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Settlement Induced Damage Modelling of Historical Buildings: the Bell Tower of the "Basilica dei Frari" in Venice

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Abstract The paper presents the case study of the *"Basilica dei Frari"* in Venice for which a non linear numerical analysis has been recently performed in order to assess its structural conditions. In fact, from the end of its construction, in the XIV century, the building suffered from structural deteriorations mainly due to *settlements* affecting the bell-tower. A main structural intervention was carried out at the beginning of the XX century, aimed at stopping the outward tilting process of the tower. The intervention was so effective that it induced an opposite effect on the tower, which started to rotate towards the cathedral. Several studies were carried out since then to evaluate the interaction between tower and church, including in recent years structural monitoring and numerical modelling, besides a strengthening intervention consisting in soil micro-fracturing. A non linear numerical model of the church-tower complex was implemented and compared to the outcomes of the available experimental data (monitoring, investigations), also considering the historical process leading to the present day conditions. To gain reliable settlement damage predictions it was necessary to adopt tensile-softening crack models in the numerical studies and perform non linear analyses able to trace the complete response of the structure. The aim of the modeling was also, besides the assessment of the structural conditions of the complex, to predict the structural effects of the physical "separation" between tower and cathedral.

Keywords: Differential settlement, non linear numerical models, structural monitoring

Introduction

S. Maria Gloriosa dei Frari Basilica was built in Venice between the first half of the XIV and the second half of XV century in gothic style (Fig. 1). During the centuries the building suffered from structural damages mainly caused by the interaction with the bell tower.

The tower, built next to the church between 1361 and 1396, was originally conceived as a complete independent structure. With its 9.50 m wide base side, it is 65 m tall and shows a double pipe brick masonry structure, supporting the internal staircase. Subsequent construction phases brought the church to its present form, completed in 1432, when St. Peter's chapel was erected between the church and the bell tower on the South-East corner (Lionello 2008). During each phase the tower was progressively linked to the church both in foundation and in elevation. It is subjected to higher concentrated loads then the surrounding structures of the cathedral, thus manifesting larger vertical displacements. Subsidence was followed by a rigid rotation and subsequent out of plumb of the tower toward South-East (outer side of the church). Structural problems in the tower had been pointed out already since the end of the XVI century, but the first documented restoration and consolidation interventions date back only to the second half of the XIX century. The first noteworthy structural intervention on the tower was performed at the beginning of the XX century and it led to enlarge and strengthen its *foundations* in order to distribute the vertical loads on a larger surface and consequently reduce the local stresses. The strengthening intervention provided to the bell-tower foundations would have been extended also to the internal sides, but due to emergency reasons that prioritized

other interventions, the consolidation of the internal substructures was never realized (Lionello et al. 2005). For these reasons the foundation's enlargement reduced the vertical settlement, but also caused an unexpected reverse of tower's rotation toward the church. The new structural configuration caused, in the following decades, the progressive formation of widespread cracks and extensive damages on structural elements of the church directly connected to the bell tower (masonry walls, columns and vaults), generating crack pattern and deformations in constant evolution.

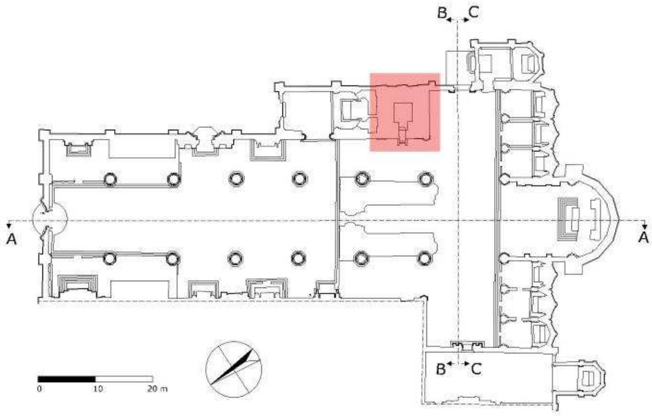


Figure 1: "Santa Maria Gloriosa dei Frari" basilica and bell-tower. Plan view of the church

On Site Investigations and Monitoring System

In 1990, after an extensive study concerning the behavior of several Venetian bell towers, a *diagnostic investigation* on the Frari bell tower started. Experimental tests carried out in the following years included: photogrammetric survey, geotechnical investigations on the soil foundation, endoscopic tests, single and double flat-jack tests, sonic tests. Moreover, a *monitoring* of the main cracks, by means of extensometers, crack gauges, temperature sensors and a direct pendulum to check the tilt and control the displacements of the tower, was implemented (Lionello et al. 2005).

The analysis of the results of the test campaign, revealed a discrete stability of the tower structure that, in spite of an out of plumb of about 80 cm, was not showing anomalous displacements.

In September 2000, after less than one century from the end of the complex intervention in foundations, further openings of some existing cracks, fall of plaster's fragments and loss of brick elements from masonry vaults, clearly testified the worsening of the structural situation. It was decided to start immediately a recognition of the visible signs of structural degradation, e.g. cracks, deformations, collapses, etc. in order to understand the nature of the ongoing cinematic mechanisms that evidently indicated changes in the global equilibrium conditions. The stone arch at the North-West corner of the tower, already rebuilt at the end of 1800's, and the adjacent vault of the left aisle showed worrying deformations, close to the geometrical stability limit. The conclusions of the first investigations and monitoring indicated the necessity of a strengthening intervention on tower's foundations in order to stop its progressive differential settlement: such intervention consisted in the so called "soil fracturing" technique, e.g. injections of cementitious grout, under controlled conditions. Such injections produce expansions and/or fracturing of the clay soil around the stone basement of the tower and are able to improve its mechanical properties (Lionello 2008).

Structural Diagnosis

The structural problems arising from the interaction of the church-tower complex were demonstrated by several surveys, experimental tests and monitoring. The most evident signs of the mechanical interaction between tower and church is a system of diagonal cracks on the masonry panel over the stone arch that connects the tower to the column at the corner between nave and transept. Cracks follow the isostatic tensile pattern in the areas where stresses exceed the *tensile strength* with the

consequent creation of a “stream of compression” (in the portions of masonry between cracks) able to transmit significant components of vertical and horizontal forces from the tower to the church (Lionello 2008). The load transfer from the bell tower to the church was proved by the execution of flat-jack tests on the masonry panel over the stone arch between tower and column (Fig. 2). The results of tests clearly indicate the compression’s component through which part of tower’s self weight is transmitted from the tower to the column. Along this line stresses vary from 0.56 to 0.95 MPa, while outside that line stresses are equal to zero.

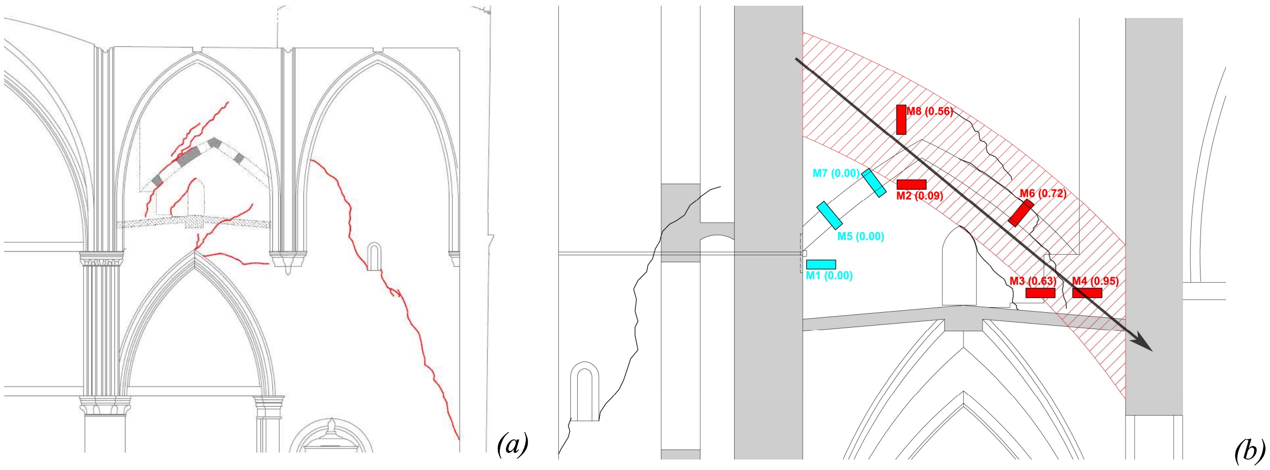


Figure 2: Full-open cracks on the South-East side of the bell tower, along the transept (a). Load transmission from the tower (on the left) to the church’s column (on the right) (b)

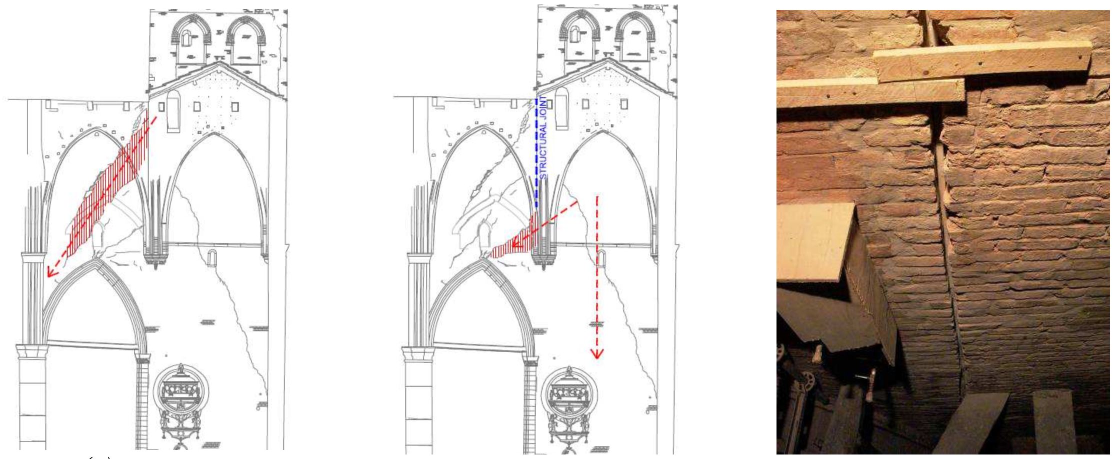
This complex mechanical system has a strong *nonlinear behavior*, which makes it quite hard to assess quantitatively the entity of the involved forces.
The consequences of internal stresses created by the mechanism described above are:

- An increase of the compression load on the column, much more higher than the contribution of the self-weight of walls, vaults and roof applied in normal conditions, i.e. without settlement.
- The formation of a strong transverse bending stress on the column, due to both the eccentricity of the vertical load (above-mentioned) applied to it and the horizontal component of the thrust.
- A decrease of the vertical load (equal to the increase on the column) on the tower that is also subjected to a bending stress due to the above-mentioned causes (eccentricity of the vertical load on the tower and horizontal component of the thrust).

The structural situation defined by experimental tests stressed the need to design an intervention aimed at solving the *interaction problem* of the church-tower complex. The intervention was validated through a finite element numerical model, which includes the portion of the structures involved in the problem. Finally the model was used to simulate the intervention and evaluate its effects on the structure.

Numerical Analysis

A structural analysis of the bell-tower and the adjacent part of the basilica was performed through a *finite element model*. To this purpose the FE code DIANA (Delft, release 9.2, 2007), able to implement several non linear constitutive laws, was used. The basic principle of the proposed intervention is to separate the bell tower from the church by means of the creation of a *discontinuity joint*, able to allow differential movements between the two structures (Fig. 3). The aim of this intervention is also to reduce the eccentric load on the column at nave-transept intersection and remove the compressive component that transfers significant vertical and horizontal forces from the tower to the church’s structures.



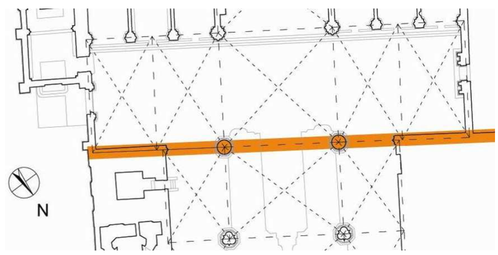
(a) *(b)* *(c)*

Figure 3: Static situation and loads transmission between tower and church before (a) and after (b) the proposed intervention. Execution of the structural joint (c)

Information on the trend of settlements during historical phases that have characterized the building's life emerged from historical documents and surveys and it was used as *input data* for numerical analyses. The combination of phased analysis with the implementation of non-linear constitutive laws of materials allowed to simulate the settlements' evolution of the structure during centuries until the pre-intervention state and then analyze the different steps of the intervention, assessing its effects on the building. The results of flat jack tests, indicating the local state of compression, have been used to *validate* the model (Lorenzoni 2009). Portions of the structure and loading conditions which are relevant in relation to the structural problem, i.e. the effects of church-tower interaction, were considered in the models. A 2D numerical model was then created in correspondence to the last line of columns at nave-transept intersection.

Table 1: Material properties and cross section of the church-tower complex implemented in the FEM

Masonry's type	*Tower*	*Church*	*Foundations*
Young's modulus E [N/mm^2]	3000	3000	3000
Poisson's ratio ν	0.2	0.2	0.2
Density ρ [kg/m^3]	1800	1800	1800
Tensile strength, f_t [N/mm^2]	0.25	0.25	N/A[1]
Fracture energy, G_f [N/mm^2]	∞	∞	N/A[1]
Comp. strength, f_c [N/mm^2]	3	3	N/A[1]
Elastic hardening, E_{har} [N/mm^2]	3	3	N/A[1]



[1] Foundations were modeled as linear elastic

Portions of structure not included in the model but directly involved in the structural interaction (e.g. vaults of transept, nave and aisles, the wooden structure of the roof, etc.) were considered by the introduction of corresponding loading and boundary conditions.

Foundations of tower and church has been modeled as well as the underlying soil in order to obtain settlements of the building during different historical stages (Fig. 5a).

Once defined the model's geometry a structured mesh has been created; it is composed by 14271 two dimensional elements and 22914 nodes (Fig. 5b). Two material laws were chosen in this numerical study i.e. the crack model for masonry and the soil model for the underlying ground.

To gain reliable settlement damage predictions of masonry buildings and allow for realistic fracture properties in quasi-brittle materials, it was necessary to adopt tensile-softening crack models (total strain rotating crack model). Masonry is assumed to have a constant behavior in tension with infinite value of fracture energy (Gf) and a linear hardening behavior in compression (Table 1). As the aim is to focus on crack behavior of masonry and thus to obtain a realistic tensional state according to

development of differential settlement between tower and church, it was decided to take a simpler, isotropic and linear-elastic model for the soil.

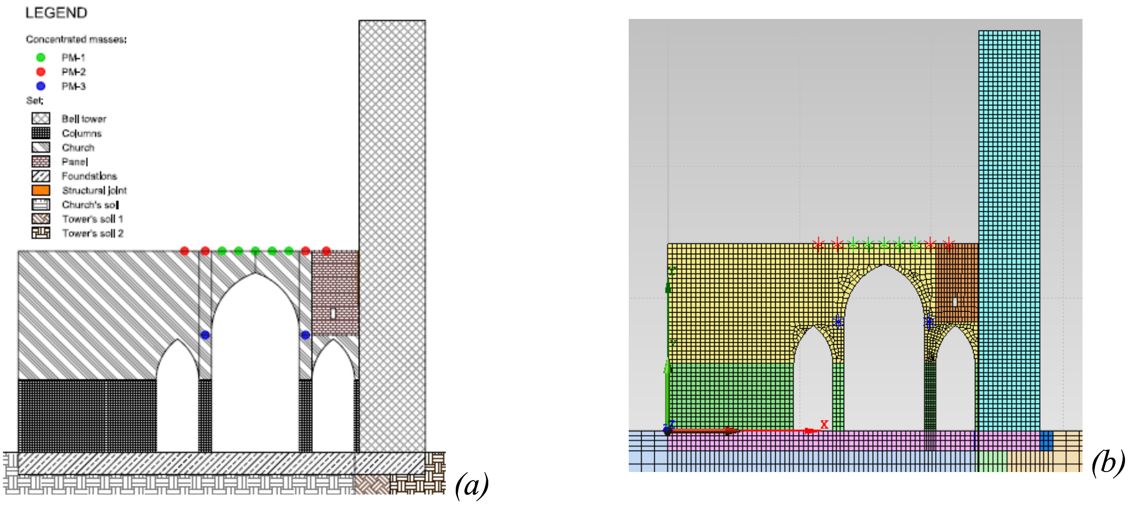


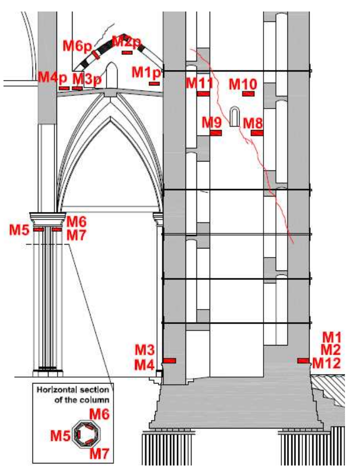
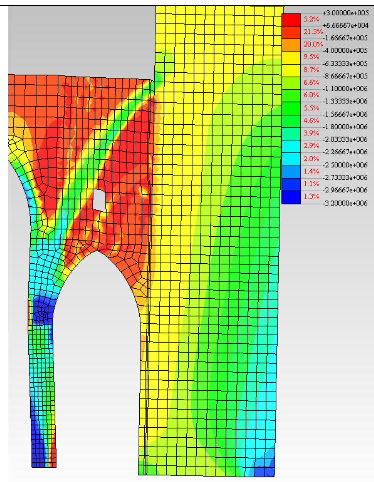
Figure 4: Scheme of the FEM: loads and material properties assigned to the model (a). Structured mesh (b)

A *phased nonlinear analysis* was then performed in order to follow the four main historical and structural stages of the monument's life. In each phase a separate analysis was performed, in which the results from previous phases are automatically used as initial values. The three main historical phases considered in the analysis are:

- Phase 1: from the end of building's construction until 1902. It simulates settlements that caused the bell-tower out-of-plumb surveyed at the beginning of XX century.
- Phase 2: (pre-intervention state): 1902-2008. It considers settlements during the last century that determined an inversion of tower's rotation.
- Phase 3: (post-intervention state): August-October 2008. It corresponds to the creation of the structural joint between church and tower, following the different phases of cut's execution.

In each phase the results of the FEM (displacements, stresses, strains, etc.) are compared to the outcomes of experimental tests and monitoring (flat-jack tests, indicated with M1, M2, …, Mn, direct pendulum, topographical survey, etc.) in order to check and demonstrate their reliability or re-calibrate the models in case of remarkable discrepancies (Table 2).

Table 2: Comparison of the tensional state between experimental data and numerical results

Single flat-jack test	Experimental σ_V [MPa]	FEM σ_V [MPa]	Positions of test	FEM
Tower base				
M1 M2 M12	1.92	1.70		
M3 M4	1.44	0.71		
Column's top				
M5	1.76	1.96		
M6 M7	3.12	3.12		
Masonry panel				
M1_P	0.00	0.06		
M2_P	0.09	0.09		
M3_P	0.63	0.97		
M4_P	0.95	1.34		
M6_P	0.72	1.21		

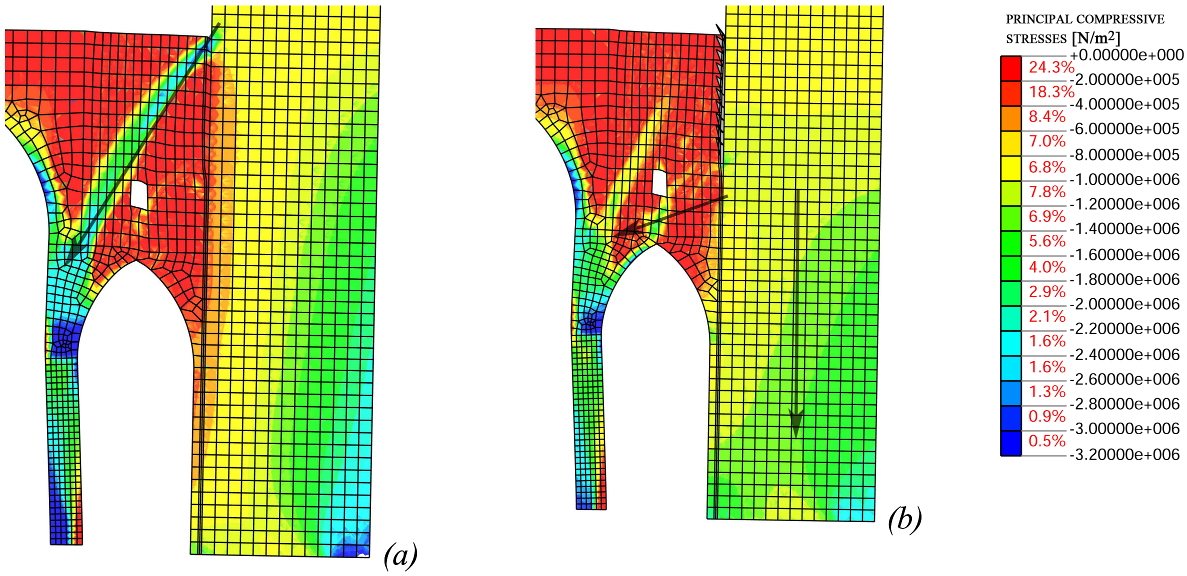


Figure 5: Development of principal compressive stresses before (a) and after (b) the intervention: the component of compression is considerably reduced after the execution of the structural joint.

Conclusions

The *numerical models results* confirm the hypothesized structural diagnosis: similar values of stress have been recorded both in experimental tests and in the modes, where is clearly visible the compressive component on the masonry panel over the arch of the left aisle. It determines a "stream of compressive forces" able to transmit significant components of vertical and horizontal load from the tower to the church's column. The creation of the structural joint between church and tower demonstrates to have *positive effects* on the global structural behavior of the building and in particular on column's stability. Analyzing the distribution of principal compressive stresses before and after the intervention (Fig. 6) it can be noted that the "stream of compression" has been almost completely removed, or at least considerably reduced.

The outcomes of the FEA confirm that the structural intervention improve considerably the global static condition of the building and in particular it has been revealed:

- a considerable reduction of the load on the church's column, estimated in about 800 kN (-17%);
- a correspondent increase of loads and stresses at tower's base (+17%);
- a return to the original static situation of the building in which the two structures were structurally more independent (Lorenzoni 2009).

References

[1] Diana (2007) "*Finite element analysis user's manual*," Release 9.2, TNO Building and Construction Research, The Netherlands

[2] Lionello, A (edited by) (2008). "*Il campanile di Santa Maria Gloriosa dei Frari in Venezia*." Ed. Verona, Italy: Electa.

[3] Lionello, A, Cavaggioni, I, Rossi, PP, Rossi, C, Modena, C, Casarin, F, Marchi, G, Gottardi, G and Ragazzini, A (2005). "*Preliminary investigation and monitoring for the design of a strengthening intervention on the Frari basilica, Venice*," in *Proc. 4th Int. Seminar on Structural Analysis of Historical Constructions*, Padova 2004, 1323-1333.

[4] Lorenzoni, F (2009). "*Settlement induced damage modeling of historical buildings: two case studies: Amsterdam and Venice*." MSc thesis, supervisor M.R. Valluzzi, Padova.

[5] Rots, J G (2002). "*Comparative study of crack models*," in *Proc. 3rd DIANA World Conference 2002*, Balkema, Lisse, 17-28