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MASONRY COMPRESSIVE STRENGTH ENHANCEMENT UNDER ECCENTRIC AXIAL LOAD

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Abstract. *This paper wants to attract the attention to a feature of masonry behavior experimentally confirmed but not universally recognized nor quantified, as it is the apparent enhancement in compressive strength under eccentric loads. This type of stress state is usual in many masonry structural members, particularly in arch bridges, as a result of the development of hinges. Experimental results in out-of-plane flexure loading are reviewed as well as the impact of flexure "over-resistance" in standards and recommendations. The paper presents new experimental results carried in the E.T.S.I Caminos of Madrid over wallettes subjected to centric and in-plane eccentric loading. An important enhancement is observed in in-plane flexure as it was in out-of-plane. A bilinear relation between enhancement factor and eccentricity is proposed. Quantitatively, factors are found up to 2.5 on the basis of linear stress distribution over the section, or 2.0 for uniform stresses. The cause of the phenomenon resides in the heterogeneous nature of masonry. Under eccentric loads, the (vertically) uncompressed part of the section does not develop lateral tensile stresses, which lead to failure. Finally some unsolved points in the quantification and applicability of the strength enhancement are highlighted.*

1 ECCENTRIC LOAD IN MASONRY ARCHES

Tradition says that arches work by their shape, under solely compression forces without flexion. In fact, bending moments –say eccentricity of the axial force– are connatural to arch behavior, and significance is focused onto section depth rather than intrados profile. This is so not only because of the action of point loads, but it takes place even under self weight since the beginning of the life of the bridge. As pointed by Heyman[1], just after the removal of scaffoldings, the arch thrusts its abutments outward, increasing the span and accommodating itself to this situation of minimal horizontal reaction by enlarging the effective rise and acquiring a three-hinged isostatic arrangement. Similarly, in a mechanism collapse situation, arches develop up to 4 hinges with the line of thrusts incredibly near the edge of the section. In both such conditions, the peak value of the stress becomes gigantic as the compressed area becomes more and more reduced.

2 "DISCOVERING" THE PHENOMENON

Recently, an experimental study[2], intended to validate a theoretical model on the interaction of in-plane ultimate forces (axial, bending and shear), *discovered* that the apparent compressive strength of brick masonry specimens under eccentric axial load largely exceeded the recorded value under pure compression. In the U.K., experiments on masonry panels, planned to investigate the effect of cyclic loading, found similar results comparing centric and eccentric tests. European codes[3] make no distinction in the design compressive strength of masonry in case of eccentric load. Nevertheless, the enhancement of strength in such cases is known quite long ago, thought consensus in its treatment and quantification seems to lack.

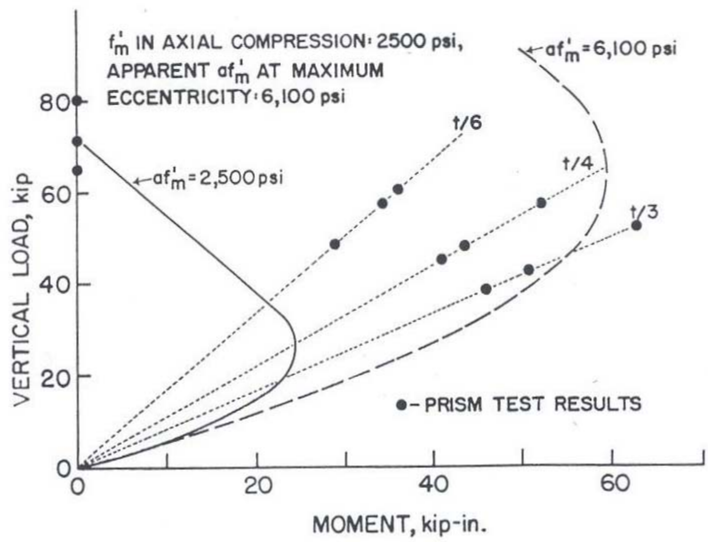

Figure 1: Results of Yokel and Dikkers for out of plane flexion, showing a maximum enhancement of 2.44.

Possibly, the first reference of an improvement in strength depending on the eccentricity of the load is due to Yokel and Dikkers[4]. They conducted out of plane flexo-compression tests on brick and concrete block masonry prisms with type N mortar. Results (fig. 1) show an apparent strength of $2.44f$ for maximum eccentricity ($e=t/3$) and an average enhancement factor of $a=f_{ap}/f=1.6$, on the basis of linear stress distribution. A discussion followed[5], other authors claiming the enhancement to be fictitious and due to a plastic redistribution of stresses. Yokel *et al* discharged such explanation on the basis of the magnitude of the phenomenon and provided new experimental data with factors $a=1.54$ and $a=1.29$ for brick and block pilasters respectively.

Hendry[6] admits the evidence of the higher value of ultimate compressive stress under eccentric load and quotes other authors who report enhancement factors increasing linearly with eccentricity (Burns gives $a=1.5$ for $e=t/3$, Fattal and Cattaneo report factor $a=1.4$ for $e=t/4$). Hendry relates the state of stress at the edge of a flexo-compressed section with the case of beam bearings on masonry, for which contact stresses may considerably exceed compressive strength of the material, proposing the expression[7] (1) for the enhancement coefficient a in terms of the ratio of the contact area to the whole area of the section:

$$a = 0.55(A_{bearing}/A_{total})^{-0.33} \geq 1 \tag{1}$$

Assuming linear distribution of stresses (1) can be expressed in terms of relative eccentricity yielding (2). Conservatively, Hendry proposes an average enhancement coefficient of $a=1.20$ for any eccentricity exceeding $0.15t$.

$$a = 0.38(0.5-e/t)^{-0.33} \geq 1 \tag{2}$$

UIC[8] includes a coefficient of enhancement which depends on relative eccentricity e/t (t being the depth of the section), expression (3):

$$a = 0.6 + 2.4\, e/t \quad (1/6 \leq e/t \leq 0.417) \tag{3}$$

ACI 530-99[9] uses two different partial material coefficients: 4 for pure compression and 3 for flexure, thus assuming an average enhancement coefficient of $a=1.33$ independently of eccentricity.

3 SOME RECENT IN-PLANE FLEXURE EXPERIMENTAL RESULTS

An experimental campaign over 23 wallettes was conducted in the Civil Engineering Faculty of the Polytechnic University of Madrid. Specimen were 9 courses high, dimensions are shown in fig. 2. Bricks were hollow ceramic with 30% voids filled with mortar prior to collocation. Mortar was 1:1:6 (cement:lime:sand). Material properties are given in table 1.

Table 1: Material properties

	Compressive strength		Tensile strength [N/mm²]		Secant Young's Modulus [N/mm²]
	Mean [N/mm²]	cV [%]	Mean [N/mm²]	cV [%]	
Bricks	42.8	10	4.0	6	-
Mortar	6.3	25	-	-	-
Masonry	11.4	2	-	-	10,000

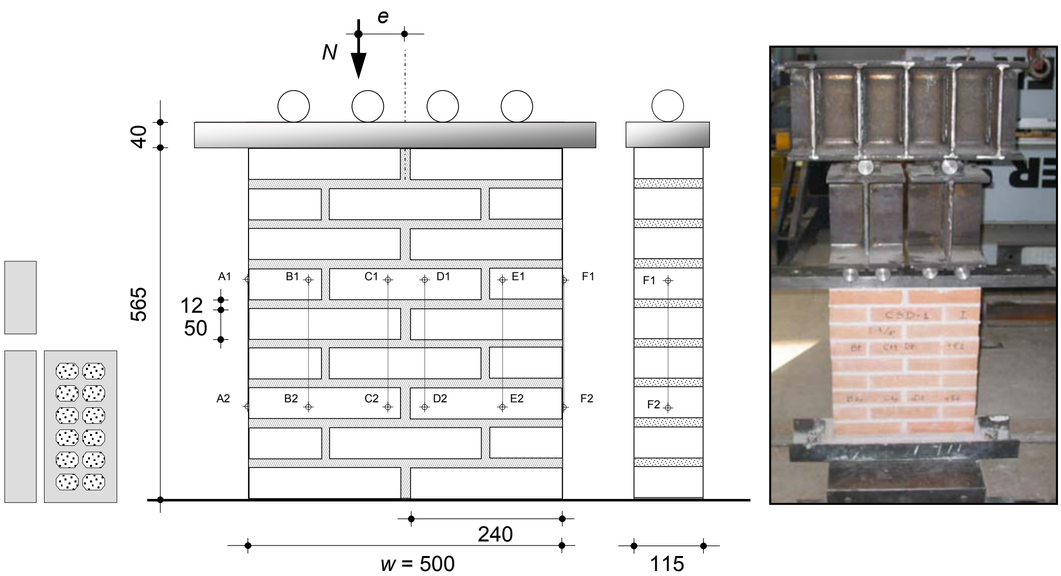

Figure 2: Brick and wallette layout and test arrangement. All dimensions in mm.

Six wallettes were tested under pure compression yielding a mean ultimate load of 655 kN (f=11.4 N/mm²). Vertical strains were recorded all around the perimeter indicating uniform strains in the specimen up to failure, for which ε_u ranged from 3.0E-3 to 3.5E-3. Then, 17 tests were conducted with eccentricities ranging from w/10 to w/2.4. Table 2 summarizes the results:

Tabla 2: Average ultimate axil load of in-plane flexo-compression tests

e [mm]		0	45	60	90	125	150	175	205
e/w		0.00	0.09	0.12	0.18	0.25	0.3	0.35	0.41
N_u [kN]		655	552	580	604	447	447	368	170
$a = f_{ap}/f$	*Linear stresses*	1.00	1.30	1.40	1.85	2.00	2.20	2.50	-
	Uniform stresses	1.00	1.00	1.10	1.40	1.45	1.60	1.90	-

Figure 3 shows the complete results in the N-M (M=N·e) plane together with the theoretical predictions, assuming linear stress distribution and different enhanced values of f. For comparison, the diagram for uniform stresses (a=1) is also displayed. It is worth no note the similarity of the results shown in fig. 3 with those in fig. 1. Yet, it may be highlighted that while in Yokel diagram the moment is out-of-plane (eccentricity measured on the thickness), the moment in fig. 3 is an in-plane force (eccentricity along the width). The conclusion is that a law defining the coefficient a of enhancement of compressive strength may be proposed as a function of relative eccentricity of the load. A threshold eccentricity e_0 is defined under which no enhancement is considered, then factor increases linearly up to a maximum value $\underline{a_{max}}$ for maximum eccentricity, similarly to UIC expression (3).

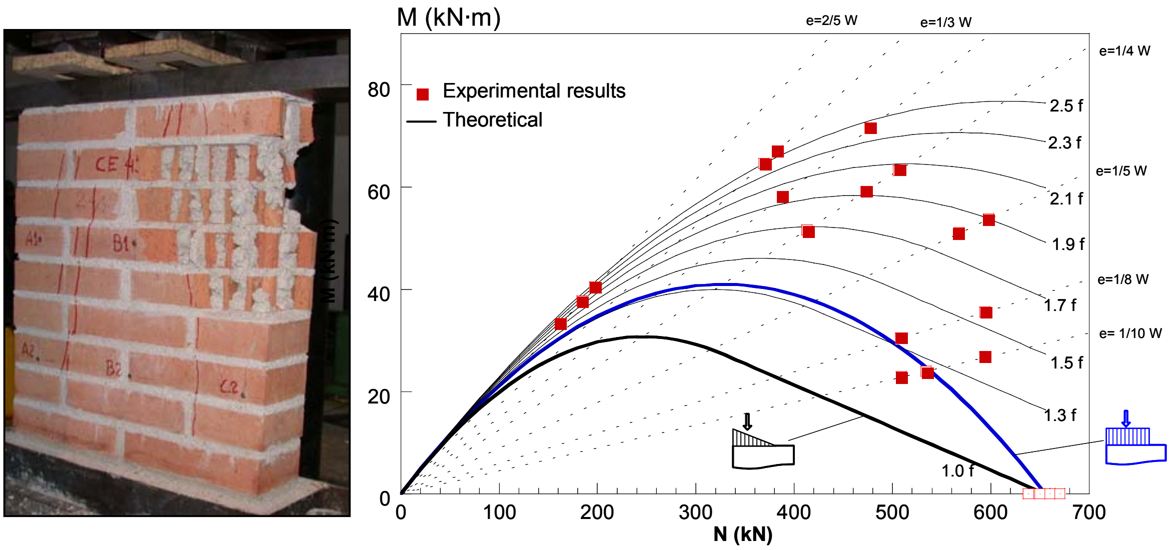

Figure 3: Typical failure and experimental N-M pairs versus theoretical predictions

Figure 4 illustrates the proposed law and the adjustment of theoretical diagrams to experiments when compressive strength is treated as dependent on eccentricity following the bilinear law (parameters for the tested masonry shown in figure 4). From this point of view, UIC expression (3) is a law of this type with parameters e_0/w=0.16 and a_{max}=1.80.

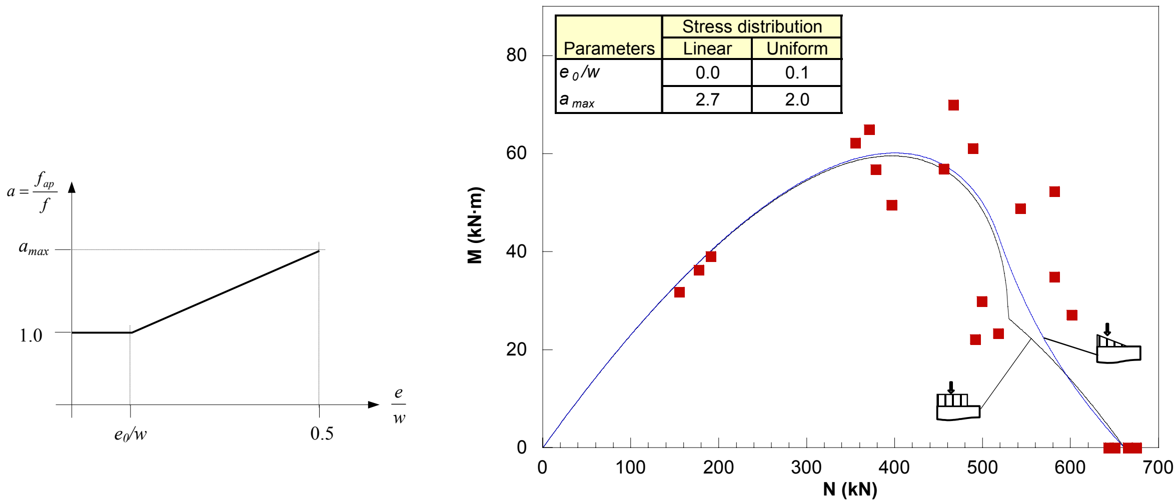

Figure 4: Proposed general law for enhancement factor *a* as a function of the eccentricity of load *e* (left). Fit to experimental results of interaction diagrams computed on the basis of the enhancement factor (right).

4 UNDERSTANDING THE CAUSES…

Different explanations have been suggested for the apparent enhancement of strength, sometimes vaguely defined and possibly influenced by knowledge of concrete behavior.

Yokel *et al* suggested that the cause of apparent strength enhancement may be that stresses under pure axial load are not uniformly distributed, as it is normally assumed. Hendry attributes the phenomenon to a strain-gradient effect arising from non-linear stress-strain relationship of masonry. Similarly, commentary to ACI-530 also points to the restraining effect of the less highly compressive fibers on the fibers of maximum compressive strength as the cause of apparent enhancement. In the opinion of the authors, the strain-gradient explanation should be applicable also to concrete, for which strain-gradient leads to ductility improvement but flexure strength increase is not so marked or does not exists at all (Sargin[10]). The answer seems to be a feature of the heterogeneity of masonry rather than being related to the *fiber* concept. Perhaps, the first convincing study about the causes was carried by Turkstra and Thomas[11]. On the experimental evidence that masonry prisms failing under flexure exhibit *cuasi* linear behavior, they conducted a linear micromodel 3D FEM calculation modeling out-of-plane flexure. Conclusions were i): As pure axial load induces generalized horizontal tensile stresses in units, eccentric load induce tensile stresses localized in small zones. ii) Horizontal tensile stresses in the direction of the wall thickness are related to the compressive load and are almost independent of its eccentricity, and iii) Maximum tensile horizontal stresses generated by bending moment were 2/3 those generated by axial load, provided maximum compressive stress for moment and load are equal.

It is known that under axial load, masonry fails because of tensile stresses developed in units. These horizontal tensile stresses are induced by vertical compression stresses as a result of the different deformability (longitudinal and transversal) of bricks and mortar. When load is eccentric, the compressed area is less; correspondingly, tensile stresses in units develop in a more reduced zone. Figure 5 shows elastic micromodel calculations on two wallets. Axial loads are different, but produce in both cases equal peak vertical stress. Induced tension (light contours) in the wall on the right (*e*/*w*=0.40) are much less significative than those in the left side (e/w=0). Wall on the right will fail when bigger tension stresses will develop. To achieve it, additional load is needed, laying a peak vertical stress value exceeding *f*.

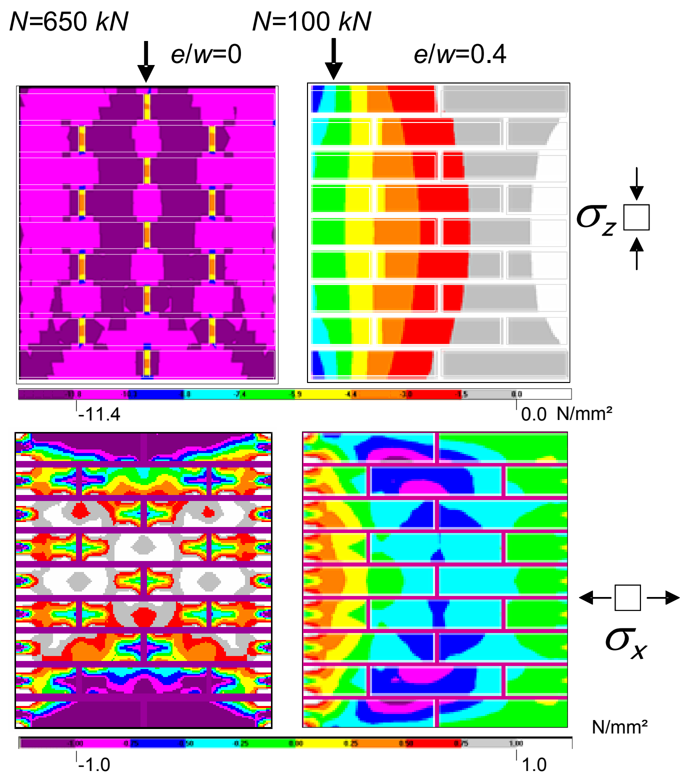

Figure 5: Vertical and horizontal stresses in two panels under centric and eccentric load.

5 … AND THE CONSEQUENCES

The revision of bibliography shows that an apparent enhancement in masonry compressive strength exists under eccentric load. In other words, compressive strength as deduced from pure compression tests is not applicable (as a failure criterion) to flexure stress states. Recent tests have shown that this phenomenon is present in in-plane flexure as well as in out-of-plane. Analysis of masonry arches, may take into account the existence of this quantitative important "over-resistance" under service loads and in collapse situation, when hinges develop.

It is, however, a little surprising that such a basic feature of masonry behavior is still not universally recognized (lacks in EC-6) and has not disserved much attention from researchers. The quantification and applicability presumably depends on different factors:

- Regarding masonry: relative deformability between unit and mortar, brick or block masonry, hollow or solid units, mortar type, bed thickness, void or filled perpend joints, etc.
- Regarding bonding: Test have been made on header bond, stretcher bond seems *a priori* less favourable. In multiring arches with a very eccentric axial load doubts arise about which eccentricity should be considered: in relation to vault or extreme ring centre.
- Regarding structural member: walls, piers, arches, vaults and domes.
- Regarding stress-state: Shear influences negatively strength enhancement (as have shown tests[2]). In piers with eccentricity of opposite signs in their ends, it is uncertain to take into account an enhancement in strength which depends on an eccentricity lacking in a huge part of the element.
- Regarding structure condition: Some damages –such as mortar loss or the mentioned ring separation– are likely to diminish or cancel the positive enhancement behaviour.

The combined use of linear and non-linear micromodels and experiments is most likely the best way to enlarge our knowledge about this *forgotten* facet of masonry performance.

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REFERENCES

[1] Heyman, J. *The masonry arch*, Ellis Horwood, Chichester, 1982.

[2] Martínez J.L. *Theoretical and experimental determination of interaction diagrams in masonry structures and application to the analysis of historical constructions*, PhD Thesis, Universidad Politécnica de Madrid, 2003 (in Spanish).

[3] European Committee for Standardization. *Eurocode 6: Design of Masonry Structures,* 2003.

[4] Yokel F.Y., Dikkers R.D. *Strength of Load Bearing Masonry Walls*. Journal of the STRUCTURAL DIVISION. Proceedings of the American Society of Civil Engineers. May, 1971.

[5] Yokel F.Y., Dikkers R.D, Fatal. *Strength of Load Bearing Masonry Walls. Closure to the Discussion. Proceedings of the American Society of Engineers*, ASCE, may 1973.

[6] Hendry A.W. *Masonry properties for assessing arch bridges*. Contractor Report 244, Transport and Road Research Laboratory,U.K., 1990.

[7] Page A.W., Hendry A.W. *Design rules for concentrated loads on masonry*. The Structural Engineer/Vol. 66/ No. 17, 1988.

[8] Union Internationale des Chemins de fer. *Recommandations pour l'évaluation de la capacité portante des pont-voûtes existants en maçonnerie et beton.* Code UIC 778-3 1995.

[9] American Concrete Institute. *ACI 530-99. Building Code Requirements for Masonry Structures*. 1999.

[10] Sargin M. *Stress-Strain relationships for concrete and analysis of structural concrete sections*. PhD Thesis, University of Waterloo. 1970.

[11] Turkstra C.J., Thomas G.R. *Strain Gradient Effects in Masonry*. McGill Univ. Montreal, 1972.