

# Recommendations for Restoration of Ancient Buildings and the Survival of a Masonry Chimney

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## Abstract

In the last years significant developments occurred with respect to the possibilities of experimental and numerical analysis of ancient cultural heritage buildings. Such advances result from the societal growing concern about the preservation of this heritage, together with the evident cultural and economic importance of this activity. Recommendations for interventions in architectural heritage structures recently issued by ICOMOS are briefly reviewed here, with a discussion about the proposed methodology. Then, a case study is fully detailed, including the aspects of survey, advanced numerical analysis, diagnosis, justification of remedial measures and detailing of the adopted strengthening. The paper also advocates that significant information can be obtained from numerical analysis, namely with respect to the understanding of existing damage and to the minimum and adequate design of strengthening. A clear understanding of the structural behaviour, based on sophisticated tools of analysis, can therefore reduce the extent of the remedial measures in the restoration of ancient structures. In this case, a masonry chimney with severe damage is strengthened keeping the old fabric, as a significant example that ancient timber and masonry structural elements can often survive if appropriate methodologies for diagnosis and repair measures are adopted.

## Keywords

Masonry; Recommendations; Historical structures; Structural analysis; Modelling; Strengthening

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# 1 Aims

Recently, Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage have been approved by ICOMOS [1]. These Recommendations are intended to be useful to all those involved in conservation and restoration problems and not exclusively to the wide community of engineers. A key message, probably subliminal, is that those involved in historic preservation must recognise the contribution of the engineer. Often engineering advice seems to be regarded as something to be sought at the end of a project when all the decisions have been made, while it is clear that better solutions might have been available with an earlier engineering contribution.

An issue related with this message is that conservation engineering requires a different approach and different skills from those employed in designing new construction. Often historic fabric has been mutilated or destroyed by engineers who do not recognise this fact, with the approval of the authorities and other experts involved. Moreover, even when conservation skills are employed, there are frequent attempts by regulating authorities and engineers to make historic structures conform to modern design codes. This is generally unacceptable because the codes were written with quite different forms of construction in mind, because it is unnecessary and because it can be very destructive of historic fabric.

The need to recognise the distinction between modern design and conservation is also of relevance in the context of engineers' fees. The usual fee calculation based on a percentage of the cost of the work specified is clearly inimical to best conservation practice, when the ideal is to avoid any structural intervention if possible. Being able to recommend taking no action might actually involve more investigative work and hence more cost to the engineer than recommending some major intervention.

Modern intervention procedures require a thorough survey of the structure and an understanding of its history. Any heritage structure is the result of the original design and construction, any deliberate changes that have been made and the ravages of time and chance. An engineer working on historical buildings must be aware that much of the effort in understanding their present state requires an attempt to understand the historical process. The engineer involved at the beginning of the process might not only have questions that can easily be answered by the archaeologist or architectural historian, but he might be also able to offer explanations for the data being uncovered.

Thus, a first aim of the present paper is to stress the role of engineering in the conservation of historical structures and the fact that an engineer, with specific knowledge in the field, must be involved from the *beginning* in the team of experts associated to the process.

The analysis of ancient constructions poses indeed important challenges because of the complexity of their geometry, the variability of the properties of traditional materials, the different building techniques, the absence of knowledge on the existing damage from the actions which affected the constructions throughout their life, and the lack of applicable codes. In addition, restrictions in the inspection and the removal of specimens in buildings of historical value, as well as the high costs involved in inspection and diagnosis, often result in limited information about the internal constructive system or the properties of existing materials. These aspects call for qualified analysts that combine advanced knowledge in the area and engineering reasoning, as well as a careful, humble and time-consuming approach. In particular, it is noted that significant advances occurred in the last decade concerning the development of adequate tools for the numerical analyses of historical structures [2].

Therefore, a second aim of the paper is to present a real case study of a masonry structure with severe damage and major constraints on strengthening possibilities. The structure is a chimney from the 18<sup>th</sup> century located in the main kitchen of the Monastery of Arouca. The Monastery is located in the North of Portugal and was founded in the 10<sup>th</sup> century. In the 13<sup>th</sup> century, the monastic community embraced the feminine Cistercian rule. The compound was significantly expanded with major works in the 17<sup>th</sup> and 18<sup>th</sup> centuries.

The works detailed here included a preliminary survey of the geometry, of the damage and of the structural system. Afterwards, an advanced computer simulation of the behaviour of the structure was carried out using a three-dimensional non-linear finite element analysis software. The diagnosis that resulted from the observation and the numerical analysis itself allowed to propose two remedial solutions, one based in keeping the old fabric and another based in the partial replacement of the old fabric. The final adopted solution is justified, and details about the design and execution are also provided.

## **2 Review of ICOMOS Recommendations [1]**

Structures of architectural heritage, by their very nature and history (material and assembly), present a number of challenges in conservation, diagnosis, analysis, monitoring and strengthening that limit the application of modern legal codes and building standards. Recommendations are desirable and necessary to ensure rational methods of analysis and repair methods appropriate to the cultural context.

Therefore, the International Scientific Committee for the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH) has prepared recommendations [1], intended to be useful to all those involved in conservation and restoration problems. These recommendations contain Principles, where the basic

concepts of conservation are presented, and Guidelines, where the rules and methodology that a designer should follow are discussed. More comprehensive information on techniques and specific knowledge can be found, e.g. [3-6]. In addition, normative and pre-normative are gradually becoming available, e.g. [7-9], at least with respect to seismic rehabilitation, which is a major concern.

## **2.1 Principles**

A multi-disciplinary approach is obviously required in any restoration project and the peculiarity of heritage structures, with their complex history, requires the organisation of studies and analysis in steps that are similar to those used in medicine. Anamnesis, diagnosis, therapy and controls, corresponding respectively to the condition survey, identification of the causes of damage and decay, choice of the remedial measures and control of the efficiency of the interventions. Thus, no action should be undertaken without ascertaining the likely benefit and harm to the architectural heritage.

A full understanding of the structural behaviour and material characteristics is essential for any project related to architectural heritage. Diagnosis is based on historical information and qualitative and quantitative approaches. The qualitative approach is based on direct observation of the structural damage and material decay as well as historical and archaeological research, while the quantitative approach requires material and structural tests, monitoring and structural analysis. Often the application of the same safety levels used in the design of new buildings requires excessive, if not impossible, measures. In these cases other methods, appropriately justified, may allow different approaches to safety.

Therapy should address root causes rather than symptoms. Each intervention should be in proportion to the safety objectives, keeping intervention to the minimum necessary to guarantee safety and durability and with the least damage to heritage values. The choice between “traditional” and “innovative” techniques should be determined on a case-by-case basis with preference given to those that are least invasive and most compatible with heritage values, consistent with the need for safety and durability. At times the difficulty of evaluating both the safety levels and the possible benefits of interventions may suggest “an observational method”, i.e. an incremental approach, beginning with a minimum level of intervention, with the possible adoption of subsequent supplementary or corrective measures.

The characteristics of materials used in restoration work (in particular new materials) and their compatibility with existing materials should be fully established. This must include long-term effects, so that undesirable side effects are avoided.

Finally, a most relevant aspect is that the value and authenticity of architectural heritage cannot be assessed by fixed criteria because of the diversity of cultural backgrounds and acceptable practices.

## **2.2 Guidelines**

A combination of both scientific and cultural knowledge and experience is indispensable for the study of all architectural heritage. The purpose of all studies, research and interventions is to safeguard the cultural and historical value of the building as a whole and structural engineering is the scientific support necessary to obtain this result. The evaluation of a building frequently requires a holistic approach considering the building as a whole, rather than just the assessment of individual elements.

The investigation of the structure requires an interdisciplinary approach that goes beyond simple technical considerations because historical research can discover phenomena involving structural issues while historical questions may be answered from the process of understanding the structural behaviour. Knowledge of the structure requires information on its conception, on its constructional techniques, on the processes of decay and damage, on changes that have been made and finally on its present state.

The recommended methodology for completing a project is shown in Figure 1, where an iterative process is clearly required, between the tasks of data acquisition, structural behaviour, and diagnosis and safety. In particular, diagnosis and safety evaluation of the structure are two consecutive and related stages on the basis of which the effective need for and extent of treatment measures are determined. If these stages are performed incorrectly, the resulting decisions will be arbitrary: poor judgement may result in either conservative and therefore heavy-handed conservation measures or inadequate safety levels. Evaluation of the safety of the building should be based on both qualitative (as documentation, observation, etc.) and quantitative (as experimental, mathematical, etc.) methods that take into account the effect of the phenomena on structural behaviour. Any assessment of safety is seriously affected by the uncertainty attached to data (actions, resistance, deformations, etc.), laws, models, assumptions, etc. used in the research, and by the difficulty of representing real phenomena in a precise way.

The methodology stresses the importance of an “Explanatory Report”, where all the acquired information, the diagnosis, including the safety evaluation, and any decision to intervene should be fully detailed. This is essential for future analysis of continuous processes (such as decay processes or slow soil settlements), phenomena of cyclical nature (such as variation in temperature or moisture content) and even

phenomena that can suddenly occur (such as earthquakes or hurricanes), and for future evaluation and understanding of the remedial measures adopted in the present.

### **3 Survey of the Case Study Structure**

#### **3.1 General Description**

The chimney is made of three brick masonry walls, making a tapered channel section leaning against a thick masonry wall from the Monastery envelope, supported by three stone granite lintels, see Figure 2. The maximum cross section of the chimney (at the base) is  $4.3 \times 4.1 \text{ m}^2$  and the minimum cross section of the chimney (at the top) is  $0.28 \times 3.8 \text{ m}^2$ , for a height of 16.4 m. The cross section of the side lintels is  $0.33 \times 0.46 \text{ m}^2$  and the cross section of the main lintel is  $0.31 \times 0.52 \text{ m}^2$ . The lintels are supported in stone columns and corbels. Hidden by the plaster, internal brick arches with 0.25 m thickness were found, see Figure 2a,c. The function of these elements is obvious, aiming at reducing the bending / tying load of the stone lintels by transmitting part of the load from the walls directly to the columns.

In order to uncover the hidden structure of the arches, four inspection openings were made in the plaster, see Figure 3. It could be observed that the rubble masonry fill between the arches and the lintels was not separated from the arches, as expected from good building practice. Two other defects were found, namely: (a) the main arch is asymmetrical with respect to the lintel, see Figure 2a; (b) the cross section of the left side lintel has a tapered shape, with a height reduction towards the external wall support, see Figure 2c.

The structure of the chimney is complemented by a set of iron ties, distributed along the height and inside the chimney. These ties aim at stabilizing the main wall, which is inclined about  $15^\circ$  with respect to the vertical position. Finally, two iron ties are also present inside the chimney, at the column corners and aligned at  $45^\circ$  with the lintels, see Figure 3. These ties are part of the original fabric and it is likely that their function was to help resisting the thrust of the system of arches / lintels. It is noted that an iron cramp was added to connect externally the main lintel and the left lintel.

#### **3.2 Observed Damage**

The main damage exhibited by the chimney consists of a sudden diagonal crack, which appeared suddenly in the main lintel, close to the right support, and resulted in temporary propping of the structure, see Figure 4a. This crack intercepts the anchoring zone of the  $45^\circ$  iron tie that connects the main lintel and the right lintel, which is a singular and weaker part of the lintel. It is also noted that this tie is corroded close to the

anchoring zone, see Figure 4b. The main lintel exhibits also several cracks close to the left support and in the vicinity of the iron cramp, which is severely corroded, see Figure 4c. In the masonry wall above the main lintel, a set of diagonal cracks is present, see Figure 4a. The inspection openings in the plaster indicate that the cracks do not intercept the hidden masonry arch, but they run through the arch extrados, see Figure 5a. This set of cracks represents significant danger and a pre-collapse situation of the left support, with a failure mechanism involving rotation of the wall with a hinge forming at the right support.

It is noted that an ancient crack is also present in the left side linter and wall, probably due to the reduction of height of the stone lintel in the corbel region, as discussed above, see Figure 4d and Figure 5b. As a result of this crack, two stone columns under each side lintel were added to the structure in an unknown date.

## 4 Structural Analysis

In order to complete the diagnosis and safety evaluation, two three-dimensional models of the chimney were prepared aiming at simulating extreme possibilities, which take into account the fact that there is notseparation between the arches and the material filling the space between the arches and the lintels. The two extreme possibilities regarding the arching effect in the walls are considered here. Therefore, the first model does not include the filling material under the arches (Model 1) and the second model considers that the walls are fully supported in the lintels (Model 2), see Figure 6. Obviously, Model 2 is more unfavourable for the stone lintels, being the most conservative approach.

In Model 1 the asymmetric location of the masonry arch has been considered in the simulation. The reduction of the cross section in the left side lintel was not considered, in order to reduce the complexity of the geometry, but the consequences of this simplification will be considered in the analysis of the results. The geometry of the columns was also considered constant, and equal to a square of  $0.40 \times 0.40 \text{ m}^2$ . It is shown later in the paper that the average stresses in the columns are very low and the adopted geometric simplification is negligible.

The models are made of quadratic solid finite elements (bricks and wedges), with approximately 500 elements and 3805 nodes, making a total of 11415 degrees of freedom. The constitutive models, loads and boundary conditions are the same for both models. More comprehensive results of the simulation are given in [10].

## 4.1 Constitutive Model

Different data has been considered for the brick masonry and granite stone. The adopted constitutive model is based on a traditional smeared crack model in tension, specified as a combination of tension cut-off (two orthogonal cracks), tension softening with an exponential law and shear retention, being the reader referred to [11] for further reading. In compression, a simple Mohr-Coulomb material model was used as compression cut-off with elastic-perfectly plastic behaviour. More sophisticated models can be used, e.g. [12], but coring and expensive testing would be required to find the necessary experimental data.

The values assumed for the material data are given in Table 1 [13,14]. Here,  $d$  is the density,  $E$  is the elasticity modulus,  $\nu$  is the Poisson coefficient,  $f_t$  is the tensile strength,  $G_f$  is the fracture energy,  $\beta$  is the shear retention factor and  $f_c$  is the compressive strength. It is noted that the properties of the granite in the lintels and columns are similar, even if they have been considered different in the analysis. Due to the fact that the lintels are simply supported in the columns, the tensile strength of the columns has been considered equal to zero. This way, no tensile stresses can appear in the connection between the lintels and the columns.

## 4.2 Loading and Supports

Two loads have been considered in the analysis, one due to the self-weight of the chimney and another due to the load transmitted by the vault of the kitchen (self-weight and live load). The density of the materials has been given in Table 1. The value for the action of the vault on the chimney has been obtained taking into account the tributary area and reads 124 kN, distributed along the two side walls.

For the numerical simulation, only the box section of the chimney has been considered. The connection with the external wall is not included in the model. Given the considerable stiffness of the external wall, the horizontal nodes of the connection to the external wall were constrained to zero. The base of the columns was also fully supported, whereas the ends of the side lintels were restrained vertically.

## 4.3 Elastic Results

### 4.3.1 Model 1 – With internal arching system

For Model 1, the elastic results in terms of maximum and minimum principal stresses are given in Figure 7 and Figure 8 for the brick masonry part of the model and the stone columns and lintels, respectively. With respect to the brick masonry, the results indicate that the maximum stresses, both in tension and



compression, occur close to the supports and key of the side arches and also in the vicinity of the support from the main arch (due to the arch asymmetry). The maximum values found, even if very localised, clearly exceed the strength assumed for the materials, with a maximum of  $0.6 \text{ N/mm}^2$  in tension (three times larger than the tensile strength) and  $1.7 \text{ N/mm}^2$  in compression (70% larger than the compressive strength). The maximum average values in the most severe locations are much lower, typically  $0.3 \text{ N/mm}^2$  for tension and  $0.9 \text{ N/mm}^2$  in compression. Figure 7b illustrates also the reduction in the compressive stresses in the span of the arches due to the arching effect of the masonry wall.

With respect to the set lintels-columns, see Figure 8, the maximum tensile stresses are found in the side lintels, close to the supports of the arches, and in the lower surface of the main lintel, being clearly higher in the left part of the lintel due to the asymmetry of the upper masonry arch. Such tensile stresses result from the bending of the lintels. Nevertheless, it is noted that the side lintels exhibit approximately uniform distributions of stresses through the thickness (uniaxial bending), whereas the main lintel exhibits a stress distribution that varies significantly through the thickness (biaxial bending), due to torsion induced by the slope of the main wall. The tensile stresses found are moderately high, but their maximum corresponds only to 80% of the tensile strength of the granite. The compressive stresses reach a value of  $3.5 \text{ N/mm}^2$  in the column-lintel connection. This value is very low, when compared to the compressive strength of the stone ( $60 \text{ N/mm}^2$ ).

### **4.3.2 Model 2 – Without internal arching system**

For Model 2, the elastic results in terms of maximum and minimum principal stresses are given in Figure 9 and Figure 10 for the brick masonry part of the model and the stone columns and lintels, respectively. In this case, the peak obtained for the tensile principal stresses is doubled due to the inexistence of the internal arching system. The maximum tensile principal stresses are located close to the supports of the lintels. With respect to the compressive stresses in the brick masonry part, the highest values are located above the mid-span of the lintels, where values close to the compressive masonry strength are reached. High compressive stresses are also found close to the supports.

With respect to the set lintels-columns, see Figure 10, the maximum tensile stresses are found in the span of the three lintels, with peak values of  $2.6 \text{ N/mm}^2$ . High tensile stresses are also found at the back supports of the side lintels, in the connection with the external wall. The zones with higher compressive stresses are close to the supports of the lintels and at the columns base but the maximum values are localised and moderate, reaching only about  $6.2 \text{ N/mm}^2$ .

## 4.4 Non-Linear Results

The linear elastic results indicated that the model without the internal arching system leads to the most unfavourable loading conditions, as expected. Therefore, this model has been selected for performing a non-linear analysis up to the collapse of the structure. The non-linear system of equations following from the finite element discretization was solved with an incremental-iterative globally convergent Newton-Raphson method with arc-length control and line-search technique [11]. Structural collapse was found for a load factor of 2.04, where the load factor represents the ratio between the applied loads and the original reference loads detailed in Section 4.2. For the ultimate load factor, the most damaged zones in the masonry walls occur close to the supports of the side lintels, see Figure 11. This damage occurs mostly in tension. Figure 12 illustrates the damage (measured by the maximum principal strains) for the lintels-columns set, which clearly defines the collapse mechanism. Three plastic hinges appeared in the side lintels, one hinge at mid-span with cracking at the lower face of the lintels (positive bending moments), and two hinges at the supports with cracking at the upper face of the lintels (negative bending moments). Figure 13 shows the force-displacement diagram for the mid-span of the side lintels, where the non-linear behaviour is clearly visible. The global response is approximately linear until a load factor of 1.0. Afterwards, a progressive non-linear response dominates until collapse, followed by a descending branch (softening regime) captured only with a reduction of the applied load. Obviously, in a real physical situation a load reduction would be impossible and the chimney would just collapse in an uncontrolled manner.

# 5 Diagnosis and Remedial Measures

## 5.1 Damage Diagnosis

The most relevant damage exhibited by the chimney is a diagonal crack in the main lintel, close to the right support. This crack intercepts the anchoring zone of an iron tie, which exhibits corrosion. The left support of the main lintel is also severely damaged with a long crack along the extrados of the internal arch in the wall. This pre-collapse situation is particularly dangerous as the propagation of the left crack leads to failure of the system. Besides other less severe damage, an old crack with considerable extension is also observed above the left side lintel.

The chimney presents some original constructive defects that are particularly relevant for the diagnosis, namely: (a) the internal arch in the main wall is asymmetric with respect to the lintel span; (b) the masonry material filling the space between the internal masonry arch and stone lintel is not separated from the wall;

(c) the cross section of the left side lintel is progressively reduced along the lintel span, with a minimum in the back support, which makes the connection with the external masonry wall.

The results of the structural analysis clearly indicate that the sudden collapse of the main lintel is not due exclusively for structural reasons, being probably triggered by corrosion of the tie that connects the main lintel and the right side lintel (here, it is noted that the environment is aggressive due to rising damp, salt and organic materials from the old kitchen activity). Certainly the local discontinuity associated with the hole for anchoring the iron tie also contributes to weaken the main lintel. The non-linear analysis carried out indicated that, in the most unfavourable conditions (without the internal arching action), the safety factor of the structure is 2.0 and, even then, possible collapse would occur in the side lintels and not in the main lintel.

The structural analysis results justify the ancient damage in the left side lintel. In fact, the numerical simulation does not take into account the cross section reduction in the left side lintel (the height varies linearly from 0.46 m to 0.30 m), which would reduce the safety factor significantly. This justifies the remedial measures adopted in the past by adding new stone columns close to the back supports of the side lintels.

Finally, the cracks and damage observed in the main lintel close to the left support are due to the corrosion of the iron cramp and also to the collapse of the right support, as the rotation of the main lintel was responsible for the long crack along the extrados of the internal arch in the main wall.

## **5.2 Alternative Repair Solutions**

The observed damage requires structural strengthening, which is rather complex due to the geometry of the column capitals and the fact that stone granite is a facing material. This means that any strengthening applied externally would become visible and would be aesthetically rather unappealing. The other preliminary issue discussed with the client (Portuguese Institute for the Architectural Heritage) was the issue of removing or keeping the two additional stone columns installed as a previous remedial measure (date unknown but before the 20<sup>th</sup> century). Conservation principles are often used as an absolute prohibition against the removal of earlier repair work. But often it is possible to find examples of repairs that are certainly old but do not constitute a valid part of the history of the structure (a typical example in timber structures, usually with no historical significance, is a prop added a 100 years ago as a measure to support a truss deteriorated by the action of beetles, abnormally high loading or just deficient original design). In the present case, it is believed that the additional columns can indeed be removed from historical reasons but the additional complexity and costs to further strengthen the chimney advise not to take this action. This has been the decision of the client.

Taking into consideration the above constraints and the absolute need of repairing the structure, two possible repair solutions have been proposed. The first solution consists of strengthening and repairing the main lintel including: (a) reconstitution of the original stone integrity by injection of epoxy resins; (b) hole drilling of the stone along its full length (4.70 m); (c) insertion of bars and injection of the hole. The second solution consists of replacing the original lintel by a new stone lintel, including: (a) propping of the complete chimney; (b) lifting the main wall with auxiliary centring of the internal arch; (c) replacement of the original stone by a new one, selected according to mechanical requirements and original granite lithotype.

In both solutions, the requirement that the chimney can be used for firing exceptionally is fulfilled. Also, stainless steel ties must replace the two iron ties that connect the main lintel with the side lintels, and the iron cramp applied in the left support of the main lintel must be removed. The rest of the ties along the chimney height must be depassivated, and protected against fire and corrosion. The left corner of the main stone lintel must be reconstituted and, finally, the crack in the left lintel and wall must be injected with epoxy resin and fluid lime mortar, respectively.

Taking into account the historical significance of the old lintel, the risk involved in the second solution of replacement and the preliminary cost estimates (the second solution is around 1.5 to 2.0 times more expensive), the solution of strengthening the original lintel with internal ties was adopted. Figure 14 illustrates various details of the solution, which includes two stainless steel rods with a diameter of 25 mm as internal ties / reinforcement of the granite lintel. The ties were designed after the integration of the tensile stresses of the linear elastic results for the numerical model without arching action, which is conservative. These rods are inserted in drilled holes of 50 mm and are provided with anchoring plates of 120 mm. After adjustment of the bolts, the drilled holes are injected with fluid lime mortar (Albaria Iniezione 200). Stone stoppers at both ends of the bottom tie are also included so that the anchoring plates are not visible. The stoppers are glued with epoxy resin and are made from the actual core removed from the lintel, after cutting. For the top tie, this operation is not needed because the surface finishing is plaster. It is noted that the usage of stone stoppers in both ends of the ties requires the drilling to be executed from both sides, which requires precision and qualified workers.

### **5.3 Execution**

The full list of works carried out included the followings: (a) temporary propping of the three lintels, with adjustable propping in the main lintel; (b) lifting of the main lintel back to its original position; (c) removal of the light projectors inside the chimney, respective cabling and the iron cramp; (d) injection of the supports

provided by the capitals of the stone columns, with fluid lime mortar; (e) injection of the cracks in the stone lintels with epoxy resin and injection of the brick masonry cracks with fluid lime mortar; (f) replacement of the iron corner ties with stainless steel ones (AISI 316); (g) insertion of stainless steel ties inside the main stone lintel; (h) reconstitution of the stone corner in the left lintel; (i) anti-corrosion and fire protection treatment of the iron ties; (f) plastering of the missing or damaged rendering with lime mortar; (g) full painting of the chimney from the exterior with traditional limewash. Figure 15 illustrates the main aspects related to the complete remedial measures and the final aspect of the chimney.

## 6 Conclusions

This paper addresses the issue of preservation of ancient structures from cultural heritage buildings. The novel ICOMOS [1] recommendations are briefly reviewed and the most relevant issues are discussed in a format of interest to other conservation specialists. The recommended methodology is presented in a simplified flow chart, stressing the importance of an “Explanatory Report”, where all the acquired information, the diagnosis, including the safety evaluation, and any decision to intervene are fully detailed. Finally, a case study of remedial works in a chimney that suffered unexpected sudden partial collapse in a stone lintel is also presented, with a focus on the possibilities and advantages of using advanced numerical analysis in the diagnosis and safety evaluation of the architectural heritage. For the case study, it was possible to conclude that: (a) the structure features original construction defects; (b) the damage and location of damage are not compatible with the structural analysis; (c) the main reason for damage is due to corrosion of an iron tie and weakening of the cross section of the tie for anchoring. It was possible to define two possible solutions for the intervention and to limit the fabric replacement to a minimum. The adopted measures are fully detailed in the text.

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Figure 9 – Principal stresses for the brick masonry part of the model, plotted in the deformed mesh configuration (Model 2, linear elastic analysis): (a) maximum (tension) e (b) minimum (compression). Results in  $\text{N/mm}^2 \times 10^3$

Figure 10 – Principal stresses for the stone columns and lintels, plotted in the deformed mesh configuration (Model 2, linear elastic analysis): (a) maximum (tension) e (b) minimum (compression). Results in  $\text{N/mm}^2 \times 10^3$

Figure 11 – Principal strains for the brick masonry part of the model, plotted in the deformed mesh configuration (Model 2, non-linear analysis and a load factor of 2.0): (a) maximum (tension) e (b) minimum (compression). Results are dimensionless

Figure 12 – Principal strains for the stone columns and lintels, plotted in the deformed mesh configuration (Model 2, non-linear analysis and a load factor of 2.0): (a) maximum (tension) e (b) minimum (compression). Results are dimensionless

Figure 13 – Force-displacement diagram for the non-linear analysis (internal arching system not considered).

The vertical displacement is measured at the mid-span of a side lintel

Figure 14 – Strengthening design: (a) front view; (b) detail of anchoring zone; (c) side view; (d) final aspect

Figure 15 – Aspects of the works carried out in the chimney: (a) temporary propping of the chimney; (b) lifting of the main lintel; (c) detail of the damage left support after removal of the iron clamp; (d) removal of plaster in the main wall for masonry crack injection with fluid lime mortar; (e) detail of the original reduction in the cross section of the main lintel for anchoring the removed iron ties; (f) aspect of the left side lintel after injection of the stone lintel (epoxy resin) and masonry wall (fluid lime mortar); (g) aspect of the holes drilled for the ties and for the anchoring plates; (h) aspects of the steel anchoring plates, bolts and injection tubes; (i) aspect of the stainless steel replacing tie; (j) final aspect of the chimney after the remedial works



## List of Tables

Table 1 – Data for the numerical simulations [13,14]

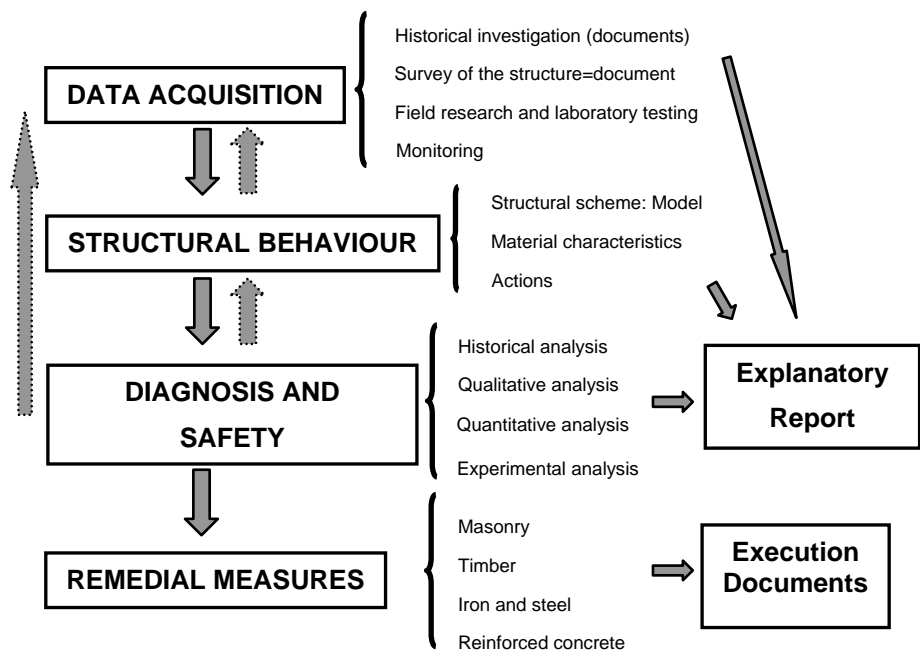
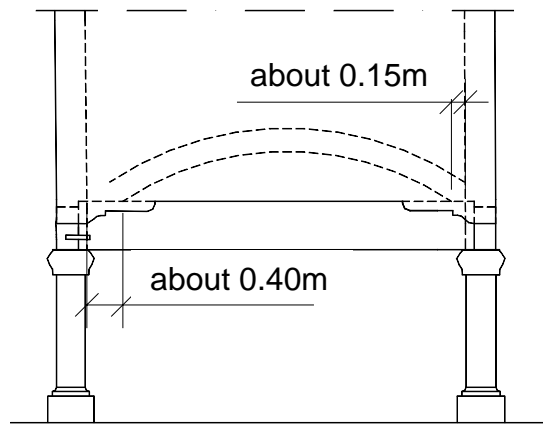
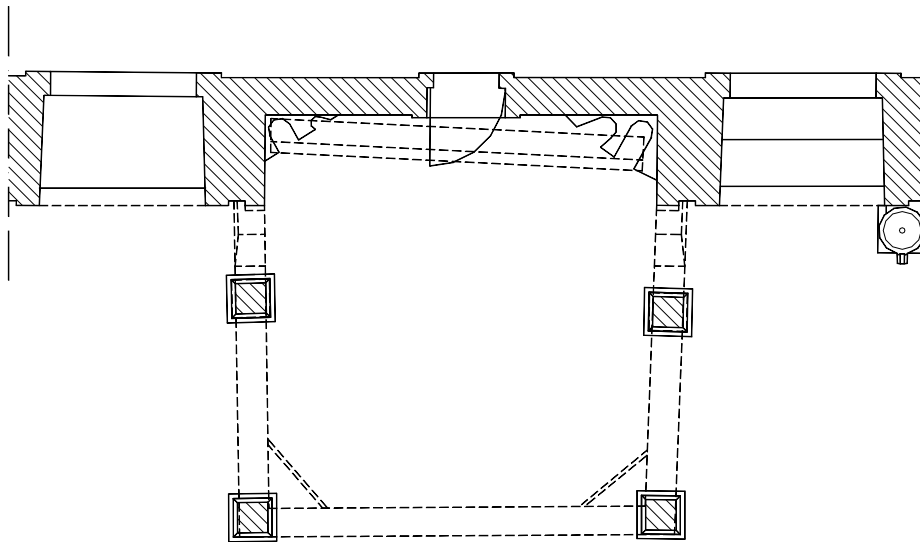


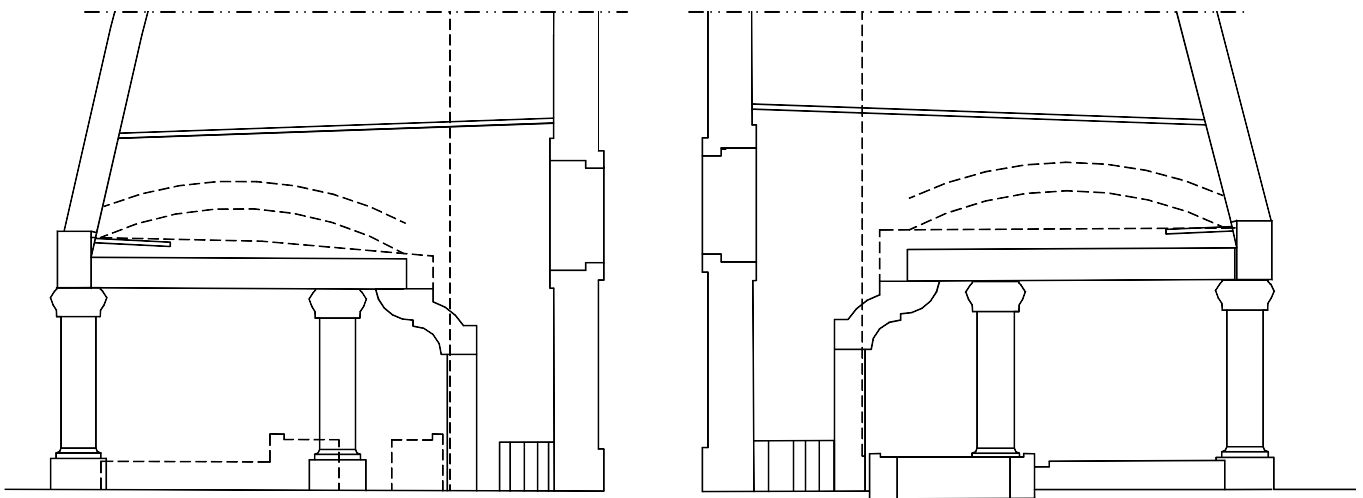
Figure 1 – Flowchart with the methodology for structural interventions proposed by ICOMOS [1]



(a)



(b)

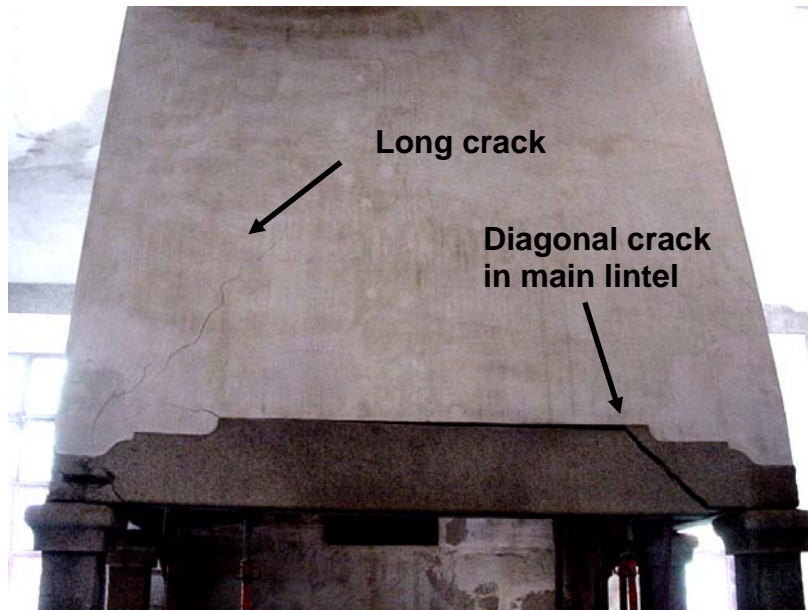


(c)

Figure 2 – Geometry of the chimney: (a) front view, (b) plan and (c) lateral views



Figure 3 – Typical aspect of an inspection opening to uncover the internal arching system



(a)

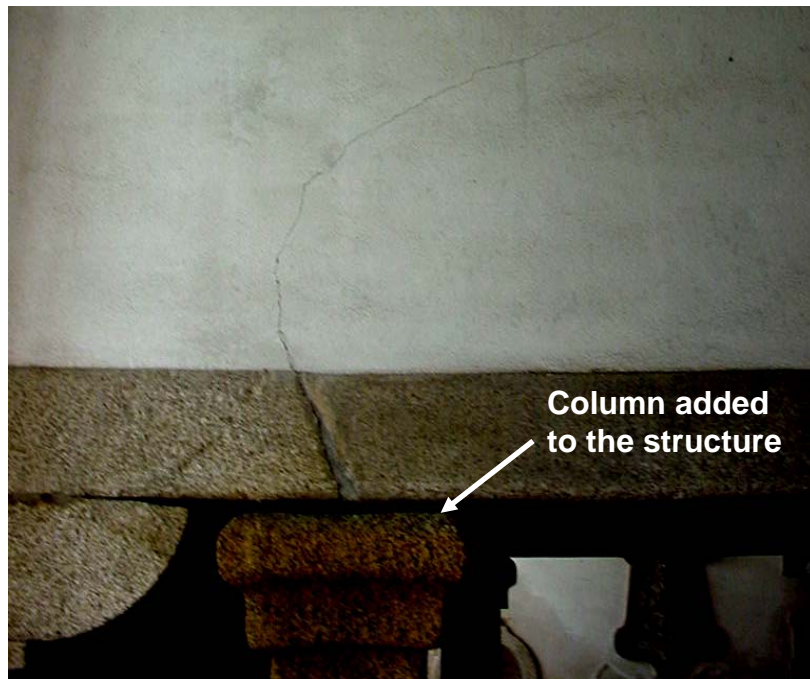


(b)

Figure 4 – Damage observed in the chimney: (a) diagonal crack in the main lintel and cracks in the upper masonry wall; (b) corrosion of the tie in the connection with the main lintel (intercepting the diagonal crack); (c) detail of the cracking and spalling of the main lintel, in the vicinity of the iron cramp; (d) crack in the left side lintel

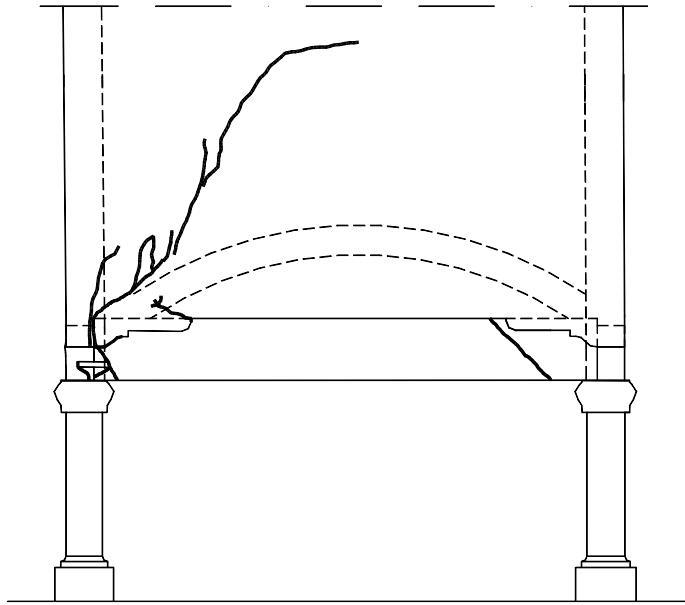


(c)

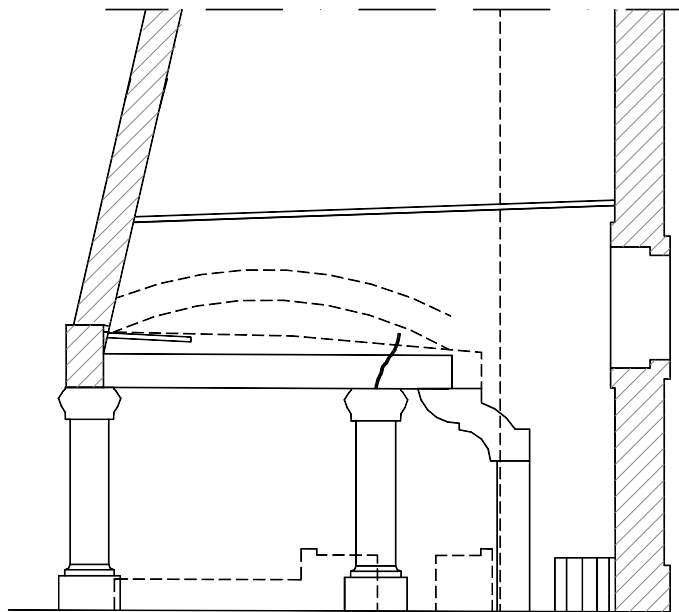


(d)

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(a)



(b)

Figure 5 – Observed cracking patterns: (a) main wall and (b) left side wall

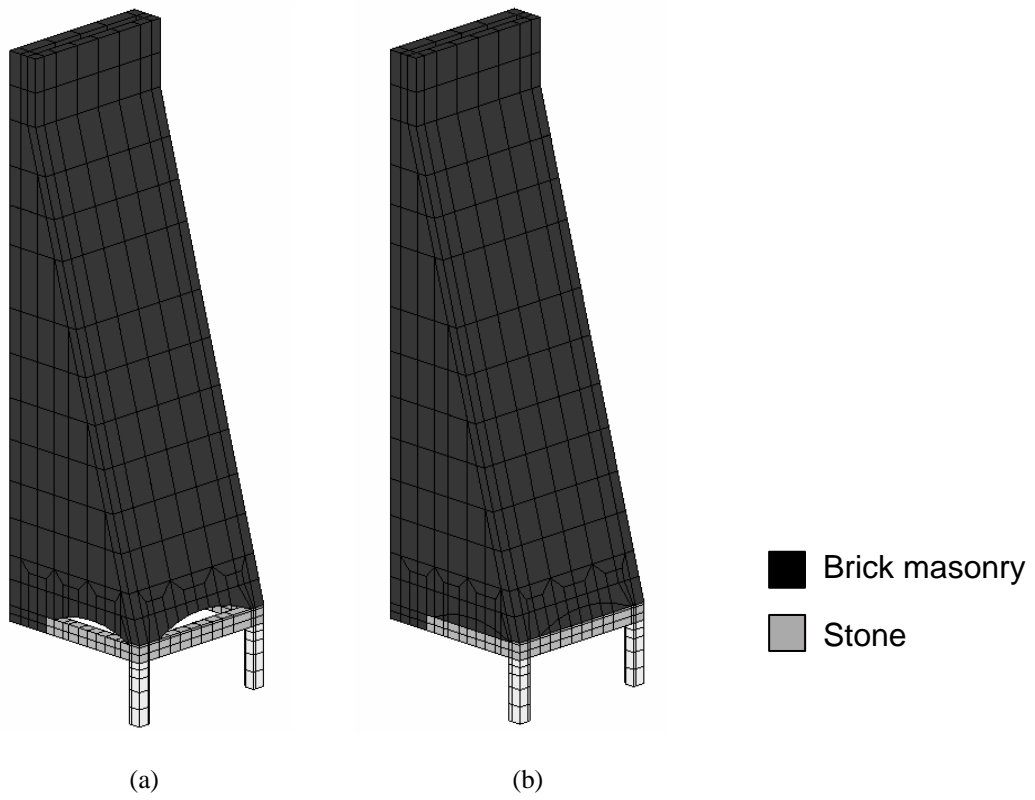
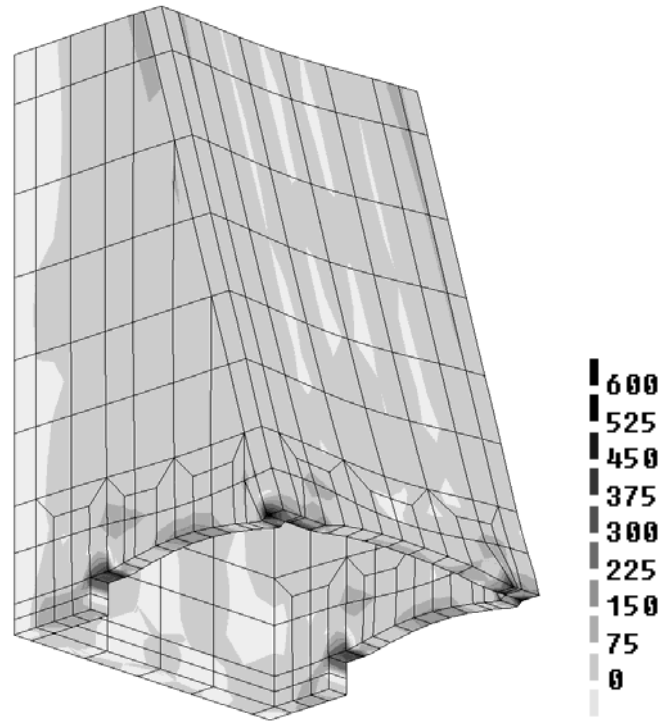
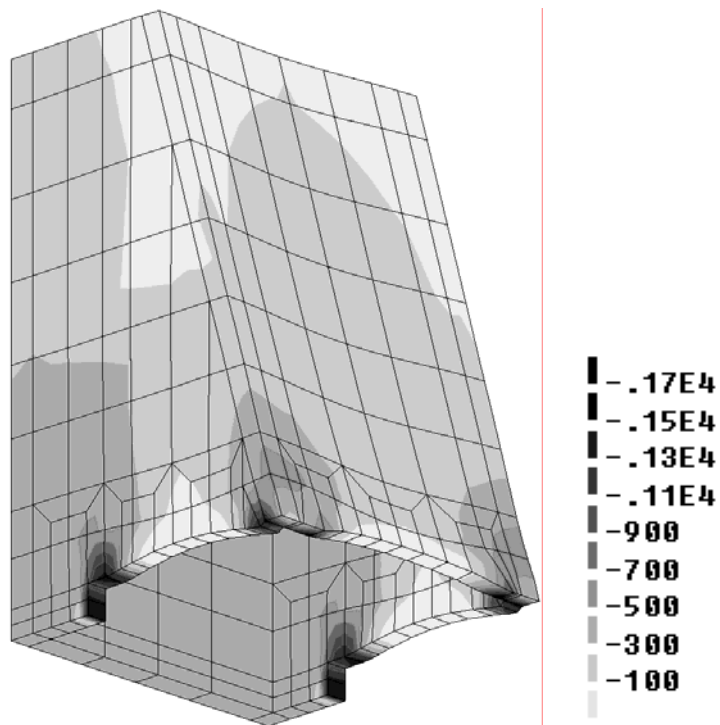


Figure 6 – Finite element model and materials adopted in the simulation: (a) Model 1 (full arching action);  
(b) Model 2 (no arching action)



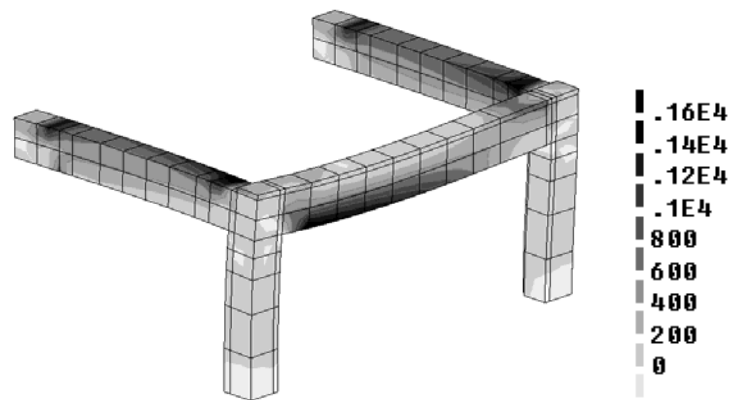


(a)

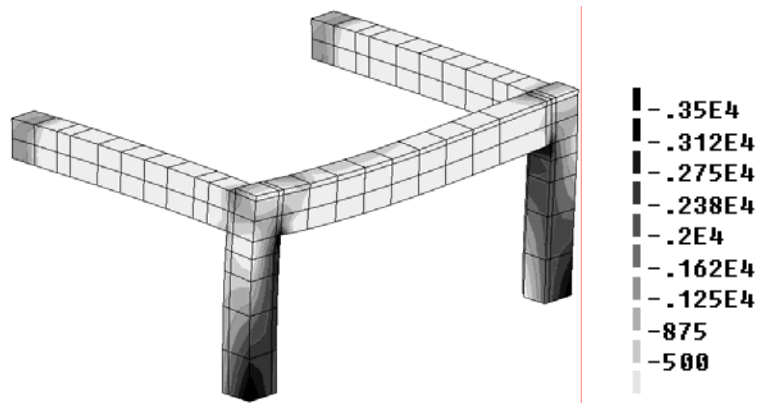


(b)

Figure 7 – Principal stresses for the brick masonry part of the model, plotted in the deformed mesh configuration (Model 1, linear elastic analysis): (a) maximum (tension) e (b) minimum (compression). Results in  $\text{N/mm}^2 \times 10^3$

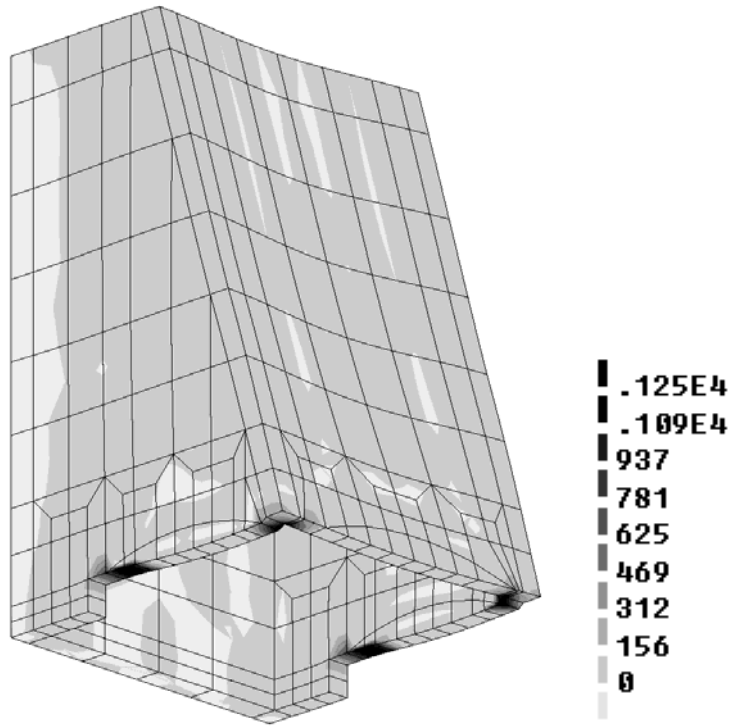


(a)

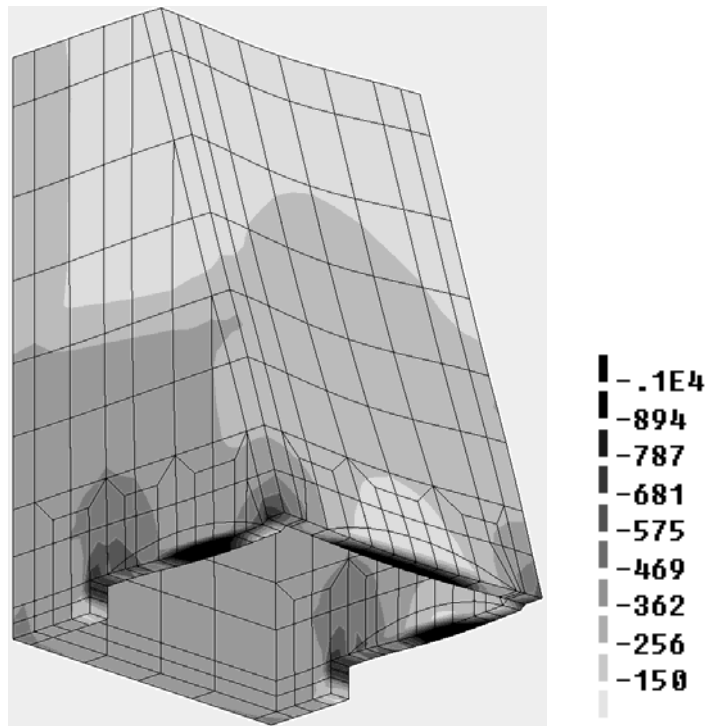


(b)

Figure 8 – Principal stresses for the stone columns and lintels, plotted in the deformed mesh configuration (Model 1, linear elastic analysis): (a) maximum (tension) e (b) minimum (compression). Results in  $\text{N/mm}^2 \times 10^3$

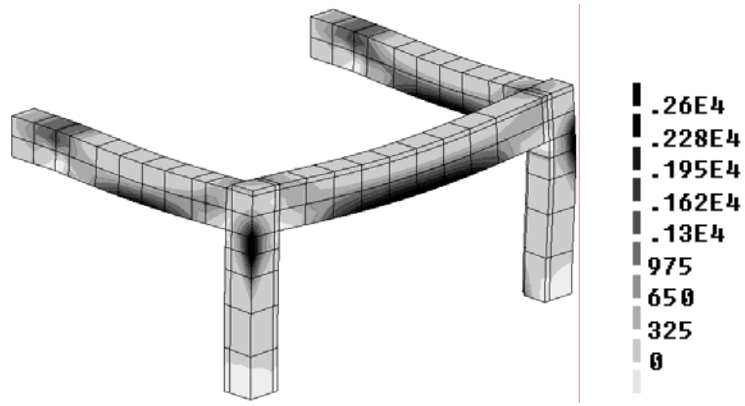


(a)

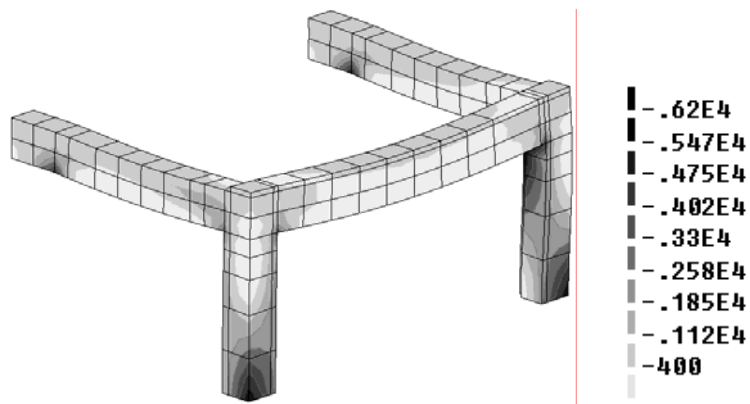


(b)

Figure 9 – Principal stresses for the brick masonry part of the model, plotted in the deformed mesh configuration (Model 2, linear elastic analysis): (a) maximum (tension) e (b) minimum (compression). Results in  $\text{N/mm}^2 \times 10^3$

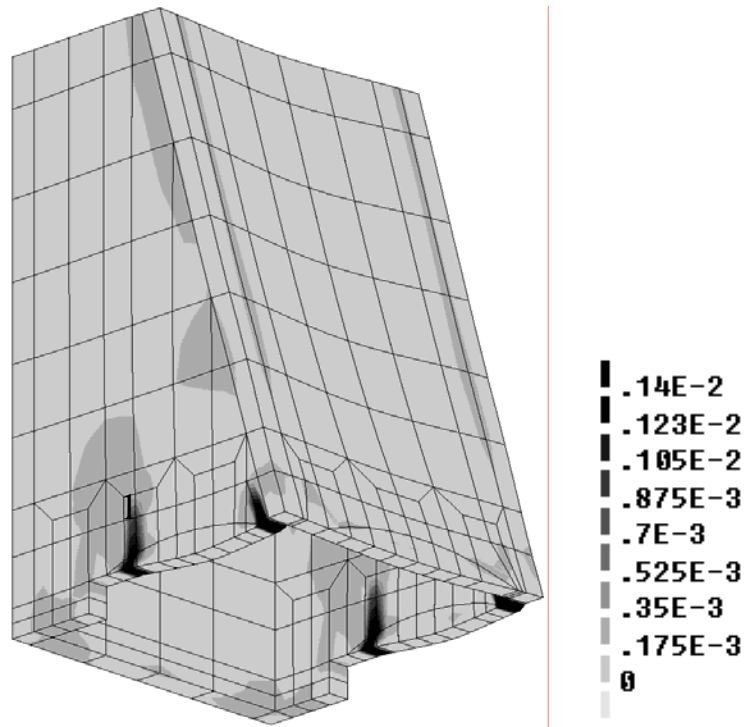


(a)

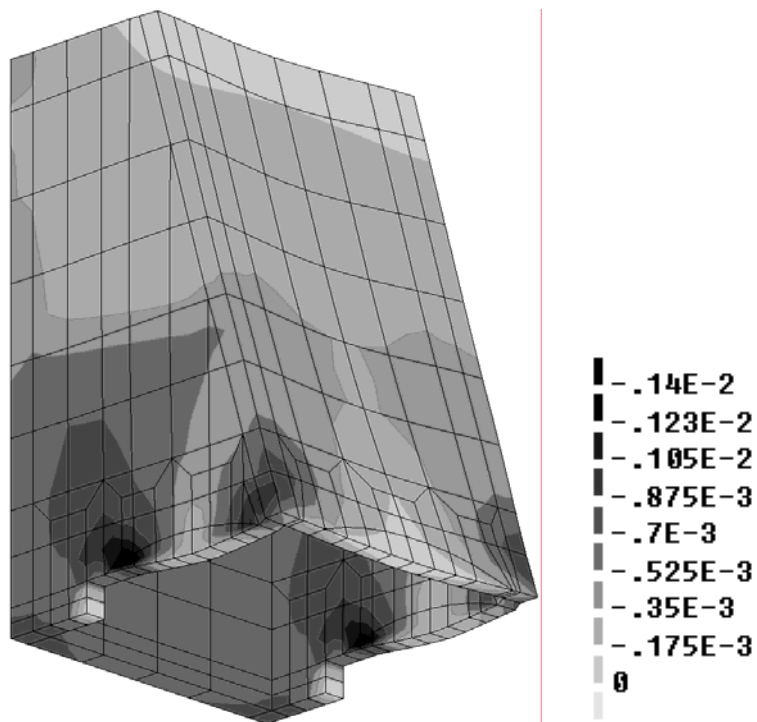


(b)

Figure 10 – Principal stresses for the stone columns and lintels, plotted in the deformed mesh configuration (Model 2, linear elastic analysis): (a) maximum (tension) e (b) minimum (compression). Results in  $\text{N/mm}^2 \times 10^3$

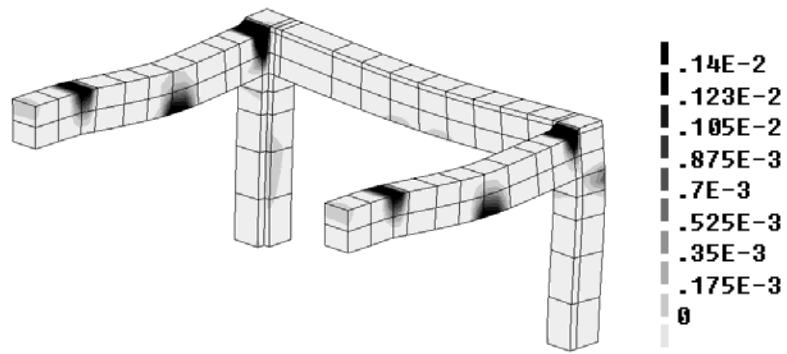


(a)

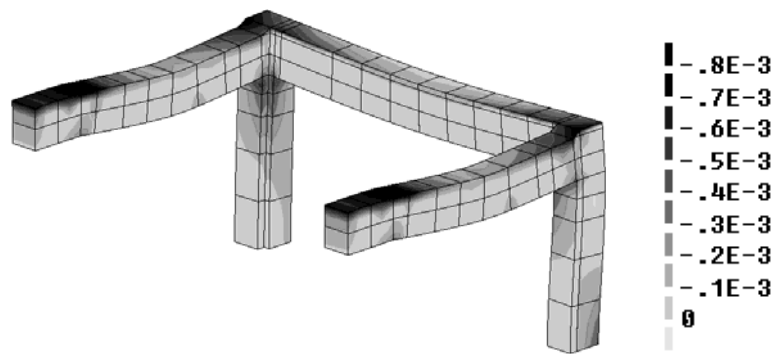


(b)

Figure 11 – Principal strains for the brick masonry part of the model, plotted in the deformed mesh configuration (Model 2, non-linear analysis and a load factor of 2.0): (a) maximum (tension) e (b) minimum (compression). Results are dimensionless



(a)



(b)

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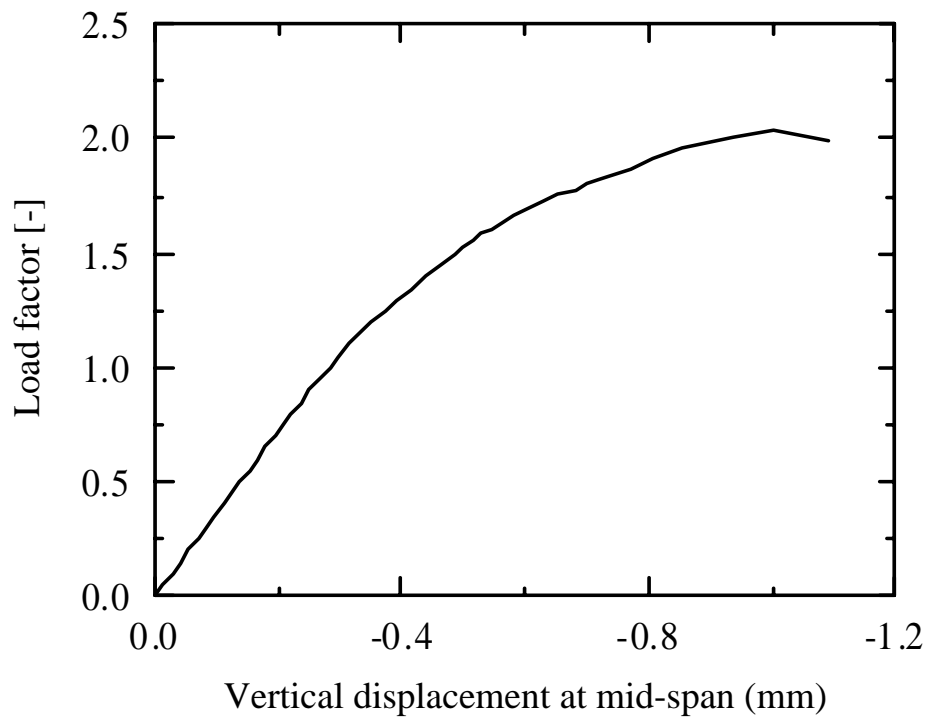
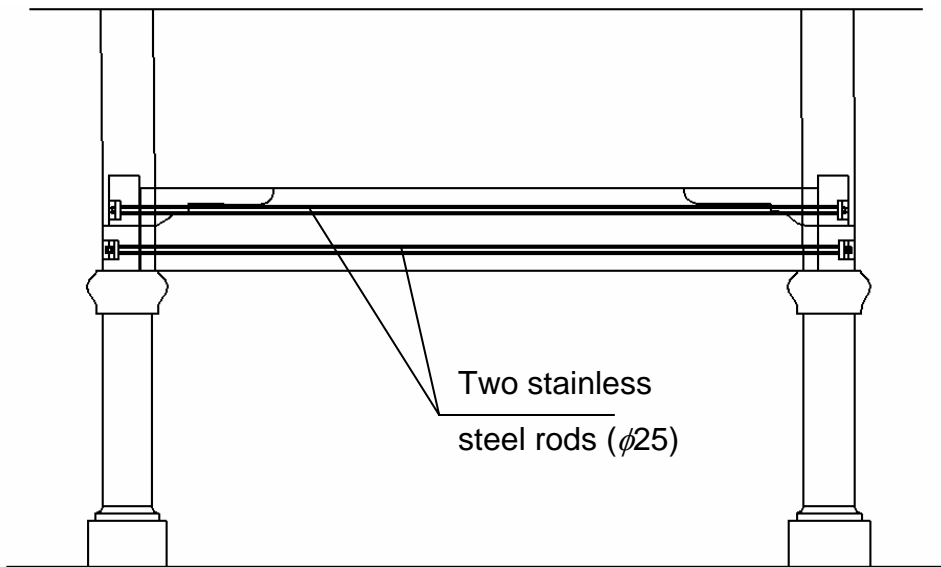
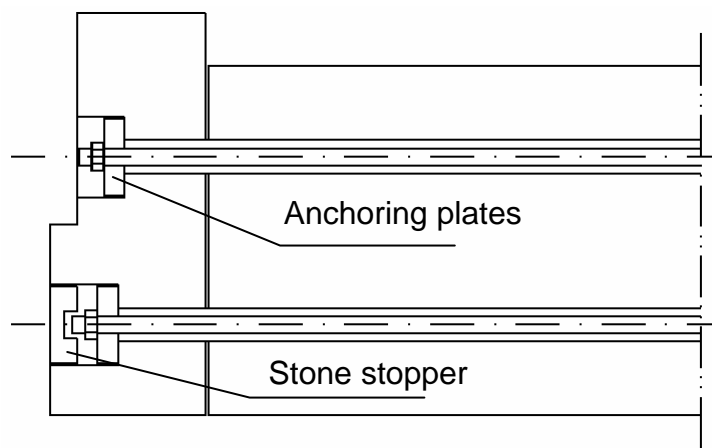


Figure 13 – Force-displacement diagram for the non-linear analysis (internal arching system not considered).

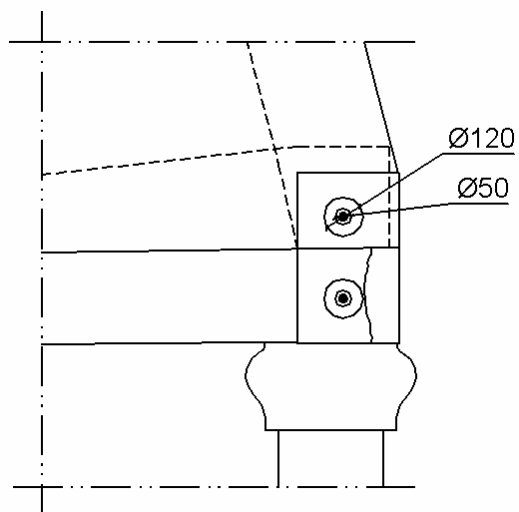
The vertical displacement is measured at the mid-span of a side lintel



(a)

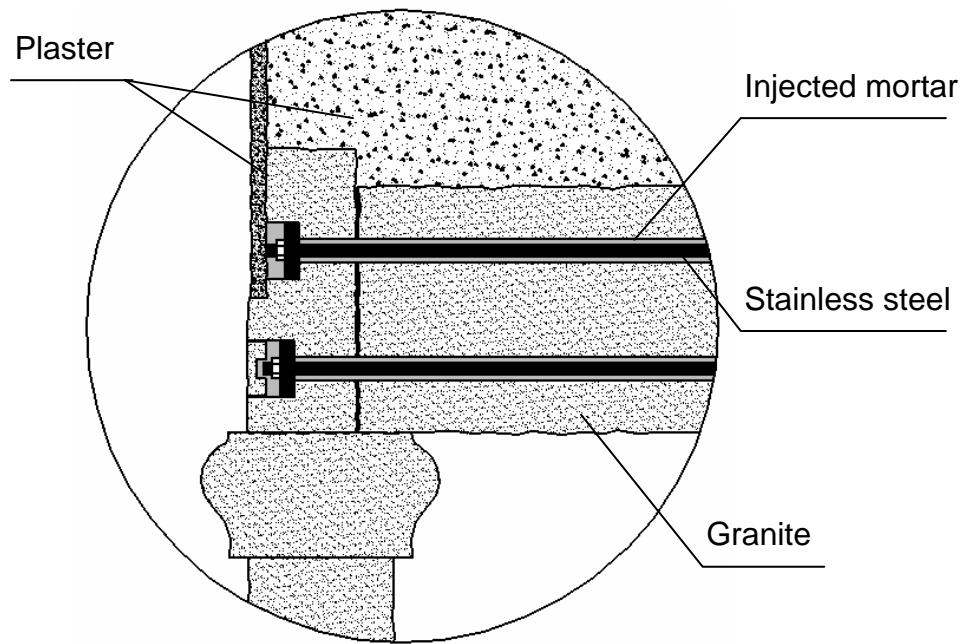


(b)



(c)





(d)

Figure 14 – Strengthening design: (a) front view; (b) detail of anchoring zone; (c) side view; (d) final aspect



(a)



(b)



(c)



(d)



(e)



(f)



(g)



(h)



(i)



(j)

Figure 15 – Aspects of the works carried out in the chimney: (a) temporary propping of the chimney; (b) lifting of the main lintel; (c) detail of the damage left support after removal of the iron clamp; (d) removal of plaster in the main wall for masonry crack injection with fluid lime mortar; (e) detail of the original reduction in the cross section of the main lintel for anchoring the removed iron ties; (f) aspect of the left side lintel after injection of the stone lintel (epoxy resin) and masonry wall (fluid lime mortar); (g) aspect of the holes drilled for the ties and for the anchoring plates; (h) aspects of the steel anchoring plates, bolts and injection tubes; (i) aspect of the stainless steel replacing tie; (j) final aspect of the chimney after the remedial works

Table 1 – Data for the numerical simulations [13,14]

	<i>Brick masonry</i>	<i>Granite stone</i>
$d$ [-]	1.8	2.7
$E$ [N/mm <sup>2</sup> ]	1 000	30 000
$\nu$ [-]	0.2	0.1
$f_t$ [N/mm <sup>2</sup> ]	0.2	2.0
$Gf_t$ [N/mm]	0.040	0.150
$\beta$ [-]	0.05	0.05
$f_c$ [N/mm <sup>2</sup> ]	1.0	60.0