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Structural consolidation methods for the Temple of Santa Maria della Consolazione in Todi (Perugia, Italy), damaged by landslides and earthquakes

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ABSTRACT: Between the years 1990 and 2001, restoration and strengthening operations were carried out on the Temple in two distinct phases: firstly, in order to face the historical foundational difficulties of this edifice, which had begun as far back as at the completion of its construction (spread roughly over a century, from 1508 to 1606); secondly, in order to repair the damages to its higher structure caused by the earthquakes in Umbria and Marche, which had begun in 1997. After an ample description of the building, the geology of the Todi Hill is defined, followed by the diagnostic and analytical phases of the project (1985–1987), the project itself (1987) and the restoration and strengthening operations (1990). Subsequently, we describe the preliminary studies, the project and the conservation and repair work carried out on the monument after it was damaged by earthquakes beginning in 1997.

1 INTRODUCTION

All of the studies from the 19th century to date which concern the architecture of Humanism, proclaim the Temple of Santa Maria della Consolazione in Todi as the exemplary image of the "ideal Church", as it is a definite expression of the solid and complex cultural and religious content which characterized the architectural ideals of that time. It represents these models as it is an image of the transition from the Middle-Ages to the Modern Ages, a period dense with values and boundaries (Fig. 1).

It is a unique one-off building which, as Leon Battista Alberti describes in his *De re aedificatoria*, embodies all the ideal features of the "Temple": a monument "which, on account of its purpose to serve and be an expression of the divine, requires more than any other 'ingenuity, ability, diligence"' so that "nothing could possibly be imagined with a more ornate appearance (...), so that the visitors are struck by amazement and admiration at the sight of such worthy things, and can barely refrain from exclaiming: what we see here is truly a place worthy of God". These idealizations represent precisely the Consolazione in Todi in its strong connection to the "truth" and to the "justice" of the cosmic organization which are expressed, above all, in the precise correspondence of the cross's axes with the four cardinal points, seen as spatial as well as temporal cornerstones of the world, through which the directions ordering the cosmos, the sun's path, the passing of the hours in the days and the four seasons cycle, are conjured up.

Figure 1. Church of Santa Maria della Consolazione in Todi (Perugia, Italy).

2 HISTORY

History ascribes the Temple to Bramante, but many researchers, not having found any documented

reference about his presence, have come to the conclusion that it is the fruit of a worthy conception of little-known architects and builders.

De Angelis D'Ossat, contrary to this theory, reports that Bramante accepted the project commission and carried it out, but, "being unable to go there personally... sent Ventura Vitoni da Pistoia in his place". Todi tradition, as early as 1574, and as testified by the Apostolic Visitor Pietro Camaiani when the works on the Temple were not yet finished, also documents that the Consolazione Temple "was designed by Bramante", and this conviction remained constant until 1872, when Adamo Rossi discovered the name of Cola di Matteuccio da Caprarola in the construction records, whereas he never found the name of Bramante during his research.

Cola appears in 1499 next to Antonio da Sangallo the Elder, working for Pope Alexander VI as a papal fortress builder. In Todi, in the construction of the Consolazione, he appears from 1508 in the records as a *magistro contractor*, a construction *conductor a cotimo* or *coptimarius* or *murator*, although from May 1509 until the end of his involvement in 1515, he is called *architettore.* Cola was also contractor of the papal fortress in Civitacastellana designed by Antonio da Sangallo the Elder, who was most likely assisted by Bramante as *sottoarchitettore* for Pope Alexander VI.

Reports from those times also tell of the collapse of a cross vault in the Foligno Duomo, built by Cola as contractor, for which Bramante's advice was sought first. Cola da Caprarola is therefore an architect-contractor, certainly connected to the Roman Sangallo and Bramante circle.

This seems to confirm Jurgen Zanker's theory, which vehemently rules out any intervention by Bramante, assuming that "the Consolazione architecture is the result of various consecutive projects, which were drawn up or modified during construction, depending on the opinion of the various architects who followed each other." And he concludes: "there is no single architect thanks to whom the Consolazione was built, but there are several of them."

It is indeed documented that from 1508 – the year in which the works began – onwards, several architects took part in the construction in a way that was anything but marginal; and it could not have been otherwise because the construction was finished in 1606, that is to say almost a century later. From the documents collected, here is the list of the architects who took part in the construction of the Temple:

- 1508 – Cola da Caprarola, architect;
- 1509 – Gabriele di Giovanni da Como, construction master;
- 1515 – Giovandomenico da Pavia, architect;
- 1518 – Baldassarre Peruzzi, architect;
- 1516/1525 – Ambrogio da Milano, architect;

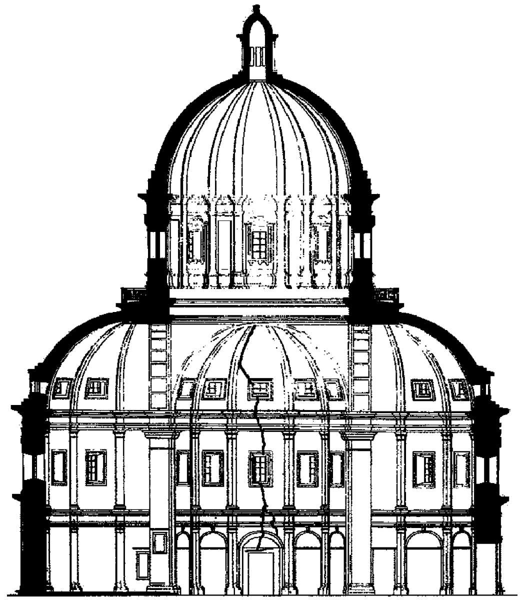


Figure 2. North-South section with identification of the main vertical crack.

- 1524/1563 – Filippo di Salvi da Meli, architect and sculptor;
- 1565 – Giovandomenico Berzugli da Carrara, architect;
- 1565 – Jacopo Barozzi da Vignola, architect;
- 1584 – Guglielmo Portoghese, architect;
- 1584/1587 – Valentino Martelli, architect;
- 1584/1594/1597 – Ippolito Scalza di Orvieto.

Even from the first decade of the 17th century, just after its building was completed, up until today, the stability of the Temple's structure has often been put to the test and has revealed its hardship in cracks and structural element collapses, which were mainly due to differential foundation subsiding or to seismic activity, as was the case during the most recent earthquake in 1997 (Fig. 2).

In 1638, the tie-beam which closed the eastern apse broke; the same tie-beam broke again another two times in the following centuries, and in 1670 an investigation was carried out, which consisted of installing topographic reference points with the intention of measuring the relative movement between the construction and the natural slope; this verification was necessary because a part of the monument is built on settled ground built with the aim of creating a square facing the valley.

After subsequent interventions and continual episodes causing instability leading to the complete exposure of the foundations of the southern apse, which had already undergone under-foundation work

in 1792, a semicircular gravity wall, designed by the engineer Luigi Poletti, architect for the Papal State, was built to the South towards the valley between 1836 and 1860, aimed at contrasting the thrust of the square's protuberance.

In 1926, following serious damage to the drum, the eastern apse underwent consolidation work, consisting mainly of creating an under-foundation to support the southeastern pilaster.

In 1953 a Technical-Scientific Committee was set up which had the task of examining the episodes of instability which had caused a large crack to open up along the median of the semi-cupola above the eastern section, and the consequent damage to the external walls.

The Committee had the task of suggesting possible remedies to finally put an end to the ongoing dynamism.

The resulting consolidation work mainly consisted of the injection of a cement-like mixture into the foundations, with the intention of making them monolithic.

3 THE BUILDING

The church, which has a Greek cross plan, stands on a structural system resting on four large pilasters at the top of a square, on the sides of which the apses are placed: three of these are polygonal and one, the one in the north, is semicircular with its centre slightly moved towards the geometric centre of the church.

The square formation, brought about by the pilasters, is also found in the terrace which can be reached by a stairway set in the shaft to the left of the choir stalls.

The pilasters, by means of large arches, sustain the drum and the cupola; the latter has a small lantern above it, also covered by a cupola. Twenty double embrasure windows, all surmounted by a tympanum, open up between one pilaster and another. This same scheme is repeated, with the same number of square windows, beneath the impost of the apsidal cupola. The drum contains a sequence of alternating windows and niches.

The Church's masonry consists of excellent quality calcareous stone originating from the Titignano quarry, near Todi, and partly also from the demolition of the medieval fortress nearby.

The Temple is set in a wide square, almost completely made with an embankment supported by a gravity wall to the south, built, as already mentioned, by the engineer Luigi Poletti between 1836 and 1860 (Fig. 3).

The building is about 50 m high and seen on a location plan, it can be contained within a circumference of a diameter of about 53 m. The apse walls are about 2 m thick, and the pilasters' imprint covers a surface

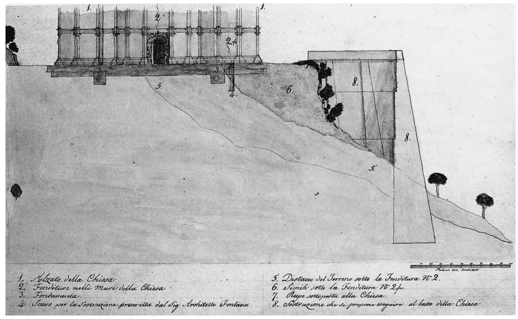


Figure 3. Southern gravity wall project (Luigi Poletti 1836).

of about 25 square meters. The apses and the pilasters are built at different depths; the former, about 20 cm wider on each side compared to the higher section, are 2 m below the ground level, whereas the latter are 4 m below (without considering the under-foundation work carried out on the north-eastern pilaster at a depth of 16.50 m, and on the eastern apse).

The weight of the structure is about 17,000 tons, composed of 6250 tons for the columns and the pendentives, 1870 tons for each polygonal apse, 2030 tons for the altar's circular apse, 1740 tons for the drum and 1370 tons for the large cupola and the smaller cupola.

The building transmits, on average, 0.95 MPa from the pilasters and 0.43 MPa from the apse walls to the foundation grounds.

The pilasters and the external walls are composed of limestone block masonry held together by lime mortar on the outside, and masonry with not regular filler on the inside.

4 GEOLOGY OF THE TODI HILL

The geologic formation of the Todi hill is due to the sedimentation of lacustrine Pliopleistocene origin in the ancient Tiber Basin, which extended to the Umbrian Valley, the Tiber Valley and other Minor Valleys. The hill, originating at the south-western edge of this basin, is made up of distinct lithological sequences, at the lower level, in two complexes:

- basic clayey complex: grey-blue silty-sandy clays, locally marly, with an average slope of a few degrees towards north-east;
- peak conglomerate complex: shingle and sand with a slimy matrix and lenticular position.

On average the soil is plastic ($Ip = 20\% \div 30\%$) and strongly over-consolidated.

These materials have irregular surfaces of tectonic origin and deposits of thin layers of sand.

The piezometric level of the aquifer layer inside the town as well as on the hillside, in a modified and

disrupted surface environment, is not always parallel to the external soil profile. This variability is due to the existence of soils with different positions and different permeability levels: high permeability in the detritus, in the sand levels and in the conglomerates ($K = 10^{-2} \div 10^{-3}$ cm s^{-1}) and low permeability in the silt ($K = 10^{-4}$ cm sec^{-1}) and, even more so, in the clays ($K = 10^{-5} \div 10^{-7}$ cm sec^{-1}).

The undrained (c_u) and drained (c') cohesion lies within the following values:

$$c_u = 0.3 \div 0.8 \text{ kg cm}^{-2};\ c' = 0.8 \div 1.2 \text{ kg cm}^{-2}.$$

This geologically young sedimentary composition, which has always been influenced by morphogenetic factors, is prone to continuous modifications due to natural deterioration, constantly increased by damaging anthropic contributions.

The consolidation interventions, financed by subsidies deriving from the Law 545/87 and the Regional Council Resolution 548/98, were carried out on all the hill's slopes subject to landslides threatening the town boundaries, which have always been prone to upheaval, nowadays still visible in the deficits and cracks in some of its structural parts. The landslides are mainly due to two predominant factors: the continuous incision in the ditches and the alteration of the clays and silts below the stratum. The incision in the ditches is due to the dynamic action of the meteoric waters and the chemical aggression of substances found in the sewers which deteriorates the organic components of the clays and silts. The continuous deepening of these incisions on the hill extends to bordering areas, causing the retrogressive loss towards the hill of the hillside stability.

It was only in the 1980s that the reasons for the constant deterioration of the clays and silts of the Todi hill below the stratum and the lessening of their mechanical capacities were revealed in laboratory experiments. Research in the last twenty years has led to new knowledge about these soils with regard to the stability of the slopes and in particular to the swelling and alteration processes of the surface layers.

It is precisely the Todi clay that was the subject of an experimental study programme undertaken at the geotechnical laboratory of the Structural and Geotechnical Engineering Department of La Sapienza University in Rome (Calabresi, Esu, Pane, Scarpelli, Rampello).

From the analysis performed on undisturbed soil samples immersed in the same water as the stratum on site, it appeared that the swelling of the material was independent from the history of the applied loads, which occurs "in more time than the consolidation process, considering the same tension level", and that the inter-particle binding intensity in the Todi clay is not very strong.

It may thus be deduced that the presence of vertical and sub-vertical discontinuities (cracks) in the soil structure represents a dominant element for the spreading of the swelling and for the lessening of its mechanical capacities.

The swelling obtained in the laboratory is quite representative of the actual swelling that occurs on site, and it is accompanied by a considerable reduction of the effective cohesion, mostly due to the increase in water content.

Taking into account the fact that clay cohesion (and silt cohesion) depends on particle cementing, on electrostatic forces which they exchange amongst each other individually, and on the binding between atoms, one becomes even more convinced that, surpassing a certain distance limit between one particle and another during the swelling of the composition, a polarity inversion sets in and repulsive forces begin to develop which cause, as already mentioned, a lessening of the cohesion. In summary, the persistence of the over consolidated clays and silts in Todi in a "wet-dry" situation due to the alternation of the stratum because of season variations, causes the constant lessening of their mechanical properties, and in particular of the effective cohesion in the wet-dry area.

The Umbrians, the Etruscans and the Romans settled in this beautiful but unsafe – because of possible landslides and earthquakes – landscape, and with time they enlarged the town, adapting it to the hill's configuration and expanding it with terracing that not only filled furrows and ditches, but also acted as a boundary wall.

This settlement, as also happened on other hills of sedimentary origin of the Tiber basin (Perugia, Montone, Ilci, Monte Castello di Vibio, and so on), was certainly advantaged by the easy drawing of water from the stratum which was not very deep, thanks to wells and drainage shafts.

5 DIAGNOSTIC PHASE

An initial study and research phase aimed at examining the Temple's stability was carried out by the Turin Polytechnic in the context of the agreement entered into with the Umbrian Monuments and Fine Arts Office No 1072 of 1985, rep. 2961.

It lasted over two years and included the carrying out of topographic and geo-mechanical measurements, drilling, laboratory tests, numerical analysis and the setting up of instruments to measure the gradient of the slope to check the static behaviour of the structural elements and their interaction with the foundation ground.

The researchers who undertook the study were Prof. Bruno Astori and Prof. Roberto Chiabrando for the topographic part and Prof. Gian Paolo Giani for the

geotechnical part. Later on, in 1990, the consolidation of the Consolazione Temple was planned as the funds connected to the Special Act for the Todi Hill and the Orvieto Rock were made available (Act No 545/87). On that occasion, further geognostic investigations were carried out which completed the ones carried out by the Turin Polytechnic, and inclinometers were installed on the square and just below the square's support wall, on which a lengthy monitoring was carried out, the results of which were always connected to the atmospheric precipitations occurring.

5.1 *Geognostic investigations and geotechnical laboratory tests (Turin Polytechnic)*

Three geognostic drillings were carried out in addition to those which had already been done in the investigations for the "Definitive consolidation of the Todi Hill and Orvieto Rock" (Law 280/78) and to those preliminary to the undersigned's project for the Temple's consolidation.

The geotechnical laboratory investigations carried out by the Turin Polytechnic consisted in: granulometry, Atterberg limits, humidity and volume weight, edometric tests, three-axial, isotropically stabilised and undrained tests.

The geotechnical parameters obtained were:

- OCR consolidation degree;
- RR recompression ratio;
- CR pressure ratio;
- SR drain ratio;
- Ed load deformability module;
- Es drain deformability module;
- σ'_{vp} maximum and effective pressure that has strained the land in its history;
- σ'_{vo} present effective vertical pressure;
- K compression module (load and drain);
- p′ average effective pressure.

Further investigations, preliminary to the planning of the consolidation of the depth upheaval, in addition to the thorough research made by the Turin Polytechnic, made it possible to identify a shifting landslide with its detaching edge immediately above the temple, involving Poletti's wall which supported the square obtained from an embankment, and the entire structure.

The modelling of the upheaval, based on the investigations, apart from small differences, coincided with the schematization of the past.

5.2 *Topographic precision monitoring (Turin Polytechnic 1985–1987)*

It was precisely the analysis of the documents regarding the Temple's history from the 17th century to date, as proof of the recurrence of the instability episodes concerning the elevation structures, the foundations and southern side protuberance on which it partially rests, which led the researchers to set up a programme of experimental measurements, which in essence were:

- the monitoring of the movements of the structures and the terrain of the foundation;
- the definition of the loads affecting the pilasters and the apse walls;
- the variation in the opening of the existing cracks;
- the mechanical characterization of the materials making up the structures and the terrain.

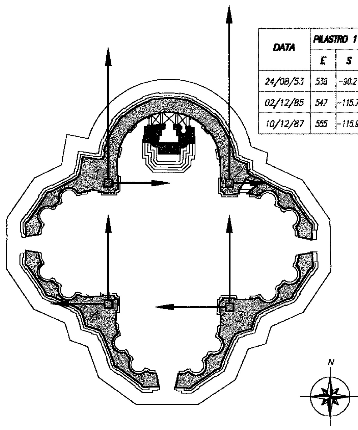
Static analyses were also carried using numerical modelling, aimed at reconstructing the history of the foundation's terrain and at calculating the differential subsiding and the rotations of the pilasters' foundation base, in order to be able to trace the reason behind the drops measured. Tests were also carried out to check the stability of the whole southern side of the Todi Hill, which has always been subject to landslides due to erosion at the foot of the Naia Torrent slope, at the Arnada Torrent inlet.

The topographic survey carried out by the Turin Polytechnic mainly concerned altrimetic monitoring, with the creation of a wire-mesh installed around the Church. Twelve points (which became fourteen in the summer of 1987) were fixed to monitor the Church's possible differential movements in relation to the surrounding slope, as well as the differential movements of the various parts making up the structure Particular attention was given to the southern area, where a benchmark was fixed near the support wall below the Temple.

Another three benchmarks were placed on the northern side (originally there was only one, but as anomalies in the behaviour of the wall were noted, another two benchmarks were added). All the external points were linked to each other by means of a closed polygon hinged to the same point where it converged and the high precision geometric levelling closed ring was bound, inside the Church.

Another fundamental operation concerned the measuring of the pilasters' drop variation using the four brackets cemented from 1953 (for the same purpose), placed at a height of approx. 12 m above the pilaster base. A wire was fixed to each of these, with a 5 kg weight at its extremity immersed in a basin full of oil to reduce oscillations (Fig. 4).

The large crack on the drum and on the semicupola of the eastern apse, which also cut through the portal, was put under control by means of a millesimal crack measuring device. During the two years of Research, seven measurement campaigns were carried out in addition to the initial one (December 1985), which served to determine the initial reference measurements.



DATA	PILASTRO 1		PILASTRO 2		PILASTRO 3		PILASTRO 4	
	E	S	O	S	N	O	N	E
24/08/53	538	-90.2	63	-106.2	151.3	-13.2	139.8	35.3
02/12/85	547	-115.7	-20	-131.5	1700	160	162.2	27.1
10/12/87	555	-115.9	-20	-132.6	170.3	15.7	162.5	265

Figure 4. Historic development of the drops.

5.3 *Mechanical characterization of the building structure with non-destructive measurements on site (Turin Polytechnic – 1985)*

The state of the vertical strain in the pilasters and in the Church's outside walls was registered in eighteen measurements with flat jacks. The cuts on the building structure were made at a height of about 1.5 m from the ground for those inside the church, and at a height of about 2.0 m from the ground for those on the outside walls. The deformability module stabilized on average values between 22,000 MPa and 28,000 MPa. Figure 4 contains the deformability module table and the corresponding histogram (from the ISMES report 12/87).

The vertical strain measured with the flat jack technique was in accordance with the results of the load testing.

5.4 *Stress-deformations finite element analysis (Turin Polytechnic – 1987)*

The finite element method (FEM) was defined with the aim of reproducing the tensional state of the Church's foundation terrains and of determining the subsiding in the apse wall and the pilaster, as well as the rotations at the pilasters' foundation base. The three-dimensional, 100 m high, 105 m long FEM model, made up of 969 nodal points and 579 iso elements, formed a parallelepiped which frames the foundation terrain, the foundations and the semi-circular gravity wall. The FEM simulation was carried out using the following steps:

- loading and drainage of the overconsolidated terrain, in accordance with the results of the edometric tests, in order to reconstruct the history of the tensional state before the construction of the Church;

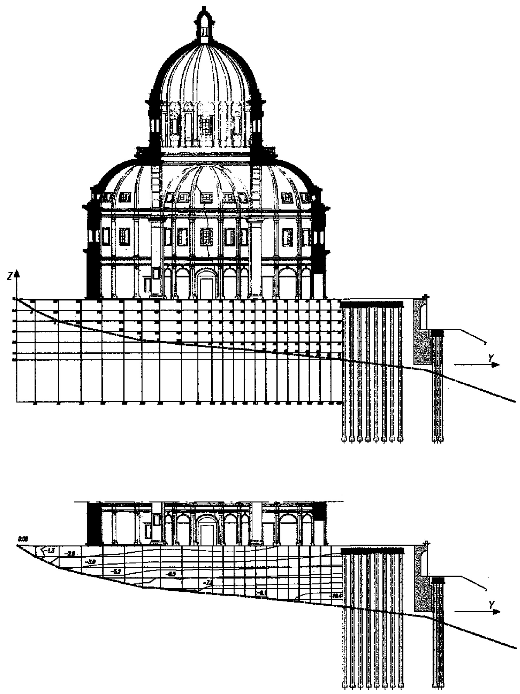


Figure 5. Formation of the upheaval and tension state of the round behind the bulkhead.

- construction of the protuberance and of the gravity wall, excavation of the foundation's terrain and construction of the foundation masonry;
- application of loads equalling the weight of the Church to the pilasters' foundations and to the apse walls.

The comparison between the results of the finite element analysis and the tension values measured in the pilasters led to the identification of vertical subsiding gradients with the same direction, having considered the drop values existing in 1953, the drop variation values between 1953 and 1985, and the drop variation value definition between 1985 and 1987.

5.5 *Finite element analysis for calculating the thrusts on the bulkhead*

The project for the consolidation of the Temple, carried out with funding by the State through Law No 545 of 29th December 1987, required Finite Element Analysis to identify the tensional state of the ground, once the sliding area had been located, for the investigation of strains on the future upheaval contrast structures (Fig. 5).

The test results were compared with the strains resulting from the calculation made with the "Janbu" method, assuming that the layer is altered and in a

movement without discontinuity. In the calculation, the unstable terrain was modelled using flat elements with 3 or 4 nodes with two leeway degrees for each junction (translation according to Y and Z) and a level state of deformation.

The mesh of the elements in the proximity of the estimated position of the contrast bulkhead was intentionally thickened to collect the values of the tensions acting on it with greater precision. The model was bound along the sliding area and to the area which was in contact with the bulkhead of piles. The weight which the Temple puts on the ground in upheaval was not taken into account, because at that stage it had already been decided to carry out an underpinning with small diameter piles over the entire foundation area. A supposition, also used for calculating the micropiles, was that the whole weight of the building be entirely transferred to the ground below the sliding area. The comparison between the results of the finite element analysis on the consolidated system and those of the stability test has shown a substantial qualitative and quantitative coherence of the two methods used.

6 CONSOLIDATION INTERVENTIONS

The Consolazione Temple was the subject-matter of two designs carried out by the undersigned for consolidation interventions:

a) on the upheaval of the foundation, Law No 545 of 29th December 1987;
b) on the damage caused by the earthquakes which occurred on 26th September 1997 and in the following days: Umbrian Regional Council Resolution No 548 of 25th September 1998.

6.1 *Consolidation of the upheaval of the foundation*

It consists of the following works:

- creating an under-foundation for the church using 232 piles with a small diameter (Ø 150 mm), obtained by directly perforating the foundations of the pilasters and the apses for a total of 5800 m;
- contrast the dynamism of the shifting land with two bulkheads (Figs. 6, 7) composed of 257 piles with a large diameter (Ø 600 mm): the first was carried out under the Tiberina main road, and the second behind Luigi Poletti's support wall, which is below the road, for a total of 6024 m. Both bulkheads were made of piles of reinforced concrete piling brought together and bound to one another by means of a reinforced stiffening slab in order to obtain their total cooperation.

This whole structural support system obtained in this way was designed to support the ground thrusts and those caused by the Temple up to a possible undermining of the valley equal to a height of 7 meters due to hydro-geological upheaval on the hillside, consolidated with the subsidies of Law 545/87.

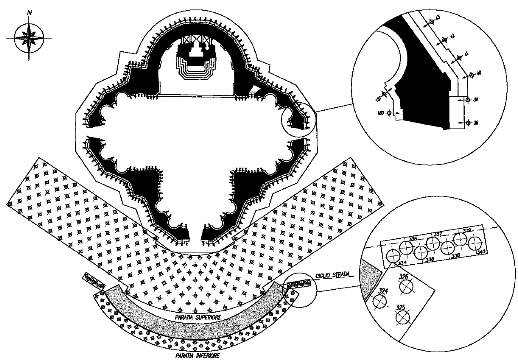


Figure 6. Planimetry of the operation on the upheaval. Bulkheads with large diameter piles and depth reinforcement with small diameter piles.

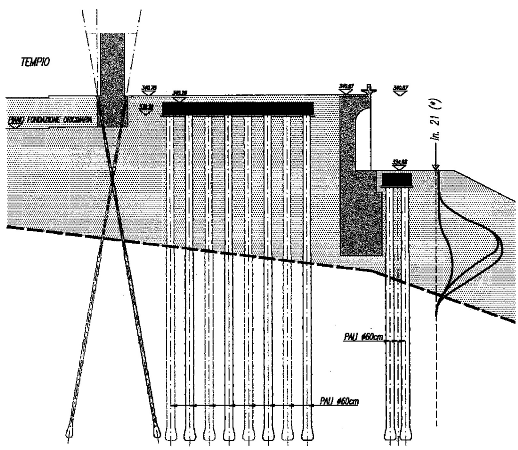


Figure 7. Small diameter piles on the foundations and front bulkheads with large diameter piles.

6.2 *Consolidation of damages caused by earthquakes starting on 26th September 1997*

The intervention has planned the following works (Figs 8 and 9):

- internal binding at the cupola base height by means of a stainless steel band AISI 304 (300 × 15 mm) which can be soldered, band "A";
- external binding at the height of the drum base by means of a stainless steel band AISI 304 (220 × 15 mm) which can be soldered, band "B";
- internal binding at the height of the apses' base by means of a stainless steel band AISI 304 (300 × 15 mm) which can be soldered, band "C";

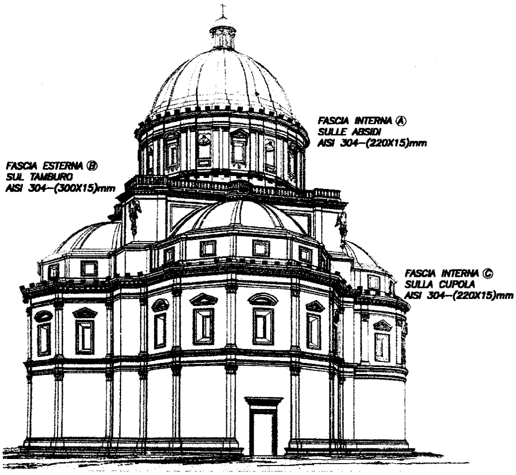


Figure 8. Stainless band disposition “A” – “B” – “C”.

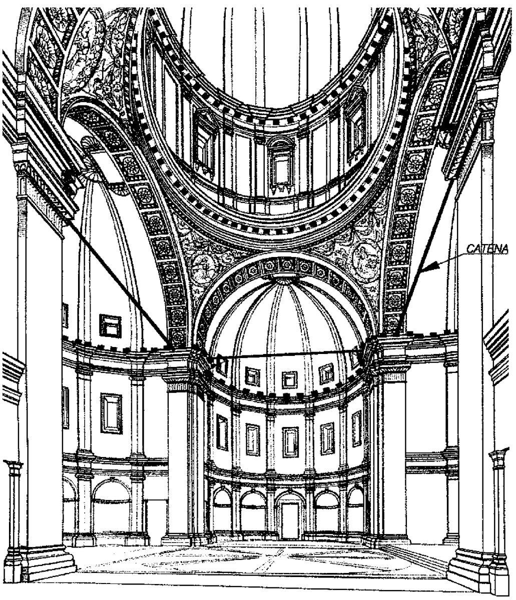


Figure 9. Stainless band disposition “A” – “B” – “C”.

- saturation of the internal and external cracks with hydraulic, sulphate-resistant mortar without alkaline water-soluble salts and with a minimum compression resistance ($R_c \geq 10$ MPa).

6.3 *Internal bindings*

For the placement of the internal bindings, it was necessary to mount scaffoldings inside the Temple in order to be able to reach both of the altitudes where the works were to take place. The internal

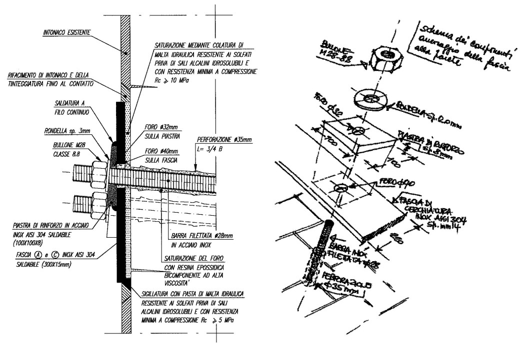


Figure 10. Detail of the internal binding.

bindings were joined to the masonry by means of reinforced perforations (Ø = 35 mm) with stainless steel threaded rods (Ø = 28 mm), the holes being filled with high-viscosity epoxy resins (Fig. 10).

Four unthreading traction tests were made on the anchorages until the load $N = C\ L\ 5\,kg\ cm^{-2}$ was reached, in which:

C = hole circumference; L = hole length.

The longitudinal positioning of the holes on the bands was performed with quincunx in order to avoid twists during the nut screwing and draught phase.

Moreover a reinforcement stainless steel AISI 304 plate (100 × 100 mm).with a hole (Ø = 32 mm) was soldered in correspondence to each hole.

All the soldering performed was Class I with a continuous wire and it was finished off with acid to eliminate the contact scorch marks.

The spaces between masonry and band were sealed with sulphate-resistant hydraulic mortar sediment, without alkaline water-soluble salts and with a minimal compression resistance $R_c \geq 10$ MPa.

6.4 *External binding*

For the placement of the external bindings, it was not necessary to mount scaffoldings; the area where the works were to be performed is in fact accessible by a staircase. All the materials were taken up to the necessary altitude with the aid of a lift.

The external binding was carried out at the height of the drum base and subjected to tension by four temporary tensioners (Fig. 11) with a final force of traction of at least $F = 300\,kg\ cm^{-2}$. In order to aid the band’s tension, “sliding pads” were used, made of neoprene parallelepipeds and PTIFE (TEFLON) plates, vulcanized with neoprene (s = 1.5 mm) and stuck with non-aggressive substances to the wall as well as to the band, to help it to slide more easily during the traction phase on the part of the tensioners. The tensioning of the band was carried out in the summer, so that the

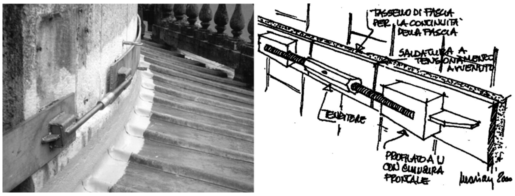


Figure 11. Tensioner apparatus for the external "B" band. The sketch shows also the junction element.

band's connection elements for the restitution of continuity at the tensioners' level could be soldered when the thermal dilatation had already occurred.

Even in this case, the spaces between the masonry and the band, once positioned, were sealed with sulphate-resistant hydraulic mortar sediment, without alkaline water-soluble salts and with a minimal compression resistance $R_c \geq 10\,\text{MPa}$.

6.5 *Conclusions*

The structural consolidation methods thought up to save this important example of Italian Renaissance architecture, which has always been under threat from landslides and earthquakes, are two-fold. The two approaches used belong to scientific fields that are close to each other but distinct: geo-technology and seismology. Specific and evolved technology belonging to the two sectors has been applied, especially that which is in no way invasive, and that respects the magnificent building in question.