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Timber Coverings of Palatine Chapel in Caserta Royal Palace

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ABSTRACT: The analysis of the timber covering structure of the Palatine Chapel in the Royal Palace of Caserta, built in the eighteenth century by Luigi Vanvitelli, is the object of the present study. The interpretation of its structural behaviour has been based upon a detailed survey and visual as well as non destructive inspections. Results of decay analysis have shown presence of confined damaged areas, which in any case do not induce exceeding safety limits, or loss of serviceability. At the aim of understanding the designing process followed by the architect in deciding the best disposition of the structural elements and specifically in choosing the slope of the underlining inclined posts more efficient to optimize structural behaviour, a comparative analysis has been done on different theoretical schemes obtained varying that slope. Numerical results show how little modification on Vanvitelli's scheme will not leave the same safety margins.

1 INTRODUCTION

Charles III of Bourbon dynasty (Madrid 1716 - 1788), become King of Naples in 1734, started an ambitious urbanization program in the hinterland, in Caserta area, whose fulcrum was represented by the grandiose complex of the Royal Palace (about two million of cube metres), evidently inspired by Versailles one. The designing and building of this masterpiece was commissioned to the architect Luigi Vanvitelli (Naples 1700 – Caserta 1773), who worked to it from 1750 to his death, (the building was completed one year later by his son Charles), Chierici (1969).

The rectangular plan is characterized by four internal courts, enclosed by rectangular bodies crossing at right angles, and meeting in a central rotunda; in the west central wing, at the upper level, there is the Chapel, whose covering system is the object of the present study, Fig.1, Vanvitelli, (1756).

The timber covering structure is composed of 12 trusses at a mutual distance of about 2.50 m, with side slopes of 25 degrees. The truss constructive scheme shows a quite special configuration, determined by different technical as well as cultural factors. The neoclassical architectural principia asked for roofs with a low profile, so that they can be exceeded in height by the classical proportioned facade tympanum. The famous British architect Christopher Wren (Wiltshire, 1632, London, 1723), faced this problem designing trusses with double lateral posts, which allowed the employment of two different slopes, raising that of side rafters and reducing until zeroing that of the central part, (queen post trusses), without reducing the span, Yeomans (1992). In Italy as well as in France, the Palladian solution continued to be preferred.

The presence of a vault extrados inside the attic, as in the Palatine Chapel, induces rising of the tie-beam over rafters leaning level, and as a consequence the side thrusts on bearing walls increase. J. R. Rondelet (Lion 1743 – Paris 1829), in his design of the new church of Saint Genevieve in Paris (begun in 1757, then transformed in the actual Pantheon), gave two different solutions of the same structural problem, in the inner naves and in the porticoes, Rondelet (1833). In the last case, inner colonnade presence allows absorbing part of the thrust, thanks to intermediate vertical posts, introduced in the truss scheme under the extremities of the tie-beam, Fig.2.

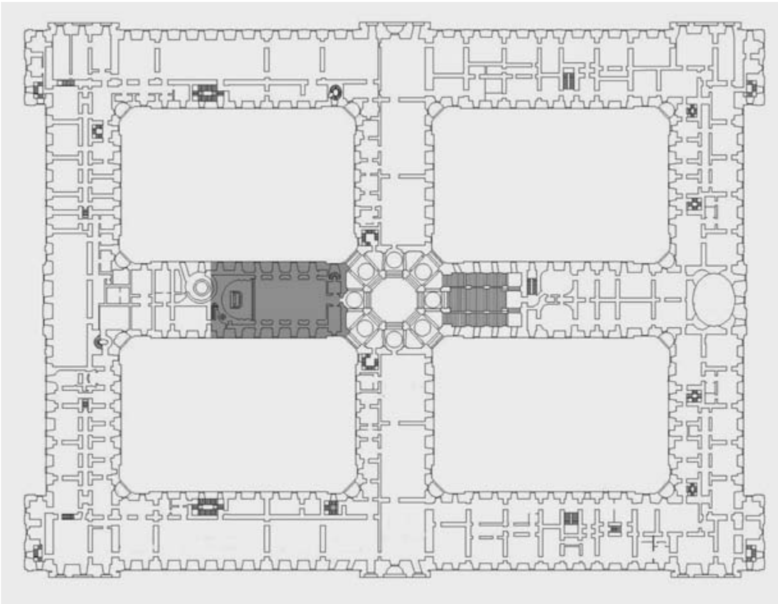

Figure 1 : First floor plan of the Royal Palace, with the Palatine Chapel evidenced.

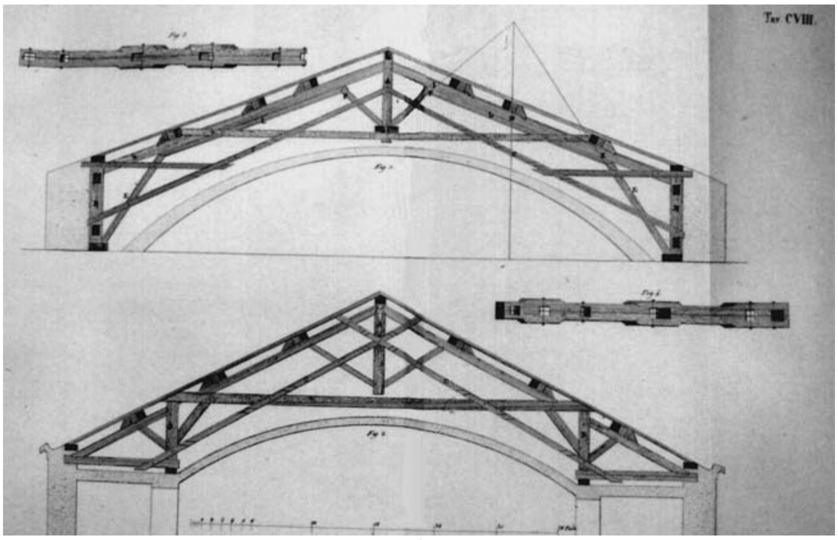

Figure 2 : Drawing of the Pantheon trusses

In his Chapel, Vanvitelli, facing the same situation, a vault extrados and a colonnade internal to longitudinal side walls, employed a similar but more efficient solution, introducing two inclined posts leaning upon the colonnade. The portion of the timber structure disposed above these posts, shows, in its central part, a simple Palladian truss scheme, with king post and inclined struts, and tie-beam made with a forged iron double tie, whose span, of about 12 m, is apprecia-ble less then the global span of the structure, about 19.70 m between side walls. So this struc-tural scheme allows by-passing the vault extrados without raising the top roofing, confining at the same time the side thrusts on masonry structures.

2 INSPECTION OF ONE TRUSS

2.1 Description of the structure

The inspected truss is part of the covering system of the Palatine Chapel, and is that nearest to the central rotunda from which the side wings of the Royal Palace depart.

As said before, the structural scheme has a special configuration, determined by the vault and masonry walls shapes. In fact, to reduce the clear span of 19.70 m between the leanings of the truss on the side walls, two timber inclined posts, in chestnut, have been introduced, with a cross section of 0.26 x 0.325 m. Those posts reduce the clear span of the truss to about 12 m, thanks to the presence of the underlining colonnade, on which they rest.

The other elements of the timber structure, in silver fir, consist of two rafters 11.65 m long, with a cross section of 0.27 x 0.34 m, and with a side slope of about 25 degrees; one king post of dimensions 0.36 x 0.27x2.24 m; and two inclined struts 1.71 m long, with a cross section of 0.275 x 0.24 m. The tie beam is made of two forged iron ties, Fig. 3.

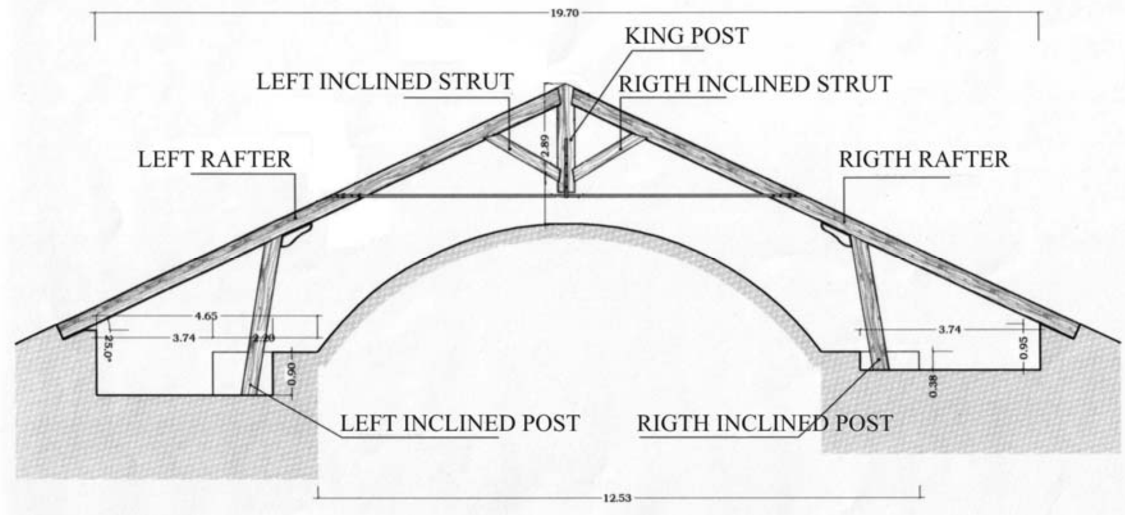

Figure 3 : Survey of the truss

This structural typology is common to the whole covering system of the Palatine Chapel, whose first six trusses has been restored in recent times, while the other six still show problems of decay. On the trusses there is a secondary bearing system made of chestnut purlins, with a cross-section of 0.21 x 0.15 m, disposed at a mutual distance of 1.10 m, on which transversal chestnut timber joists, of 0.14 x 0.08 m cross dimensions, stay to support a brick pavement, 0.025 m thick, sustaining the covering mantle, constituted of flat and bent tiles, fixed with mortar.

Presence of disconnections in this mantle has induced water infiltrations, specially concerning the studied truss, and consequently in some areas biotic decay can be noticed, principally due to fungi, but also due to past worm attacks. Decay is larger near the extremities of the rafters, where they insert into the walls.

Even if connections between the structural elements constituting the truss are realized with simple and not too much deep notches, disconnections and movements from the axial positions of the timber structure elements can't be seen. It's notable the presence of relevant shrinkage splits, specially in the rafters where this phenomenon is shown for their whole length.

2.2 Visual inspection

Visual inspection of the truss has been possible for almost the whole length of the rafters, but a deeper analysis of the timber structure above the level of the tie-beam hasn't been done, due to the impossibility at the moment to built an adequate working floor. The extremities of the rafters can't be inspected as they are completely merged into the masonry; so their conservation level has been deduced by the visual and instrumental inspection of the portions of rafters contiguous with them.

Moisture content has been measured employing an eclectic hygrometer, surveying the mean relative moisture content of each structural element on at least three reachable sides. In this way zones of limited extension in which moisture content is over the admissible values have been detected, as the areas near the rafters ends. For the remaining part of the structure, mean values vary between 12% and 15%. The widespread meteoric infiltrations, due to disconnections in the covering mantle, induce occasional humidification in some of the constituting parts of the truss, for which, during the raining periods in autumn and winter seasons, an occasional overcoming of the relative moisture content of 20% can take place. The rafter ends, completely merged inside the masonry walls, are in an aeration condition which induce a relevant enlarging of the moisture content, as the measurements taken in their proximity, which go over the value of 20%, reveal. This overcoming must be considered permanent in the rafter ends, which, consequently, show a relevant risk of biotic attack.

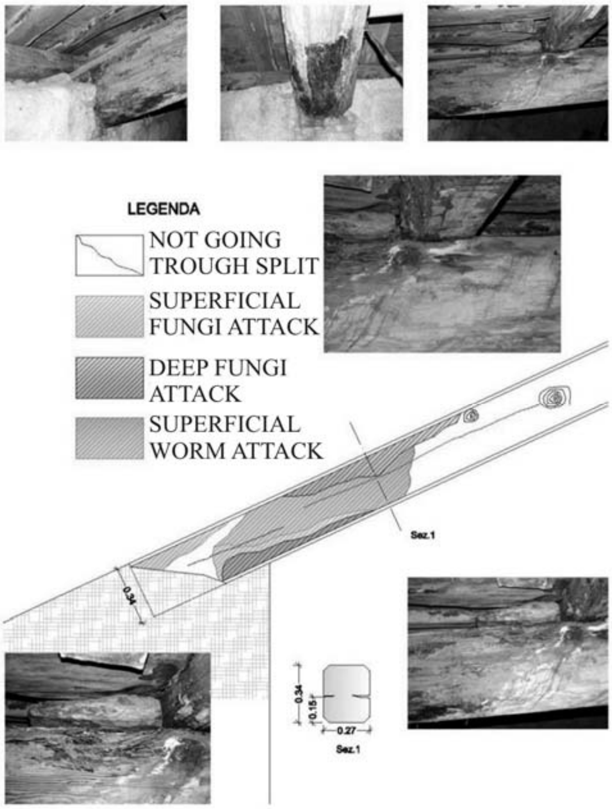

Figure 4 : Chart with the map of decay, for the left rafter.

Wood essences have been evaluated only macroscopically, observing localised areas, whose surface has been cleaned, taking off the incoherent superficial deposits, avoiding areas showing chromatic alteration. Two essence have been detected. All bearing elements except the inclined posts are made of Coniferous wood; it has been identified as silver fir. The original use of silver fir coming from Calabria has been confirmed by the discovering between the documents in Vanvitelli's own hands, kept in the National Library of Naples, Strazzullo (1973), of a letter sent to the General Inspector of Caserta building works, Mr. Neroni, in which the wood to be employed to build the covering of the Royal Palace is described. Inclined posts instead are made of hardwood timber, specifically chestnut, as the elements of secondary structure of the covering system.

The structural classification of timber elements has been done referring to the Italian regulations, UNI 11119 (2004), for which all the components of the examined truss can be ascribed to the II category, with the exception of the inclined posts, which must be considered of the III category for the relevant presence of defects.

Results of defects and decay survey for each timber element are quoted in single charts which contain geometric as well as material characteristics and the conservation status, Fig. 4.

2.3 Resistographic inspection

To analyze the bearing section in the areas of possible critical conditions, non destructive tests have been done employing a Resistograph. The Resistographic method, Rinn (1998), is a quite efficient non destructive technique, as the inspection goes in deep across the examined timber section and consequently gives more reliable data of those methods whose tested areas are limited to the outer surface. The Resistographic diagrams obtained inspecting all the reachable parts of the truss and considered more interesting at the aim of determining the internal status of some cross sections of the structural elements showing traces of decay, have been quoted in single charts, evidencing the internal extension of that decay, Fig. 5.

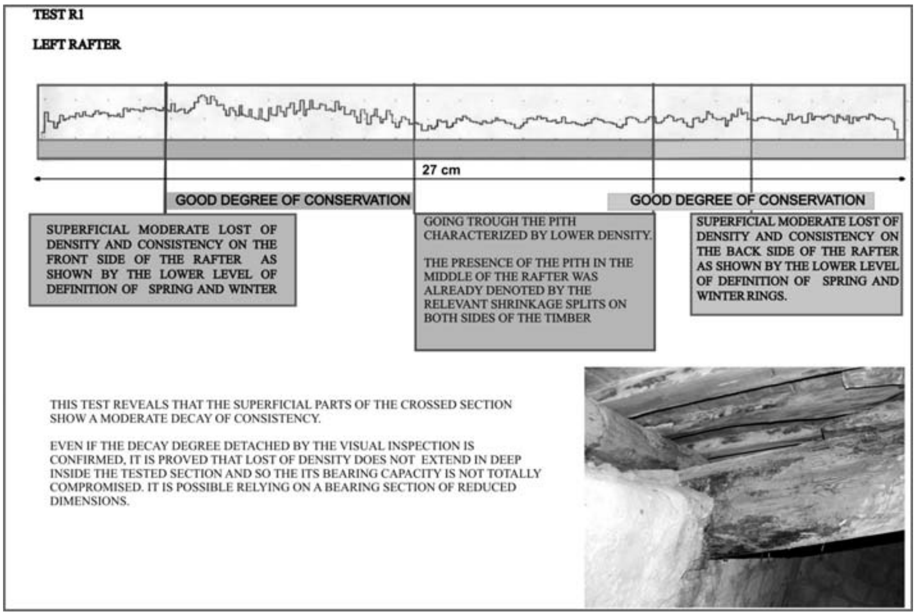

Figure 5 : Resistographic chart of the left rafter.

3 STRUCTURAL ANALYSIS

At the aim of detach the presence of critical areas relating to the distribution of stresses in the elements constituting the truss, a preliminary static analysis, in linear-elastic range, has been done employing a finite element calculation program (NOLIAN by Softing), on a plane one-dimensional scheme.

Permanents loads are due to the covering mantle and the secondary structure bearing it, and obviously to self-loads, while the accidental ones have been valuated following the Italian laws in work. To built the calculus scheme, the materials constituting the structures have been characterized using the value quoted in Table 1, where those referring to timbers have been deduced from the norm UNI (2004), for fir of II category and chestnut of III category.

Table1 : Characteristic values for materials used in the truss elements.

Materials	Density kg/mc	Longitudinal mean modulus of elasticity MPa	Shear mean modulus of elasticity MPa
Chestnut	600	8000	750
Fir	450	12000	520
Forged iron	7800	200000	70000

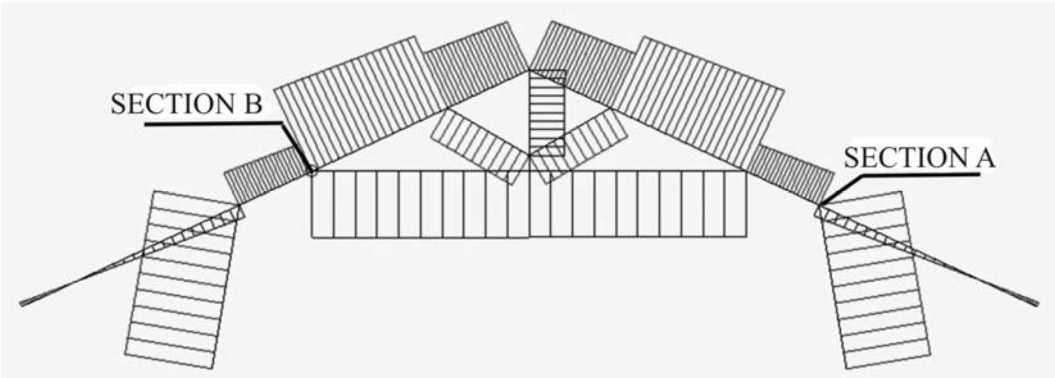

Figure 6 : Normal stresses diagram.

Mutual constraints between structural elements have been modelled to allow relative rotations, relating to the typology of the connections noticed during the survey; the iron tie-beam instead as been assumed notched at its ends, to take in account the connecting devices used to fix it to the rafters.

In modelling the constraints at the ends of the rafters, where they merge into the masonry, the state of decay has been considered, allowing the possibility of slipping, certainly prevented in the original scheme.

In the following figures, normal stresses, Fig. 6, and bending moments, Fig.7, diagrams are reported, from which it can be deduced that the most relevant stresses take place in the rafter where it is jointed with the inclined post (section A), and with the tie-beam, (section B). Corresponding values of stresses are quoted in Table 2.

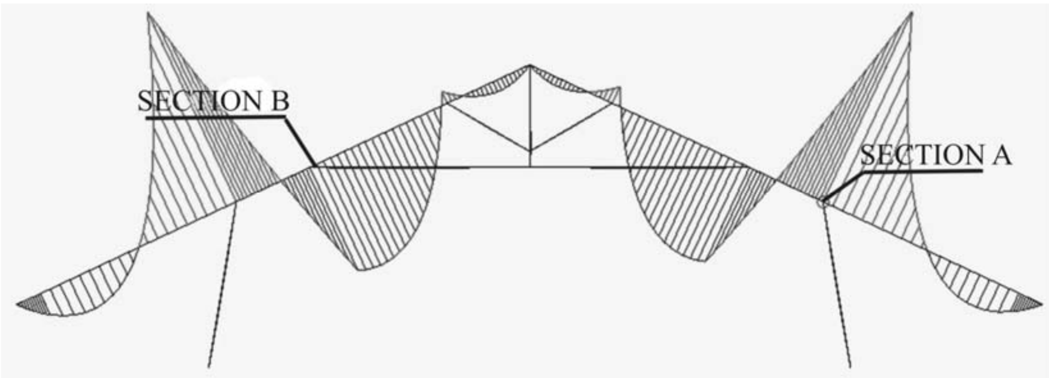

Figure 7 : Bending moments diagram.

Table 2 : Values of stresses in the critical sections of the truss.

	Normal stress N	Bending moment N x cm	Maximum stress MPa
Section A	- 38818	4486129	9.78
Section B	- 97578	2339071	6.27

Comparison with the admissible value given by the UNI norm for silver fir of II category, in static bending, which is 10 MPa, confirms that even the most stressed sections are verified, employing an admissible stress criterion. In both cases the bearing section has been assumed coincident with the geometric section, thanks to the absence of evident biotic attack.

For the critical areas near the ends of the rafters, the appreciable reduction of dimensions for the bearing section, specially on the left side of the truss, due to the presence of a relevant fungi attack, doesn’t give place, from the simple point of view of an admissible stress criterion, to the limit overcoming, thanks to the low level of induced stresses. The presence of bending effects more relevant then normal stress effects in the lower parts of the rafters is a direct consequence of the typology assumed in the calculus scheme for their end constraints.

4 EFFICIENCY OF VANVITELLI'S STRUCTURAL SCHEME

At the aim of evaluating the validity of the structural design ideated by Vanvitelli, the scheme in its original configuration has been studied, modifying the hypothesis on the end constraints of the rafters, assumed in the previous paragraph. In fact those constraints previously have been modelled to allow slipping, to take in account the decay status noticed in the extremities of the rafters; while the original disposition in cavities arranged inside masonry walls prevented slipping.

Vanvitelli certainly had understood that bringing up the tie beam to go above the vault extrados, without lifting the top of the truss, induced a critical state of stress in the portions of the rafters between the leaning on the walls and the connection with the tie-beam. The answer to this problem has been the introduction of an intermediate supporting device for which the design variable was the slope, as the position of the lower end was fixed, coincident with the alignment on the colonnade disposed inside the Chapel.

The efficiency of the solution chosen, that is inclining the post with an acute angle on the horizontal profile, making a 10° deviation from the vertical line, has been proved with a comparative analysis of results obtained from calculus schemes with different values of that slope.

The modified schemes are five, the first four of which have been drowned fixing the lower end of the post and varying the position of the connection of the upper end with the rafter, while in the last scheme the absence of the post has been assumed.

In conclusion there are six schemes:

SCHEME 1: upper end of the post coincident with joint of tie-beam with the rafter;

SCHEME 2: original scheme, with post slope deviating of 10° from the vertical line;

SCHEME 3: post in vertical position;

SCHEME 4: post slope the same as in scheme 2 but on the opposite side;

SCHEME 5: post perpendicular to the rafter;

SCHEME 6: absence of the post.

Parameters taken in account for the comparison of the above schemes are: maximum stress in the structure and total thrust made on masonry structures, obtained as the sum of the thrusts made by the rafter on the external walls and by the post on the inner colonnade. Parameter values are quoted in Table 3, while the calculated schemes are drawn in Fig. 8.

Table 3 : Values of the parameters for the calculated schemes.

	Maximum stress MPa	Thrust of the rafter kN	Thrust of the post kN	Total thrust kN
Scheme 1	6.89	12	37	49
Scheme 2	5.26	71	9	80
Scheme 3	4.90	95	0	85
Scheme 4	4.71	111	- 6	105
Scheme 5	5.04	140	- 19	121
Scheme 6	9.14	164	0	164

From results in Table 3 the favourable effect of introducing the inclined post is clear in term of maximum stress as well as of thrust on masonry structures. In fact the scheme 6 shows values of the chosen parameters definitely larger than those of the other schemes.

For the schemes with the post, the maximum stress values are quite near and in any case far from the admissible value, while the values of the thrust vary remarkably with the slope of the post. In a limit process, the best configuration, in respect to the thrust value, is obtained when the post upper end position coincides with the end of the tie-beam (scheme 1); in fact in that situation the value of the total thrust lowers to 49 kN. Technological difficulties, specially for the connection between the upper side of the post and the lower one of the rafter have certainly advised against choosing a deviation angle respect to the vertical line larger then that effectively realized. The little reductions which can be noticed in the maximum stress in the other schemes don't compensate for the increase of the thrust obtained in schemes 3, 4, 5, that is for a post disposed vertically or inclined in the opposite direction.

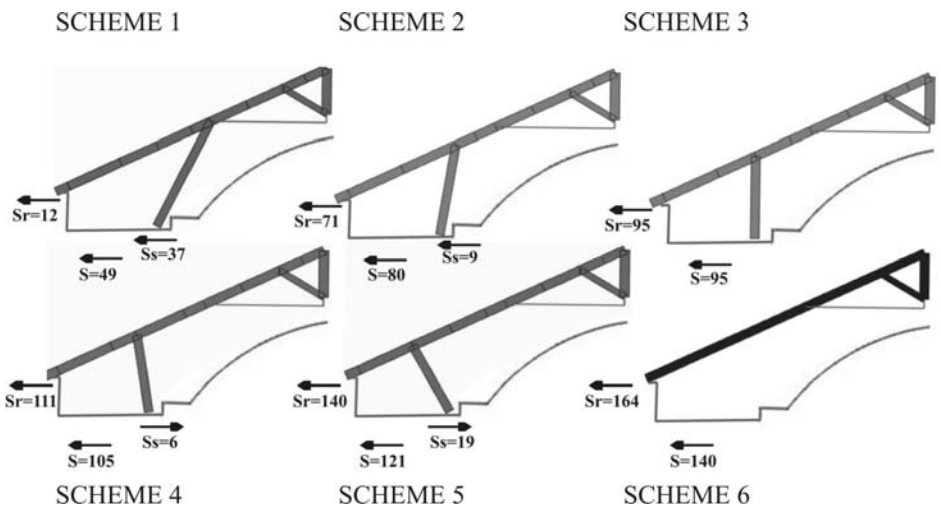

Figure 8 : Different calculated schemes with values of the thrusts.

5 CONCLUSIONS

The conservation state of the examined truss is quite good, without taking in account confined portions of rafters and specially those near the lower ends of them. The modification in the theoretical scheme corresponding to the original structure (scheme 2) analyzed in the last paragraph, induced by the decay status in those parts, gives place to a remarkable enlargement of the maximum stress value, specially due to bending effects, but obviously gives also a consistent reduction of the total thrust made on the masonry structure, quite existing only at the end of the inclined post (14 kN against 80 kN of the scheme 2). So an accurate restoration of the truss is certainly necessary, with the eventual substitution of the damaged portions at the ends of the rafters, but without modifying the actual structural scheme. In other words slipping at the end constraints ought to be left allowable, and an appropriate aeration of rafters ends inside their leaning in the lateral walls must be granted to avoid the recreation of the decay phenomena noticed now.

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