Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 70 (1993)

Artikel: Reinforcement analysis in the restoration of masonry monuments

Autor: Abruzzese, Donato / Como, Mario / Lanni, Giorgio

DOI: https://doi.org/10.5169/seals-53309

Reinforcement Analysis in the Restoration of Masonry Monuments

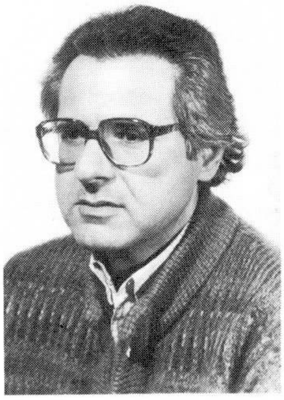
Étude de renforcement pour la restauration de monuments en maçonnerie

Verstärkungsanalyse für die Restauration von Mauerwerksgebäuden

Donato ABRUZZESE
Civil Eng.
Univ. of Rome 'Tor Vergata'
Rome, Italy

Mario COMO
Prof.
Univ. of Rome 'Tor Vergata'
Rome, Italy

Giorgio LANNI
Assoc. Prof.
Univ. of Rome 'Tor Vergata'
Rome, Italy



SUMMARY

Within the framework of the studies on masonry structures that use the no-tension Heyman model of the masonry material, the paper aims to analyse the influence of the tie rods on the lateral strength of two archetype schemes of many masonry monuments and historical buildings: the plane multi-storey wall with openings and the vault with abutments walls.

RÉSUMÉ

Dans le cadre des études sur les structures en maçonnerie - utilisant en ce qui concerne les matériaux, le modèle non-tension de Heyman - ce travail a pour but d'analyser l'influence des chaînes sur la résistance latérale de deux dispositions types de nombreux monuments et bâtiments historiques: la paroi plane de plusieurs étages avec ouvertures et la voûte avec contreforts.

ZUSAMMENFASSUNG

Im Rahmen von Studien an Mauerwerkstragwerken, welche in Bezug auf die verwendeten Materialien dem zugspannungsfreien Heyman'schen Modell entsprechen, hat diese Arbeit zum Ziel, den Einfluss der Zugstangen auf die seitliche Widerstandsfähigkeit zweier Archetypen - der flachen mehrgeschossigen Wand einerseits und des Bogens mit aussteifenden Mauern andererseits - vieler gemauerten Monumente und historischer Bauten zu untersuchen.

1. INTRODUCTION

There is an increased need to understand the structural principles of the behaviour of masonry structures, mainly for the repair and the statical analysis of monuments and historical buildings. The assumption of a consistent model of the behaviour of the masonry is a fundamental starting point because, very seldom, the traditional linear elastic analysis can be useless. The response of masonry to the applied loads has an unilateral nature: this material, in fact, can carry compressive forces, but can resist only feable tensions [1,6]. In this context the pioneering studies of J. Heyman on the masonry arch [1,2,3,4] still to day represent fundamental results. According to this approach, to interpretate the masonry behaviour, four constitutive assumptions are made:
- sliding failure cannot occur
- masonry has not tensile strength
- masonry has an infinite compressive strength
- masonry is rigid in compression.

Very dangerous for the masonry structures are the actions of horizontal forces, particularly due to earthquakes. It is urgent therefore today the demand of simple and rational methods to control the lateral strength of masonry structures and, eventually, to calculate reinforcements. In progress with the research developed by the Authors on the argument [8,9,10,11], aim of this Paper is to analyse the influence of the horizontal connections, on the lateral strength of two archetypal structures that are the main resistant systems of many masonry buildings and monuments: the plane multistory wall with openings and the vault with abutments walls, both connected by metallic tie rods.

2. LATERAL STRENGTH OF MULTISTORY WALLS WITH OPENINGS AND CONNECTING TIES

Let us consider the plane multistory wall with a regular array of openings, with N_p stories and N_m piers. (Fig.1) The wall is subjected to the action of fixed dead loads G_{ij} and imposed horizontal loads, gradually increasing with the load factor λ.

The piers are connected by means of masonry architraves, eventually reinforced by steel platbands, and also by steel ties passing through the masonry at the floor levels. The platbands or architraves will be able to sustain only compressive forces. On the contrary, the steel ties can sustain only tractions.

Both the architraves and the steel ties can develop elastic strains. In a simplified model of the masonry walls we can take into account only the deformations due to the masonry fracturing. Consequently, under the action of seismic horizontal forces, piers will remain rigid as long as the local turn over failure does not occur. The horizontal displacements of the failed piers will be therefore due only to rotations around their toes. At a generic stage of loading, on a pier act:

- vertical dead loads G_{ij} applied at the pier i and at the story j. The position of the loads G_{ij} with respect to the bottom right toe are defined by the arms b_i.
- horizontal imposed loads λG_{ij}. The elevations of the various stories, where the forces are applied, are indicated by z_j .

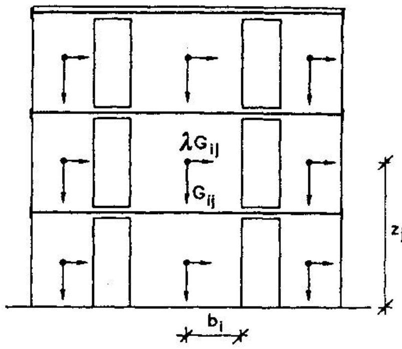


Fig.1 The multistory masonry wall

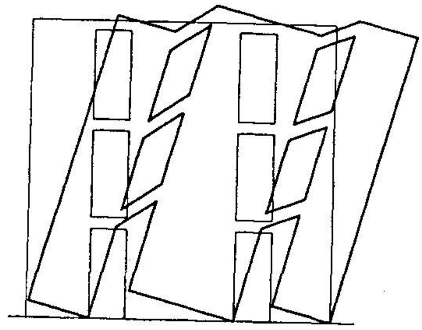


Fig.2 The sideway failure mechanism

- actions transmitted by the horizontal connections. According to the positions

$$M_i^S = \sum_1^{N_p} G_{ij} b_i \quad M_i^R = \sum_1^{N_p} G_{ij} z_j \quad M_i^N = \sum_1^{N_p} N_{ij} z_j \quad M^T = \sum_1^{N_p} T_j z_j \tag{1}$$

The quantities M^S_i , M^R_i , M^N_i, M^T_i respectively represent:
- the stabilizing moment of all the forces acting on the pier i;
- the turn-over moment of the imposed horizontal loads;
- the moment, related to the span i and respect to the pier base, of the axial forces acting in the architraves;
- the moment with respect to pier base of the tensions in the steel ties.

Each masonry pier is characterized by a proper lateral strength λ_{0i} defined by the limit value of the multiplier λ of the horizontal loads

$$\lambda_{0i} = M^S_i / M^R_i \tag{2}$$

Under the gradually increasing lateral forces the local failure in the weakest pier is attained when the load factor λ reaches the value

$$\lambda^*_{0i} = MIN(\lambda_{0i}) \tag{3}$$

The full lateral strength of the wall, on the contrary, will be attained when all the piers will have reached their proper failure condition under the limit value λ_0 of the load factor.

At the collapse of the wall the following equilibrium equations will be satisfied
- for the first pier

$$M_1^S + \lambda_0 M_1^R - M_1^N + M^T = 0 \tag{4}$$

- for the pier i

$$M_i^S + \lambda_0 M_i^R + M_{i-1}^N - M_i^N = 0 \qquad (5)$$

- for the last pier

$$M_{Nm}^S + \lambda_0 M_{Nm}^R + M_{Nm-1}^N - M^T = 0 \qquad (6)$$

Summing up these equations we obtain the collapse multiplier λ_0

$$\lambda_0 = \sum_1^{Nm} M_i^S \; / \sum_1^{Nm} M_i^R \qquad (7)$$

that is included between the minimum and the maximum values of the local collapse multipliers λ_{0i}. We immediately recognize the strengthening effect due to the introduction of ties. Because of the unilateral character of the horizontal connections, we have to associate to the equations (4), (5), (6) the following inequalities

$$M_i^N \geq 0 \qquad (i = 1, \ldots, N_m) \qquad (8)$$

At least one of the unknowns M^N_i , M^T must be equal to zero.
Let us suppose, in fact, that at the collapse the tensions in the steel ties are not equal to zero. It means that the horizontal fiber of the wall along any tie rod is stretched out. If, on the other hand, at any span of the wall the compression in the architraves were not zero, the same horizontal fiber should become shorter. Hence, to accept the stretching of the ties implies that at least in one span the compression in the architraves is zero.
On the contrary, if all the compression in the architraves were different from zero, the tensions in the steel ties will be equal to zero. Thus, since at least one of the unknown must be equal to zero, the system of equation (4), (5), (6) and inequalities (8) has an univocal solution. A simple procedure to obtain the solution of the problem has been proposed by the Authors in [11]. Once that the solution has been obtained, i.e. the collapse multiplier λ_0 together with the global moments M^S_i , M^R_i , M^N_i , M^T_i have been evaluated, it is necessary to define stresses in the ties. As usual, along the height ties are all equal each others ; thus we can write, according to the assumed model for the masonry behaviour

$$T_j = T_1 z_j / z_1 \qquad (9)$$

With this position the evaluation of the stresses in the tie rods at the ultimate state of the multistory wall can be immediately obtained.

3. VAULTED STRUCTURES

The evaluation of the vaults thrust, in order to design abutments able to stand firm against overturning, is a very old problem in the history of the mechanics of masonry structures [1]. The failure mechanism of the vaulted structure, with the determination of the position of the hinges, was another aspect of the same problem. Fig.3 shows the symmetric failure mechanism of this structure. A wide numerical investigation has been carried out by the Authors in the study of the symmetric collapse of the system with a linearly increasing vertical load applied only on the vault and fixed loads on the abutments. The failure mechanism of the vault under the above defined system of vertical loads is characterized by the occurrence of two hinges symmetrically placed at the intrados of the vault (Fig.3) with an angle ϕ equal to 30° respect to the horizontal chord. Fig. 4, on the other hand, shows the multiplier λ_0 of loads applied on the vault versus the ratio H/r, i.e. the ratio between the height of the abutments and the radius of the vault at the intrados. Contrary to the old De Durand [1] old rule, we can recognize the influence of the abutments height on the multiplier λ_{0sim}.

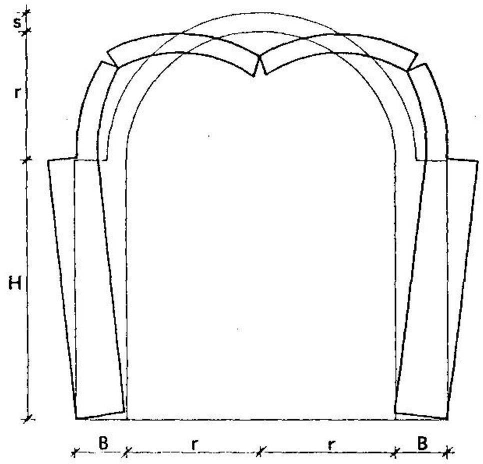


Fig.3 Symmetric failure mechanism of the unreinforced vault

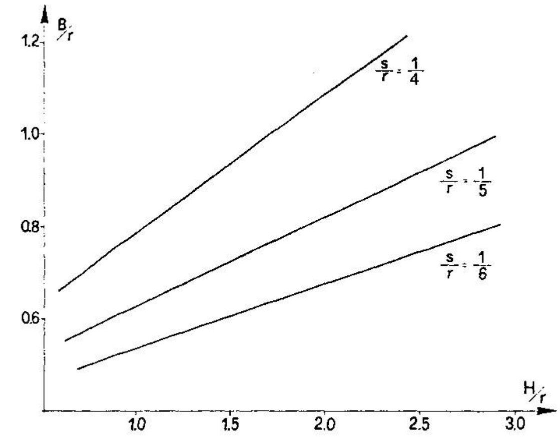


Fig.4 The influence of H/r on the collapse multiplier λ_{0sim}

The vaulted masonry structure is very weak under horizontal loads. On one side the thrust due to dead loads sums up to the thrust due to the horizontal actions. The turnover failure of one abutment with the occurrence of hingings in the vault is very frequent, as we have shown in a previous analysis[8,9]. A chain just under the springings of the vault is therefore often introduced.

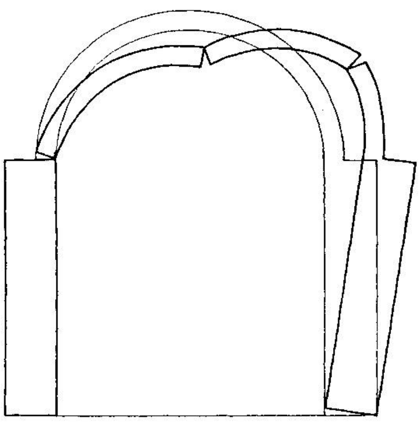


Fig.5 The four bar chain mechanism

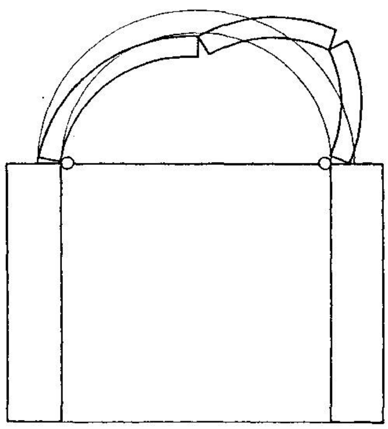


Fig.6 The local sideways mechanism of the vault alone

Let analyse therefore the vaulted structure of Fig.5 reinforced by a chain, passing through the head of the abutments. The introduction of the chain avoids the occurrence of the collapse mechanism of Fig.5, the so called four bar mechanism, with three hinges in the vault and a fourth at the toe of the right abutment.The collapse behaviour of the chained vaulted structure under horizontal loads is different. As the lateral loads increase in magnitude, the structure in fact will fail by one of the three different mechanisms:

a) *local sideways mechanism of the vault alone*

The masonry cracks in four sections of the vault with the development of a four bar mechanism with hinges only in the vault.(Fig.6)

b) *local sideways mechanism of abutments alone*

Cracks with hingings occur at the tops and at the toes of both the abutments walls. (Fig.7)

c) *global failure mechanism.*

The vaulted chained system can fail with a global "five bar chain" with the occurrence of hingings both in the vault and at the right toes of the abutments. (Fig.8) The study of this *global failure mechanism* of the vaulted system requires the analysis of the "five bar chains" with the localization of the rotation centers of the four rigid parts in which the system, at the failure, is subdivided. The presence of the chain require equal rotations θ around the toes of the two abutments. In the kinematical analysis at the collapse compatibility conditions require the opening of the fractures at the hinges.The collapse of type a) has been thoroughly examined in [8,9]. The collapse multiplier λ_0 corresponding to the mechanism b) can be easily obtained and is given by

$$\lambda_{0\,(b)} = \frac{G_v\left[\dfrac{(B-s)\,(r+s)}{(2r+s)} - s\right] + G_a B\cdot}{G_a\ H + G_v Y_{Gv}} \tag{10}$$

where G_v and G_a represent the weights of the vault and of the single abutment, B and H the width and the height of the abutments, s and Y_{Gv} respectively the thickness and the height of the center of gravity of the semicircular vault.

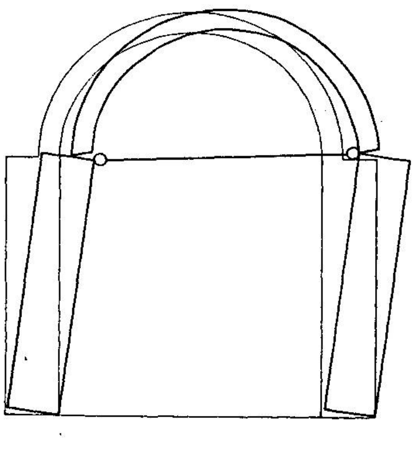


Fig.7 The local sideways mechanism of the abutments alone

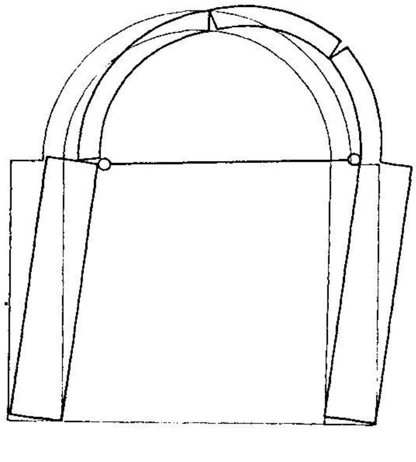


Fig.8 The global failure mechanism of the vault

The occurrence of the various different mechanism and the corresponding values of the lateral strength of the structure, depends on numerous geometrical .factors. The collapse multipliers corresponding to the mechanisms b) and c) are very near each others. The contribution to the resisting work of the masses of the vault is mainly due to the lifting depending on the abutments rotation. Less relevant, in fact, is the contribution due to the lifting depending on the hinging in the vault.
From a practical point of view, when the collapse is not only localizes in the arch, the load multiplier can be therefore evaluated by means of the eq.(10).
Once the collapse multiplier λ_0 has been obtained, as minimum of the kinematical multipliers in the set of the admissible mechanism, type a), b) and c), the thrust line, passing through the hinges, has been traced in order to control the admissible state of stress inside the vaulted system at the failure (Fig.9).
An extended numerical investigation has been developed to evaluate the type of mechanism that occurs at the failure according the values of the most significant parameters of the structure geometry, expressed by the ratios H/r, B/r, s/r.
In Fig.10, if the point (H/r, B/r) representative of the vault, falls above of the straight line
corresponding to the value of the ratio s/r, the collapse occurs with the development of the failure mechanism a).Otherwise, the failure occurs with the mechanisms b) or c).
It is interesting to evaluate the lateral strength increment obtained by the introduction of the chain with respect to same but unreinforced vaulted system. An investigation has been made to evaluate the dependence of this increment on the vaulted system geometry: the most significant parameter in this case can be represented by the above defined load multiplier λ_{0sim} concerning the symmetric collapse of the system. In Fig.11 is represented the ratio

$$\rho = \lambda_{0\ chain} / \lambda_{0\ no\ chain} \qquad (11)$$

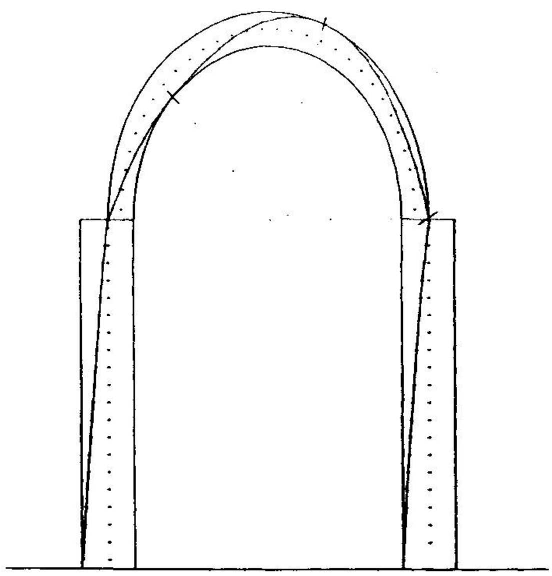


Fig. 9 The thrust line tracing to verify the results obtained by the kinematical procedure

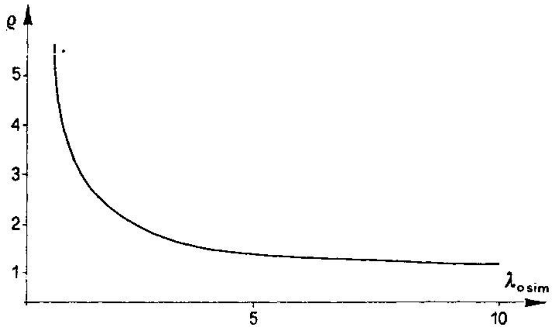


Fig.10 The occurrence diagram of the various failure mechanisms

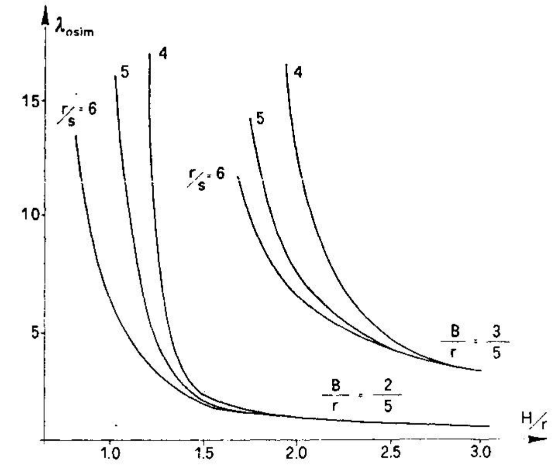


Fig.11 Strength increment due to the introduction of the chain

versus the multiplier λ_{0sim}.Of course, in presence of high values of λ_{0sim} the influence of the introduction of the chain is not significant;the contrary occurs for small values of λ_{0sim}.

REFERENCES

1. HEYMAN J., The stone skeleton. Int.Journ. of Solids and Struct.,Feb.,1966.
2. HEYMAN J., The safety of masonry arches.Int.Journ.Mech.Sci., Vol.11, 1969.
3. HEYMAN J., Equilibrium of shell structures. Clarendon Press, Oxford,1977.
4. HEYMAN J., The Masonry Arch.. Cambridge Press, Cambridge 1982.
5. BENVENUTO E., An introduction to the History of Structural Mechanics. Part II, Vaulted Structures, Springer Verlag, 1991.
6. COMO M., Equilibrium and collapse Analysis of masonry structures. Meccanica, Vol.27, No.3, Kluwer, Acad. Publ., 1992.
7. COMO M., GRIMALDI A., An Unilateral Model for the Limit Analysis of Masonry Walls. Intern. Congr. on 'Unilat. Probl. in Struct. Analysis, Springer Verlag,1985.
8. COMO M., LANNI G., Sulla verifica alle azioni sismiche di complessi monumentali in muratura, 3° Congr. Naz. Ing. Sism., Dip.Ing.Civile, Rep.n.11, Roma 1987.
9. COMO M., GRIMALDI A., LANNI G., New results on the horizontal strength evaluation of masonry buildings and monuments. 9th World Conf. Earthquake Eng. ,Tokio,1988.
10. ABRUZZESE D., COMO M., LANNI G., On the horizontal strength of the masonry cathedrals. ECEE,9, Moscow, 1990.
11. ABRUZZESE D., COMO M., LANNI G., On the lateral strength of multistory masonry walls with openings and horizontal reinforcing connections. Proc.Tenth World Conf. Earthquake Eng. Madrid,. Balkema, Rotterdam, 1992.