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Ten years of cultural monuments rehabilitation in Dubrovnik

D. Anicic
Civil Engineering Institute of Croatia, Zagreb, Croatia

ABSTRACT: The realization of the twenty years long program of rehabilitation of historical buildings commenced shortly after the city was shaken by an earthquake that struck the Montenegro region in 1979. More than 1000 historical buildings were damaged in this quake. The author was in charge of the structural design of earthquake strengthening for approximately two-thirds of the total of 60,000 m² of the building area strengthened in the first ten years following the program implementation (1981-1990).

The experience gained and problems encountered during rehabilitation work performed to this date are reviewed. Some less common structural strengthening methods are described and some newly used structural details are presented. Many structures characterized by individually variable structural typology were strengthened: residential buildings, palaces, churches, belfries, fortresses and monasteries. Different structural methods were therefore applied in this rehabilitation.

1 INTRODUCTION

It is generally known that the larger area of Dubrovnik is characterized by high seismicity. An extensive list could be made if we were to cite all destructive earthquakes that have struck Dubrovnik to this date. In the last 325 years the city was damaged by 85 earthquakes whose intensity ranged from 6 to 10 degree on the Mercalli scale and by 11 earthquakes of 8-10 degree intensity, i.e. on average one destructive earthquake every 30 years. The 1667 earthquake is considered to be the strongest one that ever struck the area of Yugoslavia. Its epicenter was located only several kilometers away from the old town of Dubrovnik. The earthquake that devastated the Montenegrin littoral region on April 15, 1979 also heavily affected the town of Dubrovnik, where its intensity was 7 degree and the distance from the epicenter amounted to approx. 80 km. This earthquake once again shook the town, it again opened the formerly repaired cracks, it further weakened the already unstable structures, but it also initiated the long-term rehabilitation program spread over the twenty year period.

342,000 m² of monuments belonging to national heritage were damaged in the 1979 earthquake. 253,000 m² of these monuments belong to the zero category. But, from the seismic aspect, these damages represented only a stimulative force that motivated the extensive rehabilitation activities: urban planning, archeological excavations, monument conservation and archives study, exploratory structural works, architectural and construction design. The living conditions in many previously unmaintained buildings had to be improved. New functions had to be attributed to public buildings. Seismic resistance of all buildings had to be increased so as to be in accordance with the recent achievements made in seismology and earthquake engineering. The existing and yet unrevealed features of cultural and historical significance had to be presented to public in an attractive way.

This paper represents a synthesis of the ten year long experience gained in the field of rehabilitation of ancient stone buildings - historical monuments (Anicic, 1982, 1990, Penelis, 1984). Individual structures were already analyzed in several papers (Anicic, 1983,1990, Moric,1989). This paper will therefore focus only on those structural solutions that differ from those normally applied or which have never before been applied in our country.

2 MONOLITHIC NATURE AND STIFFNESS OF FLOOR STRUCTURES IN THEIR OWN PLANE

One of the main deficiencies of old masonry structures lies in the fact that their floor structures are not monolithic and are not rigid in their own plane. These two properties normally enable transfer of seismic forces on walls characterized by greater resistance, i.e. on those walls that spread in the direction of the seismic force action. Thus the sufficient use is made of the wall's bearing capacity. In the case of old buildings, the floors regularly consist of wooden girders freely leaning onto walls, while stone vaults are only locally found and then usually above the first floor. At such buildings, the spatial action is considered unreliable. The damage usually occurs even after a moderate earthquake action and is due to the displacement of walls in the direction transverse to their plane.

In modern buildings, monolithic nature and rigidity of floor structures is obtained by fabrication of the reinforced concrete slabs or by making use of different types of the monolithized semi-prefabricated floors. The same method may be applied when dealing with old masonry buildings by proceeding to the replacement of the floor structure. However, in the case of cultural monuments, this is not often possible for conservatory reasons (painted ceiling, profiled and decorated beams, stuccowork, height disturbance due to difference in thickness of the old and new floor etc.). In such cases, it is usually attempted to obtain the desired floor properties by applying one of the following methods:

- by placing steel ties in the plane of the floor structure in order to link the walls of the opposite facades,
- by placing bi-directional boards nailed above the joists,
- by establishing physical link between the joists and walls using anchors or ties,
- by erecting truss or slab structures above the masonry vaults,
- by placing horizontal tie beams.

What is the effect of such partial monolithization of the floor structure? A positive practical experience was gained on a small one-story family house in the north Italy that was strengthened after the 1976 earthquake in Friuli. After that, the house was struck by another strong earthquake but remained undamaged. However, the mathematical model analysis of a four-story stone building with mixed (monolithic and wooden) floors gave less optimistic results (Moric,1989). The mixed solutions where stiff reinforced concrete floor slabs are used on some stories, while wooden floors are applied on others, do not lead to significant improvements, since the properties of such structures are much closer to those of structures where all floors are flexible (wooden). Structural engineer will therefore find it difficult to determine seismic safety of a building with flexible floors and of buildings characterized by the combination of stiff and flexible floors: here the basic principles governing the usual computation according to the established rules cease to apply and, hence, the safety can not be precisely defined by such computation. However, the systematic monolithization still ensures the integrity of the building viewed as an entity, which has been confirmed by several tests performed on seismic platforms.

When buildings are situated on the weak foundation soil, the walls should also be linked at the foundation level, i.e. below the ground surface. Strip stone-made foundations of walls are interlinked by the lateral reinforced concrete beams (40x60 to 50x100 cm) forming a grid that ensures common (synchronous) oscillation of all walls of the building. In the case of the good-quality masonry foundations, the link between the existing and new foundation strips is provided by anchors transverse to the contact surface, while in the case of very weak foundations this link is established by the reinforced concrete "plugs" inserted into openings obtained after several stone blocks are pulled out.

3 CONNECTING WALLS PERPENDICULAR TO ONE ANOTHER

In order to enable spatial action of walls that are perpendicular to one another, they should be linked along the vertical joint. Thus the rectangular cross sections are transformed into the T, I or L sections that are characterized by much greater rigidity and bearing capacity. Bending of walls in the direction perpendicular to their plane is thus prevented. Shearing force transfer along the vertical joint of the two walls perpendicular to one another is enabled by the newly placed linking elements.

Steel anchors, i.e. steel bars placed into the previously bored horizontal holes filled with cement mortar or epoxy resin, have proven to be the simple and efficient linking elements. Bar diameter and spacing along the wall height is determined through calculation. In typical cases, 1 bar 20 mm diameter at every 1.0 to 1.5 m of the wall height is considered satisfactory. In cases where vertical joint was "open" during previous earthquakes because of the wall deviation from the vertical plane, such joint was filled by cement mortar.

4 SHOTCRETING OF STONE WALLS

This procedure was applied only exceptionally when the old stone walls were of very poor quality, when their mechanical properties were reduced due to their long exposure to humidity, when they were greatly cracked during earlier earthquakes and when their deviation from the vertical amounted to several centimeters. The procedure of one-sided or two-sided shotcreting was not welcomed by the competent conservationists who subsequently strongly opposed this method so that, in the future, it should be applied only when absolutely necessary.

In addition to some well known work stages - removal of old plaster, cleaning of joints, wall surface rinsing, cement mortar application, reinforcement placing, spraying of shotcrete in 1-2 layers in the total thickness of 6-7 cm, surface smoothening - steel anchors were equally inserted into walls in order to establish a good link between shotcrete and the stone wall. The usual anchor spacing was one anchor at each 1-2 m^2 of the wall area. Special anchor treatment in order to obtain a durable anticorrosive protection includes polishing until the metal gloss is obtained, protection by epoxy resin and a bituminous coating.

In the case of the unstable stone masonry dome of the Dubrovnik cathedral, the shotcreting was performed from above, while the dome was strengthened by the reinforced concrete ring whose view from below is hidden by an ancient richly ornamented stone cornice. Thanks to this procedure, it was not necessary to detach the original dome covering from the lead sheeting.

The effect of wall shotcreting upon the increase of its bearing capacity was investigated in laboratory conditions on the specially prepared samples. Several mechanical properties needed for calculation were analyzed (Anicic, 1989). Since the tensile strength is considered to be a relevant stone masonry wall feature, the bearing capacity of diagonally stressed - strengthened and unstrengthened samples - was compared. It was found out that the bearing capacity of the strengthened wall, due to main tensile forces that cause the wall failure along the compressed diagonal, is three times greater than the bearing capacity of the well constructed but unstrengthened wall. This factor gradually reduces in time due to the strength increase and the stone-mortar adherence so that its final value is 2.5 (Anicic, 1990a). The examined walls were constructed using the M5 lime-cement mortar and the shotcrete grade M25.

Steel fabric 2xQ502 with $f_y/f_u = 609/660$ was applied, the shotcrete thickness was 2x7 cm, while the stone masonry wall was 60 cm in thickness.

5 VERTICAL PRESTRESSING

Seismic strengthening of three belfries situated in the old town of Dubrovnik and of one belfry located out of the city was performed using the vertical prestressing method. The following is obtained by such prestressing:

- greater stability, i.e. protection against overturning,
- bearing capacity sufficient to take on transverse forces,
- bearing capacity sufficient to take on the bending moment,
- greater ductility.

The procedure is simple, efficient and completely acceptable to conservationists. One cable is placed (symmetrically to the vertical axis) at each corner of the belfry and these cables are then used for the vertical prestressing of the belfry. The cables end at the lower and the upper anchororing slab made of reinforced concrete (40-60 cm in thickness) through which the compressive force is transferred to walls. The value of the applied compressive force is such that it provides axial stress in the wall. This stress is equal to the belfry weight stress at the bottom of the belfry. The following effects are obtained: a) reduction in eccentricity of the resultant of vertical and horizontal forces, b) 20 to 70% increase in resistance of the wall to the action of transverse forces, c) higher resistance to bending of the complex stone-reinforced section and d) belfry ductility along the entire height.

6 REINFORCED CONCRETE LINING AND REINFORCED CONCRETE WALLS

From the conservationist's stand-point, it is not allowed to execute the reinforced concrete lining 20 to 30 cm in thickness on the stone wall. Nevertheless, it was possible to apply such linings when the bearing capacity of walls had been completely degraded provided that the lining remains invisible so as not to disturb architectural harmony. Linings running up to the first story were designed for the interior longitudinal walls of the residential - business blocks in Dubrovnik (aprox. 15,000 m^2). These walls, invisible to visitors, had been strongly degraded by the centuries-long waste water outflow through the open narrow channels running between individual blocks. Since the soil of this part of the town can be described as the bedrock, the lining walls were anchored by a number of uniformly distributed anchors.

New reinforced concrete walls were designed in order to obtain the required bearing capacity and stiffness and to ensure a logical plan view distribution of these new spatial elements positioned at the location of the missing stone walls. Where such solution was inappropriate, either new stone walls of size similar to that of old walls were placed or the reinforced concrete walls was installed as "invisible" instead of the previously removed central stone wall layer.

7 MECHANICAL PROPERTIES OF MATERIALS

Basic mechanical properties of materials must be known if we are to perform an acceptable calculation of the building model. It is essential to know the compressive and tensile strength of the stone masonry wall and the elasticity and shear moduli in order to proceed to the analysis using the method of equivalent static forces or the limit state method. In addition, the concrete and reinforcement properties must also be known when the strengthening is made using the reinforced concrete elements. These values are determined by in-situ testing, by laboratory tests or by any combination of these two methods. If we wish to analyse in detail the behavior of each storey, then it is also necessary to define the working diagrams (horizontal force - displacement relationship) of each wall in the elastic and plastic work area.

Procedures for determining these values have considerably been improved in the preceding period and, after a great number of in-situ and laboratory tests, an adequate experience was gained regarding the visual wall quality evaluation (Anicic, 1984, 1989).

In each particular case of renewal of an old masonry building, the designer must determine, in accordance with the prevailing technical regulations and prior to the design preparation, mechanical properties of materials. It is obvious that this part of the job should be performed by the highly qualified and experienced experts.

8 CONCLUSION

This paper focuses on some less common structural solutions applied in the process of seismic strengthening of old stone masonry buildings. Together with the widely applied solutions, they form a set of solutions that give the structural engineer sufficient freedom in making an adequate choice while solving each particular problem encountered. The presented solutions have been applied during renewal of cultural monuments of Dubrovnik and, hence, they have been verified in practice.

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