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Structural preservation of Bologna National Gallery resulting from serviceability alterations

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Abstract

The paper focuses on the foundation strengthening of the Bologna National Gallery actually completed; this work, motivated from the extension of the visitor walks in the cellar floor, will provide extra space for the general reconsideration of the whole Gallery.
Several hints concerning the statement of appropriate execution procedures, analysis models and control tools are presented; all proposed methods are characterised by the common rule to enforce the safety margin above a sufficient level, though the lack of initial data which could be overcome only in the course of execution stages.

1 Introduction

The National Gallery in Bologna (fig.'s 1.1-3), is a very complex building whose history encompasses many centuries. Starting from the original structure of the XIIIth century cloister, each age has added up its own part until the late 1960.
In the last decade, the demand for common spaces where to develop timely expositions, and the need to rearrange vertical and floor routes in order to connect the permanent exposition rooms in a consequential way, led the involved Institutions to start with a global project focused on the creation of an integrated system of adjustable spaces, able to be organised by the users in several different ways.
Under the requirements of the Italian Ministry of Archaeological, Architectural and Cultural Heritage, owner of the National Gallery, the architects of Panstudio and the structural engineers of Studio Associato di Ingegneria Strutturale proposed several solutions; in the 1989 CO.RE.BE.A., a joint venture

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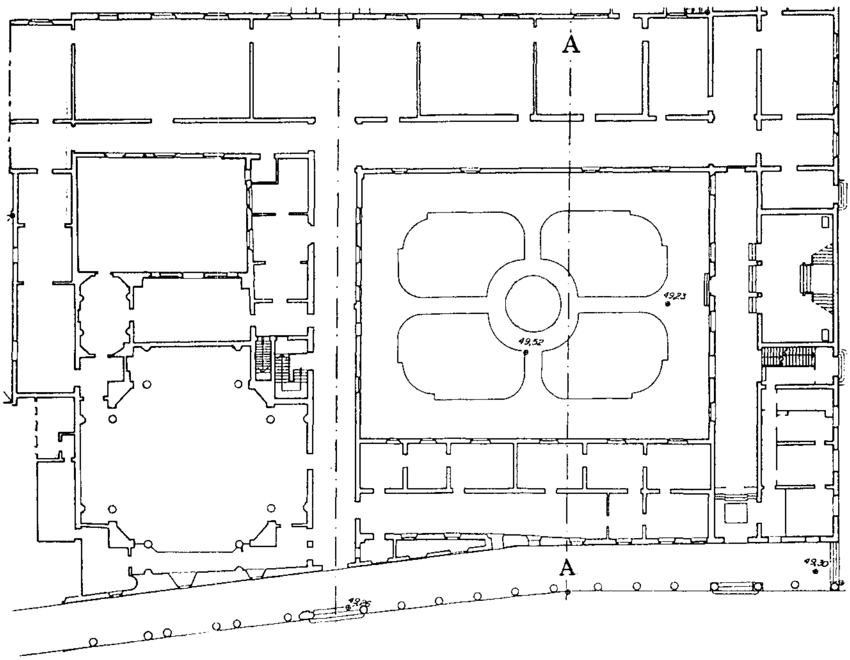


Figure 1.1: Ground Floor Plan of National Gallery and Fine Arts Academy

Figure 1.2: Main view of the building with the characteristic portico of Bologna

of licensed companies, started the works under grants of the Italian Found for investments and workmanship.
The main idea of the design reconsidered the cellars as a widening of the Gallery; although cellars have valuable brick masonry walls and cross vaulted ceilings, the under ground level position and the limited height required a strong ambient control that could be achieved only widening the geometric size of the cellars; so we decided to plan the digging out of nearby one meter of the cellar ground and completely of two internal courts.
Unfortunately, a careful check of the foundation structures showed that they would be largely unsafe after the digging works; moreover, the need of space for plant pipes and ducts required extra excavations for a system devoted tunnel.
As a consequence, once the relevant geometric and mechanical data on wall consistency and soil properties were collected, we completed the design of a generalised strengthening of foundations by means of diffuse piling and complex reinforced concrete connecting structures which contained also the piping distribution tunnel (fig. 1.4).
This complex system of restoration works interacted very deeply with the existing masonry wall feet and the surrounding soil, highlighting also strong weather and hygrothermal settlement dependency.
One of the major design concern was the selection of the final ground level in the cellars; in fact the wall foundation feet lie at very different levels so, fixing this value was a matter of careful equilibrium between the final architectural result and the underpinning work required to strengthen the foundations grounded above the final excavation level.
In order to maintain a consistent safety margin during all the construction stages, we prepared a detailed operation sequence program; all critical point of the construction where cracks have been detected in the preliminary examination, were kept under observation by means of deformometric bases. The Laboratory of the Civil Engineering Institute, University of Bologna carried out and certified the measurements during all the construction stages.
Actually, being the structural works in their final stage, the measurements show only a little seasonal evolution, which gives credit to a fully stabilised settlement condition.
In what follows we present in some detail the features of the design and construction methods utilised, the problems encountered, the solutions adopted.

2 Review of The Execution Phases

Despite the need to have data before the final design starts, in a complex restoration work this is possible in practice only rarely; so, it is important to draw flexible execution techniques that can agree with almost all the practical situations which we forecast to be relevant at a particular stage of execution.
After some discussion, we arrived at the following execution sequence:
a) general data collection (wall and soil characterisation),
b) strengthening of the walls by means of steel reinforced injections,
c) execution of the diffuse micro piling,

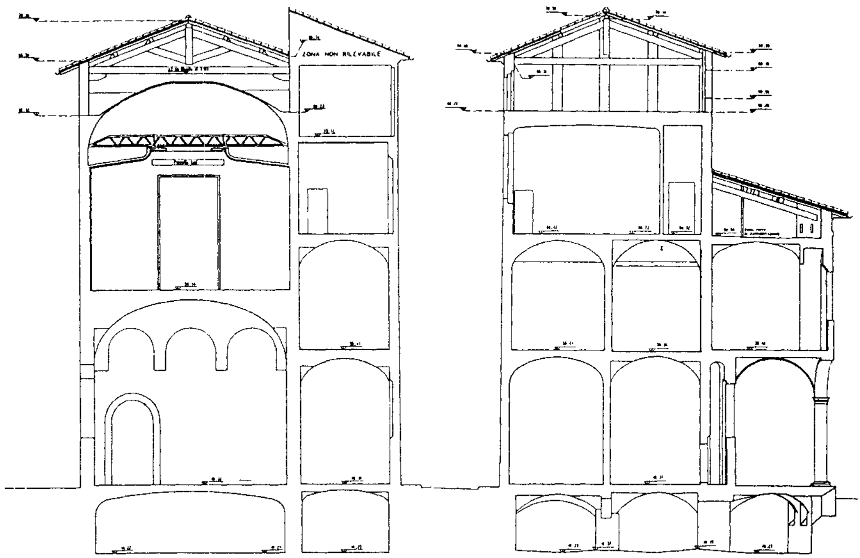


Figure 1.3: Section A-A of the two main fabrics of the building laying around the court

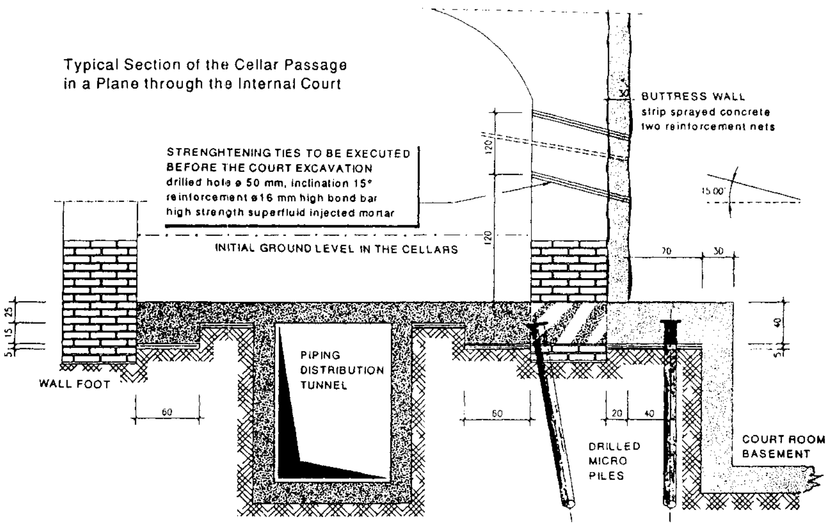


Figure 1.4: Typical section of the restoration works for the walls near the courts

surface layer, activated a drying shrinkage phenomenon that widened the cracks of the walls and columns not yet fastened to the micro piling system.
The increase and decrease of crack opening followed the weather conditions until the floor R/C slab stopped evaporation and clamped the walls, leading finally to an irreversible constant amplitude of the remaining cracks. However, it is to stress that rehabilitation works in Italy progress extremely slow, due to the difficulties in the economic management of the project.

3 Safety Evaluation of the Wall-Soil-Pile System

A very important point in the computation effort has been the evaluation of the extent needed for the piling system in order to guarantee the stability of the foundation. Unfortunately, a careful check of the foundation structures showed that they would be largely unsafe after the excavation, if no remedial work will take the place of the removed soil layer.
But it is easy to understand that the load is transferred to the piles with a certain time delay, depending on the level of consolidation attained in the soil; moreover, the piles cast in the neighbourhood of the wall develop their friction forces directly into the load diffusion cone of the wall basement. So, the usual design rule that either the piles or the basement must withstand all the load is largely on the safe side, and a more appropriate evaluation is required.

3.1 Preliminary design of the micro pile

The strengthening design of the foundation has based on the general concept that a masonry large building shows a complex organisation like a body; thus the design criteria must fit with the level of structural coherence and do not introduce strength (and henceforth stress) concentrations.
Following this we proposed highly diffused micro piles of limited length; the main data of a total of nearly 800 cast micro piles were:
- piles ø 150 mm diameter and 8.00 m length from the cellar level,
- average spacing among piles of 2.00 m,
- drilling inclination of the pile (due to vaulted ceilings), 10° out of vertical,
- reinforcement made with an encased steel tube ø 127 mm diameter and thickness 11 mm; the tube included also the valves necessary to injection.

Moreover, due to the high pressure injection casting of the high slump mortar, we adopt a magnification factor of the concrete section diameter of 1.2.
The soil properties in undrained and drained conditions were determined from 13 sampling points accurately distributed in the cellar area. In the following fig. 3.1 the main soil parameters are computed from the available data.

3.1.1 Undrained pile limit load: using the undrained cohesion value c_u = 125 kN/m^2 and a soil alteration coefficient of 0,75, we evaluate the total limit load of a pile [1]:

$$Q'_{ult} = Q'_{lat} + Q'_{base} = 1.2 \cdot (\pi \, D \, \alpha \, c_u \, L + \pi \frac{D^2}{4} N_{co} \, c_u) \, .$$

where N_{co} in this case equals 9; finally Q'_{ult} results 432 kN for a pile.

d) digging of the ground level at the final design depth,
e) casting of the R/C beams connecting the micro piles to the walls,
f) excavation of the trench required for the tunnel devoted to the piping nets,
g) casting of the tunnel with the upper slab connecting it to the R/C beams,
h) digging out of the two external courts,
i) erection of the roofs of the rooms completing the underground net,
j) execution of the non structural finishing (partitions, pavements, coatings).

In addition, before to start with phase d), the observation points were selected and monitored with mechanical gauges.

A very general fact that cannot be overemphasised is the strong link between design evolution and advancement level of works; indeed the preliminary design had a high degree of fuzziness which could be removed as far as the works stepped forward. More precisely the data yielded from each execution phase helped us to fix the extent of the structural parts to be done in the following step, till the final form of the whole strengthening design.

As an example, we fixed the excavation depth on the basis of few samples made all around the cellar's floor; after, we carried out a preliminary excavation to 70 % of the stated depth, and a monitoring of the foundation bases uncovered at this stage. Finally, extrapolating some new samples we fixed definitely the excavation depth to 90 % of the initially stated value.

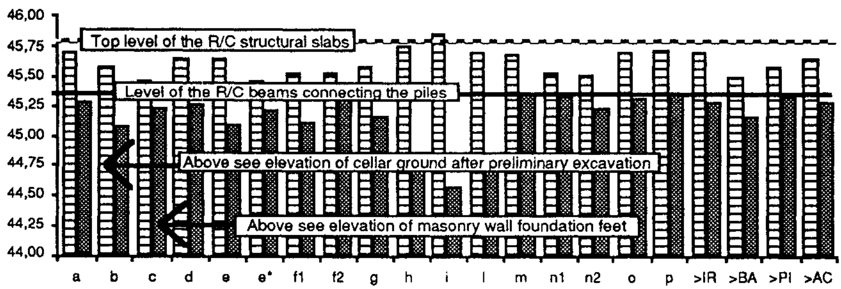
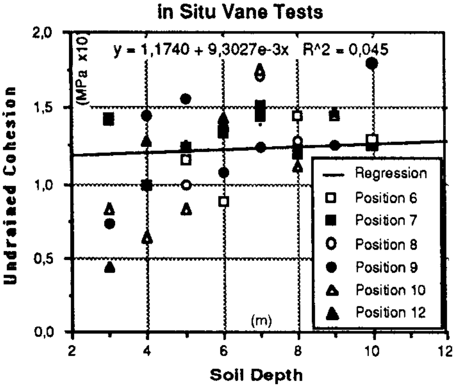


Figure 2.1: Soil elevation data for the rooms composing Gallery's cellars

A second important feature of the phase development, is the subdivision of the whole work in a number of modular separated yards that allow for a spatial and temporal characterisation of the unsafe zone. So, all drilling and digging works have been divided in a number of homogeneous sub domains, having care to continue in one new part only when the neighbouring parts were properly stabilised. Moreover, in order to avoid excessive density of dangerous operations, we shifted also in time the critical phases under execution in different yards.

During the development of this very complex restoration work we recognised clearly the coupling between structural damage index (such as crack openings) and hygrothermal conditions; this type of behaviour is a direct consequence of the very sensitive nature of the fine over consolidated clay which constitutes the main layer under the National Gallery foundations.

In fact the initial wet isothermal condition of the cellars was suddenly changed in the course of work; ventilation due to openings and the removal of the soil



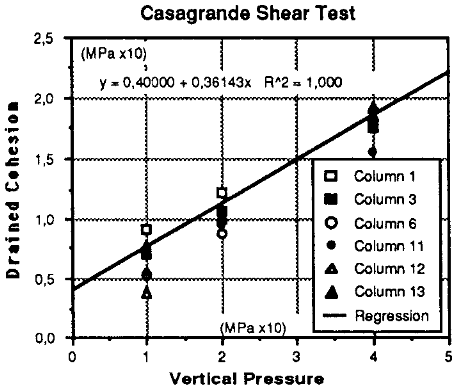


Figure 3.1: Regression used for soil parameter evaluation

3.1.2 Drained Conditions: the design of the pile at infinite time requires the knowledge of the vertical stress distribution, the horizontal pressure and the friction coefficient in the soil layers spanned by the piles; as a preliminary evaluation we can use the following formulation:

$$Q''_{ult} = Q''_{lat} + Q''_{base}.$$

$$Q''_{lat} = 1.2\,\pi\,D \cdot \left(c_d\,L + K_H\,K_F \int_0^L \sigma_{ov}(z)\,dz \right),$$

$$Q''_{base} = q_{lim} \cdot 1.2^2 \cdot \frac{\pi\,D^2}{4};$$

the horizontal pressure and friction coefficients can be computed in the form:

$$K_H = 1 - \sin(\varphi)\,, \qquad K_F = \tan(\varphi)\,.$$

The main function entering in the above reported integral is the lithostatic pressure, which is approximated as constant in its diffusion cone (fig. 3.3) :

$$\sigma_{ov} = \frac{F}{a + 2\,z\,\tan\delta} + \gamma\,z\,.$$

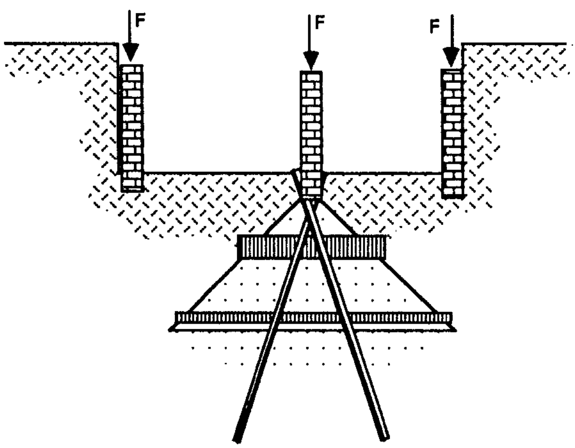


Figure 3.3: Soil vertical pressure approximation for pile lateral load evaluation

With the data a = 1.0 m, tan δ = 1 and performing the required integration we arrive at:

$$Q''_{lat} = 1.2\,\pi\,D \cdot \left\{c_d\,L + K_H\,K_F\left[\frac{F}{2}\ln(1+2\,L) + \gamma\,L\right]\right\},$$

which gives the value $Q''_{lat} \approx 270$ kN.

For the ultimate head resistance we make reference to the Terzaghi formula [1]:

$$q_{lim} = \left(7.7\,\gamma + \frac{400}{1+2{\cdot}7.7}\right)6.4 + 40{\cdot}14.83 = 1680\ \text{kN/m}^2,$$

$$Q''_{base} = 43\ \text{kN}.$$

Hence, the limit pile load in drained conditions holds 313 kN.

3.2 A more refined model for pile-soil-wall analysis

In reality the load carried by the base-pile-soil system is mainly composed of permanent load which, perhaps, is already diffused by the base alone; so it is interesting to examine the real safety coefficient descending from the very collapse situation. In this case the yielding loads of both base and pile can be summed up introducing the hypothesis of some soil ductility and arriving so at a kinematical rigid-plastic limit load of the Terzaghi-Hill type [2].

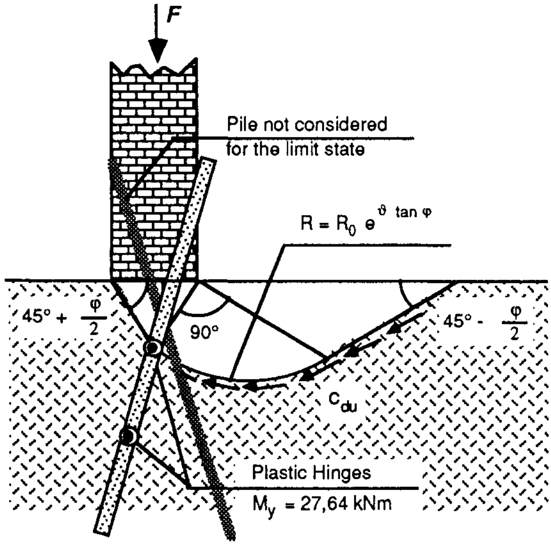


Figure 4.2: Schematic representation of the velocity field for the limit analysis

The model represented in fig. 4.2 is a very crude approximation of the real collapse mechanism, but it allows anyway for a sensible evaluation of the safety index; in fact, the presence of piles with alternate inclination leads to this type of velocity pattern only in the case of local collapse of at least one single pile spacing Δ which, perhaps, can arise only once the masonry wall is in turn collapsed into several strips separated by cracks.

The limit shear force of the pile can be computed imposing that the limit shear itself and a plastic bending moment, applied both at the tip section, will generate a plastic hinge in another (yet a priori undetermined) section.

For sake of clarity we outline the involved relationships:

$$M(x) = T_u \frac{\psi(x)}{\alpha} - M_y\, s(x) = M_y\,,$$

where: $$\alpha = \sqrt[4]{\frac{k_s\, D}{4\, E_s\, J_p}}\,,$$

$$\psi(x) = e^{-\alpha x} \sin(\alpha x)\,, \qquad s(x) = e^{-\alpha x}\left[\sin(\alpha x) + \cos(\alpha x)\right].$$

In the proposed example the second plastic hinge is located approximately 1,0 m below the soil surface. We can then express the safety condition with relation to the external and resisting forces acting on a length Δ:

$$\gamma_{pl} = \frac{M_{Rtot}}{M_S}\,,$$

where: $$M_{Rtot} = M_R(T_u) + M_R(c_{du}) = T_u\, z + \Delta \cdot \left(N_c\, c_{du} \frac{B}{2}\right),$$

$$M_S(F) = \frac{F\, B}{2}\,,$$

$$N_c = \left[\tan^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) e^{\pi \tan\varphi} - 1\right] \frac{1}{\tan\varphi}\,.$$

The Italian Code of Practice requires a minimum safety coefficient of 3.0 for shallow foundations, of 2.5 for deep foundations, and a safety ratio of 1.5 for this last type of soil stability analysis.

4 Discussion of Monitoring Results

As previously cited, several types of measurements were performed by the Laboratory of the Civil Engineering Institute during the execution of the various excavation and strengthening phases; moreover the ultimate resistance of several piles was checked by direct load test.

In fig. 4.1 we present the evolution of four deformometric bases each arranged with three mechanical gauges mounted onto the cracks appearing in the walls of the passage around the internal court at the ground level plan shown in fig. 1.2.

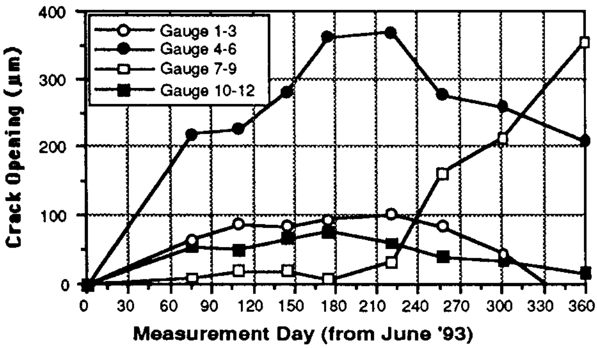


Figure 4.1: Crack opening evolution at the ground floor passage of the Gallery

As a matter of fact we can see in the diagram two opening ramps: the first, collocated in the summer '93 can be ascribed to the very dry condition of this

period; indeed the works into the cellars have been completed sometime ago and all creeping effects of these works at that time were all off.
The second increase, collocated in the spring '94, can be substantially motivated with the execution of the digging works (for a total volume of 4000 m^3) in the main internal court.After the drying shrinkage has achieved its maximum value, in the second semester of '94 all cracks remained to constant values.
All control tools used when the work was in progress demonstrated the unquestionable utility of keeping timely measurements, despite the difficulty to take into account all relevant randomness sources in the construction of a framework displaying the observed effect.
Finally, it is to mention that, a large part of decisions in the critical points of restoration work, have been supported with the extra safety margins gained with a continuous control of the crack opening evolution.

5 Conclusions

The very large amount of data necessary to set a careful design and the number of critical decisions involved in a complex restoration work like the National Gallery in Bologna require a deep understanding of the interaction phenomena.
Masonry walls, soil, water and atmosphere create a system in a metastable equilibrium that remain unchanged under slowly varying conditions for several centuries; the same equilibrium is likely to be destroyed in a few if one too of the components, under external actions, alters its normal behaviour.
In the preceding discussion we illustrated the criteria defined for the preparation of the design and for the control of the results in a practical example; the developed ideas allow for the extension of the presented procedures to other cases, where the execution techniques can lower considerably the safety margin.
Finally, the strong operational evidence of the feedback from the work advancement, to the sensing instruments, up to the design details, has been demonstrated.

Aknowledgements
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