Structural Analysis of Historical Constructions, New Delhi 2006
P.B. Lourenço, P. Roca, C. Modena, S. Agrawal (Eds.)

Seismic Rehabilitation of Cultural Heritage Through Timber Slabs and Ties

G. Cuomo, A. De Luca, and E. Mele
University of Naples "Federico II", Department of Structural Analysis and Design, Naples, Italy

ABSTRACT: In this paper non linear finite element analyses of some Italian masonry mills are performed in order to evaluate their seismic behaviour. The mill structures, located in the Cilento National Park (Italy), have been analyzed under vertical and lateral seismic load. The analyses are carried out with the purpose of studying traditional or innovative seismic protection techniques to be adopted in the retrofit project of mills, respecting the characters of the existing fabrics. From the deformed configuration of the mill structures under seismic loads, the potential for local collapse is assessed and indications for retrofit measures are derived. In particular, it has been noted that out of plane mechanisms of the single walls are the most likely collapse mode; one of the retrofit techniques which is able to strongly reduce this specific vulnerability is the insertion of a rigid slab, preferably in wood, to be realized completely without mortars and with traditional materials. This technique is almost entirely reversing.

1 INTRODUCTION AND OBJECTIVES

Mill structures in southern of Italy are considered as cultural heritage because of their large diffusion (around 300 of them are catalogued), mainly in the Cilento National Park; for this reason they are chosen as object of study in the research project of CRdC BENECON (Regional Centre of Competence on the Cultural Heritage, Ecology and Economy). The CRdC Benecon is a multidisciplinary centre that was created for the Cilento National Park eco-museum realization. Among the activities of the Benecon, the structural analysis of masonry mills located in the Park are contemplated. The project includes the rearrangement of the landscape, the restoration and the retrofit of the mill structures built in limestone or in sandstone with rubble bond pattern.

In order to preserve these buildings, both traditional and innovative retrofit techniques are considered and compared each other in the project; for this scope, non linear two and three-dimensional analyses, under vertical and lateral seismic load, are performed and reported in this paper.

The two-dimensional non linear analyses are carried out for evaluating the in plane capacity of the single masonry walls. Furthermore, three dimensional analyses of the mill structures are carried out for evaluating both the seismic capacity of the existing structures, the potential for local collapse modes and the effectiveness of different retrofit techniques (from simple connection of the walls in two directions, i.e. with wood beams, to the insertion of rigid timber floors).

The Abaqus FEM computer code is used for the non linear analyses and the smeared crack concrete model is adopted for simulating the failure of masonry.

Moreover, in this paper, an innovative dry retrofit technique, without mortar is proposed with the employment of timber floors and ties. This technique is almost entirely reversing.

2 THE STUDY CASES: THE MILLS LOCATED IN THE CILENTO NATIONAL PARK

The structures considered in this paper are masonry mills built in the XIX° century and located in the Cilento National Park, precisely in Ottati and in Perdifumo. These mills are assembled in rows and, in the context of the single row, they show homogeneous mechanical and geometrical characteristics, thus the results and conclusions derived for a specific case study can be extended to other significant cases. In particular, two mills, the so called “S. Angelo’s mill” located in Ottati and the “Mill B” located in Perdifumo, have been selected as representative of the building sample in the two different rows. The two mills differ both for the geometrical characteristics and materials; the first is built in rubblework limestone masonry and the second in rubblework sandstone masonry. The mortar, in both cases, is made of lime, sand and water.

2.1 Geometrical and mechanical properties

The “Mill B”, located in Perdifumo (Figure 1), has a square plan 6.05m × 6.60m, two story 2m high, masonry wall thickness equal to 0.65m at the first floor and 0.45m at the second floor. The structure has a pitch roof.

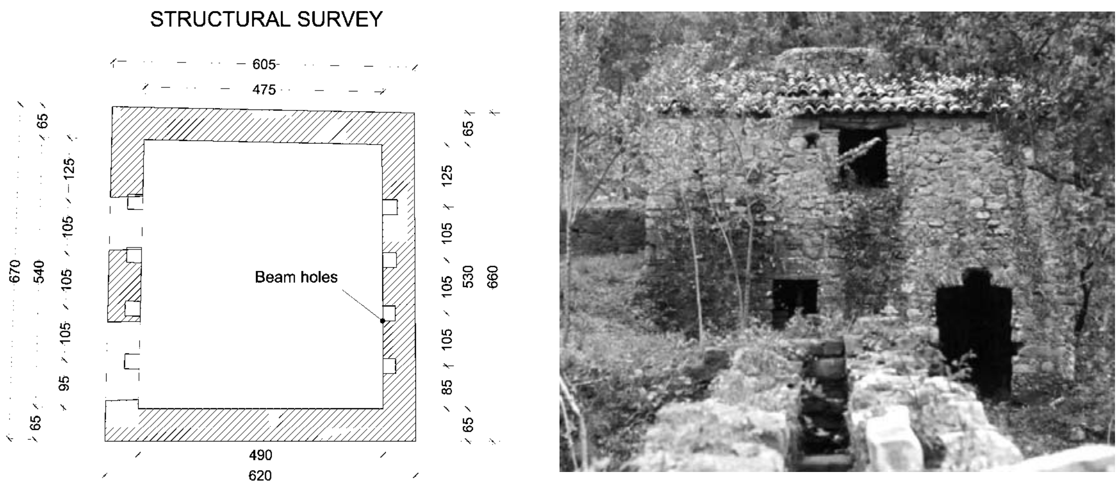


Figure 1 : The mill masonry structure namely “Mill B” located in Perdifumo.

The “S. Angelo’s mill”, located in Ottati, has a rectangular plan 12.85m × 3.70m, one story of height around 4m, masonry wall thickness from 0.80m to 1.30m. The pitch roof is not present.

Masonry mechanical properties are reported in Table 1: it is assumed 17 KN/m^3 for the self weight of the masonry, 1100 MPa for the Young’s modulus, 0.1 for the Poisson’s ratio and 2.0 MPa for the compression strength.

During the survey operations it was noted that both mills showed structural degradations, due to erosion of the mortar joints, biological attacks, presence of vegetation, loss of stone elements and diffused structural damage, in particular near the openings.

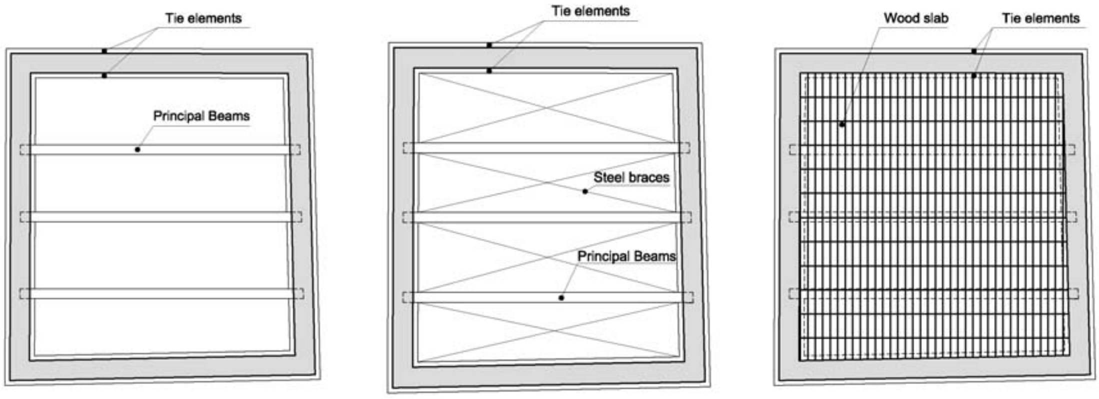
3 NUMERICAL ANALYSES AND MODELLING

For the masonry mills analyzed in this paper, two-dimensional and three-dimensional non linear finite element analyses are preformed. The two-dimensional analyses are carried out in order to assess the sensitivity of the results to the model parameters, in particular to the tensile strength, and to compute the collapse multiplier of lateral load distribution acting on the single wall.

Non linear three dimensional analyses of the mill structures are easily performed because of their small dimensions. The analyses are able to account for the out of plane behaviour of the single walls and to evaluate the effectiveness of different retrofit techniques to be adopted in the rehabilitation project.

It is also important to notice that local and out of plane collapse modes can only be derived by 3D complex analysis methods, like finite elements methods, while the simple and conventional check methods are generally not enough accurate for this scope.

Five different FE models, representing both the "as is" and four different retrofit techniques, are implemented for each mill structure and non linear analyses are performed. The analysis results are then compared in terms of collapse multiplier and local damage. The five analyzed models are: the "as is" model, representative of the structure in the original conditions; the second model, with wooden beams connected to masonry walls (Figure 2a); the third model, with bracing reinforcement of the wooden floor beams (Figure 2b); the fourth model, with two way floors, with timber deck made of wooden planks, connected to the masonry walls (Figure 2c); the fifth model, with a rigid slab, for example a reinforced concrete slab. For all models, the masonry material properties are reported in detail in the next section. In the second, third and fourth models, for the wood and steel materials an elastic behaviour is assumed, with elastic modulus respectively equal to 10000 MPa (hardwood D30 of EN338/1995 European code) for the wood and 210000 MPa for the steel.



(a) (b) (c)

Figure 2 : The different retrofit techniques: (a) connection of the wooden beams to the masonries; (b) bracing reinforcement of the timber slab; (c) insertion of a timber double crossed slab.

3.1 Masonry constitutive law, element type and discretization

Masonry is an almost brittle material and has a different behaviour in compression and tension as shown in Figure 3; a smeared crack model, as implemented in the computer FE code Abaqus (Hibbit et al., 1997), has been used to account for cracking (De Luca et al., 2002). The failure model for masonry, as implemented in Abaqus, basically requires the definition of: (i) the shape of the failure surface via the 'failure ratios' option; and (ii) the post-cracking tensile behaviour, defined by the 'tension stiffening' option. The failure surface is specified through four "failure ratios": "Ratio 1", ratio of the ultimate biaxial compressive stress to the ultimate uniaxial compressive stress; "Ratio 2", absolute value of the ratio of the uniaxial tensile stress at failure to the ultimate uniaxial compressive stress; "Ratio 3", ratio of the magnitude of a principal component of plastic strain at ultimate stress in biaxial compression to the plastic strain at ultimate stress in uniaxial compression; "Ratio 4", ratio of the tensile principal stress at cracking, in plane stress, when the other principal stress is at the ultimate compressive value, to the tensile cracking stress under uniaxial tension. Consequently, a criterion for failure of masonry due to a multiaxial stress state is defined. The failure surface for masonry is shown in Figure 3. The Table 1 reports the values selected for the model.

In the 2D FE models, the four-node shell elements, CPE4, are used, while the eight-node solid elements, C3D8, are used in the 3D FE models.

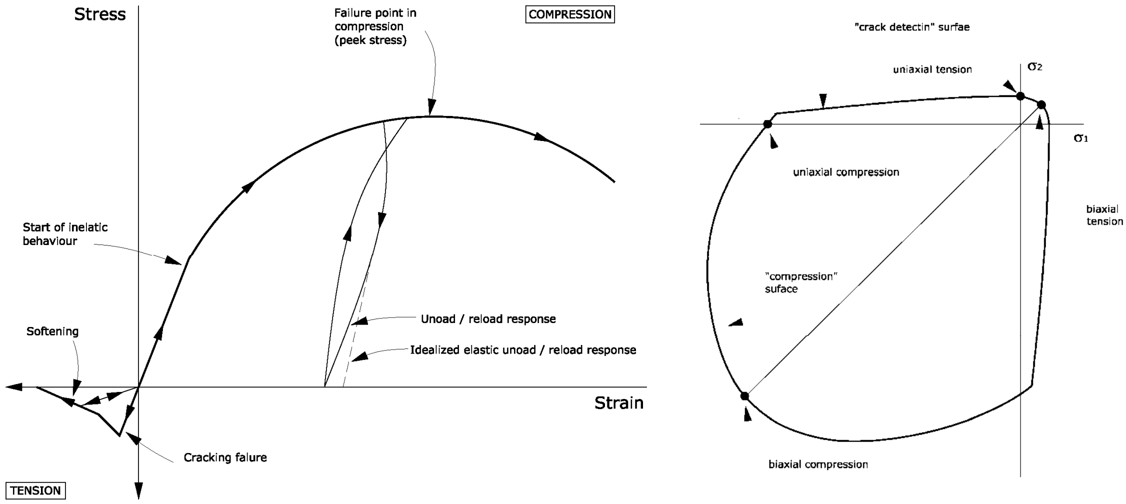


Figure 3 : A stress-strain curve and two-dimensional failure surface for masonry.

Table 1 : Material parameter values for the ABAQUS model.

	Limestone and sandstone
Weight per unit volume (daN/m^3)	1700
Young's modulus (GPa)	1.1
Poisson's ratio	0.1
Compression strength (MPa)	2.0
Tensile strength (MPa)	0.03 – 0.04 – 0.05
Failure ratios	1.16 – 0.015 , 0.02 , 0.025 - 1.33 – 0.3

4 ANALYSIS RESULTS

Two and three dimensional analyses are reported in this paragraph. Since masonry shows a small tensile strength, in particular for the stonework composed by rubble stone and degraded mortar, typical of the mill structures under study, a sensitivity analysis has been carried out by varying the tensile strength parameter.

4.1 Non linear 2D analyses

In this section two dimensional analyses of single walls, extracted from the two study cases (De Luca et al., 2003), are described. To test the sensitivity of the model to the variation of the tensile strength, different values of the Ratio2 have been adopted. In particular, values equal to 0.015 - 0.025, which corresponds to tensile strength respectively equal to 0.03 - 0.05 MPa, have been assumed in the analyses. Some analysis results are reported in Figure 4, with reference to the mill located in Ottati. In this chart the lateral load on the single wall (normalized to the weight) vs. horizontal displacement is reported; the various curves represent the static structural response with different values of tensile material strength. It can be noted that the sensitivity of the model to the tensile strength parameter is almost negligible. Therefore on the basis of these results and from similar results obtained on other masonry walls, three dimensional analyses are carried out with a tensile strength equal to 0.03 MPa, which corresponds to the best balance between convergence and accuracy.

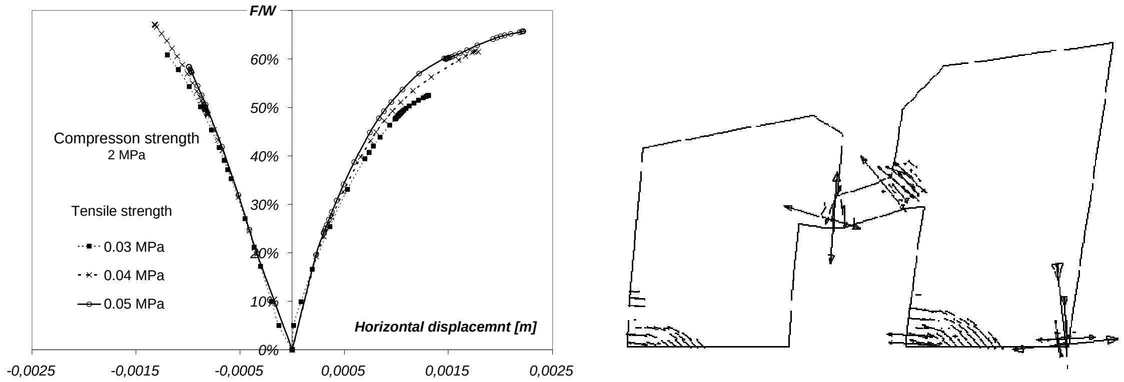
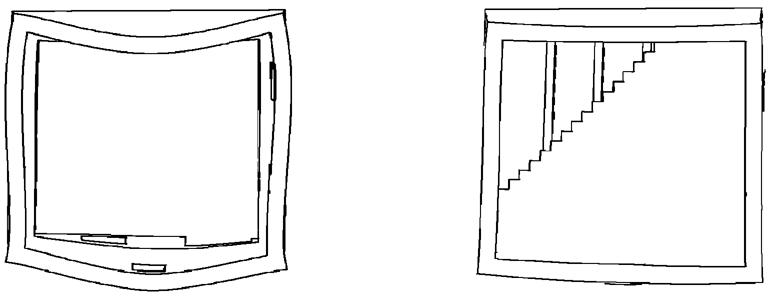


Figure 4 : 2D sensitivity analysis and plastic strains in the wall 1 of "S. Angelo mill" (Ottati).

4.2 Non linear 3D analyses

The results of the analyses conducted on the two mills show a similar trend; in addition the structural responses under horizontal load acting along the two principal directions are similar, therefore the following discussion is focused to a single mill, the "Mill B" located in Perdifumo, under loads acting along the direction 1. The Figure 5 shows the deformed shape of the mill, both in the "as is" configuration and in the timber slab retrofit configuration; from the former (Figure 5a), building out of plane vulnerabilities can be observed; the major consequence of this deformation mode is a damage and crack patterns that have also been noted during the structural survey phase. The out of plane deformations and the consequent damage are remarkably reduced in the retrofit configuration (Figure 5b).



(a) as is (b) timber slab

Figure 5 : Comparison between the deformed shape of the "as is" mill and the mill with a timber slab.

In the Figure 6 the results of the nonlinear static analysis, carried out on the FE models representing the different retrofit interventions, are reported in terms of lateral load (normalized to the weight) vs. horizontal displacement. The first consideration on these results is that the restoration through the simple connection of the wooden floor beams to the masonry walls, is not sufficient to eliminate the local and out of plane collapse modes, and the collapse multiplier of the structure does not significantly increase.

The second and most important consideration is that, with the insertion of a timber deck well connected to the mill masonry walls, improvements comparable to the insertion of a rigid reinforced concrete slab can be obtained; furthermore, local and out of plane collapse modes are eliminated and the collapse multiplier significantly increases.

It is important to notice that in order to achieve benefits similar to the insertion of timber slab by adopting the bracing reinforcement retrofit technique, it is necessary to employ steel braces with very large cross section area.

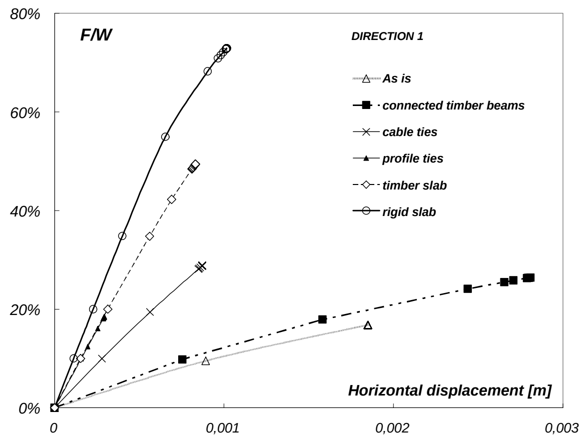


Figure 6 : Collapse multiplier for the different retrofit techniques models of the “Mill B”.

5 RETROFIT OF MILL MASONRY STRUCTURES

Based on the information derived from the structural survey and on the results derived from the finite element analyses, a retrofit intervention to be applied to all mills of the Cilento National Park has been studied in details.

The designed intervention consist in: reconstruction and reinforcement of the timber slabs; wedging and restoration of the cracks; substitution, if necessary, of the masonry near the big cracks; restoration of the stone flat arches; substitution of the timber architraves; and restoration of the roof. The reconstruction and reinforcement of the timber floors is analyzed in details and is described below.

The designed timber floors can be one-way or two-way floor (only with primary beams or with primary and secondary beams), with a single or double timber deck. The doubling of the deck is necessary to assure a high stiffness in the plan, thus inducing a box-type structural behaviour and increasing the seismic resistance. The major problem in the insertion of the slab in the mill structures, as in all masonry structures, is the efficient connection to the existing walls. The designed timber slab is connected to the structure through a mechanical dry connection without the use of mortar. This technique is studied to be structurally effective (since the mechanical connection assures the perfect collaboration between the new and the existing structures) and entirely reversing. In the Figure 7 is reported the intervention layout.

The mechanical connection between the slab and the masonry is realized, at the two facades of the walls, by means of two steel profiles (architrave/chains) connected each other by tie bolts at every first meter. The steel elements are completely external to the stonework, thus no interruption in the continuity of the masonry is necessary for the realization of intervention.

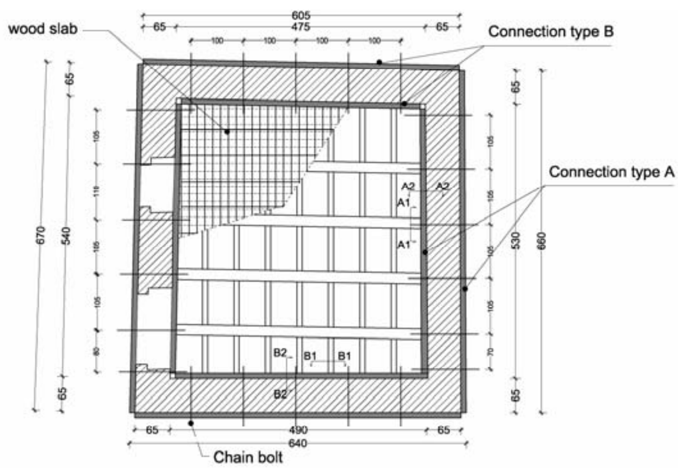


Figure 7 : Slab restoration layout.

Different solutions for the timber floor and for the connection between floor and walls have been studied and are reported in the Figure 8, 9 and 10. In particular in the Figure 8 solution with two-way floor (with primary and secondary floor beams) and double timber deck (made of wooden planks) is shown; in Figure 9 and Figure 10, the solution with one-way floor (absence of secondary beams) and double timber deck, and the solution with two-way floor and single timber deck, are respectively shown. The connection of the principal floor beams to the wall it is very simple, but the connection between secondary floor beams (or deck) and wall are more complex, requiring the use of additional timber and steel elements.

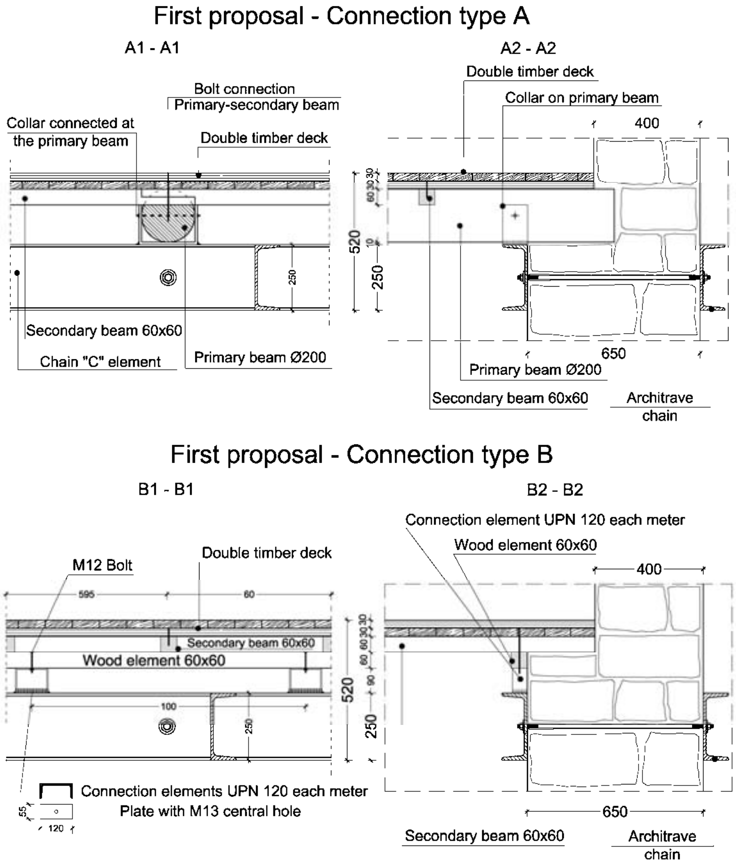


Figure 8 : First connection solution – Two way floor with double timber deck.

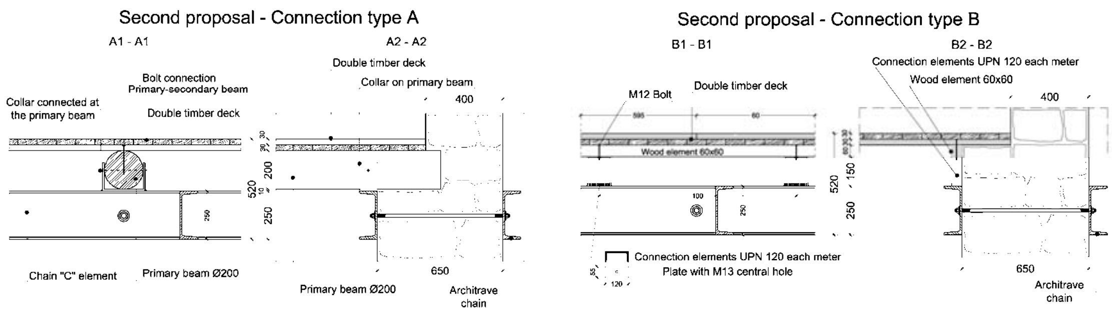


Figure 9 : Second connection solution – One way floor with double timber deck.

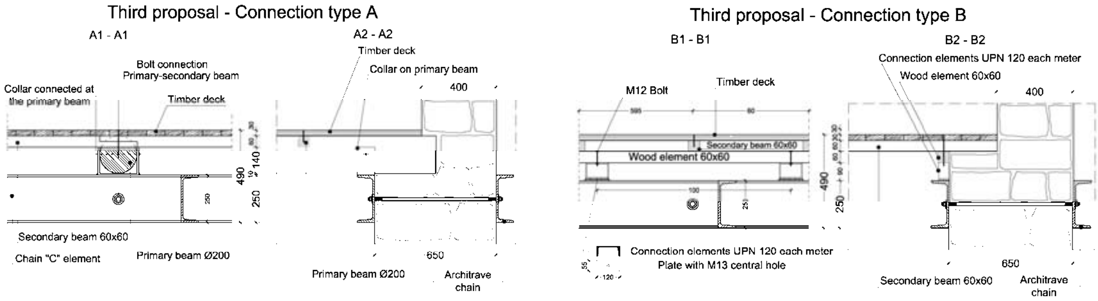


Figure 10 : Third connection solution – Two way floor with single timber deck.

6 CONCLUSIONS

In this paper, non linear finite element analyses on two masonry mill structures are performed in order to evaluate the seismic behaviour, potentials for local collapses and indications for retrofit measures. It is worthy noting that potential for local and out of plane collapse modes is detectable only through refined analysis methods, like 3D FE analysis, thus the simple conventional check methods are generally not able to grasp the real behaviour. For this purpose Abaqus FEM computer code is used, proving to be a powerful tool. The deformed configuration and the consequent potentials for local collapse modes of the mill structures under seismic loads suggest indications for local retrofit measures.

The effectiveness of different rehabilitation techniques applicable to the masonry mill structures has been investigated, and two considerations can get: first, the restoration with the simple connection of the floor wooden beams to the masonry walls is not sufficient to eliminate the local and out of plane collapses, and does not significantly increase the collapse multiplier of the structure; the second and most important consideration is that with the insertion of a double timber deck, well connected to the mill masonry walls, the same benefits produced by the insertion of a rigid reinforced concrete slab can be achieved, with the elimination of the local and out of plane collapses and a significant increase of the collapse multiplier.

In the last part of the paper we have shown the designed interventions for the restoration of the masonry mill structures, in particular the reconstruction and reinforcement of the timber floors to be realized completely without mortars and with traditional materials. Different solutions for the practical realization of the retrofit are described. The proposed techniques are almost entirely reversing and the use of mechanical connections assures the effective stress transfer between the new and old elements.

ACKNOWLEDGEMENTS

This research has been developed in the context of the activities of the CRdC BENECON (Regional Centre of Competence on the Cultural Heritage, Ecology and Economy).

REFERENCES

Croci, G., The Conservation and Structural Restoration of Architectural Heritage, *Computational Mechanics Publications*, 1998.

EN 338. Structural timber. Strength classes. *European Committee for Standardization*, 1995.

Giordano A, Mele E, De Luca A. Modelling of historical masonry structures: comparison of different approaches through a case study. *Engineering Structures* 2002;24(8):1057–69.

Hibbit, Karlson and Sorensen. ABAQUS theory manual, *Abaqus*1997.

Mele E, De Luca A, Giordano A. Modeling and analysis of a basilica under seismic loading. *Journal of Cultural Heritage* 2003;4(4):355–67.