Monitoring-driven design of a multiphase intervention for the preservation of the Frari bell Tower in Venice

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Summary

Since its construction in 14th century, the Frari bell tower has been affected by a slow but constant differential settlement with respect to the adjacent masonry structures of the Basilica. About 15 years ago, the clearly evident signs of structure instabilities provided the impetus for the implementation of modern remedial measures, first in foundation and then on the elevation structure. A ground improvement intervention using soil fracturing (also known as fracture grouting) was carried out in order to improve the mechanical characteristics of the clayey layer underlying the tower. Once the aim of improving the stability of the soil-foundation system had been achieved, a new solution was required to reduce the damaging interaction between the masonry structures, activated by the foundation settlements. A structural joint between the bell tower and the Basilica was finally executed in order to accommodate the system deformability.

The paper presents a well-documented case study, since the preliminary crucial and accurate site investigation. Then it highlights both the approach aimed at improving the overall safety - without altering the original structure and substantially modifying the current stress distribution - and the innovative methodology, adopted throughout, of a gradual and modular design, constantly driven by the outcome of an extensive real-time monitoring system of the soil-structure interaction.

Keywords: Fracture grouting/Soil fracturing; Venice; Bell tower; Monitoring; Soil-structure interaction.

1. The frari bell tower: a history of interventions

The Basilica of *Santa Maria Gloriosa dei Frari* is one of the largest and most splendid churches in Venice. It stands on the *Campo dei Frari* at the heart of the city (Fig. 1). Historical archives tell us that the Franciscans were initially granted land to build a church in 1250, but the first building was not completed until 1338. Works almost immediately (in 1340) started again on its much larger replacement, the current church, which took over a century to be built.

The Frari bell tower (Fig. 1), the second tallest in the city after that of San Marco, was built between 1361 and 1396. The bell tower structure is 9.5 m wide at the ground level, 65 m tall and weighs about 57 MN. The internal ramp staircase up to the belfry is supported by a double structure of thick brick masonry. It was originally conceived as a fully independent structure, but during the reconstruction of the Basilica, the bell tower was included into the masonry walls, at the South-East corner of transept and left aisle (see plan of Fig. 1). The connection between the bell tower and the Basilica is at the origin of the subsequent problems that have always affected their structures. In 1432 the St Peter's chapel was constructed adjacent to the Basilica and the bell tower, even if structurally independent.

The first documented signs of deterioration of the structures are dated back to the end of 16^{th} century. Between the end of 19^{th} century and the first decade of 20^{th}, three main strengthening interventions, widely documented by projects, surveys and site sketches, were carried out (Lionello, 2008). In fact, the bell tower differential settlement had caused, along the centuries, major damages both to the St Peter's chapel and to the vaults of the left aisle of the church, requiring urgent repair interventions on the masonry walls and at the level of the foundations. In the first years of the 20^{th} century the tower showed a differential settlement of already about 0.40 m with respect to the Basilica and an out of plumb toward south-east of 0.765 m at a height of 42.5 m (resulting in an inclination of about 1°). Only after these repair works the structures of the bell tower and the St Peter's chapel were connected each other at the foundation level, with a solid relieving arch made by three brick layers, and at the superstructure level, by metal ties.

In 1904, following the universal concern after the sudden collapse of the worldwide famous St Mark bell tower, an extensive investigation was carried out on many Venetian slender structures considered at stake. In the Frari case, the surveys revealed inadequate foundations with respect to

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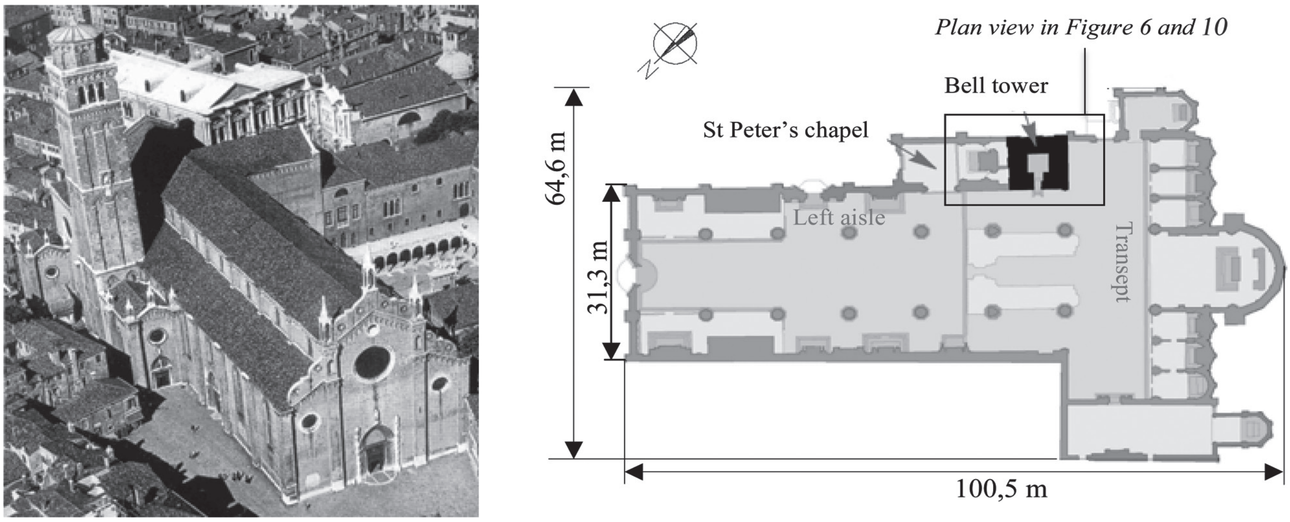


Fig. 1 – Left: a view of the Frari Square (*Campo dei Frari*) and its monuments: the bell tower, the basilica and the cloister. Right: a plan of the basilica with the bell tower and the St Peter's chapel.
Fig. 1 – A sinistra: una vista del Campo dei Frari e dei suoi monumenti: il campanile, la basilica ed il chiostro. A destra: pianta della basilica con il campanile e la cappella di San Pietro.

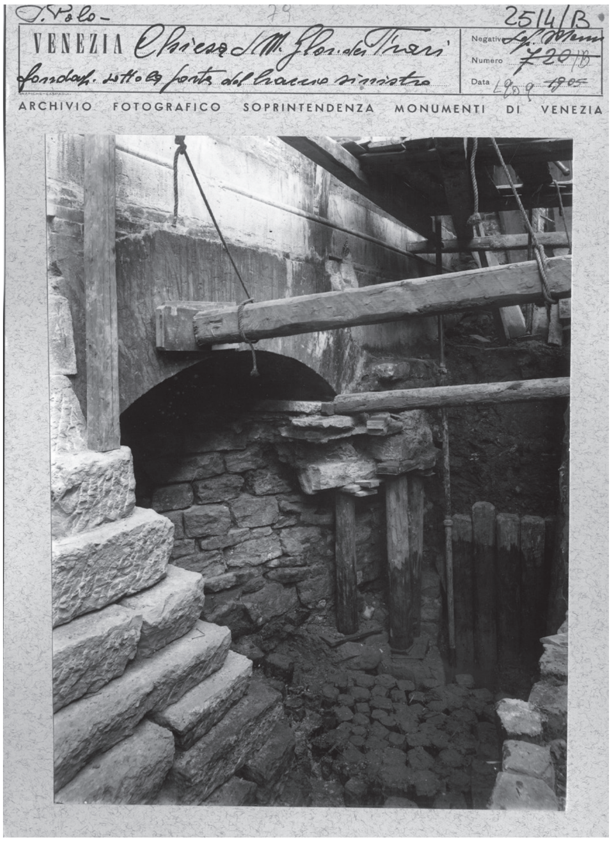


Fig. 2 – Picture of the strengthening intervention on the bell tower foundations, dated back 1904 (Photo archive of the "*Soprintendenza per i beni architettonici e paesaggistici di Venezia e laguna*").
Fig. 2 – Foto d'archivio dell'intervento di consolidamento della fondazione del campanile eseguito nel 1904 (Archivio fotografico della "Soprintendenza per i Beni Architettonici e paesaggistici di Venezia e laguna").

the bulk of the bell tower, this being therefore the main cause of the tower settlements. For this reason, a strengthening intervention on the bell tower raft foundations was carried out, consisting of widening its base, starting from the south side (toward which the tower was essentially leaning). At the time, the intervention was designed according to the traditional Venetian soil strengthening technique (Marchi *et al.*, 2006), with the insertion of closely spaced timber piles. The piles, 3.80 m long, 0.20 × 0.20 m of transverse dimensions and essentially touching each other, were made from larch and covered by a 2.00 m wide concrete bed, parallel to the side of the bell tower (Figs. 2 and 3). All the masonry walls were also treated with cement mortar to restore the cohesion of the brick masonry and improve its strength. Such raft foundation enlargement had been most probably designed as to be extended to the other sides; as a matter of fact, however, for emergency reasons that gave priority to other interventions, it was never completed (Lionello *et al.*, 2007).

2. Investigations and monitoring before the intervention

An extensive survey of the general safety conditions of Venice bell towers was planned and implemented at the beginning of the 1990's by the local *Architectural Heritage Office* of the *Italian Ministry for Cultural Heritage and Activities*. Afterwards, a rather detailed diagnostic investigation of the Frari bell tower started, including photogrammetric survey, crack-pattern survey carried out with the aid of

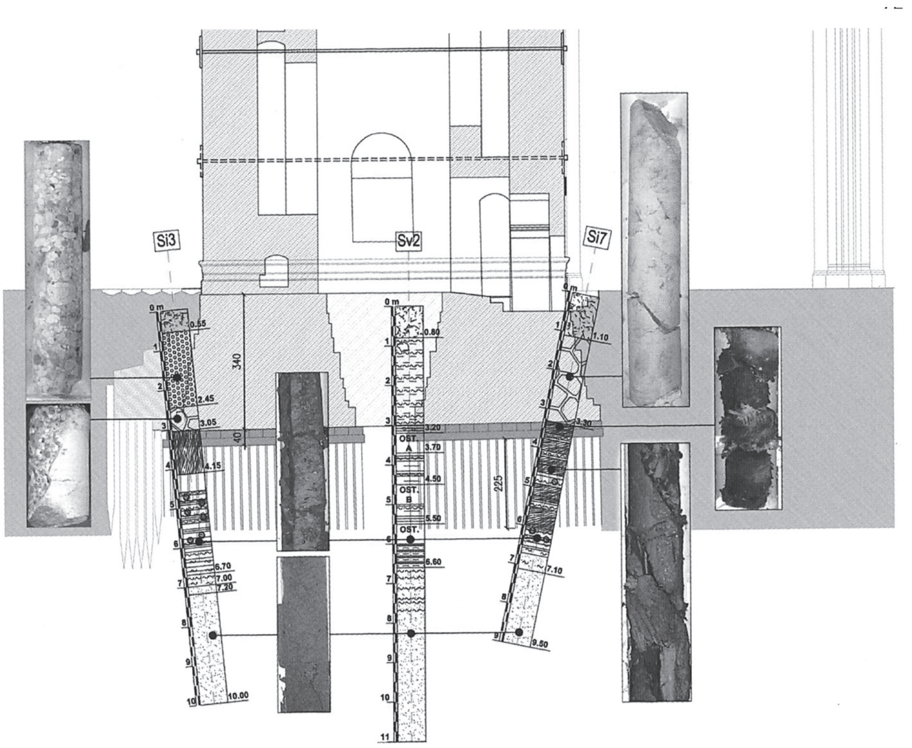


Fig. 3 – Schematic drawing of the foundation with the foundation enlargement of the 1904 on the left and pictures of samples extracted during the recent survey (Lionello, 2008).
Fig. 3 – Sezione schematica della fondazione del campanile con l'allargamento eseguito durante il consolidamento del 1904 ed immagini dei campioni estratti durante i recenti carotaggi (Lionello, 2008).

climbing technicians, endoscopies, single and double flat-jack tests on the masonry walls, sonic tests on steel ties, monitoring of the main cracks by means of extensometers and installation of clinometers for the detection of rotations of the bell tower, as well as preliminary geotechnical investigations of the foundation soil. In 2000, a further survey aimed at detecting the differential settlements of the bell tower, by comparison with 1902 situation, revealed settlements as large as 49.8 mm in the east corner, 61.3 mm in the south corner and 92.3 mm in the west corner of the bell tower base (LIONELLO *et al.*, 2004). These findings, together with the results of the photogrammetric survey of the 1990's, indicated that the bell tower was tilting in the opposite direction with respect to the "historical" trend, essentially moving back toward its vertical position (Fig. 4). This was most probably the consequence of the 1904 foundation widening.

Emergency interventions were provided to the structures more affected by the deformation processes. In particular, the installation of a timber prop system was required to support the stone arch that connects the bell tower to the adjacent pillar of the Basilica (Fig. 4). A monthly survey carried out from 2000 to 2004 confirmed the displacement trend, recording an average settlement of about 1 mm/year in the west corner of the tower.

2.1. Measurement of the state of stress

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 Basilica. At the base of the bell tower an average value of 1.92 MPa was measured on the outer side, while on the inner part a mean value of 1.44 MPa was estimated (Fig. 5). 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, 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 on the outer side and 3.20 and 3.04 MPa on the inner side (Fig. 5). A detailed analysis was also carried out on the wall above the propped vault and the results indicated the presence of a thrust line going from the bell tower to the structures of the Basilica (shown in Fig. 4), in correspondence of the upper area of the above-mentioned column, with stress values ranging from 0.56 MPa to 0.95 MPa. The presence of this thrust line was also confirmed by the results of the relevant numerical model, carried out during the investigations (LIONELLO, 2008).

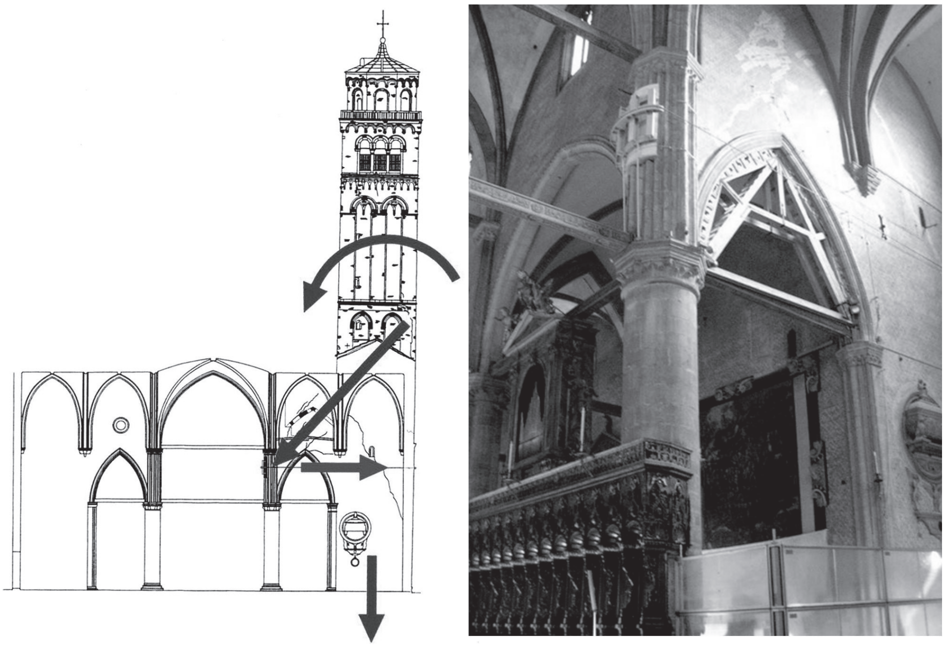


Fig. 4 – Left: possible evolution of the bell tower displacements after the bell tower foundation enlargement in 1904 and interaction with the adjacent basilica structures (Lionello *et al.*, 2004). Right: a picture of the temporary intervention by a steel cable to support part of the horizontal thrust acting on the adjacent column of the Frari basilica.
Fig. 4 – A sinistra: possibile evoluzione degli spostamenti campanile dopo l'allargamento in fondazione eseguito nel 1904 ed interazione con le adiacenti strutture della basilica. A destra: foto dell'intervento provvisionale eseguito con un cavo d'acciaio per sostenere parte della spinta orizzontale che agisce sulla colonna della basilica adiacente al campanile (Lionello et al, 2004).

2.2. Geotechnical investigations

A rather detailed geotechnical investigation was carried out in May 2003 (tests "x"/A in Fig. 6) and, on that basis, the subsequent ground improvement intervention by soil fracturing was planned. The investigations of such A-phase consisted of: 4 piezocone tests, 2 vertical and 5 inclined continuous coring boreholes, 4 continuous borings into the foundation block, together with the extraction of several undisturbed soil and foundation block samples for the subsequent execution of the laboratory tests, which enabled the stratigraphy, the subsoil properties and the geometry of the foundation block to be defined with some detail (GOTTARDI *et al.*, 2009). A section of the thus reconstructed peculiar bell tower foundation along the SE-NW direction is provided in figures 3 and 7. As usual in Venice (MARCHI *et al.*, 2006), the Frari bell tower foundation is made up of Istrian limestone squared blocks and short timber compaction piles (1.702.20 m long), with an interposed 0.40-0.50 m thick larch boarding.

The soil profile under the tower, shown in figure 7, consists of:

- Unit A, between ground level and a depth of about 3.2 m: anthropic fill;
- Unit B, between 3.2 m and about 6.7 m: dark grey, soft, silty clay, with occasional organic material, normally consolidated or slightly overconsolidated, with organic inclusions and shells In figure 8 the relevant main geotechnical characteristics, as deduced from laboratory and in situ tests, are reported;
- Unit C, between 6.7 m and 14 m: grey medium-fine sand, non plastic, from dense to very dense;
- between 14 m and the maximum investigated depth: alternation of soft clayey silt and medium-fine dense sand.

Finally, the good conditions of the foundation enlargement built on the external side of the bell tower were confirmed. A careful evaluation of settlement trend with time excluded that the movements of the tower could be entirely ascribed to secondary settlements in confined conditions. Hence, the reasons of the continuous foundation problems were mainly attributed to a slow lateral plastic flow under high stress gradients within the soft silty clay layer, squeezed between the pile ends and the underlying sand, and to the possible progressive decay of timber piles.

2.3. Monitoring system

After the end of the diagnostic investigations, a comprehensive monitoring system was installed to analyze the deformation behaviour and the structural conditions of the bell tower and the adjacent

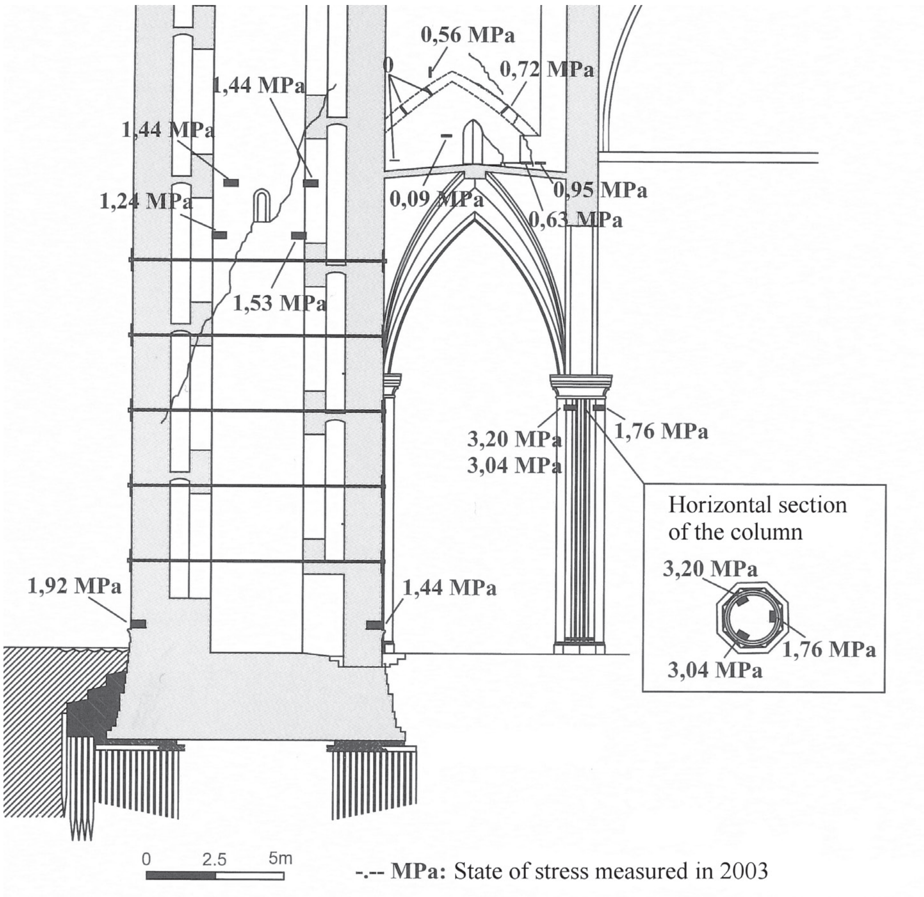


Fig. 6 – Plan of the Frari bell tower with the location of the geotechnical in-situ investigations at various stages of the intervention (Gottardi *et al.*, 2013).
Fig. 6 – Planimetria del campanile con indicazione delle indagini in sito effettuate in varie fasi dell'intervento (Gottardi et al., 2013).

portion of the Basilica during all the phases of the strengthening interventions, consisting of:
– crack-gauges and long-base extensometers installed on the main cracks of the masonry walls;
– strain-gauges to measure the deformation of the steel cable installed in the bell-tower;
– thermal gauges to measure the temperature of the internal and external air, as well as inside the masonry at different distances from the outer wall;
– geotechnical instrumentation, including electrical piezometers, multibase extensometers and biaxial inclinometers;
– direct pendulum equipped with automatic telecoordinometer, for the measurement of the absolute horizontal movements of the top of the tower.

All the instruments were connected to an automatic data acquisition system that enabled to follow in real time the effect of the works on the structures, thus enabling to introduce possible suitable modifications to the intervention design. In addition, it was considered of vital importance to measure the vertical movements of the bell tower and of the adjacent portion of the Basilica. A high-precision and accurate manual leveling system with several measuring points was thus installed and periodical surveys were carried out and intensified during the most significant phases of the works.

3. The intervention

The slow but continuous, constant rate, differential settlement of the bell tower has soon become cause of major concern for the present and future stability, not only of the bell tower but, above all, of the structurally connected Basilica. From the results of the diagnostic investigations and the following numerical simulations (LIONELLO *et al.,* 2004), it clearly emerged that the interacting structures of the bell tower and the Basilica cannot bear further differential settlements without serious consequences.

It followed the need of a strengthening intervention, at the level of the foundations, aimed at reducing the differential settlements of the bell tower. A rather innovative intervention of careful soil fracturing, also known as fracture grouting,

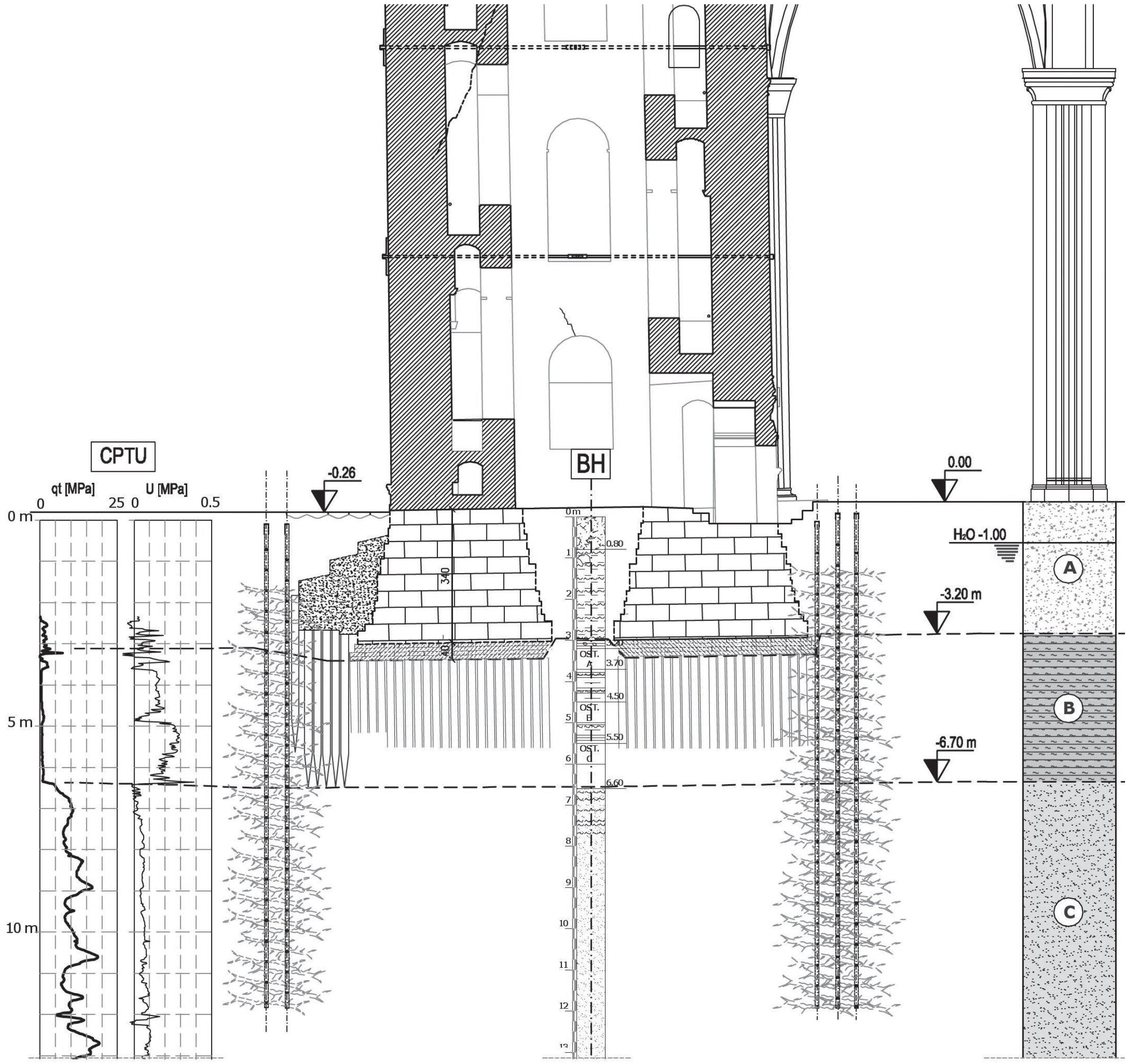


Fig. 7 – Schematic section of the tower foundation and of the relevant subsoil (SE-NW direction in Fig. 6), with the in-situ test logs, the strengthening intervention of 1904 (left) and the TAMs of the new fracture grouting intervention (Marchi *et al*, 2014).
Fig. 7 – Sezione schematica della fondazione della torre e del terreno di fondazione (direzione SE-NW in Fig. 6). In evidenza i risultati di alcune significative prove in-situ, l'intervento di consolidamento del 1904 (a sinistra) e le canne valvolate (TAMs) per l'iniezione della malta cementizia.

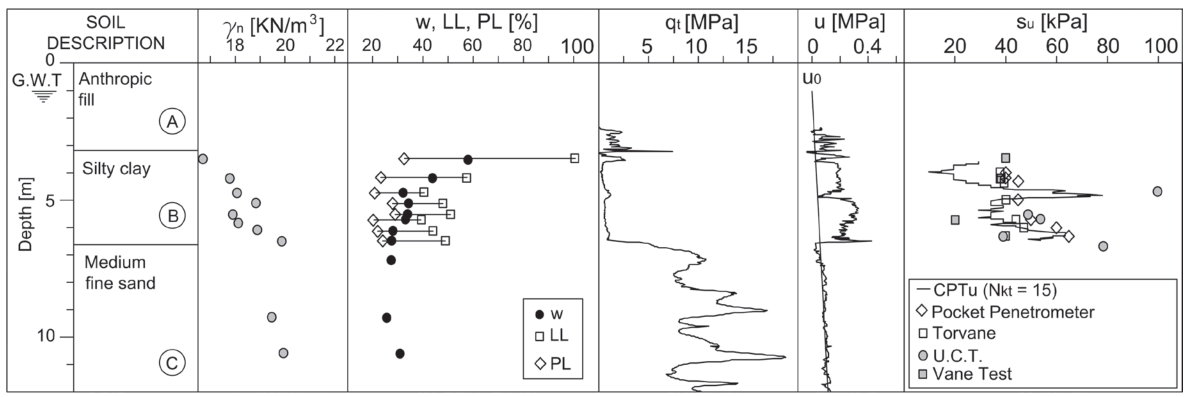
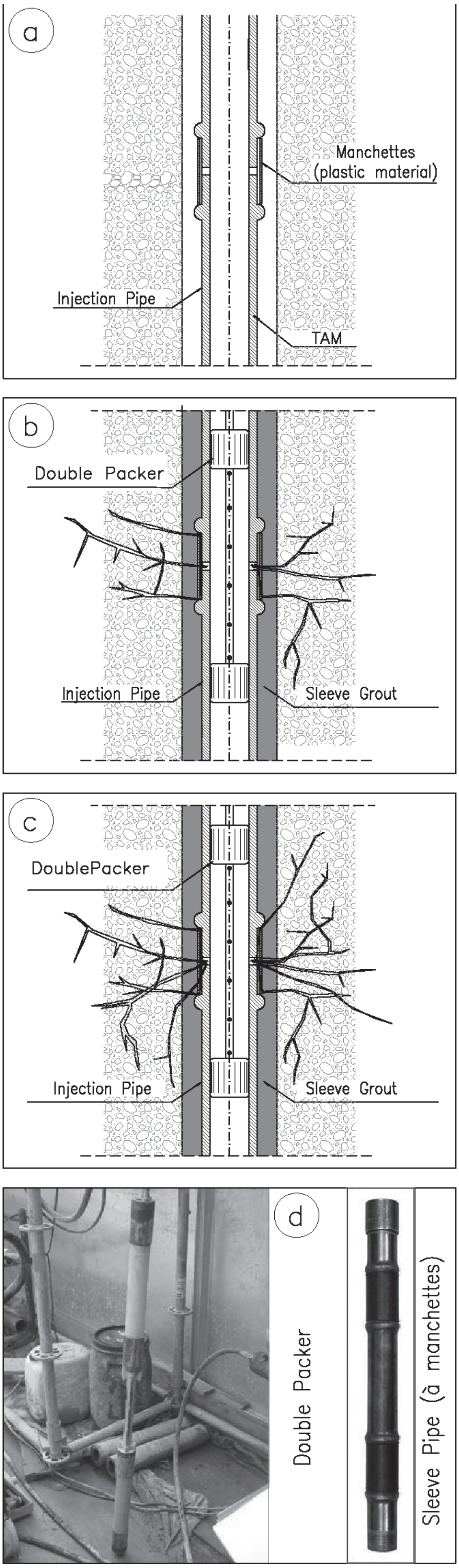


Fig. 8 – Upper soil profile with the relevant main geotechnical characteristics, as deduced from laboratory and in situ tests.
Fig. 8 – Principali caratteristiche geotecniche del terreno di fondazione, da prove in sito e di laboratorio.



was eventually designed, in order to improve the mechanical characteristics of the soft silty clay. Once the aim of improving the stability of the soil-foundation system had been achieved, a new solution was required to reduce the damaging interaction between the masonry structures, activated by the foundation settlements. A structural joint between the bell tower and the Basilica was finally executed in order to improve the system deformability. In order to guarantee the safety of the whole Basilica and bell tower, a gradual strengthening intervention was designed, with a strict and constant control during the execution of the works, a rather typical and well implemented example of the so-called "Observational Method".

3.1. Phase 1: provisional strengthening intervention

A preliminary intervention was required by the concern of deformations induced on the column of the Basilica by the thrust of the bell tower. A provisional intervention was then carried out in order to increase the safety level of this specific and most delicate component of the Basilica. A steel cable was thus positioned connecting the stone ashlars just above the capital of the column to the bell tower structure at a height of 14.40 m (round circle in Fig. 4), aimed at supporting part of the horizontal thrust acting on the column. Two strain-gauges were installed on the steel cable and the relevant tension constantly monitored during the whole intervention period.

3.2. Phase 2: soil strengthening

The principles on which the soil strengthening intervention was based were (LIONELLO, 2008):
– making compatible the remaining settlements of the complex Basilica and bell tower;
– preserving as much as possible the original foundation structure and the current stress distribution;
– avoiding a rigid foundation system for the bell tower;

Fig. 9 – Soil fracturing via multiple injections: (a) initial state: injection pipe installed; (b) first injection: fractures predominantly in one direction; (c) continued injection: fractures in various directions; (d) double packer and sleeve pipe (TAM).
Fig. 9 – Fasi di attuazione della metodologia di iniezione per idrofratturazione (fracture grouting): (a) perforazione e installazione della canna valvolata; (b) iniezione di primo ciclo; (c) iniezioni successive; (d) doppio otturatore e canna valvolata (TAM).

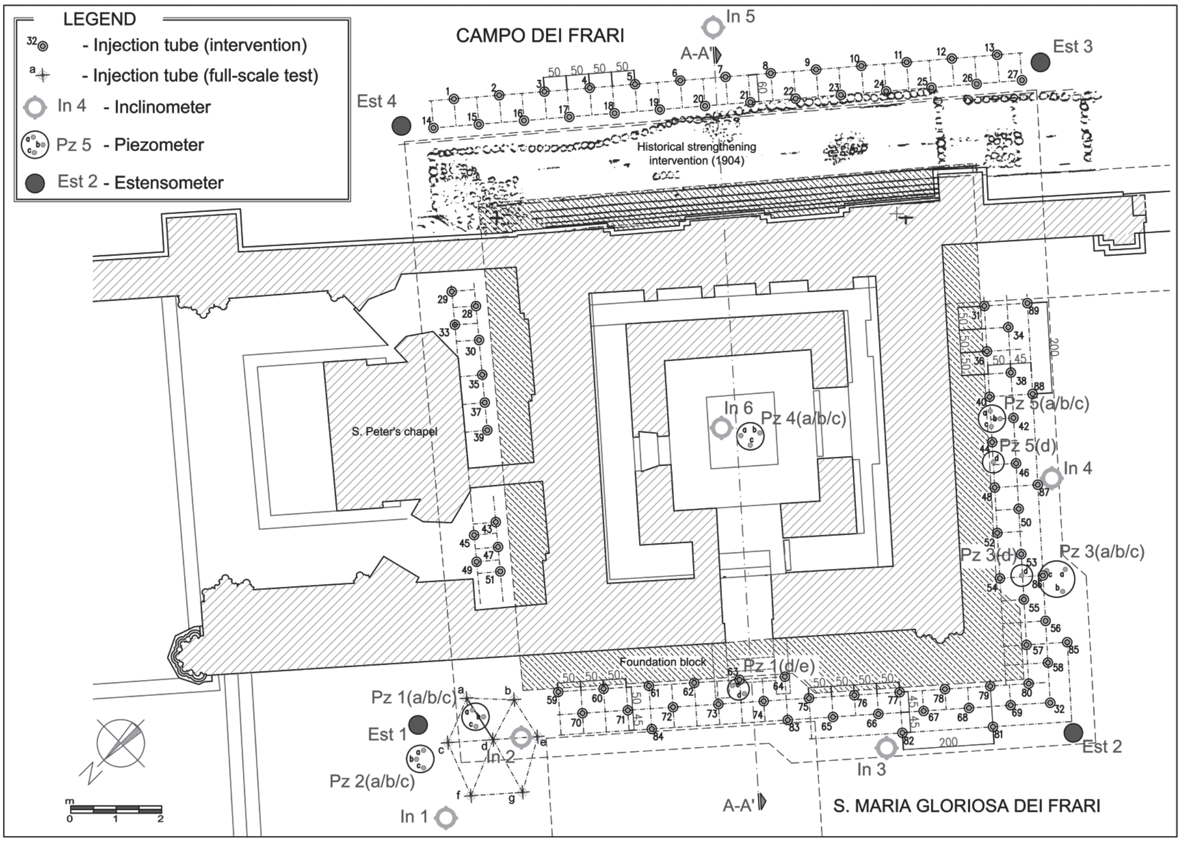


Fig. 10 – Plan of the Frari bell tower with TAMs lay-out and location of geotechnical monitoring devices (Lionello, 2008).
Fig. 10 – Planimetria con ubicazione delle verticali di iniezione e della strumentazione di monitoraggio geotecnico (Lionello, 2008).

– enabling a flexible and modular intervention, in constant agreement with the outcome of an extensive real time monitoring.

An intervention of fracture grouting was finally selected. This technique consists of installing special injection pipes (*tubes à manchettes, i.e.* TAMs, Fig. 9a) in the foundation soil, fitted with equally spaced valves at different depths. Each valve can be selectively injected by means of a double packer device (Fig. 9b). The careful and slow rate injection of suitable cement and bentonite mixtures can be repeated at successive stages, to obtain progressive increments of mechanical characteristics. The final outcome should be a reinforced soil, made up of the ori-

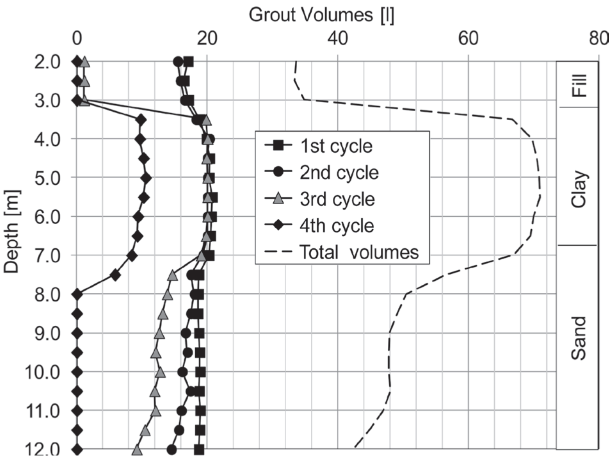


Fig. 11 – Average (for the overall 88 pipes) volumes injected from each valve, at selected depths, for the 1st, 2nd, 3rd and 4th cycle (Marchi *et al.*, 2014)
Fig. 11 – Volumi medi (sulle 88 canne iniettate) di miscela cementizia iniettati durante il trattamento, per il primo, secondo, terzo e quarto ciclo di iniezione (Marchi et al., 2014).

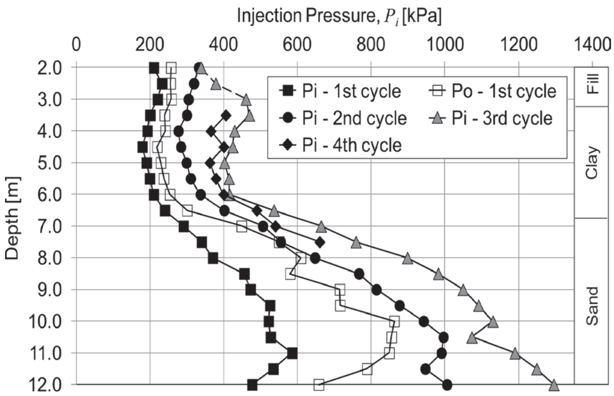


Fig. 12 – Average injection pressures vs. depth, for each cycle, as measured at the manometer (Marchi *et al.*, 2014)
Fig. 12 – Pressioni medie di iniezione misurate al manometro, per ogni ciclo di iniezione (Marchi et al., 2014)

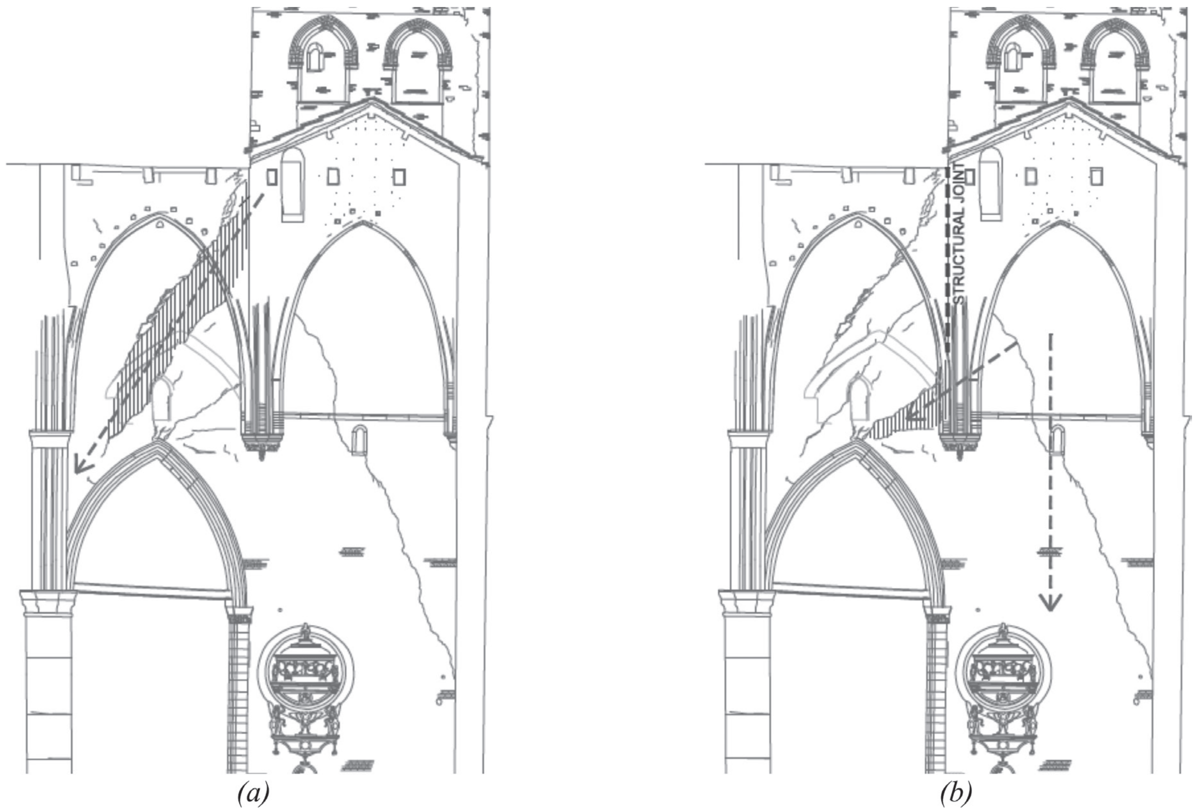


Fig. 13 – Direction of the thrust between the bell tower and the basilica: a) before the structural joint execution, b) after the structural joint execution.

Fig. 13 – Direzione della spinta tra il campanile e la basilica: a) prima dell'esecuzione giunto strutturale, b) dopo la realizzazione del giunto.

ginal material and an indented web of thin layers of injected grout (Fig. 9c).

In order to evaluate the feasibility of the soil fracturing intervention (MORI and TAMURA, 1987; PANAH and YANAGISAWA, 1989; RAABE and ESTERS, 1990; ANDERSEN *et al.*, 1994; ALFARO and WONG, 2001; SOGA *et al.*, 2005,) and calibrate the design parameters (injection pressures, injection rate, grout mixture, etc.), a full-scale test site with geotechnical monitoring devices (piezometers and multibase extensometers) was carried out on the northern corner of the bell tower, inside the Basilica. The test gave the expected results and the soil fracturing intervention was then carried out by means of eighty-eight 12 m-deep sleeve steel pipes (so-called "tubes à manchettes", i.e. TAMs) installed all around the perimeter of the bell tower (Fig. 10). Such special injection pipes had equally spaced (0.5 m) valves, selectively injectable by means of the double packer device. They were aligned along two rows, according to local geometric constraints, except for the west side where a third row was subsequently added. The inner row was placed as close as possible to the foundation side; the other two rows were spaced of about 50 cm (Fig. 10). A cement-bentonite grout was injected from each valve in three main separate stages (cycles). A fourth cycle was designed to enhance the intervention on the clay layer only and for the 55 pipes that had registered a lower injection pressure in the previous stages. The selected grout was made up of water, cement, bentonite and calcareous fill, with a water/cement ratio of about 1.5 and a cement/bentonite ratio of about 14, thus producing a very low viscosity grout (1.5×10^{-2} Pa × s).

During the intervention the flow rate was kept low and fixed at about 6 l/min. The resulting valve opening pressure and the following steady-state injection pressure were always recorded. In the final cycle, a flow rate of about half than the previous cycles was used. Relevant injected volumes are shown in figure 11. Constant grout volumes of 20 l/valve were injected each cycle in clay, whereas variable volumes between 14 and 20 l/valve in sand. In the 1st cycle all valves could be opened. In the 2nd cycle a few valves could not be opened in sand. In the 3rd and 4th cycles more restrictive criteria were adopted: a pre-fixed limit to the opening pressure of 500 kPa in clay and of 2500 kPa in sand, where therefore many more valves could not be opened. Figure 11 shows the total volumes injected in each cycle from each valve depth, divided by the total number of existing pipes (*i.e.* 88). Notice that the total amount of

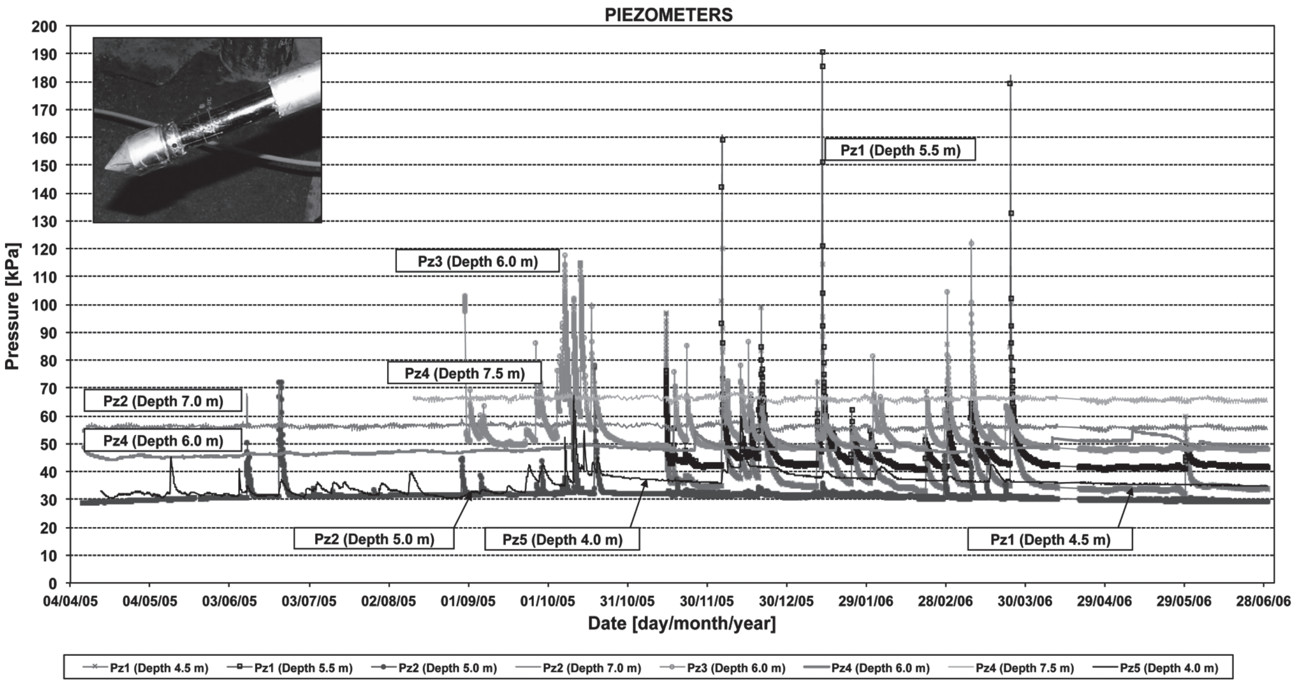


Fig. 14 – Records of most electric piezometers installed within the silty clay layer (after Lionello, 2008).
Fig. 14 – Dati piezometrici misurati durante e dopo il termine dell'intervento (Lionello, 2008).

grout injected in clay is clearly greater than in sand. At the end of the intervention, a total of about 100 m^3 of grout was injected.

Figure 12 reports the average steady-state injection pressures (P_i) vs. depth for each cycle, as measured at the manometer. P_i is clearly greater in sand and tends to increase almost linearly with depth. Injection pressures vary between about 200–500 kPa in the first cycle to about 400-1200 kPa and more in the final cycle. In particular, the low injection pressures (P_i) recorded in the third cycle between 2.00 and 3.00 m depth are not significant since they result from only 5 pipes. In addition, P_i recorded in the fourth cycle, resulting from a reduced and selected number of pipes (55), turned out to be lower than that in the third cycle; this could be also due to the lower injection flow rate used in the final cycle, as already noticed in previous experiments. However, the clearly noticeable general increase of pressures with injection cycles reflects a corresponding increase of the soil minor principal stress during the intervention, as intended (MARCHI *et al.*, 2014).

3.3. Phase 3: structural joint

Once completed the intervention on the foundation soil and whilst continuing to monitor the bell tower behavior, the relevant structural interaction with the most damaged arches, vaults and columns of the Basilica had to be tackled.

Such interaction implied in fact:

- a substantial compression stress increment on the column, much greater than the contribution of the dead weight of walls, vaults and roofs;
- the formation of a large flexural stress on the column itself, due to the eccentricity of the vertical load and to the horizontal thrust component;
- the corresponding reduction of the vertical stress along the bell tower walls.

According to a suitably conceived approach for the restoration and preservation intervention, it was therefore opted, instead of opposing against the current forces acting on the structures, to reduce their stress distribution (LIONELLO, 2011). About two years after the end of strengthening works on the foundation soil, it was decided to insert a gap between the bell tower and the adjacent church by creating a structural joint that would allow the relative movements and prevent the mutual stress transmission. It was assumed that the compression stresses flowing above the arch and transmitting substantial forces to the column of the Basilica (Fig. 13a) could be thus reduced.

The structural joint was created at the beginning of June 2008 in the position shown in figure 13b and, in order to preserve the Basilica architecture, it was carried out only above the vaults where, on the other hand, mechanical interaction was greater. During the intervention, it could be confirmed that the existing structural link had been executed only after the construction, probably together with the early 20th century interventions, when it was wrongly

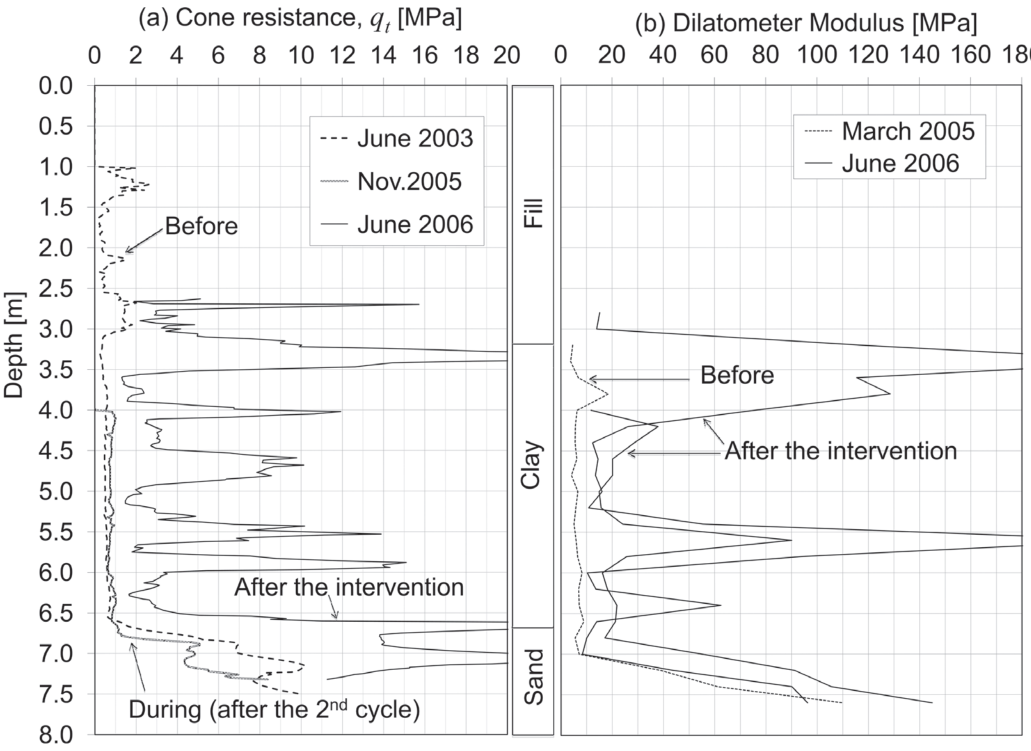


Fig. 15 – Comparison between (a) CPTU tests and (b) dilatometer modulus (MPa) before the intervention after the second injection cycle and at the end of the injections (after Marchi *et al.*, 2014).
Fig. 15 – Confronto tra (a) prove CPTU e (b) modulo dilatometrico (MPa) prima dell'intervento, dopo il secondo ciclo di iniezione e al termine delle iniezioni (Marchi et al., 2014).

pursued the approach of connecting the bell tower to the Basilica structures.

The effect of such intervention was a marked change of direction of the thrust applied by the bell tower to the adjacent column, as shown in figure 13b and predicted by numerical models.

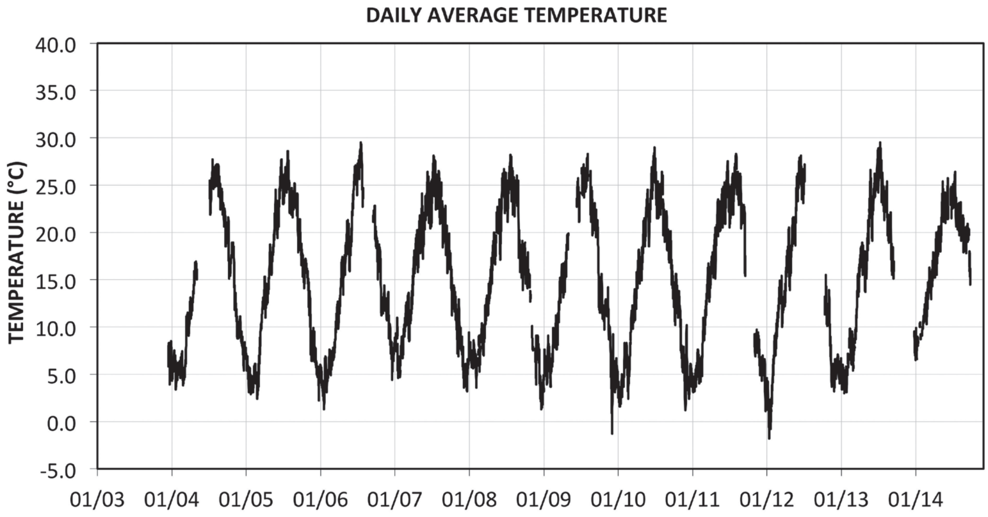
The execution of the structural joint was very slow and lasted about 6 months. During this period a detailed analysis of the information obtained by the monitoring system enabled to carry out the different steps of the intervention with a continuous check of the structural response, thus avoiding to induce damages to the bell tower and to the supporting structures of the Basilica.

4. Investigations and monitoring during and after the intervention

4.1. Soil investigations and monitoring

As reported in section 2.3, the geotechnical instrumentation installed throughout the area of the intervention enabled to carefully check the effects of the soil fracturing. As an example, in figure 14 the records of most electric piezometers installed within the silty clay layer are provided: the pore pressure peaks induced by adjacent grout injections are clearly visible as well as the subsequent relatively quick consolidation rate, which enabled to drive the operations under constant control and safety conditions.

As designed, the injections led to soil fracturing: evident and diffuse cement lenses in the soil were found in undisturbed continuous core sampling carried out at the end of the intervention. Specific additional in situ geotechnical investigations (piezocone and dilatometer tests) were planned and carried out to assess its effect on the mechanical properties of soil. The global effect of the injections could be assessed by comparing the results of in situ tests performed at different stages of the intervention. The soil strength improvement is clearly noticeable from all the CPTU tests carried out before the intervention, after the second injection cycle, and at the end of the injections (Fig. 15a) (respectively CPTu"n"/A, /D and /E in Fig. 6). Peaks of the tip resistance are due to the diffuse presence of grout lenses, although the relevant average increase among them was more effective. Figure 15b shows analogous data related to the dilatometer modulus. The relevant stiffness increase was also confirmed by laboratory oedometer tests, which provided a modulus increase of approximately 100% at 50 kPa and approximately 20% at 400 kPa (LIONELLO, 2008).



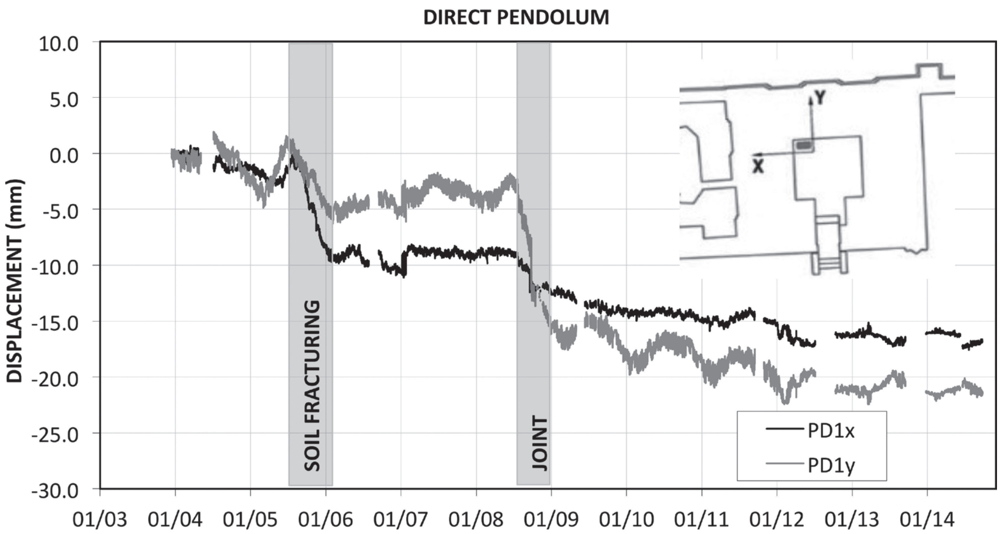


Fig. 16 – Complete history of the displacement components measured by the direct pendulum with the indication of the intervention phases: soil fracturing and structural joint execution (to 27/10/2014).
Fig. 16 – Spostamenti della torre misurati tramite pendolo diretto in un lasso temporale che comprende le varie fasi di intervento: iniezioni e giunto strutturale (dati al 27/10/2014).

However, more comprehensive information for the analysis of the structure behaviour during the soil fracturing works come from the direct pendulum records and precision leveling surveys. In figure 16, the movement trend along two orthogonal components from direct pendulum are shown. The period of observation is from December 2003 (date of installation of the monitoring system) to the end of October 2014. During the soil fracturing intervention (from April 2005 to March 2006, grey shadow), the monitoring system was also useful to define the rate of the intervention phases as well as to support decisions on the parameters of the grouting procedures (injection pressure, flow rate, etc). A significant movement of the bell tower during this intervention phase was observed, the component in x direction (toward the apse) being about 9.0 mm and in y direction (toward the Basilica) about 5.0 mm. After the end of the soil fracturing, the rate of the bell tower movements showed a quick reduction, reaching a lower value than that observed before the intervention. Movements of cracks in the stone arch which connects the bell tower to the Basilica also showed a marked increase during soil fracturing and a rapid decrease after.

In figure 17 the settlement trend of several benchmarks from precision leveling is provided. A more pronounced vertical movement of the bell tower west and south corners (points 5 and 6) is clearly observed during the soil fracturing intervention (time interval highlighted in grey), but all measuring points display a similar rate increase during the intervention and a significant reduction immediately after, also with respect to original rates be-

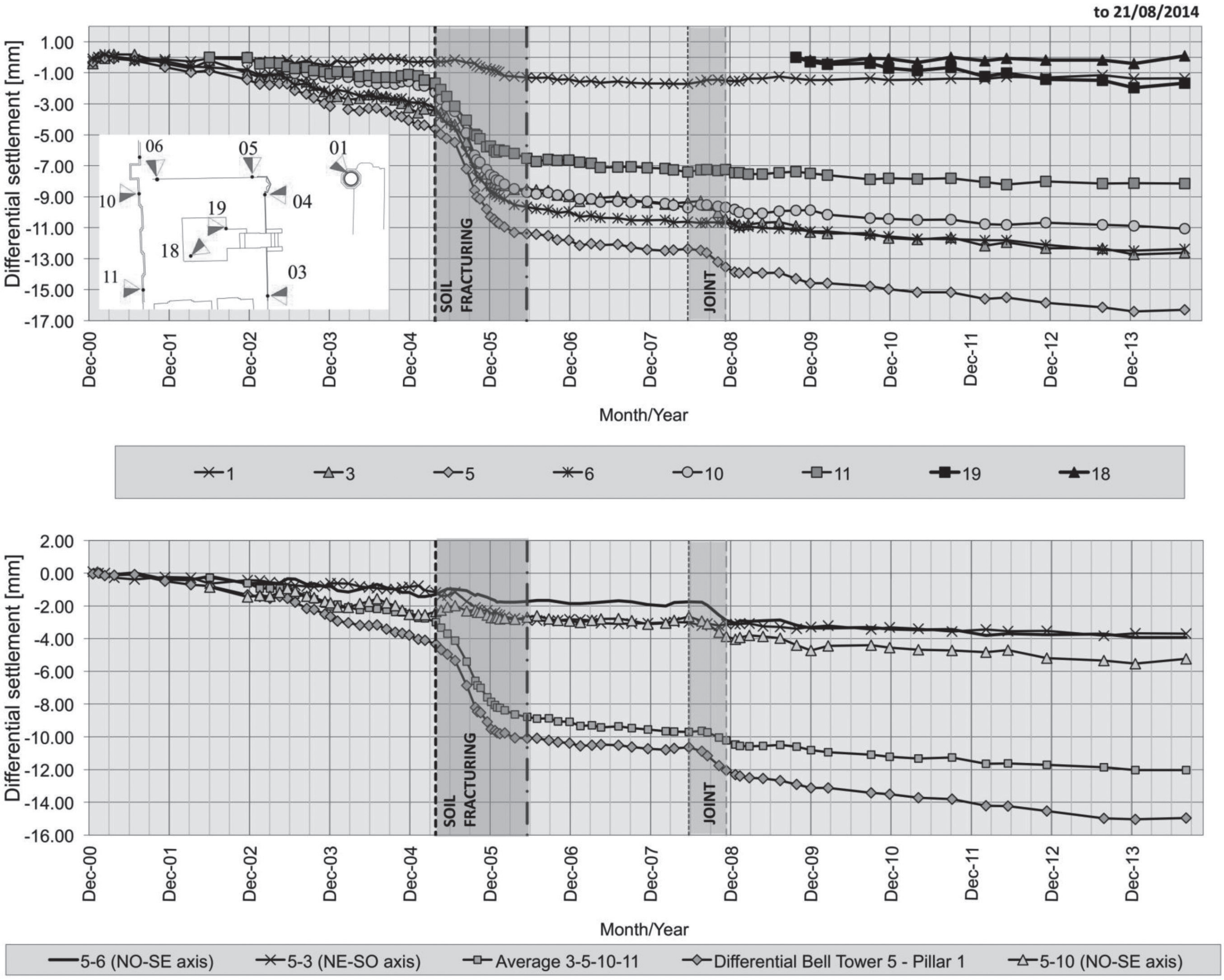


Fig. 17 – Settlements (left) and differential settlements (right) of several benchmarks from precision leveling (courtesy of FOART srl, Parma), to 21/08/2014.

Fig. 17 – Cedimenti (sinistra) e cedimenti differenziali (a destra) di diversi caposaldi, da livellazione di precisione (letture FOART srl, Parma), al 21/08/2014.

fore, consistently with design expectations. Such observation is confirmed by the differential settlement trend (Fig. 17) along main alignments and, above all, between the critical column of the Basilica and the adjacent bell tower corner (points 1 and 5): 1.01 mm/year before, 5.04 mm/year during and 0.27 mm/year after the intervention (before the joint execution).

The settlement values are also in very good agreement with the measures obtained by the direct pendulum in the period from December 2003 to March 2007 (Fig.18). Along the alignment of points 3 and 4, the differential settlements at the base of the bell tower (2.16 mm) multiplied for the ratio between the height and the base of the tower (4.70) turns out to be 10.15 mm, very close to the *x* component measured by the direct pendulum (10.00 mm). Along the alignment of points 5 and 10, the differential settlements at the base of the bell-tower (0.95 mm), multiplied for the same ratio, is equal to 4.46 mm, which is again very close to the *y* component measured by the direct pendulum (4.00 mm).

4.2. Structure investigations and monitoring

In order to follow with special care the deformation behavior during the execution of the structural joint, new crack-gauges as well as new long-base extensometers were installed. Furthermore, in order to check the thrust modification between the bell tower and the Basilica, special flat-jacks were installed in the positions indicated in figure 19: on the right the stress values measured before the execution of the structural joint, while on the left the stress changes after the intervention are shown. It can be observed a significant decrease of the state of stress in the upper part of the wall between the bell tower and the

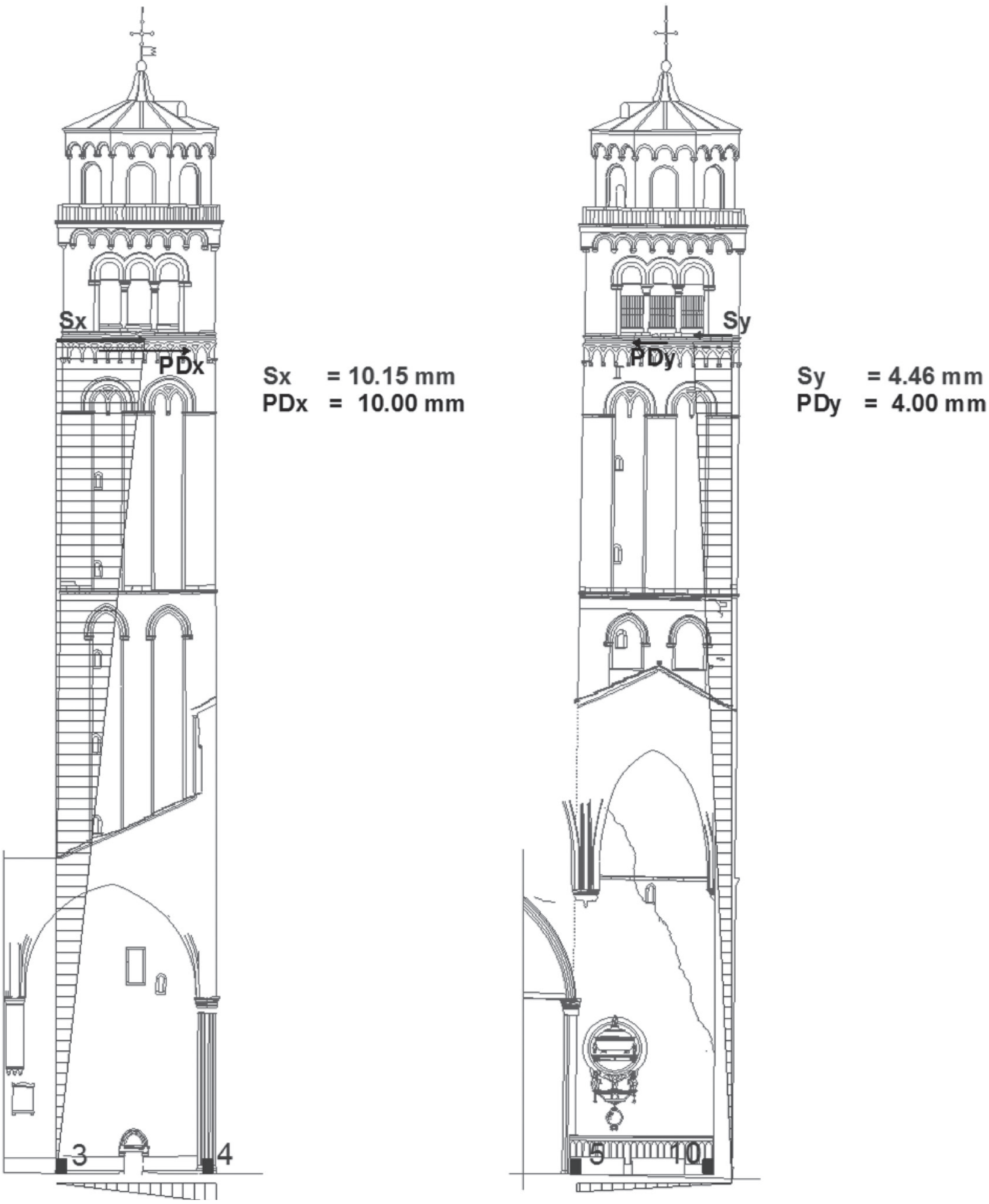


Fig. 18 – Comparison between the movements at the top of the Frari bell tower induced by the differential settlements of the foundations and those measured by the direct pendulum, in the period from December 2003 to March 2007 (Rossi & Rossi, 2012).

Fig. 18 – Confronto tra i movimenti della cima del campanile indotti dai cedimenti differenziali della fondazione e quelli misurati dal pendolo diretto, nel periodo tra dicembre 2003 e marzo 2007 (Rossi & Rossi, 2012).

Basilica, which is a clear experimental confirmation of the assumptions on the thrust reduction applied by the bell tower.

In particular, the positive effects of the structural joint can be summarised as follows:

1) a substantial reduction of the load acting on the column of the Basilica, estimated as about 800 kN (-17%);
2) a corresponding increase at the base of the bell tower;
3) a restoration of the original conditions when the two structures were statically more independent.

An interesting indication of the bell tower response to such structural modification can come again from the measurements of the direct pendulum (Fig.16, second shadowed time interval). It can

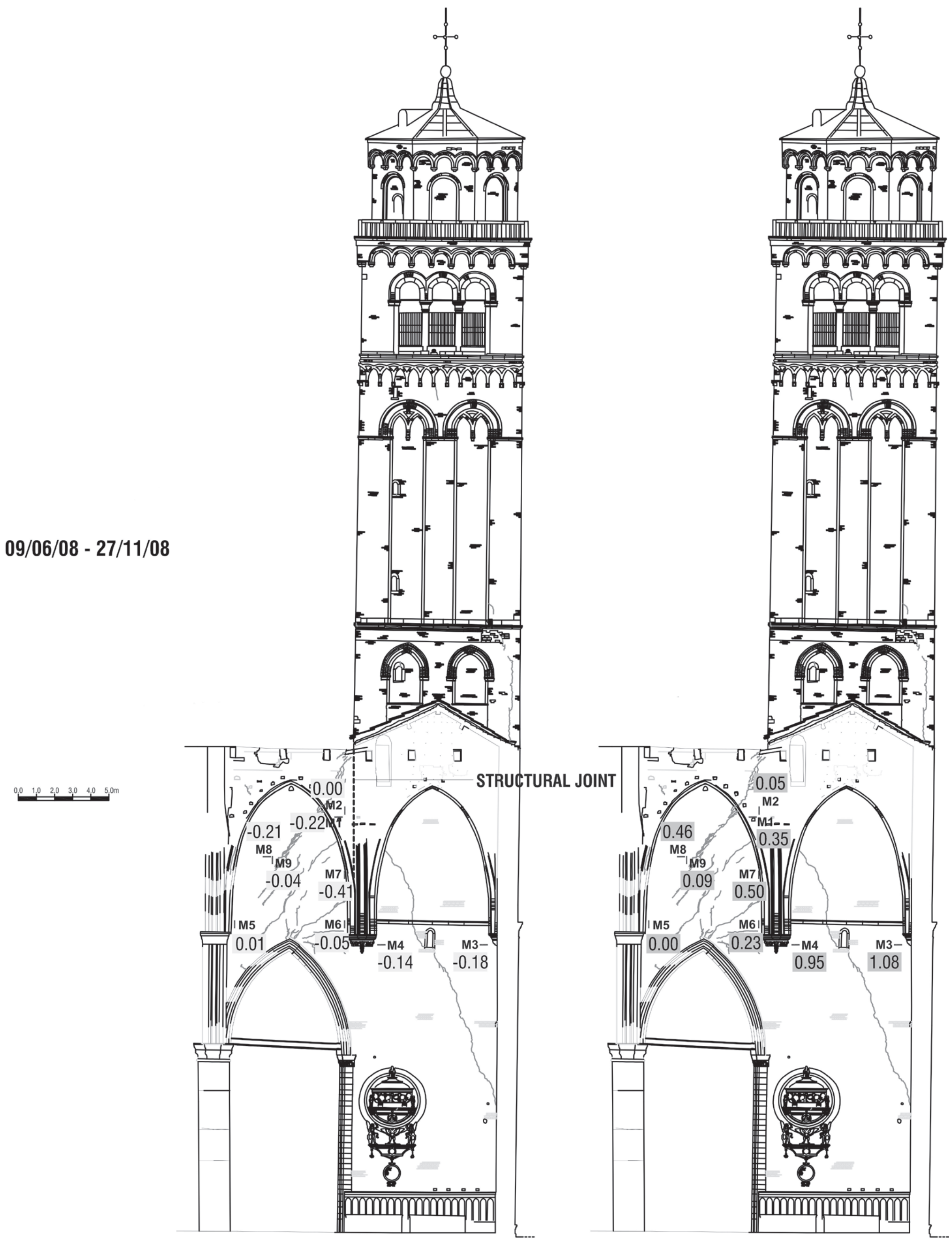


Fig. 19 – Measurements of the state of stress before the structural joint execution (on the right) and variation of the states of stress induced by the structural joint execution (on the left) (Rossi & Rossi, 2012).
Fig. 19 – Misura dello stato tensionale nelle murature prima della realizzazione del giunto (a destra) e variazione dello stato tensionale indotto dalla realizzazione del giunto (a sinistra) (Rossi & Rossi, 2012).

be observed a more evident effect along the *y* direction, i.e. consistently with the newly introduced lack of support.

It can be also helpful to analyse the movements of the masonry structures along the existing cracks together with the displacements provided by the direct pendulum and the settlements coming from precise levelling, especially when comparing the situation before (Fig. 20) and after (Figs. 21 and 22) the structural joint intervention. In the time interval - about two years - between the soil fracturing and the structural joint, the tower has settled less than one millimeter (see surveying records at the top left of Fig. 20), with a modest inclination toward the external side.

After the end of the structural joint execution and the complete removal of the temporary steel cables and of the timber prop system, the deformation behaviour has been constantly observed. The displacement components of the bell tower, measured by the direct pendulum, are more substantial

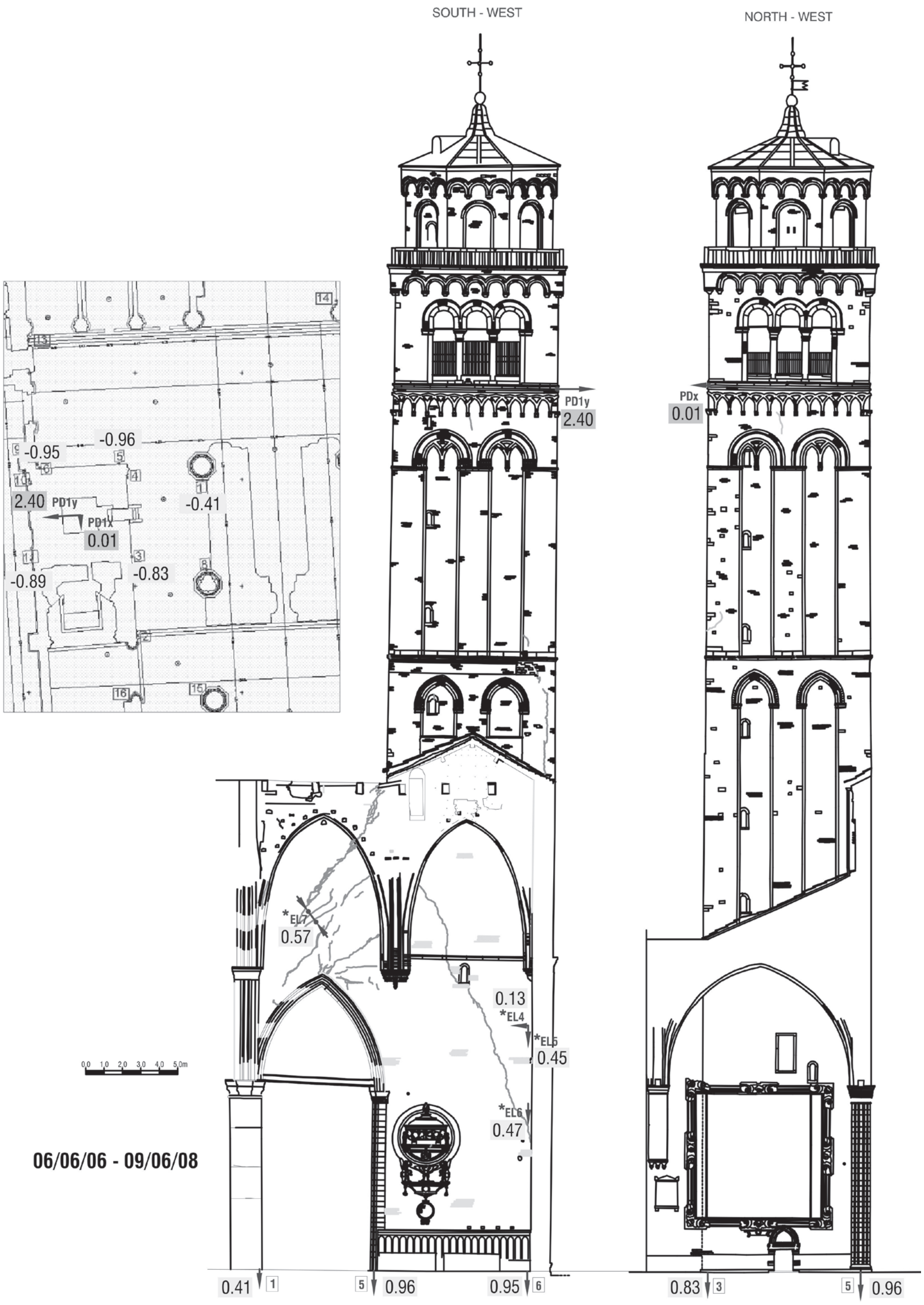


Fig. 20 – Analysis of the deformations measured during a period of two years, between the soil fracturing intervention and the structural joint execution.

Fig. 20 – Analisi delle deformazioni misurate in un periodo di tempo di due anni, tra l'intervento di consolidamento del terreno (fracture grouting) ed l'esecuzione del giunto.

and in the opposite direction, toward the Basilica. They now appear to be only partly related to the differential settlements of the foundation, the remaining part being due to deformation processes of the bell tower masonry.

Despite such substantial modification of the load distribution, however, which has inevitably produced a new increase (up to about 0.6 mm/year from January 2009 to November 2012, Fig. 17) of the differential settlement between the bell tower corner and the column (which incidentally has even showed a temporary upward relative movement, benchmark 1 in Fig.17), all the monitoring devices (Figs. 16, 17, 20, 21 and 22) show a consistent trend. The average – and still rather uniform - settlement of the bell tower has proved to be about 0.3 mm/year, in the 4 years after the joint execution (from January 2009 to November 2012) and 0.15 mm/year, in the last two years and a half (from February 2012 to August 2014). When compared to the rate of more than

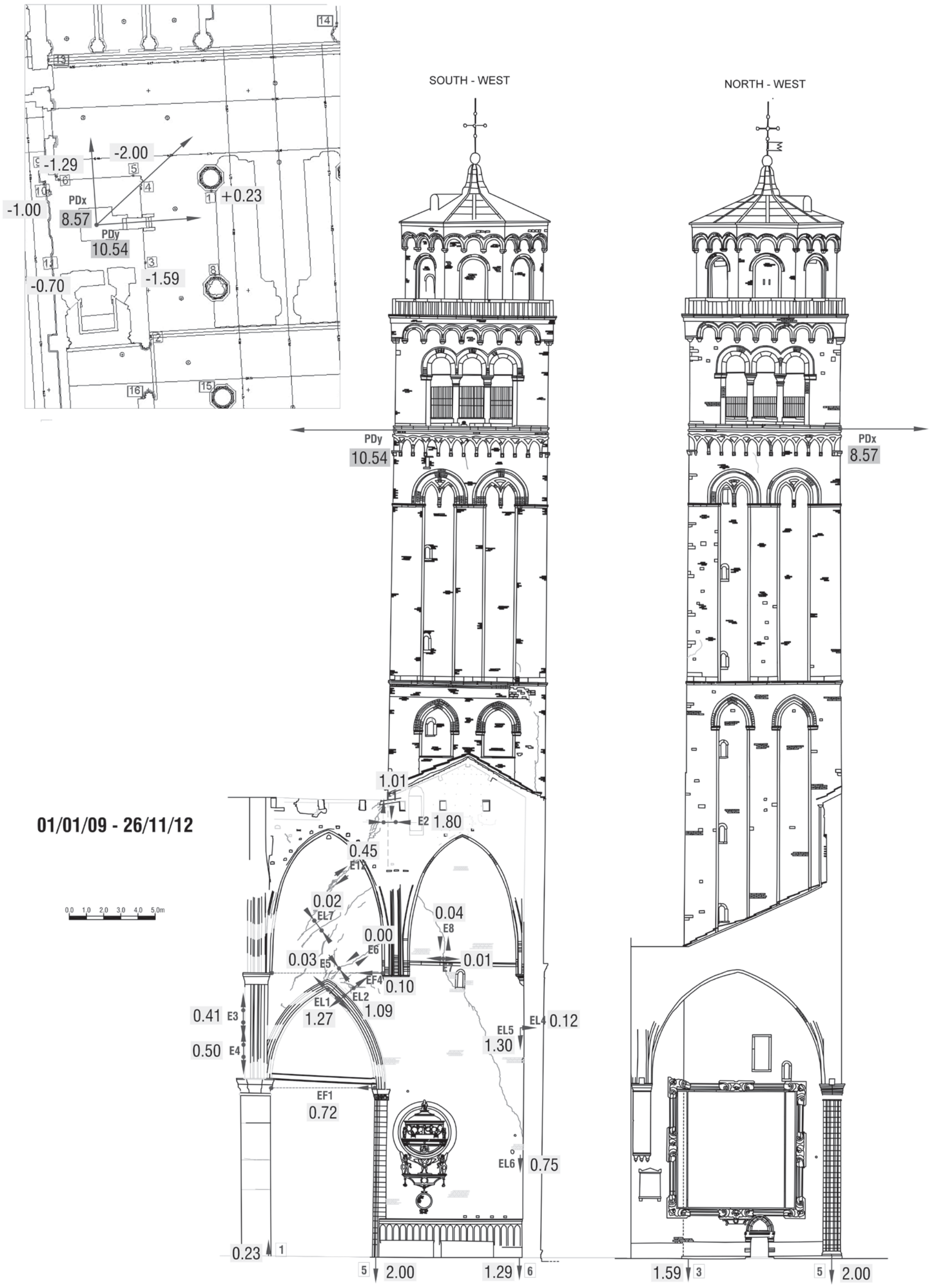


Fig. 21 – Analysis of the deformations measured during a period of about four years, after the structural joint execution.
Fig. 21 – Analisi delle deformazioni misurate in un periodo di tempo di circa quattro anni, dopo l'esecuzione del giunto.

1 mm/year before the soil fracturing intervention, it appears that its overall effectiveness can be confirmed.

It is finally extremely interesting to compare the average foundation settlement during the 5 years after the joint execution with the readings of two adjacent benchmarks (18 and 19 in Fig.17) located inside the bell tower, one at the foundation level (+1.00) (benchmark 19) and the other –more recently installed – anchored in the soil, at -6.00 m depth, just below the tip of timber piles (benchmark 18). Despite some scatter of the readings, due to the position of the benchmarks inside the bell tower, such comparison suggests that a large part of the continuous soil straining (about 80-90% of the overall foundation settlement) occurs in the first 6 m of depth. This evidence would confirm that the progressive and already established (rather unexpected in these terms, though) decay of timber piles can count for most residual movements of the bell tower.

Careful monitoring of settlement rates in the near future will enable to fully understand the ongoing mechanisms and to predict possible fu-

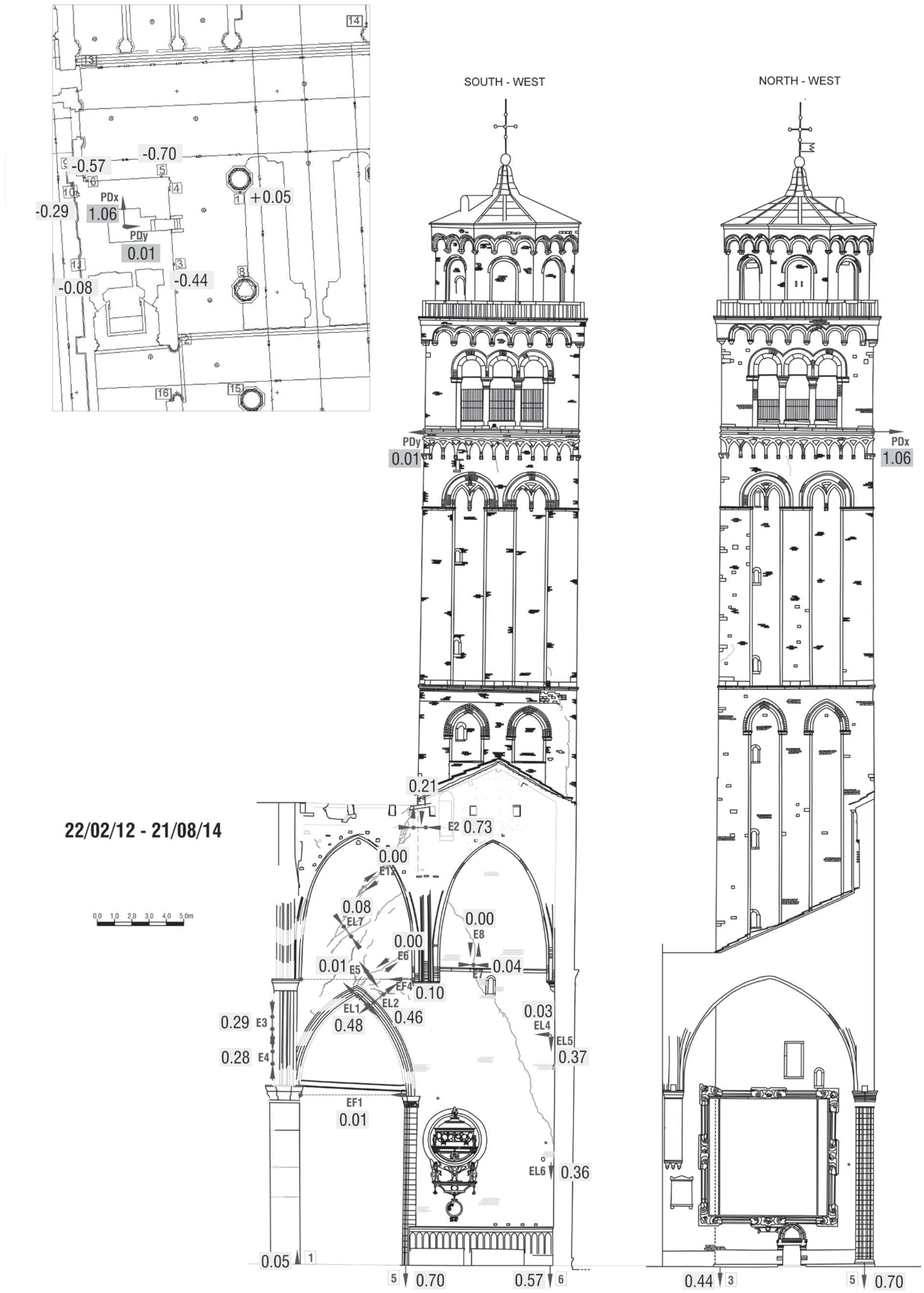


Fig. 22 – Analysis of the deformations and displacements measured in the last two years and an half.
Fig. 22 – Analisi delle deformazioni e degli spostamenti misurati negli ultimi due anni e mezzo

ture trends. However, the current reduced static interaction can accommodate further movements of the bell tower without inducing excessive stresses on the structures of the adjacent Basilica. The separation of the two structures has proved to be positive also in relation to seismic effects, if after the shakes of May 2012 no specific consequence has been recorded in the isolated structure, while new cracks have appeared in the nearby St. Peter Chapel, still interested by structural links, which will have to be carefully removed.

5. Conclusions

The Frari bell tower has been affected since its construction by a slow but constant differential settlement between the tower itself and the adjacent masonry structures of the Basilica. In the last 15 years a growing concern for the stability of the structures involved provided the impetus for the development of modern remedial measures, first in foundation and then on the elevation structure.

A ground modification intervention by soil fracturing was then carried out in order to improve the mechanical characteristics of the clayey layer underlying the tower. Once the aim of improving the stability of the soil-foundation system had been achieved, a new solution was required to reduce the damaging interaction between the masonry structures, activated by the foundation settlements. A structural joint between the bell tower and the Basilica was finally executed in order to improve the system deformability.

As regards the intervention on the foundations, innovative criteria and methodologies were used, so far devoted to rehabilitation and strengthening works on the upper structures. The material preservation of the foundation has been fully guaranteed, without any direct interventions on the relevant structures, by the insertion into the surrounding soil of injection pipes for soil fracturing. Through the articulated and extensive real-time monitoring system, purposely implemented, it was possible to focus on the intervention areas, calibrating and minimizing the amount of injected grout. The technology used has proved to be especially flexible and it will enable further injection cycles in the future, if required.

In the design of the intervention, any possible action producing a stiffness increase of the elevated structures, thus altering the overall structural behaviour, has been carefully prevented; the monitoring carried out during and after the works has confirmed the good compatibility of the strengthening intervention on the bell tower with the mechanical characteristics of the adjacent Basilica.

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Ruolo del monitoraggio nel progetto di un intervento multifase per la conservazione del campanile dei Frari a Venezia

Sommario:

a Basilica di Santa Maria Gloriosa dei Frari a Venezia ed il suo adiacente campanile, realizzato nel corso del XIV secolo, hanno da sempre evidenziato lenti ma costanti cedimenti differenziali. Nel corso del tempo tali movimenti hanno creato importanti danni strutturali sia alla basilica che al campanile. All'inizio del secolo scorso, la crescente inclinazione della torre indusse ad intervenire sul suo piano fondale, senza peraltro ottenere risultati risolutivi. Più di recente, circa 15 anni fa, gli evidenti dissesti strutturali hanno fornito l'impulso per l'avvio di una nuova campagna di indagine e le realizzazione di importanti interventi di consolidamento. Gli interventi sono stati svolti in più fasi, utilizzando criteri e metodologie innovative. In prima fase è stato condotto l'intervento in fondazione e successivamente sulla struttura di elevazione. L'intervento di consolidamento in fondazione, volto a mitigare i cedimenti dello strato superficiale di argilla limosa di scarsa consistenza, ha avuto lo scopo di rendere il più possibile compatibili i movimenti del campanile con quelli della basilica, salvaguardando al tempo stesso l'integrità delle fondazioni originarie. L'intervento di consolidamento è stato condotto mediante una serie diffusa e ripetuta di iniezioni di malta cementizia (tecnica nota come "fracture grouting"). Attraverso il sistema di monitoraggio in tempo reale, è stato possibile tarare e minimizzare la quantità di malta iniettata durante le iniezioni. Inoltre la tecnologia utilizzata si è dimostrata essere particolarmente flessibile e può consentire in futuro ulteriori cicli di iniezione, in caso di necessità. Una volta raggiunto l'obbiettivo di migliorare la stabilità del sistema terreno-fondazione, è stato condotto un intervento strutturale volto a ridurre l'interazione dannosa tra le strutture murarie, del complesso. Tra il campanile e la basilica è stato quindi realizzato un giunto strutturale con lo scopo di migliorare la deformabilità del sistema. Il monitoraggio effettuato durante e dopo i lavori ha confermato la buona compatibilità dell'intervento di consolidamento eseguito sul campanile con le caratteristiche meccaniche dell'adiacente Basilica.