Rehabilitation of Sidi M'Cid Suspension Bridge, Algeria

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Summary

The structural rehabilitation of the Sidi M'Cid suspension bridge in Constantine, Algeria, is presented. The bridge, designed by Arnodin in 1908, has a main span of 160 m and a mixed cable-supporting system with a stayed deck portion near the towers. Over time, various components of the bridge had corroded, calling for a structural rehabilitation that included partial replacement of the main suspension system as well as major interventions on both anchorages. Because the bridge spans a canyon 200 m deep, specifically designed equipment was required in order to partially replace the suspension system while maintaining the bridge under full service.

Introduction

Various small- to medium-span suspension bridges built in the late 19th and early 20th centuries are now in need of strengthening and repair [1, 2]. In many cases it is necessary to replace the main suspension cables, which due to their reduced diameter and lack of proper protection are more prone to corrosion than cables of long-span bridges.

Replacement of the suspension system is however not a simple task. Large displacements and deformations and high elastic energy accumulate in these structures during construction, making it difficult to perform partial release and replacement of the various individual components, especially the suspension cables.

The Sidi M'Cid Bridge (*Fig. 1*) is a small suspension bridge designed by the French engineer Ferdinand Arnodin at the beginning of the 20th century [3]. The bridge spans the Ruhmel River in Constantine, Algeria, where a 200-m-deep canyon has been formed in the carbonatite rocks (*Fig. 2*).

During monitoring of the bridge in 1997, the bridge showed extensive signs of corrosion in the suspension cables (*Fig. 3*), in the deck stiffening girder, and in the transverse beams. Corrosion was particularly acute at the bottom of the anchoring tunnels, which are often submerged with water that filters through the carbonatite rocks. Replacement of various components of the suspension system was therefore required, together with repair works on the steel deck girder and the waterproofing of the anchoring tunnels.

Suspension System

The Sidi M'Cid bridge is supported by two cables, each made of six spiral strand ropes 76 mm in diameter. Along the main span the ropes lie in a horizontal plane and pass over the saddle before rotating 90° into a vertical plane to loop around the U-shaped anchoring tunnels. Each suspension rope is clamped individually via steel bars to small T-profiles (*Fig. 4*). The T-profiles are connected to hangers made of 33-mm-diameter rods. The clamps and hangers are closely spaced (at 1.25-m centres) and hold the truss-type cross beams that support the deck. Of the six ropes on each side, the central four were replaced during the early 1980s, but no record is available of the works. The outer two ropes on each side are the original ones and showed extensive signs of corrosion.

The deck sections up to 30 m from the towers are supported on each side by six stays made of 35-mm-diameter ropes (*Fig. 2*). These stays are connected to the saddles and balanced on each side by two back stays made of 42-mm-diameter ropes. The stays are anchored to stiffening I-beams that run beneath the stayed portion of the deck. The horizontal force component of the stays is balanced by two traction ropes, running below the suspended part of the deck and connecting the I-beams on both sides of the bridge (*Fig. 2*). This configuration allows the deck to swing longitudinally since the horizontal stay component is self-equilibrated and the deck does not push against the abutments.

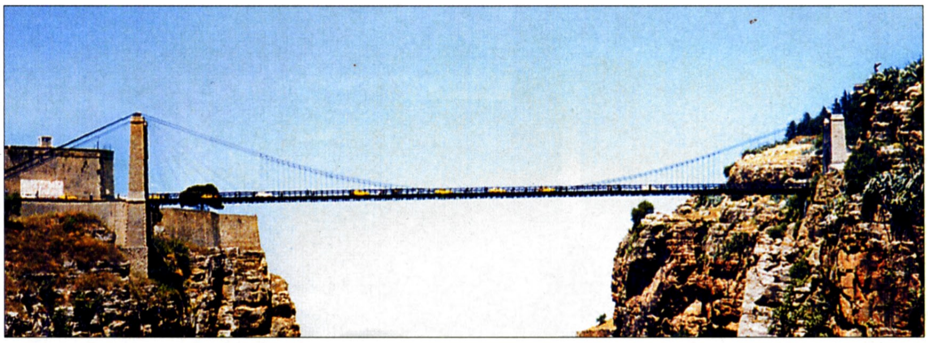

Fig. 1: Sidi M'Cid Bridge

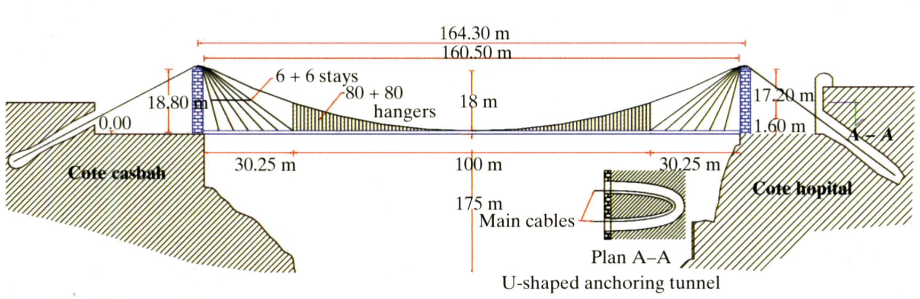

Fig. 2: Bridge profile

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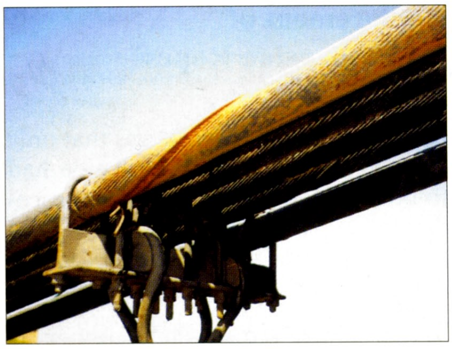

Fig. 3: Detail of hanger and cable corrosion

The saddles can slide on rollers. This mechanism, although showing some signs of deterioration, was still effective under live loads and temperature changes that are fundamental for the integrity of the stiff masonry towers. Saddle displacements of a few millimetres were measured during the monitoring period, but the friction was difficult to calculate because practical live loads cause very small saddle shifts, while temperature changes, which account for larger deformations, are very difficult to quantify. (The temperature change in the suspension and backstay ropes should have been monitored at different locations, taking into account the temperature difference between the anchoring tunnels and the outside air and other environmental factors. Sunlight exposure, for example, is exacerbated by the reduced thermal inertia of small-diameter ropes and stays.) Based on the perfect condition of the masonry towers, the friction must be well within acceptable limits.

Structural Behaviour

A number of experimental tests have been carried out to investigate the behaviour of the bridge and assess its performance, namely

- chemical and mechanical tests *in situ* and on samples
- static and dynamic monitoring of the deck and saddle displacements under test loads and temperature changes
- tension measurements in cables, hangers and stays.

Chemical analyses of the different steel elements of the bridge were performed. High-strength steel is used for suspension ropes, stay ropes, anchorage bars and nuts, cross-beam bottom-chord ropes, and traction ropes. Mild steel is used for hanger rods, parapet stiffening girders, cross-beam bracings, clamps, saddles, and all minor steelworks. Different production procedures and origins are responsible for a significant scatter in the chemical composition of the elements.

Ultrasonic and liquid penetrant methods were used to detect cracks and discontinuities in the steel anchorage components.

Suspension ropes, the structural steel of the deck, and tower masonry were mechanically tested using samples from the bridge. The tests results were in accordance with the values predicted from the chemical analyses. The ultimate strengths of the high-strength and mild steel types were around 970 MPa and 360 MPa, respectively. The masonry towers were made of poor-quality mortar and extremely tough stone blocks with a compression strength of over 140 MPa.

The bridge displacements due to temperature changes were recorded. Between 15° C and 20° C, the sag at midspan increased by 7 mm/° C. Static and dynamic tests were then performed using a 3.6-t truck. Static deflections and vibrations of the deck were recorded. Static values had to be adjusted for temperature changes that occurred during the tests.

Tensile forces in cables, stays and hangers were measured through their frequencies, where possible. Although the results were not always reliable because of the flexural inertia of some elements compared with their free oscillating length, they indicated the order of magnitude of the load distribution between different members, where the total force was known from the static behaviour of the bridge.

The overall picture that emerged from these tests confirmed what was evident at first glance, namely the suspension system was largely understressed, with all the main components working below 150 MPa under self-weight and below 250 MPa under full live load. The structural assessment was greatly enhanced by the numerical simulations of the structure, which allowed verification of the experimental findings.

A more difficult task was the assessment of the stress state of the stiffening girder because measurement of the hanger tension was unreliable. Being short and stiff, these hangers are not suited to frequency evaluation.

It appears that the deck has been overstressed at certain points during its service life since a few vertical struts in the truss girder near the tower have collapsed, apparently sheared off by fatigue.

That the parapet girders were too stiff was recognised soon after construction during loading tests [3]. Today, the stiffness of the parapet girders is probably smaller than it used to be due to increasing looseness in the parapet diagonals (*Fig. 4*), which are made of rods pretensioned with bolts at both ends. Further reduction of the parapet stiffness may have been caused by local failures (e.g. of the vertical struts) and other sources of looseness at the truss joints. Analyses carried out using a nonlinear finite-element model [5] showed that in order to match the experimental results the deck bending stiffness had to be set to less then half the value calculated from the member area, geometry and modulus, assuming perfect rigidity of all connections and joints.

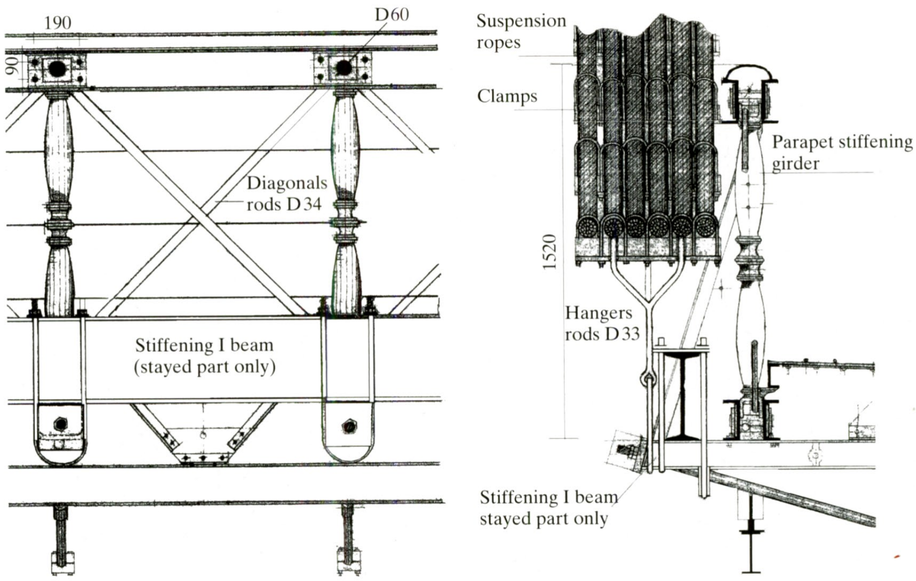

Fig. 4: Parapet truss girder

The deck boundary conditions led to a peculiar behaviour of the bridge. The response of the bridge differs substantially from that of a suspended girder and lacks reciprocity and symmetry, as can be seen in *Fig. 5* where the measured deflections under the truck test load are plotted and compared with the results from the finite-element simulations.

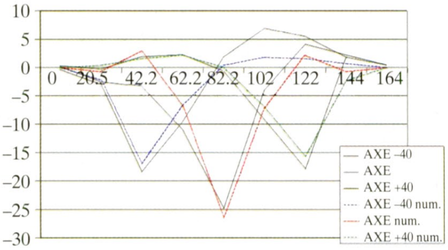

Fig. 5: Experimental (solid lines) and numerical (dotted lines) deflections of the deck under test loads

This behaviour can be explained by the fact that the deck, simply supported at the towers, is partially constrained against longitudinal movements. Under asymmetrical loading (i.e. at quarter-span) the deck tends to sway towards the loaded side until it reaches the towers. The boundaries are gaps with different openings on either side of the bridge. On the right-hand side of the bridge (Cote Hopital) the deck rests against the tower and when the bridge is loaded on that side it behaves very much as a stayed girder. When the bridge is loaded on the other side (Cote Casbah) the deck can sway with the bridge, behaving more like a suspended girder. These two types of behaviour can be seen from the hogging that take places opposite the loaded side. Suspended girders tend to hog, following the cables funicular, while stayed girders do so to a smaller extent if at all.

The results of the finite-element analyses carried out with the deck constrained longitudinally (*Fig. 5*) were in good agreement with the loading tests when the deck rests against the tower (Cote Hopital). However, the simulations underestimate the hogging when loaded on the Casbah side because the deck can shift before resting against the tower.

The effectiveness of the stays is enhanced by longitudinally constraining the deck, which also reduces the bending moment on the parapet stiffening girder. This girder is too deep to accommodate the deflection of the suspension system under asymmetrical loading, especially at the towers where vertical displacements are constrained.

Although it is debatable whether this mechanism was understood at the time of construction, it has been left in place. It introduces some axial load in the deck that is now mostly carried by the concrete deck slab, which was added to the bridge at a later stage to replace the original wooden planks.

Fig. 6: Pulley and catwalk system along the main cables

Replacement of the Suspension Ropes

A peculiarity of the work was that only the two outer ropes on each side had to be replaced. In this case, the procedure of spinning the new cables and hanging the deck onto them before releasing the old ones could not be followed [6]. The new ropes had to be spun one at a time, then tensioned to the working load before being connected to the other ropes. The work had to be carried out with the bridge under full operation since closure of the bridge would have caused serious congestion to the city's traffic.

The practical problem with releasing only one rope on each side at a time is that localised forces arise when displacement of the rope to be replaced is constrained by the other five ropes. The saddles and hanger clamps were seen as a possible source of force localisation during the de-tensioning operations. Especially for the U-shaped clamps connecting each rope to the T-profile, the possibility of releasing them by sawing them off was considered unfeasible because the rope would have applied unacceptable horizontal and vertical forces on the adjacent clamps. Given the number of clamps (over 60 on each side), the possibility of tying down the rope to be replaced along the bridge and sawing the clamps one by one was also not feasible.

A system was therefore devised where the ropes could be de-tensioned gradually, allowing them to move horizontally while recovering their stretch. This was achieved by placing a number of pulleys along the cables above the rope to be released (*Fig. 6*). The pulleys were integrated into the catwalk system and kept in place during the whole replacement operation (*Fig. 7*).

With the pulleys in place, all the clamps were released, irrespective of the phasing and order of the operation, with the rope transferring the up-lift force to the pulleys (roughly 40 kN per pulley). Once the ropes to be replaced were resting under the pulleys (*Fig. 6*), de-tensioning was carried out by releasing the ropes. This was performed by tying down the rope close to the pulley to be released and then pivoting the pulleys (*Fig. 8*).

The spacing of the pulleys was fixed so as to keep the up-lift force within the capacity of hand-operated winches

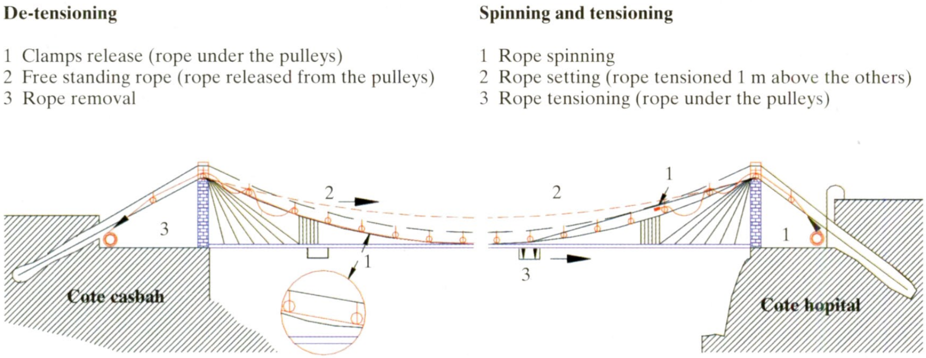

Fig. 7: De-tensioning and spinning systems

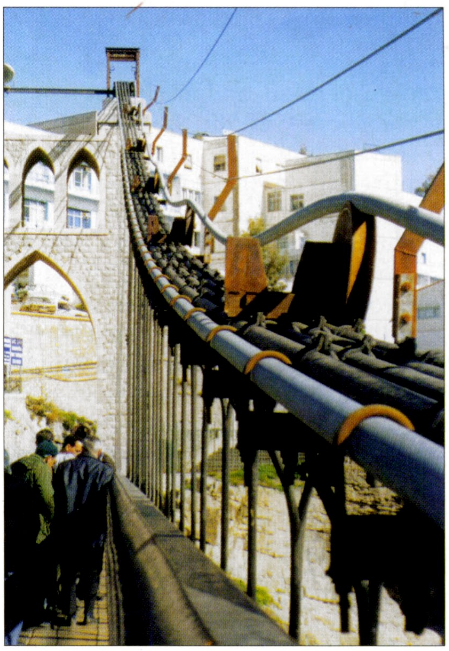

Fig. 10: Rope spinning

(maximum 50 kN). Release of the pulleys was carried out symmetrically from the towers towards mid-span. The up-lift force in the pulleys increased slightly during the releasing operation (as the deviation angle increases) before dropping towards the end of the procedure with a corresponding decrease in rope tension (from 500 to 50 kN).

A matter of concern was the behaviour of the rope over the saddles. Releasing of the rope along the suspension girder would have led to a differential pull between the two sides. It was not clear how the 100-mm rope shift required to balance the two forces would take place (either a jerk or a smooth slide). Initially it was proposed to place the rope over the pulleys on top of the towers to allow relative movement during de-tensioning. Due to a breakdown of the equipment required for the lifting operations, de-tensioning was carried out with the ropes over the saddles.

Upon release of the first rope, friction at the saddle was significant. However, once the differential force overcame the friction, the rope slipped smoothly, damped by the viscosity of the bituminous cable coat and by rust that had deposited in the saddle grooves over the years (*Fig. 9*).

Once released from all the hangers, the ropes rose approximately 1.1 m above the others and the deck sagged 12 cm, showing good agreement with calculated values and confirming the accuracy of the estimated self-weight of the deck. Complete removal of the ropes was achieved by cutting and releasing the rope from the anchorage. This operation was straightforward because the rope, under self-weight only, had a residual tension of only 50 kN.

The same pulleys were then used to spin the new ropes in place (*Fig. 10*). The new ropes were similar to the old ones except they were zinc coated, had a different socket arrangement and a higher breaking load (170 MPa). Tensioning of the new ropes took place following the same (inverted) procedure used for de-tensioning. The new ropes were tensioned from the anchorages to 1.1 m above the suspension cables. The operation was particularly simple and accurate because the rope could roll over the pulleys at the top of the tower. Tensioning of the ropes was then achieved by wincing them down under the pulleys one at a time. With the ropes tensioned in their final position, the clamps were screwed into place.

A variation with respect to the design procedures had to be made at the saddles, where the ropes had to be dropped into place inside the saddle grooves under self-weight only instead of under full deck load (i.e. before being winched down). This procedure required the ropes to be tensioned gradually from the anchors while being put into place under the pulleys in order to achieve a uniform tension. Uniform tension would have been obtained automatically, with the rope free to translate, had the rope been kept over the pulleys at the top of the tower until loading had been completed.

Fig. 8: Releasing the rope from under the pulley

Fig. 9: Placing the rope over the pulleys at the saddles

Anchorages

The suspension and back-stay ropes are anchored at the bottom of the U-shaped tunnel to a large-diameter rod fixed to the rock. This rod, placed vertically on the axis of symmetry of the tunnel, connects the ropes coming from the two sides of the bridge. Thus, the rod provides an additional restraint against differential pull in the ropes, which would otherwise be provided by friction along the tunnel. The original ropes were made of a single length with two end sockets connected via U-shaped steel bars to the rods at the two anchorages.

Replacement of the ropes had to allow for adjustment of the rope length and tension, and the original arrangement therefore had to be abandoned. During the rope replacement in the early 1980s, the problem was solved by splitting each rope into three parts and introducing two additional adjustable connections near the bottom of the tunnel (*Figs. 11* and *12*). This solution is very expensive because the three pieces require six sockets instead of two in the original configuration. The bars, nuts and washers used to join the ropes are particularly prone to corrosion, and the retrofitting work included replacement of half of the bars that had been installed less than 20 years ago (*Fig. 11*).

Fig. 11: Adjustable connection at the anchorages installed in the early 1980s

The new solution made use of two seven-strand prestressing cables for each pair of suspension ropes. These cables loop around the U-shaped tunnel, connecting the ropes from the two sides halfway down the tunnel. Differential tension in the ropes is easily absorbed by friction along the tunnel where the prestressing cables rest against the rock in a bed of cement mortar.

The solution has proved to be very efficient and easy to implement although two small side-effects had to be dealt with. The anchorage plate between the prestressing cables and the rope is quite large because the socket is thick and extra space was needed for the seven-strand prestressing block anchorages (*Fig. 13*). Making room for this element required some on-site adjustments because the existing ropes had slightly different positions at the four locations.

The torque developed by the ropes during tensioning was not counterbalanced by the prestressing cables, thus forcing the anchor plate to rotate. Because of the limited space, the anchor plate had to be restrained by greasing the surface between the socket and the plate so as to reduce friction and to force the anchor plate into position.

Conclusions

The Sidi M'Cid suspension bridge has proved to be an extremely solid and well-engineered structure. After a century in service, the bridge still provides an efficient crossing for vehicles and pedestrians, even though it was originally conceived for different types of loading.

The suspension system with its parallel rope system is very efficient and easy to maintain. Its load-bearing capacity is far in excess of the platform loading capacity and the only restraint to heavier loading is the local resistance of the deck slab and transverse beams.

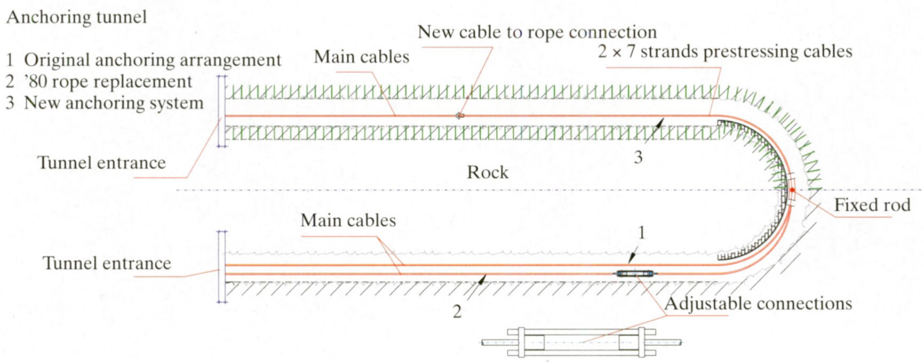

Fig. 12: Different arrangements at the anchoring tunnels

Fig. 13: New anchoring system with prestressing cables (the tunnel entrance can be seen on the right)

For this reason heavy cranes could not be used for replacement of the suspension ropes, which had to be spun across as for larger bridges. The replacement operation was carried out mostly with hand-operated winches under full traffic.

The coupling of ropes and parallel-strand cables as a means of anchorage has proved to be very efficient, although a purpose-made rope socket could minimise the size, which is likely to be a major concern for anchorages of any type.

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