1 INTRODUCTION

1.1 The reconstruction after the earthquake of 1693

The reconstruction of south-eastern Sicily after the earthquake of 1693, was one of the most complex activities consisting of architectural, urbanistic, economic and political models tested, to a large extent borrowed from the colonial experience of the crown of Spain, which had in Sicily one of its deputy kingdoms. Arising from the loss of a major part of the knowledge and skills of antiquity, there was an adoption of European architectural language in the planning of churches and palaces. Modica, Capital of the same-named County, is one of the principal centers of the valley of Noto. The church of S. Pietro, which dominates the deep incision of the...
stream, transformed rapidly in the street, according to a repeated scheme in the reconstructed cities, introducing a plain prospectus, similar to the prospectus of the cathedral of Noto. The inside basilica recall the models of the Vatican churches. They are notable for their semblance to models of German culture, of which "Gagliardi", the architect of numerous post-1693 buildings and churches, had great direct experience. The whole of the work of the reconstruction constitutes, above the architecture value, an articulated cultural model, a complex and deeply innovative heritage.

1.2 the bell tower

The Bell Tower of the Church of S.Pietro, has a structural behaviour different from the principal part of the main body of the church. The body of the tower is constituted of double walls filled with rubble, these walls being constructed of white-yellowish calcarenite. The filling is constituted of calcareous blocks in a sandy or sand-lime-matrix. The foundations and the bell cell are built in yellowish white calcarenite. A frame divides the body of the tower in two compartments. An order of butresses (doubled at the angles) frames the bell cell.

1.3 Campaign of investigations

A series of experimental tests particularly regarding the inte building complex, the determination of the principal mechanical characteristics, the level of maintenance of the supporting walls, was performed. As far as the investigations in situ are concerned, these tests included measurements of tension and elasticity of masonry, using the flat jack, endoscopic

Figure 2: Section – Plan – Frontal view
reliefs, core borings and other non destructive tests. Once appraised, the characteristic parameters of the degradation, were appraised it was possible to calculate the tension of the whole structure subject to the varied conditions of load, by use F.E.M. ending with a three-dimensional model, able to simulate the global behaviour of the tower. The results of analyses have allowed us to appraise globally and locally the safety level and static reliability of the structure towards the loads in compliance with current legislation, and to define case by case the possible necessary intervention in terms of reinforcement, consolidation to confer on the structures the resources of load-bearing required by the consolidation project.

2 MODEL OF FINITE ELEMENTS

To simulate the mechanical behaviour of the carrying structures of the building, a special three-dimensional mathematical model of the finite elements was developed. The model was used particularly for the study of static behaviour in all conditions of meaningful load and for the study of dynamic behaviour under the effect of seismic action. The model of finite elements allowed investigation of tension and deformation in every point of the modelled structures. A numerical model of finite elements of the whole body of construction using three-dimensional isoparametrics of three and four nodes type “shell”, was developed. All the supporting structures of the construction were drawn, including the internal and external supporting walls, the vaults on the staircase, and the vault covering the bell at the top of the tower. Calculation of forces took account of the main openings. The model is composed of 2570 shell type elements, divided in 71 groups and 2478 nodes in the three-dimensional space. Analyses were developed with the hypotheses of small deformations, small movement and constitutive bonds of the material of linear elastic type, using the calculation of the characteristics of tension subsequently with failure criterion studied on purpose for the masonry structures (material non homogeneous and not linear). An elastic linear type model for the study of a structure composed substantially from a non homogeneous material and of not-linear behaviour was therefore considered important for the static operation of the structure. From this indications of the order of greatness of tension and its distribution, whose deeper analysis, if thought necessary, must be developed with more sophisticated analysis, can be inferred. The calculations were effected using a reliability program from (AMV Study Software - Ronchi of the Legionaries (GO)).

3 CURRENT STATUS

3.1 The mechanical parameters of the masonry

The mechanical characteristics of the masonry were formulated on the basis of the results of the campaign of experimental tests with flat jack and some general information found in technical literature regarding the materials and the technologies of specific construction of the place.

Particularly a middle specific weight for all the masonry equal to \( \rho = 1800 \text{ Kg/m}^3 \) was considered. The coefficient of Poisson average was assumed equal to \( u = 0.12 \). As far as the choice of the elastic form of the masonry was concerned they were correctly interpreted and averaged the results of the flat jack campaign. In our case a value equal to: elastic Form 100.000 Kg/m2 was used. As far as the supporting vaults of the staircase was concerned, a constant thickness equal to 40 cms was considered. A middle filling equal to 40 cms of loose material having a weight of \( \rho = 1.600 \text{ Kg/m}^3 \) was also considered.

3.2 Characteristics of the ground and scheme foundations

The characteristics of the ground were inferred from the series of geognostic tests, showing the ground to have the mechanical characteristics of a heap calcarenitico of compact yellowish grey color. The terrestrial-structure interaction was simulated binding all the nodes to earth in foundation simply. I unload it maximum in foundation it is equal to 4,40 Kg/cmqs.
3.3 Load analysis

The considered loads are: the proper weight of the masonry; the proper weight of turn them of vaults of the staircase; the weight of the fillings of the staircase (middle thickness of 40cm), the proper weight of the vault covering the bell. The loads, above described, were analysed in a single condition of load.

3.4 Analysis of current status

A crack analysis in the current status of the whole building was performed. The purpose of analysis was mostly to define the stress in the varied components, the individualization of the maximum forces of traction and compression and location of the stressed zones of the structure.

The comparison between the stress pattern from this first analysis, with the crack pattern of fissures prior to intervention, allowed evaluation for a correct definition, with the purpose of underlining the current status and drawing attention to the major cracks.

3.5 Seismic dynamic analyses

Seismic dynamic analysis was developed with the spectral response method with the same model used previously in static analyses. At first research into the proper ways of vibration in meaningful number with the aim of a dynamic representation of the structure was effected, and subsequently spectral response category II was applied. Modal analysis of the structure involved the search for proper frequencies of excitement and relative modal forms in enough number to reach the percentage of mass participating in motion to the value of 85%. To reach this result it was necessary to extract from the model the first three wave types. The seismic effects were considered separately for acceleration of the ground in direction X (east-west) and Y (north-south). The effects of the three ways of vibration considered were combined according to the SRSS method, where every component of resultant stress of the combination is the result of the square root of the sum of the squares of each single component.

Figure 3: Model of current status
3.6 Results of numerical analyses

To make legible the results of the elaboration, they were represented, with graphic colour maps showing all the structural parts of great interest or external walls of the tower, that were those that underlined great structural weaknesses. A model of maximum and minimum tension of each of the four walls was created, according to the breaking point adopted on the map in which the varying colours indicate the forces, and the varying values were indicated on the adjacent key. Forces are indicated in kg/cmq. The negative values are the stress of compression, and the positive of traction. From the examination of the seismic dynamic behaviour of the structure it was noticed that the peaks of the values of increase-decrement of effort in the varied masonry were limited to small values. From the global point of view their influence on the values of the stress (also for the low level of static effort) doesn’t invalidate the behaviour of the structure, in which values of stress are reached that bring to breaking point, the materials in any case. From a local point of view, it was interesting that the absorption of the peaks of effort of traction and the prevention of fractures were also localized. The detailed examination of the stress model is of certain interest to the end of the project, because it allows us to foresee reinforcements or changes to the building in the project phase. Even if it is difficult to assign values to the resultant efforts from spectral analysis (being neutral eigen vectors) the overlap of the effects of the varied models (the results of a quadratic average) are of some interest to the project.

As far as the comparison with the crack pattern, is concerned in the greatest part of the cases it is thought that the low value of tension in terms of resultant traction from analysis, allows affirmation that the crack pattern is hardly referable to the static regime of the usual type of the construction but to construction defects.

3.7 Summary charts of Stress of current status

It was observed that the maximum values of the stress of compression were in all cases well below the limits of resistance of the masonry. The maximum value is in fact 4.3 Kg/cmq. As far
as stress of traction was concerned it was observed that in the building hangings they were entirely absent, while in the vaults they were of negligible entity. They were local peak values that are of modest and negligible value.

From the comparison between the values drawn from the tests in situ and the relative ones determined with the mathematical model, its own specific weight, it was deduced that the numerical model of the body of the building in object is sufficiently representative of its structural behaviour.

4 PROJECT

4.1 Identification of intervention types

The campaign of experimental tests in situ and the following numerical investigations led to the definition of the level I degradation of masonry and to the determination of the safety level, it was deduced that: the supporting structure was presented substantially whole. The masonry was generally in a good state of maintenance and the resistances in general were in the average of its type. It was considered that global stability problems, in the sense of external load calculated at the time of design, were not a problem, and neither in the performance required now. There are some cracks located in the masonry and on the vault that prejudice the static reliability of the structure in its complex.
4.2 Analyses of project conditions

An analysis was performed for the simulation of the project conditions, in which, without varying the applied loads and the consequent conditions of external load, "diatoni" in stone, "diwidag" steel bands and stiffening "frenelli" on the vault were inserted in the mathematical model in each frontal prospect.

The purpose of analysis was mostly to individualize the stress in the varied components, the individualization of the maximum stress of traction and compression and identification of the loaded zones of the structure, in its condition following the intervention of consolidation.

The data received via spectral analysis based on the first three ways of vibration were meaningful especially as far as the determination of local peaks of stress were concerned, in that they could provoke local cracking, but they were not able to prejudice the static global structure.

![Figure 5: Project conditions model - stress map](image-url)
4.3 Phases of consolidation performed

5 CONCLUSIONS

Once analysed the current status of the tower, its crack pattern, and a performed simulation of the project load conditions, we think it opportune to insert “diatoni” in stone, “Diwidag” steel bands in each frontal prospect and stiffening cross “frenelli” in brick on the vault.

The two independent parts of the supporting masonry of the tower were connected using compatible elements to confer Uniform behaviour able to resist seismic forces.

From a careful analysis of the printouts it was deduced that, the results of the stress distributed on the whole structure after the intervention, in terms or ideal stress decreased in the east and south walls, while they stayed unchanged in the remaining walls with values always under the admissible stress. The results of the calculation represented with the graphs and colour maps, in the central zone of the tower (object of the insertion of the “diatoni” in stone), underline a gradual distribution of the stress thus widening the band of low stress. It is finally considered that the project interventions and work have introduced a substantial amelioration of the quality of the building.

REFERENCES


