

Structural analysis of basilica churches: A case study

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ABSTRACT: In this paper, starting from a single case study, a contribution to the problem of modelling and analysis of churches under seismic action is provided. For this purpose, a basilica type church, the San Giovanni a Mare church, located in the historic centre of Naples, is investigated under vertical and horizontal (earthquake-type) loads. The effect of geometrical simplifications in modelling the 3D structural complex is assessed and the influence of cracking in the vaulting system of the church on the 3D behaviour is evaluated. Finally a comparison between strength demand on the single macro-elements of the church and the corresponding elastic capacity is suggested.

1 INTRODUCTION

1.1 Modelling and analysis of churches

Masonry churches are particularly susceptible to damage and partial or total collapses when undergoing seismic actions. The high seismic vulnerability of historic churches can be ascribed both to the particular configuration of this type of buildings, often characterised by open plan, presence of slender walls, lack of effective connections among the structural elements, and to the mechanical properties of the masonry material, i.e. nearly no tension strength.

The research on this topic should necessarily start from the assessment of the seismic behaviour of the building, the characterisation of the most vulnerable points of the structure, and the identification of the possible mechanisms of failure. Unfortunately, the assessment of the structural behaviour under seismic loads of complex masonry buildings, and in particular of churches, deserves objective difficulties due to several reasons that are briefly and schematically highlighted in the following. The analysis of the masonry materials, characterised by nonlinear behaviour and very low tensile strength, requires complex theoretical modelling which are not straightforward to be implemented into a F.E. model. The detailed knowledge of the masonry arrangement of the structural elements is frequently uncertain and incomplete, the mechanical properties of the material may show significant scatters throughout the building, thus the experimental characterisation very often implies simplifications which can mislead from the actual behaviour. Further difficulties are related to the complex geometry, which reflects in three-dimensional (3D) models characterised by a large number of degrees of freedom.

Therefore the need for specific modelling and analysis strategies for historic churches does exist. In this paper, starting from a single case study, a contribution to the problem of modelling and analysis of churches under seismic action is provided.

1.2 Research framework

Despite of the large diffusion of historic church buildings (in Italy are approximately 80% of the monumental heritage) and despite of their seismic vulnerability, only very recently the scientific

community has paid attention to this specific historic constructions, and has developed ad hoc approaches for the evaluation of the structural performance and of the seismic vulnerability. The first research project specifically devoted to the seismic behaviour of churches has been carried out in Italy in the 80's, after the 1976 Friuli earthquake, by the GNDT (Earthquake Protection National Group) and has been focused on the analysis and collection of the damage patterns and collapse mechanisms in the church macro-elements, defined as the parts of the construction characterised by autonomous and unitary structural behaviour under seismic actions. As a result of a wide statistical analysis carried out on a significant sample of churches (Doglioni et al., 1994) quite recurrent damage and collapse phenomenology have been recognised in the single macro-elements of churches. The methodology outlined by (Doglioni et al., 1994) has represented an useful guideline in the development of other subsequent studies on the assessment of church vulnerability, particularly after the 1997 Umbro-Marchigiano earthquake (Lagomarsino, 1998, D'Ayala, 1999, Ciampoli and Giovenale, 1999, Lagomarsino et Al., 1999).

The study presented in this paper is set in the above framework and is part of an activity developed by the authors in the context of several research projects on cultural heritage (CNR PFBBC; PRIN 97, PRIN 2000). In this paper a contribution to the problem of modelling and analysis of masonry churches under seismic actions is provided through the presentation of a case study, the basilica of San Giovanni a Mare, in Naples. It represents one of the several basilica plan churches of the historic centre of Naples, which are currently being analysed by the authors. It is author's opinion that results and conclusions derived from the analysis of a specific case could be extrapolated, through parametric analyses under appropriate hypotheses, to other cases and generalised for covering a wide building category.

1.3 Analysis method

As already stated in the premise, the analysis of the seismic behaviour of the historic masonry buildings, and in particular of churches, is a quite difficult task. Refined mechanical models, for masonry material, have been proposed in the inherent literature, but they are hardly applicable to the complete 3D analysis of complex structural systems. Nevertheless the static and dynamic analysis of the 3D structural complex provides valuable information on the global behaviour and on the interaction among the single elementary parts (the macro-elements), which constitute the structure. The analysis method adopted in the study developed by the authors (Mele et al., 1999) for overcoming the above difficulties is based on a two-step procedure:

first the structure is analysed in the linear range through a refined 3D model, with the aims of characterising the static and dynamic behaviour, defining the internal forces distribution among the single elementary parts and identifying the weak points of potential failure in the building;

secondly the single structural elements are extracted from the 3D context and analysed in the linear and nonlinear range through refined 2D models, with the aims of defining the major structural properties (i.e. vibration modes and periods, horizontal stiffness, lateral strength capacities) which can be utilised for a simplified assessment of the seismic behaviour of the whole building.

The comparison between the results obtained for the 3D models and for the single 2D structural elements allows to derive the approximations related to studying the building by means of the characteristics of its main structural elements. The comparison between the stress pattern in the single structural elements, obtained from elastic analyses of the 3D model, and the collapse mechanism of the single elements obtained either through either nonlinear or limit analysis, allows to evaluate the accuracy of qualitative information that linear analyses can provide. The comparison between the strength capacity of the single 2D elements and the strength demand derived from the 3D analysis, allows to estimate, though in approximate way, the seismic safety level of the building, and suggests the types and locations of the required retrofit interventions.

In this paper only the results of the linear analyses on the 3D structural complex are reported.

2 THE CASE STUDY

2.1 Historical and architectural features

The church of S. Giovanni a Mare, which originally was annexed to a gerosolimitan hospital, is a classical Angioino style monument. It is one of the most ancient church of Naples: the first document reporting about the church is dated 1186.

It is located in a zone close to the seaside, on a sand soil. In figure 1(a) the “insula” urban context of the church is reported, while the plan of the church, is provided in figure 1(b). The original fabric is the central core (the dashed part in figure 1(b)), namely the nave, the lateral aisles, the transept and the semicircular chancel (not indicated in figure 1(b)).

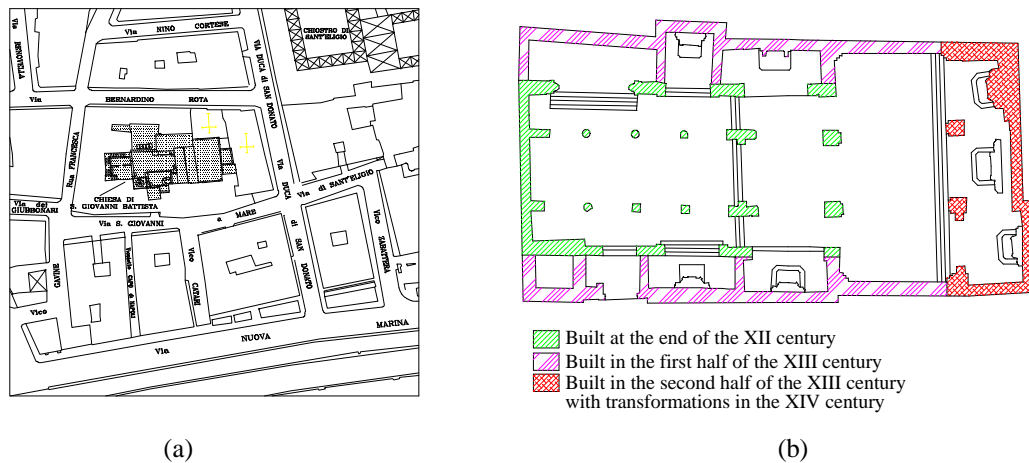


Figure 1 : San Giovanni a Mare church : (a) urban context; (b) plan with chronological modifications.

As shown in figure 1(b), several additions and alterations were made during the centuries: in the 1200, a second transept covered by high cross vaults was added, and the first transept became the fifth span of nave and aisles; in the same period also the lateral chapels were added; in the 1300 the church was re-built and enlarged: the original semicircular chancel was demolished and a new chancel zone, parallel to the transept, containing three altars, was constructed at the east end of the church; in the 1400, following the earthquake of 1456, the timber truss system of the nave and aisles was substituted by cross vaulted systems; in the same period, the bell tower and a big chapel were added; in the 1500 the hospital was disregarded. Starting from the 1800, several restoration works, which deeply transformed the decorative features of the church were made. Following the 1980 Campano-Lucano earthquake, the church suffered extensive damage and remained abandoned up to 1987, when both architectural and structural retrofit works started on. The church has been opened in the 2000. In figure 2 a longitudinal section of the church is represented, while in figure 3 two transversal sections are provided.

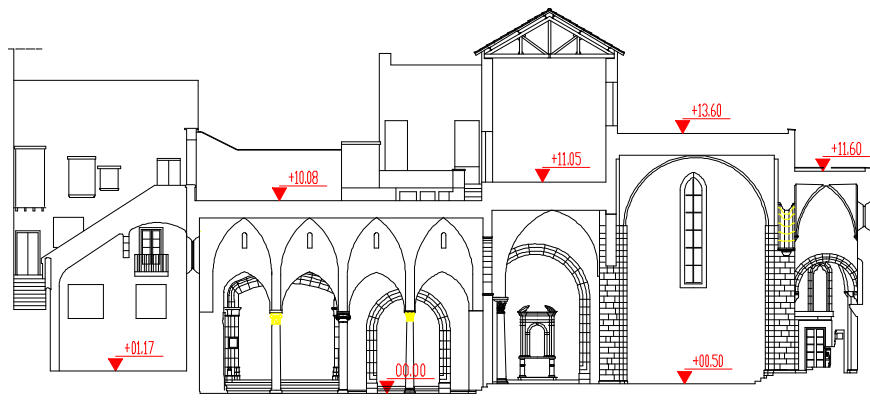


Figure 2 : San Giovanni a Mare church: longitudinal section.

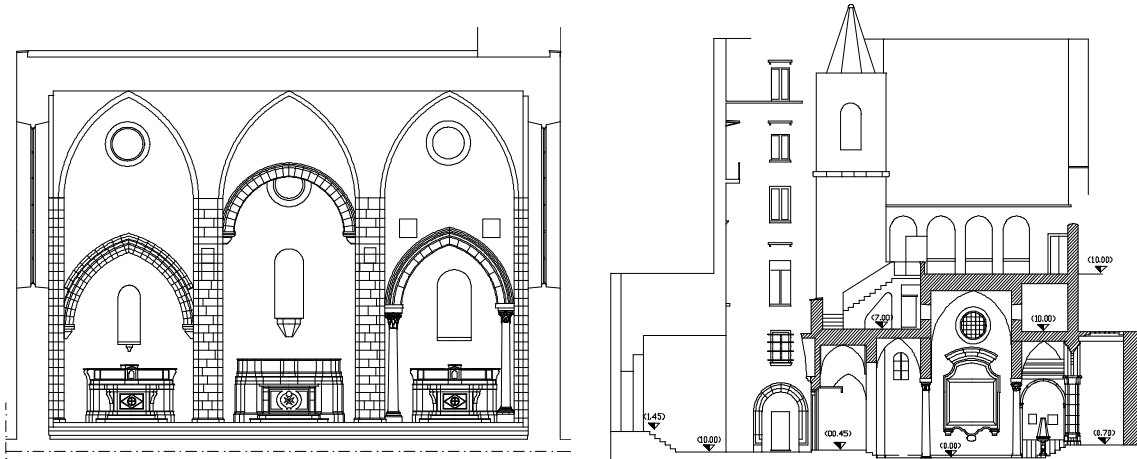


Figure 3: San Giovanni a Mare church: transversal sections.

2.2 Geometry and materials

Even though the alterations during the centuries have significantly modified the original plan, the major elements featuring the basilica type churches are still recognizable in the church of S. Giovanni a Mare, namely: the main nave, the two lateral aisles, the transept and the chancel.

The church is 37,35 m long and 18,7 m wide, has a maximum height of 13,3 m (roof of the transept) and a minimum height of 7 m (vaults of the lateral chapels). The nave width varies between 4,75 and 4.0 m, while the aisles are 2.35 m wide. An atrium is located ahead of the nave, at the west end of the church; the doorway is located on the lateral south elevation. The west part of the three naves (floor level 0.0 m) consists of four spans (three columns per two arcades), while the east part (floor level + 0.50 m), originally occupied by the first transept, consists of two lines of two pillars, which support pointed arches. The columns of the nave arcade are made of marble stones, have circular section with diameter 500 mm, have no base and each one has a different capital. The pillars in the east part of the church are made of tuff masonry and have rectangular section 1.05x1.15 m.

The structural system of the roof of the church is made of several vaults: the nave, the aisles and the transept are covered by cross vaulted systems, while some lateral chapels are roofed by either barrel or groin cross vaults. The chancel zone has quadripartite ribbed vaulting systems, with stone ribs having no structural function, but only decorative purpose.

The walls have thickness varying between 0.80 and 1.00 m, and are made of tuff masonry; for this material, reliable values of the mechanical properties are: Young modulus $E = 1100$ MPa, tangential elasticity modulus $G = 518$ MPa, $\nu = 0.071$; weight $\gamma = 17$ kN/m³, compression strength $\sigma_c = 1.7$ MPa, tension strength $\sigma_t = 0.165$ MPa.

2.3 Modelling: schematic plan and macro-elements

Due to the considerable irregularities of the church fabric, a simplified arrangement of the plan has been set up for the structural analysis of the church. From this “design” plan (figure 4), quite more regular than the actual one, the single macro-elements which constitute the structural system of the church can be identified. In particular the transversal elements are appointed as T1–T6 and respectively represent: the east end wall of the chancel (T1), the triumphal arch between the chancel and the transept (T2), the transversal section separating the transept and the naves (T3), two transversal section in the nave zone (T4 and T5), the west façade (T6). In the longitudinal direction the church has a roughly symmetrical layout, with the elements quite similar two by two. The longitudinal elements are appointed as L1 – L6, and respectively denote: the north and south longitudinal elevations (L1 and L6), the arcades separating the lateral chapels from the aisles (L2 and L5), the arcades between the aisles and the nave (L3 and L4). These single macro-elements have been extensively analysed through two-dimensional (2D) models, both in the linear and nonlinear range; for sake of brevity these analyses are not reported in this paper.

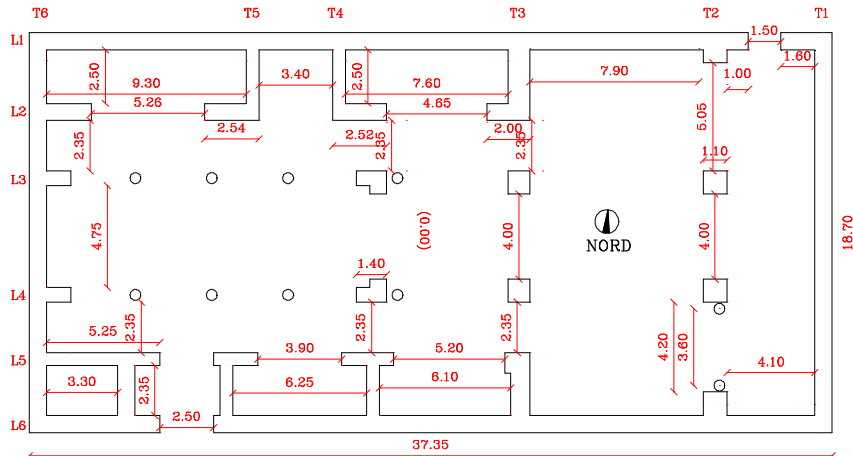


Figure 4 : San Giovanni a Mare church : schematic plan.

Four 3D models of the entire church, represented in plan in figure 5 and appointed as plan-a, plan-b, plan-c, and plan-d models, have been developed. The model plan-a (fig.5(a)) is given by the spatial assemblage of the 2D models of the single macro-elements, while the model plan-b (fig.5(b)) accounts for some misalignments of the macro-elements, as actually present in the church; in the model plan-c (fig.5(c)) the presence of the bell tower has been considered. While in these three models the hypothesis of a rigid floor connection at different levels has been adopted, in the fourth model, appointed as plan-d, this hypothesis has been removed. The position of the rigid diaphragm connection among the macro-elements at different levels is indicated in fig. 5 (d).

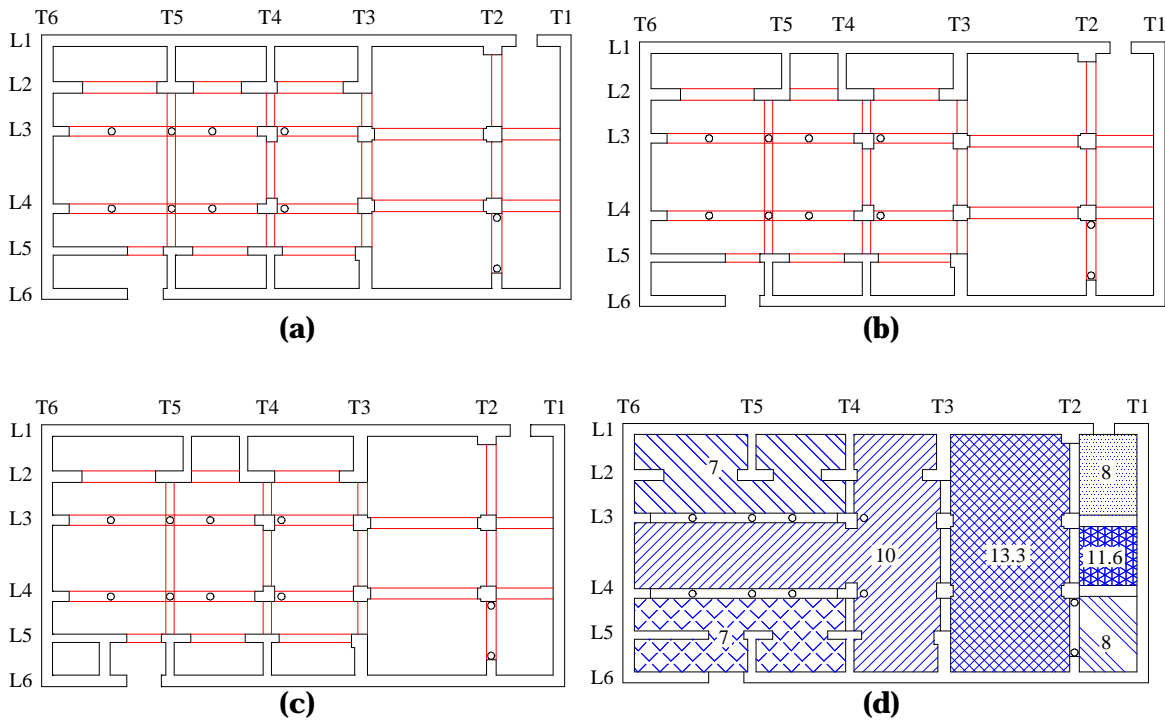


Figure 5 : (a), (b), (c): Plans of the 3D models; (d) levels of rigid floor connection

2.4 The vaulted roof system and the vertical loads

In figure 6 (a) the roof vaulted system of the church is outlined. The major part of the roof elements are cross vaults (either groin or ribbed), which transfer concentrated loads in four supporting points, the vertexes of the covered quadrilateral surface. Some barrel vaults, which transfer loads along the two springing lines, are present in the zone of the lateral chapels.

This sketched plan, which roughly define the roof load transfer path, has been utilised for deriving the vertical loads at the top of the single macro-elements. A preliminary evaluation of the weight of the vaulted system (weight of: vault web and ribs (if any), spandrel infilling, finishes) has been carried out and an equivalent unitary load value has been derived. For this purpose the church plan has been divided in different fields according to the size of the covered surface. In particular three zones, respectively appointed as A, B, and C (fig.6(b)) have been distinguished:

Zone A: lateral aisles and minor chapels, chancel (vault spans less than 6 m);

Zone B: nave and major chapels (with barrel vaults);

Zone C: transept, with the largest (quadripartite) vaults of the church.

For the three zones, different thickness values of the vault structure and of the infill material have been determined, giving rise to different values of the total weight (table 1).

On the basis of these unitary loads and of the geometric tributary areas, the vertical loads acting at the top of the single macro-elements have been evaluated.

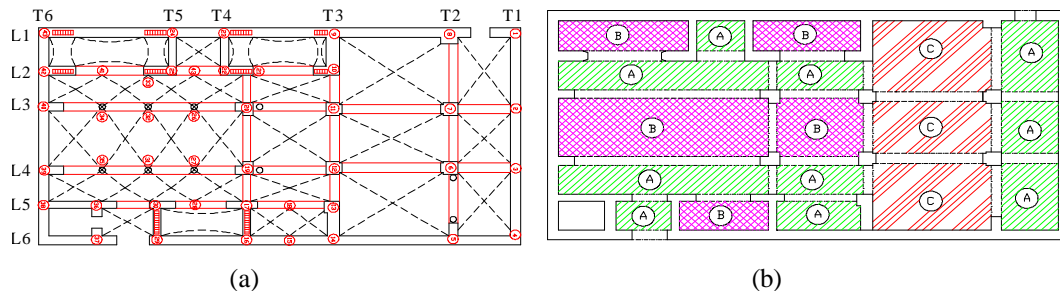


Figure 6 : (a) vaulted roof system; (b) zones of different roof weight.

Table 1 : vault thickness and weight

	Total thickness	Structure thickness	Infill thickness	Total weight
	Mm	mm	mm	KN/m ²
Zone A	900	300	600	11.5
Zone B	1500	450	1050	18.1
Zone C	2000	600	1400	23.8

3 THE 3D STRUCTURAL MODEL

3.1 FEM model

In order to grasp the global behaviour of the church, the four three-dimensional models of the structural complex (plan a, b, c, and d, as illustrated in the previous section) have been developed using the Finite Element Method (FEM) computer code SAP 2000. The models consist of about 6500 joints and 6000 shell elements, plus some 1D elements for the columns and pillars supporting the nave vaults. In figure 7 some prospective view of model b and c are provided. Linear analyses of the 3D models of the church have been performed in static and dynamic range. In the static analyses the models have been subjected to the vertical loads deriving from the masonry own weight and from the roof loads, as well as to conventional horizontal actions, equivalent to seismic loads, applied at points corresponding to distributed and lumped masses. Two separate load conditions have been considered for the seismic analysis, respectively with horizontal actions applied along the longitudinal direction of the church and along the transversal direction.

In the following the results of the analyses performed on the four models of the church are given and discussed with the aim of evidencing the main aspects of the global seismic behaviour and of estimating the distribution of the internal forces among the different structural elements of the church.

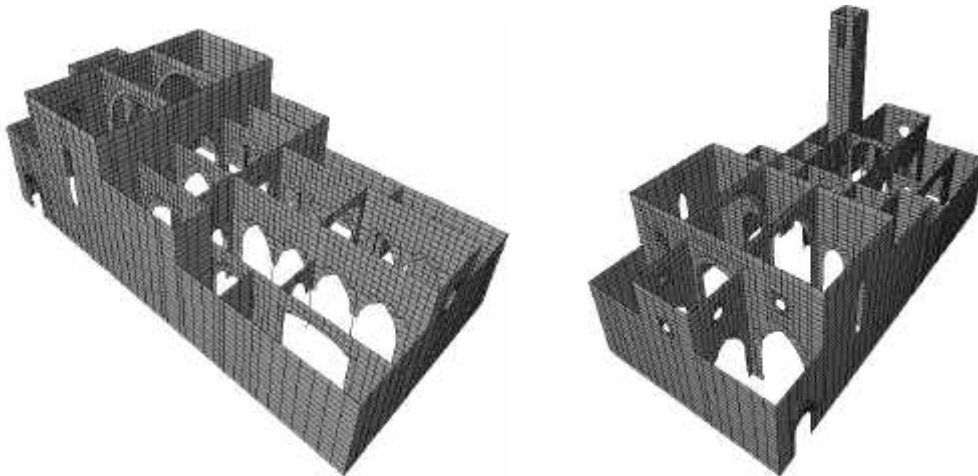


Figure 7 : prospective view of the FEM models. (b) and (c).

3.2 Dynamic behaviour: mode shapes and periods

In the figures 8, 9 and 10 the plan representation of the first vibration modes respectively of the plan a, plan c and plan d models of the church are provided together with the relative periods. In the tables 2, 3 and 4 the participation factors of the total mass (in terms of ratio between the single mode participant mass, M_i , and the total mass M_{tot}) of the most representative vibration modes in the longitudinal and transversal direction are reported, respectively for the plan a, plan c and plan d models of the church. The plan b model has been omitted since it provides results very similar to the plan a.

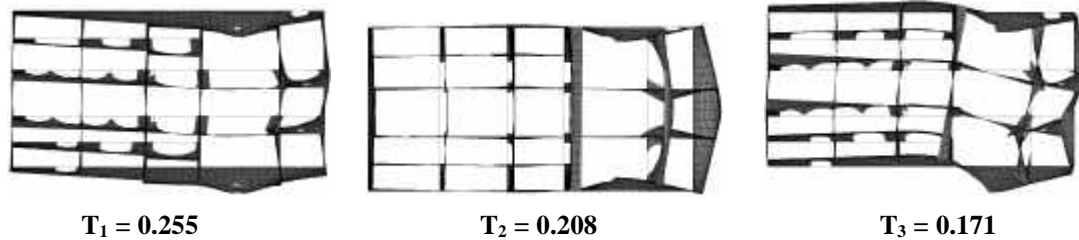


Figure 8 : plan (a): mode shapes and periods.

Table 2 : plan (a) : periods and mass participation factors.

Mode	Transversal direction			Mode	Longitudinal direction		
	Period (sec)	M_i/M_{tot} (%)	$\Sigma M_i/M_{tot}$ (%)		Period (sec)	M_i/M_{tot} (%)	$\Sigma M_i/M_{tot}$ (%)
1	0,255	71,840	71,840	2	0,208	55,585	55,588
3	0,171	1,227	73,077	4	0,148	14,246	69,869
7	0,115	0,698	73,956	5	0,133	1,472	71,341
8	0,112	3,461	77,418	6	0,132	6,791	78,132
15	0,087	6,487	84,256	11	0,097	3,000	81,344

The first vibration mode of the plan-a model involves about 72% of the total mass in the transversal direction and gives rise to remarkable out of plane deflections of the longitudinal macro-elements, which therefore are likely to provide a contribution in the absorption of the transversal seismic actions. The second and fourth mode respectively excite 56% and 14% of the total mass in the longitudinal direction, producing less pronounced out of plane displacements of the orthogonal macro-elements. The higher mode shapes involves local deformation modes, with maximum 6-7% of the total mass (like in the 15th mode in the transversal direction and the 6th mode in

the longitudinal direction). The third and eighth mode show torsion deformations of the church, particularly concentrated at the first and second transept: however the low values of mass participating factors reported in table 2, suggest that these modes have a minor influence in the vibration of the complex. Very similar considerations apply to the plan-b model.

Also in the plan-c model, where the bell tower has been included, the global dynamic behavior is unchanged, with the exception of the first two vibration modes which involve local deformations of the tower, in the transversal and longitudinal direction, with participating mass equal to 6% and 5% respectively. The considerable slenderness of the tower (20m high, with square plan approximately 3 x 3m) affects the value of the first and second period ($\cong 0.45$ s), giving rise to values which are about twice the global ones of the entire structure ($\cong 0.24$ s). The 9th, 12th and 13th modes evidence torsion of the bell tower.

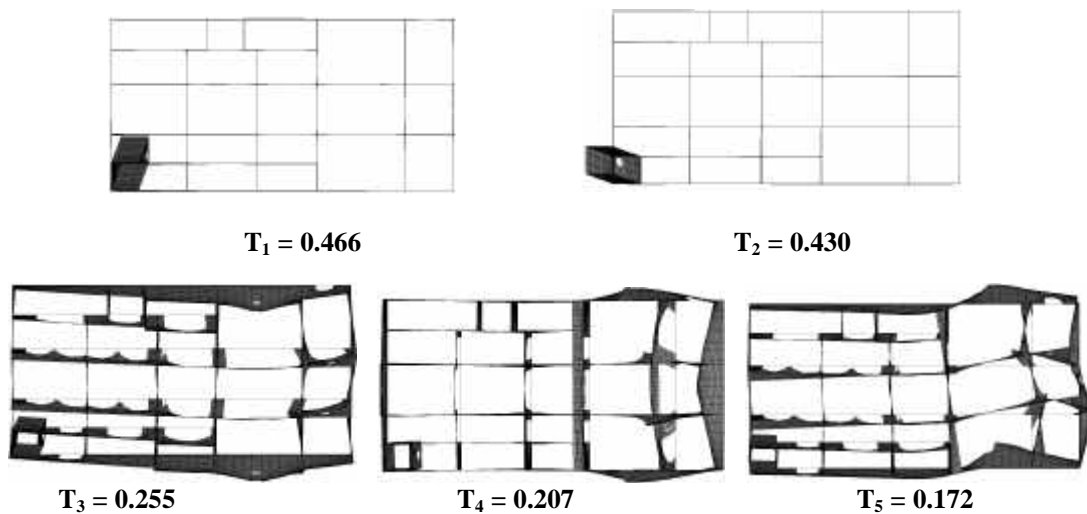


Figure 9: plan (c): mode shapes and periods

Table 3 : plan (c) : periods and mass participation factors.

Mode	Transversal direction			Mode	Longitudinal direction		
	Period (sec)	M_i/M_{tot} (%)	$\Sigma M_i/M_{tot}$ (%)		Period (sec)	M_i/M_{tot} (%)	$\Sigma M_i/M_{tot}$ (%)
1	0,466	6,399	6,399	2	0,430	5,043	5,961
2	0,430	0,388	6,787	4	0,207	51,939	57,902
3	0,255	66,321	73,108	6	0,148	14,145	72,088
5	0,172	1,109	74,220	7	0,134	3,535	75,624
9	0,118	3,631	78,036	8	0,132	3,444	79,068

The plan-d has been obtained from the plan-a model by removing the rigid diaphragm connection among the macro-elements, thus the two models are directly comparable for assessing the effect of extensive and severe cracking in the vaulted roof system. The dynamic behaviour of the plan-d model is significantly different with respect to plan-a, both in terms of the mode shapes and of period values. The first mode involves longitudinal deformations, while the second one is a transversal mode. The increase of the global deformability reflects on the of the first four period values, much higher than the ones of the previous models: the first period is equal to 0.403 sec, while only starting from the sixth mode the period becomes comparable to the ones registered in the other models (0.27 sec). Also the modal participating mass is very different, since the first 35 modes give rise to a total participating mass approximately equal to 73% in the transversal direction and to 69% in the longitudinal direction. With the exception of the first and second modes, the modal participating mass is always less than 10%, thus confirming the loss of the box-type behaviour.

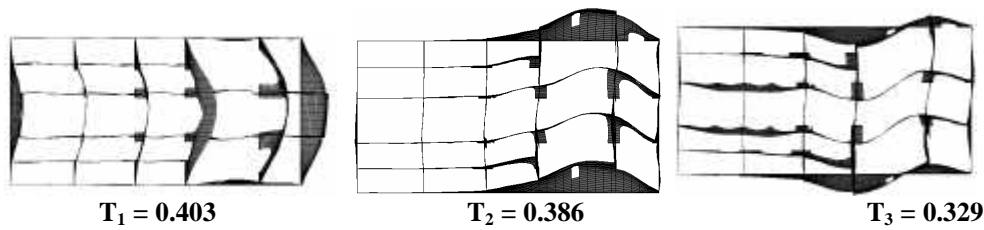


Figure 10 : plan (d): mode shapes and periods.

Table 4 : plan (d) : periods and mass participation factors.

Mode	Transversal direction			Mode	Longitudinal direction		
	Period (sec)	M_i/M_{tot} (%)	$\Sigma M_i/M_{tot}$ (%)		Period (sec)	M_i/M_{tot} (%)	$\Sigma M_i/M_{tot}$ (%)
2	0,3865	37,369	38,344	1	0,4029	41,356	41,356
3	0,3292	11,722	50,066	6	0,2696	6,6508	48,219
4	0,3085	8,3982	58,464	16	0,1592	3,7172	52,023
7	0,2297	2,3652	60,902	17	0,1555	10,318	62,342
12	0,1877	3,5296	64,432	19	0,1461	4,897	67,239
14	0,1709	1,6875	66,119	23	0,1388	0,5913	67,83
15	0,1621	4,1999	70,319	24	0,1329	0,8296	68,659

3.3 Static analyses

In figure 11 the results of the analyses on the four 3D models are compared by reporting the base shear force V_i in the single macro-elements, normalized to the global base shear action V_{tot} , applied to the building. The histograms in figures 11 (a) and (b) are respectively referred to seismic action in the transversal and longitudinal direction of the church.

The comparison among the 3D models suggests a negligible influence of not large geometrical simplifications, as the ones operated in the case study (plan-a, and -b models), confirming the observations coming from the dynamic analyses. Also the addition of the bell tower (plan-c model) does not produce remarkable differences. The trend in the shear force distribution for the first three models suggests a concentration on the stiffest elements of the church, namely the perimeter ones, both in the transversal and in the longitudinal direction; for the plan-c model, a slight increases of the shear force absorbed the west façade (T6), due to the presence of the bell tower mass, can be observed. On the contrary, the analysis accounting for vaults cracking (absence of rigid floor connections among the macro-elements, plan-d model) evidences quite sensitive modifications in the force transfer path and in the deformation modes: the loss of a monolithic behavior of the building produces decrease of shear force in the perimeter elements and an increase in the more deformable, central elements. Finally the contribution of the elements located along the orthogonal direction with respect to the seismic action is not negligible: the longitudinal elements globally absorb 21÷27%, of the total applied shear, while for the transversal elements the contribution is about 10÷15%.

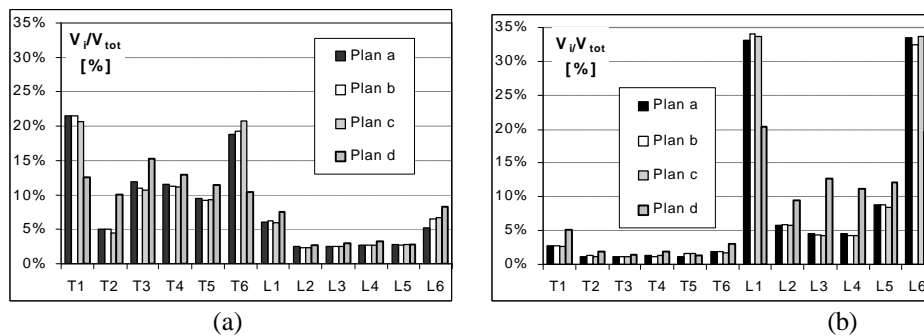


Figure 11 : distribution of shear forces among elements : (a) transversal, (b) longitudinal seismic action.

In figure 12 the values of the base shear V_i in each macro-element, normalised to the total vertical load W_i acting on the element are reported. These values, which provide a measure of the strength demand on the elements, are compared to the normalised horizontal load multiplier leading to the first cracking, which provides a measure of the element elastic strength capacity. Due to the negligible differences among the models plan-a, -b and -c, this comparison between elastic strength demand and capacity has been carried out only for the plan-a and plan-d models (figure 12 (a) and (b), respectively), allowing to assess, though in an approximate way, the level of safety of the building in the two conditions.

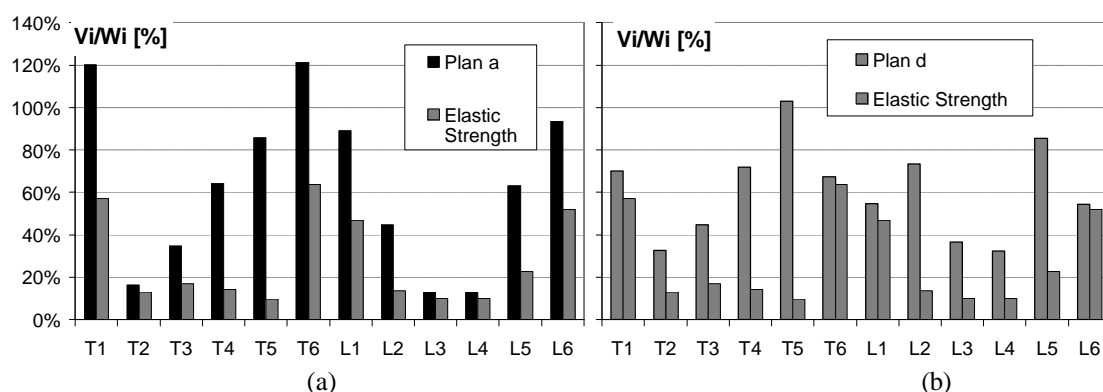


Figure 12 : comparison between elastic strength demand and capacity : (a) plan-a, (b) plan-d model.

4 CONCLUSIONS

The analysis carried out in this paper through a complete three dimensional model of a real church has provided the main characteristics and the behaviour of this particular type of building (basilica).

The contribution of the masonry panels out of their plane has been quantified through dynamic and static analyses. The effects of different levels of “simplifications” of the geometry have been proven to provide negligible effects in terms of shear distributions. Finally, the effect of rigid diaphragms has been evaluated. It has been demonstrated that rigid diaphragms not necessarily lead to an improvement of the performance since the stiffer elements are overloaded beyond their capacity.

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