STRUCTURAL MONITORING OF THE MEXICO CITY CATHEDRAL (1990-2013)

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Abstract. The Mexico City Cathedral underwent, along the decade of 1990, a major rehabilitation aimed at coping with effects of extreme differential settlements. As part of a comprehensive program focused, first, at a partial correction of existing differential settlements, and then, at reducing their future growth, remedial interventions were carried out to improve structural safety of the building. Monitoring the response of the structure along and after the rehabilitation was an essential component of the program, because it allowed adjusting the actions that were being taken on the subsoil and on the foundation and, subsequently, was the basis for the detection of any sign of inappropriate structural performance. The paper gives, first, a brief description of the whole project and of the effects of the various stages of rehabilitation on the patterns of differential settlement and on the structural response of the building. Then, it describes the monitoring systems and evaluates their results, especially those obtained in the 14 years elapsed since completion of the rehabilitation program. Particular emphasis is placed on the results of the seismic network, which has provided a great amount of valuable information on the peculiar characteristics of the seismic input and on the dynamic response of the structure. It is concluded that the actions taken have significantly prolonged the life expectancy of the building, having significantly reduced the growth rate of its differential settlements; however, it is essential to continue close monitoring to detect early signs of changes in the patterns of subsidence or of the structural response to them.
1 INTRODUCTION

The Mexico City Cathedral has been severely affected by differential settlements since its construction began in the 16th century. With its total weight of 12,700 kN, this monument stands on very soft clay layers, which had been previously consolidated, in some areas, by the weight of earlier monuments, over whose remains the Cathedral was erected. Differences in soil deformability, have been the main cause of differential settlements. Since early 20th century, intense water extraction from the underlying aquifer has severely aggravated subsidence, and the distortion of the building caused by differential settlements has seriously undermined its structural safety. As an example, in 1990, the differential settlement between the main altar and the southwest corner of the temple had reached 2.4 m, and was increasing at a rate of 12 mm/yr. Some of the main columns supporting the roof showed an out of plumb exceeding 3%. Severe cracks in the roof, floor and walls constantly reappeared in spite of frequent repairs.

A major rehabilitation program was started in 1991 to restore the Cathedral and the adjacent parish church, to a more stable and safe condition. A description of the geotechnical problems and of techniques used to correct the differential settlements and to reduce their future rate of growth, can be found in [1, 2 and 3]. Structural studies performed in support of the project, and to monitor structural behavior during the rehabilitation, have been described in a previous paper by these authors [4].

2 STRUCTURAL CHARACTERISTICS AND PAST PERFORMANCE OF THE CATHEDRAL

The Cathedral (Figure 1) has five longitudinal naves. The roof of its central nave is a cylindrical stone masonry vault supported by arches and by 16 stone columns. A close array of robust masonry walls divides the external naves into small chapels. These walls, together with the facades and some buttresses, constitute a peripheral belt that provides great lateral strength and stiffness to the monument.

![Figure 1: The Cathedral of México.](image)

The Cathedral is supported by a grid of foundation girders, 3.5 m deep, which rest on a foundation mat, about 2 m thick. Timber piles with a 0.2 m diameter and a length of 2-3 m are spaced at every 0.6 m beneath the foundation mat.

The primary construction material is a masonry conglomerate of volcanic stones bonded by a lime-sand mortar. Stone ashlars were used for columns and arches. There is clear evidence throughout the structure of changes having been made to shapes and dimensions of structural members, from early stage of construction, in order to cope with out of plumbing and lack of alignment, that were caused by differential settlements. For example, heights of columns vary as much as 0.85 m; and several rows of sills with variable heights had to be used to correct the
inclination of the façade; furthermore, the spans and rises of arches and vaults were adjusted to obtain a uniform roof level.

Differential settlements measured in the floor of the Cathedral before initiation of rehabilitation works are shown in Figure 2. Two major mechanisms of deformation were identified. One is the sinking towards the southwest corner and the other is the “emergence” of the central nave in its northern part. The first mechanism produced a pattern of transverse cracks in the roof and arches, especially near to the southern facade, which, with its very heavy towers, rotated outward, thus separating from the rest of the church. The second mechanism produced the outward rotation of columns and of lateral walls, as well as the opening of vaults and arches, originating a pattern of longitudinal cracks in the roof, floor and foundation. It also generated horizontal cracks at the base of longitudinal walls and some diagonal cracks in transverse walls. This second mechanism is critical from the structural point of view, because it involves tilting of the columns of the transept, which carry the heavy weight of vertical the central dome. The greatest deviation between the top and the base of the column was 0.53 m, which amounts for 25% of the column side.

Figure 2: Curves of equal differential settlement (in meters) of the floor of the Cathedral and of its adjacent buildings. September 1993. Reference point (0.0 level) is at the foot of the altar.

3 THE REHABILITATION PROGRAM

A detailed survey of differential settlements of the floor and of column tilts was carried out in 1990; its results caused great concern about the structural safety of the church, mainly regarding seismic effects. Consequently, a major rehabilitation project was implemented. Success of under-excavation for correcting leaning of several buildings after the 1985 Mexico earthquake, led to try to achieve a partial correction of the differential settlements of the Cathedral through the use of this same technique, which consists of a controlled extraction of small portions of soil under some parts of the structure, until a pre-established reduction of differential settlements is achieved. Previously, the building roof was thoroughly shored with a robust steel tubular structure and the critical columns of the central nave were temporarily strengthened by strapping them with steel collars as shown in Figure 3.
The overall objective of under-excavation was to produce a rotation towards the northeast of the Cathedral as a rigid body, plus an inward rotation of the lateral naves of the church aiming at reducing tilts of columns and walls. An average rate of correction of the most critical differential settlements of 13 mm per month was sustained for about two years. Afterwards, the structure showed an increasing opposition to further distortions, causing to end this phase of the rehabilitation process. A second phase of the project was initiated, aiming at achieving a more uniform deformability of the soil beneath the different parts of the church, by soil hardening through mortar injections in zones showing the highest rates of settlement. Mortar grouting into the layers of soft clay was carried out mainly in the southwest corner of the church, and decreasing volumes were grouted towards the church center. Additional injections were performed along the northwestern and northeastern sides of the church, in order to reduce their trend to outwards rotations. As a further step for reducing differential settlements, stone masonry foundation girders were retrofitted by attaching concrete walls at both sides and steel beams on their tops, in order to provide continuity among their segments, and to form a stiff grillage that could offer greater opposition to relative settlements of the columns bases, thus mitigating the trend to a convex deformation of the floor. After several phases of soil hardening, the rates of differential settlements were drastically reduced, as it will be discussed later in this paper.

Remedial interventions in the main structure started soon after the end of under-excavation. They aimed mainly at restoring the integrity of the original structure and at reducing its vulnerability. A thorough injection of the widespread cracks in masonry vaults and walls was performed to consolidate stone masonry, together with the substitution of several damaged ashlers in arches and lintels. One of the arches supporting the choir that had lost most of its curvature was dismantled and rebuilt with a more favorable geometry. The main concern related the structural safety of the church regarded the 16 columns along the main nave, because of the heavy gravity loads they were supporting, of their great slenderness and of their significant out of plumb. The situation of the four columns of the transept, which supported the main dome, was considered critical and was studied in much detail. Some vertical cracks were interpreted as signs of incipient crushing due to high concentrations of compressive stresses caused by the uneven contact between consecutive ashlers. That was because, for leveling ashlers and maintain them in a fixed position until the mortar will set, wedges of the same stone were placed all along the perimeter of the columns (Figure 4). The difference in stiffness between these wedges and the surrounding lime mortar made that load from the upper ashlers had to be transmitted.
mainly through the wedges, thus producing high stress concentrations in stones areas close to these wedges, while the rest section remained at a very lower stress. In order to provide a more uniform stress distribution throughout the joints, the stone wedges were removed and a high quality mortar was injected to cover the entire surface of the joint.

Figure 4: Hard stone wedges at the periphery of the horizontal joints between courses of ashlars.

On the bases of the columns, beneath the floor level, a sturdy steel ring was placed and connected to steel beams that were added at the sides the masonry walls constituting the original foundation grid. Besides giving continuity to the foundation grid, these rings are intended to prevent cracking of the bases of the columns due to rotations caused by differential settlements (Figure 5). Another important structural intervention concerned the confinement of the central and lateral naves, aimed at restricting the overturning of the lateral facades. A solution through three sets of tie-rods was implemented. Two of them were hidden over the ledges located at the level of the rise of the vaults of the lateral naves and of the transept.

Figure 5: Confining steel ring around column bases.

4 STUDIES TO ASSESS THE STRUCTURAL SAFETY AND TO GUIDE DECISIONS ABOUT REHABILITATION

During preparatory works for the first phase of the rehabilitation program, a detailed survey of the structural elements and of the overall behavior was performed. Existing cracks were mapped, along with the traces of previously repaired cracks. Internal imperfections were detected through ultrasound equipment and, occasionally, small diameter cores were extracted to ascertain internal structure of some critical elements, as well as for performing laboratory tests to obtain mechanical properties of the material. Compressive stresses in columns and walls
were estimated by the flat jack test [5]. Results showed great differences of stresses acting on different parts of the same column section; their variation indicated a significant eccentricity of the applied load. The most critical situations are at the top of the columns of the transept, where the compressive stress reaches 8 MPa, which is not far from the strength of the weak stone ashlars used in the upper part of the columns.

The assessment of the structural safety of the buildings was performed, first, for the structure in the way it was conceived, and then for the structure as it was built, including the geometrical changes made to compensate for the effects of differential settlements, leading mainly to the deviations in verticality of columns. Afterwards, the structure was modeled as it was at the beginning of the rehabilitation program, with the damages and distortions caused by the differential settlements occurred throughout its existence. Then, the safety of the structure as it was at the end of its rehabilitation was assessed, taking into account the partial correction of the differential settlements and of columns out of plumbs, by under-excavation, as well as the repair of damages in columns and arches and the retrofitting of the foundation and the confinement of the lateral facades.

Numerical analyses of the effects of different loading and deformation conditions were performed, through a FEM linear model of the complete structure, whereas detailed studies considering nonlinear behavior were performed on critical members, as the four columns under the central dome. In general terms, loads induced by the weight of the building as it was originally conceived, generate only compressive stresses in both the roof structure and the support members. Results reveal the adequacy and effectiveness of the proposed structural scheme, but also draw attention to very high unit load that extraordinary weight of construction transfers to the ground (129 kPa), which is much greater than what a subsoil of this nature could bear without an excessive subsidence.

Then the model was modified taking into account differences between actual construction and project, and the long term effects of differential subsidence. The main cause of concern were the stability of the vaults and arches, which had lost part of their curvature due the opening of their supports, and the stability of the columns, which are affected by the bending produced by the rotations of their base. Analyses indicated that stresses and displacements were still well within safety limits. Nevertheless, considering the risk of an accidental overload be generated by an abrupt settlement of the foundation during under-excavation, the decision was taken of shoring the roof and of temporarily strengthening the critical columns of the central nave, as previously described.

The model of the structure as it was at the end of its rehabilitation, was calibrated with the results of field studies, of the monitoring system, and particularly, of the network of seismic instrumentation to be described in the following chapter of this article. One of the main factors that led to the conclusion that the structure was well into safety limits, was that the seismic analyses determined that the lateral displacements of the structure relative to the ground, were of a few millimeters, for the severe earthquakes motions prescribed by design code codes for buildings of particular importance. From the monitoring results after completion of the selective hardening of the soil, the growth of differential settlement expected in the following 25 years were estimated, and the resulting increase in the leanings of the columns was determined. It was found that the latter were still within safety margins, so it was decided to limit structural interventions to the few basic actions as described in Chapter 4. Additionally, it was recommended to maintain a continuous monitoring of the structure for timely detect significant changes in the trend of subsidence, and any increase of the displacement of the critical structural elements.
A basic factor for decision taking on rehabilitation steps was the monitoring of a large numbers of indicators of the building response to external actions and to the different stages of rehabilitation, especially during the under-excavation phase. The basic indicators were initially obtained by monthly topographic surveys of settlement of the bases of the columns and walls, as well as of their out of plumb. Settlements were measured at 277 points of the floor, and tilts at 184 points on columns and walls, and at 80 points on towers and facades.

A couple of years after starting under-excavation works, a continuous monitoring system was installed, which allowed real time follow up of the main indices of the response. The system was designed, built and installed by the ISMES Institute of Bergamo (Italy) in 1994, and is still in operation. It consists of 22 wire gauges that record changes of span (opening or closing) of arches and vaults; 10 electronic tilt meters to record changes of columns out of plumbs; and 6 thermometers that record solar radiation and temperature changes inside and outside church. The location of these devices is shown schematically in Figure 6. Through an integrated computer system, every four hours graphs showing the evolution of each of the measured parameters are updated and sent to two remote control stations.

One of the most important studies conducted as part of the rehabilitation program consisted of monitoring, through a network of accelerometers, seismic activity on the site, and the cathedral’s response to it. The initial arrangement was through eight instruments located in different positions of the roof, basement and tower, plus one placed in the free-field, 20 m away from the building (Figure 7). Over time, the number and position of the sensors had varied, but the network has continued to record the events from 1997 to date, when five of them are operating. During this period, dozens of seismic events of varying intensity, from low to moderate, have
been recorded. A basic purpose of the network is to detect changes of the dynamic properties of the structure, which could be attributable to structural damage that may affect the safety of the building.

![Figure 7: Seismic instrumentation network.](image)

6 EVALUATION OF STRUCTURAL PERFORMANCE THROUGH RESULTS OF THE MONITORING NETWORK

Monitoring of the Cathedral has continued after the end of the main rehabilitation program, although the limitations of financial resources and those derived from modifications to the operation of the temple, have driven to removing some of the transducers, either temporarily or permanently, and to space the topographic surveys. Despite these limitations, valuable information is being obtained, from which some basic conclusions will be presented in the following paragraphs, for each type of monitoring.

**Topographic survey of differential settlements.** The most important results for evaluating the success of the actions taken to correct the patterns of the subsidence are presented in Figure 8, in which the growth rates of differential settlements in some critical zones of the cathedral, immediately before beginning the rehabilitation, are compared with those observed in recent years. It can be noted that, prior to the correction works, in several parts of the temple the difference of settlements between nearby reference points was growing at a rate of more than 10 mm/yr, whereas in recent years the average growth of differential settlements between the same points has been reduced to less than 2 mm/yr.

Throughout the entire project, the main index of the differential settlements has been the gap between a reference point located at the foot of the main altar and another at the foot of the SW bell tower. The differential settlement between the two reference points had reached 2.4 m just before the start of under-excavation works, as it was shown earlier in Figure 3. The evolution of this gap is shown in Figure 9, in which three stages of behavior can be distinguished: during under-excavation (1993-97), during soil hardening phase (1998-2001), and after the end of works. Because of under-excavation, the point at the foot of the main altar subsided 1.35 m, while the one near to the SW corner dropped only 0.51 m, implying a reduction of 0.84 m of the gap between the two points. The rate of differential settlement between the two points decreased from 14 mm/yr in 1989 to only 2 mm/yr in 2000. The graph of the differential settlements between the two reference points shows that since 2002 there has been no further loss of the reduction achieved with the under-excavation. On the other hand, it should be noted that the settlement rate has been increasing since 2007 in both points, most likely because of an increase in the rate of water extraction from deep aquifers.
Columns out of plumb. As it has been previously mentioned, the major concern, in terms of structural safety of the building, regards the stability of the columns supporting the central nave, which is severely threatened by their out of plumbs. Figure 10 a) shows the direction and magnitude of the out of plumbs of columns and walls before the start of the rehabilitation works. As it can be appreciated, out of plumbs exceeded 2% of their height, in 11 out of the 16 columns...
of the central nave. Figure 10 b) shows corrections of column out of plumbs, after the sub-excavation. It can be seen that in almost all columns the change was in the right direction, although the amount of the correction was rather modest: in five columns, out of plumb was still 2% or more.

Figure 10: Out of plumbs of columns and walls (in % of their height), and their corrections by under-excavation.

A more detailed view of the variation of columns out of plumbs throughout the entire process is given in Figure 11, which shows the evolution of the lateral displacement at the heads of three columns of the transept, as obtained from measurements of the electronic tilt meters. In all three cases, it is observed that, after the phases of under-excavation and of injection, out of plumb continues growing, but at a small rate, not exceeding 1 mm/yr. Farthest north column, C5, maintains a slight corrective movement in the NS direction, whereas in EW direction, its out of plumb is steady. Column D6 is moving eastbound, thus increasing its distance from column C6, which is rather stable. The most worrying situation is the inclination of the towers of the main facade, whose history of lateral movements is shown in Figure 12. The lateral displacement in the EW direction has ceased in the west tower, while in the east towers is growing at 1mm/yr, eastbound. On the other hand, in both towers there is still a significant southbound motion, which for east tower grows at a rate of 2.3mm/yr, and for the west tower at 1.5 mm/yr. This movement of the towers carries an outward rotation of the whole façade, which has caused the reopening of a transverse crack in the vaults of the first bay of the nave. Elsewhere in the temple, columns rotations are very small, especially in the NS direction. The above described results show that only a small part of the tilts of columns and walls had been corrected by the under-excavation, and in some cases part of the correction is now being reversed, particularly with regard to the columns of the north bays of the nave, and to the whole façade of the church.
Figure 11: Evolution of out of plumbs of three columns of the transept.

Figure 12: Evolution of out of plumb of the two towers.

Opening of the span of arches and vaults. Results of electronic wire gauges monitoring the closing or opening of the span of the arches correlate well with those measuring leanings of the columns supporting each arch. Figure 13 shows evolution of the opening of six arches of the main nave: from the one at apse through those at the transept, and until that at second bay, near the main facade. Only the segment from year 2007 onwards is shown, so to highlight the current trend. It can be noted that all arches spans are growing at an annual rate that is maximum in the northern arches (1.1mm/yr), and decreases to 0.4 mm/yr for the southern arch. On the other hand, the longitudinal arches flanking the central nave have significantly smaller movements, except for those of the first bay, whose span increases due to the outward rotation of the main facade. The opening of the arches of nave has led to the reopening of a longitudinal crack along the length of the key of the vault, which has been already repaired once, after the end of the
rehabilitation program, and has recently reopened again. A transverse crack at the key of the first bay has also appeared again, due to the outward rotation of the facade.

Figure 13: Increase in the span of the arches of the central nave (Note the seasonal variation).

7 RESULTS FROM THE SEISMIC MONITORING NETWORK

The interpretation of results obtained from the seismic network during the rehabilitation phase were published shortly after the completion of the works, [6]. They will be summarized below, along with the new results obtained since then.

Ground motion at free field. The rather peculiar characteristics of the seismic ground motion recorded in the historic center of Mexico City, can be appreciated from Figure 14, which shows a record of the accelerometer placed on the free field, near the site of the Cathedral. Acceleration time history shows an almost harmonic vibration of very long duration, and with a dominant period of vibration near to 2 s. The latter varies in different sites, according to the depth of the very soft clay layers underlying the ground.

Motion at the basement. The ground motion at free field is modified by the presence of the massive structure of the cathedral, due to a kinematic interaction between the soil and the structure. In Figure 15 the ratios of the Fourier spectral amplitudes obtained from acceleration histories recorded at the basement of the cathedral, to those recorded at the free field are shown.
As it can be appreciated, for low frequency waves, as those dominating the vibration at free field, the ratio is consistently equal to one, while for wave frequencies exceeding 1.1 Hz, the amplitude is significantly smaller for the motions recorded at the basement. The cause is a kinematic soil-structure interaction because seismic waves traveling in the free field are filtered when crossing a rigid massive body as the cathedral basement, thus reducing their amplitude. The reduction takes place mostly for high frequency waves whose length is less than the size of the building.

Figure 15: Ratios of the Fourier spectra ordinates obtained from acceleration histories recorded at the three instruments located a basement to that recorded at the free field.

Figure 16 shows the acceleration response spectra for three of the most intense events recorded at the basement and at the free field. The dominant period, which was slightly higher than 2 s in the 1993 event, has decreased to about 1.8 s for both events of year 2004. These reductions are similar for the motion at the basement and at the free field. The decrease in the dominant period has been observed in most sites of the lake zone of Mexico City, and is attributed to consolidation of the clay layers because of the intense water extraction from the underlying aquifer. A comparison of the spectra of the ground motion at the basement with those at the free field shows that in the former there is a general significant reduction of spectral amplitude of the motion, and that this reduction is greater for periods of less than 0.5 s. These reductions are a favorable effect of the soil structure interaction.

Figure 16: Response spectra of three strong motions recorded at the free field, a), and at the basement, b).

Motion at the roof. Acceleration histories recorded at the roof and at the basement for the segment of one of the recorded seismic events are compared in Figure 17. The two graphs are
similar, in terms of amplitude and of prevailing vibration frequencies. The increase in maximum acceleration from basement to roof is less than 20\%, and the cycles of vibration of both records are governed by the natural period of vibration of the soil layers (about 2.1 s). This indicates that the structure follows the motion of the ground, almost as a rigid body, without a significant amplification of the induced shaking; therefore, lateral displacements relative to the ground and internal forces induced to the structure are rather small.

![Figure 17: Comparison of acceleration histories recorded at the roof and at the basement.](image)

From the spectral analysis of the relative motion between roof and basement, main dynamic properties of the structure can be obtained. Fundamental periods of vibrations in the two horizontal directions as determined from the most recent recorded event are shown in Figure 18. As it can be seen, the fundamental periods in the transverse and in the longitudinal directions are very similar, 0.49 and 0.44 seconds, respectively. These periods are very far from the 2.1 s period dominating the input at the basement, which, being almost harmonic, has little content of waves with frequencies in the neighborhood of the fundamental modes of vibration of the structure. This difference explains the very small amplification of the acceleration at the building roof with regard to those induced at its basement.

![Figure 18: Identification of modal frequencies of the structure from ratios of Fourier spectral amplitudes recorded at the roof and at the basement.](image)

Figure 19 shows the variation in time of the fundamental period of vibration of the building in its two main horizontal directions, as determined from records of events that occurred since the end of the under-excavation phase to date. The irregularity of the graphs can be attributed mainly to differences in magnitude and epicentral distances of the seismic events considered. In both directions the differences are small, but a trend is seen to a progressive increase of the fundamental period with time. While in the transverse direction the variation is small, in the longitudinal direction the fundamental period shows a more significant and almost linear increase with time, that could be a sign of a progressive reduction of the lateral stiffness of the structure in that direction.

![Figure 19: Variation in time of the fundamental period of vibration of the building.](image)
8 CONCLUDING REMARKS

The rehabilitation of the Cathedral was innovative in many ways. First, for the techniques that were developed and applied to correct the behavior of the foundation, but also for the control of the different steps of intervention, based on the results of a comprehensive system for monitoring a set of parameters of the building response.

The results were very successful, especially in terms of the drastic reduction in the growth of differential settlements, which have been increasing at a very small rate since the end of the phase of soil hardening under the areas of maximum subsidence. With regard to the structural safety, the significant reduction of differential settlement through under-excavation was accompanied only by a modest reduction of the leanings of the columns of the main nave and of the towers. Slope of several columns and walls still exceeds 2% of, and is still growing, albeit at a small rate. The most worrisome case is the eastern bell tower that could reach a critical situation within a few decades, unless some provision is taken within the next few years.

The seismic monitoring network has allowed a very complete view of the characteristics of the ground motion during earthquakes and how they are modified by the soil structure interaction; most importantly, it has allowed to appraise the characteristics of the seismic response of the structure and their variation during the 17 years since the network installation. The ground motion at the site is characterized by an almost harmonic vibration of a very low frequency. In modern tall, flexible buildings whose fundamental frequencies of vibration are also low, this kind of ground motion is greatly amplified, and seismic effects are very severe. On the contrary, massive stiff colonial buildings, which have quite high vibration frequencies and internal damping, tend to vibrate with same amplitudes than the ground. In a way, they can be thought as floating over the soft clay of the old lakebed. Moreover, the soil-structure interaction gives rise to a significant reduction in the amplitude of waves whose period is close to that of the fundamental modes of vibration of the structure.

Analytical and experimental studies have led to a better understanding of the structural behavior and to the identification of the main sources of vulnerability. Analyses indicate that, in the current situation, the structural safety is adequate; however, the building could be in danger if differential settlement rates were to rise significantly.

Although the behavior of the structure has substantially improved because of the rehabilitation, some problems remain that require careful monitoring. Leading among them are the steepness of the columns and of the main façade, which continues to increase, albeit at a much smaller rate than before, and could merit in the near future a local intervention to curb its growth. It is therefore essential that the monitoring system continue to be operated in its entirety and its results be constantly evaluated.
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