Structural interpretation of post-earthquake (19th century) retrofitting on the Santa Maria degli Angeli Basilica, Assisi, Italy

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ABSTRACT: A sequence of ground shakings occurred from October, 1831 to March, 1832 in Central Italy. Due to the high intensity and the repetition of the events, the Basilica of Santa Maria degli Angeli, Assisi was damaged severely, suffering the collapse of the vaults of central and left naves. After the first emergency interventions, engineer-architect Luigi Poletti (1792–1869) was appointed to design the retrofitting and restoration works. From an architectural point of view the works carried out by Poletti were rather respectful, but from a structural point of view they introduced interesting innovations. Contrary to the collapsed construction, Poletti rebuilt the roof over the central nave with an original raised tie beam and iron braced truss, not resting on the below vault, while the vault had now a tapered transversal section. Poletti’s solution are considered from a structural point of view, performing numerical simulations by means of limit analysis and finite element models, with reference to his original drawings, to previous and following earthquake damages, and to contemporary technical literature.

1 INTRODUCTION

On March 15, 1832, after at least six severe shaking which took place in the previous months, the vaults over the central and left naves of the Basilica of Santa Maria degli Angeli, Assisi (Central Italy) were collapsed. The event had a huge impact in Italy, due to great importance of the church.

Such importance is related to a small chapel called “La Porziuncola”, where Saint Francis discovered his vocation and founded the Order of Friars Minor, and with the neighbouring “Cappella del Transito”, where according to the tradition Saint Francis passed away.

The Basilica which currently shrines such significant testimonies is one of the biggest Catholic churches. As a matter of fact, it is approximately 126 m long, 55 m wide and the top of its lantern is about 75 m tall. The building has three naves separated by four pillars on each side (Figure 1). The central nave is covered by a barrel vault with a clear span of roughly 18.5 m. On each bay of the lateral naves are five cross vaults which correspond to five chapels. The transept and the long choir are covered by barrel vaults as well. At the intersection of central nave and transept is the “Porziuncola” chapel sheltered by a dome whose internal diameter is approximately 20 m. A bell tower is on the right side of the choir, while there is an open gallery between the body of the building and the façade.

The construction of the Basilica, without the extant façade, required almost 120 years due to both its size and the poverty of the Friars. Although during such a time span no severe earthquake occurred, thereafter many earthquakes took place.

2 CONSTRUCTION HISTORY AND SEISMIC DAMAGES

2.1 Construction history of Santa Maria degli Angeli

The construction history of the Basilica of Santa Maria degli Angeli has been dealt with in several previous works (refer e.g. to di Boveglio & d’Isola Maggiore 1834, Perilli 1842, Vignoli 1989, and references therein). Therefore, only the most significant
phases of its construction history are reported here and neglecting the information about the convent.

After the death of Saint Francis (1226) only small buildings, a refectory, some dormitories, a chapel, a choir, an infirmary, were built in the area.

In 1568 Pope Pius V decided to construct an adequate Basilica, because of the importance of the site and because of the mass of pilgrims already attracted. The Friars asked the Perugia (Central Italy) architect Galeazzo Alessi to provide a model for the design of the new building which he did during the same year. The following year materials were supplied to the site and the prominent architect Jacopo Barozzi da Vignola visited the place, probably as design supervisor on behalf of the Pope. On March 25th the church was officially founded. Alessi delivered a plan of the building which he did during the same year. The following year materials were supplied to the site and the prominent architect Jacopo Barozzi da Vignola visited the place, probably as design supervisor on behalf of the Pope. On March 25th the church was officially founded. Alessi delivered a plan of the building which he did during the same year.

After five years the foundations were completed and it took twenty years to finish the masonry work of the three naves and related chapels. In 1593 their wooden roof was built (p. 111). During the first decade of the 17th century three pillars of the dome were built. It was in 1622 that the construction of the church restarted slowly in the choir. In 1637–40 the fourth pillar was erected, and in between 1644 to 1646 three main arches under the dome and the choir vault were completed. In 1645–48 the right transept was built, but the left one was raised in 1650–52. Even though the construction of the dome started in 1662, just a year later was stopped by a harsh controversy (both monetary and technical) between the master builder and the Friars. The works restarted between 1668 and 1674, when they were stopped due to an out-of-plumb of the façade. The damage was ascribed to the too limited thickness of its upper section, and the void created by the stairs in the lower one. Between 1674 and 1675 the façade was strengthened and partially rebuilt. Only after that the works on the dome started again and were completed on September 25, 1677. Then, the bell tower was built in 1679–85, thus completing the church with the exception of the façade (Bartelli 1989).

2.2 Earthquake damages

The first information about earthquake damages in the church date to the April 17, 1747 earthquake (VII MCS felt intensity, Guidoboni et al 2007): a survey reported cracks in the choir, damages at the naves pillars, out-of-plumb of the façade (Boschi et al 1998, 81 and 86).

On July 27, 1751 another earthquake (VI MCS, Guidoboni et al 2007) hit the Basilica again in the choir (Boschi et al 1998, 62). It is possible that is was after these earthquakes that the lantern was looped by means of iron ties at three different levels (Menegotto 1993 and 2003).

Generic slight damages are reported for the July 28, 1799 earthquake (Boschi et al 1998, 122), felt with a VI MCS intensity (Guidoboni et al 2007).

A very severe seismic sequence struck the Basilica between 1831 and 1832 (di Boveglio & d’Isola Maggiore 1834, ch. 14; Perilli 1842, p. 6–7; Vignoli 1898, 147–153; Boschi et al 1998, 150–151). On October 27 the main door and the small cupola (on top of the main dome) were damaged. On November 6 such damages were worsened. The January 13 and 27, 1832 and the March 13 shakes cracked the vaults and the pillars on the naves. Finally, the March 15 event caused the collapse of the vaults of the main and the left naves, as well as of the pillars between the two (Figure 2). The right nave, although not crumbled, was damaged badly, and probably survived because two of its pillars have been previously looped with iron and timber. On the contrary, the dome was not severely stricken.

The first emergency interventions, under the direction of engineer Antonio Mollari, were the reinforcement of the survived pillars, through iron and timber, and the construction of wooden pyramid for the protection of the “Porziuncula”.

On August 11, engineer architect Giuseppe Brizzi was asked to advice for permanent interventions. Among other secondary indications, he suggested to rebuild the roof on trusses not resting on the vault,
although this would have reduced the drum of the dome.

On August 22 engineer architect Luigi Poletti (1792–1869) is first mentioned. Poletti was a pre-eminent practitioner at that time (Dezzi Bardeschi et al 1992) and during his professional lifetime was frequently involved in interventions after catastrophic events (Reale et al 2004).

Poletti recommended to shore the choir’s vault, the right nave’s arches, and the three arches between the naves and the transept.

Between 1832 and 1835 the master builder appointed to execute his instructions added iron rings to the dome, repaired two pillars in the right nave and tear down the upper section of the façade. The interventions on the two pillars were not faultless. According to Perilli (1842, 5) the original masonry of the pillars was a rubble core one, with a thin external brick masonry skin, and the master builder did simply repair them. The description of the construction technique is very interesting, since it may help to explain the severe damages observed in the building.

After many discussions in 1836 Poletti was confirmed in his role of designer and Mollari was appointed as supervisor of the execution. Between March and September of the same year the right nave was torn down and rebuilt (with solid brick masonry, Perilli 1842, 8) up to the arches, while the vault was completed between April and June 1837. Between July and October 1836 the demolition of the top of the façade was completed.

The central nave vault was rebuilt in 1838. The construction of the roof took place in 1839. Finally, the façade was partially rebuilt between March and August, 1840.

On February 12, 1854 another earthquake hit the area (VII felt intensity in Assisi, Guidoboni et al 2007). The Basilica suffered cracks in the choir apse, in the transept, in the cupola, in the lateral naves, and in some chapels. The dome and the main nave were only slightly damaged (Boschi et al 1998, 164). Architect Giovan Battista Tiberi suggested extensive use of iron ties (Boschi et al 1998, 165), especially in the lateral naves and in the chapels. His proposals have been partially carried out, avoiding the visible ties, and locating them in the attic (refer to pictures in Lunghi & Lunghi 1989, 194–198; and description in Boschi et al 1998, 166).

Shear and spalling cracks occurred in the lantern above the dome by the April 19, 1984 earthquake (Boschi et al 1998, 214), whose felt intensity was VII MCS (Guidoboni et al 2007). Such damages induced careful provisional and permanent interventions (Menegotto 1993 and 2003), which have lived through the following seismic event.

The 1997 Umbria-Marche seismic sequence has also damaged the Basilica: the façade (rebuilt for architectural reasons during the 20th century) rotated slightly, cracks were detected on the lateral naves vaults and at the junction between naves and dome (refer to: Capalbini et al 1999, where also a complete geometric survey of the basilica is available).

Although from an architectural point of view, Poletti restoration was substantially faithful to the 16th century design (Perilli 1842, 7; Vignoli 1989, 152), from a structural point of view he introduced important innovations which will be discussed in the next section. It is important to stress that the portions reconstructed under his supervision survived all the subsequent events with only minor damages (Boschi et al 1998, 151). As will be shown, the strategy pursued by Poletti was not to recur to special earthquake-resistant solutions. Instead he tried to reduce the vulnerability of the edifice first of all by means of rule-of-art construction techniques, such as solid brick masonry instead of rubble core masonry. Secondly he saved weight on vault. Finally, he reduced the thrust exerted by horizontal structures, both vault and roof.

3 STRUCTURAL EVALUATION OF THE INTERVENTIONS

3.1 The tapered vault

Unfortunately there is no graphical evidence about the construction shape of the pre-1832 vault. However, in di Boveglio & d’Isola Maggiore (1834, pl. 7; Figure 2) the mark of vault with the triumphal arch has a constant thickness. Moreover, Perilli (1842, 6) explicitly states that the previous vault had a constant thickness.

Poletti’s drawing dated July 19, 1836 shows the cross section of his new vault (Figure 3). Although this is of some interest, it has dragged little attention
so far. Poletti designed a tapered vault (i.e. a vault with non constant thickness). The actual vault geometry is slightly different from the one originally drawn (Lunghi & Lunghi 189, 194–195). The vault has a crown thickness of 32 cm, and (through three 16 cm steps) a haunch thickness of about 80 cm. Therefore, the span (at springers) to crown thickness ratio is about 57.7, the span (at haunches) to crown thickness ratio is about 45.7, while the crown to haunch thicknesses ratio equals to 2.5.

It will be shown that such a profile has several advantages and it is therefore legitimate that Poletti purposely pursued it in order to reduce the vault’s thrust, considered among the reasons of the 1832 collapses.

There is no much attention in the historical literature to tapered arches or vaults. Probably the first reference is due to architect Andrea Palladio (1508–1580) in a 1567 appraisal on the new Cathedral of Brescia in Northern Italy (Puppi 1988, 123–125). Writing about the dome, Palladio states that the thickness should grow from the lantern to the drum in order to reduce the load at mid-span. Such thickness reduction is obtained by using two spheres, of diverse radii, with their centres at different heights. According to Huerta (2004, 200–201), Palladio used this method to design all the domes in his treatise (Palladio 1570), and the same was done previously by architect Sebastiano Serlio (1475–1554). Palladio’s appraisal was reported by Zamboni (1778) and later by Rondelet (1832–1835, Book 9, Sec. 6, Ch. 4, Notes and Pl. 195), so it is possible that Poletti was aware of it.

Architect Carlo Fontana (1638–1714), in his famous treatise on the Vatican Temple (1694, 361–367) suggested a partially different rule for pointed domes, using four different centres, at the same height, in order to get a thickness decreasing from the lantern to the drum. This rule was based on the observation of several existing domes and on the exam of previous geometric rules (Huerta 2004, 272).

The first reference to tapered arches and barrel vaults is probably due to the French military engineer Amédée-François Frézier (1682–1773). In his treatise (1737–1739, 2: 87) he describes a method to draw arches of non constant thickness based on the first scientific theories on arches due to La Hire, Parent and Couplet (for such theories refer e.g. to: Benvenuto 1991, 321–326, 331–336, 338–344). Frézier’s method is rather similar to that by Serlio and Palladio, since the intrados and extrados curves are obtained considering two centres of different height. Once the crown thickness has been established, their positions are fixed either by fixing the springer thickness or by lowering the centre of the intrados curve by a fraction of its radius. In Frézier’s examples the springer/crown thickness ratio equals 3 and extrados/intrados radius ratio equals 7/6. The French engineer gave also rules for vaults bearing heavy loads (e.g. bridges), vaults bearing small loads (e.g. vaults loaded by some wooden truss), and vaults subjected to self weight. In the latter case he recommends a thickness at the crown equal to 1/4 of the span, a thickness which should be doubled in the 30° section close to the haunches (2: 96–97).

Figure 3. Cross section highlighting the tapered vault, dated July 19, 1836 (BCALPM 1836, detail). Courtesy of: Biblioteca Civica d’Arte “Luigi Poletti”, Modena, Italy.
In 1748 in an unpublished manuscript (Huerta 2004, 358–360) the French engineer Jean Rodolphe Perronet (1708–1794) suggested to design bridges with a thickness doubling from the crown to the haunches. In the case of semi-circular arches the crown thickness should be equal to 5/144 of the span plus 1 foot. A rather similar rule was suggested by the French Engineer Bernard Forest de Bélidor (1697–1761) in Frézier’s and Perronet’s approach was followed by the French architect Jean Baptiste Rondelet (1734–1829) in his famous treatise (French edition 1812–1817, Italian translation 1832–1835). Therefore, he recommends to design arches of non constant thickness by lowering the centre of the extrados curve (Pl. 27 and Book 9, Sec. 6, Ch. 1, Art. 1), and to differentiate the crown thickness as a function of the span and of the type of live load (heavy, medium and zero; Book 3, Sec. 3, Ch. 1). However, also based on some experiments he performed, the thicknesses suggested are sometimes much smaller. For the case at hand, and considering an average stone, he suggests a thickness of 25 cm at the crown which should be doubled at the haunches, by means of a linear increase. Moreover, Rondelet clearly states that a tapered vault is much more convenient than a constant-thickness one. As a matter of fact, he writes that a semi-circular arch of constant thickness subjected to its self-weight needs a thickness equal to 1/17 of the span, while a tapered one needs a crown thickness which is one fifth of the previous (ibidem). Furthermore, he declares that a tapered vault will exert a much lower thrust of the abutments than a constant thickness one of equal span (Book 9, Sec. 6, Ch. 1, Art. 2; and ibidem Ch. 3). In this last chapter Rondelet gives tables for the crown thickness of semicircular vaults with horizontal extrados, with solid infill up to half height and thickness constant or tapered. The crown thickness of the constant one is one third bigger than the crown thickness of the variable vault.

Rondelet’s approach had a great fortune, as evidenced by other widespread manuals of 19th century (Breymann 1885, Vol. 1, Ch. 8, Sec. 9 and Pl. 100).

The vaults drawn by Rondelet or Breymann are meant to have a smooth thickness increase. Such a result can be obtained if the vault is meant to be made of cut stones. However, if bricks are used (as in Santa Maria degli Angeli) it is easier to have discrete steps in the cross section. As far as it is known to the authors, such case is rarely considered in the literature. Körner (1895, 286) recommends to avoid such a design, since the thrust line will be very close to the extrados of central (thinnest) segment, while cracks and bulging will appear on the two sides of the vault close to the first thickness variation. As Rondelet and Breymann he endorses an even growth of the cross section.

In order to better understand such favour for tapered vaults as well as Poletti’s design, his vault is analysed here by means of the safe theorem of limit analysis (Heyman 1966). Two lines of thrust are drawn, minimising either maximum top and bottom stresses or the thrust exerted on the haunches (Méry 1840). In the second case maximum compression stress was assumed 2.2 MPa (OPCM 2005). Three different geometries are considered: (1) Poletti’s, (2) constant thickness \( t = 0.54 \text{ m} \) giving a vault of same weight, (3) constant thickness 1/3 bigger than Poletti’s crown thickness \( t = 0.43 \text{ m} \), thus following Rondelet’s abovementioned tables. Two load cases are taken into account: (1) self-weight only, (2) self-weight plus a 18.5 kN load applied at mid span over a 0.5 m length (representing the load of the roof resting on the vault, whose load-per-unit-area has been estimated equal to 2.0 kN/m²).

The analyses are performed with a computer code, drawing the aforementioned thrust lines and the associated stress fields (Dr. Cesare Tocci, personal communication). Sample outputs are presented in Figure 4 and Figure 5.

In Table 1 is presented the comparison between Poletti’s vault and the two vaults of constant thickness. In terms of optimal stress Poletti’s solution grants a 3 geometrical safety coefficient (Heyman 1982, 24) with a 62.0 kN thrust. The 0.54 m constant thickness vault gives the same geometrical coefficient with a much higher, 74.6 kN, thrust.

The 0.43 m constant thickness exerts a little smaller horizontal force, 62.0 kN, but with a much lower, 2.5 geometrical coefficient.

In terms of minimum thrust, Poletti’s vault gives again the best value, 53.6 kN, with the 0.43 m vault granting a similar 54.6 kN. However, such figure is also due to the lower self-weight. Therefore, the ratio between horizontal / vertical components is 0.64 in the...
Table 1. Analysis of Poletti’s tapered vault compared to two constant thickness vaults. Self weight only considered.

<table>
<thead>
<tr>
<th>Minimum</th>
<th>Poletti</th>
<th>t = 0.54 m</th>
<th>t = 0.43 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress</td>
<td>3.0</td>
<td>3.0</td>
<td>2.5</td>
</tr>
<tr>
<td>Thrust</td>
<td>1*</td>
<td>1*</td>
<td>1*</td>
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</tbody>
</table>

Geometrical safety coefficient

<table>
<thead>
<tr>
<th>Maximum compression stress (MPa)</th>
<th>Geometrical safety coefficient</th>
<th>Minimum</th>
<th>Poletti</th>
<th>t = 0.54 m</th>
<th>t = 0.43 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical reaction at haunch (kN)</td>
<td>Maximum compression stress (MPa)</td>
<td>Stress</td>
<td>Thrust</td>
<td>Stress</td>
<td>Thrust</td>
</tr>
<tr>
<td>83.3</td>
<td>0.28</td>
<td>2.2</td>
<td>0.32</td>
<td>2.2</td>
<td>0.36</td>
</tr>
<tr>
<td>83.3</td>
<td>0.36</td>
<td>2.2</td>
<td>67.0</td>
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</tr>
<tr>
<td>1.36</td>
<td>1.1</td>
<td>2.2</td>
<td>76.3</td>
<td>76.3</td>
<td></td>
</tr>
<tr>
<td>54.6</td>
<td>1.36</td>
<td>2.2</td>
<td>76.6</td>
<td>76.6</td>
<td></td>
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</table>

*Conventional value.

The overall performance of Poletti’s solution is thus much more satisfactory than the other two, hence justifying both his design and the approval of tapered arches in the technical literature.

3.2 The roof truss

The design of the roof, which Poletti performed in September 1838, is a very good example for conflicting architectural and structural. The former must have inspired the original design characterised by a lower roof resting on the vault in order to let the dome tower over the entire edifice. Structural motivations, strongly backed by observed earthquake damage, suggested to recur to a truss leaving the vault clear and reducing the thrust exerted on the walls.

About his final solution, Poletti stated proudly that “[…] the roof truss [has] polygon shape of my own invention, sound as the triangular, and with the two achievements of not touching the vault and not raising too much […]” (Vignoli 1989, 150). Poletti portrayed his roof in several drawings (inventories 547, 548, 550, 552, 553 at the Biblioteca Civica d’Arte “Luigi Poletti”, Modena), but there is only one dated (BCALPM 1838; Figure 6). Most probably inventories 547 and 553 were previous studies.

According to a document of the following year quoted by Vignoli (1989, 159) the original design of the truss was due to Antonio Rutili, a Foligno (Central Italy) engineer of the public waters prefecture who in 1833 strongly recommended Poletti’s retrofitting for the City Hall of Foligno (ASPSF 1833). However, Perilli (1842, 10) attributes the design to Poletti.

Poletti’s roof structure consisted of a king post truss with raised tie beam. As usual in the Italian tradition the king post does not touch the tie beam. Collar and principal rafters are met at their joint by an extended ashlar strut. Iron braces partially triangulate struts and tie beam. Poletti must has been aware of the need to provide a tension-resistant joint between raised tie beam and principal rafter. Therefore he decided to guarantee the tensile strength by mean of an iron tie parallel to the timber one.

Lunghi & Lunghi (1989) provide a detailed cross section of the naves, reporting the size of both timber and iron structural elements of the roof and interesting photos of the roof system. All timber elements (principal rafters, raised tie beam, additional struts) have a square cross section, with a 39 cm side.

The problem of a roof truss built on a vault which projects itself above the wall top, thus making the use of a tie beam impossible, has been dealt with at least since Middle Age (refer e.g. to Courtenay 1985).

Limiting the attention to 19th century, the topic of the problem of a roof truss built on a vault which projects itself above the wall top, thus making the use of a tie beam impossible, has been dealt with at least since Middle Age (refer e.g. to Courtenay 1985).

Breymann (1885, Vol. 2, Ch. 6, Sec. 6b) shows a roof with a raised tie beam connected, by means of additional braces, to the foot of the principal rafters. In a note by the Italian editor L. Mazzocchi such braces
are prolonged to the principal rafters, thus obtaining a scissors-braced truss.

Poletti’s roof solution seem to have dragged more attention than the vault one, although such interest has not always brought to a correct interpretation of its features.

According to Lunghi & Lunghi (1989) Poletti’s iron braces are useless, since “assuming on both sides hinges as restraints for the structure, this is fully compressed and therefore the braces are not reacting” (p. 193). Although not clearly stated by the authors, they seem to have assumed fixed restraints. Under such hypothesis it is true that the inclined iron elements are compressed. However, if a slight lateral sagging is assumed then this is not true anymore.

A similar phenomenon has been observed in the analysis of cross vaults. If they are examined assuming fixed restraints almost no tension is observed (Alexander et al 1977). However, if the supports can give way, elements previously compressed will be subjected to tension (Barthel 1989).

The roof is analysed here with a linear elastic finite element code (CSI 2004), under self-weight only (a 1.35 partial safety factor has been considered, for later code verifications), and assuming an increasing lateral sagging on both sides. Although the considered values are still rather small compared to the free span of the roof (18.45 m), Poletti’s iron braces are subjected to tension if the restraint gives way laterally 2 cm only. Such a lateral spread is not unrealistic both due to lack-of-fit problems, masonry local settling under the truss thrust, deformation of the unrestrained principal rafters ends. As a matter of fact, modelling such unrestrained end as a clamped inclined beam subjected to the whole vertical reaction of the masonry, the lateral deflection will be approximately 6.5 cm. It is possible that Poletti expected such a lateral spreading and, therefore, he added the braces. The presence of the braces is able to markedly reduce the horizontal thrust exerted by the truss (Table 3), a problem which Poletti must have taken account of, since the roof has been considered a reason of the collapse, and since this was the same reason of the different French, English and American solutions previously mentioned. Furthermore, the additional struts remove roughly 80% of the vertical reaction from the principal rafters, and thus reduce the risk of their excessive bending as well as they transfer the load much further down the wall.

It is interesting to note that as the sagging increases maximum bending moments in principal rafter, raised tie beam, and strut tend to become more and more similar (Table 3). Although he certainly did not perform explicit calculations, Poletti must have been aware that the braces will have induced deflections in the raised tie and in the struts. That is probably the reason why he gave to the three fir elements the same cross section. Finally, it is interesting to note that, according to the Italian standards (DT206 2006; UNI 11035-2: 2003), the design ultimate moment (capacity) in the case of permanent loads is equal to 128 kN m. Nonetheless, it is true that some of the principal rafters and of the extended ashlar struts had to be reinforced by means of L-shaped steel profiles (Lunghi & Lunghi 1989, 196–197).

4 CONCLUSIONS

Confronted with the need to retrofit a pre-eminent building while restoring (not reconstructing) Santa Maria degli Angeli Basilica (Assisi, Central Italy) after the severe seismic sequence of 1831–1832, architect Luigi Poletti did not to recur to special earthquake-resistant solutions. Instead he tried to reduce the vulnerability of the edifice first of all by means of rule-of-art construction techniques, such as solid brick masonry instead of rubble core masonry. Secondly he saved weight on the vault of the central nave and reduced its thrust assuming a tapered cross section. As has been shown in the paper, by means of numerical simulations of thrust lines and associated stress fields, such solution is more effective compared to uniform thickness ones, having the same weight or complying with recommendations of the contemporary technical literature.
Poletti tackled also the problem of the design of a roof not resting on the vault and not concealing the dome. Therefore, he designed a king post truss with raised beam, additional extended ashlar struts and iron braces. As shown in the paper, the problem of a roof truss built on a vault which projects itself above the wall top has been dealt with both in actual buildings and in the technical literature. Nonetheless, Poletti solution is original and effective, as proven in the paper by means of a finite element model. The additional struts reduce the vertical reaction at the foot of the principal, thus limiting the bending deflection, while the braces are effective in case of a lateral spreading of the structure and in reducing the thrust exerted on the walls.

More convincing than any numerical simulations, Poletti’s retrofitted elements survived with minor damages all the following earthquakes.

**REFERENCES**


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Table 3. Analysis of Poletti’s roof truss under self-weight. (WOB = Without Braces; WB = With Braces)

<table>
<thead>
<tr>
<th>Sagging cm</th>
<th>Thrust (kN)</th>
<th>Maximum Bending Moment (kN m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>WOB</td>
<td>WB</td>
</tr>
<tr>
<td>Principal rafter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Raised tie beam</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strut</td>
<td></td>
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</tr>
</tbody>
</table>

| 0          | 140 | 44.1 | 10.4 | 2.6 | 2.6 |
| 2          | 133 | 45.9 | 29.3 | 2.6 | 13.1 |
| 5          | 121 | 68.9 | 71.7 | 2.6 | 38.6 |


