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INNOVATIVE TECHNOLOGIES FOR THE SEISMIC IMPROVEMENT OF MASONRY AND A SYSTEMATIC APPLICATION AFTER EARTHQUAKE

Antonello Salvatori [1] **Marianna Leonori**[2] **Simone Salvatori**[3]

[1] University of L'Aquila
Piazzale E. Pontieri 2, Monteluco di Roio, L'Aquila, Italy, e-mail: antonello.salvatori@univaq.it

[2] EDIL CAM® Technical Office Systems
CAM® processing executor, mail: Marianna.leonori@edilcam.it

[3] Consultant Engineer
Torino, Italy, e-mail: simsalv97@gmail.com

Abstract

The conservation and protection of historic buildings, which can often conflict with the tools of seismic protection, can be afforded by considering two main approaches to the problem. The first concerns the reduction of the seismic demand on the structure, the latter increases the resistance capacity of the structural elements. Excluding cases of monumental buildings, where the needs of conservation are very often prevalent over the anti-seismic techniques (when and where applicable), in the case of most historic buildings, protection can be compatible by applying the correct innovative methods of anti-seismic retrofitting.
A system of structural consolidation based on the creation of a three-dimensional grid, made with tension tie-rods steel strips, which confines limited portions of masonry, is analyzed, and an analytical model is examined. The formation of a three-dimensional grid allows to obtain the condition of triaxial compulsion, capable of giving greater compressive strength to the masonry volume, due to the effect of induced confinement, and at the same time the bindings constitute reinforcement and offer load-bearing capacity in traction and without increasing mass. This system is the first to have introduced and engineered three-dimensional consolidation by mechanical action. Several masonry mechanisms involve the behaviour of the consolidated masonry, which are all examined in the paper.
A noticeable application to a historical masonry building aggregate, dating eight centuries, is illustrated.

Keywords: Seismic isolation, earthquake, structure monitoring.

1 INTRODUCTION

The needs of conservation and protection of historic buildings often conflict with the needs of seismic protection and safety to be ensured both for the structures and for the occupants, especially in those

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buildings of historical interest, headquarters of public institutions or schools, in which the exposure factor becomes relevant for the purpose of evaluating the intervention to be carried out on the building. There are two main approaches to the problem of seismic retrofitting or improvement of existing buildings. One is linked to the reduction of the seismic demand on the structure, and therefore we are in the field of isolation and dissipative systems, the other linked to the increase in the resistance capacity of the structural elements, and therefore to the whole vast family of structural consolidation systems. Apart from the borderline cases of monumental buildings, where evidently the needs of protection are very often prevalent over the types of anti-seismic intervention (when and where applicable), in the case of historic buildings (public and private) of an ordinary nature, protection can be-come compatible through the correct application of traditional or innovative methods of anti-seismic restoration.

A system of structural consolidation based on the creation of a three-dimensional grid, made by means of tension tie-rods, in steel strip, which individually circle limited portions of masonry, is analyzed, and an analytical model is examined. The formation of a three-dimensional grid allows to obtain a final condition of triaxial com-pulsion, capable of giving greater compressive strength to the wall volume due to the effect of induced confinement, at the same time the bindings constitute reinforcement and offer load-bearing capacity in traction and without increasing mass. This system is the first to have introduced and engineered three-dimensional consolidation by mechanical action. The CAM® system consists of the application of a metal reinforcement, made with steel strips measuring 19x0.90 mm that run on both sides and in the thickness of the walls, so as to obtain a three-dimensional reticular system of horizontal, vertical and transverse tie rods. Each tie rod can be made as an overlapping of coils individually tensioned with a special pneumatic machine, calibrated in its pulling action. The seams are sequential and connected, usually forming a double warp. Special metal elements consisting of deep-drawn plates allow the forces acting on the wall plane to be closed (thus exempting the masonry) at the node points and at the same time distribute the compression action that is activated in the transverse direction; the set of ribbons that cross the wall thickness constitutes a mechanical connection. The tensile strength Nt,Rd is assumed to be equal to the lower of the two values:

- Npl,Rd plastic resistance of the current section A to the characteristic yield stress.
- A minimum resistance equal to 70% of the resistance of the intact strip is guaranteed at the breakage of the reduced section at the junction.

Several masonry mechanisms involve the behaviour of the consolidated masonry, which are all examined in the paper.

A noticeable application to a historical masonry building aggregate, dating eight centuries, is illustrated.

2 2009 L'AQUILA EARTHQUAKE

L'Aquila city was struck down by a 6.3 Mw and seismic moment M0 = 3.7 x 1018 N m (according to INGV) earthquake in 2009 April 6th. Its historical centre and all the surrounding suburbs were severely damaged, causing 309 casualties, and more than 1500 people were injured. L'Aquila has been the first Italian important city directly destroyed by a near fault earthquake since Messina earthquake (1908). Many buildings collapsed completely, both in masonry structure and in reinforced concrete ones. Many buildings suffered heavy structural damage, like shear cracks in the pillars, shear cracks in the concrete walls, nodal ruptures, and even total or partial collapses. Some more recent buildings evidenced noticeable structural damage, mainly due to design errors and constructive inadequacy.

Registered data showed response spectra very different due to the local amplification effects.as shown in Fig. 1. Response spectra evidenced a local strong amplification in correspondence of the high frequencies (0 – 3 Hz) in the suburbs (Mount Pettino west area, in correspondence with an active local fault), where several reinforced concrete buildings were heavily damaged also in structural elements.

The damage can be associated to dynamic resonance in correspondence of the highest values in the response spectra, due to the reinforced concrete building characteristics (main frequency often in the range 2.0 – 10.0 Hz). Some differences, due to soil amplification effect, were found in the center of the city, as shown in fig. 3. The local site effect reveals itself noticeable in relationship with the dynamic behaviour of seismic isolated buildings.

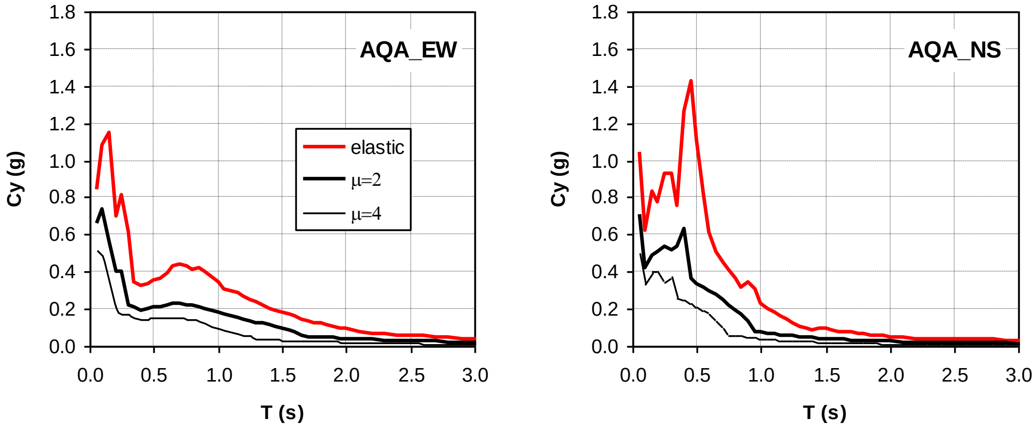

Figure 1. Response spectra in L'Aquila west area (near fault), amplification effect at 10.0 – 2.0 Hz

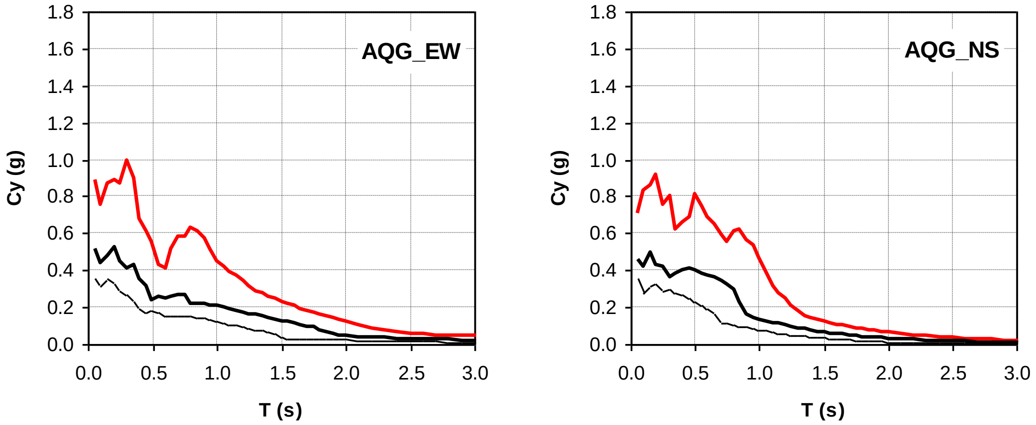

Figure 2. Response spectra in L'Aquila central area, with secondary amplification effect at 0.6 – 0.7 Hz

3 POST EARTHQUAKE RECONSTRUCTION AND CAM® RETROFITTED BUILDINGS

The issue of seismic adaptation and improvement of masonry buildings of historical and artistic interest assumes national importance both in relation to the recent earthquakes (L'Aquila 2009, Emilia 2012, Amatrice – Norcia 2016), and to the protection of assets of cultural interest.

As is well known, these buildings are, due to their intrinsic artistic and architectural characteristics, subject to the special protection referred to in Legislative Decree 42/2004, and to the related guidelines for structural interventions.

In particular, the needs of conservation and protection of historic buildings often conflict with the needs of protection and seismic safety to be ensured both for the structures and for the occupants, especially in those buildings of historical interest, headquarters of public institutions or schools, in which the exposure factor becomes relevant for the purpose of evaluating the intervention to be carried out on the building.

The references proposed by the Guidelines of the Ministry for Cultural Heritage propose, in absolute, the intervention of seismic improvement as an intervention inherent in the restoration of buildings.

In addition, it should be noted that the many ordinances issued following the L'Aquila earthquake of 6 April 2009, against a minimum percentage of seismic improvement between 60% and 80% for all buildings damaged by the earthquake not subject to protection, explicitly provide for a seismic improvement, for historic buildings subject to protection, in which it is not necessary to obtain, as a result of structural interventions, a minimum percentage of seismic improvement.

Apart from the borderline cases relating to monumental buildings, where evidently the needs of protection are very often prevalent over the types of anti-seismic intervention (when and where applicable), in the case of historic buildings (public and private) of an ordinary nature, protection frequently becomes an obstacle to the correct application of traditional or innovative methodologies of anti-seismic restoration, being, moreover, deriving from subjective assessments of the elements responsible for the assessment of the protection itself.

The work describes some interventions on buildings of historical interest, located in L'Aquila, damaged by the 2009 earthquake, in which the techniques of recovery and restoration of the buildings conform with the needs of seismic improvement.

The application of the C.A.M. method in these buildings involves and solves many of the typical aspects of the protection of cultural heritage, in particular, it is effective in solving many of the problems inherent in masonry buildings at the same time. After

The probabilistic concepts indicated in the guidelines for cultural heritage are in contrast with the deterministic evaluation of the seismic event based on geophysical evaluations, so that an erroneous belief is induced that the anti-seismic intervention based on a reduced time scale (the nominal life) will protect the construction for a smaller number of years in probabilistic terms.

This, in reality, does not correspond with the need to protect the building of interest with respect to the MCE (Maximum Credible Earthquake) for the location site. In fact, the maximum event, depending on geophysical quantities (in art not known), is not affected by the memory effect, so the probability of occurrence of the maximum event is unchanged with each new intervention.

Consequently, the reduction in a low percentage of the seismic vulnerability of protected building does not entail the real protection of the building itself, nor are the real safety conditions met for the number of years expected (return period) for a given event of low probability. This is because, a priori, an event that falls within a return period with respect to that established on the basis of the percentage of seismic improvement of the interventions proposed for historic buildings cannot be discarded from possible events, even if probabilistic and non-deterministic.

Furthermore, on the basis of the guidelines, the interventions must generally be aimed at individual parts of the structure, containing their extension and number as much as possible, and in any case avoiding significantly altering the original distribution of stiffness in the elements, where it is possible to evaluate the seismic behavior of the main masonry elements according to the concept of stiffness. On the contrary, the evaluation of the global behavior of the structure can make it possible to simultaneously make a seismic improvement with respect to the entire structure, and with respect to the local kinematics that can be activated in the complex building organism.

The types of damage following the seismic events mentioned can be divided into two large classes:

GLOBAL
Bad connections between walls
Lack of connections between horizontals and walls
Collapse due to instability
Detachment-overturning of the masonry elements

LOCAL LEVEL
Bad connections between vestments
Disorderly wall texture and poor-quality binder
Displacement Collapse

The main types of collapse, which involve the aspects highlighted, can be effectively counteracted through the use of technologies that combine structural strength with the conditions of equilibrium that are very often lacking in masonry structures.

In fact, in the collapse due to articulation-displacement between rigid bodies, there is a lack of equilibrium, while, in order to also use the ductility resources of the masonry, it is necessary to involve and use the resources of resistance of the material, through the increase of the global equilibrium, involving the complete connection of the systems (walls, floors, ...), the equilibrium of the systems of closed forces (pushing roofs), the re-centering of the actions (eccentric floors on the thickness), and the increase of the local equilibrium (compaction of the masonry element, compensation of the internal thrusts of the expulsion of the material in order to increase the resistance of the masonry).

Therefore, the application of the C.A.M. method to protected buildings guarantees both anti-seismic safety and, through its reversibility, the conservation and historical protection of masonry buildings.

4 THE C.A.M.® METHOD IN THE RESTORATION OF MASONRY BUILDINGS

The CAM system consists in the creation of tension tie rods using steel strips ($s \leqslant 1$ mm) that individually circle limited portions of masonry, forming a three-dimensional coercive grid (post tension in the masonry plane, post tension in the direction of the masonry thickness).

The acronym CAM® is the trademark and patent that represents the technology of *Active Seams of Artifacts*. It contains a system of structural consolidation that in a nutshell can be said to be based on the creation of a *three-dimensional grid*, made by means of *tension tie-rods*, in *steel strips*, which *individually* circle limited portions of masonry

The individual structural components of the system are very simple:

high-strength stainless steel strips

deep-drawn plates

Angles and plates of embossed sheet metal

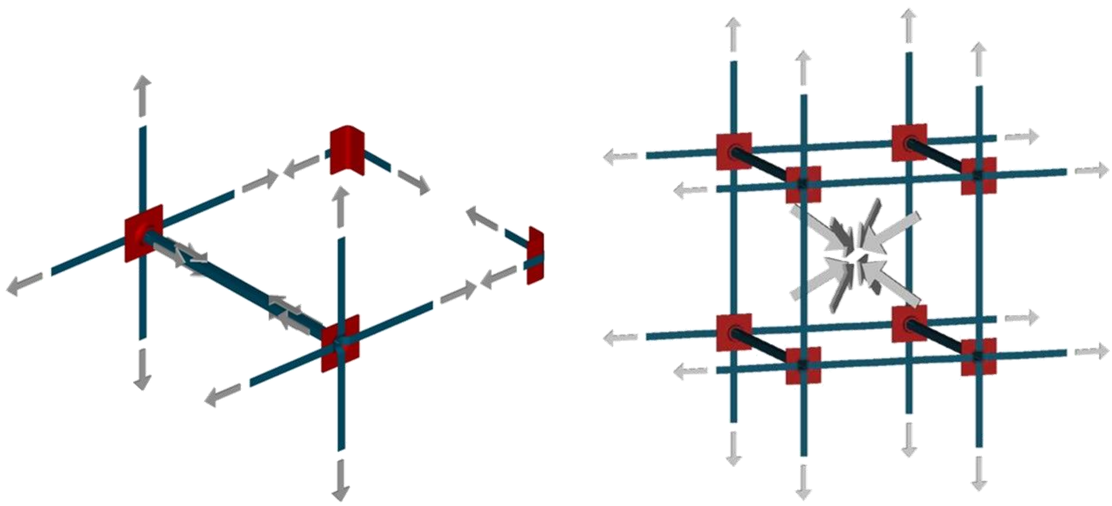

Figure 1 -Triaxal behavior of prestressed tridimensional strips (courtesy CAM® System)

The formation of a three-dimensional grid allows to obtain a final condition of triaxial compulsion, capable of giving greater compressive strength to the wall volume due to the effect of induced confinement, at the same time the bindings constitute reinforcement and offer load-bearing capacity in traction and without increasing mass.

This system is the first to have introduced and engineered three-dimensional consolidation by mechanical means.

From a more technological point of view, the CAM® active seams system consists of the application of metal reinforcement, made with steel strips (19 mm wide and less than 1 mm thick) that run on both sides and in the thickness of the walls, so as to obtain a three-dimensional reticular system of horizontal, vertical and transversal tie rods. Each tie rod can be made as an overlapping of coils or circles, individually tensioned with a special tensioning machine, calibrated in its pulling action.

The seams are sequential and connected, usually forming a double warp. Special metal elements consisting of deep-drawn plates allow the forces acting on the plane of the wall to be closed at the node points and at the same time distribute the compressive action that is activated in the transverse direction; the set of ribbons that cross the wall thickness constitutes a mechanical diatone.

The post-tensioning in the transversal direction of the masonry is realized, in stratified masonry with several facings, a real artificial diatone, which solidifies and binds the facings together, preventing one of the typical collapses of such masonry, and therefore the disastrous consequences that this entails in global terms.

The type of reinforcement therefore achieves a mechanical connection of the facings, the absorption of thrusts inside the facade and the re-centering of the eccentricities.

The compressive force on the masonry is an active force, which does not need seismic action to excite its resistance (as happens in other types of intervention, such as reinforcement with reinforced concrete). This prevents the masonry facings from having transverse relative displacements (and therefore cracks), before the intervention of the artificial diatone, allowing the response in terms of resistance (and therefore ductility) and not only in terms of limit equilibrium.

This results in a three-dimensional coercion grid inside the masonry, effective according to the pitch of the strips and their overlapping.

The effectiveness of the post-tensioning in the steel strips, and therefore of the prestressing of the masonry, is increased according to the correct ratio of the spacing of the strips to the thickness of the masonry, in order to allow the correct compression of the masonry in all areas of the same (Figure 4).

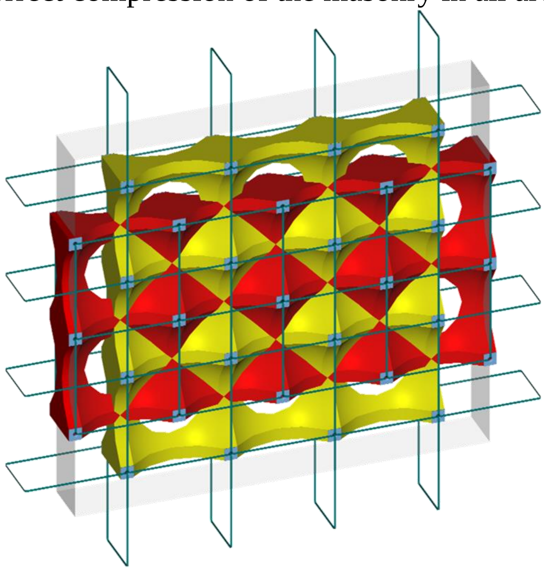

Figure 2 – Confinement of compressed masonry (courtesy CAM® System)

For masonry with an irregular texture, like stone masonry, the shear strength is governed by diagonal tensile failure (limit failure for reaching the shear strength in the criterion of shear failure diagonal cracking), while for regular masonry failure may also occur when the strength along the mortar beds is reached (limit failure for reaching the shear strength in the sliding shear failure criterion); for these types of masonry, both failure mechanisms must be checked and the shear strength will be the lower of the two.

The normal modulus E and tangential elasticity G are to be considered relative to non-cracked conditions, so that, depending on the type of analysis conducted, the stiffnesses can be suitably reduced.

This reduction is generally assumed to be 50% (see p.to 4.1.1.1 and 7.2.6 NTC2018). In linear analyses, the non-cracked geometry is modeled and therefore it is legitimate to consider the final behavior of the cracked masonry by reducing the stiffness up to a maximum of 50% of their initial value. In the pushover analysis, if the behavior of the masonry panel is described with the entire development of the M-φ curve, the mechanical parameters E and G can be taken with their full value unless the behavior of the masonry is modeled by means of a three-phase diagram (elastic phase – cracked – plastic).

Table C8.5.I of Italian code NTC 2018 shows, for the behaviour of the most recurrent masonry types, non-binding indications on the possible values of the mechanical parameters, identified through the survey of the construction aspects (§C8.5.2.1) and relating to specific conditions: lime mortar of modest characteristics (average compressive strength f_m estimated between 0.7 and 1.5 N/mm^2), absence of recurrences (listatures), facings simply juxtaposed or poorly connected, texture (in the case of regular elements) in a workmanlike manner, unconsolidated masonry. For seismic verification purposes only, in the event that the mortar has particularly poor characteristics (average compressive strength f_m estimated at less than 0.7 N/mm^2) a reduction coefficient of 0.7 for strengths and 0.8 for elastic modulus is applied to the values in the table. The parameters indicated in the table are mainly aimed at verifying seismic actions.

Compressive strength is also used for checks with regard to non-seismic actions, provided that the possible possibility of local instability phenomena, associated with an insufficient connection between the facings, is also considered.

Of course, this kind of procedure doesn’t take into account for shear forces absorption by steel stripes, so, often, resistance and ductility for masonry seems to be underestimated.

In order to interpret past phenomena and/or to try to estimate future phenomena in the long term, it is good to consider that in the masonry material there are phenomena consisting of slow, plastic and viscosity deformations, with mechanical behaviors that are also considerably different from those in the short term, which can give rise to a redistribution of stress peaks.

The values indicated for regular masonry refer to cases in which the texture complies with the rule of the art. In cases of incorrect weaving (vertical joints not adequately staggered, horizontality of the rows not respected), the values of the table must be adequately reduced.

Of course, it is advisable to carry out as many direct or indirect tests on the masonry (single/double flat jacks, endoscopies, sonic tests, etc.) in order to consciously assume the values shown in the table.

It is also specified that walls made with two facings simply placed side by side or with "sack" fillings of poor mechanical characteristics present a high risk of instability, which can be accentuated by the presence of horizontals resting only on one of the facings and by the absence of effective anchoring between the floors and the external facings of the walls. The risk of instability, which is greater in stone walls, is also present in cases of squared stones on external surfaces.

If there is no risk of instability of the individual facings, the wall can be considered as composed of two walls simply placed side by side, each with a thickness equal to its effective section.

After excluding the possibility of detachment mechanisms between the facings, in the event that the internal core is large with respect to the facings, and in particular if poor, it is advisable to reduce the resistance and deformability parameters of the external facings.

In the case of an internal core of substantial thickness, the equivalent mechanical properties of the masonry, to be attributed to the entire thickness of the wall, are to be obtained starting from those of the facings (Table C8.5.I, possibly modified by the coefficients of Table C8.5.II) and of the core, through appropriate evaluations.

In the particular case of an internal core with negligible mechanical characteristics, the equivalent properties of the wall panel can be obtained, precautionarily and simplified, neglecting the thickness of the core.

These clarifications, explicitly reported by the Circular, focus attention on the primary and most dangerous mechanism for masonry buildings, which is the disintegration of the masonry mass.

The basic hypothesis of any theoretical consideration that can be made on a masonry building is precisely about assuming masonry as a homogeneous and isotropic material. The knowledge of our built environment leads the technicians to the awareness that this assumption is certainly a stretch, both for regular textures and even more for poor masonry textures. This is why the possibility that the breakage of the masonry face occurs due to disintegration must be evaluated as a preliminary step, i.e. the masonry material does not behave as a homogeneous material but as a set of microelements in contact with each other with a very low resistance to horizontal actions (the binder is the only friction).

A building that manages to activate the resistance in its panel plane succeeds in activating 'ductile' collapse mechanisms, in which the protection of life is at least ensured by horizontal actions of low magnitude.

Fragile mechanisms, i.e. those mechanisms that lead to the premature breakage of the building or portions of it, are therefore connected to the lack of constraint:

- At the local level, i.e. at the level of wall texture
- On a global level, i.e. as an absent constraint between macro-elements such as walls - walls and walls - slab.

In the case of **disordered masonry texture**, in which the binder has poor adhesion characteristics or in which the size is made with small and/or rounded stone elements or in the case of sack masonry with double facing and pushing core, the masonry breaks due to disintegration. Masonry does not behave as a homogeneous material but as a set of stone elements resting on top of each other whose resistance to horizontal actions is extremely low.

The masonry load-bearing structures are thus forced to balance external actions not so much thanks to a resistance of the material but to a resistance by equilibrium to the dislocation of the blocks.

The mechanism of brittle collapse due to displacement of stone elements is the first brittle mechanism to be avoided. There is no simplified scheme for evaluating the mechanism of disintegration, while the mechanism of breaking the double vestment can be calculated through the following scheme.

If the masonry device is made compact and homogeneous in its mechanical behaviour, it will be possible to 'read' the masonry structure as a set of bodies such as macro-elements of the wall and floors. It is the absence of connection between these macroelements that leads to the formation of the subsequent fragile mechanisms of collapse such as overturning facades or portions of buildings.

5 IDENTIFICATION OF STRUCTURAL CRITICALITIES AND ANALISYS STRATEGY

Regardless of the analysis carried out, the first approach must necessarily be aimed at the evaluation of the overall behavior of the structure.

Global intervention must be understood and constructed as the sum of local interventions, which must take into account the intrinsic situations and boundary conditions, in order to homogenize and make *behaviors, resistances and rigidities coherent and congruent*.

In general, the interventions must be aimed at improving the structural functionality therefore linked:

- the *re-centering between the centers of gravity of masses and stiffnesses* (regularization of the main modal shapes, minimization of torsional effects),
- the reduction of any irregularities in plan and elevation,
- the elimination of local collapse mechanisms,
- to the increase in strength and ductility of the individual structural components.

As already mentioned, the CAM® mechanical system does not alter the distribution of mass and stiffness, but guarantees an increase in terms of resistance (the use of tape is to all intents and purposes additional reinforcement) and by confining limited portions of masonry it allows significant increases in ductility. The great advantage, however, to which its application is linked is certainly compared to the local collapse kinematics, as the CAM® diffused mesh allows to sew together not only any facings of a sack masonry but also walls orthogonal to each other.

In fact, the greatest contribution that the mechanical system offers, often implicitly through the application of a widespread mesh, is in the context of limiting local collapse mechanisms. This aspect, even if not always assessable in terms of numbers, is nevertheless absolutely preparatory to the good overall behavior of the building.

Only by realizing the box effect, the masonry building behaves like a three-dimensional structure and therefore only then did the results obtainable from the global analysis through a three-dimensional calculation model have meaning.

The *linear analysis* will lead to the identification of the crisis as *overcoming the maximum resistance*. With reference to what has been said, the following critical situations will be identified on the individual element:

- ***CRISIS due to COMPRESSION:*** exceeding the limit value of the axial load N_{rd}.
- ***CRISIS due to PRESSURE BENDING*** in the plane and out of plane: exceeding the limit value of the resisting moment $M_{rd,x}$ and $M_{rd,y}$, indicating with the subscripts x and y the two directions of the element in the plane.
- ***SHEAR CRISIS*** (sliding or diagonal cracking): exceeding the limit value of the shear resistant load $V_{d,sc}$ and $V_{rd,fiss}$ whose subscript refers to the type of failure identified by the position of injury.

The *project of the intervention* will therefore consist in *the calculation of the resistant increase* that the element reinforced will have to achieve.

The limit of such an analysis therefore appears to be that it would tend to bring attention from the behavior global to the local one of the single elements, of the individual elements, which lead to the crisis.

In the context of a *non-linear analysis*, the attention is strongly shifted towards the overall behavior of the structure. The analysis may be interrupted by the premature collapse of a few elements and therefore by the inability to involve large portions of the structure.

When, for example, plasticization occurs almost simultaneously in large portions of the structure, this is symptomatic of a good structural distribution of strengths and deformation capacities.

Therefore, in this case, it will be necessary *to identify which elements do not collaborate in the good overall behavior* and to identify the cause understood as reaching the limit of resistance or deformability. One strategy is, for example, the increase in the shear capacity of the element so that the limit failure is the flexural one, therefore with a greater displacement capacity.

What has already been described also applies to the non-linear analysis: the extension of the intervention, albeit minimal and 'out of calculation', allows to increase the constraints between macro-elements. It is therefore not advisable to limit oneself to the local consolidation of the single element that goes into crisis, but it is good *to proceed iteratively until a good global involvement of the available resources is obtained*.

The confinement pressure f_1, of a wall panel of width L and thickness t confined with CAM tapes with horizontal pitch p_{fh} (from which $p_{fh,eff} = 2 \cdot p_{fh}$) is given by:

$$f_1 = \frac{1}{2} min\{f_{yd,h} \cdot \rho_{sx}; f_{yd,h} \cdot (\rho_{sy} + 2 \cdot \rho_{by})\} = \frac{f_{yd,h}}{2} \cdot min\{\rho_{sx}; \rho_{sy,tot}\} = \frac{f_{yd,h}}{2} \cdot \rho_{s,min}$$

Where ρ_{sx}, ρ_{sy}, ρ_{by} are the geometric percentage of C.A.M.® strips in the various direction og each masonry panel.

The performance coefficients in the horizontal and vertical in plane directions are:

$$k_H = \left[1 - \frac{1}{3 \cdot L \cdot t} \cdot \left((c_{sx} - 2 \cdot R)^2 + \left(p_{fh,eff} - 2 \cdot R\right)^2 \cdot \left(n_{fori} - 1\right) + (t - 2 \cdot R)^2 + (c_{dx} - 2 \cdot R)^2\right)\right]$$

$$k_V = \left(1 - \frac{p'_{fv}}{2 \cdot min\{L,t\}}\right)^2 = \left(1 - \frac{p_{fv} - b_f}{2 \cdot min\{L,t\}}\right)^2$$

The analysis of this formulation shows that:

- The closer the geometry of the reinforcement (the single mesh) is to a square shape in plan, the greater the magnitude of the confinement effect on the strength (maximization of the horizontal efficiency coefficient)
- The thicker the bindings, the more the effectiveness of the reinforcement improves (maximization of the vertical efficiency coefficient)

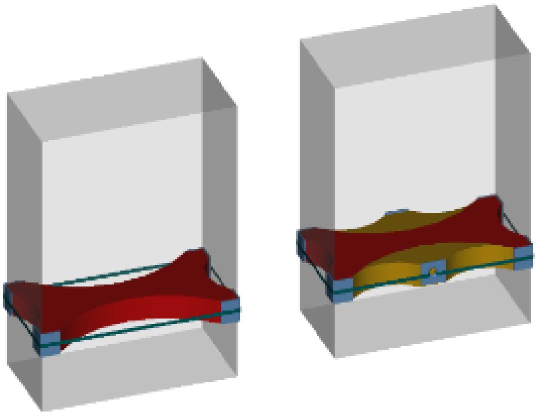

Figure 3 - Evidence of the increase in material involved as a result the thickening/regularization of the mesh (horizontal efficiency)

The confinement carried out with the CAM® System increases the resistance capacity to centered compression of the masonry element and consequently also increases its ultimate deformation and therefore its ductility.
The press-bending design of masonry, similarly to concrete, can be performed with both linear and nonlinear calculations. In the case of nonlinear verification, the choice of the stress-strain diagram is fundamental, from which the stresses in the masonry and reinforcement will be deduced, once the deformations are known.
For this diagram it is possible to adopt appropriate models constituting the behavior of the material, defined based on the calculation strength f_{md} and the ultimate deformation εmu, set equal to 0.0035, while for the intermediate values ε_{m2}, ε_{m3}, ε_{m4} refer to the standard in which they are specified for the different models.
The effect of the pre-stress of the tapes can be taken into account by considering the initial deformation state, similarly to what is done in the calculation of the prestress. The tapes will start from a non-zero stress and deformation state.
The calculation must therefore consider both the initial deformation state of the masonry and the additional reinforcement, identifying the overlapping regions. Each ligature will be considered for the actual state of stress, based on the achievable deformation. It must therefore be considered that, even in the portion of masonry subject to compression, there will be tensile reacting belts, if the deformation is maintained such as not to 'discharge' the belt from the pretension imposed on it in the initial phase.

5.1 SHEAR RESISTANCE

Shear failure presents considerable interpretative difficulties related to the dispersion of the values of the experimental strength of the panel (typical effect of brittle breaks) and the difficulty of predicting the distribution of local stresses at the time of failure. For these reasons, the evaluation of shear strength is essentially based on simplified methodologies. Typically, the most widely used approaches are based on the maximum tensile stress criterion and the Mohr-Coulomb criterion. The first criterion considers the failure due to diagonal cracking and is typical of chaotic masonry (the only criterion proposed in the Regulations for the verification of existing masonry), while the second is the failure due to sliding, typical of block or listed masonry in which the break is located along the mortar recurrences (the only criterion proposed in the Regulations for the verification of newly built masonry).
To highlight the functioning of the two criteria just enunciated, it may be useful to 'read' the trend of the stresses inside a wall panel supposedly double-wedged and subject to a horizontal action at the head.
The trend of the axial stress and shear diagram is constant, while because of the bending (maximum in the head and foot) the trend of the internal compressions (compression reacting area) will be as shown in the figure.
In the extreme section, the bending stress is maximum, therefore there will be a minimum length of the area subject to compression (depth of the neutral axis), therefore maximum compressive stress. Linked to this, there will be, in a frictional criterion, maximum shear resistant tension; in this section the shear stress will also be maximum, as it too is expressed only in the reacting area.
The direction of the crack will be orthogonal to the main tensile stress, from which the criterion of knotted shear to the tensile strength of the material derives.
The resistance to be considered, however, since both are possible breakages even on existing buildings, will be the lower of the two.
In both collapse mechanisms, the horizontal taping of the CAM® system is immediately legible as additional reinforcement (wall brackets), however the vertical appeals also contribute to the resistant mechanism in hindering sliding along the breaking surface

The proposal in this work regards an additional term to the usual shear resistance formula:

$$V_t = l' \cdot t \cdot f_{v,d}$$

Which can therefore only be applied to the types of masonry for which the resistant parameter $f_{v,o}$ is reported, where:

$$V_t = l' \cdot t \cdot \left(\frac{f_{v0}}{\gamma_{M*FC}} + 0.4\sigma_n\right)$$

$$V_{t,lim} = l \cdot t \cdot \frac{f_{btd}}{2.3} \cdot \sqrt{1 + \frac{\sigma_0}{f_{btd}}}$$

Or, according to EC6:

$$V_{t,lim} = l \cdot t \cdot \frac{\frac{0,1}{FC \cdot \gamma_m} \cdot \left(\frac{f_k}{0,45 * 2,5^{0,3}}\right)^{1/0,7}}{2.3} \cdot \sqrt{1 + \frac{\sigma_0}{\frac{0,1}{FC \cdot \gamma_m} \cdot \left(\frac{f_k}{0,45 * 2,5^{0,3}}\right)^{1/0,7}}}$$

The additional term considers the shear resistance increase due to post tensioning of the stripes, without considering the fracture of masonry blocks, which can considered secondary respect to the limits to deformation induced by the prestressed steel strips.

According to this relationship, the proposed additional term is :

$$V_{t,\, liminc} = 2 \cdot n \cdot A_{nas} \cdot f_{yd,nas} \cdot \cos\beta + 2 \cdot n \cdot A_{nas,d} \cdot f_{yd,nas}$$

For each portion of confined masonry, where:
A_{nas} is the area section of each steel strip in horizontal / vertical direction,
$A_{nas,d}$ is the area section of each steel strip in shear direction
$f_{yd,nas}$ is the steel calculus axial limit stress
β is the angle of shear force inside the masonry
So the total resistance is:

$$V_t = V_{lim} + V_{t,lim\ inc}$$

By this, the increase in shear resistance of confined masonry can consider not only usual terms, but also the increment in shear resistance due, in every case, to the presence of the steel strips.

6 APPLICATION OF THE C.A.M.® SYSTEM TO A MASONRY BUILDING DAMAGED BY EARTHQUAKE

Several applications both of the C.A.M.® system have been applied to many masonry buildings damaged by earthquake (in particular 2009 L'Aquila earthquake).

In particular, a noticeable application has been performed to the ancient S. Antonio masonry complex in L'Aquila (dating about 1100 a.C.) (including the ancient Romanic church of S. Antonio).
The complex structure consisting of different walls including a church, built starting from the year 1000 and aggregated in various ways, suffered considerable damage during the seismic event of 2009 which highlighted deficiencies and alterations that took place over centuries.
The application of the modified formula permitted to obtain a noticeable increase in shear resistance of the masonry panel.
The structures are mainly in masonry with additions in more recent times that can be read from the insertion of portions in AC leaning against the main factory.
The following picture denoted strong structural criticalities which, given the valuable historical architectural asset, had to be remedied in the most targeted way possible without invalidating the original 'construction lexicon'.
The CAM® mesh to sew the building together, limiting the local collapse mechanisms and giving an increase in strength and ductility to the masonry without altering mass and stiffness, together with the further interventions of reinforcement of the vaults and reconstruction of the horizontals, allowed the achievement of the required target of repair and seismic improvement.
The flexible System is well suited to the constraints of a protected building, allowing valuable elements (frames, upholstery, coats of arms, etc.) to be left visible without failing to meet the requirements of a more strictly structural nature.
This kind of retrofitting by C.A.M.® system permitted the building to afford new central Italy earthquakes (till M_w 6.5, 2016 Norcia earthquake) without any damage.

Figure 4 – S. Antonio church and masonry buildings aggregate

Figure 5 – A CAM® application detail joining masonry with story and roof

7 CONCLUSION

A system of structural consolidation based on the creation of a three-dimensional grid, made with tension tie-rods steel strips, which confines limited portions of masonry, is analyzed, and an analytical model is examined, with the proposal of a new formulation which take into account for a shear steel resistance term. The formation of a three-dimensional grid allows to obtain the condition of triaxial compulsion, capable of giving greater compressive strength to the masonry volume, due to the effect of induced confinement, and at the same time the bindings constitute reinforcement and offer load-bearing capacity in traction and without increasing mass. This system is the first to have introduced and engineered three-dimensional consolidation by mechanical action. Several masonry mechanisms involve the behaviour of the consolidated masonry, which are all examined in the paper.

A noticeable application to a historical masonry building aggregate, dating more than nine centuries, is illustrated, demonstrating that considering this term improves the shear behaviour of the whole masonry, also when masonry blocks are cracked by compressive or shear forces.

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