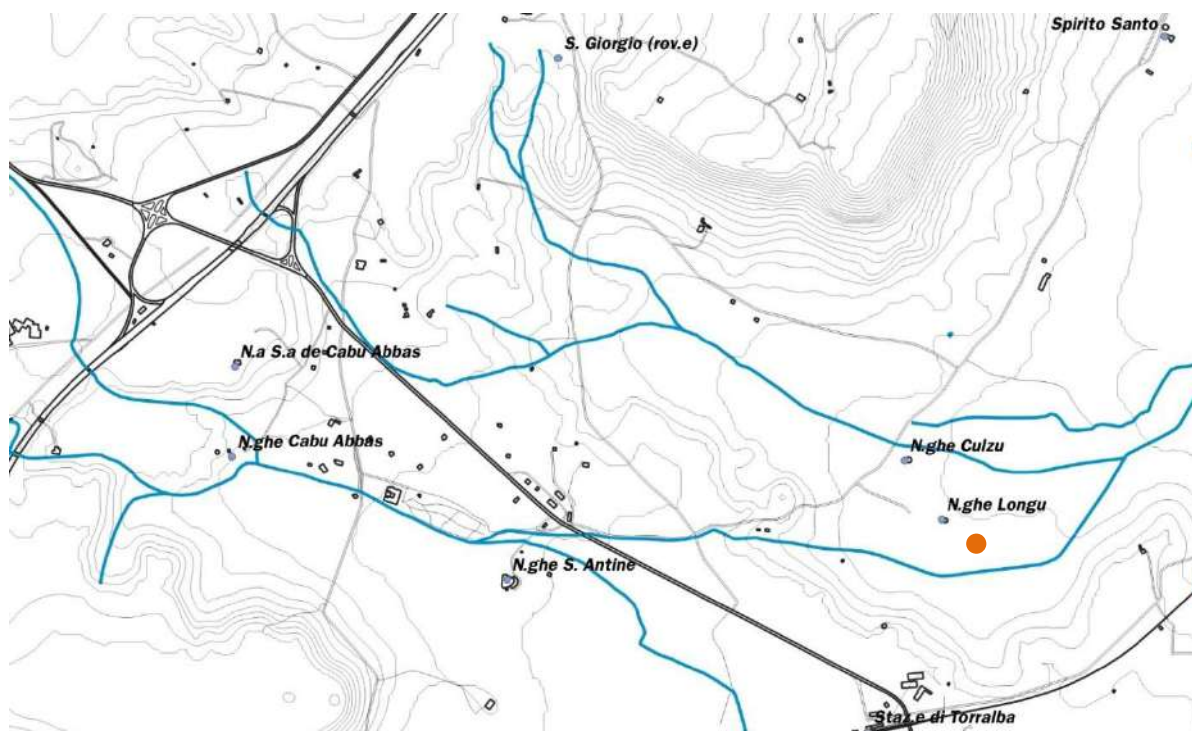


Figure 217. Current state of the Cabu Abbas River basin. The Nuraghe Longu stands out as a median and control point between the two waterways.



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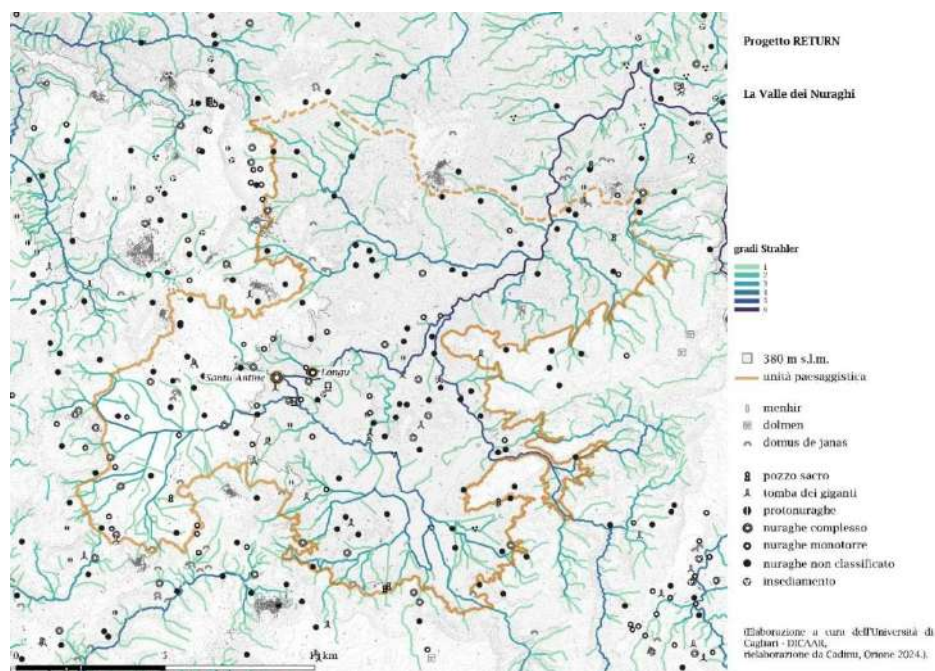


Figure 218. Norme PAI, Art. 30ter, Hierarchy of the hydrographic network of the Valle dei Nuraghi classified according to the Strahler numbering (graphic elaboration by Silvia Orione, 2025).



Figure 219. The vegetation along the Riu Cabu Abbas waterline seen from the top of the Nuraghe Santu Antine. In the center on the ridge is the church of Nostra Signora di Cabu Abbas (photo by Marco Cadinu).



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Figure 220. View of the Valle dei Nuraghi, hydrographies (photo of Marco Cadinu).



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Figure 221. In the foreground, the Cabu Abbas spring cabin and Nuraghe Cabu Abbas in the background (photo by Marco Cadinu).



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Figure 222. In the foreground, water outcrops in front of the perimeter wall of the ancient mining concession of the Cabu Abbas spring cabin (photo by Marco Cadinu).



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Figure 223. Typical water outcrops in the Valle dei Nuraghi (photo by Marco Cadinu).

6.1.7 Wildfire risk

Within the framework of regional planning, fire risk has been assessed and thematised through the *Sardegna Geoportale* database. This planning effort reveals that the study area presents a significant degree of variability in terms of exposure to fire hazard. Fire risk increases considerably in proximity to the primary forest of *Su Tippi*, due to obvious factors related to denser vegetation. *Su Tippi* extends across approximately 320 hectares in a north-west to south-east direction, from *Monte Cuccuruddu*, near the outskirts of *Cheremule*, to the territory of *Torralba*, and is separated from the *Cabu Abbas* area by the S.S. 131.

The large scale and landscape prominence of the forest call for careful fire risk assessment, in line with recommendations issued by regional planning and protection instruments.

In this context, it is notable that the visual and physical discontinuity created by the SS 131 acts as a barrier to the propagation of wildfires downhill (Figure 224).

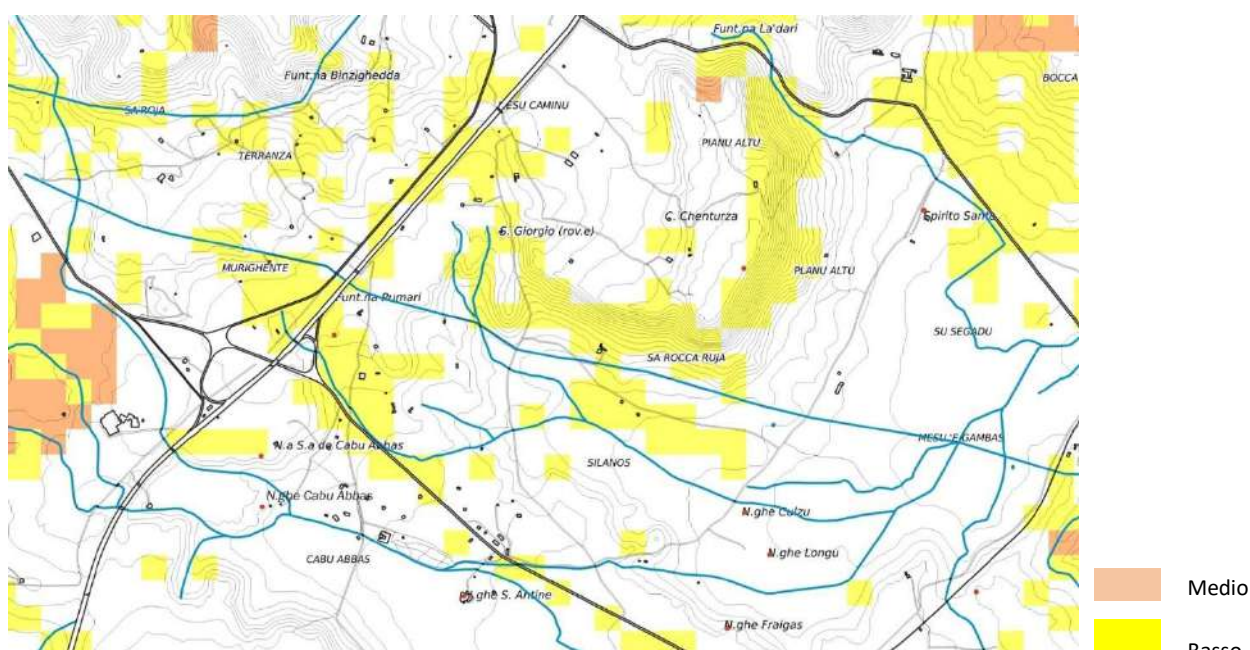


Figure 224. Forest fire risk map, graphically elaborated from the forest fire risk and danger maps of the Sardinia Region.

6.1.8 Other vulnerabilities

- Hazard from “over-tourism”

The direct interaction of visitors with the architectural heritage can accelerate the natural deterioration process to varying degrees (general wear, structural damage, damage to wall surfaces, alteration of humidity levels within the structures, etc.). The increase in tourist flows can particularly affect monumental assets and the surrounding ecosystems, potentially compromising their integrity.



Overcrowding at an archaeological site constitutes a physical risk to the artefacts as well as a potential loss of the site's authenticity and aesthetic value.

Among the risks posed by tourism is the possibility of disrespectful behaviour, a risk that proportionally increases with visitor numbers. Excessive tourism also entails another risk related more to the asset's image, namely its commodification: in the absence of adequate service facilities and flow regulation, so-called overtourism could alter the relationship between landscape scenarios and visitors, potentially resulting in a loss of attractiveness of the asset and the entire system.

The recent significant growth in visitor numbers (exceeding 50,000 annually), and the anticipated strong increase following the hoped-for inclusion of the site on the UNESCO lists, require precise organisational decisions, currently at the core of programmes being developed by DICAAR under commission from the Sardinia Region.¹

¹ The aforementioned candidacy is promoted by APS– La Sardegna verso l'UNESCO, supported by several organizations, and has been officially included on the Tentative List since November 15, 2021. The working group of DICAAR (Department of Civil, Environmental, and Architecture of the University of Cagliari) includes numerous collaborators who are making a crucial contribution to the landscape and infrastructure review of the territories of the 32 candidate Nuragic sites. Marco Cadinu initiated the consultancy process in 2020, joined by Carlo Atzeni and Pier Francesco Cherchi since 2022. The ongoing general planning program, aimed at the infrastructural reorganization and revitalization of the cultural landscapes surrounding the candidate monuments, has been funded by the Sardinia Region since 2024, which tasked the group, coordinated by Marco Cadinu and Carlo Atzeni, with producing a comprehensive vision for the full integration of the 32 monuments and the relevant landscape units, with due regard for the necessary infrastructural adjustments and the standards required by UNESCO protocols. The members of the working group are: Ivan Blečić, Michele Agus, Chiara Cabras, Stefano Cadoni, Marco Lecis, Stefano Mais, Andrea Margagliotti, Francesco Marras, Alessandro Meloni, Silvia Mocci, Sara Montis, Marco Moro, Silvia Orione, Gavina Piras, Manuela Piras, Fabrizio Pusceddu, Benedetta Stefania Rubattu, Maria Carla Saliu, Carla Sechi.



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Figure 225. Nuraghe Longu, relationships between landscape and human scale (photo by Alessandro Meloni).

- *Landslide risk*

Within the planning documents of the Municipality of Torralba, certain areas are identified as characterised by geomorphological hazard. In the area under study, for instance, the Romanesque church of Nostra Signora di Cabu Abbas, closely connected to the hydrological system of the homonymous watercourse, falls within a zone of high hazard (Hg3), while the intersection between the 131bis and S.S. 131 road infrastructures presents zones with an even higher hazard level (Hg4).

The Church of Nostra Signora di Cabu Abbas occupies a position of strong landscape significance, situated on a prominent relief overlooking the northern slope of the Valle dei Nuraghi, facing the Nuraghe Santu Antine. However, this very topographical location exposes it to a high geomorphological hazard.

The term "geomorphological hazard" refers to the probability of occurrence of natural events, such as landslides or erosion, capable of compromising the stability of the soil and any structures built upon it. In the case of Cabu Abbas, the slope on which the church stands shows signs of slope instability due to morphological, geological, and anthropogenic factors, including: natural disintegration of the outcropping rocks; absence or thinning of vegetation cover, which reduces surface cohesion; soil erosion during periods of heavy rainfall; and modifications to the local hydrological regime, potentially linked to agricultural or infrastructural interventions. These conditions increase the vulnerability of the church but also the possibility that instability phenomena may compromise the landscape and archaeological legibility of the slope in which it is located, thus altering the visual and symbolic balance with the nearby Nuraghe Santu Antine.

For this reason, the church constitutes a sensitive node within the monumental network of the Valle dei Nuraghi, which it overlooks. Therefore, its protection concerns not only the material conservation of the building but also the mitigation of geomorphological risks that threaten its relationship with the landscape and cultural context in which it is situated.

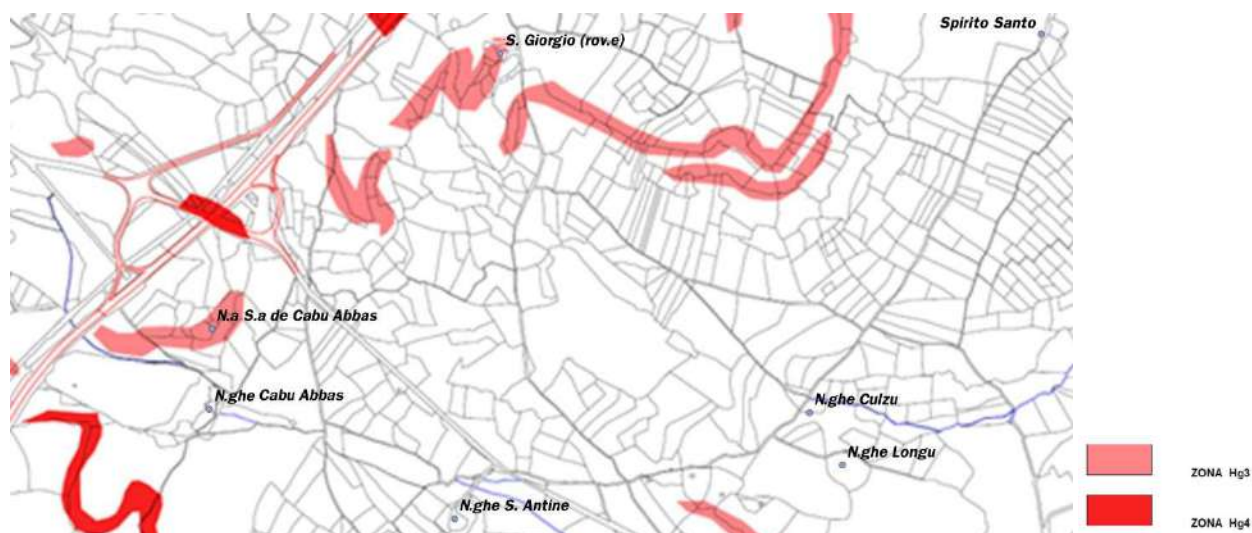


Figure 226. Except from the geomorphological hazard map of the Municipality of Torralba.



Figure 227. Relationship between the hill on which the Chiesa di Nostra Signora di Cabu Abbas is located and the Valle dei Nuraghi below (photo by Silvia Orione).



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Figure 228. The Nuraghi Valley seen from Cabu Abbas hill. The Nuraghe Santu Antine is highlighted (photo by Silvia Orione).

- *Risk of Deterioration and Loss of Heritage*

In the observation of Nuraghe Longu, it emerges that another nuragic monumental site is located in its immediate proximity: Nuraghe Culzu. The close distance between the two structures necessitates a reflection on their relationship, although any physical traces of this connection have, to date, been compromised.

Nuraghe Culzu, situated near Nuraghe Longu, represents a paradigmatic example of a cultural asset at risk of silent disappearance. Despite its strategic position within the nuragic landscape of the valley, the site is in an advanced state of abandonment. Overgrowing vegetation compromises its legibility, while material alterations—such as the removal of summit stones, reportedly used for the construction of dry stone walls in nearby fields, likely during the implementation of the *Editto delle Chiudende* in Sardinia (19th century)—have undermined its structural integrity.



From a risk assessment perspective, Nuraghe Culzu, in its exemplary condition representative of similar nuraghi, falls into three interconnected categories: risk of material degradation due to neglect and the erosive action of atmospheric and biological agents; perceptual risk, stemming from the increasing difficulty in recognising the structure as part of a coherent monumental system; and cultural and relational risk, due to the loss of visual, historical, and symbolic connection with nearby Nuraghe Longu and the wider archaeological network of the valley.

Its current state of conservation is such that, to an untrained eye, the site might appear merely as a pile of stones rather than as a significant node in the nuragic network. This compromises not only its intrinsic value but also the overall reading of the archaic landscape, founded on visual, spatial, and symbolic relationships between monumental sites. The lack of recognisability and minimal protective interventions feeds a risk of cultural invisibility, making an urgent mitigation strategy necessary—one that includes light but effective actions such as selective vegetation clearance, securing the site, signage, and integration into the territorial narrative routes.

In the case of Culzu, therefore, its current condition threatens the loss of the significance of its figure and facilitates its slow fading within the landscape, gradually resembling more and more a minor topographic mark.



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Figure 229. In the foreground, Nuraghe Longu. In the background, on the left, Nuraghe Culzu (photo by Silvia Orione).



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Figure 230. In the center, in the distance, Nuraghe Longu in its context. On the left, Nuraghe Culzu is covered by vegetation, making it difficult to read (photo by Alessandro Meloni).



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Figure 231. The current state of Nuraghe Culzu (photo by Silvia Orione).

6.1.9 Risk Mitigation Perspectives and Landscape Rebalancing

The current condition of the *Valle dei Nuraghi* stands as a paradigmatic case of Sardinia's archaic landscape in its contemporary configuration.

It is a landscape composed of multiple elements—agricultural land, churches, nuraghi, watercourses—which, though heterogeneous in nature, are held together by an invisible network of relationships. It is precisely this subtle, often hidden but essential network that confers identity, coherence, and quality upon the territory.

This relational network, refined and stratified over time, constitutes the true value of the historic landscape: a network composed of monuments, ancient infrastructures, pathways, and territorial hierarchies sedimented across the centuries. Today, our responsibility is twofold: on the one hand, to

safeguard the materiality of the monuments; on the other, to recognise and protect the intangible connections that unite them and make them part of a coherent system.

Within this framework, the Nuraghe Longu emerges as one of the many components of the net that shapes the so-called *Valle dei Nuraghi*: a territory dotted with dozens of protohistoric monuments, coeval expressions, and remnants of both earlier and later epochs. Over the course of history, the valley has also accommodated Roman infrastructures, Byzantine cemeteries (currently under investigation), medieval settlements, and traces of continuity of use, all of which attest to the pervasiveness and resilience of this monumental landscape.

It is this extensive and complex geographical and historical network that constitutes the field of investigation of the DICAAR research group within the RETURN project. A field which today is increasingly threatened by interventions that, if left unregulated, risk compromising the coherence of the archaic and, more broadly, of the historic landscape. As a result, all new infrastructural processes—such as the construction of roads, roundabouts, wind and photovoltaic power plants, antennas, and large productive buildings for agriculture and livestock—must be included within a framework of evaluation that places the integrity of the landscape system at its core².

New infrastructures, often driven by production logics that regard the territory primarily as a resource to be exploited, generate widespread risks that do not merely affect individual monuments but the entire network of relationships that ensures their legibility and historical significance.

For this reason, it is essential not to limit action to the material conservation of built heritage, but to preserve—especially in light of the risk assessments discussed above—the relational functioning of the territory: a subtle, ancient, and stratified system that represents one of the highest forms of continuity between past and present. Failure to protect this invisible mesh would, in fact, amount to creating new forms of risk for the contemporary historic landscape.

² The Sardinian Landscape Regional Plan, dated 2006, contributes to the management of the singular criticalities



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DI RIPRESA E RESILIENZA

Return

7 Bibliography

- [1] Direzione Ambiente e di Scienze della Terra, “Studi di Microzonazione Sismica del territorio comunale di Firenze,” Firenze, 2021.
- [2] Italian Department of Civil Protection, Guidelines for Seismic Microzonation (ICMS). Working Group on Seismic Microzonation, Presidency of the Council of Ministers, Rome, Italy, 2008.
- [3] ABAQUS, Theory and user’s manuals 2018, Pawtucket (RI, USA): Hibbit, Karlson and Sorensen, 2018.
- [4] S. Sgobba, R. Puglia, F. Pacor, L. Luzi, . E. Russo, C. Felicetta, G. Lanzano, M. D’Amico, R. Baraschino, G. Baltzopoulos and I. Iervolino, “REXELweb: a tool for selection of ground-motion records from the Engineering Strong Motion database (ESM),” in *7th International Conference on Earthquake Geotechnical Engineering (ICEGE) 17- 20 June 2019, Rome, Italy*, 2019.
- [5] J. Lubliner, J. Oliver, S. Oller and E. Oñate, “A plastic-damage model for concrete.,” *Int J Solids Struct*, 1989.
- [6] S. Degli Abbati, A. D’altri, D. Ottonelli, G. Castellazzi, S. Cattari, S. De Miranda and S. Lagomarsino, “Seismic assessment of interacting structural units in complex historic masonry constructions by nonlinear static analyses,” *Computers & Structures*, 2018.
- [7] M. Valente and G. Milani, “Seismic assessment of historical masonry towers by means of simplified approaches and standard FEM,” *Construction and Building Materials*, no. 108, pp. 74-104, 2016.
- [8] MIT2019: Circolare 21 gennaio 2019, n.7 Istruzioni per l’applicazione dell’<<Aggiornamento delle “Norme tecniche per le costruzioni”>> di cui al decreto ministeriale 17 gennaio 2018. G.U. n. 47 del 26/02/2009, Supplemento Ordinario n. 27, Roma, 2019.

- [9] V. Alecci, M. De Stefano, R. Luciano, A. Marra and G. Stipo, “Numerical Investigation on the Use of Flat-Jack Test for Detecting Masonry Deformability,” *Journal of Testing and Evaluation*, vol. 49, no. 1, 2021.
- [10] NTC, Aggiornamento delle “Norme tecniche per le costruzioni”. S.O. alla G.U. n. 42 del 20/02/2018. (in Italian), 2018.
- [11] A. Cabboi, C. Gentile and A. Saisi, “From continuous vibration monitoring to FEM-based damage assessment: Application on a stone-masonry tower,” *Constr. Build. Mater.*, vol. 156, p. 252–265, 2017.
- [12] M. Ponte, R. Bento and S. D. Vaz, “A Multi-Disciplinary Approach to the Seismic Assessment of the National Palace of Sintra,” *International Journal of Architectural Heritage*, pp. 1-22, 2019.
- [13] M. Dell’Oste, A. Gubana, A. Morassi and E. Puntel, “Dynamic Testing in Support of the Seismic Assessment of a Century Old Masonry Building Complex.,” *Buildings*, 2022.
- [14] G. Di Lorenzo, A. Formisano, L. Krstevska and R. Landolfo, “Ambient vibration test and numerical investigation on the St. Giuliano church in Poggio Pienze (L’aquila, Italy).,” *J Civil Struct Health Monit*, vol. 9, pp. 477-490, 2019.
- [15] A. M. Marra, V. Alecci, D. Pugliese and M. De Stefano, “Full-scale dynamic measurement system for FE-model updating of masonry building aggregates,” Vancouver, Canadian, 2022.
- [16] P. Fajfar, “Capacity spectrum method based on inelastic demand spectra,” *Earthquake Engineering and Structural Dynamic*, 28, pp. 979-93, 1999.
- [17] G. M. Calvi, R. Pinho, G. Magenes, J. J. Bommer, L. F. Restrepo-Vélez and H. Crowley, “Development of seismic vulnerability assessment methodologies over the past 30 years,” *ISET J. Earthq. Tech.*, vol. 43, pp. 75-104, 2006.



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DI RIPRESA E RESILIENZA



- [18] F. & NIBS, Multi-hazard loss estimation methodology: earthquake model, Washington: HAZUS-MH MR4, Technical manual, Federal Emergency Management, Mitigation Division, 2003.
- [19] C.-D. 212/2013, *Istruzioni per la valutazione affidabilistica della sicurezza sismica di edifici esistenti*, ROMA - CNR 14 Maggio 2014, 2014.
- [20] G. B. M. F. L. M. A. M. S. Bartoli, "Bayesian model updating of historic masonry tower through dynamic experimental data," in *Proceedings of the X International Conference on Structural Dynamics (EURODYN 2017)*, Rome, September 10-13, 2017.
- [21] G. Galloway, G. Seminara, G. Blochl, M. Garcia, A. Montanari and L. Solari, "Reducing the Flood Risk of Art Cities: The Case of Florence," *Journal of Hydraulic Engineering*, vol. 146, no. 5, pp. 1-7, 2020.
- [22] MIT, Istruzioni per l'applicazione dell'Aggiornamento delle Norme tecniche per le costruzioni di cui al decreto ministeriale 17 gennaio 2018 (in Italian), 2019.
- [23] C. del progetto: C. Regione Toscana, "Microzonazione Sismica Relazione tecnica Regione Toscana Comune di Pienza (SI)," 2020.
- [24] S. Shahbazi, A. Barra, Q. Gao and M. Crosetto, "Detection of buildings with potential damage using differential deformation maps," *ISPRS Journal of Photogrammetry and Remote Sensing*, vol. 2018, p. 57–69, 2024.
- [25] A. Barra, C. Reyes-Carmona, G. Herrera, J. Galve, L. Solari, R. Mateos, J. Azañón, M. Béjar-Pizarro, J. López-Vinielles, R. Palamà, M. Crosetto, R. Sarro, J. Cuervas-Mons and O. Monserrat, "From satellite interferometry displacements to potential damage maps: A tool for risk reduction and urban planning," *Remote Sensing of Environment*, vol. 282, p. 113294, 2022.



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e della Ricerca



Italiadomani
PIANO NAZIONALE
DI RIPRESA E RESILIENZA



- [26] S. Bianchini, L. Solari and N. Casagli, "A GIS-Based Procedure for Landslide Intensity Evaluation and Specific Risk Analysis Supported by Persistent Scatterers Interferometry (PSI)," *Remote Sensing*, vol. 9, no. 11, p. 1093, 2017.
- [27] G. Grünthal, "European Macroseismic Scale 1998," Centre Européen de Géodynamique et de Séismologie, Walferdange, Luxembourg, 1998.
- [28] S. Giovinazzi and S. Lagomarsino, "A macroseismic model for the vulnerability assessment of buildings," in *13th World Conference on Earthquake Engineering*, Vancouver, Canada, 2004.
- [29] S. Lagomarsino, S. Cattari and D. Ottonelli, "The heuristic vulnerability model: fragility curves for masonry buildings," *Bull Earthquake Eng*, vol. 19, pp. 3129-3163, 2021.
- [30] V. Cardinali, E. Di Rienzo, M. Tanganelli and M. De Stefano, "Compound-based approach for large scale seismic vulnerability assessment: application to the Garfagnana area in Tuscany," *Bull Earthquake Eng*, vol. 23, p. 1969–1997, 2025.
- [31] J. B. Burland and C. P. Wroth, "Settlement of Buildings and Associated Damage," Cambridge, England,, 1974.
- [32] J. B. Burland, "Assessment of risk of damage to buildings due to tunnelling and excavation," *Earthquake Geotech. Eng*, vol. 3, p. 1189–1201, 1997.
- [33] M. Son and E. Cording, "Evaluation of building stiffness for building re-sponse analysis to excavation-induced ground movements," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 133, pp. 995-1002, 2007.
- [34] M. Son and E. J. Cording, "Estimation of Building Damage Due to Excavation-Induced Ground Movements," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 131, p. 162–177, 2005.



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Italiadomani
PIANO NAZIONALE
DI RIPRESA E RESILIENZA

Return

- [35] S. Lagomarsino, A. Penna, A. Galasco and S. Cattari , “TREMURI program: An equivalent frame model for the nonlinear seismic analysis of masonry buildings,” *Engineering Structures*, 56, pp. 1787-1799, 2013.
- [36] A. Penna, S. Lagomarsino and A. Galasco , “A nonlinear macroelement model for the seismic analysis of masonry buildings,” *Earthquake Engineering and Structural Dynamics*, 43(2), pp. 159-179, 2014.
- [37] G. Lilliu, *La Civiltà Nuragica.*, Sassari, Italy: Carlo Delfino Editore., 1982.
- [38] G. Lilliu, *La civiltà dei Sardi dal Paleolitico all’età dei nuraghi*, Torino: Nuova ERI, 1988.
- [39] G. Ugas, “La Sardegna nuragica. Aspetti generali,” in *La Sardegna nuragica. Storia e materiali. Corpora delle antichità della Sardegna*, Sassari, Carlo Delfino Editore, 2014, pp. 11-34.
- [40] A. Depalmas, “I Nuraghi. Le torri dell’isola,” in *L’Isola delle Torri*, Sassari, Carlo Delfino Editore, 2015, pp. 76-83.
- [41] A. Moravetti, “Appunti sulla civiltà nuragica,” in *L’Isola delle Torri. Giovanni Lilliu e la Sardegna Nuragica*, Sassari, Carlo Delfino Editore, 2015, pp. 37-57.
- [42] A. Usai, “Paesaggi nuragici,” in *L’Isola delle Torri*, Sassari, Carlo Delfino Editore, 2015, pp. 58-69.
- [43] G. Ugas, “Strutture insediative e seminterrate e ipogeismo sepolcrale nella Sardegna preistorica,” in *Ipogeismo nel Mediterraneo: origini, sviluppo, quadri culturali, atti del congresso internazionale (Sassari – Oristano, 23-28 maggio 1994)*, Muros, Stampacolor: 888-908., 2000.
- [44] G. Salis, “Bronzi di pregio,” in *Corpora delle antichità della Sardegna, La Sardegna Nuragica. Storia e materiali*, Sassari, Carlo Delfino Editore, 2015, pp. 331-346.
- [45] T. Cossu, M. Perra and A. Usai, , *Il tempo dei Nuraghi, la Sardegna dal XVIII al VIII secolo a.C.*, Sassari: Carlo Delfino Editore , 2018.

- [46] E. Contu, "L'architettura Nuragica," in *Ichnussa, la Sardegna dalle origini all'età classica*, Milano, Credito Italiano, 1981, pp. 5-175.
- [47] E. Contu, *La Sardegna preistorica e nuragica: La Sardegna dei nuraghi.*, Sassari, Italy: Arti Grafiche Editoriali "Chiarella", 1997.
- [48] R. Cicilloni, *Le domus de janus con corridoio d'ingresso di tipo "megalitico" o "dolmenico"*, Tanda G. (ed.), 2015.
- [49] G. Ugas, *L'Alba dei Nuraghi*, Cagliari: Fabula, 2005.
- [50] P. Melis, "I Nuraghi," in *Corpora delle antichità della Sardegna. La Sardegna Nuragica, Storia e Monumenti, Catalogo dei siti*, Sassari, Carlo Delfino Editore, 2017, pp. 29-53.
- [51] G. Lilliu, *La civiltà nuragica*, Sassari: Carlo Delfino Editore, 1982.
- [52] R. Cicilloni, F. Porcedda, M. Cabras, L. Spanedda and J. Camara Serrano, "Primeros resultados del análisis de la presencia humana durante la Edad del Bronce en el Sarcidano (Sudeste de Cerdeña, Italia)," *Bollettino di Archeologia online*, vol. VIII, no. 1-2, pp. 1-3, 2018.
- [53] J. Camara Serrano and L. Spanedda, "L'organizzazione sociale nuragica. Note e ipotesi," in *Corpora delle antichità della Sardegna. La Sardegna Nuragica, Storia e Monumenti*, Sassari, Carlo Delfino Editore, 2015, pp. 150-159.
- [54] G. Paglietti, "Modalità di aggregazione delle capanne circolari nel villaggio di Su Nuraxi di Barumini," in *In La Preistoria e la Protostoria della Sardegna. Volume II – Relazioni Generali, Atti della XLIV Riunione Scientifica dell'Istituto Italiano di Preistoria e Protostoria (Cagliari, Barumini, Sassari 23-28 novembre 2009)*, Istituto Italiano di Preistoria e Protostoria, 2012, pp. 745-750.
- [55] G. Depalmas, "Nuove ricerche presso il nuraghe Lugherras di Paulilatino (Or) – campagna 2006-2007," in *La Preistoria e la Protostoria della Sardegna. Volume IV – Poster, Atti della XLIV Riunione*



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DI RIPRESA E RESILIENZA



Scientifica dell'Istituto Italiano di Preistoria e Protostoria (Cagliari, Barumini, Sassari 23-28 novembre 2009), Firenze, Istituto Italiano di Preistoria e Protostoria, 2012, pp. 1355-1360.

- [56] A. Depalmas, "Il paesaggio del sacro nella Sardegna nuragica, Architetture celebrative e spazi cerimoniali nei luoghi di culto e nei santuari," in *Atti dell'Undicesimo Incontro di Studi Valentano (Pitigliano 14-16 Settembre 2012) Centro Studi di Preistoria e Archeologia*, Milano, Negroni Catacchio N. (ed.), 2014, pp. 481-496.
- [57] A. Depalmas, "Dal nuraghe a corridoio al nuraghe complesso," in *Il Tempo dei Nuraghi*, Nuoro, Italy, Ilisso Edizioni, 2018, pp. 54-60.
- [58] G. Salis, "Il tempio circolare nel complesso archeologico di Sa Carcaredda a Villagrande Strisaili," in *L'Isola delle Torri. Giovanni Lilliu e la Sardegna Nuragica – Catalogo della Mostra*, Sassari, Carlo Delfino Editore, 2015, pp. 281-286.
- [59] A. Antona, "Tombe di giganti in Gallura. Nuove acquisizioni," in *La Civiltà Nuragica: Nuove acquisizioni II*, MIBAC – Soprintendenza Sardegna, 2008, p. 713–728 .
- [60] A. Moravetti, "La tomba e l'ideologia funeraria, in," in *La civiltà nuragica,,* Milano , Electa, 1985, p. 120–168 .
- [61] E. Castaldi, Tombe di giganti nel Sassarese, in, Roma: Origini, III, 1969.
- [62] E. Contu, Il significato della "stele" nelle tombe di giganti, Sassari 1978 : Quaderni Archeologia e Conservazione, 1978.
- [63] C. Bittichesu, "La tomba di Bùsoro a Sedilo e l'architettura funeraria nuragica," in *Archeologia del Sinis, Atti II Convegno,,* Cabras, 1996.



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PIANO NAZIONALE
DI RIPRESA E RESILIENZA



- [64] R. Cicilloni, F. Porcedda, G. Cabras, L. Spanedda and J. Camara-Serrano, "Analisi territoriali in un'area della Sardegna centromeridionale: modelli ubicativi durante l'età del Bronzo," *Archeologia e Calcolatori*, vol. 30, pp. 329-346, 2019.
- [65] R. Cicilloni, G. Cabras, F. Porcedda and J. Camara-Serrano, "Protohistoric landscapes in Sardinia (Italy): territorial control and exploitation of natural resources in the middle and late Bronze ages," *CPAG*, vol. 31, no. 159-179, 2021.
- [66] L. Manca and S. Demurtas, "Protonuraghi a camera naviforme," in *La Sardegna nel Mediterraneo tra il Bronzo medio e il Bronzo recente, XVI-XIII sec. a.C., Atti del III Convegno di studi Un millennio di relazioni fra la Sardegna e i paesi del Mediterraneo*, Cagliari, Edizioni della Torre, 1992, pp. 107-125.
- [67] G. Ugas, *L'Alba dei Nuraghi*, Cagliari: Fabula, 2005.
- [68] A. Moravetti, "Ricerche archeologiche nel Marghine-Planargia.," in *Il Marghine – monumenti: parte prima*, Sassari, Carlo Delfino Editore, 1998, pp. 645 - 648.
- [69] G. Gasperetti and M. Logias, "Scavi nel villaggio del Nuraghe Appiu a Villanova Monteleone," in *Memorie dal sottosuolo: Scoperte archeologiche nella Sardegna centro-settentrionale*, Quartucciu, Scuola Sarda, 2013, pp. 73 - 83.
- [70] A. Moravetti, *Sardegna archeologica dal cielo, dai circoli megalitici alle torri nuragiche*, Sassari: Carlo Delfino Editore, 2010.
- [71] A. Moravetti, "Il territorio in epoca neolitica e prenuragica (circa 6000 a.C. - 1500 a.C.)," in *Casula F.C (ed.)*, 1990, pp. 19-26.
- [72] E. Atzeni, "Cagliari Preistorica," *Atti* 1986, 21-57, 1968.

- [73] N. Canu and R. Cicilloni, *Il Santuario di Santa Vittoria di Serri tra archeologia del passato e archeologia del futuro*, Roma: Edizioni Quasar, 2015.
- [74] E. Contu, *Edificio megalitico rettangolare di Domu de Orgia in località Cuccureddi, Esterzili, Sassari*, 1948.
- [75] M. Fadda and F. Posi, *Il Villaggio Santuario di Romanzesu. Sardegna. Guide e Itinerari*, vol. 39, Sassari: Carlo Delfino Editore, 2006.
- [76] D. Varnes, "Landslide hazard zonation: A review of principles and practice," *UNESCO*, 1984.
- [77] E. Vecchi, M. Cigagna, D. Fiorino, B. Grosso, E. Pilia and G. Vacca, "Mapping of the multi-risk analysis for the cultural heritage of Sardinia from the Pre-Nuragic and Nuragic periods: Initial results of the RETURN project.," in *Computational Science and Its Applications - ICCSA 2025, Springer Nature Switzerland*, 2025.
- [78] M. Salis, A. Ager, F. Alcasena, B. Arca, M. Finney, G. Pellizzaro and D. Spano, "Analyzing seasonal patterns of wildfire exposure factors in Sardinia, Italy," *Environmental Monitoring and Assessment*, vol. 187, 2015.
- [79] A. Cardil, M. Salis, D. Spano, G. Delogu and D. Molina Terren, "Cardil, A., Salis, M., Spano, D., Delogu, G., & Molina Terren, D. (2014). Large wildland fires and extreme temperatures in Sardinia (Italy)," *iForest - Biogeosciences and Forestry*, vol. 7, no. 3, p. 162–169, 2014.
- [80] F. Bell, *Engineering geology* (2nd ed.), Butterworth-Heinemann., 2007.
- [81] B. Fitzner, *Damage diagnosis on stone monuments*, International Journal for Restoration of Buildings, 2002.
- [82] E. Winkler, *Stone in architecture: Properties and durability* (3rd ed.), Springer, 1997.

- [83] J. Sippel, T. Weiss, S. Siegesmund and Á. Török, “Decay of natural stones caused by fire damage,” in *Natural Stone Resources for Historical Monuments*, London,, Geological Society, Special Publications, 271, 2007.
- [84] E. Winkler, *Stone in architecture: Properties and durability* (5th ed.), Springer, 2014.
- [85] Y. Biro, Á. Török, A. Várkonyi and Á. Török, “Effect of fire-related temperatures on natural stones,” *Construction and Building Materials*, vol. 212, p. 92–101, 2019.
- [86] Regione Autonoma della Sardegna, Piano Stralcio di Bacino per l’Assetto Idrogeologico (PAI), <https://autoritadibacino.regione.sardegna.it/pianificazione/piano-stralcio-di-bacino-per-lassetto-idrogeologico-pai/>, 2006.
- [87] D. Hobbs, “Rock compressive strength,” *Colliery Engineering*, vol. 287–292, p. 41, 1964.
- [88] J. Paone , D. Madson and W. Bruce, *Drillability studies—Laboratory percussive drilling*. USBM RI 7300, Washington, D.C.: USBM, 1969.
- [89] S. Tandanand and H. Unger, *Drillability determination—A drillability index of percussive drills*. USBM RI 8073., Washington, D.C.: USBM, 1975.
- [90] H. Rabia and W. Brook, “An empirical equation for drill performance prediction,” in *Proceedings of the 21st Symposium on Rock Mechanics (pp. 103–111)*. University of Missouri., 1980.
- [91] D. Singh, “Determination of some engineering properties of weak rocks,” in *K. Akai (Ed.), Proceedings of the International Symposium on Weak Rock (pp. 21–24)*, Balkema, 1981.
- [92] R. Göktan, “Theoretical and practical analysis of rock rippability,” Ph.D. thesis, Istanbul Technical University, 1988.

- [93] M. Fener, O. Gunaydin and M. Ferentinou, "A comparative evaluation of indirect methods to estimate the compressive strength of rocks.," *Rock Mechanics and Rock Engineering*, vol. 38, no. 4, p. 329–343, 2005.
- [94] O. Toraman, O. Bayat and M. Çelik, "Predicting the crushability of rocks from the impact strength index.," *Minerals Engineering*, vol. 23, no. 9, p. 752–754, 2010.
- [95] M. Saliu, A. Akindoyeni and I. Okewale, "Correlation between blast efficiency and uniaxial compressive strength.," *International Journal of Engineering and Technology*, vol. 3, no. 8, p. 799–805, 2013.
- [96] G. Sarıışık, T. Topal and Y. Yılmaz, "Classification of parameters affecting impact resistance of natural stones.," *Journal of Testing and Evaluation*, vol. 44, no. 4, p. 20140276, 2016.
- [97] A. Palström, RMI - A system for characterizing rock mass strength for use in rock engineering, 1995.
- [98] T. Hansen, "Rock properties.," *Norwegian Rock and Soil Association*, vol. Publ. No. 5., 1988.
- [99] E. Hoek and E. Brown, *Underground excavations in rock.*, Institution of Mining and Metallurgy., 1980.
- [100] Z. Bieniawski, *Rock mechanics design in mining and tunneling.*, A.A. Balkema., 1984.
- [101] S. Grillo, S. Mocci, G. Pia, N. Spanu and L. Tuveri, *Il manuale tematico della pietra* (U. Sanna & C. Atzeni, Eds.). Itaca. (In Italian), 2009.
- [102] R. Lama and V. Vutukuri, *Handbook on mechanical properties of rocks.*, Trans Tech Publications., 1978.
- [103] G. Massacci and G. Medici, "Le piroclastiti sarde come pietre ornamentali," in C. Marini (Ed.), *Atti del Convegno "Le Materie Prime Minerali Sarde" (Cagliari, Italy, 23–24 Giugno 1997).*, CUEC. (In Italian), 1997.

- [104] c. Meletti, R. Camassi and V. Castelli, “A reappraisal of the seismicity of Sardinia, Italy.,” *Bulletin of the Seismological Society of America*, vol. 111, no. 2A, p. 1148–1158, 2021.
- [105] A. Gorshkov, G. Panza, A. Soloviev and E. Brandmayr, “On the seismic potential of the Corsica-Sardinia block.,” *Rendiconti Lincei. Scienze Fisiche e Naturali*, 32, p. 715–728.
- [106] M. Anselmi, G. Saccorotti, D. Piccinini, C. Giunchi, M. Paratore, P. De Gori, M. Buttinelli, E. Maggio, A. Plaisant and C. Chiarabba, “Microseismic assessment and fault characterization at the Sulcis (South-Western Sardinia) field laboratory.,” *International Journal of Greenhouse Gas Control*, vol. 95, p. 102974, 2020.
- [107] A. Bortolussi, V. Dentoni, C. Levanti, S. Cara, F. Pinna and B. Grosso, “3D Statics Analysis of Nuragic Towers,” *under review*, 2025.
- [108] G. Lilliu, *La Civiltà dei Sardi. Dal paleolitico all’età dei nuraghi* (3rd ed.), Torino, Italy: Nuova Eri Edizioni Rai., 1988.
- [109] E. Contu, *La Sardegna Preistorica e nuragica*, Sassari: Carlo Delfino Editore, 1997.
- [110] A. Depalmas and R. Melis, “The Nuragic people: Their settlements, economic activities and use of the land, Sardinia, Italy,” in *Landscapes and Societies: Selected Cases*, Dordrecht, The Netherlands, Springer, 2010, pp. 167-186..
- [111] A. Usai, “The Nuragic adventure: Monuments, settlements and landscapes,” in *In Gardening time. Monuments and landscape from Sardinia, Scotland and Central Europe in the very long Iron Age*, Cambridge, UK, McDonald Institute for Archeological Research, 2021, pp. 151-158.
- [112] M. Perra and A. Usai, “Il tempo dei Nuraghi,” in *Il Tempo dei Nuraghi*, Nuoro, Italy, Ilisso Edizioni, 2018.

- [113] E. Blake, "Sardinia's nuraghi: Four millennia of becoming," *World Archaeology*, vol. 30, no. 1, pp. 59-71, 1998.
- [114] A. Stiglitz, "Beyond the Nuraghe: Perception and reuse in Punic and Roman Sardinia," in *Gardening time. Monuments and landscape from Sardinia, Scotland and Central Europe in the very long Iron Age*, Cambridge, McDonald Institute for Archaeological Research, 2021, pp. 75-82.
- [115] A. Moravetti, "Considerazione sui protonuraghi," in *Corpora delle antichità della Sardegna. La Sardegna Nuragica, Storia e Monumenti, Catalogo dei siti.*, Sassari, Carlo Delfino Editore, 2017, pp. 11-27.
- [116] A. Usai, "Le modalità di insediamento e il controllo del territorio," in *Il Tempo dei Nuraghi*, Nuoro, Italy, Ilisso Edizioni, 2018, pp. 40-53.
- [117] G. Ugas, *Shardana e Sardegna* (2nd ed.), Cagliari, Italy: Edizioni Della Torre, 2017.
- [118] W. Cavanagh and R. Laxton, "Notes on building techniques in Mycenaean Greece and Nuragic," in *Sardinia in Studies in Sardinian Archaeology III: Nuragic Sardinia and the Mycenaean World*, Oxford, UK, BAR International Series 387., 1987, pp. 39-55.
- [119] F. Campus and S. Cappai, "Dalla pianificazione territoriale alla fabbrica del nuraghe," in *Il Tempo dei Nuraghi*, Nuoro, Italy, Ilisso Edizioni, 2018, pp. 61 - 84.
- [120] A. Vanzetti, G. Castangia and A. Depalmas, "Complessi fortificati della Sardegna e delle isole del Mediterraneo occidentale nella protostoria," in *Scienze dell'Antichità: 19*, Roma, Edizioni Quasar, 2013, pp. 83 - 123.
- [121] F. Lo Schiavo, "Contatti e Scambi: I Nuragici per mare e per terra," in *Il Tempo dei Nuraghi*, Nuoro, Ilisso Edizioni., 2018, pp. 204 - 314.



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PIANO NAZIONALE
DI RIPRESA E RESILIENZA



- [122] R. Leighton, "Nuraghi as ritual monuments in the Sardinian Bronze and Iron Ages (circa 1700–700 BC)," *Open Archaeology*, vol. 9, no. 1, pp. 229-255, 2022.
- [123] A. Vanzetti, "Nuragic memories: A deep-seated pervasive attitude," in *Gardening time. Monuments and landscape from Sardinia, Scotland and Central Europe in the very long Iron Age*, Cambridge, UK, McDonald Institute for Archeological Research, 2021, pp. 151 - 158.
- [124] M. Rassu, *Geometrie nuragiche. Tipi, prototipi, archetipi*, Ghilarza, Italy: NOR Edizioni., 2020.
- [125] M. Rassu, *Ingegneria nuragica. Tecniche costruttive e organizzazione del cantiere*, Ghilarza, Italy: NOR Edizioni, 2021.
- [126] W. Cavanagh and R. Laxton, "An investigation into the construction of Sardinian nuraghi," in *Papers of the British School at Rome*, 1987, pp. 1-74.
- [127] S. Cappai, "A hypothesis on a building technique to determine the shape of the Nuragic tholoi," in *Proceedings of the First International Congress on Construction History*, Reverte Madrid, Madrid, Spain, 2003, pp. 535-544.
- [128] S. Cappai and G. Pulina, "Le caratteristiche tecnico-costruttive delle tholoi nuragiche," in *La Sardegna Nuragica. Storia e Monumenti*, Sassari, Italy, Carlo Delfino Editore, 2017, pp. 55 - 66.
- [129] E. Contu, *Ichnussa: La Sardegna dalle origini all'età classica* (3rd ed.), Milano, Italy: Garzanti Sheiwiller, 1990.
- [130] M. Roberti and O. Spina, "Discrete element analysis on the Sardinian "Nuraghe"," in *Historical constructions 2001: Possibilities of numerical and experimental techniques*, Proceedings of the 3rd International Seminar, 2001, p. 719–727.

- [131] N. Christensen and W. Mooney, "Seismic velocity structure and composition of the continental crust: A global view.," *Journal of Geophysical Research: Solid Earth*, vol. 100, no. B6, pp. 9761-9788, 1995.
- [132] R. Wüst and C. Schlüchter, "The origin of soluble salts in rocks of the Thebes mountains, Egypt: The damage potential to ancient Egyptian wall art," *Journal of Archaeological Science*, vol. 27, no. 12, pp. 1161-1172, 2000.
- [133] J. Barber, G. Cavers, A. Heald and D. Theodossopoulos, "Memory in practice and the practice of memory in Caithness, northeast Scotland, and in Sardinia," in *Gardening time. Monuments and landscape from Sardinia, Scotland and Central Europe in the very long Iron Age*, Cambridge, UK, McDonald Institute for Archaeological Research, 2021, pp. 7 - 15.
- [134] J. Atkinson, *The mechanics of soils and foundations* (2nd ed.), Boca Raton, FL: CRC press, 2007.
- [135] Y. He, B. Liang, J. Yang, S. Li and J. He, "An iterative closest points algorithm for registration of 3D laser scanner point clouds with geometric features," *Sensors*, vol. 17, no. 8, p. 1862, 2017.
- [136] R. Barzaghi, A. Borghi, D. Carrion and G. Sona, "Refining the estimate of the Italian quasi-geoid," *Bollettino di Geodesia e Scienze Affini*, vol. 3, pp. 145-160, 2007.
- [137] A. Albertella, R. Barzaghi, D. Carrion and A. Maggi, "The joint use of gravity data and GPS/levelling undulations in geoid estimation procedures.," *Bollettino di Geodesia e Scienze Affini*, vol. 1, pp. 47-57, 2008.
- [138] G. Vacca and A. Dessi, "Low-Cost Geomatics Surveys for Emergency Interventions on Cultural Heritage. The Case of Historic Wall in Cagliari," in *International Conference on Computational Science and Its Applications*, Cham: Springer, 2023, pp. 650-664.



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Italiadomani
PIANO NAZIONALE
DI RIPRESA E RESILIENZA



- [139] M. Shahbazi, G. Sohn, J. Théau and P. Menard, "Development and Evaluation of a UAV-Photogrammetry System for Precise 3D Environmental Modeling," *Sensors*, vol. 15, p. 27493–27524., 2015.
- [140] E. Sanz-Ablanedo, J. Chandler, J. Rodríguez-Pérez and C. Ordóñez, "Accuracy of Unmanned Aerial Vehicle (UAV) and SfM Photogrammetry Survey as a Function of the Number and Location of Ground Control Points Used," *Remote Sens*, vol. 10, p. 1606, 2018.
- [141] R. Szeliski, *Computer vision: algorithms and applications*, Springer Nature, 2022.
- [142] G. Vacca and A. Dessi, "Geomatics supporting knowledge of cultural heritage aimed at recovery and restoration," *The International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences*, vol. 43, pp. 909-915, 2022.
- [143] S. Grillo, E. Pilia and G. Vacca, "Protocols of Knowledge for the Restoration: Documents, Geomatics, Diagnostic. The Case of the Beata Vergine Assunta Basilic in Guasila (Sardinia)," *International Conference on Computational Science and Its Applications*, vol. Cham: Springer International, pp. 670-685, 2022.
- [144] ISPRA, "Carta geologica d'Italia alla scala 1:100 000: Foglio 193 Bonorva [Carta digitale]," Servizio Geologico d'Italia, 2010.
- [145] E. N. I. d. Unificazione, *UNI 10924:2023 Conservazione del patrimonio culturale – Malte per elementi costruttivi e decorativi – Classificazione e terminologia*, Milano: UNI, 2023.
- [146] C. Atzeni, G. Pia, U. Sanna and N. Spanu, "Characterisation and conservation of vesicular basalt materials of Bronze Age Nuraghi (Sardinia)," *Preservation of Natural Stone and Rock Weathering: Proceeding of the International Workshop*, no. Madrid, Taylor & Francis/Balkema, p. 113–118, 2007.

- [147] G. Casula, S. Fais, F. Cuccuru and M. Bianchi, “An Integrated Petrographic, Geomatic and Geophysical Approach for the Characterization of the Carbonate Rocks of the Calcarei di Cagliari Formation.,” *Minerals*, vol. 14, no. 501, 2024.
- [148] S. Fais, G. Casula, F. Cuccuru, P. Ligas, M. Bianchi, A. Plaisant and A. Pettinau, “A Contribution to the Geological Characterization of a Potential Caprock-Reservoir System in the Sulcis Coal Basin (South-Western Sardinia).,” *Energies*, vol. 12, p. 4524, 2019.
- [149] S. Fais, F. Cuccuru, G. Casula and M. Bianchi, “Characterization of Rock Samples by a High-Resolution Multi-Technique Non-Invasive Approach.,” *Minerals*, vol. 9, p. 664, 2019.
- [150] A. Aydin, “Upgraded ISRM Suggested Method for Determining Sound Velocity by Ultrasonic Pulse Transmission Technique,” *Rock Mechanics and Rock Engineering*, vol. 47, pp. 255 - 259, 2013.
- [151] S. Grillo, *Atlante di petrografia dei materiali lapidei di interesse storico e archeologico*, Roma: Quasar, 2015.
- [152] V. Acocella, “La materia lapidea e l’identità territoriale,” in *Architettura e paesaggio: riflessioni sulla pietra*, FrancoAngeli, 2004, p. 45–46.
- [153] E. Heritage, *The maintenance and repair of traditional farm buildings: A guide to good practice*, Swindon: English Heritage, 2011 .
- [154] Itasca Consulting Group, Inc., UDEC – Distinct Element Code, Version 7.00 [Software], Itasca Consulting Group, 2025.
- [155] Itasca Consulting Group, Inc., 3DEC – 3 Dimensional Distinct Element Code, Version 7.00 [Software]., Itasca Consulting Group, 2025.
- [156] J. Lemos, “Discrete element modeling of masonry structures,” *International Journal of Architectural Heritage*, vol. 1, no. 2, p. 190–213, 2007.

- [157] J. Lemos, “Discrete element modeling of the seismic behavior of masonry construction,” *Buildings*, vol. 9, no. 2, p. 43, 2019.
- [158] R. Dimitri, L. De Lorenzis and G. Zavarise, “Numerical study on the dynamic behavior of masonry columns and arches on buttresses with the discrete element method,” *Engineering Structures*, vol. 33, no. 12, p. 3172–3188, 2011.
- [159] B. Pulatsu and E. Erdogmus, “Parametric study on masonry arches using 2D discrete-element modeling,” *Journal of Architectural Engineering*, vol. 24, no. 2, p. 04018007, 2018.
- [160] B. Pulatsu, E. Erdogmus and P. Lourenço, “Simulation of masonry arch bridges using 3D discrete element modeling,” in *Structural Analysis of Historical Constructions*, Springer, 2019, p. 1123–1130.
- [161] P. Asteris, V. Sarhosis, A. Mohebbkhah, V. Plevris, L. Papaloizou, P. Komodromos and J. Lemos, “Numerical modeling of historic masonry structures,” in *Handbook of research on seismic assessment and rehabilitation of historic structures*, Hershey, PA, IGI Global Scientific Publishing, 2015, pp. 213-256.
- [162] N. Mendes, S. Zanotti and J. Lemos, “Seismic performance of historical buildings based on discrete element method: An adobe church,” *Journal of Earthquake Engineering*, vol. 24, pp. 1270-1289, 2018.
- [163] P. Wang and G. Milani, “Application of a general purpose distinct element model (DEM) for the seismic vulnerability prediction of masonry aggregates,” in *Proceedings of COMPDYN 2023*, 2023.
- [164] G. Mirabella Roberti and O. Spina, “Discrete element analysis on the Sardinian “Nuraghe”,” in *Historical constructions 2001: Possibilities of numerical and experimental techniques - Proceedings of the 3rd International Seminar (pp. 719–727).*, 2001.



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PIANO NAZIONALE
DI RIPRESA E RESILIENZA



- [165] P. Cundall, "Formulation of a three-dimensional distinct element model - Part I: A scheme to detect and represent contacts in a system composed of many polyhedral blocks," *International Journal of Rock Mechanics and Mining Sciences*, vol. 25, no. 3, p. 107–116, 1988.
- [166] R. Hart, P. Cundall and J. Lemos, "Formulation of a three-dimensional distinct element model - Part II: Mechanical calculations," *International Journal of Rock Mechanics and Mining Sciences*, vol. 25, no. 3, pp. 117-125, 1988.
- [167] G. Lilliu, *I nuraghi: torri preistoriche di Sardegna*, Edizioni "La Zattera", 1962.
- [168] G. Lilliu, *La civiltà dei Sardi dal neolitico all'età dei nuraghi*, Torino: ERI, 1962.
- [169] F. Remondino, "Heritage recording and 3D modeling with photogrammetry and 3D scanning," *Remote sensing*, vol. 3, no. 6, pp. 1104-1138, 2011.
- [170] R. M. & Associates, *Rhinoceros (Version 7.0) [Computer software]*. <https://www.rhino3d.com/>, 2023.
- [171] F. Laner, Accabadora. *Tecnologia delle costruzioni nuragiche*, Collana di Architettura. Franco Angeli Editore, 1999.
- [172] A. Fraddosio, N. Lepore and M. Piccioni, "Further refinement of the Corbelling Theory for the equilibrium analysis of corbelled domes," *Curved and Layered Structures*, vol. 6, p. 30–40, 2019.
- [173] M. Como, *L'Architettura delle Tholoi Micenee. Aspetti costruttivi e statici*, Università degli Studi Suor Orsola Benincasa, 2007.
- [174] A. Bortolussi, V. Dentoni, C. Levanti, S. Cara, F. Pinna and B. Grosso, "3D Statics Analysis of Nuragic Towers," *under review*, 2025.
- [175] J. Heyman, "The Stone Skeleton: Structural Engineering of Masonry Architecture," *International Journal of Solids and Structures*, vol. 2, no. 2, p. 249–279, 1966.



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Italiadomani
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DI RIPRESA E RESILIENZA



- [176] S. Huerta, Arcos, bóvedas y cúpulas Geometría y equilibrio en el cálculo tradicional de estructuras de fábrica, Madrid: Instituto Juan de Herrera, 2004.
- [177] C. Baggio and P. Trovalusci, "Limit analysis for no-tension and frictional three-dimensional discrete systems," *Mechanics of Structures and Machines*, vol. 26, no. 3, p. 287–304, 199.
- [178] M. Pavlovic, E. Reccia and A. Cecchi, "A Procedure to Investigate the Collapse Behavior of Masonry Domes: Some Meaningful Cases," *International Journal of Architectural Heritage*, vol. 10, no. 1, p. 67–83, 2016.
- [179] A. D'Altri, V. Sarhosis, G. Milani, J. Rots, S. Cattari, S. Lagomarsino, E. Sacco, A. Tralli, G. Castellazzi and S. de Miranda, "Modeling Strategies for the Computational Analysis of Unreinforced Masonry Structures: Review and Classification," *Archives of Computational Methods in Engineering*, vol. 27, no. 4, p. 1153 – 1185, 2020.
- [180] A. Gerges, "Modelling masonry as a continuum: Finite element analysis using ABAQUS," (Doctoral dissertation). University of Cagliari, Department of Civil, Environmental Engineering and Architecture (DICAAR), 2024.
- [181] A. Iannuzzo, M. Angelillo, E. De Chiara, F. De Guglielmo, F. De Serio, F. Ribera and A. Gesualdo, "Modelling the cracks produced by settlements in masonry structures," *Meccanica*, vol. 53, no. 7, p. 1857–1873, 2018.
- [182] M. Pepe, M. Sangirardi, E. Reccia, M. Pingaro, P. Trovalusci and G. de Felice, "Discrete and Continuous Approaches for the Failure Analysis of Masonry Structures Subjected to Settlements," *Frontiers in Built Environment*, 6, art. no. 43., 2020.
- [183] A. Gesualdo, "Quantifying the role of elasticity in the dynamics of unilateral structures: the free oscillations of a masonry-like panel," *Ingegneria Sismica*, vol. 39, no. 4, pp. 43-71, 2022.

- [184] G. Milani, M. Rossi, C. Calderini and S. Lagomarsino, “Tilting plane tests on a small-scale masonry cross vault: Experimental results and numerical simulations through a heterogeneous approach,” *Engineering Structures*, vol. 123, p. 300–312, 2016.
- [185] M. Angelillo, A. Fortunato, A. Gesualdo, A. Iannuzzo and G. Zuccaro, “Rigid block models for masonry structures,” *International Journal of Masonry Research and Innovation*, vol. 3, no. 4, pp. 349-368, 2018.
- [186] M. Webster, “Water-temples of Sardinia: Identification, Inventory and Interpretation,” Uppsala University, MA Thesis, 2014.
- [187] S. Cappai and A. Sotgiu, “Pinnettas de pedra: A guide for the valorisation of dry-stone artifacts,” in *Heritage 2022*, Valencia, 2022.
- [188] INGV (Istituto Nazionale di GeofisiVulcanologia), “MPS04: Probabilistic seismic hazard model for the Italian territory.,” 2014.
- [189] P. Miano, *Paesaggi a rischio*, Siracusa: Lettera Ventidue, 2024.
- [190] G. Bonini and R. Pazzagli, *Quaderni 16. Paesaggi dell'acqua*, Gattatico: Edizioni Istituto Alcide Cervi, 2018.
- [191] C. Tosco, *Il paesaggio come storia*, Bologna: Il Mulino, 2007.
- [192] M. Cadinu, “Architetture nuragiche e paesaggio protourbano. Relazioni antiche ed estese dimensioni attuali.,” in *Restauro Archeologico, XXX, Special Issue/2022*, Florence 18–19 November 2022, 2022.
- [193] M. Cadinu and S. Orione, *Cabu Abbas: fonte dello stratificato paesaggio millenario del Nuraghe Santu Antine e della Valle dei Nuraghi.*, *Il Tesoro Delle Città, Strenna XV*, 2024, pp. 92-129.



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DI RIPRESA E RESILIENZA



- [194] G. Deriu, L'assetto territoriale dell'odierno Meilogu dal Basso Medioevo ai nostri giorni (secoli XI-XXI), Cagliari: eBook, 2012.
- [195] A. Boninu, "Testimonianze di Età Romana nel territorio di Torralba," in *I Nuraghe S. Antine nel Logudoro-Meilogu*, Sassari: Delfino, 1988, pp. 305-314.
- [196] A. Mastino, "Postumio Matidiano Lepido - Un nuovo preside clarissimo," in *Il Nuraghe S. Antine nel Logudoro-Meilogu*, Sassari, Delfino, 1988, pp. 315-326).
- [197] S. Mais, "Giovanni Antonio Carbonazzi e la costruzione della Strada Reale Carlo Felice (1822-1829)," *Aristana*, pp. 45-59, 2023.
- [198] S. Mais, Ponti, strade e opere pubbliche. Giovanni Antonio Carbonazzi (1792-1873) ingegnere nel Regno di Sardegna, Wuppertal: Steinhäuser Verlag, 2020.
- [199] A. Carandini, La forza del contesto. Come estrarre dai beni inanimati, immersi nel sonno della storia, il potenziale capace di risvegliarli?, Roma-Bari: Editori Laterza, 2017.
- [200] G. Calvi and G. Magenes, "Experimental research on response of URM building systems," in *Proceedings of the U.S. – Italian Workshop on Guidelines for Seismic Evaluation and Rehabilitation of Unreinforced Masonry Buildings*, 1994.
- [201] S. Lagomarsino and S. Giovinazzi, "Macroseismic and mechanical models for the vulnerability assessment of current buildings," *Bull Earthq Eng*, pp. 445-463, 2006.
- [202] R. Cicilloni and F. Porcedda, Serri Survey Project: ricerche archeologiche nella Sardegna centromeridionale, Firenze: Saperi condivisi (M. Miari, ed.), Istituto Italiano di Preistoria e Protostoria, 2019, pp. 20-23.
- [203] A. Moravetti, "La tomba e l'ideologia funeraria," in *La civiltà nuragica*, Milano, Electa, 1985, pp. 120-168.

- [204] R. Siano, P. Roca, G. Camata, L. Pelà, V. Sepe, E. Spacone and M. Petracca, “Numerical investigation of non-linear equivalent-frame models for regular masonry walls,” *Engineering Structures*, vol. 173, p. 512–529, 2018.
- [205] S. Kahraman, “Evaluation of simple methods for assessing the uniaxial compressive strength of rock,” *International Journal of Rock Mechanics and Mining Sciences*, vol. 38, no. 7, p. 981–994, 2001.