THE BISHOP PALACE IN DUBROVNIK: STRUCTURAL ANALYSES AND PROPOSAL OF REMEDIAL MEASURES

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SUMMARY

Frequent interventions have been practised in the past on the Bishop Palace of Dubrovnik, often required to repair damages produced by earthquakes. The new project of structural strengthening has been elaborated by G. Croci Consultant Engineering in cooperation with the University of Zagreb; the project was carried out according to UNESCO guidelines: a special care was taken to ensure the compatibility and reversibility of the designed interventions.

1. SOME HISTORICAL NOTES

The Bishop Palace, built it in 1350 by the Sorkocevic family, lies inside the old town of Dubrovnik, in front of the Cathedral. The original aspect of the Palace was modified in the 16th century, and largest modifications occurred after the damage produced by the earthquakes of 1667 (a new main façade was built in front of the Cathedral, the internal arrangement was changed, French windows with stone balustrade in the second storey of the North façade were built, and were then walled up in the 18th century). In 1979, a strong earthquake made the building uninhabitable.

2. THE PRESENT CONDITION OF THE BUILDING

Bearing walls are “sack masonry”: external layers are made of well dressed stone with lime mortar, while the filling is of irregular crushed stone pieces. Tests on the masonry gave quite

Fig. 1 : North Façade

Fig. 2 : A cornice under the balustrade

good values of resistance, but it was also noticed that the heterogeneity of the masonry due to frequent modifications, and diffuse damage caused by the seismic action, contributed to weaken the structure in large areas. Cracks are now visible near most of the doors and lintels, and on the East Façade. A particular problem was detected on the North façade (Fig. 1), were parapet walls, built in the 18th century, are not actually connected to the structure and do not co-work with the rest of the wall, weakening the façade at this level. Cracks on the cornices under the balustrade of the second storey are now visible under almost all of the windows (Fig. 2). In the preliminary project elaborated by University of Zagreb several interventions were proposed, not only to strengthen the structure, but also to permit the new activities that are going to be performed in the building and to recover its original aspect. Thus, in the preliminary project, the demolition of the parapet walls of the balustrade of the first storey of the North façade was proposed, in order to return the façade to its original 17th century aspect.

3. ANALYSIS OF THE GLOBAL STRUCTURAL BEHAVIOUR

A finite element model of the whole building was arranged in order to study the structural behaviour under dead loads and during seismic actions (Fig. 3, 4). Numerical analyses were then performed using the “SUPERSAP” software from Algor Inc.

No significant problems were detected as regards vertical static loads; whilst a special care was needed for the study of the effect of seismic actions.

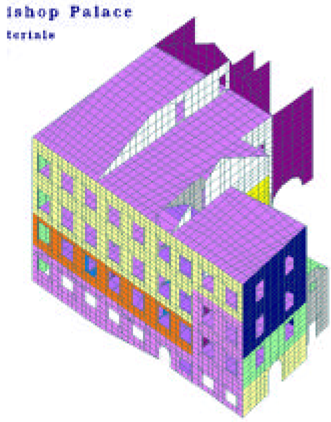

Fig. 3 : The F.E model

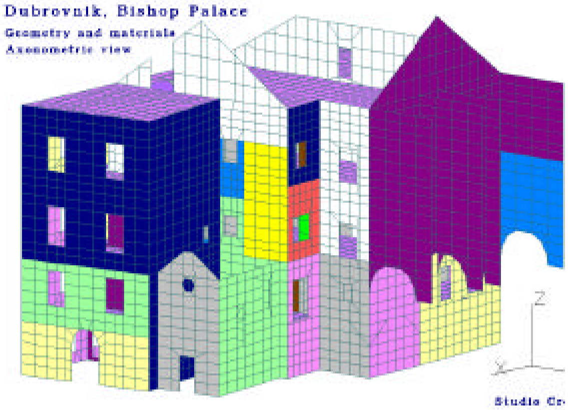

Fig. 4: The F.E model

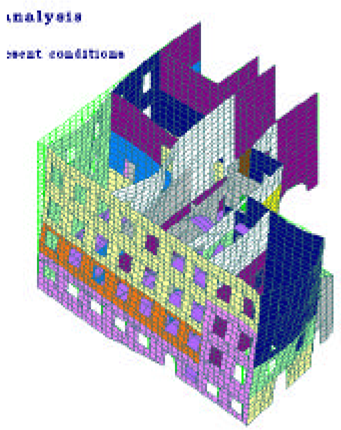

Fig. 5 : 1^{st} Mode (Present conditions)

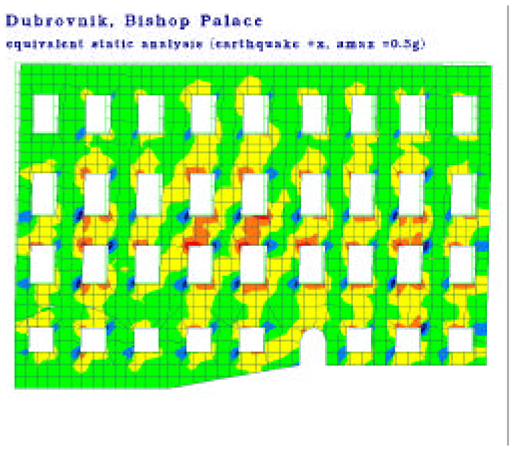

Fig. 6 : Max. principal stress on North Façade

Firstly, response spectrum analyses were performed, that showed that the first mode of the building was in the East-West direction (Fig. 5), involving the North Façade in its own plane, and producing relevant tensile stress (Fig. 6). As significant interventions had been proposed on this wall, already damaged by earthquakes, a local model of the North Façade, with an increased density of the F.E. mesh, was arranged to study this area in detail.

4. LOCAL ANALYSES OF THE NORTH FAÇADE

The aim of the local analyses on the North façade was mainly to evaluate the effect of the earthquake expected for the town of Dubrovnik, both in the present conditions and after the proposed intervention. In order to take into account the non-linear elastic structural behaviour, "step analyses" were performed thanks to a "post-processor" of the SUPERSAP, allowing to individuate the elements out of a fixed three-dimensional resistance domain.

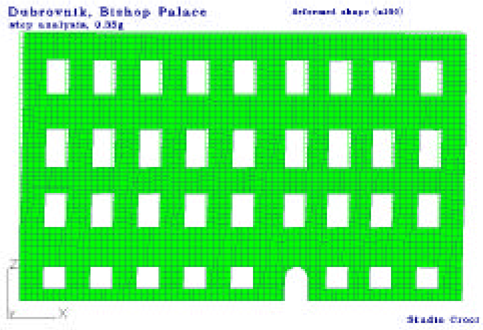

Fig. 7

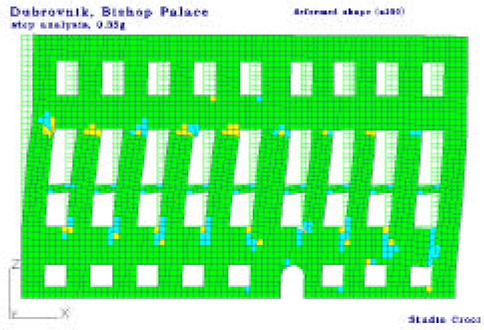

Fig. 8

It should be taken into account that, for the same action, the response of the complete model (100%) was higher than that of the local model (85%). This difference can be explained considering that in the global model the inertia forces due to the whole structure are taken into account, and torsional effects due to the eccentricity of the barycentre of the masses respect to the barycentre of the stiffness are considered. In other words, an acceleration of 1g applied on a local plane model corresponds to an acceleration of 0.85g on the whole model.

A first local model was performed supposing that the parapet walls of the second storey were actually co-working with the rest of the structure, as it could be obtained strengthening the zone behind the balustrade of the French windows. The results show that in this case the façade has no problems respect to horizontal actions: at the value of 0.35g, corresponding to 0.35*0.85= 0.3g on the whole structure, the façade is still in the elastic range, and no elements are broken (fig. 7).

The second model was obtained eliminating the elements corresponding to the parapet walls of the North façade, according with the intervention prescribed in the preliminary project.

The analyses show that the absence of co-working parapet walls at the second storey level has important consequences on the structural behaviour of the façade, that can be summarised as follows (Fig. 8):

?? The façade has a weak zone under the balustrade of the second floor.

?? A “shear type behaviour” of the portions over the ground floor is clearly visible from the deformed shape, due to the reduced stiffness of the walls between the windows of the second and third storey.

?? First cracks appear for ag=0.175g (corresponding to 0.175*0.85= 0.15.g on the whole structure), and at a_g= 0.40g (corresponding to 0.40*0.85= 0.34 g) the portion over the first floor collapses

As first cracks appear in the zone under the windows of the second storey, it is confirmed that this zone of the building is weak, as it was suggested by the survey of the damage.

At present time, with the parapet walls not completely co-working with the structure, the building is in an intermediate condition between the first and the second model.

Finally, a third model was arranged to detect the possible effects of a remedial measure to prevent the structure from possible collapses emphasised in the previous analysis.

A steel beam composed by two joint steel UPN200 was supposed to be introduced in the weak zone under the windows of the second storey, simulated in the numerical model with “beam” elements. The analysis shows that this intervention could significantly increase the resistance

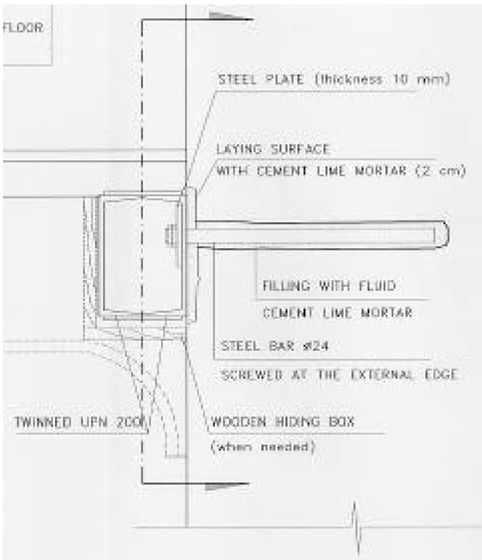

Fig. 9

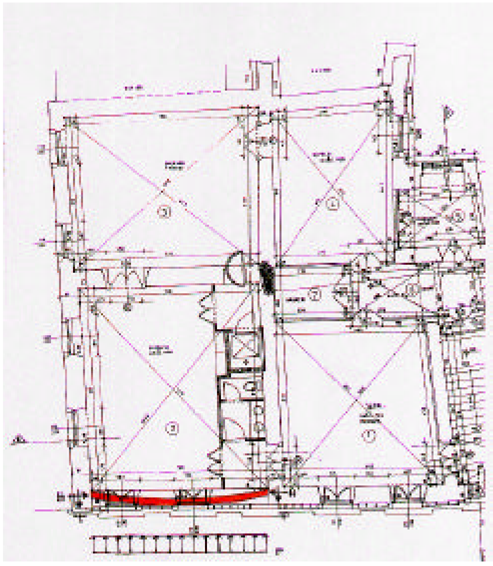

Fig. 10

of the façade respect to the situation explored in the previous analysis:

?? The first cracks appear at a_g = 0.225g instead of 0.175g (corresponding to 0.191g and 0.15 on the whole structure).

?? At the value of a_g = 0.35g (corresponding to 0.3g on the whole structure, value foreseen by the Croatian Code for the seismic analyses), the structure has still a significant margin of safety respect to collapse.

?? The ultimate load increases from 0.4g to 0.5g (corresponding to 0.34 and to 0.425 on the whole structure)

5. LOCAL INTERVENTION ON THE NORTH FAÇADE

The introduction of a steel beam composed by two twinned UPN200 was showed to be a measure effectively contrasting the weakening caused by the demolition of the parapet walls.

The effectiveness of the intervention can be judged comparing (table 1) the ratio between horizontal acceleration causing the collapse of the façade (a_{ult}) and the maximum acceleration foreseen by the Croatian Code (a_{max}). Actually, introducing the twinned UPN beams the safety increases more than three times (42/13=3.2).

Table 1 – Efficiency of the second storey reinforcements

	Maximum previewed horizontal acceleration a_{max} [g]	Collapse horizontal acceleration a_{ult} [g]	a_{ult}/a_{max}
Model pb (without parapet, without UPN200 beam)	0.3	0.34	1.13
Model pt3 (without parapet walls, with twinned UPN200 beam)	0.3	0.425	1.42

Besides, the proposed intervention seemed also compatible with the characteristics of the building, as the UPN beams can be hidden in the thickness of the floor wooden structure, that, at this level, under the second storey, often have false decorative ceilings (Fig. 9).

6. CHAINS, ROOF AND FLOORS

Apart from the solutions mentioned above, other interventions were proposed in order to improve the structural behaviour of the Bishop Palace. Among them horizontal chains were proposed at the floors and roof level, in order both to contrast the thrust developed by the arch effect produced by seismic actions perpendicular to the plane of the wall (Fig. 10), and to prevent the detachment of the façade. Horizontal chains were showed to be indispensable also for the timber structure of the roof (Fig. 11), now thrusting on the walls, due to the absence of chains at the level of the base of the inclined timber elements (Fig. 12).
Finally, several proposals for the strengthening and stiffening of the floors with horizontal steel elements were elaborated (Fig. 13), that were shown to be useful to ensure the resistance needed for the live loads that must be taken into account for the foreseen new utilisation of the Palace, and to stiffen the floor (Fig. 14) in order to improve the behaviour of the building during seismic actions. In all the proposals mentioned above a special care was taken in the choice of the technique and of the materials applied: only reversible measures were suggested, avoiding the use of concrete and trying to respect the characteristics of the building.

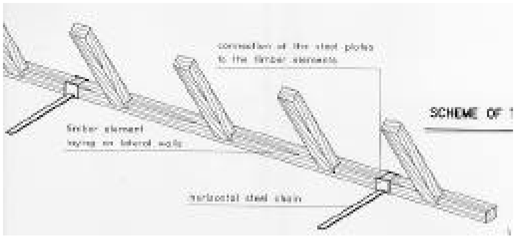

Fig. 11 : The arch effect on masonry walls

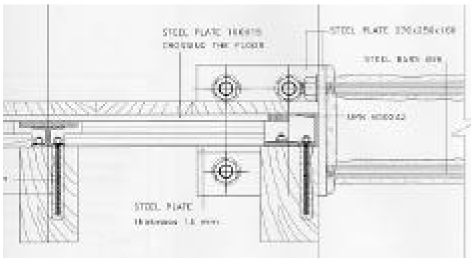

Fig. 13

Fig. 12

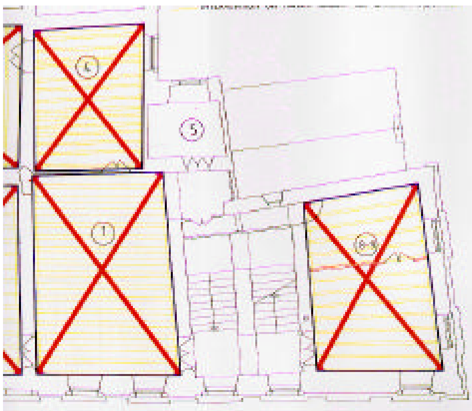

Fig. 14