# Strengthening of the Parish Church in Farigliano

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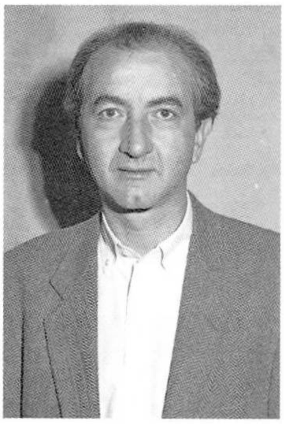
## Consolidation de l'église paroissiale de Farigliano

## Verstärkung der Pfarrkirche in Farigliano

**Renato LANCELOTTA**
Prof.
Politechnico di Torino
Torino, Italy

**Giuseppe PISTONE**
Dr. Eng.
Politechnico di Torino
Torino, Italy

**Renzo ROVERE**
Dr. Eng.
Public Works Ministry
Roma, Italy



**SUMMARY**

The foundations of the Parish Church of Farigliano are dug in soft, recently deposited materials with a high degree of compressibility. This explains the precarious behaviour exhibited in recent years by the static conditions of the structure. This report describes the urgent measures taken to protect the building, including the installation of a hydraulic levelling system.

**RÉSUMÉ**

Les fondations de l'église paroissiale de Farigliano reposent sur un sol souple constitué de dépots récents et ayant un degré élevé de compressibilité. Cela explique l'état précaire des conditions statiques de la structure au cours de ces dernières années. Ce rapport décrit les mesures d'urgence prises afin de protéger l'édifice, y compris l'installation d'un système hydraulique de mise à niveau.

**ZUSAMMENFASSUNG**

Das Fundament der Pfarrkirche in Farigliano wurde in einen weichen, aufgeschütteten Boden mit hoher Kompressibilität gegraben. Das erklärt das prekäre statische Verhalten des Gebäudes in den letzten Jahren. Die vorliegende Abhandlung beschreibt die notwendigen Massnahmen, um das Gebäude zu schützen, einschliesslich eines hydraulischen Hebungssystems.

## 1. INTRODUCTION

On February 29 1887, the ancient parish church of Farigliano was severely damaged by an earthquake and its stability was irremediably impaired. In his report of December 1887 to the Parish Priest, Ing. Chiechio, the designer responsible for static analysis, recommended the construction of a new church precisely on account of the exceedingly high costs to be incurred for the consolidation of the ancient building.

Having selected the site for the new church, it was immediately discovered that the soil was hardly suitable to carry the considerable loads to be transferred by the building (see the report of October 18, 1888, parish archives). As a result, under the guidance of In.g Chiechio, the entire area was excavated to a depth of 5 m, and 5 m long durmast wood piles were driven into the ground to correspond with the impost faces of the foundations.

From the archive documents, it can be seen that the site had been selected for various reasons, none them, unfortunately, of a technical nature.

The church was erected very rapidly and was completed in 1890 (fig. 1).

The building is fashioned in the shape of a Latin cross with a three-nave front body and a polygonal apse; at the confluence of the transept and the central nave rises the presbytery, surmounted by an octagonal drum covered by a dome (fig. 2).

Under the entire church floor there is a basement obtained from the digs carried out initially to improve the soil.

From the structural standpoint, the building has outer bearing walls of masonry made of pebbles and air-hardening lime, interrupted at regular intervals by horizontal brick courses, spaced about 80 cm apart, featuring excellent workmanship and strength.

Thick, continuous brick walls also define the entire presbytery area inside, while the naves are delimited by thin marble columns surmounted by arches and by a longitudinal masonry wall which defines the central nave. The columns continue down into the basement with square section brick piers.

The roofing of the basement consists of 12 cm thick triangular vaults and arches; "in folio" vaults cover the side aisles, while the central nave is topped by a wooden lacunar roofing.

These elements add up to a box section possessing great stiffness in the portion comprising the apse and transept, while the front portion of the building above ground is rather weak on account of the poor horizontal rigidity offered by the slender columns, the thin vaults and the lacunar covering.

## 2. STRUCTURAL DAMAGE

From the very outset, diffused fissures appeared in the thin vaults of the side naves, in addition to cracks opening in different points of the curtain walls.

In 1988, the settlements of the structure - which were already quite evident - were compounded by the flooding of the basement.


Fig. 1 - View of the church.

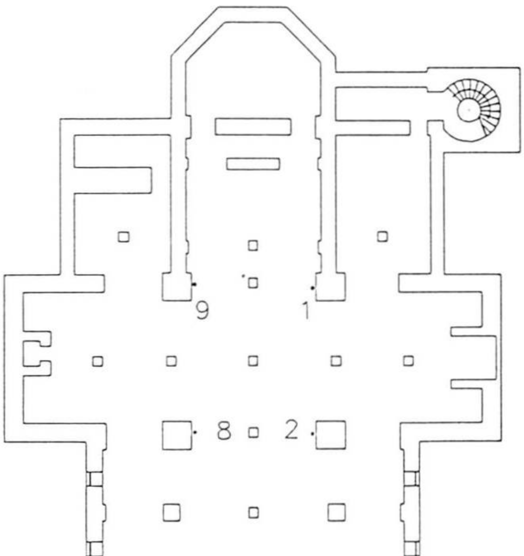



Fig. 2 - Plan of the basement showing the arrangement of the hydraulic levelling system measuring points.

Fig. 3 - Fissures in the thin vaults of the side aisles.

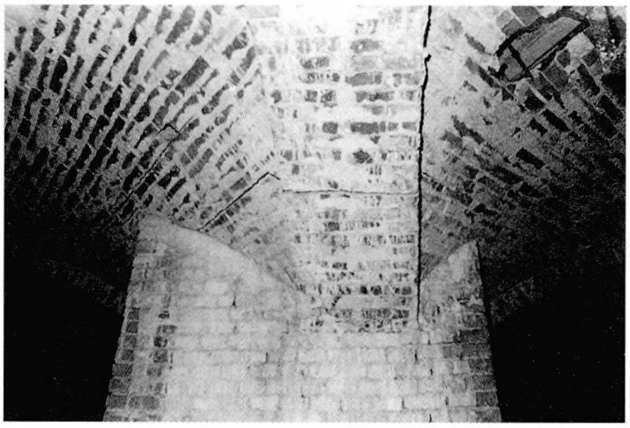


Fig. 4 - Fissures in the the 12 cm thick vaults on the underground level.

Fig. 5 - Fissures in the the wall defining the central nave.

At this point the overall picture was as follows:

- systematic fissures in the thin vaults covering the side aisles, of ancient origin, but probably made worse by recent damages; their width came to over 4 cm and their position in the building's plan revealed a markedly symmetrical arrangement; in particular, their 45° degree slant isolated, in each vault, a central part which was obviously in compression, as borne out by the relative slip of the crack edges and its enhanced curvature (fig. 3).
- a series of lesions, of recent appearance, involving the vaults and arches in the basement. The most important cracks were located all around the four main piers at the confluence of the transept and the central nave, and were as wide as 19 mm at the arches (fig. 4);
- a few major cracks affecting the church proper, in the wall at the top of the columns defining the central nave (fig. 5);
- full height lesions of more recent formation (August 1989) in the right-hand side lateral wall of the presbytery, where the transept meets the nave, and in the facade wall.

## 3. SOIL FOUNDATION CONDITIONS

From the outset, the pattern of lesions and relative settlements observed in the building structures was ascribed to the conditions of the subsoil. From the geological viewpoint, the foundation soil is on the edge of an alluvial terrace overlying the stiff clay of the nearby valley (of Miocene origin). Part of this terrace is of a colluvial nature, and is made of soft materials with great compressibility.

The data collected from 4 bore-holes have shown that the upper stratum of this recent silty clay of low to medium plasticity has a thickness of 17 m. Because of the chaotic structure of this foundation soil and on account of the scarce significance of laboratory tests performed on small samples, in situ compressibility has been evaluated through a back-analysis of the settlements observed in a lower level wall. This has yielded an operational value of soil stiffness as low as 1.5 MPa, which explains the large values of the cracks, fissures and settlements observed in the masonry structures.

Fig. 6 - Collegamento tra basamento del pilastro e testa dei micropali.

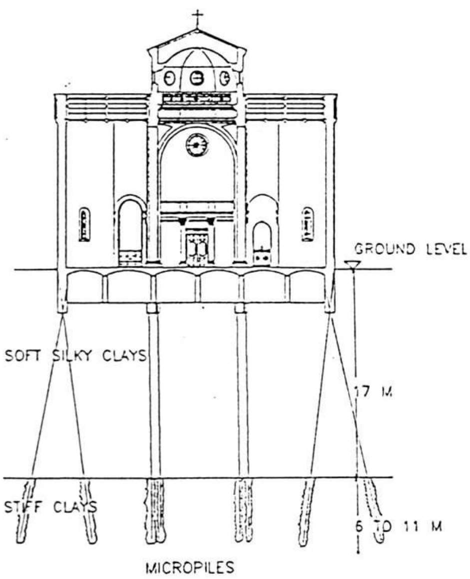



Fig. 7 - Cross-section at the transept showing the stratigraphic composition of the soil and micro-piles.

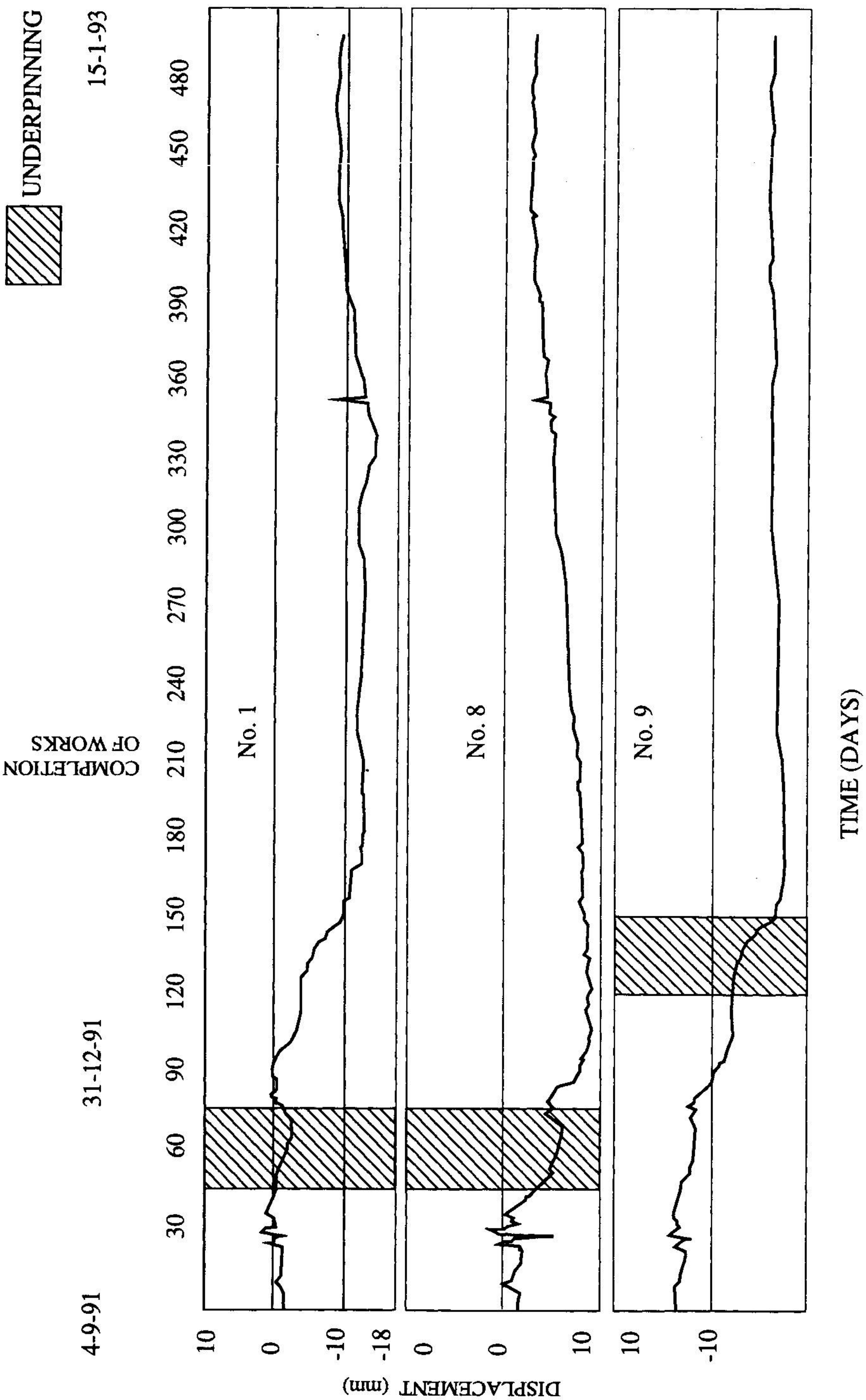

UNDERPINNING
15-1-93
31-12-91
4-9-91
COMPLETION OF WORKS
30 60 90 120 150 180 210 240 270 300 330 360 390 420 450 480
No. 1
No. 8
No. 9
TIME (DAYS)
10 0 -10 -18
0 0 10
10 -10
DISPLACEMENT (mm)

Fig. 8

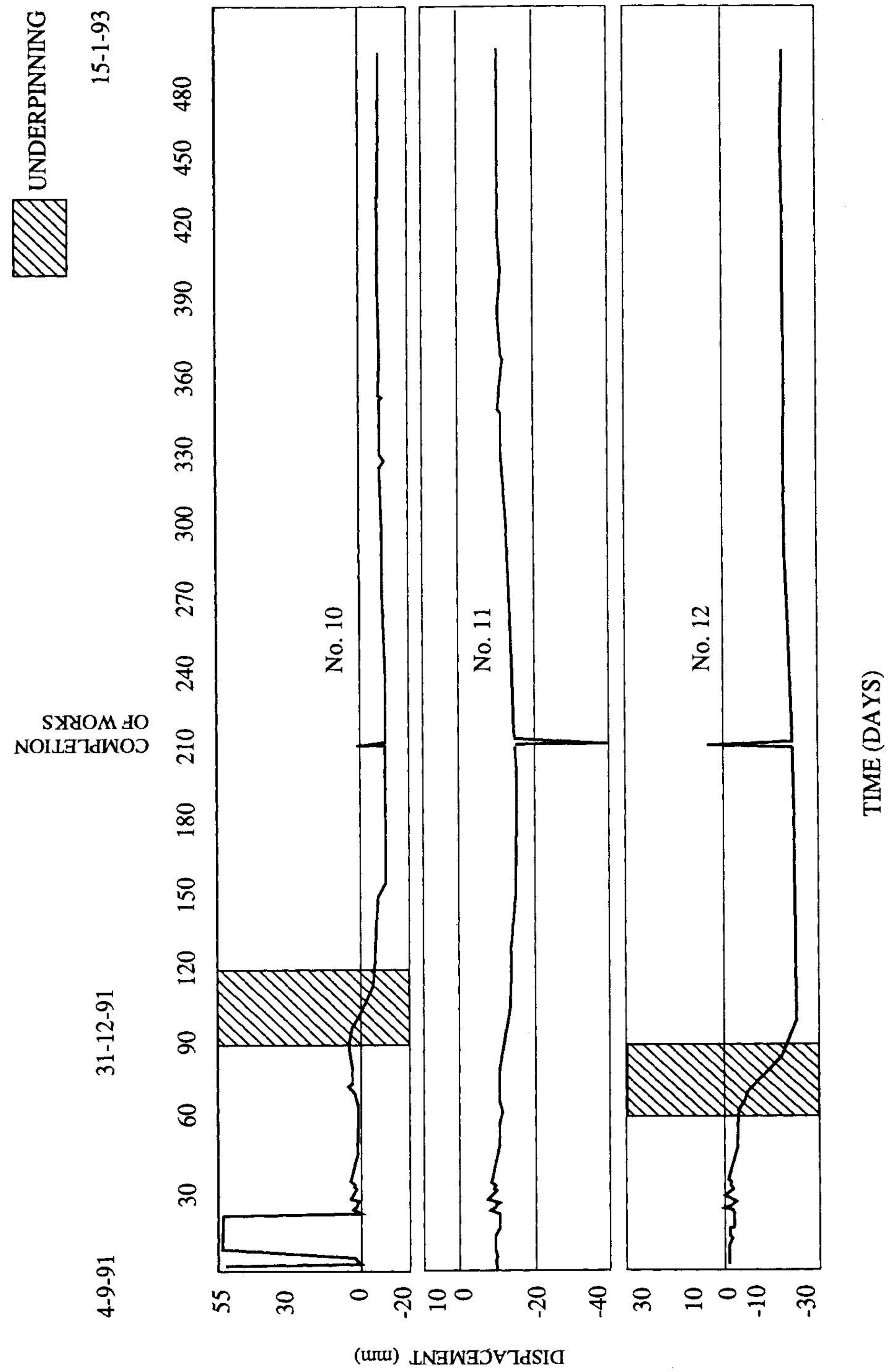

UNDERPINNING
4-9-91
31-12-91
15-1-93
COMPLETION
OF WORKS
30 60 90 120 150 180 210 240 270 300 330 360 390 420 450 480
No. 10
No. 11
No. 12
TIME (DAYS)
55 30 0 -20
10 0 -20 -40
30 10 0 -10 -30
DISPLACEMENT (mm)

Fig. 9

In this situation, the first major protective measure taken was the underpinning of the foundations by means of micro-piles. To prevent the formation of undesirable lateral loads as would be produced by down-drag if the upper layer settled more than the micro-piles, raking piles were not used and the vertical piles were connected to the masonry foundations through reinforced concrete slabs (figs. 6 and 7).

The load applied to each of these micro-piles ranges from 130 to 400 KN and the negative skin friction varies between 120 and 200 KN.

According to these loads, the bearing length in the lower stiff-clay ranges from 6 to 11 m. In addition, in view of the large difference in settlements necessary to attain the desired resistance, only skin friction has been taken into account, neglecting base resistance, so as to prevent the occurrence of further movements of the structure.

## 4. MONITORING AND RELIABILITY EVALUATION OF THE PROTECTIVE MEASURES

Systematic measurements of the vertical settlements of the columns and lateral walls started in June 1991.

Figure 2 shows the arrangement of the hydraulic levelling system and the most significant diagrams are shown in figs. 8 and 9.

Due to the re-activation of the movements in 1988, the first part of these diagrams clearly shows that the settlements were still evolving in 1991.

The time periods during which underpinning work was underway are also indicated by shaded portions of the diagrams and it can be seen that after the work was completed the structure did not suffer from any further movement.

A few additional remarks should be made: first, in order to prevent down-drag, the bearing length of the micro-piles was limited to the lower part of the stiff clay stratum. This required a careful control of the injection pressure in the upper part, so as to reduce soil disturbance and prevent any increase in pore water pressure with subsequent consolidation.

Secondly, the settlement pattern shown here has to be understood in terms of relative settlements, because the values given are referred to point 11- a fixed point in the facade wall which certainly underwent minimal movements. This explains, in some cases (see points 1 and 8, for example), the unusual apparent reduction in the extent of the settlements with time.

## ACKNOWLEDGEMENTS

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