Confined masonry members: a method for predicting compressive behaviour up to failure

A. Mandara & G. Palumbo

Department of Civil Engineering – Second University of Naples – Italy

ABSTRACT: The behaviour of masonry elements confined with steel tied plates and subjected to axial compression is faced in this paper. The problem is investigated by means of an analytical approach, calibrated against the results of a non linear F.E.M. numerical analysis, carried out with the ABAQUS code. The theoretical procedure allows the evaluation of the effect of confinement in both horizontal directions, accounting for the inelastic behaviour of both masonry and steel. In spite of the reduced number of factors involved, the proposed model proves to be reliable in the prediction of the main behavioural aspects of confined masonry and, for this reason, it can be considered as an useful tool for practical calculations.

1 INTRODUCTION

Confinement of vertical load-bearing masonry or stone elements is a traditional practice used for either recovering existing damage or for increasing their ultimate capacity. The basic principle of confinement relies upon the application of a compressive action in one or more directions transverse to that of the applied load, so to achieve conditions of multi-axial compressive stress. This leads to both an increase of the member compressive strength compared with the case of uniaxial stress, and also to a remarkable improvement of the ductility properties at failure (Mandara 2002).

Confinement interventions can be applied not only to single structural elements, like columns or walls, but also to a greater extent, for example encircling the whole building with a suitable system of tensioned members. In all cases, a quite rational structural system is obtained, in which in-situ materials are exploited in the most rational and effective way. The most common way to apply confinement is by means of metal elements working in tension, such as tie-bars or tie-beams, fastened to the masonry by means of contrasting end-plates, as shown in Figure 1 (Mazzolani 1996; Mazzolani 2002; Mandara 2002). To this purpose, most adopted material in such interventions is steel, both mild, low carbon and high strength grade, even though in the last years innovative materials such as titanium alloys and Shape Memory alloys have started to be applied with interesting results (Mazzolani & Mandara 2002). Tension in the ties can arise either as a consequence of member lateral expansion, or due to external prestressing. In this view, confinement can be defined passive or active, respectively. In some cases, a combined procedure can be also used, in particular when a long-term adjustment of the confining force is required. In this case, due to the use of externally fastened plates, the intervention can be arranged in such a way to be easily controlled or removed, if necessary. This aspect complies with a commonly adopted policy, aiming at preventing existing buildings, in particular when they possess monumental value, from irreversible and/or inappropriate restoration operations (Mazzolani & Mandara 2002).

Even though commonly acknowledged as an effective provision and traditionally adopted in current practice, an exhaustive framing of confinement problems from the theoretical point of view is, surprisingly, not yet reached. As a consequence, no procedures for the prediction of the load bearing capacity of confined masonry members are available at all. For this reason, a theoretical procedure for the prediction of the effect of confinement, which accounts for the actual inelastic behaviour of both masonry and steel, is dealt with in this paper. A F.E.M. model running on the ABAQUS code is used for the calibration of the proposed procedure. To this purpose, the simulation model has been preliminarily fitted on the basis of some experimental results available in literature (Ballio & Calvi 1993). The outcome of the numerical simulation is then used to fit the parameters of the theoretical model. As a conclusion of the study, some general indications about the prediction of failure type are also given.
In spite of some simplifications introduced into analytical developments, the method proposed can be considered as a first attempt to the direct evaluation of the load-bearing capacity of confined masonry members.

2 THE THEORETICAL MODEL FOR CONFINED MASONRY

2.1 General

The development of the model presented herein started some years ago for the case of masonry uniformly confined along one transverse direction by means of tied end-plates (Mandara & Mazzolani 1998). In this case, the collapse can occur due to bar yielding only (Figure 2a). Other possible failure types are the collapse by crushing (punching) of the masonry in the confined area (Figure 2b) and the collapse by shear-tension failure of masonry in unconfined areas (Figure 2c). According to tests referred to in (Ballio & Calvi 1993), the main geometrical magnitudes influencing the collapse mechanism are the plate-to-bar cross section ratio \( A_p/A_s \) and the ratio between the unconfined wall length and the wall thickness \( (l/t) \).

As expected, the use of small plates gives place to failure by punching of the masonry, whereas a comparatively great distance between plates involves collapse by shear-tension in the unconfined area. Relatively high values of \( A_p/A_s \), instead, produce failure by bar yielding, which is also the collapse type providing the best performance in terms of ductility.

The procedure set out in (Mandara & Mazzolani 1998) has been further refined in (Mandara & Scognamiglio 2003), in order to take into account collapse mechanisms other than bar yielding, and is presented in this paper in its complete formulation, referring to the general case of confinement along two orthogonal directions. As the model was purposely conceived for design applications, it has been based on the simplified assumption of homogeneous continuum. For this reason, in case of masonry with complex texture, it requires the preliminary application of suitable homogenisation criteria in order to get the equivalent masonry properties (Nemat-Nasser &
Hori 1999). The main assumptions are the same as in (Mandara & Mazzolani 1998), namely:

- isotropic behaviour of masonry;
- elastic-perfectly plastic behaviour of steel bars;
- fully rigid steel confining plates, which results in the confining force to be evenly distributed across the wall side surface; as a consequence, the steel bars can be assumed as concentrated;
- pseudo-elastic relationship between the applied stress \( \sigma_m \) and the confining stresses \( \sigma_{c,x} \) and \( \sigma_{c,y} \) holding in both elastic and post-elastic range, which permits the use of Navier-like equations written in terms of secant modulus \( E_{m,s} \);
- masonry behaviour in compression, both confined and unconfined, described by means of an appropriate non-linear \( \sigma - \varepsilon \) law, with experimentally fitted parameters.

The model is presented here referring to the case of bi-directional uniform confinement, that is the case of members with square or rectangular cross-section confined in both transverse directions (Figure 3).

### 2.2 Constitutive relationships for confined masonry

Referring to Figure 4, the equilibrium equations along the wall transverse directions must be satisfied:

\[
A_{s,x} \sigma_{s,x} = -A_{p,x} \sigma_{c,x} \\
A_{s,y} \sigma_{s,y} = -A_{p,y} \sigma_{c,y}
\]

(1)

The global strain in the masonry in both load \( \varepsilon_{m} \) and transverse direction \( \varepsilon_{c,x} \) and \( \varepsilon_{c,y} \) is expressed by means of Navier-like equations written in terms of secant modulus, which means that both the elastic and plastic part of the global deformation are taken into account at the same time:

\[
\varepsilon_{m} = \frac{1}{E_{m,s}} \left( \sigma_{m} - v(\sigma_{c,x} + \sigma_{c,y}) \right) \\
\varepsilon_{c,x} = \frac{1}{E_{m,s}} \left( \sigma_{c,x} - v(\sigma_{m} + \sigma_{c,y}) \right) \\
\varepsilon_{c,y} = \frac{1}{E_{m,s}} \left( \sigma_{c,y} - v(\sigma_{m} + \sigma_{c,x}) \right)
\]

(2)

where \( E_{m,s} \) is the secant modulus of compressed masonry. When steel bars are in elastic range \( \varepsilon_{c,x} = \varepsilon_{c,y} = \varepsilon_{c} \) and \( \varepsilon_{c} = \sigma_{c}/E_s \) being the steel elastic modulus (Figure 4). By means of suitable algebraic manipulations of Equations (1) and (2), the confinement stresses \( \sigma_{c,x} \) and \( \sigma_{c,y} \) can be obtained:

\[
\sigma_{c,x} = \sigma_{m} \left( 1 - v \right) \beta + \sigma_{c} \beta \\
\sigma_{c,y} = \sigma_{m} \left( 1 - v \right) \beta + \sigma_{c} \beta
\]

(3)

By substituting Equation (3) into any of Equations (2) the following expression for \( \sigma_{m} \) can be obtained (Mandara & Palumbo 2003):

\[
\sigma_{m} = \frac{\varepsilon_{E_{m,s}} \left( 1 + v \right) E_{m,s} \beta + \varepsilon_{c} E_{m,s} \beta}{\varepsilon_{E_{m,s}} + \varepsilon_{c} + \varepsilon_{E_{m,s}} \left( 1 - v \right) \beta + \varepsilon_{c} \beta}
\]

(4)

Assuming that both \( E_{m,s} \) and \( v \) are a function of the compressive strain \( \varepsilon_{m} \), then Equation (4) may be considered as the \( \sigma - \varepsilon \) law of the confined masonry. The above equations hold until the stress in the tensioned
bars does not exceed the steel yield stress $f_y$. When this occurs, assuming that bars in both directions are yielded, from Equations (1) it results:

$$\sigma_{c,x} = -f_y \frac{A_{s,x}}{A_{p,x}}; \quad \sigma_{c,y} = -f_y \frac{A_{s,y}}{A_{p,y}}$$

(5)

where the steel yield stress $f_y$ has to be taken as negative. Confining stresses $\sigma_{c,x}$ and $\sigma_{c,y}$ are given in Equation (5). By substituting Equations (5) into any of Equations (2), the stress in the load direction becomes:

$$\sigma_m = E_{m,x} \varepsilon_m - f_y \left( \frac{A_{s,x} + A_{s,y}}{A_{p,x} + A_{p,y}} \right)$$

(6)

The above formulation can be easily extended to cases other than bi-directional confinement. For example, in case of walls confined along the transverse direction $x$, the position $\varepsilon_{c,y} = 0$ can be made. Correspondingly, the following equations are obtained:

$$\sigma_m = E_{m,x} \varepsilon_m - f_y \left( \frac{A_{s,x}}{A_{p,x}} \right)$$

(7)

$$\sigma_{c,x} = \frac{(v + v^2)\sigma_m}{1 - v^2 - (v + v^2)\left(1 - v^2 + \left(\frac{E_{m,x}}{E_{s,x}}\right)\left(\frac{A_{p,x}}{A_{s,x}}\right)\right)}$$

(8)

for steel bars in elastic range, and:

$$\sigma_m = \frac{E_{m,x} \varepsilon_m - (v + v^2) f_y A_{s,x}}{A_{p,x}} \left(1 - v^2\right)$$

(9)

$$\sigma_{c,x} = -f_y \frac{A_{s,x}}{A_{p,x}}$$

(10)

for steel bars in plastic range.

Similarly, for columns or males confined in the $x$ direction only, one may assume $\varepsilon_{c,y} = 0$ and the corresponding equations for steel bars in elastic range are:

$$\sigma_m = \frac{E_{m,x} \varepsilon_m \left(1 - v^2\right) + A_{p,x} E_{s,x} + A_{s,x} E_{m,x}}{A_{p,x} E_{m,x} + A_{s,x} E_{s,x}}$$

(11)

$$\sigma_{c,x} = \frac{A_{s,x} E_{s,x} \varepsilon_m}{A_{p,x} E_{m,x} + A_{s,x} E_{s,x}}$$

(12)

whereas for yielded bars they become:

$$\sigma_m = E_{m,x} \varepsilon_m - f_y \frac{A_{s,x}}{A_{p,x}} \varepsilon_f$$

(13)

$$\sigma_{c,x} = -f_y \frac{A_{s,x}}{A_{p,x}}$$

(14)

2.3 Application of the procedure

For the above method to be applied, appropriate functions for $E_{m,s}$ and $v$ have to be assigned. The secant modulus $E_{m,s}$ can be obtained starting from a suitable $\sigma - \varepsilon$ relationship for plain masonry. A number of $\sigma - \varepsilon$ relationships susceptible to be used exist in the technical literature, derived from experimental tests on either plain concrete, or fitted directly on masonry specimens. In this procedure, a model derived from the Saenz's law for concrete has been proposed (Sargin 1971):

$$\sigma_{m,s} = \frac{k}{\left(\frac{E_{m,s}}{E_{m,u}}\right)^2 - 2} \left(\frac{E_{m,u}}{E_{m,s}}\right) + \left(\frac{E_{m,u}}{E_{m,s}}\right)^2$$

(15)

where $E_m, \sigma_{m,u}$ and $E_{m,u}$ are the initial elastic modulus, the ultimate compressive stress and the corresponding strain, respectively. Such a model represents a slight variation of that proposed in (Mandara & Mazzolani 1998) and provides a good description of both pre-and post-collapse compressive behaviour of masonry. Compared to the original formulation of the Saenz’s law, there are some differences in the model presented herein, namely a strength enhancement factor $k$ due to confinement in order to take into account the increase of masonry resistance produced by the combined state of stress and the exponent $v$ introduced instead of a numerical factor equal to 2, in order to obtain a more accurate reproduction of the softening branch of the $\sigma - \varepsilon$ relation. Assuming values of $v$ other than 2 causes the actual maximum of the $\sigma - \varepsilon$ curve to be slightly different from $\sigma_{m,u}$, but gives a much better approximation of the material post-collapse behaviour. Both $k$ and $v$ have been found being basically dependent on both mechanical and geometrical properties of the masonry wall. An appropriate expression for $k$ can be put into the form:

$$k = 1 + \alpha \left(\frac{\sigma_{c,x}}{\sigma_m} + \frac{\sigma_{c,y}}{\sigma_m}\right)$$

(16)

where $\alpha$ is a numerical coefficient to be fitted experimentally, which depends on the masonry features. The $\sigma_{c,x}/\sigma_m$ and $\sigma_{c,y}/\sigma_m$ ratios can be evaluated from equations given in the previous paragraph as a function of $v, E_{m,s}, E_s, A_{p,s}, A_{p,y}, A_{s,x}$ and $A_{s,y}$.

The expression of secant modulus can be easily calculated from Equation (15), obtaining:

$$E_{m,s} = \frac{E_m}{1 + \alpha \left(\frac{\sigma_{c,x}}{\sigma_m} + \frac{\sigma_{c,y}}{\sigma_m}\right)}$$

(17)
In the evaluation of \( \sigma_{c,x}/\sigma_m \) and \( \sigma_{c,y}/\sigma_m \) a trial-and-error procedure would be necessary for their calculation. In fact, since both \( \sigma_{c,x} \) and \( \sigma_{c,y} \) depend on \( E_{m,s} \) through \( \sigma_m \), its value depends on \( k \) itself, too. Nevertheless, it can be observed that, for practical purposes, the terms of Equation (16) may be computed in an approximate way assuming \( k = 1 \) in the evaluation of \( E_{m,s} \), independently of the bar yielding. The related inaccuracy can then be covered by an appropriate value of the parameter \( \alpha \).

An accurate estimation of \( \nu \) is extremely important for the reliability of the model, that is for a correct evaluation of the confinement effect. Unfortunately, the meaning of the Poisson’s modulus in masonry is not exactly the same as in an elastic continuum, in particular when the collapse load is approached. The transverse expansion of the masonry is, in fact, strongly influenced by the onset of cracks along the load direction. Also, the actual masonry texture, that is the block size and configuration as well as the mortar properties, should be considered in the assumption of the \( v(\varepsilon_m) \) function. In this view, a direct evaluation of the transverse expansion ratio \( v \) leads to a so-called “apparent” Poisson’s modulus, whose mechanical meaning is far different from the one of an elastic, isotropic continuum. Some existing tests (Faella et al. 1993), in fact, indicate apparent values of the coefficient \( \nu \) at the ultimate strength equal to or higher than 1.5-2, depending on the masonry features. Such values, clearly incompatible with the physical meaning of \( \nu \), cannot be assumed in case of confined masonry, as the development of cracks is counteracted by confining ties. In this case, cracks occur as well, but with a rather different aspect from the case of unconfined masonry. In absence of reliable data on \( \nu \) under combined stress conditions, assuming that in such a case a reduction of the void volume due to the local crushing of the masonry could take place, it seems more appropriate to assign a law for \( \nu \) reaching values not higher than 0.5 (no volume change) in the large displacement range. This is more complying with the assumption of isotropic continuum made for deriving the \( \sigma - \varepsilon \) law. At the same time, the condition that \( \nu = 0 \) for \( \varepsilon_m = 0 \) should be fulfilled, this corresponding to what commonly observed in tested specimens. As being stated, a possible law for \( \nu \) can be put into the form:

\[
\nu(\varepsilon_m/\varepsilon_m) = \left( \frac{\varepsilon_m}{\varepsilon_{m,s}} \right)^b \left[ a + b \left( \frac{\varepsilon_m}{\varepsilon_{m,s}} \right)^b \right]^{1/b} \tag{18}
\]

where coefficients \( a, b \) should have to be determined in order to adequately reproduce the results coming from either numerical simulation or direct experimentation. Since the Poisson’s ratio is responsible for the lateral expansion of the masonry and, consequently, for the confining pressure of the steel plates, the coefficients \( a \) and \( b \) can be fitted in such a way that the bar yielding occurs at the same point as in the real cases. On the contrary, the \( \alpha \) factor, taking into account the effect of confinement on the masonry strength, can be evaluated by considering the actual value of ultimate load bearing capacity, as deduced by experimentation or by F.E.M. analysis.

3 PARAMETRIC ANALYSIS

3.1 The F.E.M. model

The non-linear F.E.M. code ABAQUS release 6.2.1 (2002) has been used for the calibration of the theoretical model described before, starting from experimental data available in (Ballio & Calvi 1993). The analysis pointed out the influence of confinement on both the ultimate strength and ductility of compressed masonry, showing all the effectiveness of this practice and also giving some hints on how to obtain a given failure mechanism.

Eight-node reduced integration C3D8R elements have been used. The standard material model *CONCRETE embedded in ABAQUS has been adopted for modelling masonry (Mandara & Scognamiglio 2003; Mandara & Palumbo 2003). Because of its good accuracy in the reproduction of the progressive development of cracking, the consequent tension stiffening effect and the actual shape of the interaction domain at failure, it can be also used for interpreting the response of masonry with an acceptable degree of approximation. In true, such behaviour is much more complex than concrete due to the strong non-isotropic feature of masonry, as well as to the block-to-mortar interaction. The latter effect is dominant in case of large block stone masonry, that is where block size is potentially larger than than that of confining plates, or when the quality of mortar used is very poor. As this would involve a failure mechanism strongly different from that predicted assuming material isotropy, the validity of this calibration should be limited to relatively small sized blocks, with mortar of good quality, so as to approach the isotropic behaviour as much as possible.

The behaviour of steel bars has been described using the *ELASTIC and *PLASTIC material keys, by means of which an accurate elastic-perfectly plastic \( \sigma - \varepsilon \) law has been reproduced, allowing for a small hardening in order to reduce numerical convergence problems. Contact problems between the steel plates and the underlying masonry have been properly taken into account, in order to consider possible slip phenomena. To this purpose the *SURFACE INTERACTION facilities have been exploited. Due to the post-critical softening in the compressive response involved by any collapse mechanism, the *RIKS algorithm has been used. Also, allowance for geometric second order effects has been made.

Results of 10 tests on ancient masonry wall specimens coming from the collapsed Civic Tower of Pavia,
Italy, (Ballio & Calvi 1993) have been assumed as a benchmark for the set up of the numerical procedure. Specimens have been chosen in such a way to achieve all possible collapse types (Figure 2), namely: 1) collapse by yielding of steel bars; 2) collapse by crushing (punching) of the masonry in the confined area; 3) collapse by shear-tension failure of masonry in unconfined areas. Figure 5 depicts the wall deformed shape predicted by ABAQUS for the three collapse mechanisms highlighted in the tests, together with the contour of equivalent Von Mises stress and horizontal displacement of the wall. Based on the distribution of such displacements, it is easy to distinguish collapse mechanisms from each other. The comparison between the numerical simulation and the test results is shown in Figure 6.

3.2 Description of the analysis

An extensive parametric analysis has been carried out by means of the F.E.M. model described in the previous paragraph (Figure 7), assuming several material properties. The analysis has been concerned with the
Table 1. Synopsis of masonry mechanical properties assumed in the parametric analysis and relevant material calibration parameters ($z$, $a$, $b$ and $\alpha$).

<table>
<thead>
<tr>
<th>Masonry type</th>
<th>$E_m$ (MPa)</th>
<th>$\sigma_{m,u}$ (MPa)</th>
<th>$\varepsilon_{m,u}$</th>
<th>$t$ (mm)</th>
<th>$\Phi$ (mm)</th>
<th>$z$</th>
<th>$a$</th>
<th>$b$</th>
<th>$\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3300</td>
<td>3.5</td>
<td>0.0025</td>
<td>480</td>
<td>8,12,16</td>
<td>1.5</td>
<td>1</td>
<td>2.1</td>
<td>0.2</td>
</tr>
<tr>
<td>2</td>
<td>2300</td>
<td>2.5</td>
<td>0.0025</td>
<td>300</td>
<td>8,10,12</td>
<td>1.8</td>
<td>3</td>
<td>2</td>
<td>0.5</td>
</tr>
<tr>
<td>3</td>
<td>660</td>
<td>2.5</td>
<td>0.007</td>
<td>300</td>
<td>4,6,8</td>
<td>2</td>
<td>3</td>
<td>1.8</td>
<td>0.5</td>
</tr>
</tbody>
</table>

In order to consider collapse mechanisms other than yielding of steel bars, partial confinement with uniformly spaced plates has been also considered (Figure 9). In such a case, depending on both plate spacing and $A_{kx}/A_{p,x}$ ratio, collapse by either local crushing or shear-tension may occur. For a given $A_{kx}/A_{p,x}$ value, the corresponding $i/t$ value has been assumed as relevant parameter to establish whether the wall collapse occurs due to bar yielding, masonry punching or shear-tension failure of masonry between plates. As long as the $i/t$ ratio increases, the corresponding wall collapse load decreases, according to the trend shown in Figure 10, where the ratio between confined and unconfined strength $R_c/R_{nc}$ is plotted against $i/t$ ratio, for $A_{kx}/A_{p,x} = 50$, 100 and 400. In general, for $i/t \geq 1.5$, the effect of confinement vanishes completely. Correspondingly, the failure mode moves from bar yielding to punching or shear/tension depending on the value of $A_{kx}/A_{p,x}$.

As a conclusion, a synopsis view of all possible failure conditions is given in Figure 11, where the relevant collapse mechanisms are also indicated. An approximate border line between them has been traced, which can give useful indication from the design point of view. In practice, collapse conditions other than that
involved by bar yielding should be avoided, in that they cause confinement ineffectiveness and/or local crushing of masonry, and hence, a brittle behaviour at collapse for the wall.

Concerning practical calculations, an easy tool for the prediction of collapse load for a given value of \( i/l \) ratio is needed. The solution to this problem would involve the definition of a very complex mechanical model, taking into account all relevant aspects of the collapse mechanism. Such difficulty arises due to the fact that in case of bar yielding or local punching the wall collapse occurs due to masonry crushing between confining plates, whereas in case of shear-tension failure it is predominantly a matter of local instability of outer masonry leaves.

Appropriate interaction models should be used to represent the actual collapse phenomenology of confined masonry. A simplified procedure is proposed herein for engineering purposes, based on the combined use of both F.E.M. results and theoretical prediction for uniformly confined masonry. In this case, results coming from the theoretical model can be used for \( i/l = 0 \), whereas F.E.M. data can be exploited to reproduce the variation of the wall strength as long as the \( i/l \) ratio increases. Curves in Figure 10 can be used to this purpose, in order to fit a simple relationship relating \( R_c/R_{nc} \) ratio to \( i/l \) ratio. The following equation is proposed:

\[
\frac{R_c}{R_{nc}} = \left[ \left( \frac{R_c}{R_{nc}} \right)_{i/l=0} - 1 \right] \left[ \frac{0.1}{\left( \frac{R_c}{R_{nc}} \right)_{i/l=0} - 1} \right]^{-0.87/(i-l)} + 1 \tag{19}
\]

whose results are plotted in Figure 11 as well. Such equation can be used to predict the resistance of confined masonry for a given value of the \( i/l \) ratio when the resistance ratio of uniformly confined \( (i/l=0) \) to unconfined masonry \( (R_c/R_{nc})_{i/l} \) is known. Information on the related collapse mechanism is then obtained from Figure 11.
4 CONCLUSIONS

The behaviour of masonry members confined by steel ties has been faced in this paper. To this purpose, an ad-hoc theoretical procedure has been developed, which is able to predict the response of uniformly confined masonry members up to failure. The procedure has been calibrated on the basis of results coming from a non-linear F.E.M. simulation carried out by means of the ABAQUS code. The analysis has led to a thorough understanding of the global behaviour of masonry in such loading conditions, highlighting all relevant collapse mechanisms. With respect to existing models, mostly concerned with confined concrete in compression, the theoretical procedure discussed herein is based on a reduced number of parameters, to be fitted on the basis of either experimentation or numerical simulation. With a suitable choice of these factors, the model exhibits a satisfying degree of accuracy, while remaining comparatively easy to apply in practical cases. Also, collapse mechanisms other than that involved by bar yielding have been investigated by means of numerical simulation, leading to the attainment of some general conclusions on the most suitable range of geometrical and mechanical properties to be adopted in practice. At the same time, F.E.M. results have been further exploited in order to set a simplified procedure for the evaluation of the member load bearing capacity as a function of confinement ratios \( \tau / t \) and \( A_{p,x} / A_{x} \). Results achieved confirm the procedure to be suited to engineering purposes.

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