OVERALL STRENGTH CRITERIA OF MASONRY SHEAR PIERS: DISCUSSION AND VALIDATION OF AVAILABLE MODELS

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SUMMARY

The aim of this paper is to review the models proposed in literature and codes for the verification of in-plane loaded masonry piers, discussing their reliability, with reference to different types of masonry and stress fields, and their applicability in seismic verifications with reference to the experimental evaluation of the parameters. In particular, the following issues are discussed: the consistency of the hypotheses on which they are based; the coherence of the above mentioned hypotheses with the type of material considered; the coherence between the criteria and the failure modes which they aim to describe. Numerical simulations, as well as direct comparison standard strength criteria, are provided.

INTRODUCTION

In accordance with performance-based earthquake engineering concepts, in the last decade non-linear static analysis procedures have found wide application in seismic verification methods. As well-known, these procedures are based on a comparison between the displacement capacity of the structure and the displacement demand of predicted earthquakes. The definition of the displacement capacity requires the generation of a force-displacement ("pushover") curve able to describe the overall behaviour of the structure from its elastic state, to damage degradation and finally to collapse. This curve can be generated by performing a non-linear static analysis, i.e. by subjecting the structure, idealized through an adequate model, to a static lateral load pattern of increasing magnitude (describing seismic forces). The performing of this type of analysis on masonry buildings usually requires a strong idealization of the structure. A common idealization is that of "equivalent frame", which is particularly suitable for the analysis of standard masonry buildings, made up of well connected walls with a quite regular pattern of openings. In the "equivalent frame", each resistant wall is supposed to resist only in-plane forces and is decomposed into a set of masonry panels (piers and spandrels). Each panel is supposed to act as a beam, characterized by non-linear behaviour. In this context, the definition of simplified models adequate to assess the load-bearing capacity of panels, in terms of both strength and displacement, is of fundamental relevance.

The models proposed in the literature and in seismic codes in recent decades for the seismic verification of in-plane loaded masonry panels, are generally based on the approximate evaluation of the stress state produced in the panels by the forces acting and on the assessment of its admissibility with reference to a limit strength domain. Their experimental validation has been performed adopting a predefined test layout, considering a limited set of masonry types and, often, a limited stress field. Their application in the simulation of different types of

test, different types of material and more extended stress states does not always lead to acceptable results. The reasons may be of different nature.

- Simplified methods assess the stress state in a limited number of points of the panels (usually, a local stress in a single point or mean stress on a reference section is considered). Not always is the choice of the reference point or section on which to perform the assessment of the stress state adequate. Forces acting on shear walls produce strongly non homogeneous stress states, both in the linear range, due to the boundary conditions and the applied forces, and in the non-linear range, due to the stress redistribution derived from the damaging process of the material.
- Simplified models are based on hypotheses which are valid only for a limited type of material and stress fields. Since the hypotheses introduced are far from the reality of the panels considered, the model is just as unreliable.

The aim of this paper is to review the models proposed in the literature and codes for masonry piers, discussing their reliability, with reference to different types of masonry and stress fields, and their applicability in seismic verifications with reference to the experimental evaluation of the parameters.

IN-PLANE DAMAGE MECHANISMS OF MASONRY PIERS

The observation of the seismic damage on complex masonry walls, as well as laboratory experimental tests on masonry panels, showed that masonry piers subjected to in-plane loadings may have two different types of behaviour, to which different failure modes are associated:

- <u>Flexural behaviour</u> – This typically leads to the development of tensile flexural cracking at the tense corners, inducing the pier to act as a rigid body rotating about the toe (*Rocking*). The ultimate limit state is obtained by crushing of the compressed corners.
- <u>Shear behaviour</u> – This may produce two different modes of failure. In *Sliding Shear Failure*, the development of flexural cracking at the tense corners reduces the reacting portion of the section; failure is attained with the sliding on a horizontal bed joint plane, usually located at one of the extremities of the panel. In *Diagonal Cracking*, failure is attained with the formation of a diagonal crack, which usually develops at the centre of the pier and then propagates towards the toes. The propagation can follow a stair-stepped path, passing through the head and bed joints, or a straight path, passing through the blocks.

The occurrence of these different failure modes depends on several parameters: the geometry of the pier (such as the height to width ratio, namely λ); the boundary conditions; the acting axial load; the mechanical characteristics of the masonry constituents (mortar, blocks and interfaces), in terms of both absolute and relative values; the masonry geometrical characteristics (block aspect ratio, in-plane and into-the-thickness texture, orientation of joints). Many experimental tests in the past attempted to analyse the influence of the parameters on the failure modes of masonry shear piers. In general, it has been assessed that: *Rocking* failure tends to prevail in slender piers; in moderately slender piers, *Diagonal Cracking* tends to prevail over *Rocking* and *Bed Joint Sliding* failures with increasing levels of vertical compression; *Diagonal Cracking* propagating through blocks tends to prevail over *Diagonal Cracking* propagating through mortar joints with increasing levels of vertical compression; *Diagonal Cracking* through the blocks tends to prevail over *Diagonal Cracking* through mortar joints when the ratio between mortar and block strength increases; increasing interlocking of blocks (block aspect ratio plus masonry pattern type) may induce a transition from *Diagonal Cracking* through mortar joints to *Rocking* or to *Diagonal Cracking* through blocks.

It is worth noting that it is not always easy to distinguish the different types of mechanism, since many interactions may occur between them. As will be seen in the following, this issue is reflected in verification models proposed in the literature and in codes, which often do not allow the reader to identify the failure modes considered.

Models considering the flexural behaviour of panels and describing the *Rocking Failure* usually choose the base section of the pier to assess the stress state. The normal stress acting on the bed joint plane is assumed as reference stress (σ_c); it is calculated on the basis of the beam theory, neglecting the tensile strength of joints and assuming an appropriate normal stress distribution at the compressed toe. The failure is associated with the attainment of the unilateral compressive strength of masonry normal to the bed joint plane (vertical direction y). Equilibrium leads to the following general expression:

$$\sigma_c = \frac{1}{k_{r2}} \frac{\bar{\sigma}_y}{(1 - k_{r1}\kappa)} \quad \le f_m \ , \tag{1}$$

where k_{r1} is a coefficient taking into account the slenderness and the boundary conditions of the panel; k_{r2} is a coefficient which takes into account the assumed normal stress distribution at the compressed toe (a common assumption is an equivalent rectangular stress block); κ is the ratio between the maximum overall horizontal force (V_u) and the vertical force (P) applied on the panel; $\bar{\sigma}_y = P/dt$ is the mean vertical stress acting on the section (d and t are the panel width and thickness, respectively); f_m is the compressive strength of the masonry.

Different models have been developed to describe the failure modes associated with the shear behaviour. It is possible to recognize two main types of models: models describing the failure of masonry considering its constituting components (joints or blocks) separately and models considering indistinctly the development of a crack along a principal stress direction.

Most of the models developed in the literature and adopted in codes to describe the shear failure of masonry along the joints are based on the Mohr-Coulomb criterion. However, the latter has been interpreted in different ways. The crucial point, not always clarified, is the correspondence between the adopted form and the failure mechanism predicted (*Bed Joint Sliding* or *Diagonal Cracking* through joints). The different forms can be synthesized through the following expression:

$$\tau_c = k_{1d}\bar{\tau} \quad \le k_{1s}\left(\tilde{c} + \frac{1}{k_{1s}} \tilde{\mu} \bar{\sigma}_y \right) \ , \tag{2}$$

where $\tilde{c}$ and $\tilde{\mu}$ are the cohesive and friction components of the mortar joints, respectively; k_{1d} depends on the slenderness of the panel and represents the ratio between the shear stress at the centre of the panel and the mean shear stress $\bar{\tau} = V_u/dt$; k_{1s} is a coefficient which takes into account the actual compressed part of the reference section, assumed as the ratio between its uncracked and total length.

Models describing *Bed Joint Sliding* shear failure usually consider a horizontal section at the base of the pier to assess the stress state. In this case, the mean shear stress $\bar{\tau}$ is assumed as reference stress (τ_c) and $k_{1d} = 1$. Since the base section of the pier may not be entirely compressed due to flexural effects, the mean stress is calculated on the actual compressed section through the coefficient k_{1s}. The parameters $\tilde{c}$ and $\tilde{\mu}$ are assumed to have a "local" meaning, thus representing the cohesion and the friction coefficient of mortar bed joints (Table 1).

Models describing *Diagonal Cracking* through joints usually consider the centre of the pier to assess the stress state; the actual parabolic shear stress distribution along the transversal section is taken into account through the parameter k_{1d}. Since the middle transversal section of

the pier is considered, it is assumed as entirely compressed (k_{1s}=1). The parameters $\tilde{c}$ and $\tilde{\mu}$, in this case, should not have a "local" meaning, since they must describe a complex failure of masonry, involving both mortar head and bed joints. The most corroborated formulation to define these parameters was developed by Mann and Müller (1980). This formulation is based on two main hypotheses: the bricks are much stiffer than mortar joints; the mechanical properties of head joints are negligible. Since no shear stresses can be transferred through head joints, the block is subjected to a torque; equilibrium can be attained only by a vertical force couple, leading to a non-uniform distribution of the compressive stresses on bed joints. Assuming a Mohr-Coulomb type criterion, these hypotheses lead to a definition of $\tilde{c}$ and $\tilde{\mu}$ which allows one to take into account the geometrical characteristics of the masonry pattern (see Table 1, where $\varphi = 2h/b$, h and b being the height and the width of blocks, respectively).

Table 1. Meaning of the parameter in shear failure modes.

Failure mode	k_{1d}	k_{1s}	$\tilde{c}$	$\tilde{\mu}$
Bed Joint Sliding	1	*Function of the assumed constitutive law***	c	μ
Diagonal Cracking (through joints)	*Function of the slenderness* (λ)*	1	$c\dfrac{1}{1+\mu\varphi}$	$\mu\dfrac{1}{1+\mu\varphi}$

* For $\lambda \leq 1.0$, k_{1d}=1.0; for $\lambda \geq 1.5$, k_{1d}=1.5; for 1.0<λ<1.5, k_{1d}= λ.
** Considering a NRT material and linear distribution of stresses, k_{1s}= $3(0.5-\kappa\psi\lambda)$, where ψ is a parameter taking into account the boundary conditions of the panel.

On the basis of the same mechanical hypotheses adopted for the description of the *Diagonal Cracking* through joints, Mann and Müller (1980) developed a criterion for the cracking of blocks. They observed that for increasing values of the compressive stress *Diagonal Cracking* occurring through joints is preceded by *Diagonal Cracking* occurring through blocks. Adopting the same micromechanical hypotheses introduced above, they observed that, since no shear stresses can be transferred through head joints, an approximately double shear force must be transferred through the blocks. The criterion adopts the principal compressive stress acting in the centre of a block as reference stress σ_c; this must not exceed the tensile strength of the block itself f_{bt}. The criterion may therefore be expressed in the following form:

$$\sigma_c = \frac{\bar{\sigma}_y}{2} + \sqrt{\left(k_{1d}k_{2d}\bar{\tau}\right)^2 + \left(\frac{\bar{\sigma}_y}{2}\right)^2} \leq f_{bt} \quad . \tag{3}$$

where k_{2d} is the ratio between the mean shear stress applied on the block and the local shear stress at its centre. It has been demonstrated that k_{2d}=2.3 for standard masonry where φ=0.5. Among the models which consider indistinctly the development of a crack along a principal stress direction, the most widely used was originally proposed by Turnšek and Čačovič (1971). It considers as reference stress σ_c the principal tensile stress acting at the centre of the pier σ_I. Masonry is considered as an isotropic material and the hypothesis of neglecting the horizontal normal stresses σ_x, which have zero mean on the horizontal section but are not null in the centre of the wall, is introduced. It is assumed that the reference principal stress σ_c must not exceed a "reference" tensile strength of masonry f_t:

$$\sigma_c = \sigma_I = \frac{\bar{\sigma}_y}{2} + \sqrt{\left(k_{1d}\bar{\tau}\right)^2 + \left(\frac{\bar{\sigma}_y}{2}\right)^2} \leq f_t \quad . \tag{4}$$

DISCUSSION OF THE CRITICAL ISSUES

In this paragraph two issues of different nature will be discussed. The first is related to an "extrinsic" feature of the criteria introduced above, i.e. their use in verification methods. The second is related to an "intrinsic" feature, i.e. the reliability of their hypotheses.

Each resistance criterion defines a mechanical interpretation of a specific failure mechanism of the masonry pier. In order to obtain the limit domain, the choice of the most suitable criteria and their combination are necessary. The suitability is related to two main aspects: the actual occurrence of each failure mode for different levels of compression; the coherence between the mechanical hypotheses introduced and the type of masonry considered.

Even if it is well known that many parameters influence the failure mode, various experimental research programs and earthquake damage assessment showed that *Diagonal Cracking* has a fundamental relevance among the possible failure modes, whereas *Bed Joint Sliding* failure is restricted to a few instances, usually corresponding to rather stiff piers subjected to low levels of vertical load and/or low friction coefficient.

Many codes, due to the strong simplifications needed for their applicability, propose criteria or limitations which are not directly referred to specific failure modes. For example, the proposal, contained in Eurocode 6, to consider only the compressed part of the transversal section in the shear failure (EC6 3.6.2), leads to recognizing the end section as reference one. Therefore, the proposed criterion seems to refer to a *Bed Joint Sliding* failure mode. Following this assumption, the friction coefficient should be representative of "local" properties of the masonry, depending on the type of blocks and interfaces adopted. However, the Eurocode assumes it as a constant value equal to 0.4, independently from the type of masonry. For new brick masonry, the value of 0.4 could appear rather low. It might be interpreted as the result of the application of Mann and Müller's theory, obtained considering a local friction coefficient μ = 0.6 and a value of φ typical of standard masonry (as done in DIN 1053-1).

The reliability of the prediction given by a criterion depends on the coherence between the mechanical hypotheses on which it is based and the type of masonry considered. For example, it is reasonable to expect that the criterion proposed by Turnšek and Čačovič works well for "nearly isotropic" masonries; its reliability diminishes with increasing levels of anisotropy (Figure 1). In this latter case, the criterion proposed by Mann and Müller seems more suitable.

Figure 1. Masonry types.

The use of a model depends on: the technical possibility of evaluating the parameters required through experimental tests; the sensitivity of the strength prediction on the uncertainty of these parameters. In common practice, the most widely used typologies of experimental tests are: the triplet test, the diagonal compression test and the racking test. The first is suitable to define local mechanical properties of masonry, such as c and μ. The second allows one to define the reference tensile strength of masonry (f_t). The third can be used with different purposes: the result of a single test could be theoretically sufficient to define the reference tensile strength of masonry (f_t); two tests could theoretically provide global parameters of

masonry, such as $\tilde{c}$ and $\tilde{\mu}$. In the case of a racking test and diagonal compression test, the coherent interpretation of the results is an issue of fundamental relevance. Indeed, since depending on the compression level at which different failure mechanisms may occur, it is always necessary, to interpret the experimental results on the basis of the occurred failure mechanism and of the masonry typology considered. This issue will be further clarified in the final paragraph.

Furthermore, it is worth pointing out that practical or technical reasons do not always allow one to perform the experimental tests required for a given model (this is the case, for example, of existing buildings in which the extraction of material to be tested should be minimized): as a consequence, the choice of a criterion risks to be forced by the available experimental results. It could therefore be useful to establish a correspondence between the different mechanical parameters. It is worth noting that these correspondences (such as that between the cohesion c and reference tensile strength of masonry f_t from the diagonal compressive test) have been scarcely clarified in the literature.

Finally, as regards the sensitivity of the strength prediction on the uncertainty of the experimental parameters it can be observed that, relating to the flexural behaviour, in the presence of low axial load (frequently occurring in complex masonry buildings), the value of the compressive strength f_m has a limited influence on the ultimate strength of the masonry panels. On the contrary, relating to the shear behaviour, it is worth noting that the adoption of micromechanical models which require "local" parameters, may induce a spread of the results in the transition to the scale of the panel.

Regarding the reliability, that is the "intrinsic" feature introduced above, the most significant issues to delve into are related to the theoretical coherence of the assumptions which the various criteria are based on. For this purpose a set of numerical non linear analyses has been performed, as widely discussed in the following section. They are mainly directed at investigating two questions: firstly, by what amount the actual stress distribution differs from the simplified distribution assumed in each criterion, considering that a transition from the elastic to the non-linear range may occur; secondly, whether the choice of establishing the maximum shear capacity of the panel referring only to some specific points/sections is correct, considering that further resistance resources may be possible in relation to the stress redistribution.

NON-LINEAR PARAMETRICAL ANALYSES

A set of parametrical analyses with a different combination of aspect ratios of the piers and different levels of axial loads has been performed. The finite element method, together with a non-linear constitutive model for masonry (Calderini and Lagomarsino 2007) has been adopted. The model was developed with a micromechanical approach, considering the plane stress hypothesis and neglecting the mechanical resistance of the head joints (thus assuming them as geometrical discontinuities).

The range of the axial load variation assumed was such as to cause a mean vertical stress $\bar{\sigma}_y$ varying between the values 0.05÷0.8 of the masonry compressive strength f_m. Three configurations of walls have been investigated, respectively characterized by slenderness λ = 0.65 (Panel 1), λ =1.35 (Panel 2), and λ = 2 (Panel 3). A fixed-fixed boundary condition was imposed. The same mechanical properties were adopted for all the piers. The latter choice is motivated by the will to particularly deepen the influence of the geometrical aspects rather than the mechanical ones on the maximum resistance attainment of the pier.

The mechanical properties assumed are representative of a running bond pattern masonry characterized by interlocking φ =0.5, tensile strength/cohesion ratio equal to 0.2, compressive strength/cohesion ratio equal to 27. They correspond to the ones characterizing the piers tested in Ispra by Anthoine et al. (1995); preliminary numerical analyses showed the effectiveness of the non-linear constitutive model to reproduce the masonry behavior (Calderini and Lagomarsino 2007).

Occurrence and development of the failure mechanisms

For low values of $\bar{\sigma}_y$, the failure modes that occurred may be classified as *Rocking*. For higher values of $\bar{\sigma}_y$, in the case of Panel 1 and Panel 2, the prevailing mechanism is *Diagonal Cracking*: in Panel 1, it is associated with two inclined pseudo-parallel cracks, whereas in Panel 2 with only one crack propagating from the centre of the panel. The increasing of $\bar{\sigma}_y$ leads to a transition from *Diagonal Cracking* through joints to *Diagonal Cracking* through blocks. In the case of Panel 3, the prevailing mechanism is *Rocking*, even though for high values of $\bar{\sigma}_y$ it has been noticed the development of diagonal cracking starting from the end sections interested by flexural cracking (named in Figure 4 as "*Mixed behaviour*"). This circumstance was also observed in experimental tests.

In order to evaluate whether the pier has further resistance resources after the attainment of the limit strength of the material in the reference points/sections or not, the evolution of the stresses τ and σ_I (principal tensile stress) was monitored.

For Panel 1 and Panel 2, in those cases in which a shear response is activated, the attainment of a peak force is well recognizable. In particular, the difference between the values of V in correspondence of the load step in which τ and σ_I attain the maximum values and its maximum value V_u resulted quite moderate (generally within 5%). This observation validates the hypothesis that the limit strength of the pier can be reasonably predicted on the basis of the attainment of limit strength condition of the material in only few reference points/sections.

Transition of the stress distribution from the linear to the non linear range

In order to analyse the coherence of the hypothesis which the above mentioned criteria are based on, the study of the stress distribution appears of fundamental relevance. Particular attention should be paid to the transition from the initial elastic phase to the non-linear one.

In the following, for some meaningful sections (the end sections and the one at the center of the wall), the evolution of the stress distribution corresponding to different drift values, marked on the force-displacement (V-u) curves summarized in Figure 2, is described. In Figure 2 the following dimensionless notation is adopted: for the central section, the stress components σ_x (perpendicular to the head joints) and σ_y (perpendicular to the bed joints) are referred to the mean vertical stress $\bar{\sigma}_y$, while the tangential component τ is referred to the mean shear stress $\bar{\tau}$; otherwise for the end section the adimensionless form is computed with reference to the masonry compressive resistance f_m. The figure is referred to a mean vertical stress $\bar{\sigma}_y$ = 0.6 MPa.

As far as the σ_x stresses are concerned, we can distinguish a first phase (the approximately "elastic" one) in which they are quite moderate, almost negligible. They result in tension for both Panel 2 and Panel 3 because of the expansion effect derived from the axial load application; on the contrary, for Panel 1, they result in compression: this phenomenon is ascribable to the confinement effect at the center of the panel as a consequence of its very squat configuration. Proceeding to the non-linear response, we can observe significant

changes: they progressively translate from tension to compression; this phenomenon greatly increases until the attainment of the maximum resistance of the panel (that is the peak of the *V*-*u* curve). It can be observed that they result in a more significant entity corresponding to decreasing values of the slenderness.

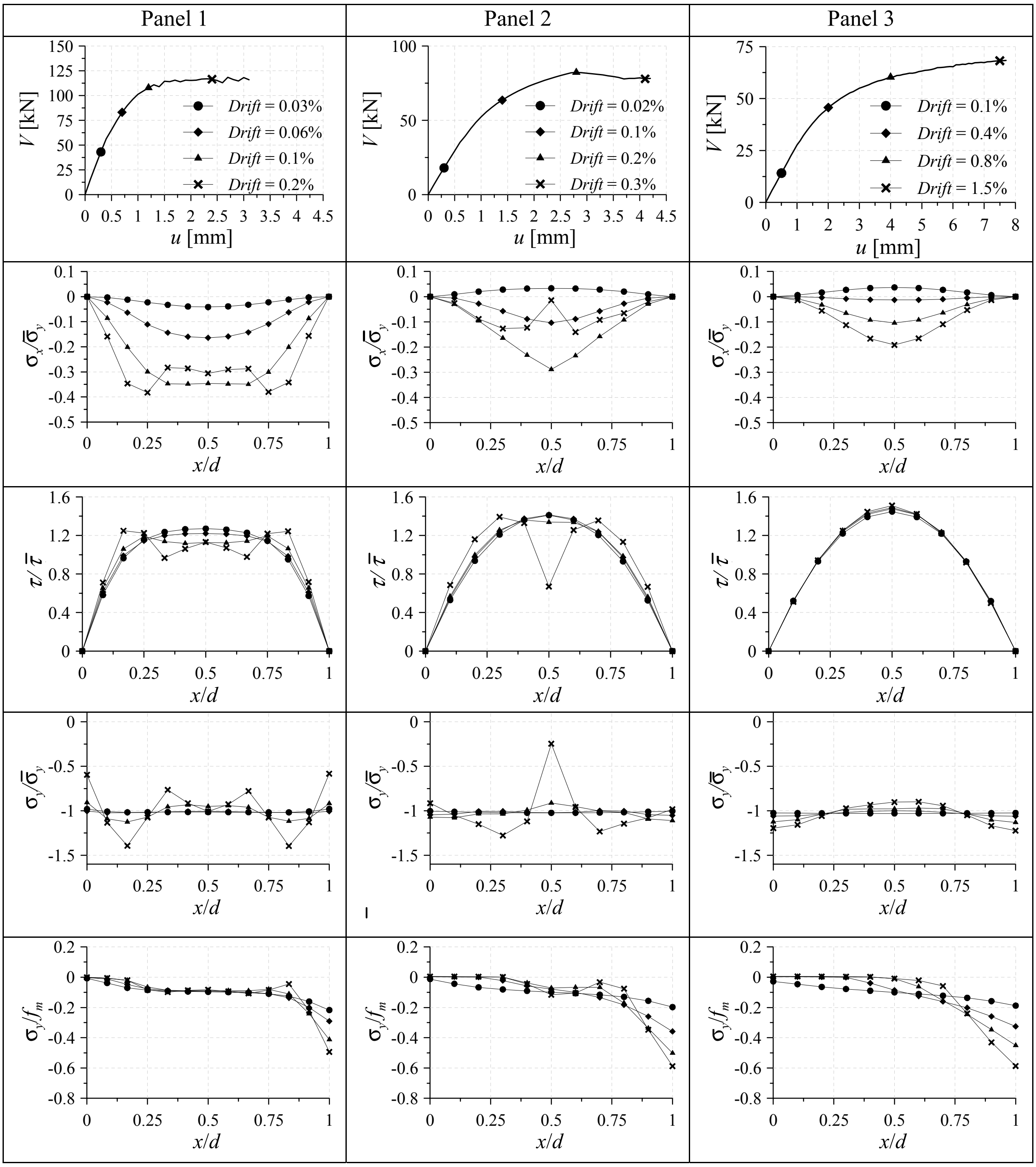

Figure 2. Stress evolution in the reference points/sections of the panels, with reference to different drift values.

Referring to Panel 2, we can observe the development of a crack starting from the center of the wall just corresponding to the attainment of the maximum resistance; for greater values of drift, the damage worsens causing an inactive region. This fact is evident observing the sudden fall of all stress components at the central cross section for drift of 0.3 % (softening

phase). Analogous occurrence can been found on Panel 1 as a consequence of the development of two diagonal parallel cracks. On the contrary in the case of Panel 3, in which the prevalent collapse mechanism is related to a flexural response, it is not possible to observe this fall.
Aiming to evaluate the coherence of the k_{1d} coefficient, the tangential stress distribution will now be investigated. Starting from the initial parabolic distribution, the peak of the $\tau/\bar{\tau}$ ratio decreases: as a consequence of the non linear phenomena, the distribution flattens attaining to the maximum resource even for elements contiguous to the central one. For Panel 1, the ratio $\tau/\bar{\tau}$ passes from 1.33 to 1.15, whereas for Panel 2 from 1.44 to 1.37. On the contrary, for Panel 3 the initial value equal to 1.48 remains almost unvaried; it is worth highlighting that this value results particularly coherent with the hypothesis of De Saint Venant idealization, undoubtedly justifiable for this geometry. We can conclude that the hypothesis of assuming the k_{1d} coefficient as a function of the slenderness is generally consistent enough, even though the usually assumed lower bound k_{1d}=1, for $\lambda < 1$, does not seem precautionary. In fact, for Panel 1 the k_{1d} coefficient does not come down to the 1.15 value.
Finally, regarding the σ_y component, the analysis of the stress evolution in the end section allows us to highlight the progressive reduction of the effective un-cracked section length, particularly evident in the case of Panel 3 governed by the *Rocking* failure mechanism.
As shown by the comparison of the stress distribution between the pseudo-elastic and non-linear phases, one of the greatest disagreements consists in the onset of compressive stress component of σ_x at the central section. This progressive evolution can be interpreted analysing the variation of the mechanism which governs the panel response. Indeed, as a consequence of the spread of the tensile flexural cracking at the end section, the panel gradually starts to behave as an equivalent strut. Figure 3 shows this phenomenon for Panel 2, by means of the representation of the compressive principal stress directions in correspondence of two successive drift values. The occurrence of this phenomenon has been monitored for each panel, by varying the axial load. The entity and the effect of the stresses σ_x diminishes with increasing values of $\bar{\sigma}_y / f_m$.

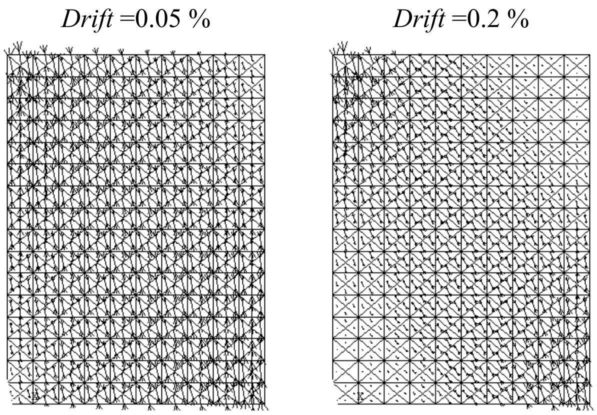

Figure 3. Compressive principal direction for different drift values (Panel 2, $\bar{\sigma}_y$ = 0.6 MPa).

Comparison between numerical and analytical strength domains

Figure 4 shows the comparison between the limit strength domain obtained from numerical analyses (V_u) and the domain predicted by the above illustrated criteria. In Equation (4) a mean value of f_t (obtained from the numerical results for $\bar{\sigma}_y$ = 0.6 MPa) has been considered. In general, a good correlation with the criteria expressed by equations (1) and (2)-(3) can be

observed. The good agreement between the numerical analyses and the criterion proposed by Mann and Müller, rather than that of Turnšek and Čačovič, might only be related to the similarity of the hypotheses adopted in the two cases. However, this fact can be also attributed to the coherence of these latter hypotheses with the type of masonry examined. Indeed, it is worth noting that experimental tests carried out on Panel 2, characterized by the same masonry considered here, showed the occurrence of *Diagonal Cracking* though joints (Anthoine et al. 1995).

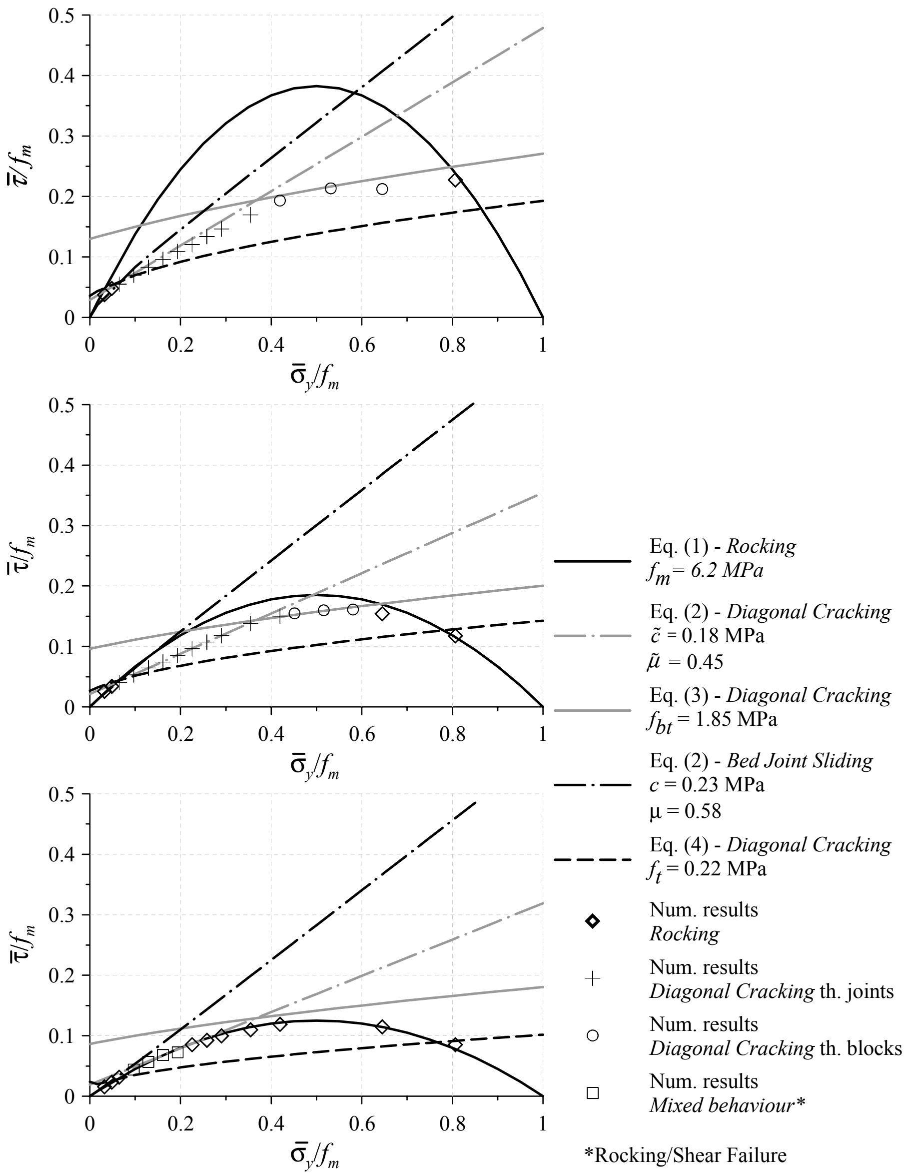

Figure 4. Comparison between numerical and analytical strength domains.

The comparison between the domains leads to the following conclusions:

- The assessment of the stress state in a few points of the panel seems to be sufficient to predict the strength of the entire panel.
- The effect of compressive stresses σ_x previously evidenced does not seem to significantly affect V_u; this can be stated considering that neither of the criteria considered takes into account this stress component.

- Even though the criterion of Turnšek and Čačovič seems to provide quite a good prediction of the strength for low values of the ratio $\overline{\sigma}_y / f_m$ (which after all correspond with the typical ranges considered in experimental tests), it might lead to a strong underestimation of the strength for higher axial loads.

FINAL REMARKS

The discussion of the models reported in the literature and the non-linear parametrical analyses lead to following conclusions.
On the basis of the results obtained, the assessment of the stress state in a few points of the panel seems to be sufficient to predict the failure of the entire panel.
The correct use of the criteria requires the assessment of the following points: the consistency of the hypotheses on which they are based; the coherence of the above mentioned hypotheses with the type of material considered; the coherence between the criteria and the failure modes which they aim to describe.
On of the most critical points seems to be the alternative use of the models proposed by Turnšek and Čačovič (1971) and Mann and Müller (1980), since markedly different predictions may be obtained with the two models. The first appears more suitable to describe nearly isotropic masonries. Isotropy can be related to two extreme cases: regular pattern masonries characterized by components (block and mortar joints) of similar strength and stiffness; strongly irregular masonries in which the randomness of the pattern does not allow one to recognize predefined planes of weakness. The second appears to be more suitable to describe regular masonries characterized by "weak joints -strong blocks" (in terms of both strength and stiffness).

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