Preliminary investigation and monitoring for the design of a strengthening intervention on the Frari basilica, Venice

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ABSTRACT: The "latest" 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. From the end of the construction the building suffered from structural deterioration mainly due to settlements affecting the bell-tower. At the end of the 19th and at the beginning of the 20th century the church underwent major repairs because of the increasing sinking and leaning of the bell-tower, causing worrying damages on the basilica adjacent structures. During the 20th century the bell-tower still suffered from differential settlements; the early 900's foundation strengthening did not stop the tower to settle. This paper presents the main results of experimental and numerical investigation recently carried out in order to assess the present safety conditions of the structures and to feed the necessary strengthening intervention at the level of the bell-tower foundations.

1 THE BELL-TOWER

Built between 1361 and 1396, the "Frari" bell-tower is conceived as a complete independent structure; first designed as a separated body, the bell-tower, with its 9.50 m wide base side and 65.00 m tall, shows a double pipe brick masonry structure, supporting the internal staircase.

The reconstruction of the Basilica, started in 1340 and finished in the second half of the XV century, entails however the inclusion of the south-east corner of the structure of the bell-tower inside the masonry walls of both transept and left aisle, at their junction (Figure 1).

In 1432 takes place the construction of the St Peter's chapel, leaned against the bell-tower, on its north-east side. The chapel's structure is nevertheless independent from both the bell-tower (the discontinuity joints are well defined) and the basilica, being simply adjacent to its masonry structures.

The bell-tower started showing the first signs of deterioration at the end of the XVI century: in a letter of 1592, the general father of the Franciscan order requested the Minor Friars of the Frari basilica to spend a bequest of 1000 "scudi" to execute some repair interventions on the bell-tower, due to its dangerous safety conditions.

Few notices, from documents of the archives, are related to events concerning the structure of the bell-tower, until the XIX century. Between the end of 19th century and the first decade of the 20th, three main strengthening interventions, widely documented by projects, surveys and site sketches, are carried out. In fact the subsidence of the bell-tower – in the first years of the 20th century, a differential settlement of about 0.30 m respect the basilica's structure and a out of plumb toward south-east of 0.765 m on a height of 42.5 m are reported – caused major damages both to the St Peter's chapel and to the vaults of the left aisle of the church, requiring necessary repair interventions on the masonry walls and at the level of the foundations.

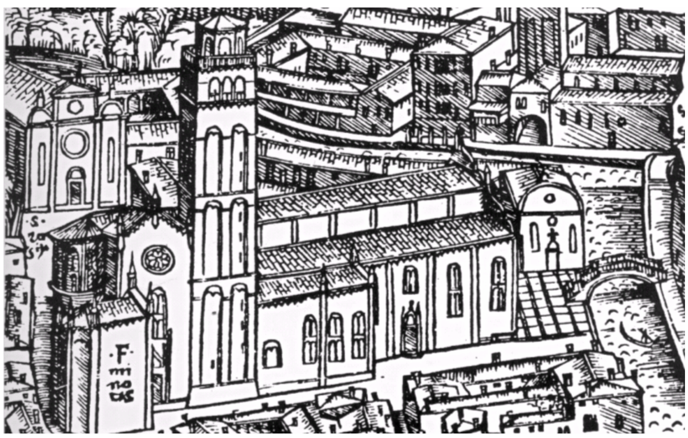

Figure 1. Frari basilica and bell-tower, detail from a view of Venice, J. de Barbari, 1500. On the right of the bell-tower, the St Peter's chapel.

Only subsequently to this repair works the structure of the bell-tower and the ones of the basilica and the St Peter's chapel are connected at their foundations, and in their upper structures by metal ties.

The first documented intervention on the structures of the bell-tower was carried out from 1862 to 1866. The initial layout substantially considers two types of activities: interventions on the bell-tower, concerning repair and maintenance works on the inner structure of the tower with mending on the external brick masonry layer, and interventions on the masonry walls of the St Peter's chapel next to the bell-tower. On a survey drawing of the St Peter's chapel South wall, dating back to 1862, the masonry wall adjacent to the bell-tower looks heavily damaged because of the subsidence of this last, in spite of the complete separation between the two structures. The intervention design includes therefore the demolition - for all its height and for a width of 2.70 m - of the damaged wall section, and its successive reconstruction.

In 1864, during the execution of the interventions, the original foundations of the external walls of the St Peter's chapel are brought to light. They were composed by big squared stone blocks, mainly disarranged for lack of mortar.

As an alternative to the construction of new foundations for the masonry wall to be rebuilt, it was decided to sustain this last with a solid relieving arch, made by three brick layers, spanning from the bell-tower substructure to the St Peter's chapel existing foundations. A first structural connection at the level of the foundations was hence provided, with subsequent changing in the mechanical behaviour of the complex bell-tower-St Peter's chapel.

The second repair intervention is realized between the years 1867 and 1873, starting with a general restoration of the roof of the church. Other urgent repair works are carried out, such as the disassembly and the following reconstruction of the stone arch of the left aisle adjacent to the bell-tower, and of the masonry vaults of the St. Peter's chapel.

The severe decay conditions of the left aisle arch stone ashlars, badly disarranged as consequence of the arch's deformation (due to the differential settlement of the supports), required urgent remedies (Figure 2). The disassembly and subsequent reconstruction of the arched lintel with repair of the damaged stone elements and the substitution of the ones beyond retrieval, and the insertion of a full thickness relieving arch in the masonry wall just above the damaged stone structure were carried out.

In correspondence of the most heavily damaged masonry wall parts, repair intervention consisting in partial substitution of brick units and positioning of stone ties (1.00 × 0.50 × 0.30 m), with subsequent plastering, were performed.

It is not possible to estimate, from written documents, which part of the envisaged repair works were carried to an end. Almost all of the stone ashlars of the arch were disassembled and substituted with new elements: it is quite simple to evaluate the different state of conservation and of finish between the first ashlars close by the capitals, likely dating back to the arch first construction, and the more recent ones, composed by well squared new elements.

The reconstructing of the stone ashlars was executed, according to the traditional technique, in a dry manner, meaning without mortar interposed in the joints, and with the positioning of a thin sheet of plumb between the two adjacent faces.

There are more uncertainties, on the other hand, about the insertion of the relieving arch on the upper masonry wall: on its opposite face respect the transept, at present without plaster, it is evident the presence of an arch inserted in the masonry. However, it seems that the relieving arch was built at the same time than the surrounding wall, and not inserted in a successive period; this fact is also supported by the observation of the shift between the keystone of the relieving arch and the one of the underlying stone arch.

The intervention on the stone arch was hence aimed at eliminating the effects of the structural decay, still not acting on the cause of the damage.

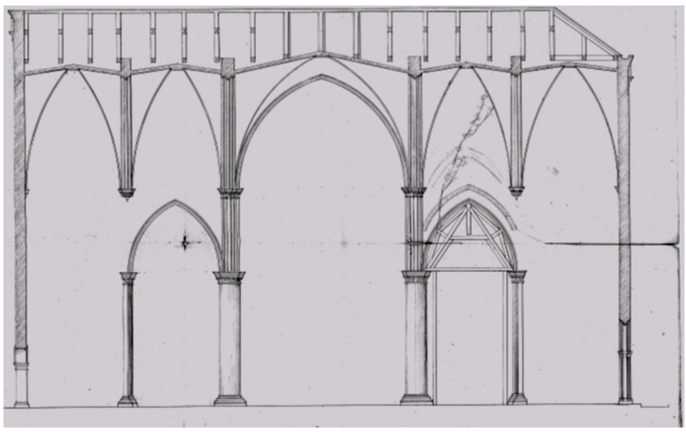

Figure 2. Crack pattern survey, 1867

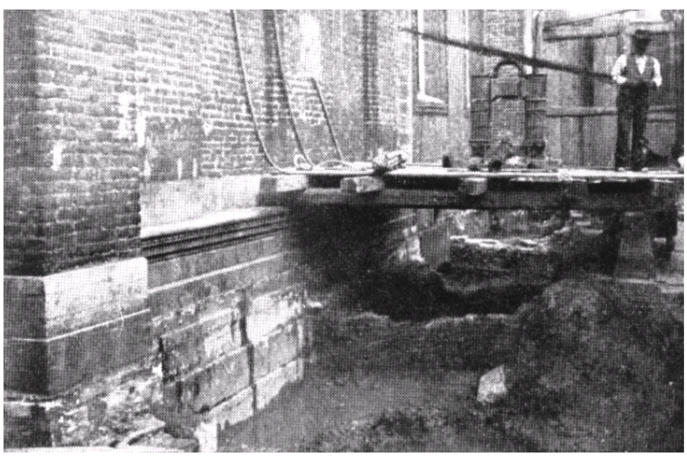

Figure 3. Strengthening intervention on the bell-tower foundations, 1904

During the first years of the XX century, in fact, some further repair works on the stone arch were executed; to restore the continuity between the once again disarranged ashlars, the joints were filled with cement grout and, in correspondence to the wider joints, some brass rods were positioned and surrounded by mortar.

The settlements of the bell-tower caused considerable damages to the masonry vaults of the contiguous St Peter's chapel; from a report of 1867, it emerges that the vaulted structures were in such bad conditions to make necessary their demolition and successive reconstruction with new ones, made with a plastered timber structure. During the execution of the works, such measures were not taken and, by noticing that the moulded masonry structures of the ribs could have been preserved, it was decided that only for the webs a reconstruction was necessary.

In 1902, subsequently to the collapse of the St Mark's basilica bell-tower, even the "Frari" bell-tower, together with many other Venetian structures considered at risk, is studied and monitored to evaluate its continuative and progressive settlement.

The surveys carried out at the level of the foundations revealed an inadequate base respect the bulk of the bell-tower, being this the main cause of the settlements of the tower. For this reason, a strengthening intervention on the bell-tower foundations takes place, consisting in the widening of the foundation base, starting with the south side (toward which the tower was leaning).

The project considered the traditional Venetian soil strengthening technique, with the insertion, close by the old bell-tower foundation raft, of timber piles (made by larch, length 3.80 m, transverse dimensions 0.20 × 0.20 m), covered by a 2.00 m wide concrete bed, parallel to the side of the bell-tower (Figure 3). All of the masonry walls went repointed with cement mortar that, in addition to restore the cohesion of the brick masonry, improved its strength.

During the same strengthening intervention, the excavation of the St Peter's chapel area adjacent to the structure of the bell-tower was also performed, reaching the foundations level. The substructures of the two adjacent buildings are joined by pouring concrete, with a previous roughening of the masonry surfaces. On the external walls, some works aimed at merging the masonry layers and to connect the two structures were performed. In addition to the connections at the foundations level, the structure of the bell-tower and of the side chapel were also linked at different heights on the perimeter walls.

In December 1902, some perforations were executed on the external walls of the bell-tower, with the purpose to position steel ties between the orthogonal walls. A plumb line was positioned below the floor of the belfry, with a total length of 42.50 m.

The strengthening intervention provided to the bell-tower foundations would have been extended also to the two internal sides; as a matter of fact, for emergency reasons that prioritized other interventions, the consolidation of the internal substructures was never realized. In the St Peter's chapel, the timber vaults erected in 1867 were demolished and substituted with vaults made with hollow bricks, 0.10 m thick, connected by using a lime-cement mortar and capped by a cement mortar layer (thickness 0.08 - 0.10 m) cast at the extrados of the vaults.

2 EXPERIMENTAL INVESTIGATIONS AND MONITORING

In 1990, subsequently to an extensive study concerning the behaviour of the Venice bell-towers, a diagnostic investigation on the Frari bell-tower started.

The investigations carried out in the following years included: fotogrammetric survey; geotechnical investigations on the foundation's soil; endoscopies, single and double flat-jack tests on the masonry elevation structures; sonic tests on steel ties.

Moreover, a monitoring of the main cracks, by means of extensometers, and the positioning of clinometers, for the detection of eventual rotations of the bell-tower, started.

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 0.8 m, was not showing anomalous displacements.

In September 2000, some worrying sign of structural deterioration appeared: new crack patterns, especially in the St Peter's chapel vaults, widening of already existing fissures (Figure 4a), falling of small portions of plaster and bricks from the vaults. An emergency intervention was provided to the structures more affected; in particular, lime mortar grout injections were provided to the cracked St Peter's chapel vaults. Moreover, the serious disconnectedness that affected the stone ashlars of the left aisle arch adjacent to the bell-tower structure, once again consequence of the arch's deformation, required the installation of a timber prop system (Figure 4b).

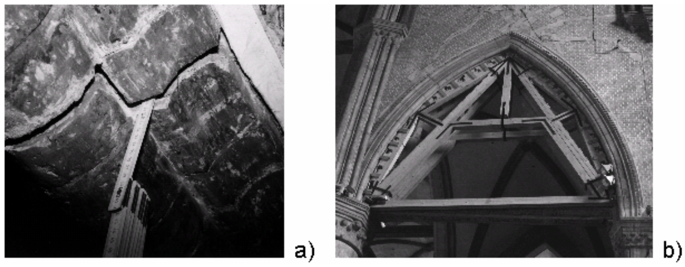

Figure 4. a) the remarkable widening of cracks in the St Peter's chapel vaults; b) the emergency centering consolidating the left aisle arch adjacent to the bell-tower.

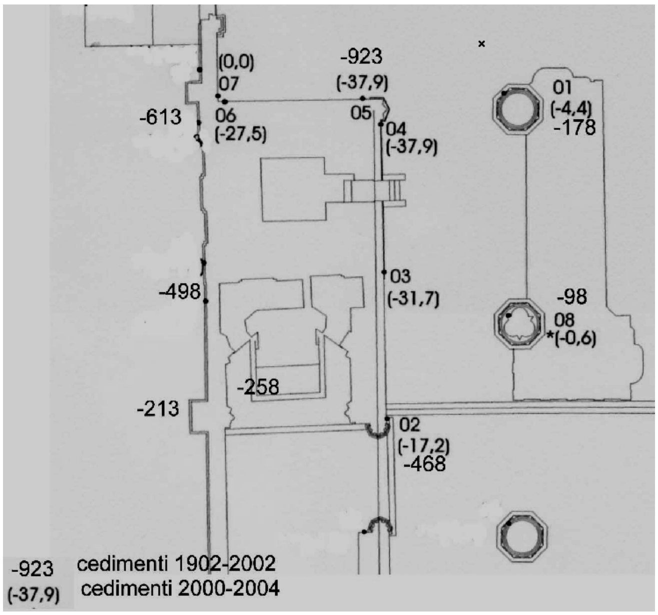

Figure 5. Differential settlements; comparison between the 1902-2002 and the 2000-2004 surveys (measures in 1/10 mm).

A survey aimed at detecting differential settlements in different points of the complex, by comparison with the one of 1902, revealed that the average subsidence of the structure of the church was included between -10 and -20 mm. The area close by the bell-tower and the adjacent masonry structures, shown, on the contrary, far greater differential settlements: -49.8 mm in the East corner, -61.3 and -92.3 mm in the South and West corners of the bell-tower base, respectively.

The differential settlements layout noticed, together with the comparison between the fotogrammetric survey of 1995 and a successive one of 2000, indicated that the bell-tower is tilting in the opposite direction respect the "historical" tendency, meaning that it is going back towards its vertical. Furthermore, a monthly survey carried out from 2000 to 2004 confirms the above mentioned displacement trend, recording an average settlement of 1 mm/year in the South-West corner of the tower respect the most "stable" point inside the basilica (Figure 5).

In 2001, in order to check the deformations of the bell-tower and of the adjacent structures of the basilica, an automatic monitoring system was installed, including the following instruments (Figure 6):

- 6 long base extensometers with invar wire kept in tension by a weight, installed in order to measure the relative displacements between the walls of the bell-tower and the adjacent structures;

- 8 crack-gauges installed on the main cracks of the South-West side of the bell-tower and of the wall above the stone arch.

The results of the monitoring indicate that the opening of the cracks is only partly caused by the settlement noticed at the foundations level.

The continuous movement of the bell-tower, the presence of crack patterns with fissures even 15 mm wide, suggested a deeper level of analysis by means of further investigation on the structures.

Between June and September 2003, two investigation campaigns were performed.

Flat-jack testing technique was used to measure the existing state of stress on the masonry structures of the bell-tower and of the adjacent structures of the Basilica. At the base of the bell-tower an average value of 1.92 MPa was measured on the external side, while on the inner part a mean value of 1.44 MPa was estimated (Figure 7).

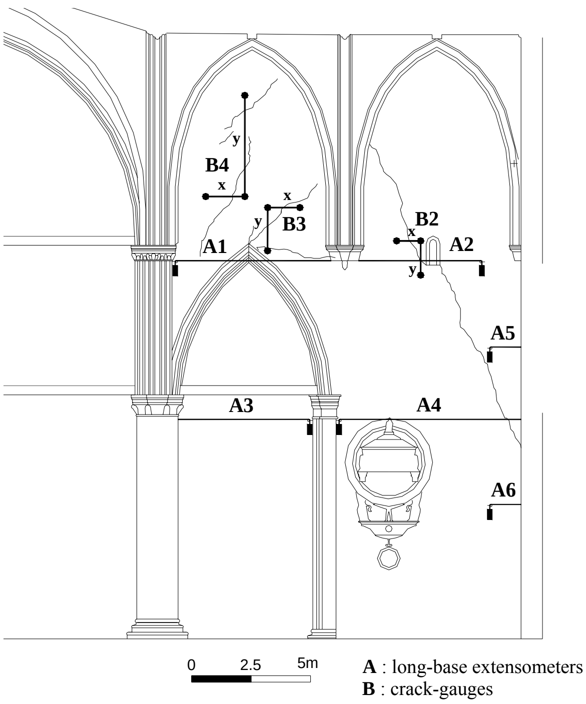

Figure 6: Structural monitoring system - scheme of instruments installation - 2001.

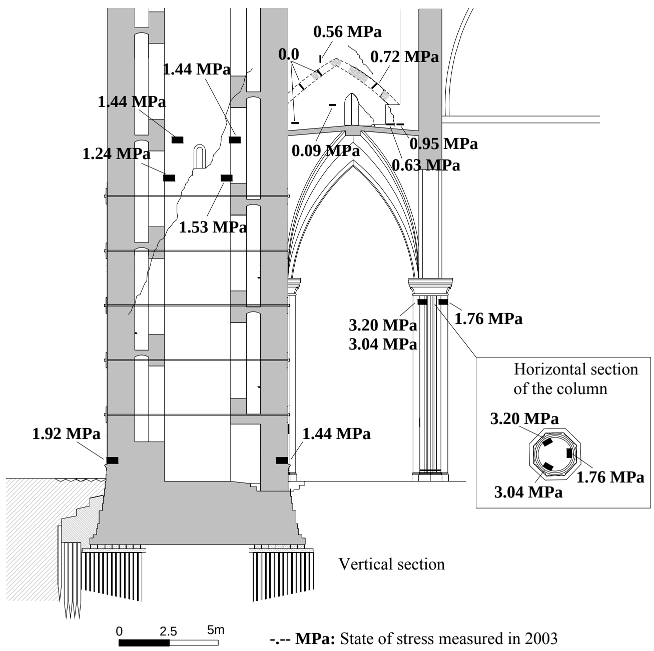

Figure 7. Measure of the existing state of stress on the masonry structures of the bell-tower and of the adjacent structures of the basilica by flat-jack technique (2003).

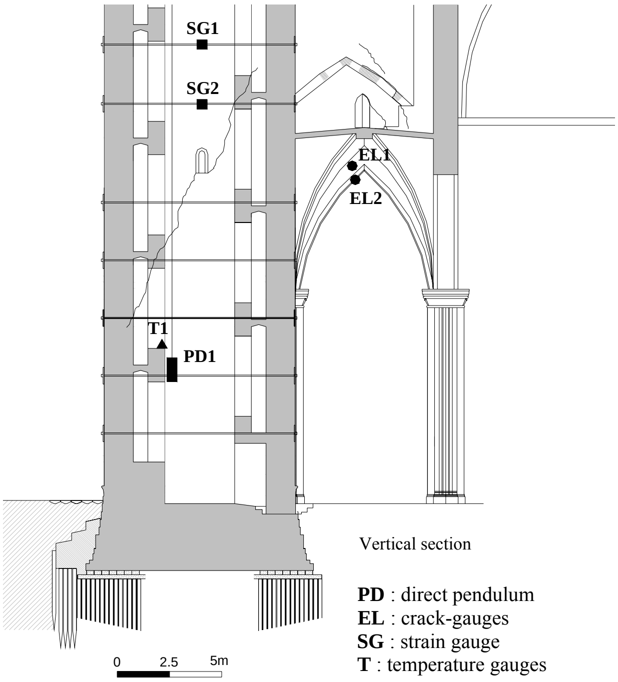

Figure 8: Scheme of installation of the instruments of the automatic monitoring system - 2003.

Inside the bell- tower, flat-jack tests were carried out on both sides of an inclined oblique crack which runs along the South-West side of the tower. The similar stress values measured, at two different levels (Figure 7), clearly show that the crack, even if passing through the entire thickness of the wall, does not induce particular stress concentration on the structure. Very high values of compressive stress were measured at the top of the column sustaining the propped arch: 1.76 MPa at the external side and 3.20 and 3.04 MPa on the inner side (Figure 7).

A detailed analysis was also carried out on the wall over the propped vault. Several tests were carried out, and the results indicate the presence of a thrust line going from the bell-tower to the structures of the basilica, in correspondence of the upper area of the above-mentioned column. In fact, horizontal and vertical flat jack tests show states of stress ranging from 0.56 MPa to 0.95 MPa.

In 2003 the automatic monitoring system was also enlarged by the installation of the following instruments (Figure 8):

- direct pendulum equipped with automatic telecoordinometer, for the measure of the absolute horizontal movements of the top of the tower;
- 2 crack-gauges installed on the cracks of the propped stone masonry arch;
- 2 strain-gauges in order to measure the deformation of the steel cable (see par. 3.3) installed in the bell-tower;
- a thermal-gauge to measure the temperature of the internal and external air, and also inside the masonry at different depths from the outer wall.

All the instruments are connected to an automatic data acquisition and recording system which is equipped with a modem to transfer the data to a remote controller.

3 STRUCTURAL MODELLING

A structural analysis of the bell-tower and the adjacent parts of basilica was performed. The aim of the modelling was the identification, through the historical process that led to the actual situation, of the masonry structures' present behaviour.

The portion of structure modelled, chosen after subsequent simplifications, includes the bell-tower and the adjoining parts of the church that were mostly affected by the interaction with the tower.

Some "historical" models were implemented in order to assess, at different times, the static of the structures. The final configuration of each analysis, considering the differential settlements of the bell-tower in the lapse of time gone by, was used as starting model geometry for the successive calculation.

The results of the latest model, representing the present situation, are finally discussed and compared with the experimental outcomes.

3.1 *The models*

For all the numerical simulations presented hereafter, the only load condition considered is the self weight. The load corresponding to some parts of the real structure not modelled (timber structure roof of the basilica, belfry), was imposed as external forces. The crossed vaults' filling was included as surface load.

The mechanical properties chosen to describe the materials arise from the results of previous tests performed on the masonry structures. In particular, the value of the elastic modulus (3300 MPa, average of the results of double flat jack tests performed on the bell-tower masonry) was given to all of the sets of materials. The same density (2000 kg/m^3) was also assumed for all the materials.

The material is considered homogeneous and isotropic, and the analyses performed are linear elastic.

This is a really strong assumption, especially when dealing with differential settlements in the range of half a meter. Both a non linear behaviour in terms of material and geometry should have been considered, see Lourenço (2001).

In spite of this, an iterative process aimed at reducing the inconsistencies arising from the linear elastic behaviour was carried out. In each model, after running the analysis, a higher deformability was assigned to the elements subjected to an excessive tension respect the assumed strength of the material, in the successive analysis.

This (costly) strategy, repeating the analysis as long as necessary to reach a final stress distribution consistent with the prefixed tensile limit, was however

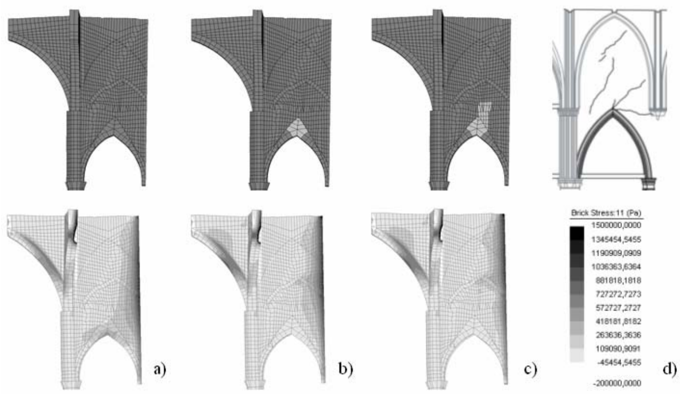

Figure 9. Successive weakening of the tensed elements; from (a), where all the material has the same properties through (b) and (c) it is possible to reach a final "acceptable" configuration. The tensile stress progressively decreases, and it is possible to notice some peaks (a) where a remarkable crack originates (d).

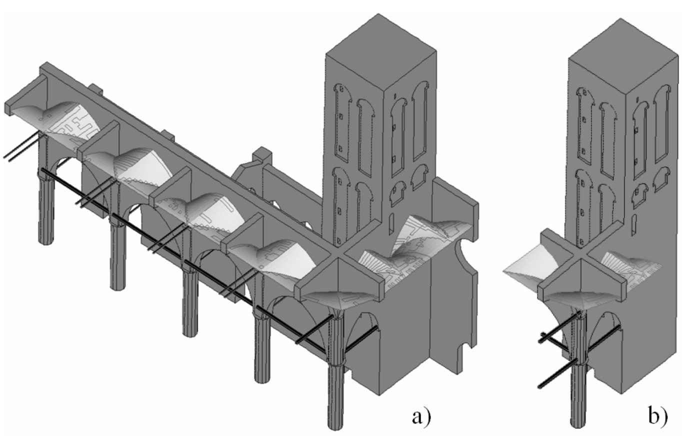

Figure 10. a) "Global" model including the bell-tower, the St Peter's chapel, the left aisle, half the main nave and half the transept with its façade; b) "simplified" model including the bell-tower, the first transversal arch with its supporting column and the four quarter-vaults above this last.

corroborated by the fact that a tensile stress concentration appeared in the model where a real crack pattern is evident.

Weakening the elements subject to high tension, a smeared crack pattern was assumed, and the propagation of some principal cracks was followed by the subsequent iterative process (Figure 9), see Rots & Invernizzi (2001)..

3.2 *Results of the modelling*

A FE model, considering a wide portion of church, the bell-tower and the St. Peter's chapel was first implemented (Figure 10a). The "simplified" model, used for the calculations, includes 32,760 brick, 1400 shell plate and 360 beam elements (Figure 10b).

The bricks were used for the brick masonry, the plate elements for the vaults, while the beams reproduce the ties. The average dimension of the solid elements is 0.4 m in the bell-tower, 0.5 m for the plate element of the vaults.

Two previous phases were analyzed before implementing the latest model: after the construction (1450) and before the strengthening intervention on the bell-tower (1903). Each of these models shows the geometry corresponding to the respective year (by means of imposed rotations and translations at the base of the bell-tower), and the softening of the elements damaged in the previous stage. Each model was then calibrated with the available experimental data (historical drawings, surveys, monitoring and on site tests).

The vertical displacement imposed to the bell-tower in the period 1450-1903, respect the basilica's structures was 0.3 m. The tilting was simulated with a rigid rotation at the base toward outside (South-East) of an angle of about 0.9°, determining an out of plumb of 0.765 m at the height of 42.5 m.

The following model (1903-today) reflects the tendency of the XX century. An "inverted" rotation of 0.06° was imposed to the bell-tower, with an average settlement of 84 mm. Moreover, taking into account the geometric survey of 1990, a rotation of 0.1° toward the transept (South-West) was also considered.

Some results can finally be drawn: 1) preferential channels of compressive stress are localized inside the masonry wall above the propped arch (Figure 11a);

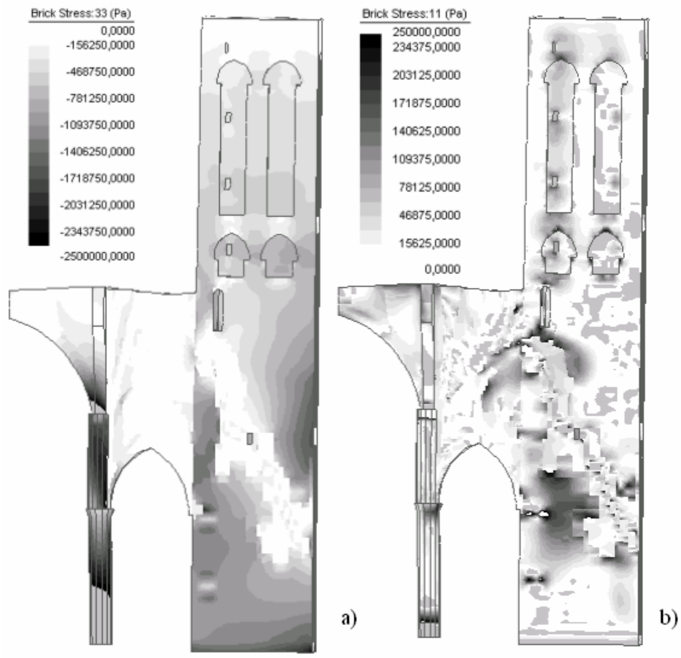

Figure 11. "late" model results: a) principal compressive stresses; b) principal tensile stresses. It is possible to appreciate the "smeared" approximation of the main fissure on the bell-tower's pipe and the crack pattern on the wall above the transverse arch. It is also possible to notice the deform shape of this last, because of differential settlements, and a slight curvature of the pillar toward the main nave.

2) the high tensile stresses found in the same masonry wall, due to the settlements of the bell-tower, reduced by weakening the stiffness of the material, are the cause of the wide crack patterns and may also represent the loss of shape of the stone arch (Figure 11b); 3) the high and uneven stress, found just below the capital of the column, can be ascribed to the horizontal thrust determined by the movements of the bell-tower; 4) a certain amount of tensile stress was found in correspondence of the bell-tower window opening on the transept, presumably indicating the presence of the main fissure on the external pipe of the bell-tower.

3.3 *Final remarks*

The limits of the linear elastic analysis carried out are significant. However the historical process methodology seems to be a decisive issue in the comprehension of phenomena affecting the structure. The introduction in the analysis of phenomena not mentioned here, such as, for example, the complete loss of resistance of the early timber ties, the repair intervention substituting the damaged materials with new ones, in the deform shape of that specific period, the insertion of new steel ties at different stages, addressed the analysis to different results.

More accurate results can be found by applying the same modelling concept with more suitable solution strategies, first of all the material non linearity.

A provisional steel cable, aimed at taking part of possible extra-horizontal thrust acting on the column, was positioned, connecting the stone ashlars just above the capital of the pillar to the bell-tower structure at a height of 14.40 m (Figure 12). The steel tie is also used as a monitoring device: two strain gauges are applied to the coupling sleeve, recording each loss or increase of tension in the cable.

4 GEOTECHNICAL ANALYSIS

A detailed analysis of the subsoil stratigraphy and geotechnical properties together with the exploration of the exact geometry and typology of the foundation block was primarily achieved and the definition of an accurate geotechnical model of the foundation finally completed.

The only previously available data on the subsoil conditions dated back to 1991, consisting of two boreholes carried out inside the bell-tower to the depth of 18 metres.

A new extensive geotechnical investigation campaign was carried out in May 2003, including (Figure 13):

- 2 continuous vertical boreholes: Sv1 in the area outside the church, to a depth of 25.50 m; Sv2 inside the bell-tower, to a depth of 21.50 m;
- 5 inclined boreholes, labelled from Si3 to Si7 in Figure 13, where the drilling depth and inclination are respectively reported in brackets;
- 4 continuous borings into the foundation block. EI1 and EI2 were inclined of 30°, drilled for a length of 4 m and located inside the basilica; SVE1 and SVE2 were short vertical borings, carried out on the NE side of the bell-tower, on the back of St Peter chapel;
- 4 static penetrometer tests with monitoring of pore water pressure (piezocone tests), labelled from CPTU1 to CPTU4 and pushed to variable depths between 17.00 m and 19.20 m;
- Standard Penetration Tests (SPT), in boreholes;
- Extraction of several soil and foundation samples, most of which undisturbed, for the subsequent execution of the laboratory tests, aimed at the evaluation of physical and mechanical properties.

In the end, all borings were carefully filled with cement grout. Boreholes Sv1 and Sv2 were also supplied with two piezometers for the water level measurement.

4.1 *Stratigraphic section*

Data interpretation enabled to draw a rather detailed geotechnical characterisation of the soil layers beneath the Frari bell-tower, all belonging to the recent Quaternary deposits typical of the Venetian lagoon (Cola & Simonini, 2002).

The following stratigraphic units were defined:

- Unit A, from ground level to -3.0 m: anthropic fill, made of loose sand in silty matrix, with frequent inclusions;
- Unit B, from -3.0 m to about -6.7 m: dark grey, silty clay, from very soft to soft, with occasional organic material, normally consolidated or slightly overconsolidated. Between -4.5 m and -5.5 m, it is noticeable the presence of several few-centimetres-size shells. Towards the bottom end of the unit, the silty clays turns in yellowish silty sand, probably

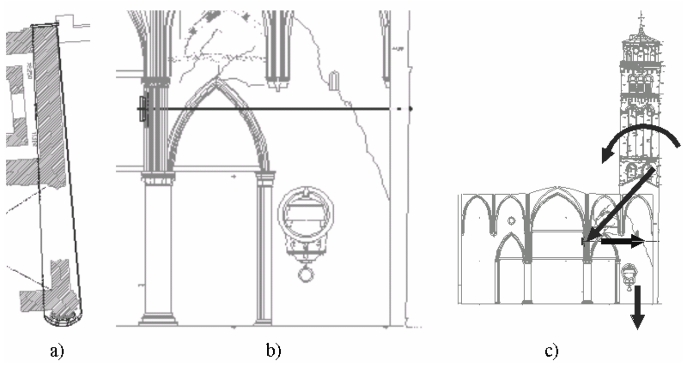

Figure 12. a) Plan view and b) elevation of the provisional steel cable; c) possible evolution: rotation of the bell-tower; transmission of thrust to the top of the column; the steel cable is brought into play.

oxidised and slightly over consolidated for desiccation;

- Unit C, from about -6.7 m to about -14.0 m, -14.5 m: grey medium-fine sand, non plastic, from dense to very dense.
- Units D, E and F: alternation of soft clayey silt and medium-fine dense sand.

Figure 14 summarizes the main geotechnical properties of each stratigraphic unit, as deduced from laboratory and in situ tests. In particular, the following elastic and shear strength parameters in drained conditions were eventually assumed – for units B and C - in settlement and stability calculations: for Unit B: E' = 3500-5000 kPa, c' ≈ 0 kPa, φ' = 28°; for Unit C: E' = 35,000-40,000 kPa, c' = 0 kPa, φ' = 35°.

Figure 15 shows a section of the foundation subsoil, drawn along the SE-NW direction. The relevant vertical and inclined boreholes, together with CPTU tests, are superimposed to the above mentioned stratigraphic characterisation.

The typology and geometry of the foundation block is also shown, according to previously available data and to the outcome of the recent site investigation. The foundation is made up of limestone (pietra d'Istria) squared blocks, about 0.20-0.40 m thick, in a remarkable state of preservation. Among blocks the original mortar was also locally found.

Below foundation blocks there is a wooden floor of squared larch boards, 0.40-0.50 m thick. Under the boarding, a thin layer of clean sand was found, probably used to prepare a uniform bedding plane for the foundation. Further below the typical wooden piles layer is made up of short (from 1.5 m to 2.0 m) consolidation piles, pushed into the fully saturated, soft clay, very close each other, often side by side, to make it denser and stronger. Further accurate tests on the various materials of the foundation block are currently in progress.

The foundation is almost square, with sides 10.8-11.0 m long. However, inside the bell tower the pad foundation is not continuous, but there is a 2 × 2 m square area with no foundation elements. The present site investigation campaign also provided confirmation of the 1902 intervention, aimed at the foundation enlargement on the external side of the bell-tower.

4.2 *Origin of continuous settlements*

The slow, but constant rate, continuous differential settlement of the bell-tower, described in paragraph 2, is cause of major concern for the present and future stability, not only of the bell-tower but also of the structurally connected basilica. However, simple calculations based on the geotechnical model briefly described seem to exclude that the movements can be entirely ascribed to secondary settlements (Butterfield et al., 2003). The viscous response of units B and C can account for only the 20-30% of the current settlements and should show a decreasing rate with time. Some progressive failure of the soft silty clay layer, squeezed between the piles end and the unit C, must therefore be taken into account. A thorough understanding of settlement causes is clearly crucial for the subsequent intervention design.

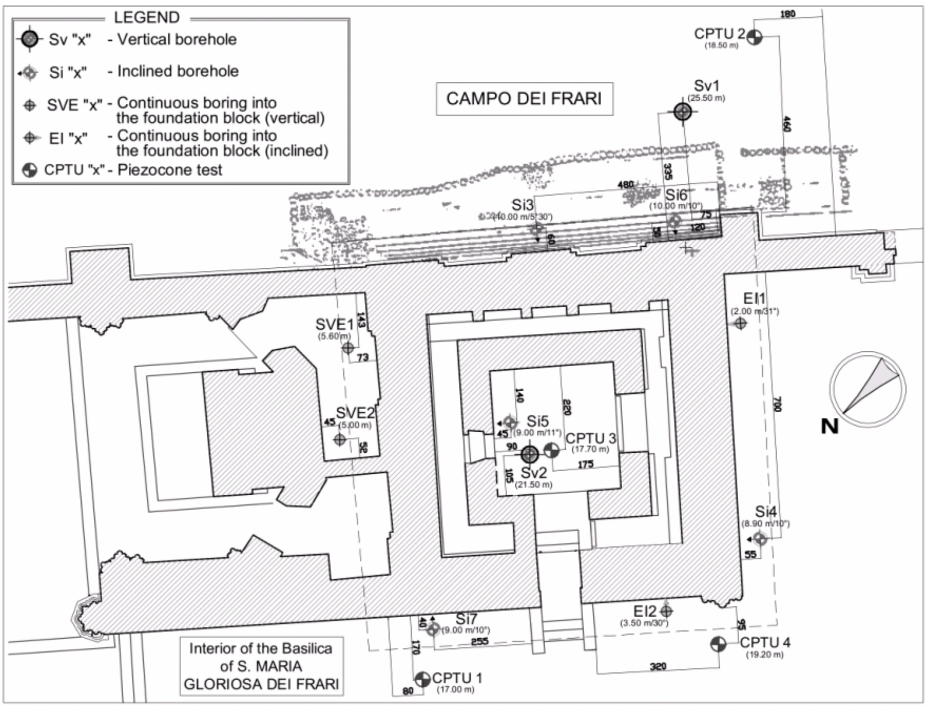

Figure 13. Geotechnical investigation programme

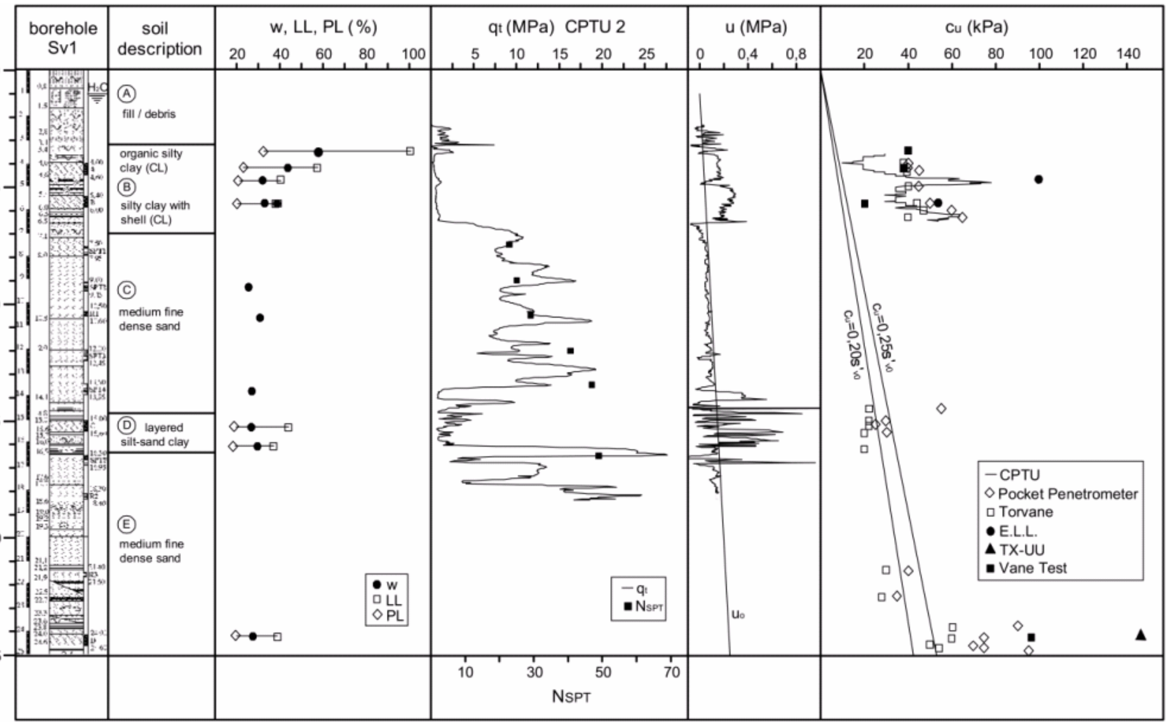

Figure 14. Soil profile

Numerical modelling of such rather complex situation, aimed at the quantitative evaluation of each single cause, is currently in progress

5 CONCLUSIONS

From the results of the experimental and numerical simulation it emerges that the bell-tower/basilica interacting structures can not bear, without serious consequences, further differential settlements. It follows the need of an intervention, at the level of the foundations, aimed at reducing the settlements of the bell-tower; after the strengthening intervention, the entity of vertical displacements of the complex basilica/bell-tower should be in the same range.

The principles on which the strengthening intervention should be based are: 1) avoiding to alter the actual foundation arrangement, in order to preserve the existing substructure; 2) avoiding to propose a rigid foundation system for the bell-tower; 3) operate following the principles of the "observational method". The previous principles derive from some general considerations: the existing foundation system should not be included in the strengthening intervention, in order to safeguard the historical foundation and to avoid substantial stress variations at the level of the substructures; the bell-tower foundations must not be rigid, in order to avert the inverse problem, meaning a higher settlement for the basilica's structures with respect to the bell-tower; the "observational method" is based on the principle that it is possible to learn from monitoring of the operations carried out until that moment, and so optimize and modulate the successive interventions phases.

An approach aimed at improving the mechanical characteristics of the foundation soil can fulfil all such prescriptions and was therefore taken into consideration. It could be pursued through grout injection, via soil fracturing; the fine grained nature of unit B, in fact, does not enable permeation grouting.

The soil fracturing technique consists in installing special injection tubes in the foundation soil, with valves at different depths. The careful and slow-rate injections of suitable cement and bentonite mixtures can be repeated at successive stages, to obtain progressive increments in terms of mechanical characteristics, according to the above-mentioned "observational method".

The final outcome should be a reinforced soil, made up of the original material and an indented web of thin layers of injected grout.

The soil would be eventually confined by such thin layers and subjected to a much increased stress state. An increment in terms of deformation modulus of the consolidated soil (with subsequent creep reduction) and an improvement of the soil shear resistance should finally be obtained. With the aim of validating and calibrating this rather innovative strengthening method, generally used as compensation grouting but here applied in difficult conditions to a very soft, fully saturated fine-grained material, a full-scale test site was arranged on the Northern corner of the bell-tower, inside the basilica.

Figure 16 shows the relevant lay-out, placed according to an irregular triangular mesh for local obstacles. A total of seven injection tubes, 12.00 m long, 12.5 mm thick and with a diameter of 88.9 mm, were installed. Several mixture compositions were used, injected from bottom to top at 0.50 m spacing. Grout flow was between 1 and 6 l/min, with maximum injection pressures of 5 bars in clay and 15-25 bars in sand. Vertical and horizontal displacements of the surrounding soil were monitored, as well as the possible pore pressure build-up, which fortunately turned out to be rather limited. In the end, specific additional boreholes and piezocone tests were also carried out.

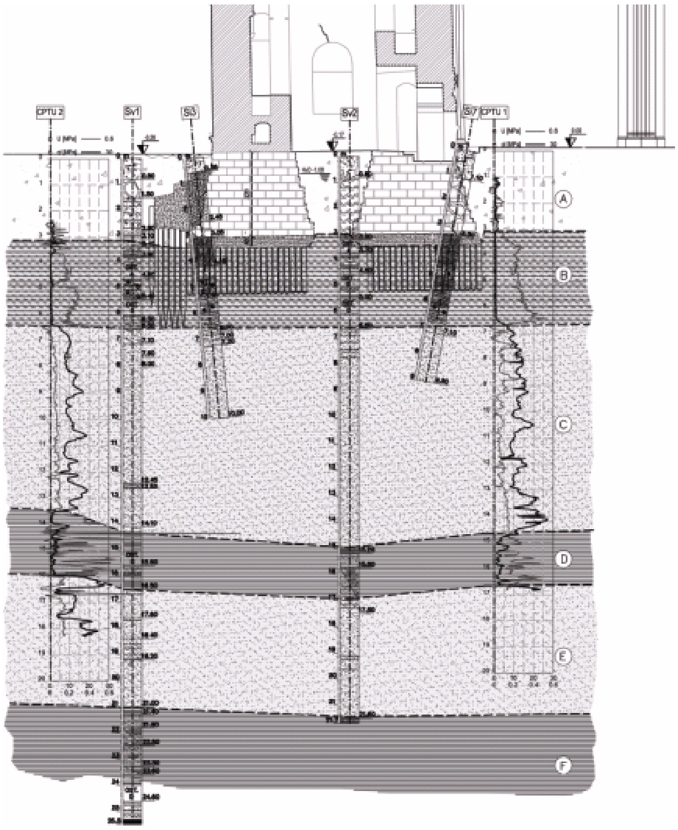

Figure 15. Foundation subsoil stratigraphy

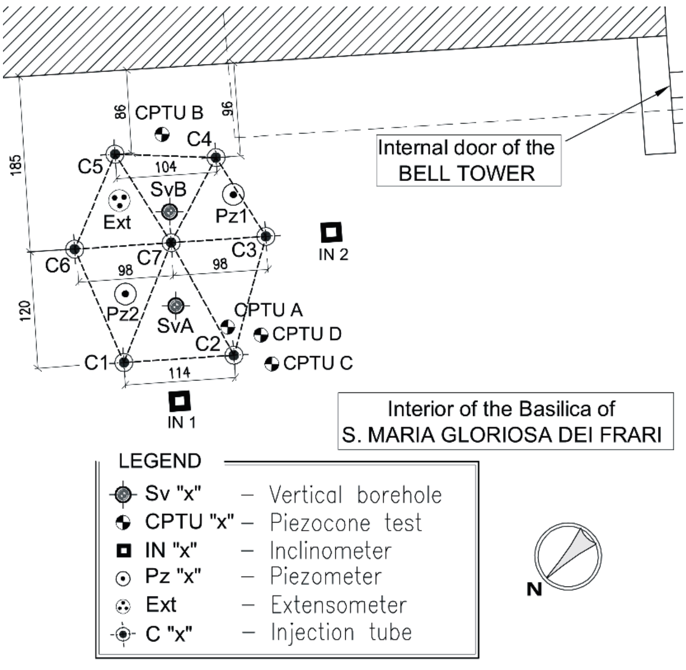

Figure 16. Soil fracturing full-scale test site

The importance of the drilling technique was especially pointed out, in order to prevent any consequence of local stress release, as well as the need of injecting rather elevated grout volumes in the cohesive soft soil, in order to achieve significant results.

The outcome of such very helpful test site, still under consideration, will enable to calibrate the possible following intervention, in terms of suitable mixtures and injected volume and pressures.

REFERENCES

Butterfield, R., Gottardi, G., Simonini, P. & Cola, S. 2003. A new interpretation of the compressibility of venetian silty-clay soil. *Proceedings of the British Geotechnical Association International Conference on Foundations, ICOF 2003, Dundee, Scotland, UK, 2-5 Sept. 2003.* T.A. Newson (ed.), pp. 149-159.

Cola S. & Simonini P. 2002. Mechanical behaviour of silty soil of the Venice lagoon as a function of their grading characteristics. *Canadian Geotechnical Journal, Vol.39, pp. 879-893.*

Gatti, P., Gavarini, C., Piana, M., Piombetti, A. 1992. Study on the dynamic behaviour of the Venice bell-towers, *Proc. 3rd International Conference on NDT, Viterbo, October 1992.*

Gobbetto, Frezza, Fullin, 1992. Campanile della Basilica dei Frari – Relazione statica riassuntiva.

Lionello, A., Cavaggioni, I. Il monitoraggio del Campanile di S. Maria dei Frari a Venezia tra conoscenza e conservazione. 'ΑΝΑΓΚΗ 33'- Milano.

Lourenço, P.B. 2001. Analysis of historical constructions: from thrust-lines to advanced simulations. *Proc. III Int. Seminar on Structural Analysis of Historical Construction, Guimarães, Portugal.*

Rossi, P.P. 1997. The importance of monitoring for structural analysis of monumental buildings. *Keynote lecture International Colloquium IABSE, Bergamo.*

Rossi, P.P. 1995. Possibilities of the experimental techniques for the structural analysis of historical constructions. *Proc. Int. Seminar on Structural Analysis of Historical Construction, Barcelona, Spain, Nov. 1995*

Rots, J.G. & Invernizzi, S. 2003. Regularized saw-tooth softening. *Computational Modelling for Concrete structures, Eds. N. Nicanic, R. de Borst, H. Mang, G. Meschke, Proc. EURO-C 2003, Balkema, Lisse, pp. 599-617.*

Rots, J.G. 1997. *Structural Masonry – An experimental/numerical basis for practical design rules.* Rotterdam: Balkema.

Scolari, A. 1920. La Chiesa di S.ta Maria Gloriosa dei Frari ed il suo recente restauro, *Studi di arte e storia, a cura della Direzione del Museo Civico "Correr", Vol.1°, 1920.* State Archive of Venice, St Maria Gloriosa dei Frari, catastico, B.n. 94, c. 51.

TECNITER, 1991. Report on survey and investigations on the structure of the Frari bell-tower. Milano.

TECNITER,1992. Study on the dynamic behaviour of the structure of the Frari bell-tower. Milano.

The St Maria Gloriosa dei Frari bell-tower, *the historic Archive of the Landscape and Architectural Heritage Office, Venice*

The St Maria Gloriosa dei Frari bell-tower, *the State Archive, Venice.*

Zanforlini, M. *Documentary research about St Maria Gloriosa dei Frari on behalf of the Landscape and Architectural Heritage Office, Venice.*