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Investigation of the Roman Aqueduct Structure of Acqui Terme

Recherche concernant la structure de l'aqueduc romain de Acqui Terme

Tragwerksuntersuchung am römischen Äquädukt in Acqui Terme

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SUMMARY

The research concerns the remnants of a Roman aqueduct of stone block masonry at Acqui Terme in Piemont, Italy. Its aim was to acquire exhaustive data on the mechanical behaviour of the aqueduct, to serve as a basis for assessments concerning the safety and preservation of this monument.

RÉSUMÉ

La recherche concerne les vestiges d'un aqueduc romain constitué de blocs de pierre et situé à proximité de la ville piémontaise de Acqui Terme, en Italie. Cette recherche visait à acquérir des informations exhaustives sur les aspects les plus significatifs du comportement mécanique de l'aqueduc, afin de poser les fondements pour les évaluations en matière de sécurité et de conservation de ce monument.

ZUSAMMENFASSUNG

Die Arbeit bezieht sich auf die Reste eines antiken römischen Äquäduktes aus Bruchsteinmauerwerk, das sich in Acqui Terme, im italienischen Piemont, befindet. Ziel ist das Sammeln von Daten hinsichtlich der wichtigsten Aspekte des mechanischen Verhaltens des Äquäduktes als Grundlage für die Beurteilung der Sicherheit und Erhaltung des Monuments.

1. INTRODUCTION

The Roman aqueduct of Acqui Terme was built in the Augustean period to supply water from the Erro stream to the town. At one end, this structure, extending over 14 km, ran about 20 m above ground to bridge the depression created by the Bormida River. This portion of the aqueduct was supported by piers evenly spaced 9 m apart and stone masonry arches; only two portions remain of this part of the aqueduct: one, closer to the town, consists of seven piers and four arches; the other, next to a hill, includes eight, badly deteriorated shorter piers.

The investigation illustrated in this paper was conducted on behalf of the Archaeological Superintendence of the Piedmont region with the aim of acquiring data on the mechanical properties of the original material as a basis for an evaluation as to the monument's safety conditions and state of preservation.

2. COMPOSITION OF THE PIERS AND ARCHES

At the end of the 19th century (1896), when erosion had already damaged severely both the facing and the inner masonry (photo 1), restoration work was started and carried out in several stages, mostly on the arches.

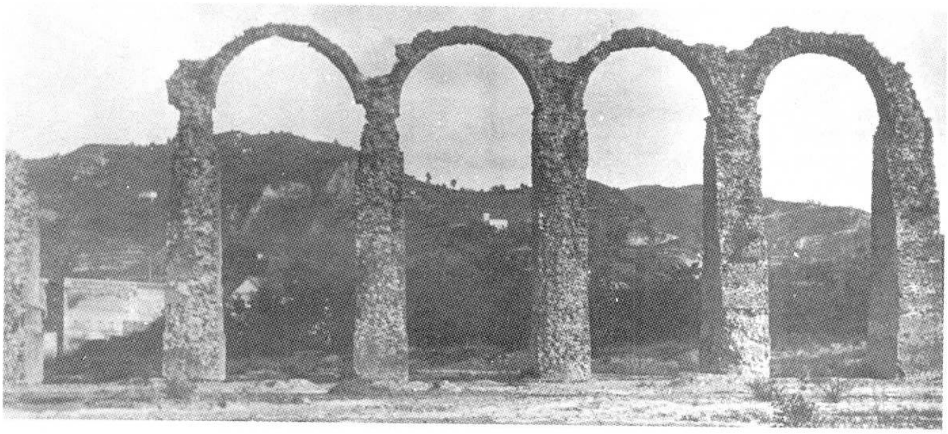


Photo 1 - General view of the aqueduct in 1896.

The composition of the original masonry may therefore be discerned best in isolated piers which underwent but limited repair work (photo 2). The facing layer consists of sandstone blocks, of variable width (25 cm on average) 8 cm thick, squared on the outer surface. Mortar joints, whether horizontal or vertical, have a thickness of between 2 and 3 cm, which is generally seen to increase towards the innermost part of the masonry. The average value of the ratio between the area of the mortar and the total area of the facing layer is about 0.3 (photos 3 and 4). The inner part of the masonry instead is made of sandstone blocks exhibiting irregular edges between the two parallel faces. Narrower and thinner than the facing blocks, all these elements were carefully laid horizontally one at a time. The mortar beds of the inner blocks reveal marked discontinuities and their thickness, on average, is greater than that of the facing blocks: the average value of the ratio between the mortar area and the total area of an inner vertical section, in fact, is about 0.5 (photos 5, 6). In the inner part of the masonry there are many cavities, mostly located in the median portion of the thinner vertical joints. These cavities were produced both by insufficient flow of the mortar from the bottom up as the blocks were laid and by insufficient filling of the upper mortar bed.

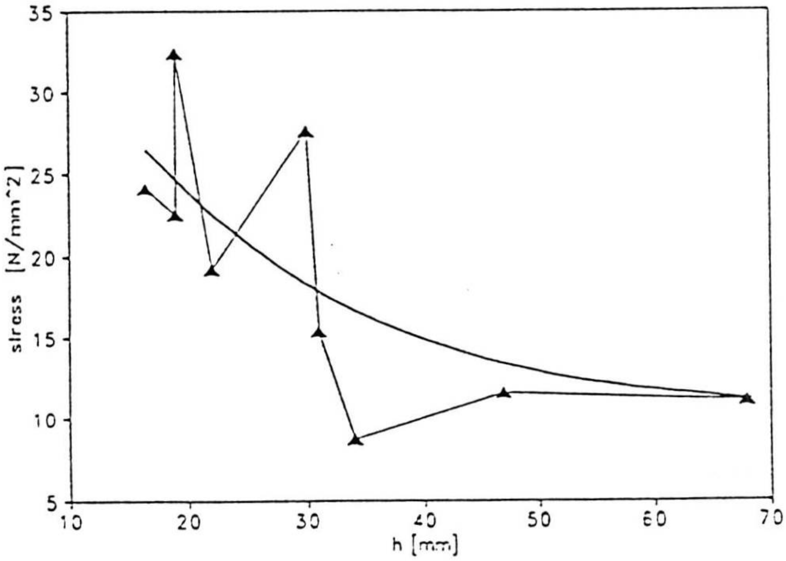


Fig. 1 - Strength as a function of the specimen height.

Photo 2 - Non restored isolated pier.

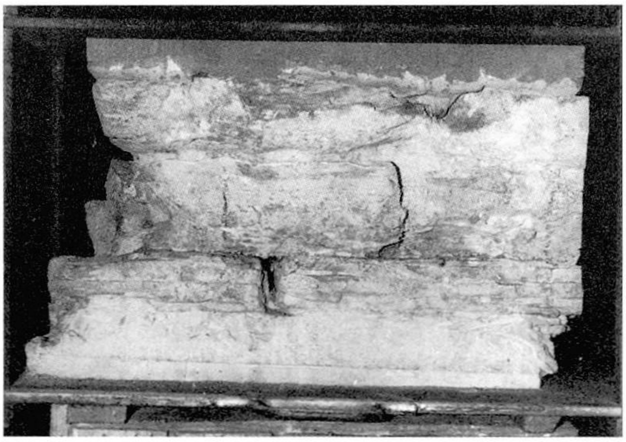


Photo 3 - Sample obtained from facing masonry.

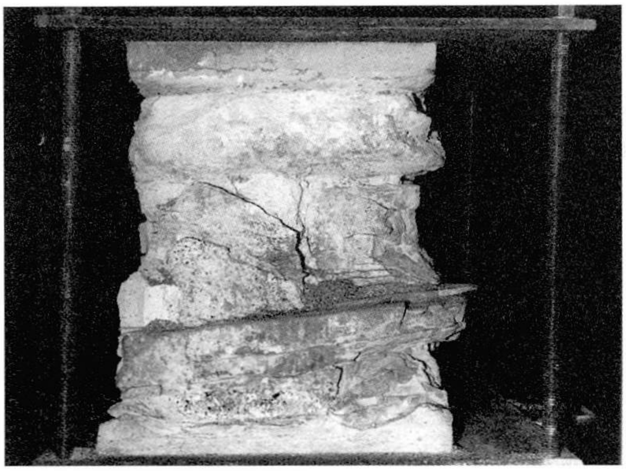


Photo 4 - Sample obtained from facing masonry.

Photo 5 - Sample obtained from inner masonry.

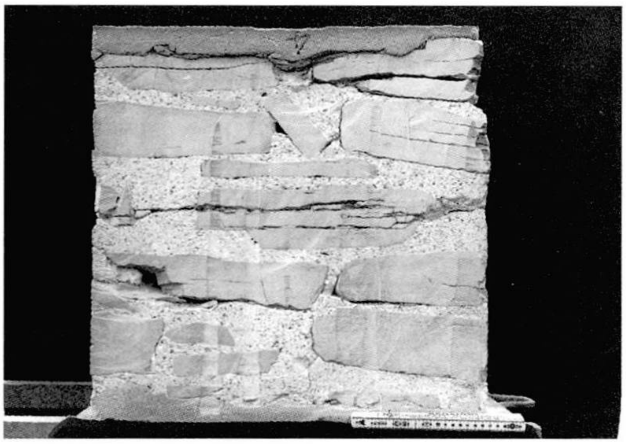


Photo 6 - Sample obtained from inner masonry.

This macro-porosity, already observed by Choisy in Roman masonry walls of this kind [1], was surely associated with the need to make limited use of water in the mortar mix and to improve its strength, which was very high in this particular case; another consideration was not to undermine the stability of the piers at the building stage. In all likelihood, the primary cause of erosion resulting in the monument's deterioration was this macro-porosity associated with freezing and thawing cycles; this was also one of the most significant factors, together with the thickness and discontinuity of the mortar joints, affecting the mechanical behaviour of the masonry as observed throughout the testing programme.

3. TAKING THE SAMPLES

With the agreement of the Piedmont archaeological authorities, three blocks were obtained from the ruins of the first pier in the set of eight without arches adjacent to the hill. Sample selection was performed so as to make sure the specimens would be representative of the overall conditions of the materials. The samples taken from the cladding included two specimens: Nos. 1 and 2, 38 and 50 cm high, respectively, including the concrete headers, with a height-to-width ratio of between 1.5 and 2 (see photos 4, 5). As for the inner part of the masonry, it was deemed necessary to obtain a large test piece which would clearly represent the irregularity of its fabric and be able to average out the singularities in mechanical behaviour entailed by the latter. Specimen No. 3 turned out to be almost cube shaped, measuring 55 x 57 x 56 cm (photos 6, 7). However, the typical behaviour of this masonry attenuates, at least partially, the influence of the pressure plates' transverse containment on the loading surfaces: the height/width ratio, of considerable significance when dealing with homogeneous materials or well laid brickwork, loses its importance when a limited offset of the vertical joints compared to the height of the stone courses promotes the early formation of cracks along such joints and causes the formation of resistant sections on which the influence of the plates is minimal.

In parallel with the taking of samples, cores were also drilled for chemical-physical analyses and mechanical tests on the stones and the mortar.

4. MECHANICAL PROPERTIES OF THE STONES AND THE MORTAR

The samples taken were analysed and tested by A. Frisa Morandini of the Geo-Resources and Territory Department of the Turin Politecnico.
Petrographic characteristics were deduced from microscopic visual examination of thin sections and carbonate measurements (through powder calcimetry).

The stone was classified as "calcareous cement sandstone" with an average carbonate content of 38%. Extensive outcrops of this sandstone, of the Tertiary Basin of Liguria and Piedmont, can be found in the hills surrounding Turin and in the Langhe and Monferrato areas where numerous small quarries were opened in the past for the production of cutting stones for the building industry. The mechanical properties of the stone (whose mean values are listed in Table 1) were obtained by testing to failure Ø 31 mm cylinders with an average height of 75 mm.

	Rock	Mortar
Weight by volume [N/m³]	26,750	18,100
Mono-axial compressive strength [N/mm²]	63	19
Tangent elastic modulus in compression [N/mm²]	11,000	9,900
Secant elastic modulus [N/mm²]	9,500	11,600

Table 1

The values of the tangent and secant elastic moduli were determined for a load corresponding to half the failure load under mono-axial compression.

A comparison between the physical and the mechanical characteristics of the different specimens revealed that the compressive strength of the less compact test pieces having higher porosity and higher soaking coefficient values was about 20% lower than that of the others. Mortar employs a binder consisting of air-hardening lime, with partially hydraulic properties, presumably produced by good baking. The aggregate was a rather course siliceous sand, with up to 10 mm diameter.

The average binder/aggregate ratio is about 1/3. The specimens were seen to contain no gypsum nor any other chemical alteration product and the presence of lichen colonies on the face joints reveals that to this day no chemical physical alteration has taken place in the mortar.

In the mortar, compressive strength and the elastic modulus were determined on cylinders of different heights: strength was seen to decrease with increasing specimen height (fig. 1), this being an aspect of special interest in the case being considered, on account of the sizeable discontinuities present in the mortar beds of the inner masonry, in terms of both thickness and area.

The high values of strength and of the elastic modulus in this Pozzolan free mortar may presumably be ascribed to a favourable aggregate grain size, combined with the binder's partially hydraulic properties, and the thorough working of the mortar in the presence of a low water/binder ratio.

The comparison between the mechanical properties of the mortar and those of the stone revealed a ratio of about 1/3 for compressive failure strength while the elastic modulus of the two materials, under a load corresponding to about 50% the failure load, was almost the same.

5. TESTS ON MASONRY ELEMENTS

Specimen behaviour was studied by applying the load in cycles of increasing intensity, starting from relatively low values. The tests were performed by means of a ball-joint press at the laboratory of the Structural Engineering Department of the Turin Politecnico. More precisely, the test pieces were subjected to three loading cycles of the same intensity and after that the load was increased in steps of between 1 and 2 N/mm². The readings were taken as a function of the load increments imposed. Unloading never fully relieved the pressure on the elements. The instrument used consisted of horizontal and vertical strain gauges: the vertical readings made it possible to detect the deformations taking place in the masonry specimen as a whole (thanks to gauge bases from 20 to over 30 cm long). Specimen faces in contact with the pressure plates had been levelled beforehand with gypsum. Load-displacement and stress-strain diagrams were plotted on the basis of the test results.

During the tests, local failure was seen to occur systematically in small portions of the masonry and various stones located at different places were also seen to fail, through additional shear and bending effects: all these phenomena gave rise to partial load distributions and the reduction of isolate stress peaks.

In the case of sample No. 1, though care had been taken to apply the load to its centre of gravity, a marked compression and bending effect occurred due to the presence of a stiffer and stronger core in one of the corner area. Failure took place at 1180 KN (8.6 N/mm²), during the second loading cycle of the last triplet, at a strain value markedly higher than that observed during the previous cycle where the specimen, however, had reached a load of 1300 KN (9.4 N/mm²).

In specimen No. 2 the compression and bending effects were much smaller, on account of a more regular composition (fig. 2), and failure occurred under a load of 1420 KN (13.1 N/mm²).

The behaviour of specimen No. 3 was relatively uniform, with no sizeable compression and bending effects: a series of diffused micro-cracks appeared rather early over all the faces. At 1000 KN (3.3 N/mm², 40% of the failure load), widespread crack formation was already evident and it kept increasing over the successive cycles, the load level remaining the same (photo 7). This phenomenon continued steadily until failure which occurred at 2400 KN (8.0 N/mm²).

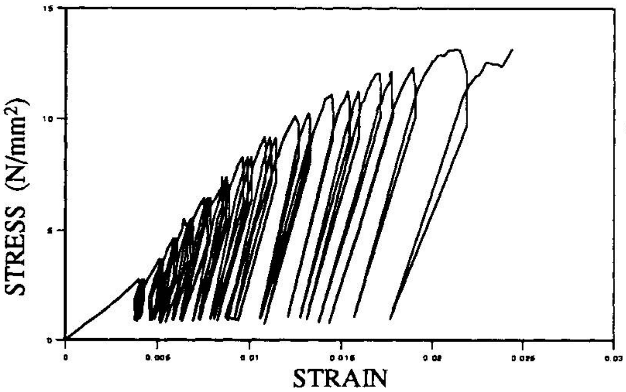


Fig. 2 - Stress-strain curve of strain gauge 7 - specimen 2.

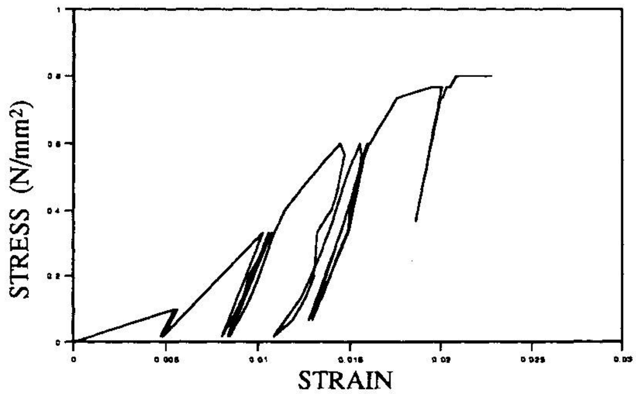
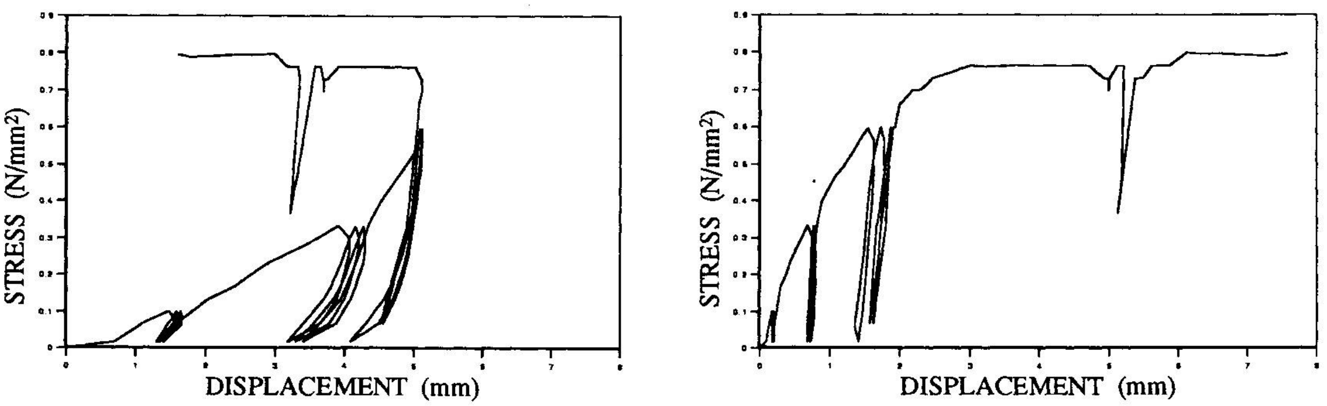


Fig. 3 - Average stress-strain curve - specimen 3.



Figs. 4 - 5 - Stress-strain curve of strain gauge 4 (vertical) and 9 (horizontal) - specimen 3.

The progressive opening of cracks did not prevent the material from exhibiting overall elastic behaviour in the individual loading cycles up to very high stress values, as confirmed by the diagram in fig. 3. This diagram illustrates a typical behaviour which was observed throughout the tests: the initial load increase, for each testing level, was accompanied by high strain values and marked residual strain, while in subsequent cycles (the load being the same) strain was lower and residual strain was virtually negligible.

A comparison between the diagrams plotted from the readings taken by two strain gauges of about the same length on the same face of specimen No. 3 (figs. 4, 5), - a vertical and a horizontal gauge, Nos. 4 and 9, respectively - shows that the strain values are of the same order of magnitude and the horizontal one entails a transverse strain modulus about three times as high as that of the masonry at the elastic stage. Moreover, the downward portion of the curve in fig. 5 reveals a major overall settlement of the specimen under the load, this behaviour closely resembling that of specimen No. 1 already described above (fig. 6). Thus we find that the transverse strains that have been observed are greater than would be expected on the basis of the Poisson modulus of the components, and are independent - even in terms of their direction - of vertical ones in the proximity of failure. Failure involved the masonry specimen as a whole, as shown in photo 13. Strain stabilisation was especially slow at a load level corresponding to 90% of the failure load and required a waiting period of about 20 minutes, this being a confirmation of the major influece exercised by the permanence of the load on the collapse of very irregular masonry.

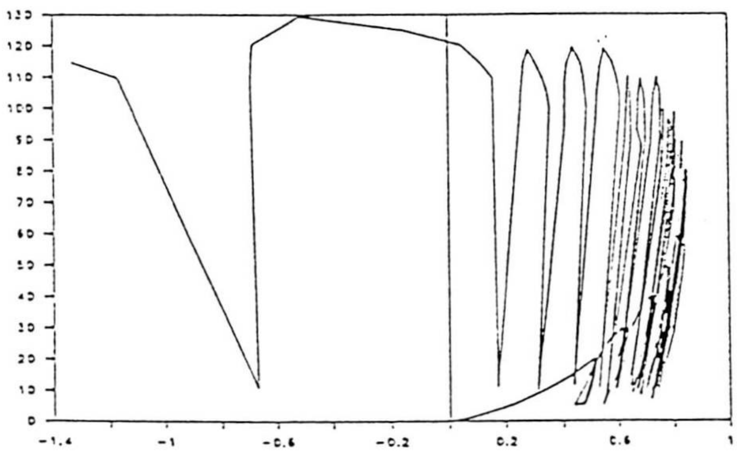


Fig. 6 - Load-displacement curve of strain gauge 5 - specimen 1.

Photo 7 - Collapse of specimen No. 3.

6. CONCLUSIONS

The stones and mortar employed in the original construction of the Acqui aqueduct have very similar deformability, as borne out by the values of the tangent and secant elastic moduli.

The deformability of the masonry is characterised by the fact that the material takes on a markedly elastic behaviour under repeated loading cycles with a pre-determined maximum stress limit, whilst it exhibits an anelastic behaviour each time the load is increased. This phenomenon, which is typical of masonry in general [2], was seen to be very pronounced in this case. We may therefore distinguish two different deformability moduli: one can be inferred from the envelope curve of the maximum values in the stress-strain diagrams, this curve being very close to the one that would be obtained under a steadily increasing load. The central portion of the curve is nearly linear and very long, with a value of between 600 and 800 N/mm^2, which is lower than 1/10 of the values for stone and masonry.

The other modulus characterises the elastic behaviour of the masonry during the loading cycles: it is appreciably constant almost up to failure, and its value is about 3000 N/mm^2 (1/3 that of the components). The ratio between the values of the modulus under a constantly increasing load and the values measured during constant width loading cycles is 1/4. The great difference observed in the deformability of the masonry and that of its components should be ascribed to the state of cracking which occurs in the masonry specimens at the early stage of loading; this isolates blocks and fragments within the specimen which undergo relative rotation and slip during the test; this conclusion is borne out by an examination of the cracks, almost always of variably width due to individual fragment rotation, and by the strain values read on the horizontal gauges, all of them much higher than would be observed in an elastic material.

In addition to being associated with size effects in the specimens, the noticeable decrease in the strength of the masonry compared to the strength of mortar and stone is caused primarily by the fact that the stone material is not homogeneous with limited overlapping, the influence of these factors being greater in the inner part of the masonry: the average stress value obtained by dividing the applied load by the specimen's area, in fact, does not account for the great differences in stress existing in the specimen at different points of the cross-section as a function of variations in local rigidity; it is precisely these high local stress values that result in the formation of circumscribed cracks and the material becoming plastic, with the consequent stress redistribution (a phenomenon that can be clearly observed in figs. 5 and 7), in addition to resulting in a reduction in the material's overall strength.

This also accounts for the lower strength of the inner masonry compared to the facing layers, though the latter were seen to be greatly influenced by their small size and by the composition of the facing elements.

7. REFERENCES

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