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# DIAGNOSIS AND STRUCTURAL REHABILITATION OF A XVI CENTURY STONE MASONRY ARCH BRIDGE

**Abraham R. Sánchez Ramírez[1] and Roberto Meli[2]**

[1] Institute of Engineering, UNAM
Ciudad Universitaria, Delegación Coyoacán, México D.F. CP 04510, Mexico.
e-mail: ASanchezR@iingen.unam.mx

[2] Institute of Engineering, UNAM
Ciudad Universitaria, Delegación Coyoacán, México D.F. CP 04510, México.
RMeliP@iingen.unam.mx

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**Abstract**. *The paper deals with a stone masonry arch bridge, which is the outstanding part of an important work of hydraulic engineering, built in the sixteenth century in central Mexico: the "Father Tembleque Aqueduct". This article describes the studies performed to determine the state of damage and the structural safety of the bridge, in its present condition, under effects of wind, earthquake and gravity loads. Although most of the arches show an excellent state of preservation, signs of increasing deterioration have raised concerns about the long-term safety of some of the tallest arches. Given the afore-described situation, these authors have proposed a rehabilitation project aiming at restoring the structure to near its original condition. Retrofitting includes the restitution of masonry in areas where it is dislocated or damaged; consolidation of ashlar and raw masonries, and confinement of some columns and arches, with stainless steel ties, to restore the joint action between ashlar and raw masonries.*

## 1 INTRODUCTION

The most important work of hydraulic engineering built in the sixteenth century in the Americas is the “Father Tembleque Aqueduct” in central Mexico. Its outstanding feature is a segment of 66 arches with a total length of 1,020 m, called the “Major Arch Bridge” (Figure 1). Its importance is emphasized by the many technical challenges that were faced to erect this work, which was carried out with very limited technical and material resources, on the initiative of a monk with no previous experience known. It was executed by local labor, whose skills and ingenuity, inherited from the pre-colonial traditions, are evidenced by the high quality of the work and the effectiveness of some construction techniques.

Figure 1: The Major Arch Bridge of the Father Tembleque Aqueduct.

The aqueduct was designed and built by Friar Francisco de Tembleque between 1543 and 1561, with the purpose of supplying water to the town of Otumba, in the State of Mexico, bringing it from some springs that arose about 40 km away. The friar had difficulty in getting the consent of the authorities, both ecclesiastical and colonial, which were very skeptical of the viability of the work and the ability of the monk and his Indian labor, to meet the technical challenges posed by the project, [2], [7]. The aqueduct was in operation for nearly two centuries, and stopped working in the late eighteenth century, probably because the inhabitants of Otumba began to stock up on local wells.

Of the 17 years required to finish the work, five were taken to build the, so called ”Major Arch Bridge”, which was necessary to save Tepeyahualco Canyon. This great work, which is the subject of this article, was admired throughout the colonial age [1], and has survived thanks to the excellent quality of its construction and the respect that has been subjected to by the local communities.

This paper summarizes the studies conducted for the evaluation of the structural safety of the major arch bridge. It begins with an overview of the hydraulic work and its construction process, followed by a description of its current state of damage and of the structural analyses that were performed for the diagnosis of its safety under gravity loads and the effects of earthquake and wind actions. It ends with a summary of the structural rehabilitation project.

## 2 DESCRIPTION OF THE AQUEDUCT

The aqueduct was resolved in most of its route with small channels and ditches on walls of small elevation; in its initial segment it required three bridges to save the deepest canyons. A general map of its route is shown in Figure 2. The route preferably follows the contours of the land and seeks the easiest way to cross the various canyons that must traverse. The length of the aqueduct is 41 km. The channel through which the water flows is 30 cm wide by 20 cm high, and its flow of operation is estimated at 40 liters per minute. The altitude of the channel at the start of the aqueduct is 2440 m, and at the final tank is 2360 m, which means a drop of 80 m, corresponding to an average slope of 0.002, i.e. 2 meters per kilometer. This gives very little room for error considering the rudimentary measuring instruments and building tools that were available for the work force.

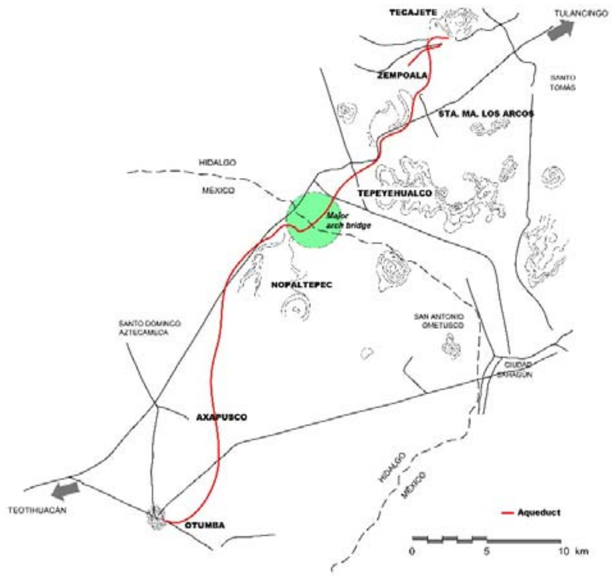



Figure 2: Route of the aqueduct.

The Major Arch Bridge covers a length of almost a thousand meters, has 66 arches and two end segments of solid masonry. Its main arch is 38 m high and covers a 17 m span, and within it another arch of smaller height was built. The arches are supported on rectangular columns whose in plan dimensions are 2.8 m in the longitudinal by 2.6 m in the transverse direction. In the latter direction, the section is reduced twice by 60 cm, first at 7 m below the kick of the arches and again at 19 m from the same reference. Columns were built with a raw basalt stone masonry in the central part of the section, and with large ashlar stones at the corners (Figure 3). Some of these ashlars were prepared with teeth protruding into the raw masonry, in order to achieve the binding of the two materials. Between columns, two parallel voussoir arches are supporting a raw masonry upper part and the channel on its top (Figure 4). Teeth penetrating into the upper raw masonry were also provided in some of the voussoir ashlars. Columns are founded on a very firm soil or on solid rock, through a 30 cm enlargement of their masonry section. Foundation depth ranges from 20 to 40 cm.

Figure 3: Arrangement of stone masonry in columns.

Figure 4: Arrangement of stone masonry in arches (view from the side and view from the intrados).

It is assumed [4] that the arches of low rise and small span were built by using wooden formwork, while for the largest bridge spans, a solid wall was first built like a tall rampart, by alternating stone masonry segments corresponding to columns, and adobe segments corresponding to arches, of which the adobe walls constituted the formwork. The wall was constructed as a series of continuous horizontal bands about one meter high by 2.8 m wide. This gave enough room for the mason to work placing stones or adobes, while stepping on the already built lower band, through which materials were hauled from the nearest end, where the wall had little height (Figure 5). Two crews were likely to advance simultaneously from the center towards the ends of the bridge. The adobe walls were intended to be demolished, once the bridge had been completed; nevertheless, it seems that only the top of the adobe walls was demolished, enough to free the arches, while the bottom part was left as part of the bridge structure, where it was gradually disintegrated by weathering. Currently, there are only a few remains of the adobe walls in two of the bridge spans. On the top of the arches, the channel was built in raw masonry, and was protected by stone slabs and a gabled stone roof.

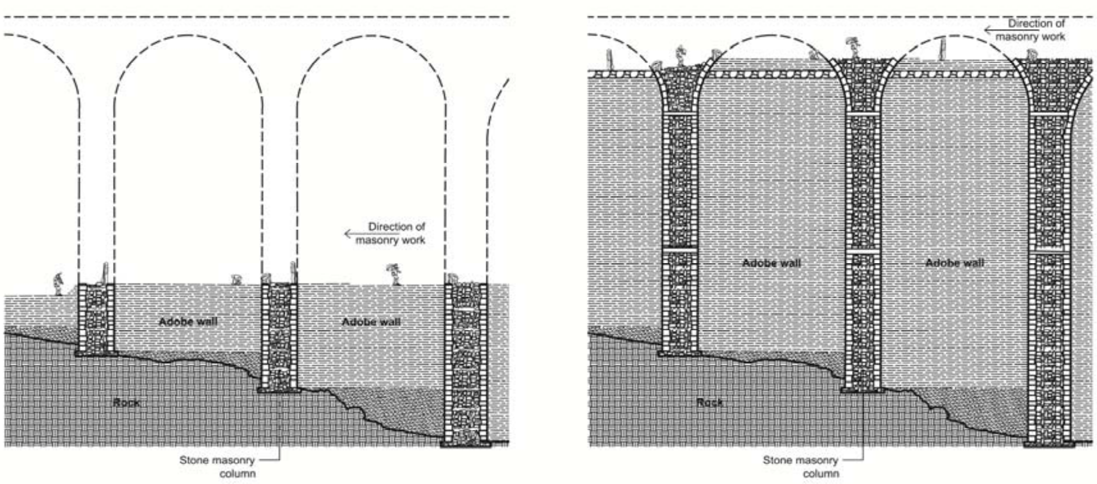



Figure 5: Building procedure for the highest segment of the arched bridge.

## 3 FIELD STUDIES

Some results of the geometric and topographical surveys that were carried out for the master plan of rehabilitation of the aqueduct are of interest to assess the quality of the execution of the work. It was found that the maximum deviation of the dimensions of the sides of column sections from their average value do not exceed 10 mm. Out of plumb of column faces never exceed 15 mm, nor 1/1000 of the column height. In many cases, tilting of opposite faces of a column is in different directions, thus indicating that is due to small inaccuracies in the construction and not to a rotation at the column base (Figure 6). A remarkable regularity is found in the slope of the channel bottom, as well as in the vertical and horizontal alignments of the columns.

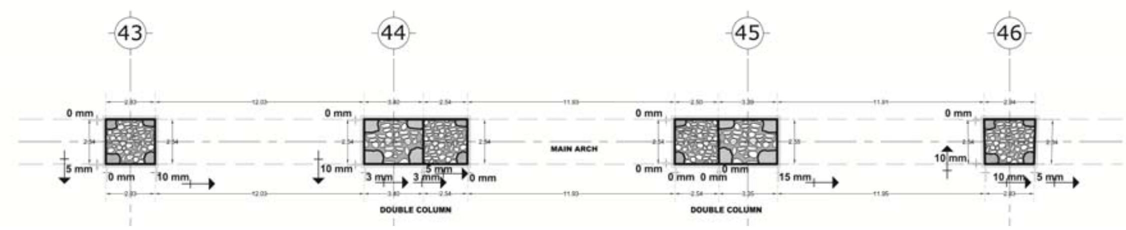



Figure 6: Measured out of plumb of the column faces.

Surveys were conducted at the base of each column, in order to know the characteristics of the foundation and its conservation status, as well as the type and properties of the subsoil. The tallest columns in the canyon area are directly founded on basaltic rock. In the rest of the bridge, columns rest on a layer of compact sandy clay with maximum thickness of 1 m over the basaltic rock. Tests were carried out to determine the geological origin of the two types of stone used for the masonry work. It was determined by laboratory tests that the ashlar stone has a density of 2.3 and a compressive strength of 25 MPa, while for the stones of the raw masonry these values are 2.0 and 7.0 MPa, respectively.

Columns, arches and channel were thoroughly inspected to obtain a comprehensive assessment of the structural damage. Six main types of damage were identified. Damage types throughout the bridge were recorded as shown in Figure 7 for the central area. Each type of damage is described below, along with comments about its origin and significance.

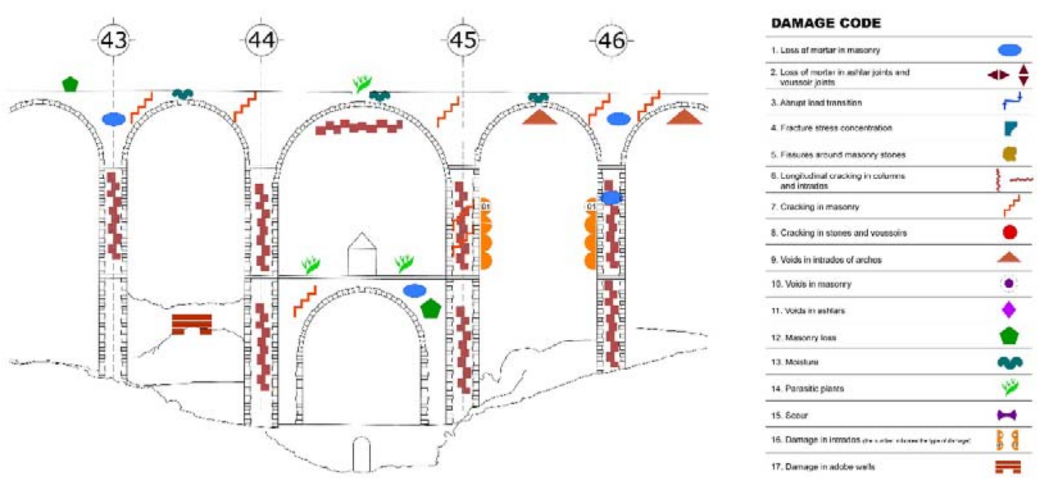



Figure 7: Types of damage recorded in the central part of bridge.

**Loss of mortar in raw masonry**. Different zones of the aqueduct show a partial loss of the mortar used as a binder in raw masonry. In most of the observed sites, mortar loss was only superficial; however, there are some parts where the problem is exacerbated and the lack of mortar is such that some of the stones have fallen, leaving cavities of remarkable size (Figure 8). These cavities reduce the structural capacity of the masonry work and accelerate its deterioration. Similar effects are produced by the mortar loss in horizontal joints between the corner ashlars of the shaft of the columns (Figure 9) or between the voussoirs of the arches.

Figure 8: Loss of mortar in raw masonry.

Figure 9: Loss of mortar in horizontal joints between ashlars (notice stone wedges placed to level corner ashlars).

**Fracture of ashlar stones**. Breaking of ashlars is caused by stress concentrations (Figure 10) occurring mainly after a mortar loss in horizontal joints between columns ashlars. Under these conditions, the load transfer from the upper to the lower stones no longer occurs through the entire mortar joint, but exclusively through several small stone wedges that the builders had placed to align and level the corner stones of the columns (Figure 9). These stress concentrations in the small areas covered by the wedge stones, in most cases only give rise to chipping of the stone edges, but, in some cases, the whole stone is fractured.

Figure 10: Cracking of raw masonry, affecting a corner ashlar.

**Separation of the corner ashlars from the raw masonry**. Extended cracks were found at the boundary between column ashlars or arch voussoirs and the raw masonry (Figure 11). The problem is attributed, on the one hand, to the stones lacking of protruding teeth to anchor them to the raw masonry and, as will be discussed below, to the great difference in stiffness between stone ashlars and raw masonry. this gives rise to high compressive stresses in the corner stones, which, eventually, separate from the inner raw masonry get to work as four isolated columns, that, because of their greater stiffness take most of the gravity load and tend to separate from the rest of the shaft and bend outwards. A similar situation occurs in the arches, between voussoirs and raw masonry (Figure 12). The vertical cracking of the columns is the most widespread and most worrying problem. There is evidence that several of these cracks had been repaired in the past by grout injections; nevertheless, most of them are now opened again, indicating that this solution was insufficient.

Figure 11: Vertical crack separating corner ashlars from the raw masonry. (It can be noted that the crack had been repaired in the past and is now open again).

Figure 12: Typical cracking at the intrados of an arch, due to separation of the corner ashlar from the internal raw masonry.

**Defective support of arches on columns**. This deficiency comes because the base of the arches is much smaller than the upper end of the columns, and therefore, arch voussoirs stand only on raw masonry and not on the corner ashlars. This situation fosters separation of the column ashlar corners from the inner raw masonry, through elongated longitudinal cracks at the interface between the two materials or, occasionally, in the adjacent raw masonry (Figure 13). The most critical case of this type of damage occurred in one of the tallest columns and had been corrected, probably since the time of construction. Repairs involved tearing off a considerable part of the column shaft, and substituting both ashlar and raw masonries by a new masonry with smaller stones and with a more effective bonding array.

Figure 13: Damage due to eccentric load transfer from the arch to the top of the column.

**Cavities in raw masonry**. In the intrados of several arches, raw masonry shows cavities of considerable depth (Figure 14). Most of these cavities can be attributed to runoff water from the upper canal, due to its cracking or to loss of its waterproofing coating. There are also cavities in the faces of columns and arches, essentially caused by the loss of mortar. A particularly delicate case is that of cavities and loss of masonry stones at some of the column bases, because storm water runoff eroded the masonry mortar (Figure 15).

Figure 14: Cavities in raw masonry.

Figure 15: Scour damage at a column base.

## 4 STRUCTURAL ANALYSIS

Several types of analyses were carried out to evaluate the structural safety of the major arch bridge in its current state, mainly in order to serve as a basis for defining rehabilitation needs, but also to assess the knowledge at the time of construction, through the appropriateness of the original structure. Analyses were made on a finite element model representing the whole bridge (Figure 16), although in most cases only the behavior of the critical segment, which is comprising the arch over the canyon and its two immediate adjacent arches on each side, was studied. The mechanical properties of the component materials (density, Young modulus and tensile strength) were derived from standard laboratory tests on stones of the same geological origin than those with which the structure is built.

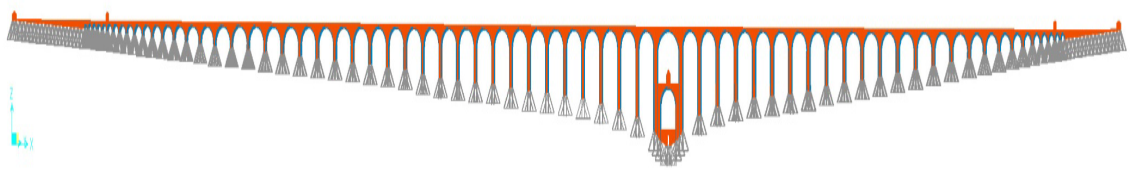


Figure 16: Finite element model of the whole bridge.

The analysis of the structure under gravity loads was performed to quantify its total weight, in addition to stresses and displacements. The weight of the whole structure is 14,285 t, of which 12,225 t correspond to raw masonry, and the remaining 2,060 t to the ashlar masonry of the columns corners and the voussoirs of the arches. Loads on columns varied with their height, reaching 625 t for the tallest columns. Figure 17 shows the distribution of vertical stresses generated by gravity loads along the central segment of the bridge. Great concentration of stresses occurs at the borders of the columns sections, and the maximum stresses occurs the corners of the column base which, for the tallest column adjacent to the central arch reaches 13 MPa, while the average stress on the column section is only 0.6 MPa. As previously pointed out, this situation can be attributed to the ashlar stone possessing greater stiffness than the rest of the masonry. It should be bear in mind that the model did not consider the separations between the two materials, and, therefore, underestimates stress concentrations occurring in some of the actual columns.

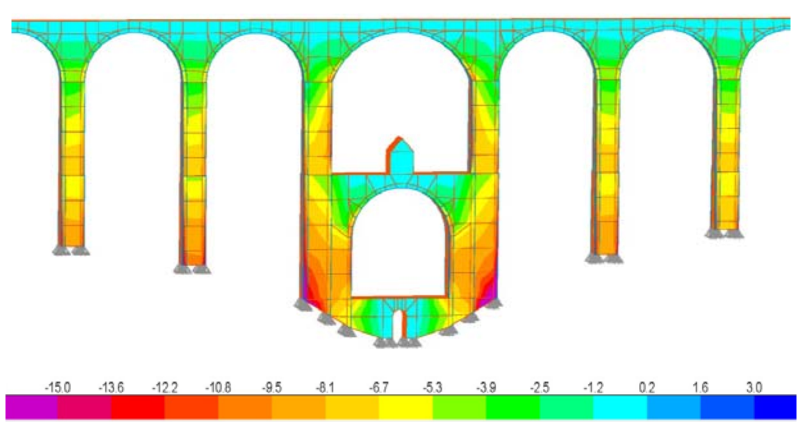



Figure 17: Vertical stresses in the central part of the bridge under gravity loads.

The site is more than 250 km away from the subduction zone of the Mexican Pacific Coast, where earthquakes of great magnitude are generated; therefore, the level of seismic hazard for the arch bridge is moderate, also because the structure is founded on very firm soil. The code specified design spectrum is shown in Figure 18 and includes correction factors taking into account the importance of the work, the type of soil and the non linear behavior of the structure.

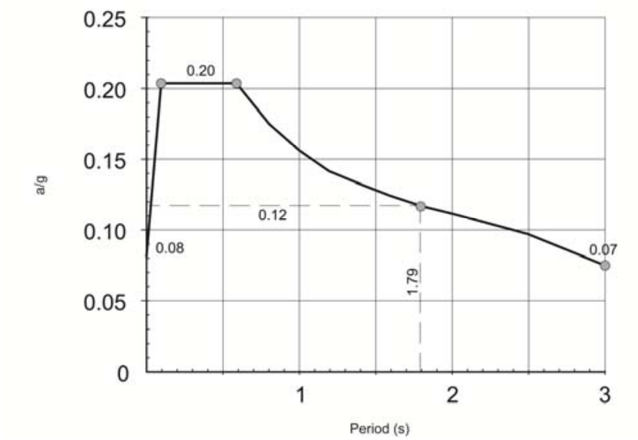



Figure 18: Seismic design spectrum.

The finite element model that had been employed to study the effects of gravity loads across the whole bridge was also used to determine its dynamic properties and response to the design spectrum, through a modal dynamic analysis. The first mode of vibration has a period of 1.79 s, and corresponds to the vibration of the structure in the direction perpendicular to its plane. As can be appreciated in Figure 19, lateral displacements are concentrated in the central zone of the bridge. The relatively long period of vibration of the fundamental mode is due to the flexibility of the cantilevered columns in the transverse direction, and is favorable in this case of a very firm ground, because it falls into the descending branch of the design spectrum, where the spectral ordinate is rather small (0.12 g). In the first six natural vibration modes correspond to the displacements in the transverse direction, and their participation in the overall response of the bridge show a fast decrease with the order of the mode.

Figure 19: Natural modes of vibration (plan view).

A spectral modal analysis was performed for the combined effect of the dead load plus the design seismic action in the direction normal to the plane of the bridge. The main results of this analysis are the following. The greatest lateral drifts occur in the central and tallest segment of the bridge. The maximum lateral displacement is 13.5 cm. Maximum vertical stresses occur at the corners of the columns bases, reaching 9.7 MPa in compression and 4.7 MPa in tension. As shown in Figure 20, in raw masonry stresses are much smaller than in the ashlars. Maximum computed tensile stresses widely exceed those that can be withstood by the masonry; therefore, even seismic events of of much lower intensity that the design earthquake should have occurred in the lifetime of the bridge were likely the cause of the horizontal cracks now appearing in the mortar joints of ashlar and raw masonries.

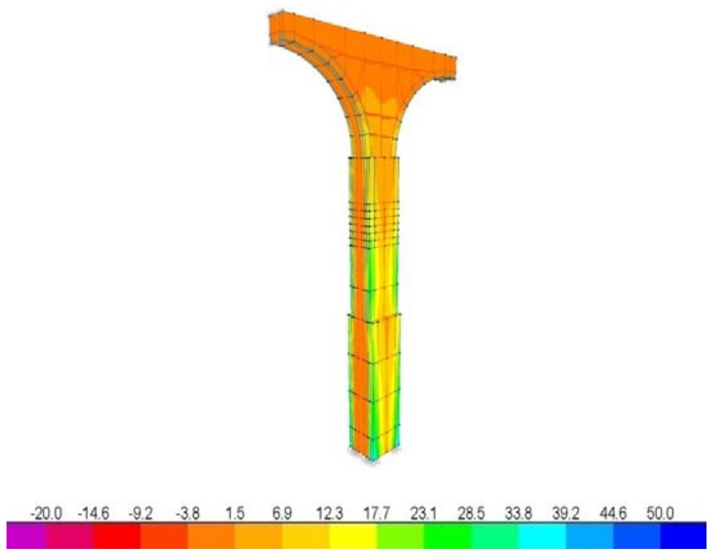



Figure 20: Vertical stresses from the modal spectral analysis in the tallest column.

It must be considered that, in a strong earthquake, flexural cracking will reduce the stiffness of the structure and, therefore, increase its fundamental vibration period, then shifting it to the region smaller ordinates of the design spectrum. On the other end, concentrations of compressive stresses could have be dissipated by small local crushing.

The site of the aqueduct is far from areas affected from extreme weather events; nevertheless, due to its relatively flat topography, is often subject to rather strong winds. Wind forces were computed for an event with a return period of 200 years, corresponding to a regional wind speed of 130 km/h. A simplified wind analysis, ignoring dynamic effects, was carried out for the tallest column of the bridge. Results are shown in Figure 21, in terms of the equivalent lateral thrusts exerted by wind along the column height, and are compared to those of a similar simplified static analysis performed for computing seismic effects. The total lateral force due to wind at the column base is equal to 17.3 t. In comparison, the lateral base force applied by design quake is almost 3 times greater. A more sophisticated analysis of the effects of wind forces was not considered to be necessary, given that the structural safety against collapse is clearly governed by seismic effects. Nevertheless, it must be taken into account that, the simplified analysis shows that the wind forces are large enough to induce significant tensile stresses in the tallest columns. Therefore, even winds of smaller intensities than those prescribed by the code, as those having return periods of a few decades, are strong enough to induce stresses exceeding the tensile strength of the masonry, and, therefore, to contribute to the horizontal cracks found at the mortar joints.

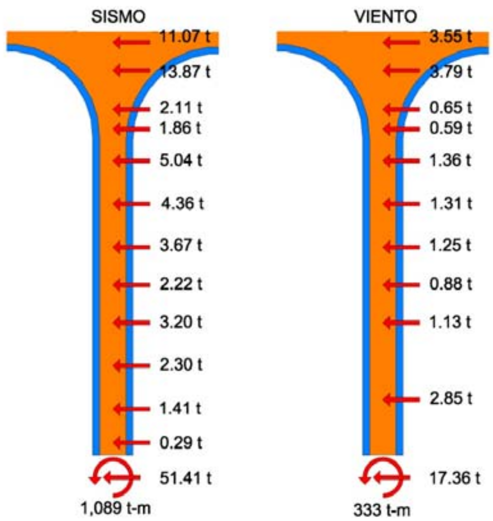



Figure 21: Comparison of results from simplified static analyses for wind and seismic actions.

## 5 DIAGNOSIS OF THE STRUCTURAL SAFETY

Presently, most of the major arch bridge shows an acceptable state of preservation. Its structural damage could be qualified as moderate, and the structure retains its original geometry with no distortions that could be related to differential settlements of the foundations. Additionally, the afore described analyses indicate that in most of the bridge structure, level of stresses due to its own weight was within safety limits, when the structure was in its original undamaged condition. The analyses have also shown that original structure had an adequate safety against collapse under expected extreme earthquake or wind events, as those postulated by the building codes. Nevertheless, even moderate seismic events could induce in the tallest columns tensile stresses causing cracks at the mortar joints.

On the other hand, imperfect bonding between the two materials has caused, in the tallest arches and columns, a progressive separation of the corner ashlars from the inside raw masonry, thus affecting the composite action between the two parts of the structure. This situation could initially have been caused by high tensile stresses due to gravity loading, and then significantly aggravated by the loss of mortar due to rain and spilling from the canal, as well as by additional stresses due to earthquakes. In the early 80s several of the vertical cracks at the interface between the two had been consolidated, however, now they are all open again. This indicates, first, that the consolidation of the cracks was insufficient to stop the damage and, second, that the problems are now far more severe than in the past, because crack thickness is larger than before and new cracks have appeared. In conclusion, due to the afore-mentioned problems, the tallest columns are unsafe in their present situation.

The separation of ashlar stones from the raw masonry is attributed to the loss of bond between the two parties. In columns, the four corners are much stiffer than the central part of the section. This situation causes the gravity loads be concentrated in the corner stones, which, therefore, tend to bend outwards and to separate from the core of the section. The slenderness of the corner elements increases their tendency to bend out of the plane of the bridge. Something similar to that described for columns, occurs in the arches due to the absence of an effective link between the two parts that tend to separate: the outer voussoir bands and the inner raw masonry. Another critical situation of the same kind occurs at the top of these columns, where arch voussoirs stand only on raw masonry and not on the corner ashlars, thus fostering the separation the separation of the two parts. The advance of the separation of the corners generates that some of them may become fully detached from their inner and fail by buckling, as happened in the past in one of the columns. Therefore the three mentioned problems requires priority interventions to restore the integral action of the two materials.

An additional situation of risk requiring priority attention is found in the bases of the columns of the central arch, on the ravine, where degradation of the raw masonry due to scour at the columns base has generated an imperfect contact between the column base and the underground rock, thus giving rise to high stress concentration at the ashlar corners. This makes it necessary to protect the bottom of these two columns to prevent that scour of the mortar continue to advance due to storm water runoff, thus affecting the safety of the structure.

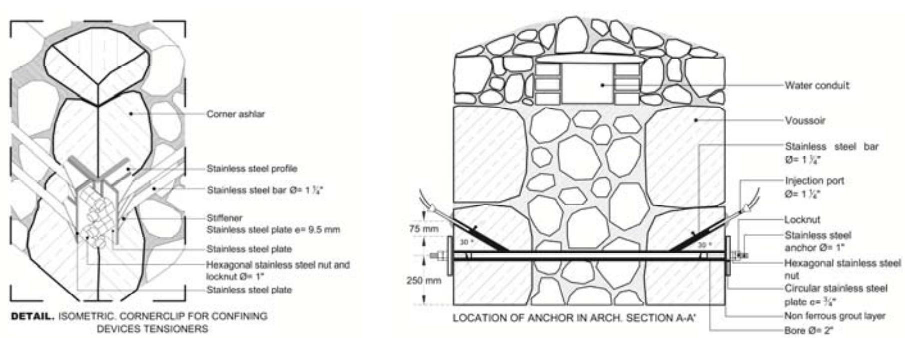
## 6 PROPOSED STRUCTURAL REHABILITATION ACTIONS

To restore the structural integrity of the aqueduct, several actions have been proposed by these authors [6]. The most important ones are described in the following paragraphs; they are concentrated in the tallest columns of the bridge.

**General rehabilitation of masonry**. This concept includes various actions aiming at restoring the integrity of the masonry work. It includes removing the parasitic flora; replacing the mortar that has been lost in the joints between ashlars and between stones of the raw masonry; replenish the stones that have fell or are broken; and perform consolidation grouting to fill cavities and internal cracks in the masonry.

**Confinement of columns**. To restore the composite action of the corner ashlars and the inner raw masonry, four stainless steel angles will be placed at the column corners and interconnected through eight bars of the same material (Figure 22a). These bars are provided with threads and nuts on the ends, so the device can be tightened to confine the column section avoid the development of vertical cracks in the columns. These confining devices will be mainly placed at the upper end of the columns to restrain the opening due to the eccentric thrust of the arch, as well as at the column base where stresses due to gravity and lateral loads are maximum. They constitute a reversible intervention, which can be removed by loosening the nuts (Figure 23).

**Confinement of arches**. To tie together the corner voussoirs and the raw masonry at the bottom part of the arches, thereby avoiding reopening of longitudinal cracks that have occurred in the bottom of the channel, it has been proposed to place transverse tie rods crossing the width of the arches. (Figure 22b). These bars will be embedded in the stone voussoirs and the raw masonry, and will be prestressed.




a). Columns b). Arches

Figure 22: Devices for confinement of columns and arches.

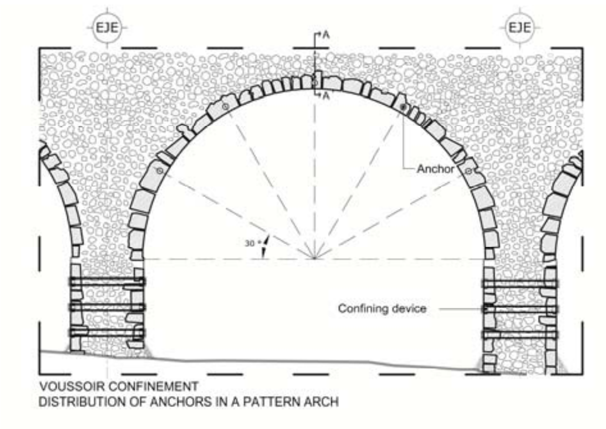



Figure 23: Location of confining devices for columns and arches.

**Other actions**. The column bases and foundation of the span crossing the ravine should be restored to their original shape, and the topography of their surrounding land must be modified to avoid scour at the column bases. Furthermore, the surface of the subgrade portion of the column must be protected with a waterproof coating until at least 0.5 m above the ground level. The remains of the adobe walls deserve to be preserved, as evidence of the construction system used to build the arched bridge. They should be protected to avoid increasing degrading.

The rehabilitation work should be taken advantage of, for performing several studies about the mechanical properties of the materials and the inner status of the structure, mainly by means non destructive techniques, but also with minimally destructive ones, as video probes. The main purpose would be to gather information for calibrating analytical models used to determine the response of the structure and thus have a more precise about the structural behavior it will have in the future. Ambient vibration tests will be conducted in different parts of the structure to measure its dynamic properties at local and global levels.

## 7 CONCLUDING REMARKS

The major arch bridge, thanks to the excellent quality of its design and construction, has managed to survive for nearly 450 years; however, it now shows some structural damage in its columns and arches. The main problem is a progressive separation between the parts built of stone ashlars and those of raw masonry, which are losing their composite action. These damages had been repaired in in the past, but with the passage of time have appeared again and have spread.

The results of the theoretical analyses of the structure indicate that, under the action of its own weight, the bridge has an acceptable level of safety. They also reveal that severe earthquakes like those specified by the design code for checking structural safety against collapse, would generate high compressive and tensile stresses that could be absorbed by the structure in its original undamaged condition, but that, in its present conditions, could produce partial collapse of some of the tallest columns. On the other hand, even moderate earthquakes, which should have occurred several times in the lifetime of the structure, would induce stress exceeding the tensile strength of the masonry, and therefore, are considered to have been the main cause of the present structural damage. Wind effects are less critical than those of earthquakes, but high enough to cause cracking at the bases of the tallest columns, and could have also contributed to the damage.

Therefore, there a need to restore the structural integrity of the bridge, for which these authors have proposed the consolidation of the masonry, along with a series of retrofitting actions aiming at restoring the composite action between the two kinds of stonework. The rehabilitation of the structure must be addressed soon to prevent further damage and deterioration that could cause a partial collapse of the bridge. The rehabilitation must be accompanied by a rigorous monitoring of the structural health of the bridge and by a maintenance program, in order to preserve this important work for much longer.

## ACKNOWLEDGMENTS

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## REFERENCES

[1] F. Calderón de la Barca (XIX Century), *La vida en México, Porrúa,* 2003.

[2] C. Cantú Bolland, El Puente-acueducto Tembleque. *Cuadernos de Arquitectura Virreinal,* No. 16, Facultad de Arquitectura, UNAM, 1995.

[3] A. Ciudad Real (XVI Century) *Tratado curioso y docto de las grandezas de la Nueva España.* Inst. Inv. Estéticas, UNAM, 1993.

[4] C. González Lobo, La obra de Fray Francisco de Tembleque en la región de Zempoala-Otumba. *Bitácora-arquitectura,* No. 12, 2001.

[5] H. Ramírez de Alba et al., Revisión estructural arquería mayor del acueducto del padre Tembleque. *Memorias VIX Congreso Nacional de Ingeniería Sísmica*, Boca del Río, Veracruz, 2013.

[6] R. Sánchez Ramírez, R. Meli, Diagnóstico y proyecto de rehabilitación de la arquería central del acueducto del padre Tembleque en Zempoala, Hidalgo. *Téchnical Report to DGSM Conaculta,* México, 2014.

[7] O. Valdés, *El padre Tembleque, Editorial Jus,* 2010.