Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Modelling of repair and retrofit of historical buildings in the Balkan region

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ABSTRACT : Taking into account the high seismic risk of the Balkan region as well as the artistic and historical values of various masonry structures, attempts have been made to establish and elaborate efficient methods for their repair and retroffiting. The idea of the retroffiting proposals is to suggest adequate, simple, cheap and nondestructive methods of strengthening, following the existing system of horizontal wooden ties along the facade walls of the structures. For this purpose simple models with embedded slightly prestressed cables in the already existing holes were analysed. The finite element analysis of a separate wall of the selected structure confirms the positive effect on the dynamic performance of the wall element during a strong seismic excitation. More sophistacated analysis as well as experimental tests are needed in order to establish precise repair and strengthening methodology for these structures. The obtained results can be extrapolated on various types of historical masonry structures where similar structure elements are encountered.

1. INTRODUCTION

The Balkan region is an area extremly rich and abundant in cultural monuments and histrorical buildings dating from various periods, starting from Roman ramparts and amphitheaters, through Byzantine churches up to more recent traditional dwelling houses. Taking into account the high seismic risk of the region as well as the artistic and historic values of these structures attempts have been made to elaborate efficient methods for their fitting and retrofitting.

In order to define an appropriate method for restoration and strengthening it is necessary to define the resistance capacity of the selected structures in original state, especially under the effect of horizontal dynamic loading.

Among different types of historical structures a representative groupe of four Byzantine churches in the region of Macedonia was chosen. These monuments represent one of the most typical and valuable type of historical structures in the Balkan region and in other regions belonging to the ancient Byzantine impire. The selected structures date from the period of the late fourteenth century, the so called "age of decadence ", when the architectural forms and the famous fresco painting developed its full richness in proportions, colours and style.

From the constructional point of view the structures consist of massive facade bearing walls followed by system of columns and arches placed in both orthogonal directions. (fig.1 and fig.2) The masonry is in most of the cases of very good quality, consisting mostly of lime stone in deep layers of strong lime mortar and layers of bricks used for decoration.

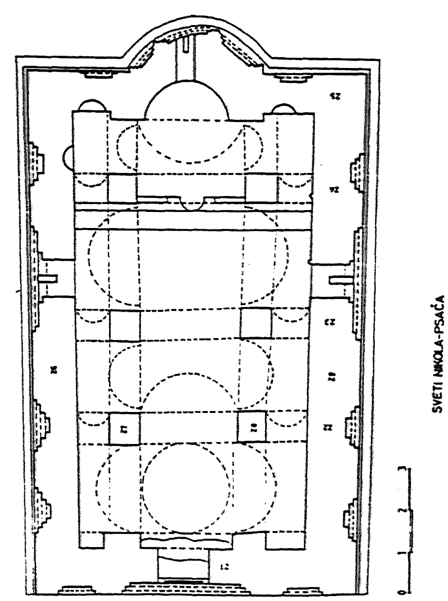

Figure 1. Plan

The performed analysis of the selected buildings as well as the obtained and recorded behaviour during past earthquakes shows a remarkable strength capacity of the structural system. This capacity can be associated to the very large wall sections, to the high resistance of the vaulted elements -arches and vaults- under seismic loading, and especially to the traditional aseismic elements, such as horizontal wooden ties and

Figure 2. Elevation

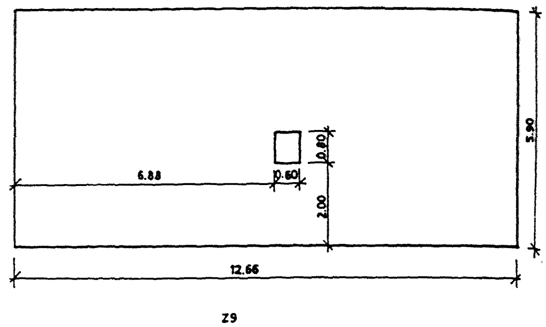

Figure 3. Typical facade wall element

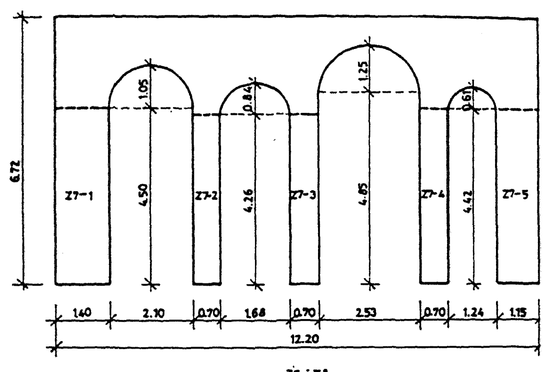

Figure 4. Typical element consisted of columns and arches

timbers, placed along the facade walls, connecting the individual structural elements - the walls - and thus contributing to the overall stiffness of the structure as a whole. Yet, due to the extensive weathering, soil sliding and settlement and the very high seismic risk, interventions in the stuctural system seem often to be necessary and unevitable. The idea of the retroffiting proposals is to suggest adequate, simple, cheap and nondestructive methods. In the same time it is necessary to achieve several lines of defense of the structural bearing system, similar to the effect of the originaly existing ties, often decayed and nonfonctioning at present. For this purpose simple models with slightly prestressed cables embedded in the already existing or especially drilled holes were considered. Such interventions do not affect the original wall covering - fresco painting and finishes - and are not agressive for the exisiting built - in materials, compared to the other methods used in the current practice, such as: grouting, gunitting and so on.

2. MODELLING OF THE REPRESENTATIVE STRUCTURE

From the four analysed structures the analysis of the church St: Nikola in the village of Psaca will be presented (fig. 1 and fig. 2).
The structural system was analysed as a simple cantiliver beam system, fixed at the ground level with the mass concentrated at the top. The bearing system of the structure, as described above, consists of massive facade walls followed by a system of columns and arches placed in the both orthogonal directions. On figure 1 these bearing elements are marked with "zi". Typical layout of such elements is presented on figures 3 and 4. The system of wall elements can work together due to the stiff vaulted roof structure and the connecting wooden ties and belts, so all the structural elements (walls and columns) were considered as elements fixed at the bottom and at the top.

In the analysis minimal values for the ultimate tensile and compressive stresses of the wall section were assumed (ultimate tensile stress - ft = 100 kpa and ultimate compression stress - fc = 1000 kpa). With these values a onedirectional pseudodynamic -static- analysis with an ideal elastoplastic behaviour of the walls and columns was performed. Two different failure criteria while determining the ultimate shear capacity of the bearing elements were considered; ultimate shear capacity due to direct influence of shear and ultimate shear capacity due to bending. Assembling the shear capacity of all the individual elements the ultimate shear capacity of the whole structure in the both representative directions was evaluated. The analysis shows good resistance capacity of the structure. The structure is actually able to resist a seismic force equal to 30% of its weight with a safety coefficient against collapse of more than 1.5.
Further on, a dynamic time history analysis with three different acceleration records was performed. A certain "ductility" capacity was allowed for the masonry elements; a ductulity of 1.1 for the elements failing in shear and of 1.5 for the elements failing in flexion. The dynamic response of the structure is quite satisfactory. The required ductulity capacity for different earthquakes is presented on fig. 5. Two levels of peak acceleration were considered, the lower of 25% g corresponding to the "design" earthquake, and the higher of 38% g corresponding to the

"maximum" expected earthquake. The required ductility, even for the maximum expected acceleration, is a bit higher or almost equal to the assessed global ductility capacity, taken as 1.35.

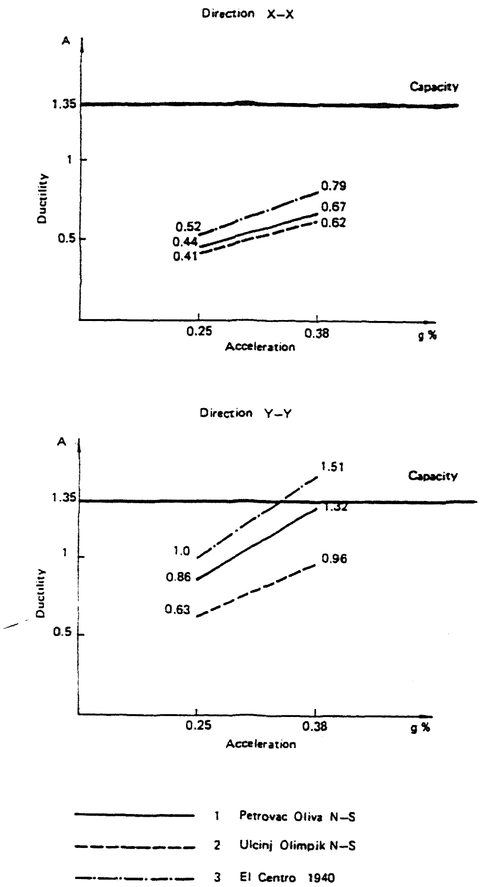

Figure 5. Required ductility

Nevertheless, it should be stressed that this analysis is quite rough. It does not take into account the actual geometry of the wall element, the configuration of the openings, the varation of the tickness and, most of all, the influence of the wooden ties and interconnecting timber elements.
In order to have a better idea of the actual stress - strain distribution at limit state for the bearing system of the structure a more sophisticated finite element analysis of a separate wall element was performed.

3. FINITE ELEMENT ANALYSIS OF A SEPARATE WALL ELEMENT

For this purpose, from the four facade walls (fig.2) of the structure, the west facade wall was chosen. It has one opening which is actually the principal entrance of the church. The layout of the wall is presented on fig. 6. On this element a plane elastic analysis was performed - fig. 7. The wall was fixed at the bottom and at the top simulating the effect of the stiff vaulted roof system. On this wall a horizontal force equal to 30% of its weight with a triangular distribution was applied. This is the same amount of ststic loading that was applied to the wall in the simplified analysis described in chapter 2. According to the analysis this force induced limit state of stresses in the bottom wall section with a safety coefficient of 2.0 (actual stresses ft = 50 kpa, ultimate stress ft = 100 kpa). In the finite element analysis the same force induced a maximal stress of 80 kpa, wich is 60% higher. This difference results primarly to the local concentration of stresses at the corners and around the openings of the wall. The same difference of values counts for the obtained displacement from the two analysis. The deformed view of the wall is presented on fig. 8.

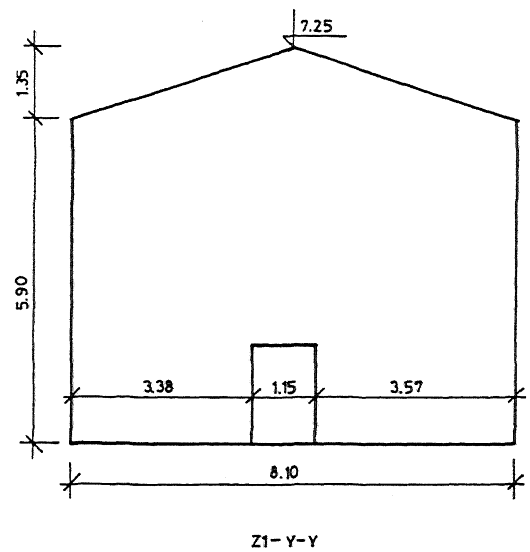

Figure 6. Layout of the selected wall

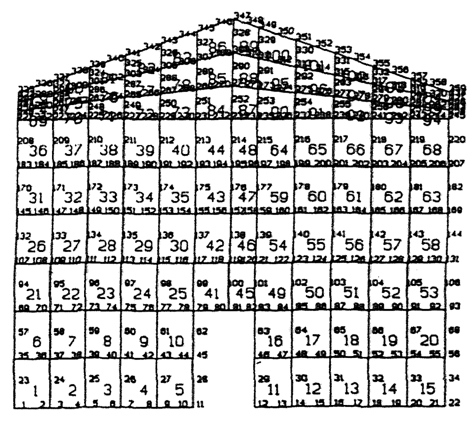
Figure 7. Finite element modelling

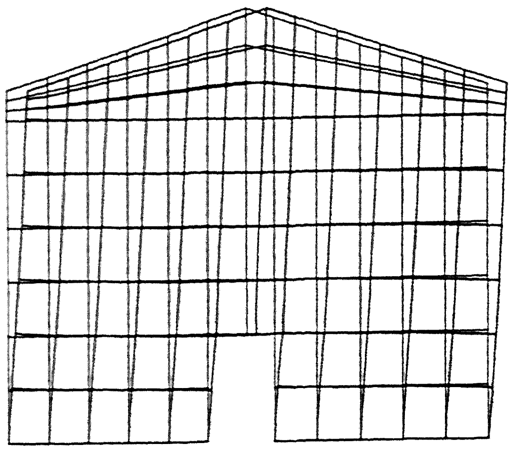

Figure 8. Deformed view of the wall

The finite element analysis was repeated with the same boundary conditions taking into account the wooden ties placed on two levels in the wall mass - fig.9. In this case an almost 40% higher lateral force induced the same level of stresses compared to the force in the case where the wooden ties were not taken under consideration. The level of the induced tension stress in the wooden ties was close to its limit values.

Finaly the wooden elements were replaced with adequate amount of reinforcing steel bars. A very similar stress - strain condition of the wall was obtained. This analysis corresponds to the previously explained idea of retroffiting.

It is clear that the simplified methodology of analysis (chapter 2) underestimates the ultimate shear capacity of the wall. Taking into account the effect of the existing timber belts a higher resistance capacity of the structure can be determined, corresponding much better to the real behaviour of the structure.

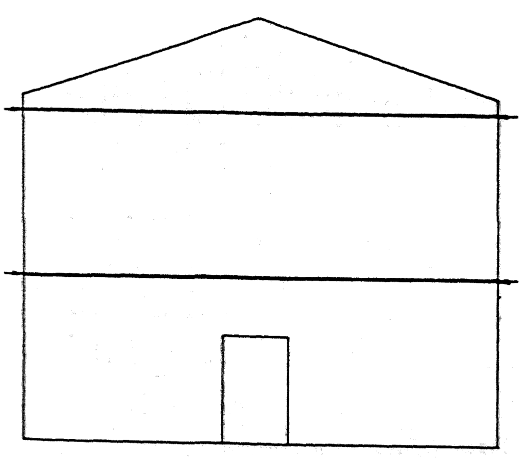

Figure 9. Disposition of the timber belts or embedded steel ties

4. CONCLUSIONS AND PROPOSALS

The idea of embedding prestressed steel ties in the already existing holes in the wall mass is actually to contribute to the synchronized behaviour of the individual wall beraing elements, thus enlarging the postelastic capacity of the structure and preventing its collapse. As for the wall elements, it should be stressed that a more preferable failure mechanism is expected to occur. Actually, at the limit state of deformations and stresses of the wall a different crack pattern following the embedded steel ties would develop, thus " ductilising" the brittle diagonal failure so typical for massive masonry elements. Raising in the same time the ultimate shear capacity of the structural system, as shown in chapter 3, a much better performance of the structural elements and the structure as a whole can be achieved.

Further analysis are nessecary to define the postelastic strength and deformation capacity, in original and in repaired state. The assembly of the structure as a whole is also to be anlysed. Varying the material properties - maximum and minimum assumed values of tension and compressive stresses -, in elastic and in postelastic state, more sophisticated elastic as well as nonlinear models can be developed.

Experimental analysis are nevertheless essential to verify the real behaviour of these structures. So far, very little references on this topic can be found. Taking into account the small size of the buildings, dynamic tests of walls and even whole structures are easily feasable. Such research work would help to understand and verify the real behaviour of these structures under earthquake loading comprising the possible contribution to the development of strengthening and restauration measures.

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