Structural Monitoring of the Mexico City Cathedral

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Summary

A large monitoring system has been implemented to measure the structural response of the Mexico City Cathedral, which is being submitted to a rehabilitation program aiming at correcting a significant part of the differential settlements that severely undermine its structural safety. In addition to conventional survey, an automatic, on-line system has been installed primarily to measure out-of-plumbness and changes in span of arches and vaults. Stresses in critical columns and walls are being measured by the flat-jack technique and compared with those obtained from analytical models. Stress measurements have indicated high compressive stresses in some column sections whose capacity is near to be reached.

1. Introduction

The Mexico City Cathedral has been severily affected by differential settlements experienced by its foundation since the very beginning of its construction. Initially, large settlements were caused by soil consolidation due to the extremely large weight of the monument (1.27 MN) built on the very soft clay deposits of the Mexico City Valley. More recently, the intense pumping of the acquifer has been the main cause of subsidence. The monument was erected on the remains of Aztec temples and pyramids, which had preconsolidated the soil under some parts of the construction while in the rest the soil remained much softer, thus giving rise to large differential settlements. A front view of the Cathedral and of the adjoined Sagrarium is shown in Figure 1, along with a soil profile.

Subsidence has produced structural damage since the time of the construction of the temple, and an almost uninterrupted repair activity has been required, since the age of the temple construction. In recent years, the distortion of the building was such that its structural safety was severely undermined. The differential settlement between the Southwest corner and the Apse reached 2.42 m in 1991 and was increasing at a rate of 12 mm/year. Some of the main columns supporting the roof showed an out-of-plumbness near to 3%, and severe cracks in roof, floors and walls constantly reappered, in spite of frequent repairs. The total settlement of the monument since the beginning of its construction has been estimated as 7.5 m.

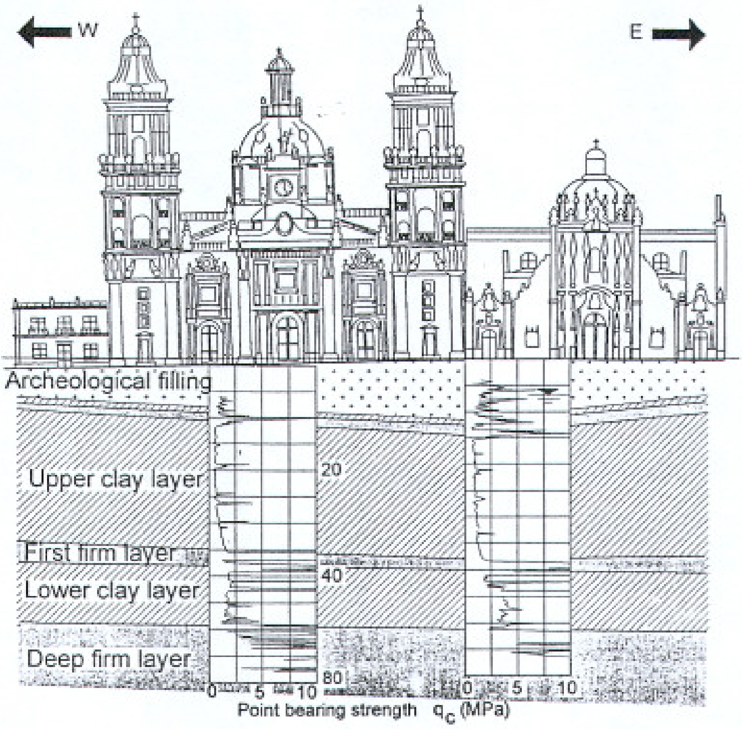

Figure 1 Front view and soil profile of the Cathedral and the Sagrarium (right side)

Considering that the regional subsidence of the Mexico City Valley could not be elimintaed in the near future and, thereofre, the present trend and rate of differential settlements will persist, a major rehabilitation project was started in 1991. The geotechnical aspects of the project are described in Ovando et al. (1996), and the structural aspects in Meli and Sánchez (1997). A brief description of the project follows.

A technique called underexcavation was adopted to reduce differential settlements. Briefly, a controlled subsidence of the most elevated parts of the ground is produced by extracting soil from the deep strata of soft clay. Small-diameter boreholes are made from the bottom of 32 shafts opened at a depth of about 25 m. The closure of the holes due to the weight of the soil and of the structure produces settlements at the surface. Through a careful programming of the amount and position of excavated soil, a preselected pattern of ground settlements can be achieved with good precision.

Preparatory works started in late 1991 and the underexcavation began in August 1993. Previously, the roof had been shored with steel towers whose height could be adjusted following the geometrical changes of the building caused by the induced settlements.

The overall objective was to produce a rotation of the monument, as a rigid body, towards the Northeast, plus an inward rotation of the lateral naves in the northern part of the church, and a rotation of the Sagrarium towards the Northwest. Additionally, some local corrections were attempted, trying to further reduce the out-of-plumbness of particular columns and walls. The described correction of differential settlements induces stresses in the structure, because the preexisting cracks have been filled with mortar and no room is available for the reversal of the deformation.

The underexcavation process requires a strict control of the operations and a close monitoring of the building response, in order to verify that the desired pattern of settlements is achieved, and that the structural behavior is adequate.

The complexity of the soil-foundation-structure system and of the underexcavation process did not allow to establish a precise excavation program and goal, neither an accurate prediction of the structural response. An "observational method" was used, where the underexcavation program has been adjusted from a careful evaluation of the measured response. After three years of operation enough knowledge has been reached to establish detailed programs of activity and final goals.

2. Structural Monitoring

2.1 Conventional Monitoring

Since the beginning of the project and before starting the operations, a measurement program was established, first, to establish a baseline for the most significant parameters of the structural response, and then, for controlling the performance of the monument during the underexcavation process.

In the initial stage, rather conventional techniques were used for the monitoring program. The following are the most relevant measurements performed.

a) Change of level of several hundred points of the floor and roof of the building, through precision survey.
b) Changes in the slope of columns and walls through plumbs and portable tiltmeters.
c) Changes in the span of arches and vaults, through convergence measurements.
d) Change of width of the most important cracks, through portable gauges.
e) Load taken by the shoring structure, through strain gauges placed in the vertical steel pipes.

This monitoring system has been improved through the duration of the rehabilitation and has provided the basic data for the control of the process.

The most relevant results are those of the bimonthly leveling of the floor. Figure 2 shows the pattern of differential settlements before the beginning of the correction. The extraordinary amount of these settlements can be easily appreciated, along with the large distortion they produce to the structure.

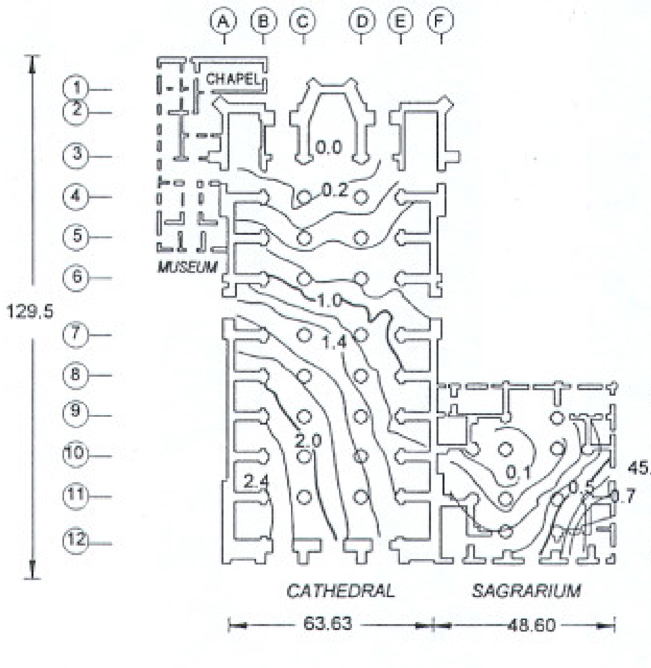

Figure 2 Differential settlement, in meters (Dec 1989)

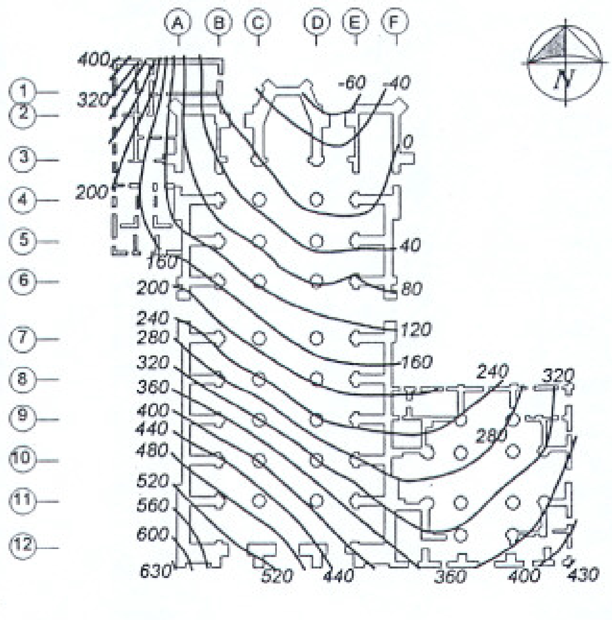

Figure 3 Differential settlements between Oct 1991 and Dec 1996, in millimeters

Meli 3

As shown in Figure 3 the pattern of settlements produced during the correction process is very nearly opposite to the existing one, demonstrating the appropiateness of the adopted procedure.

Figure 4 shows the variation with time of the correction of the differential settlements between the Southwest corner and the Apse. As it can be appreciated, a correction of 620 mm has been already reached and, by maintaining the same rate of correction, a reduction of about 1 m can be achieved by the end of 1998.

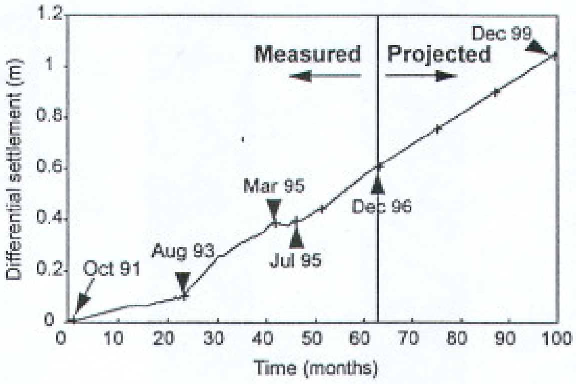

Figure 4 Evolution of the reduction in the differential settlement between the South West corner and the Apse

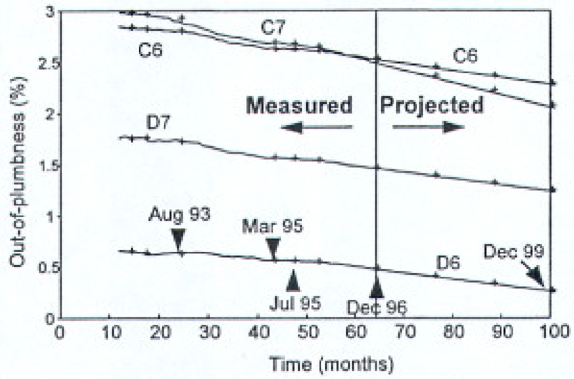

Figure 5 Correction of out-of-plumbness of the central columns

Among the results of other measurements, the reduction of out-of-plumbness of the central columns is to be pointed out. From Figure 5, it can be appreciated how each of the four central columns has been reducing its out-of-plumblness, but that the rate of reduction has been relatively small; therefore, the final correction at this regard cannot be expected to be very large.

2.2 Real time monitoring

As the underexcavation works proceeded, it became clear that a more automatic and continuous monitoring system would be of great advantage, in order to timely detect any symptons of dangerous structural performance. For that purpose, ISMES Spa designed, constructed and installed a system constituted by the following parts:

a) 10 direct pendulums with telecoordinometers, to measure the horizontal motion of the upper parts of the structure, in columns, towers and facade walls.
b) 22 long-base extensometers, to measure the changes in span of vaults and domes.
c) 5 temperature sensors in the extrados and intrados of the roof.
d) 1 radiometer

Signals of each transducer are collected in a data adquisition system, where they can be read directly, or remotely via modem.

The system has been working without problem; readings are extremely stable and, due to the large size of induced motions, the noise produced by ambient effects is neglegible. As an example, Figure 6 shows the displacements experienced by upper parts of one of the central columns, from the completion of the instrumentation, in July 1994, to December 1996. An almost constant correction rate can be appreciated, except for the period from April to June 1995, where the operations had to be interrupted and the original trend of settlements was restablished. When the underexcavation was resumed, the correction took inmediately the same path and rate. A more detailed view of the results is given in Figure 7, where the same results are amplified for a period of one month. The graph is still very stable and the effect of the sudden motion due to a small earthquake can be appreciated.

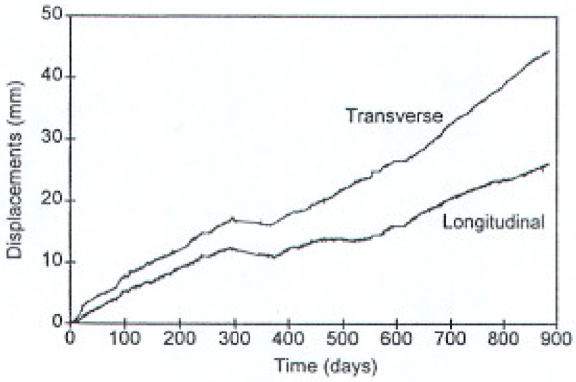

Figure 6 Displacement of the upper end of central column C7

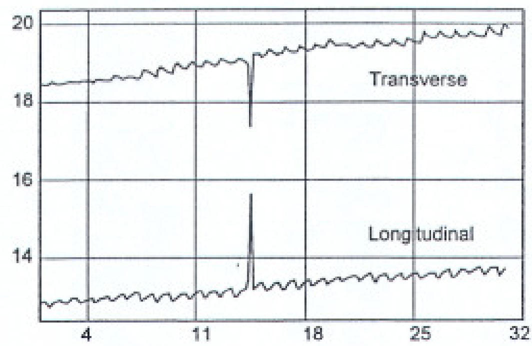

Figure 7 Displacements of central column C7during Sep 1996 (mm)

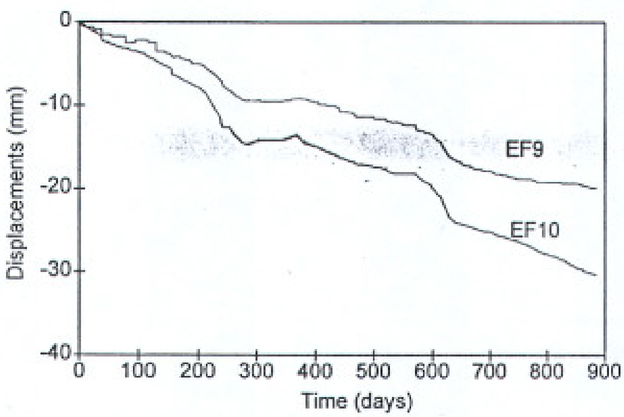

Figure 8 Span reduction of the central nave from Jul 94 to Dec 96

In a similar fashion, the evolution of the closing of the central nave in the southern part of the temple is shown in Figure 8. As it can be observed, initially the rate of span reduction was relatively high, nevertheless, once the existing cracks were closed, the rate of span reduction was significantly diminished, despite attempts to reduce the stiffness of the vaults by cutting some masonry diaphragms in the extrados.

3. Stress Monitoring and Analysis

As it has been already pointed out, the structural safety of the central columns is of great concern, because of the large loads they support and of their great out-of-plumbness. Initially, several studies were made to determine the internal array of stones and their mechanical properties. Detailed non destructive testing of the quality of the stone was also performed, to detect variation in the quality along the height of the columns. In fact, it was found that the stones used in the upper part of all columns were of a much lower quality than those of the lower part (10 Mpa versus 30 Mpa, average compressive strength).

Several analytical procedures were used to estimate the state of stress in the structure and particularly in the columns; from the traditional methods based on the flow of gravity forces, to finite element models of the whole structure or of parts of it.

In order to validate analytical results, in situ measurements were performed of the state of stress in the structure, through the flat-jack technique that has been developed by ISMES for its application to masonry structures (Rossi, 1995).

Stresses were measured by this technique in columns at different heights and in some walls. The main results are shown in Figure 9. Maximum compressive stress was 9 Mpa for one of the central columns. A large variation of stresses within the column section was observed, following a roughly linear trend, thus indicating the presence of significant flexure in the columns.

In general terms, measured stresses were rather consistent for different sections and for repeated measurements at the same section. Some readings were affected by a problem derived from the particular way the flat-jack technique was applied to columns in this case. The cuts to introduce the flat-jack were made in the mortar of the horizontal joints. In some of these joints, small pieces of stone had been placed to level the upper line of stones; therefore, large concentrations of stress were produced due to the high rigidity of these stones, whereas the mortar in their vicinity was releived of stresses. This problem was studied in detail by doing several measurements in column D-10, where some very low stresses had been measured. From these considerations some measurements were discarded because they were made in the vicinity of very stiff points.

Results of finite element analyses showed a rather uniform distribution of compressive stress through the column section, even when the actual settlement configuration was induced at the base of the structure. On the contrary, the measurements indicated a large and almost linear variation of stresses in most sections. The difference is due to large cracking and non linear behavior of the structure. Only when a local model with more representative boundary conditions is analysed, a better ressemblance between measured and computed stresses is achieved.

It can be concluded that valuable indications about the level of stresses in a structure of this kind can be obtained by the flat-jack technique, when due consideration is given to specific local conditions.

4. Seismic Monitoring

Seismic actions constitute a significant hazard to the structure, even if it has demonstrated to be able to withstand earthquakes during more than four centuries, without major harm. Presently, the great out-of-plumbness of columns and walls makes the structure more vulnerable to these effects. The seismic response of such a heavy and stiff structure founded on a very soft soil is significantly affected by soil-structure interaction. Additionally, the low tensile strength of masonry and the large cracking make its behavior very different from that of modern buildings where there is continuity between structural components.

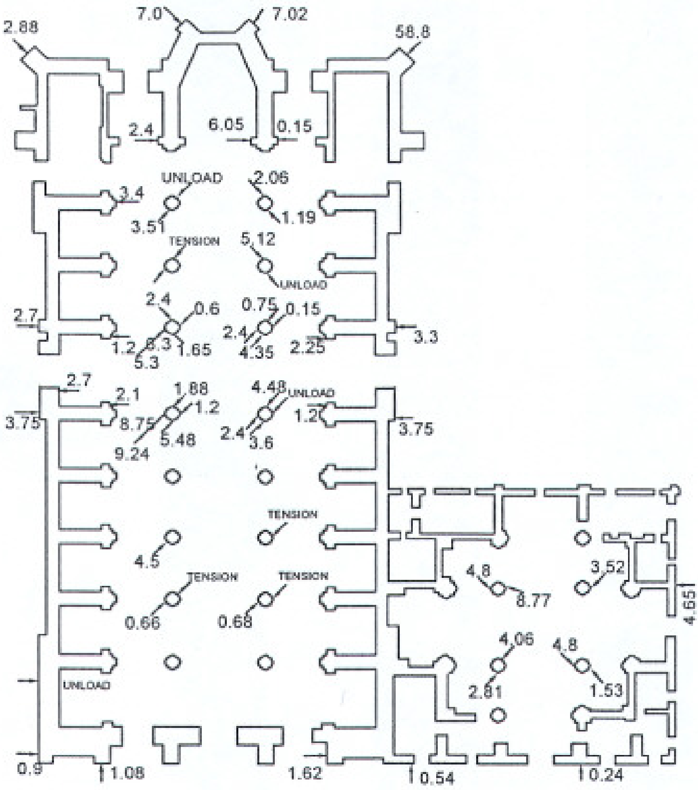

Figure 9 Compressive stresses measured through the flat-jack techniques (MPa)

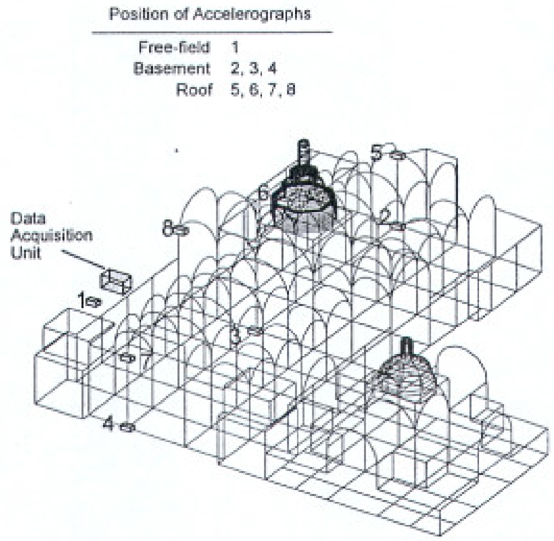

Figure 10 Accelerographic Network

In order to obtain basic information about the seismic response of buildings of this kind, a network of eight accelerographs has been recently installed (Figure 10). One instrument is located in the ground outside the temple, to provide a reference motion in the free-field. Three accelerographs are placed in the basement to identify possible differences in the motion imposed to the buildings due to variation in soil properties and lack of monolithism of the foundation. Four additional accelerographs are placed in the roof to measure the amplification of the motion at different points, and the possible out-of-phase motions of parts of the structure separated by the main lines of cracking.

In the few months after its completion, the network has already provided two sets of valuable recordings. Only a few relevant results can be commented within the scope of this paper. The building vibrates essentially as a rigid body with small differences in the motion at different points of its foundation, and with a very small amplification of the ground motion at the roof. Clear evidence of out of phase motion between the northern and southern part of the building is detected, nevertheless the amount of the differential displacements is extremely small. The fundamental period of vibration of the building is 0.39 and 0.45 seconds in the longitudinal and transverse direction, respectively. The prevailing vibration period of the ground at the site is 2.50 s; therefore, the situation is very far from resonance.

5. Conclusions

The monitoring system installed constitutes an essential part of the process for the correction of the differential settlements, by giving basic guidance to define the amount of underexcavation needed to achieve the desired configuration of differential settlements, without producing signficiant harm to the building.

The real time monitoring has been highly successful and has allowed to detect sudden changes in the response of the building and to timely make the necessary modifications.

The flat-jack technique has proven to be a reliable procedure for obtaining a reasonable estimation of the state of stress in the structural members, if the interpretation takes into account the possible sources of difference between the stress in a specific point and the general trend of stress configuration.

The accelerographic network is likely to provide a valuable insight into the seismic response of this kind of monumental buildings founded on soft soils.

Aknowledgements

The project for the rehabilitation of the Mexico City Cathedral is performed by the Direction of Monuments and Historical Sites of the Ministry of Education, under the direction of Sergio Zaldivar. The real time monitoring system has been donated by ENEL, the Italian Agency for Electric Power, and installed by ISMES. The latter institution also executed most of the flat-jack measurements.

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