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Hormigón y Acero 2017; 68(283):e23–e34

HORMIGÓN y ACERO
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Robustness: The quality Ribera missed in 1905

Robustez estructural: la cualidad que echó de menos Ribera en 1905

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Received 7 April 2017; accepted 12 April 2017
Available online 20 November 2017

Abstract

On April 8th 1905 the roof of the 3rd reservoir of *Canal de Isabel II* in Madrid collapsed, being one of the most disastrous accidents that has occurred in the history of Spanish construction: 30 people died and 60 were injured. At the time, the event was subject to great speculation and investigation, both in the engineering community as well as in the court of law. Despite this, it did not result in convincing conclusions regarding the cause of such a disaster. As far as the scope of this article is concerned, this collapse serves as a great example of the disproportionate consequences a lack of structural robustness can have. Modern codes contain specific requirements whose compliance is believed to implicitly guarantee a robust design. However, such requirements were not accounted for by the designer José Eugenio Ribera, one of Spain's leading civil engineers. As a result, a single execution defect could lead to disastrous consequences.

Keywords: Robustness; Third reservoir; Vaults; Collapse

Resumen

El 8 de abril de 1905 se produjo el colapso de la cubierta del Tercer Depósito del Canal de Isabel II, hecho terrible que provocó la muerte de 30 personas durante la construcción y dio pie a una polémica extraordinaria de la que sin embargo no se extrajeron conclusiones convincentes. A los efectos del presente artículo, este episodio es un buen ejemplo de estructura proyectada con ausencia de análisis de robustez, cualidad que se supone de implícita satisfacción si se cumplen los requisitos normativos de los códigos modernos, pero que fue obviada por el proyectista de la cubierta, el insigne y emprendedor José Eugenio Ribera, lo que motivó que un defecto en la ejecución tuviera consecuencias desproporcionadas.

Palabras clave: Robustez; Tercer depósito; Bóvedas; Hundimiento

1. Introduction: robustness as a structural quality

A structure is understood to be robust if the failure of a specific element does not provoke disproportionate consequences. For example, the failure of one of the catenary cables on the Golden Gate Bridge would provoke a disaster, whereas the failure of one its hangers would only cause damage of a limited scope.

It is considered to be a structural quality which is accepted as appreciated but in most cases lacks quantitative precision. Something similar seems to happen with the concept of ductility, where implicit satisfaction is assumed if current modern regulatory codes are met.

However, this has not always been the case.

In fact, the concept of robustness is relatively new: the partial destruction of the 22-story Ronant tower block in London in

DOI of original article: http://dx.doi.org/10.1016/j.hya.2017.04.011
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http://dx.doi.org/10.1016/j.hya.2017.10.001
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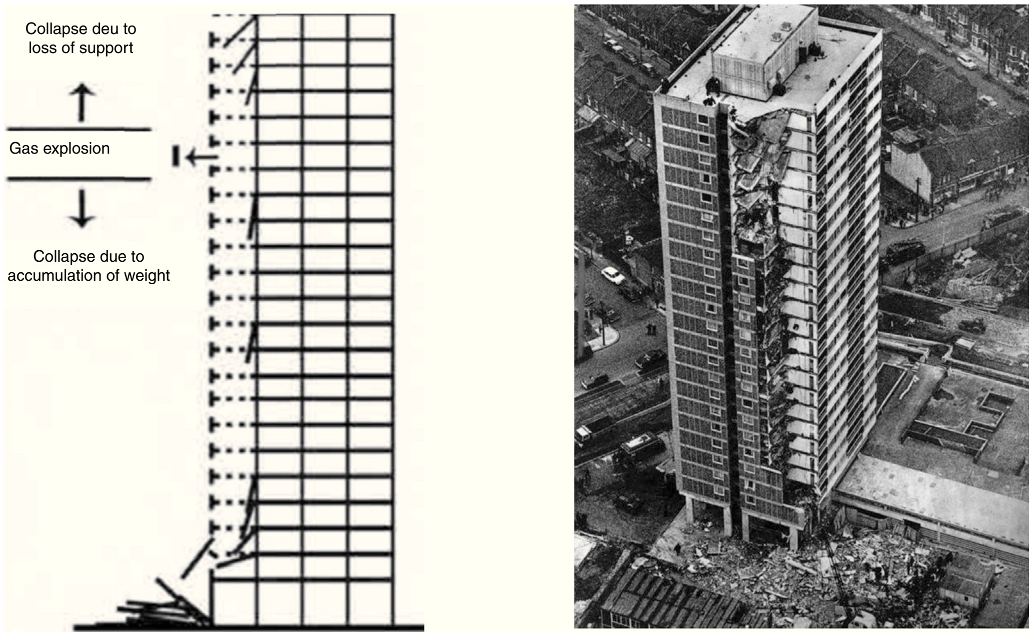


Figure 1. Partial collapse of the Ronan Point tower on May 16th 1968. London [2].

1968 (Fig. 1) was the cause of the incorporation of the concept of progressive failure into the British codes (and hence into the rest of the regulations), along with the consideration of accidental or unusual actions [1]. The accident happened in the early hours of the morning of the 16th May 1968, when one of the neighbours was about to prepare a cup of tea. On lighting a match, the stove gave off an explosion of gas which then blew out the windows and the external walls of the living room. As a result the whole corner of the building collapsed like a line of dominos [2]. The external wall enclosure was made up of floor to ceiling pre-fabricated reinforced concrete panels set on concrete floor slabs. So, when one of these walls failed it consequently provoked the collapse of all those above it, and with the accumulation of weight, the corresponding destruction of those below. These employed the Larsen-Nielsen system which filled the joints between the walls and the slab with mortar but without any reinforcement whatsoever. Therefore, on suffering a horizontal action, as with the explosion, its resistance capacity was highly reduced and therefore failed. But more important than the magnitude of this horizontal action is that it caused the progressive collapse as a result of a partial failure.

More recently we could witness the total collapse of the World Trade Center towers in New York as the result of the impact of two BOEING 767s (Fig. 2). The structural damage produced by the impact was aggravated by the fire from the planes' fuel which caused the area to lose its capacity to withstand the load born upon it, and on collapsing provoked the progressive failure of all the floors. As a result of these attacks, a manual titled the FEMA-426/BIPS-06 [3] was issued in the United States to mitigate the effects of terrorist attacks on buildings. At the same time, it served as a wake-up call to consider robustness in structures, and nowadays a number of teams are working on this matter.

Disasters such as the ones stated before have motivated current structural design to include calculation criteria and structural details to guarantee structural robustness, which is understood to be “the capacity of a system to withstand a local failure without suffering disproportionate damage in relation with the cause which originated the failure”.

Figure 2. Attacks 11th September 2001. New York.

However, this quality may not be present in existing structures, which could become a conditioning factor with regards to its analysis and possible refurbishment.

This article addresses the need for the technician who confronts the analysis of an existing construction to be aware of this quality, which is hence clearly expressed in cited examples.

This is also the case of the collapse of the roof of the Third Reservoir of Canal de Isabel II in Madrid in April 1905 [4]. This is an excellent example of a projected structure lacking analysis of robustness. 30 people died and 6 people were injured in this accident, which is one of the most serious to happen in building in Spain and yet is still little known. The following section will look at this case to analyze its cause and subsequently reflect on the subject of robustness in existing structures.

2. The collapse of the third reservoir of Canal de Isabel II

2.1. From the construction of Canal de Isabel II to the collapse of the roof

April 1905 found Madrid at the height of its growth, receiving more than 10,000 people annually. This growth triggered a number of significant urban modifications, including, since its creation in 1858, that Canal de Isabel II offered the city's water supply.

By the end of the century, existing reservoirs were deemed insufficient, and in 1881 a new one had been projected in the Campo de Guardias lying below the place known today as Santander Park.

Its construction was controversial from the very beginning, provoking several changes which were not solved until the 10th December 1901 when a Royal Order issued an international competition. This highly disputed competition was won by José Eugenio Ribera, one of Spain's leading engineers, whose solution employing a system of vaults was the most economical and could be executed in the shortest period of time.

The construction of the roof was carried out at the same time as the four chambers in which the reservoir was divided. The works began on the 4th chamber which was located to the south in spring 1904. In April 1905, this 4th chamber was practically covered, whilst in chambers 1 and 2, the columns and the main girders had been placed and the placement of the vaults was beginning. Between the 5th and 6th of April, a series of load tests were carried out, consisting of placing an 80 cm layer of soil along the length of one strip of vaults. The soil was taken from the adjacent vaults hence leaving them without soil cover. The following day, 7th April, the works of distributing the soil over the vaults continued. On the 8th, early in the morning, the collapse happened (Fig. 3).

Figure 3. General view of the disaster area (*Nuevo Mundo*, April 1905).

2.2. Characteristics of Ribera's roof

In 1902 Ribera had constructed the Reservoir of Roces III in Gijón employing a system of vaults which was widely used at this time, called the Monier system. These are very flat vaults (1/10), 5 cm thick covering a 3.8 m span (Fig. 4) However, in contrast to the Monier system where the vaults sprang from abutments, these were supported on the top of beams on 6 m tall columns.

The success of the construction of this reservoir served him to propose the same typology for the new reservoir in Madrid. This time the geometry would be slightly riskier (Fig. 5) with vaults spanning 6 m and 5 cm thick (span/thickness = 1/20). The columns were 8 m tall and 25 cm square on the sides (slenderness 1/32). Though the slenderness of the vaults was endorsed by the aforementioned Monier system, the slenderness of the columns was far greater than current standards; Ribera simply justified this by offering his prior positive experience in the San Sebastián Ceramic Factory which had employed even slenderer columns and with greater loads.

This configuration is repeated over 21 spans of beams and 36 spans of vaults until covering the area of 216 m × 85 m in each chamber (339 m × 216 m in total) (Fig. 6).

As in Gijón, the reservoir was below ground level and the vaults were covered with a regular 20 cm depth of soil.

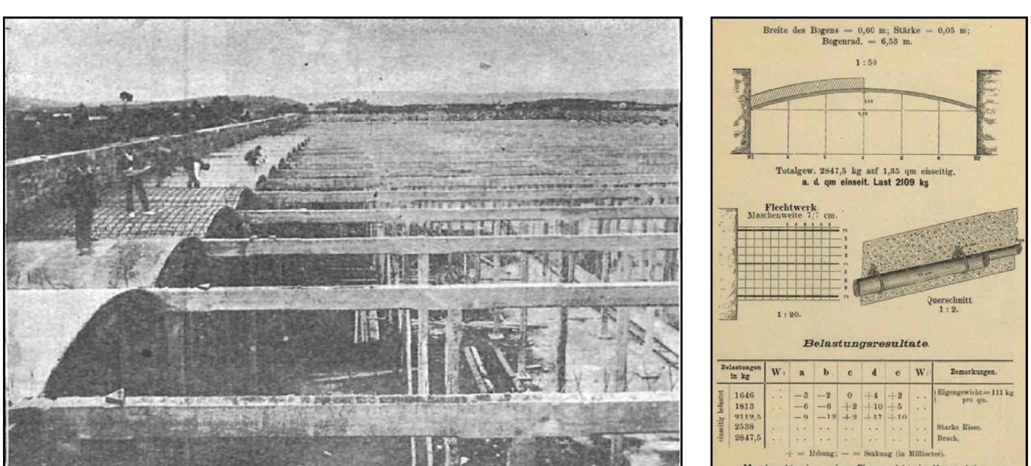


Figure 4. Ribera's Reservoir in Gijón in 1902, and the Monier system for the vaults.

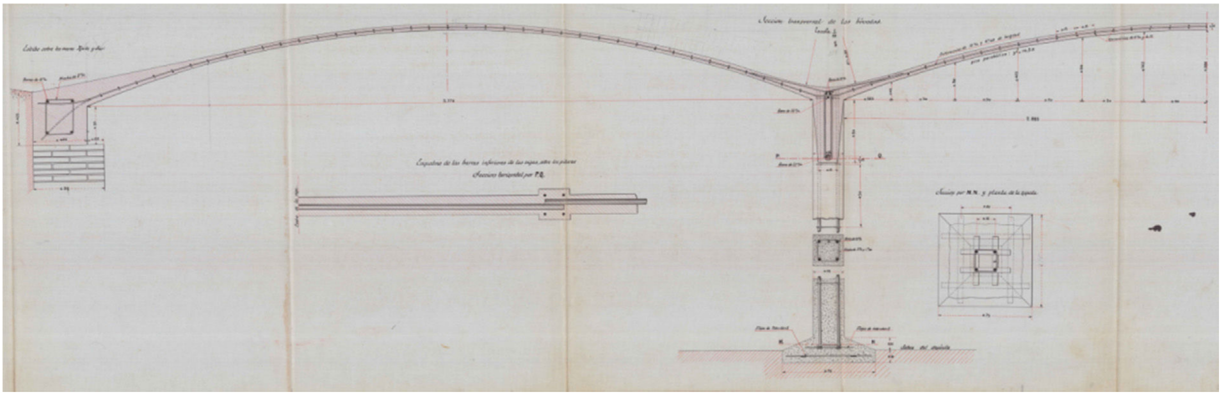


Figure 5. Transversal section of the roof proposed by Ribera.

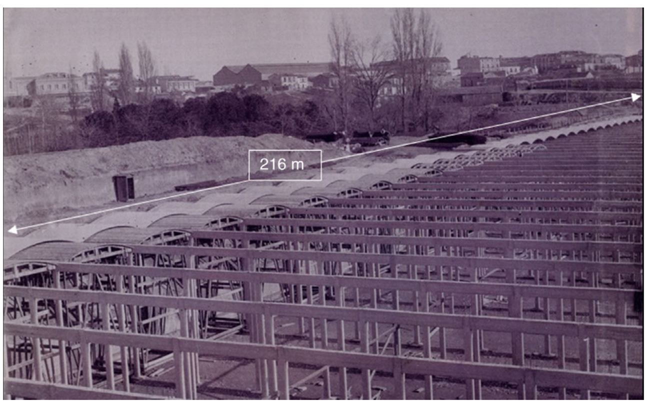


Figure 6. Construction of the Third Reservoir of Canal de Isabel II. Madrid, 1905.

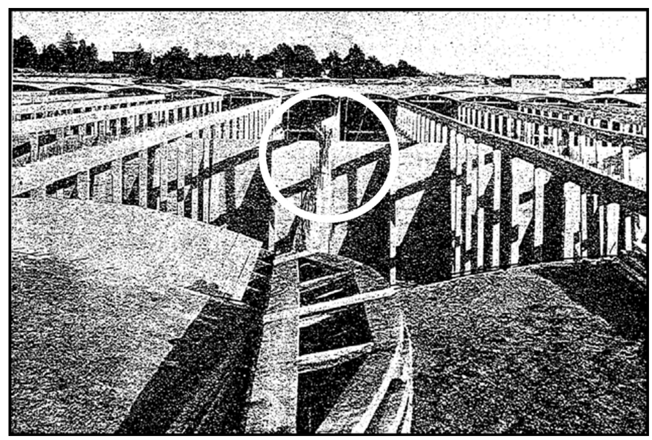


Figure 7. Deformations of a beam in the first chamber, June 1905. Revista de Obras Públicas, 14 March 1907.

2.3. The investigations at the time and the acquittal of Ribera

To investigate the causes of the accident, the same 8th April a Royal Order set up a commission while the Town hall began its own investigations.

As a result of the tragedy Alfredo Álvarez Cascos (Director of Canal), Carlos Santa María (Works Director) were accused along with José Eugenio Ribera, who assumed total responsibility on behalf of the contracted party.

The trial took place at the Provincial High Court of Madrid with the hearing taking place two years later between the 1st and 8th April 1907. Ribera's defence was presented by Melquiades Álvarez as his lawyer and José Echegaray, recently proclaimed Nobel Prize winner, as expert witness. The court acquitted Ribera along with the other two accused.

A determinant factor in Ribera's acquittal was the deformation observed in a number of beams in the first and second chambers at the beginning of June, just two months after the collapse (Fig. 7). These gave rise to numerous articles and opinions attributing the failure of the vaults, to a certain degree, to have been caused by the high temperatures suffered in the month of April. In fact, surprising as it may seem (it is to be taken into account the collapse occurred at 7 o'clock in the morning) most of the existing bibliography to this respect has this hypothesis as the main cause of the collapse. Regarding the real effect these temperatures may have had upon the roof, along with other the actions which could be applied shall be addressed in the following section.

Going beyond the final court sentence, the collapse had greater and further repercussions. This echo was due, on one hand, to the criticisms related to the construction of the Third Reservoir from the preliminary Project and all the engineers which had been involved, but, above all, due to the doubts created in the use of reinforced concrete because of the uncertainties derived in its use. However, the attitude at national level from a number of technical forums in defence of Ribera as a constructor and concrete as a construction material, along with international contributions from personalities such as Fritz von Emperger, Director of the prestigious magazine *Beton und Eisen*, or Hennebique himself, who could see his flourishing company endangered, permitted confidence to be upheld in the new material.

With regards to Ribera, the investigation brought to light once again, the audacity to say that he is, without doubt, one of the great Spanish engineers who took on a decisive role in the introduction of reinforced concrete in Spain. In the project of the Third Reservoir he risked too much, provoking a disaster which accelerated the transition towards a new period in structural concrete that would be under the protection of greater scientific knowledge and the first building regulations. In this period, Ribera would also have a leading role.

2.4. Analysis of the third reservoir's roof

2.4.1. Approximation of the resistant behaviour of the roof

Ribera's desire to reduce to a constructive minimum the thicknesses of both the vaults and the columns led him to design an extraordinarily slender structure whose behaviour was not fully known, and even today is considered complex.

In fact, an initial approach to Ribera's design of the roof, through a typological analysis, brings to light how this construction would suppose a great advance in the development of a structural typology which would have enormous exposure and extension between the 1930s and the 1970s: namely the slender concrete shells [4]. This advance seems to be involuntary (not even calculating the concrete section to withstand the axial loads), although it is possibly full of intuition as shown with the load tests.

However, the collapse supposed a standstill and a backstaging of this structural form, which years later, along with an important mathematical development, would offer a change towards some of the most daring and suggestive forms known in engineering.

With regards to such behaviour, in [4] we analyze the distinct types of failure and the main causes which may occur in a structure such as the roof of the Third Reservoir based on simple conditions of equilibrium in the vaults and simple calculations in the portal frames.

This analysis, at times crude and always approximate, nevertheless offers us the possibility to discard a number of failure modes and address the investigation. A series of results are highlighted as follows:

- A thermal increase in the roof was discarded (supposing it even happened) as the sole cause of a mechanism of collapse. Effectively, the axial forces of the vaults are far lower than those which these can withstand in absence of other live loads (apart from the uniformly distributed soil loads). Additionally, the behaviour of the beams with the already constructed vaults would be very different to what produced the instability in the first chamber.

  The precision of these results demanded a very complex analysis because of the uncertainty of the actions to be considered as well as the noticeable non-linear behaviour of the structure, as the forces depended on the structural stiffness of the different structural elements, and that, on the level of imposed deformations and curvatures. In addition, the analysis of said thermal increases could not be separated from the analysis of the roof regarding the rest of the imposed deformations, especially those due to shrinkage (increased by cooling).

  Given the importance that this aspect received in the trial, the following section shall look further in depth at the behaviour of the roof when faced with imposed deformations.
- Another question which was discarded was a defect in materials as being the possible cause of the collapse of the vaults. The reason is that the stress that these are working under is very low, even with the levels of soil reached during the load test referred to in previous sections (0.80 m).
- Regarding actions of the project, that is to say the uniformly distributed soil loads, columns, beams and the vaults, would all be safe covered by wide safety margins.
- Other types of failure are also discarded where the accident could have been caused by a rotation of the footings or failure in the portal frames.
- However, when faced with non-symmetrical loads on vaults, the roof is extremely sensitive and the mere check of equilibrium of the vaults with their gross cross-section leads us to extremely unlikely results which would not explain that the roof of the fourth chamber had been almost fully completed: differences in soil thickness on one side and another on a 3 cm thick vault distributed over the tributary width of the columns would produce a mechanism of collapse.

  It was therefore necessary to look further into a series of aspects such as the contribution of reinforcement in the vaults capacity, the possibility of transversal distribution within a same vault alignment, as well as, and especially, the effects of shift in the supports in the stability of the vaults. These analyses shall be commented on in the following sections.

2.4.2. The roof faced with imposed deformations

This investigation has discarded that imposed deformations could be the origin of the collapse:

The vaults, as soon as they are subject to slight bending, and as a result of their geometry and conception, would crack and tend to form resistant statically determined mechanisms and relax the forces in the rest of the spans.

In this sense, Fig. 8 shows the deformation of the transversal section of the vaults assuming that the crown of the central one were the neutral point for horizontal displacements and, simultaneously, a three-hinge arch was formed. This hypothesis would be coherent with the deformation scheme presented by the

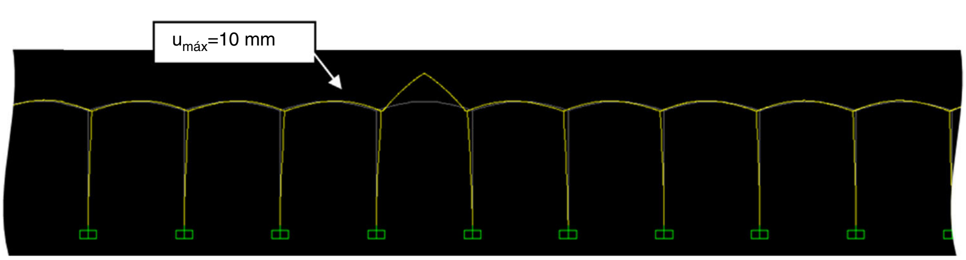


Figure 8. Deformation of the vaults supposing rotation in the central area (for reasons of clarity, only the 10 central vaults are shown of the existing 36 spans).

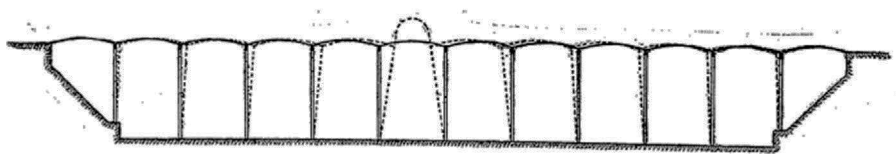


Figure 9. Deformation of the roof faced with thermal increase in accordance with the publication by Fritz von Emperger.

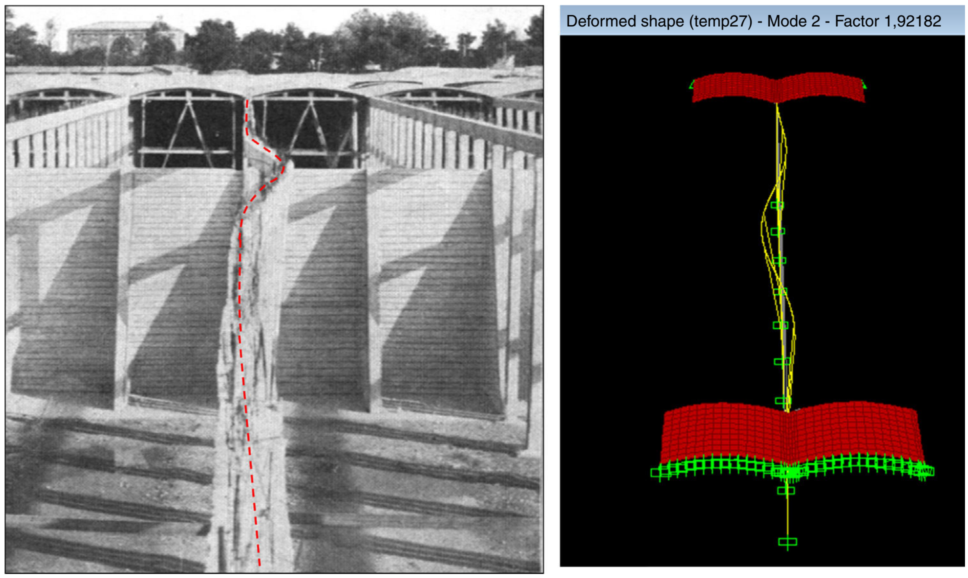


Figure 10. Observe the similarity between one of the failure modes found and the instabilities in the first and second chambers.

Austrian engineer Fritz von Emperger as a part of his analysis as the cause of the collapse (Fig. 9).

The magnitude of the displacements shown in the deformed shape brings to light the minor effects this local discontinuity produced upon the rest of the transversal section of the roof structure. This fact is confirmed if the internal forces are analyzed, showing that they are very low and far from those which would lead to failure of the springing of the arch, which is the weakest in the case of the roof [4]. Once again, stability is guaranteed.

The only "negative" effect that the indirect actions could have in the transversal direction of the roof would be the flattening of the vault shape as a result of shrinkage and creep, although the loss of shape is rather negligible.

Neither would the imposed deformations upon the longitudinal portal frames have an effect on the resistance of the whole structure once the vaults are constructed.

This situation contrasts with the beams in the first and second chambers which, without the vaults having yet been placed, deformed shortly after the collapse in the month of June (Fig. 7). When movement is not limited at the head of the columns, beams become very sensitive to lateral instability induced by axial compression due to restrained movement produced by a thermal increase (specifically at midday and not early morning when the collapse of the fourth chamber occurred).

Díaz-Pavón's doctoral thesis titled *Investigation on the causes which could cause the collapse of the roof of the Third reservoir of Canal Isabel II in 1905* [4] goes into great depth regarding this situation, obtaining safety margins for buckling albeit the fact that neither the length of the beam nor the inertial forces on the columns and beams, nor the thermal increase itself is the same as what really happened in June 1905, allow the justification of such a failure. In this sense, it is to be seen how the buckling mode shown in Fig. 10 resembles the instability in the photograph corresponding to the deformations seen in the first chamber. The fact that it apparently looks like the second mode of buckling would be influenced by a number of variables, especially by geometric imperfections.

These results contrast with the situation in the fourth chamber.

Indeed, its situation when it collapsed was greatly different as the vaults laterally braced the beams and the thermal increases at 7:30 in the morning could not be so high.

Even taking into consideration the thermal increase, the ratio between the theoretical critical axial load of these beams and the load due to thermal increase is in the order of 142 as is shown in Fig. 11. Moreover, the first six modes of buckling detected by the model are found in the plane of the portal frame and not orthogonally which hence lays bare the enormous contribution the vaults have in stabilizing the beams.

2.4.3. *The roof facing gravitational action*

As previously mentioned, when regarding design actions, that is to say, uniformly distributed soil loads, the columns, beams and vaults would be safe, covered by wide safety margins. However, regarding non-symmetrical loads in vaults, the roof is very sensitive.

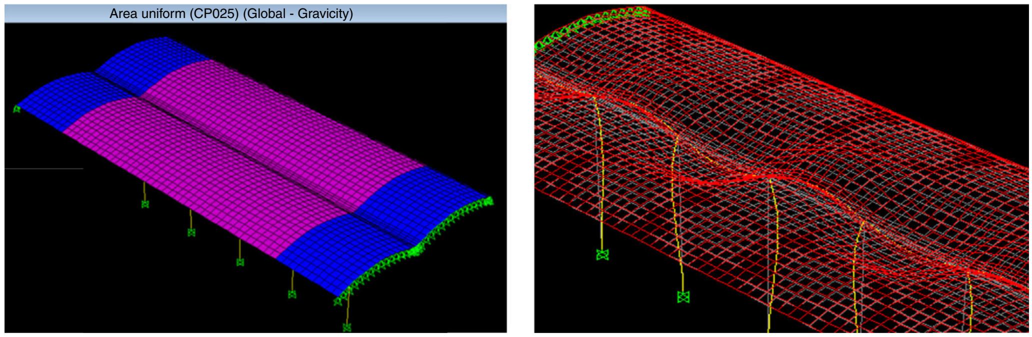


Figure 11. Buckling mode of finished roof. Instability is discarded in the plane of the vaults.

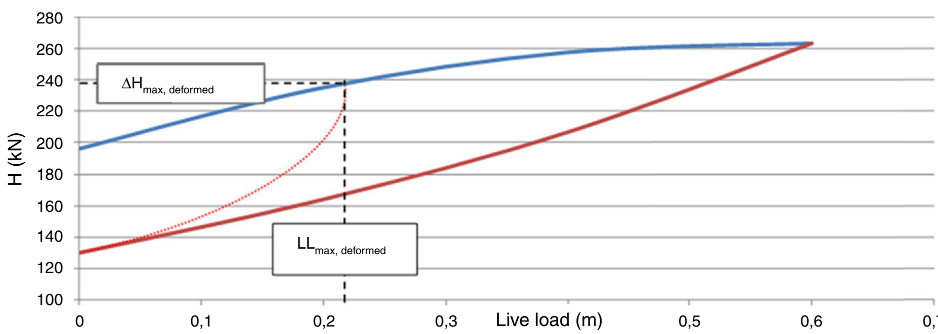


Figure 12. Reactions in the support corresponding to the lines of pressure of maximum (upper curve) and minimum thrust (lower curve), in the case of uniformly distributed 25 cm deep soils and live-load (LL) on haunches. The discontinuous line graphically represents the thrust when a displacemnt of the support occurs.

This sensitivity regarding asymmetric actions is complex to evaluate. The asymmetric load provokes a displacement in the support (a shift away from) which would cause the vault, which mobilizes initially the line of minimal thrust, would continue to increase such thrust as the displacement increases (greater span, less rise, more thrust: the discontinuous line in Fig. 12). The evolution of thrust and shift is clearly non-linear and therefore difficult to foresee. The deformation of the arch depends on the stiffness of the structure as a whole and such stiffness depends on the loads, whose increase could cause the relaxation of the structure (by cracking) so making the deformations ever greater.

This behaviour could appear difficult to interpret. It is evident that a masonry arch (or vault) – and therefore with no bending stiffness –, if the abutment is not able to provoke the necessary reaction, would open and end up collapsing. Whereas, if the section does have bending capacity, the arch would deform, reaching a state of equilibrium which, in the limit, might not need any horizontal reaction and would begin behaving as a *beam* with a curved path.

In the vaults of the Third reservoir, the *beam effect* in any case would be very small, and the reaction, and therefore the axial loads, would increase to maintain the equilibrium of the *arch*.

With regards to carrying out an approximation of this behaviour different models of the structure of the Third Resevoir were carried out [4] which, although necessarily simplified, took into consideration mechanical (cracking and creation of plastic hinges) and geometrical nonlinearities (2nd order effects), which have brought to light the extraordinary flexibility of the structure when faced with an increase in non-symmetric loads.

In addition, the aforementioned models reveal the important role of the presence of rebar in the vaults' behaviour. Moreover, this reinforcement, so dimensioned as to withstand the full axial load in the vaults when faced with uniformly distributed loads, offer the structure a great bending capacity which is especially significant when the thickness of the soils were reduced or non-existent, so conditioning the type of mechanism which forms on the roof.

As a result of said analyses, the graph in Fig. 13 depicts the relative displacement between supports which increases in proportion to the loads so measured as thickness of equivalent soils. The case shown is to apply a uniform 25 cm load, an additional load on the haunches as well as other configurations of possible non-uniform loads.

As may be appreciated in all cases, in reduced differences of loads, the behaviour is clearly elastic and the displacement between supports is very small. However, when the vaults begin to crack the flexibility increases greatly and the displacements shoot up, increasing at an uncontrolled rate for very low increases in load. In this way, the point where the roof collapses is reached, which in any case is caused by the loss of shape of the loaded vault (snap-through failure mode). Failure caused by instability of a column was discarded.

Akin results are obtained for the roof without soils (beyond non-uniform loads). For example, if the load accumulates asymmetrically along half span, the flexibility of the roof is very pronounced as from 10 cm, reaching point of collapse at less than 15 cm.

These values should be considered, in any case, as approximate: on one hand they would be further reduced if the weight

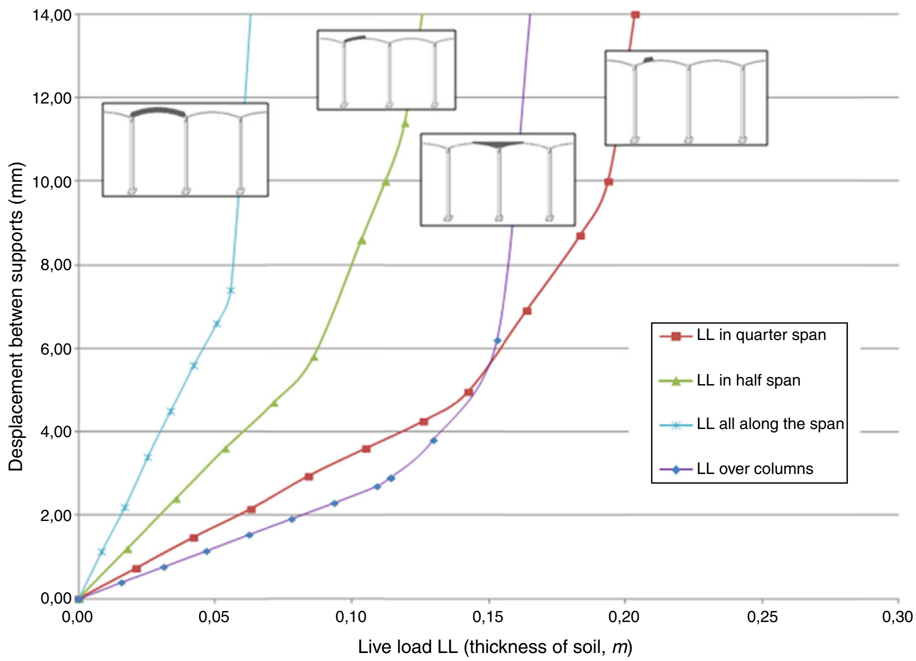


Figure 13. Effects of excess asymmetric loads upon the roof with 25 cm of soil.

(and the impacts) of the workers and wheelbarrows is taken into account (take into consideration the volume of distributed soil over the tributary width of a vault in half span and 0.10 cm deep supposes a weight of 20 kN, whereas the weight of a single loaded wheelbarrow would be of around a tenth of that).

They could also be lower if, instead of the nominal geometry of the vaults, defects of execution or losses in shape due to deformations were taken into account, although the latter, as indicated in the previous section, would provoke a very small effect in our case.

On the other hand, as is demonstrated in [4], the capacity of longitudinal distribution along the same alignment of vaults (behaving as shells), makes the results obtained excessively conservative especially in regard to local situations of loads, that is, those which are not presented longitudinally (for example maintenance loads once in service).

Finally, it was checked that the safety margins of the roof, if it had been completed, in relation to the most unfavourable circumstances which could be applied (non-uniform distribution of soil volumes as a result of a horizontal level of soil, instead parallel to the extrados of the vaults, as contemplated in the Project, plus maintenance loads) are very wide.

This has brought to light how the constructive typology chosen by Ribera, with the evident risks it supposed for the construction (and even once finished and in service), was however a suitable solution for the purpose it was designed for. An example of which is the Roces III Reservoir in Gijón, which had a slightly less risky geometry and is still today in service albeit with a few changes which are commented upon later on.

2.5. Prior considerations before establishing the cause of the collapse

Of the possible causes which could give rise to a collapse as in the 4th chamber and which is analyzed in [4], and because of its due importance, it is necessary to highlight some aspects of the nominal situation of the roof, in other words, its design.

It is to be highlighted here that the configuration of the roof, and hence its structural conception, was pointed out in some of the initial technical reports as the cause of the failure and its extension to the whole of the roof.

In the investigation which was carried out it was nevertheless confirmed that the project was totally justified and well-defined, starting off with a number of considerations regarding the type of actions the roof could be submitted to. It would therefore be possible to consider if said hypothesis were admissible or not.

The important differences in calculation criteria regarding distinct structural elements in relation to the current ones, a number of concepts not totally developed at the time, and the resolution of construction details, many of which not used today, have demanded an in-depth analysis. The conclusion is that the design of distinct structural elements, in the hypothesis of uniformly distributed gravitational loads, was correct in all cases, presenting as well, in general, wide safety margins, even considering current day design criteria.

There is one single aspect which, in the stated nominal situation would not have had any initial importance, but could have influenced in the extension of the failure to the whole roof once started, and even favoured the initiation of said failure, as shall be commented on in the following points. It addresses the detail of the footings of the columns over the foundations (Fig. 14). Apparently inherited from the tradition of masonry and steel, the

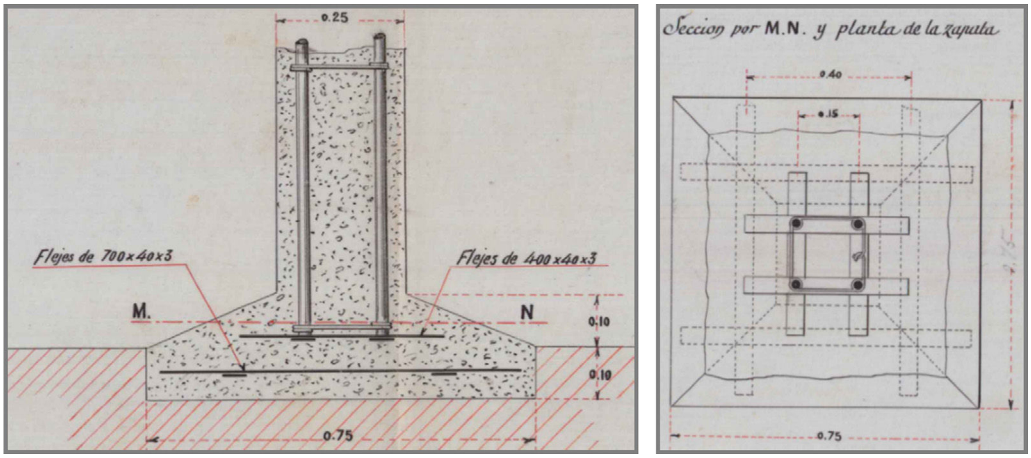


Figure 14. Detail of column footing on the foundation.

reinforcement of the columns starts above the foundations, without ensuring the anchorage to it, and therefore without offering the section of the column footing the capacity of bending (further than this being a mass concrete element, that is, what is due to the eccentricity of the axial loads in respect to the column's axis, which is very reduced due to its outstanding slenderness).

Logically, regarding the hypothesis considered in the Project of uniformly distributed loads, this configuration lacks importance as the bending in the column would be practically zero. It would have greater importance if said loads were not totally symmetrical as analyzed in the following section.

Otherwise, apart from minor details, the structure was perfectly defined and did not offer uncertainties regarding its structural behaviour, independent to the fact that some reinforcement configurations differ to those which years later became more popular as a result of a better understanding of the "new" material.

Therefore a design error has not been detected which would justify the start of the collapse. Indeed, failure started as a result of one of the causes explained in later sections, the structure progressively became a mechanism and the failure extended to the whole chamber, as in fact happened. In effect, the geometry (with a very great slenderness and consequently low stiffness regarding bending) and the configuration of the columns (even having been correctly embedded) caused them to have a low capacity to withstand the differences of horizontal forces which are transmitted between adjacent vaults, causing these to suffer inadmissible deformations which end up provoking the collapse, as is explained in the following section.

2.6. The cause of the collapse

As has just been explained, the investigation which was carried out concluded that the severity of the accident in the 4th chamber of the Third Reservoir was due to an extremely sensitive configuration of the structure when faced with any local failure which, once begun, would cause this failure to extend to the rest of the roof (Fig. 15).

It was also deduced that this failure was probably originated by the existence of non-uniformly distributed loads on the vaults. Effectively, differences of up to 10 cm between one alignment of vaults and another were found which caused the collapse. Confiding the stability of the roof to thicknesses of fill of this magnitude, during the construction phase on a roof as is analysed, was very risky.

Ribera was aware of this risk, and in the Project he reiterated the importance of the measures to adopt to avoid such accumulations of non-symmetric loads on the roof. However, the fact of having completed the construction of most of the roof and the success of the load tests could well have relaxed somewhat the instructions regarding the distribution of soils, so justifying the producing of errors and the subsequent failure of the vaults.

We could also highlight that the failure of the vaults was the result of the loss in shape of one of them. It is important to stress that this failure mode is associated with a problem of global behaviour as the individual resistance of each of its elements: vaults, beams and columns was sufficient.

In any case, both during the construction process, due to the aforementioned reasons, as with the finished roof in the service period in face of any accidental action, the local failure of a vault or support would cause the adjacent elements to immediately become unbalanced, not being possible the stabilization with the remaining vaults in the same alignment and hence causing the collapse to extend to the whole roof.

It has been discarded however that the heat wave in the days prior to the accident, which served as Ribera's defence, could have influenced in any way as the origin or the extension of the collapse.

We can therefore conclude that even if the collapse could not have been caused by a defect in design, as Ribera was fully aware of the risks he was undertaking in the construction phase; his structure was excessively flexible and risky.

3. Considerations on robustness of a series of structures

Going beyond the cause of the collapse, the case of the Third Reservoir is a clear example of a structure lacking in robustness as seen by the magnitude of the accident. In this way, Ribera must have reflected upon this defect in the structural configuration he had adopted, and a few years after the collapse, and possibly as a

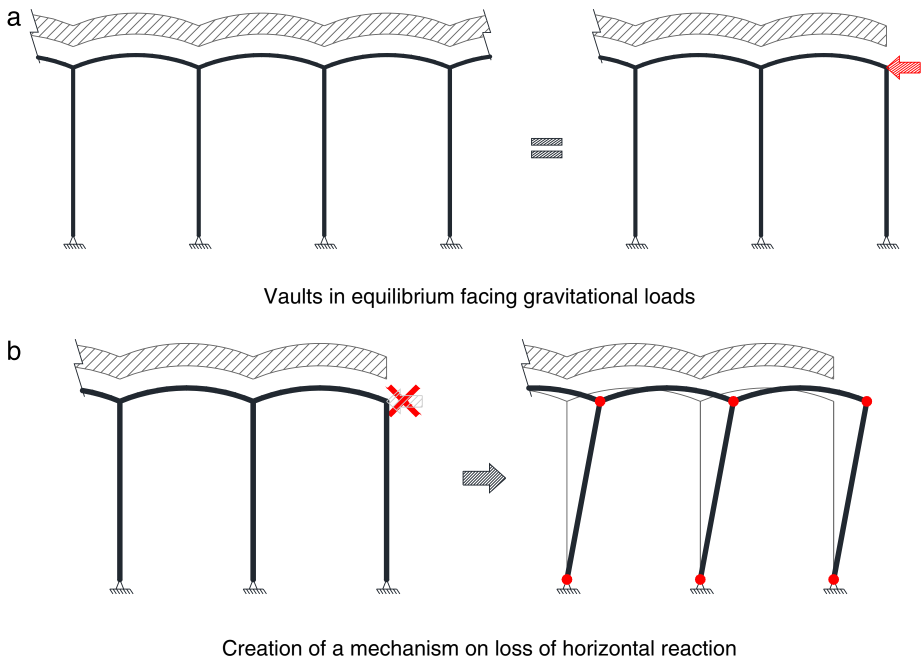


Figure 15. Resistance model of Third Reservoir.

Figure 16. The Roces III Reservoir (Gijón) today (Courtesy EMA, Gijón).

results of the "fears" that something similar could happen in the Gijón Reservoir (even though this was not as sensitive as the one in Madrid due to its geometry), provided the roof with a number of transversal beams which did not exist in the original Project (and thus cannot be seen in the photographs of its construction) (Fig. 16).

In conclusion, he converted a very sensitive roof in relation to any local failure into a very robust structure along with additional mechanisms to withstand the loads so that if an element failed (for example by local failure of a vault), the loads could still be born through other resistant mechanisms. Similar procedures may be found in other roofs with alike typologies, whether originally conceived as this or as measures of reinforcement. Two examples may be seen in Fig. 17.

Logically to achieve these additional measures is not always possible. In fact, in most cases it is not even posed. Returning to the Golden Gate bridge which we started the article with, nobody contemplates additional cables to suspend the deck. The robustness in this case is a result of an increase in inspections, additional safety measures etc., which minimize the probability of failure of these cables.

A further example which shows the importance of this sensitivity towards structural robustness is that of intervention, very frequent in changes of use or simple maintenance in arch bridges of the XIX century or the first half of the XX century. An example of this can be seen in Fig. 18 which shows the Pont de la Concorde, constructed by Perronet 1787–1791. The lightness of its construction contrasts with the stiffness of former bridges, especially Roman bridges which are the paradigm of everlasting structures. In fact, what is more than well known is that Perronet revolutionized the design of masonry bridges regarding the concept of robustness we are interested in, varied the width of the piers to nearly a tenth of the length of the spans when until that moment the same relation was running between 1/3 and 1/5 in Roman bridges and even greater in mediaeval ones [5].

The geometry of low arch bridges in the XIX century makes them more sensitive (in comparison to the former semi-circular vaults and pier-abutment bridges) to phenomena such as scour

Figure 17. Plasteres panelled vaults in the Batlló factory, Barcelona (R. Guastavino, 1875) and the Basilica Cisternin Istanbul (532 A.C.) (right).

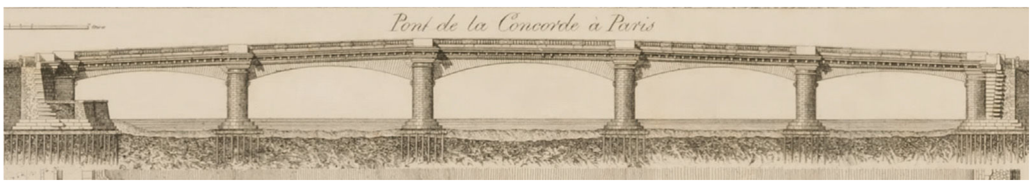


Figure 18. Pont de la Concorde (Perronet, 1787–1791).

around the piers so provoking slight rotations of the foundation and the consequential failure of a span and hence the failure of the remaining spans.

In these conditions it is important for the technician, who is carrying out the intervention, to be sensitive to the lack of robustness so that he may fulfil the necessary investigations to ensure, with greater safety margins than in a Roman bridge if possible, that a failure of a vault or a pier does not take place (very unusual circumstance), or that aforementioned events such as scour may be avoided. This last failure is much more frequent: Fig. 19 shows graphically the collapse of a bridge in León as a result of this [6].

As a final example, and getting back to the Third Reservoir, we can return to the footing detail of the columns on the foundations. The configuration shown in Fig. 14 is not exclusive to Ribera, being equivalent of that used, for example, by Hennebique or which may be found in some of the first publications on reinforced concrete of the time (Fig. 20).

However, unlike aforementioned masonry and steel constructions where the bracing was entrusted to the layout of the walls themselves or the triangulations of the steel elements, those of reinforced concrete are structures in which, except in specific cases where acquired knowledge has made them disappear (such as those uncomfortable hinges in swing bridges) the nodes should be stiff.

Said stiffness – or the capacity of the node to transmit bending moments – is precisely that which offers the structure "a monolithic nature" and "stability", by creating structures which are highly statically indeterminate and whose possibility of failure when faced with horizontal actions is practically zero. This is the way in which, indirectly, we make robust structures today.

Also in the intervention of these first reinforced concrete structures we should be aware of certain configurations in detail to be able to decide the type of measures necessary to incorporate in each case.

4. Final thoughts

The severity of the accident of the Third Reservoir was the result of a very risky structural configuration, conceived without the necessary resources which could have mitigated the effects of a local failure.

The assumption of risks in engineering has allowed, without any doubt, its progress. But it is also a fact that the repeated occurrence of accidents as a result of occasionally neglecting design criterion which would have led to a specific structural configuration.

In the case of the Third Reservoir, Ribera adapted the Monier system configuration of vaults upon fixed abutments to a roof which was supported on very slender columns whose stability was confined to an excellent distribution of the loads over the same. He was totally aware of the sensitiveness of this configuration (until this moment it had always been braced), but the success of the construction of the Gijón Reservoir without such bracing, and the state of design in those years at the turn of the century, with concrete being consolidated as the new fashionable material, encouraged him to push the known limits and even take on unnecessary risks such as the carrying out of load tests with the subsequent movements of the soil on that overly sensitive roof which finally provoked its collapse.

This oversight in determinant structural factors as a result of an excessively relaxed state of design is recurrent in the history of engineering, up to the point where Petroski, one of the great educators on engineering failures, baptized it as the "syndrome of success" [7]. Examples of the aforementioned are the collapse of the Dee Bridge by Robert Stevenson in 1846, the collapse of the Quebec Bridge over the River San Lorenzo in 1907, or the infamous Tacoma Bridge in 1940.

As also happened in some of these cases, the collapse of the Third Reservoir has become forgotten and its cause never was fully analyzed [8]. Moreover, it is surprising that it has not

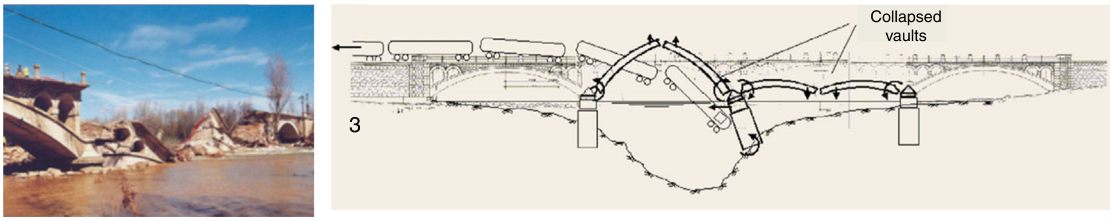
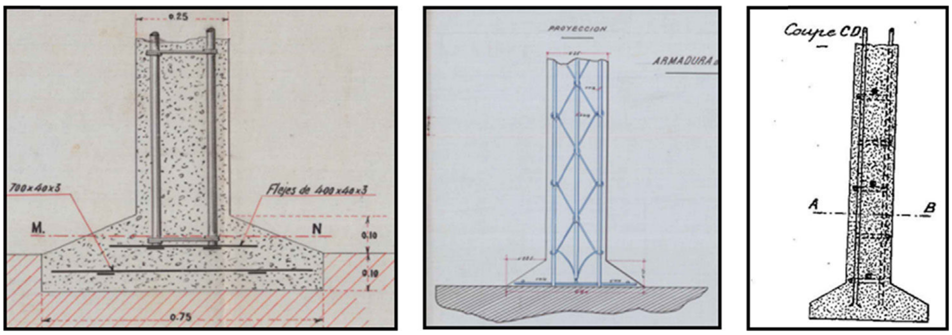


Figure 19. Graph showing the formation of a collapse mechanism due to pier rotation in the Veguellina de Órbigo bridge, León (constructedin the early 1930s).



Sistems *Ribera* Sistems *Compañía de Sestao* Sistems *Hennebique*

Figure 20. Different types of detail of column footing on foundation.

been detected until now, despite the great technical level of civil engineers of the time, who were highly trained in mathematics, geometry and mechanics. It is also surprising that the designer and the technical inspectors were unable to identify the failure mode associated with the kinematic collapse mechanism associated to low vaults supported on tall, slender columns, along with unfortunate construction details. Add to that, that prior experience and the apparently successful load tests led to a misleading sensation of confidence and security which as seen was unfounded.

May this be a worthwhile example for the engineers of today.

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[1] This article was written from results obtained in the doctoral thesis titled INVESTIGATION ON THE CAUSES WHICH COULD CAUSE THE COLLAPSE OF THE ROOF OF THE THIRD RESERVOIR OF CANAL DE ISABEL II IN 1905 [4], presented in November 2015 to the Escuela Superior de Caminos, Canales y Puertos de la Universidad Politécnica, Madrid. All relevant bibliography used is compiled in mentioned article.Further references are listed in the reference list.