Reconstruction of La Fenice theatre in Venice. Foundation problems

Reconstruction du thêatre La Fenice à Venise. Problèmes liés aux fondations

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ABSTRACT: The rebuilding of the theatre involved very delicate and exacting works to solve foundation problems. The underwater excavations had to be supported laterally by impervious barriers, particularly conceived in order to ensure the stability of the old, high masonry walls to be preserved. Safety against piping and excessive seepage from the bottom has been obtained by two-fluid jet grouting treatment. Works were made more difficult by the particular location of the theatre.

RESUME: La reconstruction du thêatre a entrainé un recours à des traitments très particuliers en fondation. Les excavations sous nappe de nouveaux espaces souterrains ont du être soutenues par des barrières laterales imperméables capables de garantir la stabilité des hautes vieilles murailles qui devaient être conservées, tout en prévénant des tassements dangereux. Ces travaux se sont revélés encore plus difficiles à cause de la particulière position du thêatre.

1 THE BUILDING YARD

The theatre was built in the eighteenth century on a Venice area at that time already densely urbanized. The theatre was destroyed in January 1996 by a fire. The reconstruction was inspired by the idea that it had to be "how and where it was".

According to the project of Aldo Rossi the old exterior masonry walls and some main structures inside the theatre, still intact after the fire, were not to be modified, whereas all the remaining parts

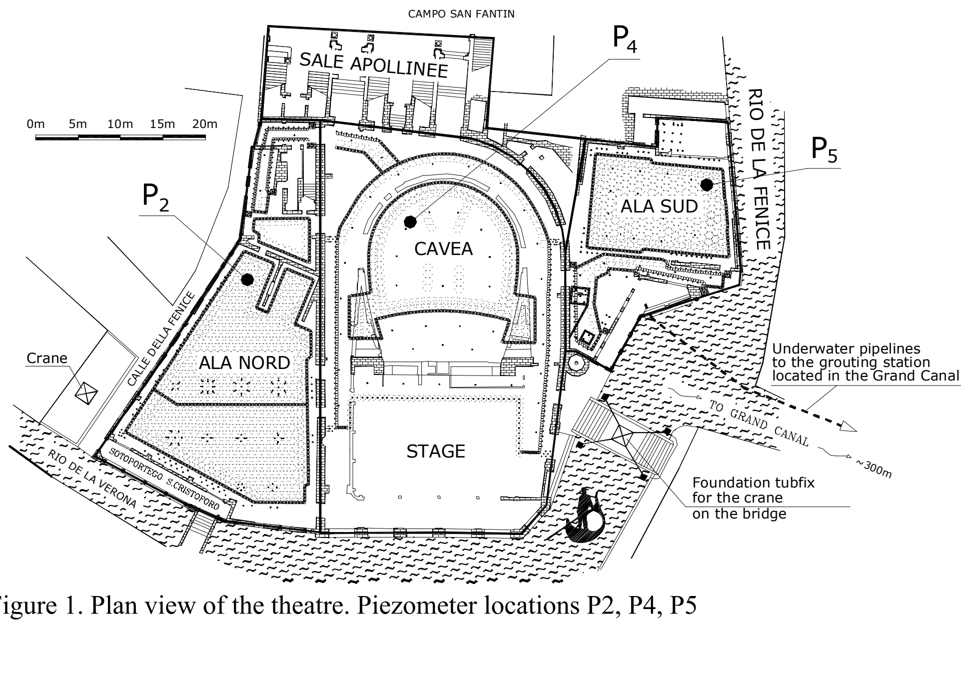

Figure 1. Plan view of the theatre. Piezometer locations P2, P4, P5

were to be rebuilt with new structures totally detached from those to be preserved.

The theatre may be divided into four main areas (figure 1):

- the main entrance and foyer (Sale Apollinee),where only conservation operations were carried out
- the theatre hall (Cavea) and the stage, where underground excavations were undertaken to place musical instruments and a new stage machinery
- a large building with dressing and technical rooms, warehouses and offices (Ala Nord), completely rebuilt
- a smaller building with a new rehearsal hall and other rooms (Ala Sud), to be rebuilt partly underground.

In such a situation it was necessary to create new underground spaces, down to -2,90 m below average sea level (0,00): the maximum hydraulic head at the bottom of the exacavation results equal to 4,30 m, considering a high tide rise up to 1,40 m a.s.l. (figure 3).

The consolidation and waterproofing works had to be planned taking into account several serious difficulties both of technical and of logistic kind. For instance: the closeness to the excavation of the wooden piles and of the masonry walls to be preserved (20-25 m high), a soft soil with weak mechanical characteristics, very limited space where to operate and store equipment, supply only by small boats, no use of large machinery.

The theatre is 300 m far from the Grand Canal and is surrounded by narrow canals (figure 1). Spaces were so limited that, for example, the grouting station had to be placed on a floating board in the Grand Canal itself.

Before the planned works, archaeological excavations put in evidence ancient structures such as masonry foundations, wooden piles, floors, wells, built prior to the theatre. The Authorities imposed the conservation of some interior masonry walls resting on very irregular foundations.

2 SOIL PROFILE AND GEOTECHNICAL CHARACTERISTICS

Soil investigations were carried out by means of borings, in situ and laboratory tests on selected samples. Permeability, at various depths, was determined by in situ Lefranc and laboratory oedometric tests. Since some old wooden piles under the old stage were pulled out after the fire, piezocone tests were performed to check the soil cone resistance. In the following table 1, the average values of the geotechnical characteristics are shown.

Table 1. Geotechnical characteristics

Soil Layers	Sand [%]	Silt [%]	Clay [%]	W_L [%]	W_P [%]	I_C	γ_n [t/m^3]	w_n [%]	N SPT	c_u [MPa]	E_{oed} [Mpa]	$k_{h\ oed}$ [cm/sec]
A	12	63	25	41.6	22.2	0.27	1.81	36.3	-	0.031	3.35	7.3×10^{-8}
B	83.5	13	3.5	-	-	-	-	-	20	-	-	-
C	2	69	29	38.0	22.5	0.54	1.90	29.7	-	0.055	9.60	7.9×10^{-8}

From the results obtained , soil profile was determined for each area. The stratigraphy can be summarized as follows (see also figures 3 and 4):- a filling layer 1÷2 m thick - a layer of soft clayey silt 5÷6 m thick (A) occasionally intercalated by sandy beds - a layer of slightly silty sand (B) 5÷6 m thick - a silty clayey layer (C) stiffer than the upper one.

3 WATERTABLE

In the involved area, water level was continuously monitored by piezometers located at different depths. From these data, correlations were drawn between water table and sea levels consequent to tide movements.

In the silty-clayey soils, the water level is slightly influenced by tide oscillations. On the contrary, in the sandy layers the water level varies nearly instantaneously, but the further from the canals the piezometers are, the more damped are the variations.

In figure 2, damping values of the hydraulic head taken from 3 piezometers versus the sea level and their linear interpolation are shown. The largest damping was observed in the most internal area of the theatre (Cavea), whereas the smallest one in Ala Sud close to Rio de la Fenice.
Introducing the water level assumed for the temporary work design (+1,40 m a.s.l.) the hydraulic head was obtained in the various zones for stability analysis against hydraulic uplift. For instance, in Ala Sud the damping value was 56 cm and the resulting water level was +0,90 m a.s.l..

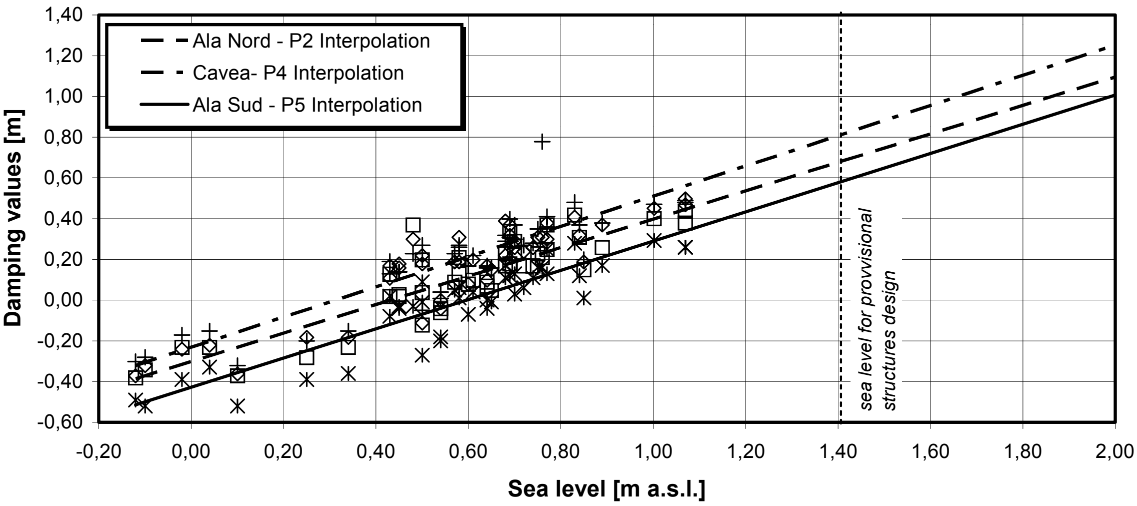

Figure 2. Difference between sea level and watertable levels in piezometers vs. sea level.

4 THE DESIGN OF FOUNDATION TREATMENTS

The difficulties above described and the necessity of preserving the high old masonry walls (20÷25 m high) made it necessary to find out particular, innovative, not invasive solutions suitable to avoid any damage or disturbance to old structures.
Reinforced concrete diaphragms were not feasible because of the dimensions of the rigs and the presence of wooden piles and old masonry foundations. Neither ordinary sheet piling was possible to be done for the same reasons and anyway because of its low inertia.

Then a combined solution was chosen (called "diaphragmed berlinese"):

- steel piles Ø 127 mm (10 mm thick), 40 cm apart making a structure suitable to support lateral loads, known as "berlinese".
- plain plastic piles Ø 230 mm placed among steel piles in order to achieve a continuous impervious barrier (see figure 3a).

As the old masonry walls had to be preserved and the soil bearing capacity was low, it was necessary to transfer the load of the new structures to the sand layer lying between -6 and -12 m u.s.l..

So, tubfix type micropiles (with grouted bearing bulb, obtained by repeated cement injections through "manchettes" valves) were carried out. These steel piles (Ø 101,6 mm and 12,5 mm thick) were also part of "berlinese".

Tubfix have both a temporary function since they carry the lateral loads and a permanent one since they transfer vertical loads to the sand layer. Therefore tubfix were made of galvanized steel, taking particular care during their execution (see figure 3 b).

In deep excavations areas the two-fluid jet grouting technique (cement mix + air) was used to improve safety against seepage and piping risks. Continuous block treatments were made by staggered columns 0,80 m apart in general; the spacing has been reduced to 0,70 m along perimetric strips close to old masonry walls.

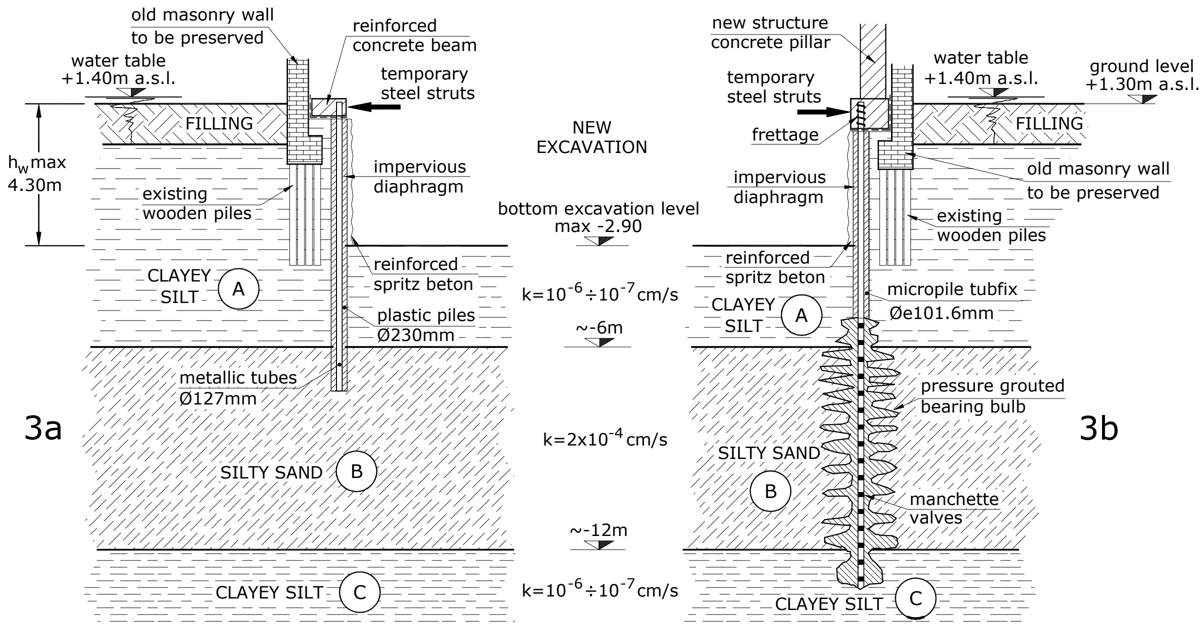

Figure 3. a): The "berlinese": impervious diaphragm wall retaining the lateral loads. b) Tubfix micropiles, inserted in the "berlinese", supporting the weight of new structure of the theatre.

5 BERLINESE AND GROUTED PLASTIC PILES

During excavation, steel piles bend under earth and water lateral pressure. In case of significant bending, soil between diaphragm and wooden piles is no more confined, causing stress release, so that old masonry walls could undergo dangerous differential settlements.

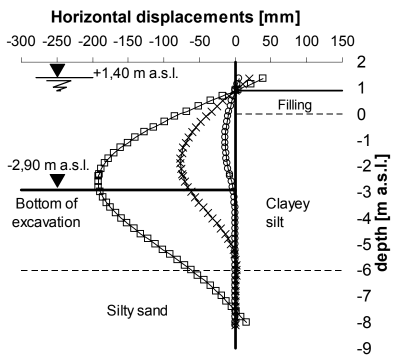

Figure 4. Calculated displacement of the "berlinese" piles

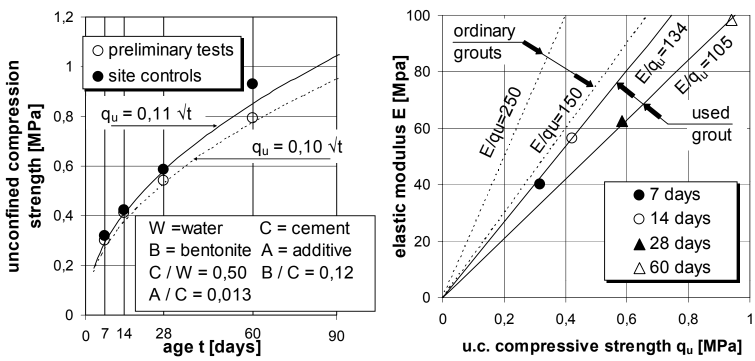

Figure 5. Plastic grouts mechanical properties
a) u.c strength vs. curing time b) E vs. u.c. strength

In order to reduce pile bending, the pile heads were connected by reinforced concrete beams supported by temporary steel struts and the mechanical characteristics of the soil under the bottom of excavation were improved by means of jet grouting treatments.

The treatment effectiveness was proved by finite element analysis. In figure 4 the computed horizontal pile displacements are shown; three cases were analysed: no jet treatment (⊟) , 3 m thick jet grouted layer above the pile bottom (✳) and 5 m thick jet grouted layer below the excavation bottom (⊖). The last solution was adopted since it strongly reduces pile bending. During the excavation, no significant displacements of masonry walls were shown by monitoring systems.

Steel piles could bend to a different extent causing vertical cracks right inside the grouted piles so that the diaphragm could not be impervious any more.

Therefore plastic piles Ø230 mm were cast using cement-bentonite grouts with an additive enabling to optimize the rheological and mechanical properties (stability, workability, lower elastic modulus to compressive strength ratio). In figure 5a mean values of strength vs. curing time t are shown, comparing the results of preliminary tests to those of the controls on samples taken on the site; the two sets of data are very much alike, showing an increase of q_u fairly proportional to the square root of t.

As regards deformability, the mean values of elastic modulus E are plotted in figure 5b vs. u. c. strength at different curing times. The E/q_u ratio (about 100÷130) shows a good relative deformability, enhanced by the additive, which allowed the use of the greatest bentonite/cement ratio consistent with workability and strength demands; in fact the E/q_u values for ordinary cement-bentonite mixes are sensibly higher as shown in the same plot (statistical range between 150 and 250).

6 FOUNDATION MICROPILES

The concrete pillars of the new structure are very close to the old masonry walls. In order to avoid the old wooden piles to be disturbed by the tubfix, the following design solutions were adopted:

- tubfix piles were placed further than 50 cm away from the old foundations
- bulbs were grouted approx. 2 m under the old wooden piles which support the old masonry walls.

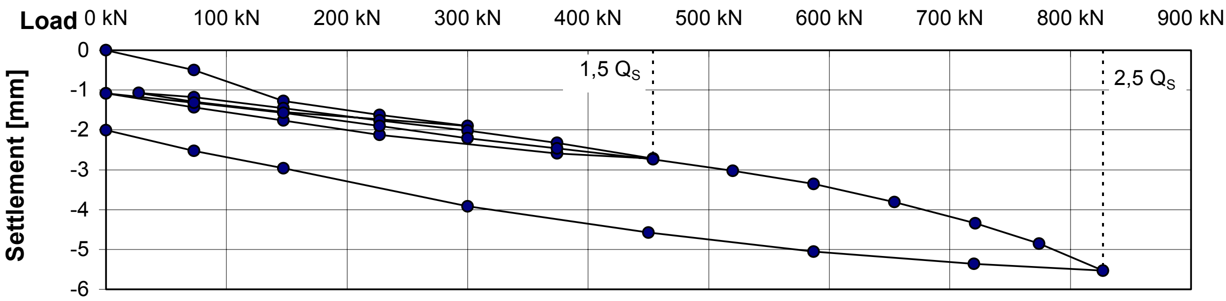

Figure 6. Tubfix micropile - Load test data.

In order to check the micropile bearing capacity, loading tests were carried out (see figure 6). Load was increased up to 830 kN that is 2,5 times the limit service load (Qs).

Injection pressures and grout volumes were carefully monitored for each "manchette". After each injection, the need of further quantity of grout was determined, according both to the grouting pressure reached and to the old structural monitoring data. No more than 3 injections per manchette were performed.

7 THE HORIZONTAL HYDRAULIC BARRIER

The horizontal cutoff has been realized where the excavation was too deep to assure safety against piping and excessive seepage.

In order to optimize the operating parameters for the selected two-fluid jet grouting technique, a field trial has been carried out. Three terns of treated soil columns have been executed at 0,8 m c/c (according to design) with variable specific volume VM of injected cement grout (see table 2).

After about one month three corings have been performed in each tern (A in the center of one column, C in the tern center, I inclined 20% to the vertical, crossing a couple of columns) with recovery of numerous undisturbed samples.

Table 2. Bifluid jet grouting parameters (Ø 100 cm columns)

		Cement grout (C/W = 1)			Air		Linear specific energy		
Tern	V_{ris} [m/h]	P_m [MPa]	VM [m^3/m]	Q_m [m^3/h]	P_a [MPa]	Q_a [m^3/h]	E_{sm} [MJ/m]	E_{sa} [MJ/m]	E_s [MJ/m]
1	16,9	35	0,400	6,75	1,00	200	14,00	3,93	17,93
2	15,6	35	0,450	7,00	0,85	200	15,75	4,19	19,94
3	13,7	35	0,500	6,85	0,85	200	17,50	4,39	21,89

In figure 7 mean values of coring parameters RR and RQD are compared.

On an average the coring quality trends to improve with VM increasing; at a parity of VM, the core recovery data of vertical borings are fairly similar.

These results point out the continuity of the treatment obtained with the design spacing.

The laboratory tests have given in summary the following results:

- bulk density increasing with depth from 16 kN/m^3 to about 17,5 kN/m^3
- a similar trend as regards strength (figure 8) with general mean values increasing from 1,2 MPa in clayey silt to 2,8 MPa in the underlying sandy layer.

The higher strength values have been obtained in type A corings, the lower ones in inclined borings I. On an average, strength is of the same order in terns 2 and 3, and sensibly higher than in the first tern. According to these results, the jet grouting parameters of tern 2 have been chosen.

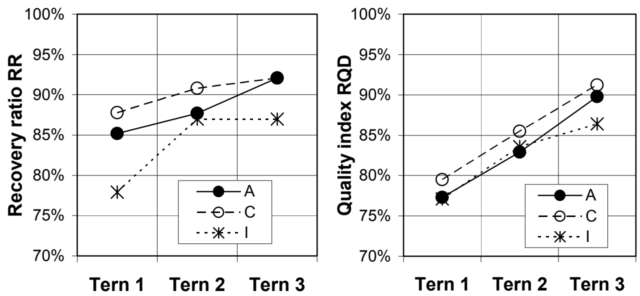

Figure 7. Jet grouting field trial: average core recovery parameters (control boreholes)

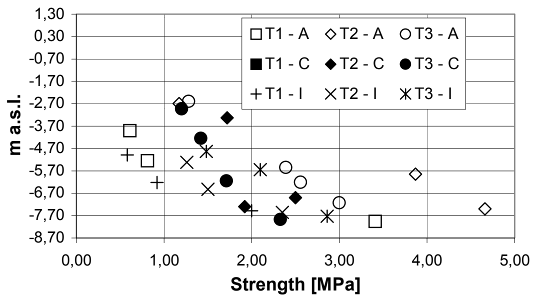

Figure 8. Jet grouting field trial: specimen strength vs. depth

8 CONCLUSIONS

Works were carried out regularly. A few problems were met only during jet grouting under the Ala Sud wall close to Rio de La Fenice: anyway damages to masonry were prevented by continuous monitoring.

All excavations were carried out with no water seepage even where the hydraulic head reached its maximum value.

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