SMAR 2019 – Fifth Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures

SMAR 2019

Structural upgrading of the longest and skewed span of the Yverdon Viaduct

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ABSTRACT: The Yverdon Viaduct, built in 1983 and situated in the canton of Vaud, Switzerland, is the object of a refurbishing project aiming to guarantee its remaining service life. The consulting office DIC s.a. ingénieurs in Aigle (Switzerland) was mandated in 2014 by the Swiss Federal Roads Office (FEDRO) to propose a strengthening intervention of the longest and skewed span (73.70 m) of this Viaduct. This span underwent the consequences of a conceptual default (insufficient prestressing) which led to important deformations and appearance of cracks at the lower slab of the multicellular box deck.

Three alternative strengthening solutions were proposed. Two of them were discarded, despite their superior static efficiency, due to the particular constraints of the roads authority.

The conceptual design proposes the implementation of ten post-tensioning cables inside the second and the fourth cell of the concerned span. The post-tensioning cables have a rectilinear geometry and their ends are fixed at the existing skewed crossbeams of the span's edges. New skewed crossbeams (diaphragms) made from self-compacting concrete where cast in place at a distance equal to one third of the span from each edge and serve as deviators of the cable forces.

This article presents all steps of an interesting real case study, from conceptual design of variants up to the execution phase. It shows how simple engineering concepts can be both very efficient and adapted to the numerous restraints as well. Particular attention is paid to modelling as well as to experience obtained from execution from which important lessons can be derived for both the engineering community and the authorities.

1 INTRODUCTION

1.1 The Yverdon Viaduct

The Yverdon Viaduct consists of five discrete sections, each of them 23 m wide and about 600 m long, separated from each other by expansion joints. It is a continuous multi-span structure with spans of about 32 m long, supported by a series of cylindrical piers, two piers at each crossbeam. The cross section is a six girder open section. The fifth section of the Viaduct presents a particularity. It was conceived to bridge the future Entreroches canal, aimed to link the Leman lac to the lac of Neuchatel. The canal, which was never built, was planned to cross the viaduct's axis at an angle of 35°. Thus, the fifth section of the Viaduct presents a 73.70 m long skewed span (parallelogram in plan), monolithically connected with the adjacent spans which present in plan a trapezoidal geometry, as depicted in figure 1. The cross section of the longitudinal girder of the skewed span is a 5-cell hollow box. Each of the two skewed crossbeams of the main span is supported, on a series of four cylindrical piles. The main span together with the attached spans to it, were made of cast in situ prestressed reinforced concrete, whereas the webs of the other spans are made of prefabricated prestressed reinforced concrete.

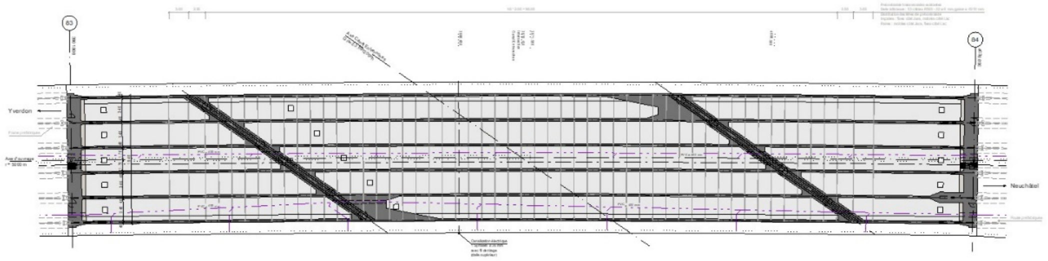

Figure 1. Situation plan of the skewed span and the attached spans to it.

2 EVALUATION OF THE CONDITION AND ASSESMENT OF THE STRUCTURAL PERFORMANCE

2.1 Evaluation of the condition of the viaduct and the main span

The Viaduct of Yverdon was regularly inspected since its creation. The visual inspection of the year 2013 concluded that the fifth viaduct and particularly the part with the closed cross section presented some important deteriorations. Ordinary pathological findings of such a viaduct after 30 years in service (figure 2a, b, and c) such as humid zones, leakage of water from deck drainage tubes inside the multi-cell box girder and corrosion of vertical steel rebars of the webs, were revealed. Furthermore, the lower slab of the multi-cell box girder of the main and skewed span presented, in the middle of it, some cracking diagonal to the longitudinal axis (figure 2d). This type of cracking should not appear under service load and is a clear sign of a structural deficiency.

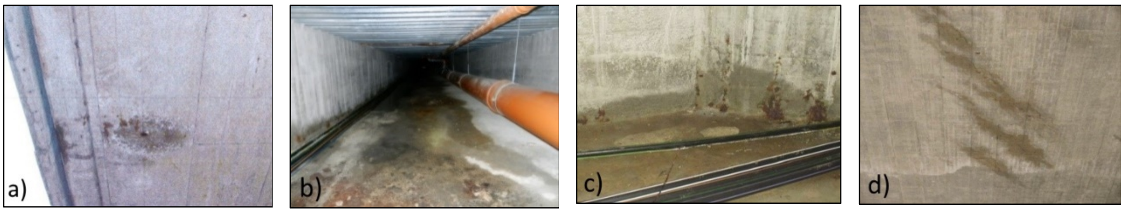

Figure 2. Photos from the visual inspection in 2013. (DIC s.a. engineers, CH-Aigle)

2.2 Assessment of the structural performance

After these findings, a structural verification was ordered and confirmed that the lower slab of the multi-cell box girder was insufficiently reinforced. In fact, the direction of the cracks was perpendicular to the tension stress field confirming thus the structural deficiency. The skewed and main span presents a remarkable slenderness, the static height of it being only 1/40 of the span. In order to avoid significant deformations and cracking at serviceability limit state, the permanent loads of such structures have to be properly balanced by prestressing. However, it was found that the balanced loading obtained from the existing prestressing was only 0.43 of the dead loads, far away from the value of modern practice, where a balanced degree varies from 0.8 to 0.9. Additional static calculations have revealed that the deflection of the main and skewed span under service load is in order only if the subtraction of the structural hogging of 16 cm is taking into account. Further investigation by means of electronic microscopy of concrete samples revealed the possibility of initiation of Alkali-aggregate reaction currently at sleeping stage (figure 3).

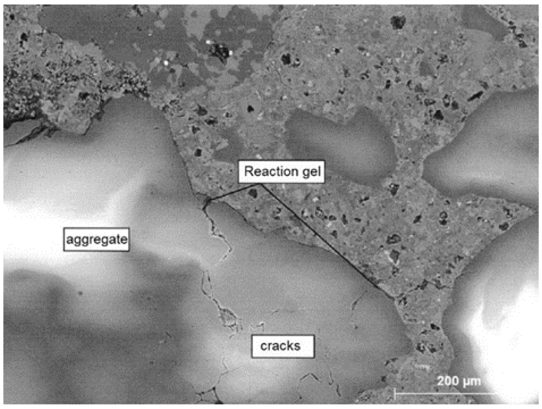

Figure 3. Photo from the electronic microscopy examination for Alkali-aggregate reaction (TFB, 2013)

The results from the visual inspections, the material investigation for Alkali-aggregate reaction and the findings of the structural verification forced the highway authority to launch a rehabilitation and strengthening project in order to avoid further damage, since waiting the planned project of the complete renovation of the highway section was risky and would lead to an increase of future intervention costs caused by acceleration of the degradation process.

3 INTERVENTION PROJECT

3.1 Presentation of the three proposed strengthening concepts

Three solutions were proposed to the local bureau of the federal road authority. The first solution, proposes the construction of a new skewed crossbeam under the middle of the main span, which lays to a new series of four columns, founded through group of piles. This new structural element is aimed to prevent further vertical deck deformation and stands for an intermediate support for the main span when subjected to excessive traffic loading. However, due to its high investment cost in comparison to the structural gain as well as to the particularity of a hybrid static system (as actually for dead load and frequent traffic, whereas with intermediate support for excessive traffic), this solution was abandoned. The second solution consist of introducing of a system of external tendons. Each tendon is anchored at the crossbeams and transfers the uplifting actions to the corresponding web of the skewed multi-cellular box girder by means of steel vertical struts. This solution, despite its structural excellent potential, presents a strong visual impact, with the risk to create unnecessary noise to the public media concerning the bearing capacity of the bridge not excluding an eventual public survey procedure for the chosen form, which would lead to delay of the execution work. For these reasons, this solution was also rejected.

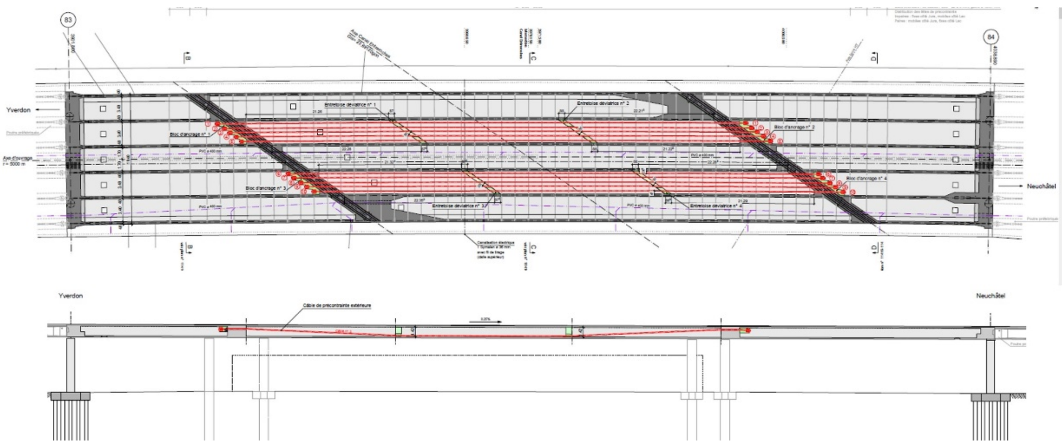

Figure 4. The third proposed technical intervention. Situation plan and longitudinal cut at the second box.

The third solution consists of adding additional post-tensioning external cables, inside the 2nd and the 4th cell of the 5-cellular box girder (figure 4). In such a way, the execution work may be performed without interruption of traffic as well as with no visual impact after reinforcement.

3.2 Structural aspects of the maintained rehabilitation and structural upgrading project

3.2.1 Validation of the structural modelling

Initially, as mentioned in section 2.2, a structural model was built to explain the cracking developed at the lower slab of the multi-cell box girder of the main span. In order to simulate as correctly as possible the real behaviour a finite element model was applied. The 3D model was built using the SCIA software. In order to diminish the calculation time, the existing prestressing of the longitudinal girders and of the cross sectional skewed beams was modelled by means of a single "equivalent" cable. The existing prestressing cables in the transverse direction were introduced with their exact geometry. Typical losses of prestressing due to friction, anchorage slip, relaxation and creep were taken into account. All useful information found in the archives was used in order to validate the structural model. In the 1970s, it was usual to perform a test loading of all new bridges. Such a loading took place also for the Yverdon Viaduct by means of 16 trucks.

The measured deformation of the span during this test loading was compared to the deformation calculated by the 3D finite element modelling. The geometrical form of the deformation due to the skewed geometry was confirmed and the absolute values of the displacement match well, with the real structure being slightly more rigid than the model. Due to the proper reproduction of the deformation, the model was considered as appropriate for the simulation of the real structure and served as a basis for the model of the reinforced structure.

3.2.2 Modelling of the reinforced structure

The 3D finite element model used for the static verification of the existing structure prior to strengthening served as a base. The additional rectilinear post-tensioning cables, five in the second box and five in the fourth box of the multi-cellular box girder were introduced to the model with their real geometry, as depicted in figure 5a. Figure 5a presents the half of the main skewed span. One may see also the outline of the new skewed cross diaphragms, which were built at one third of the main span, at the deviation points of the new rectilinear post tensioning cables. According to Static Calculations Report (2014), the additional post-tensioning cables result to an uplift of the main span, which in its middle reaches the highest value of 24 mm (figure 5b).

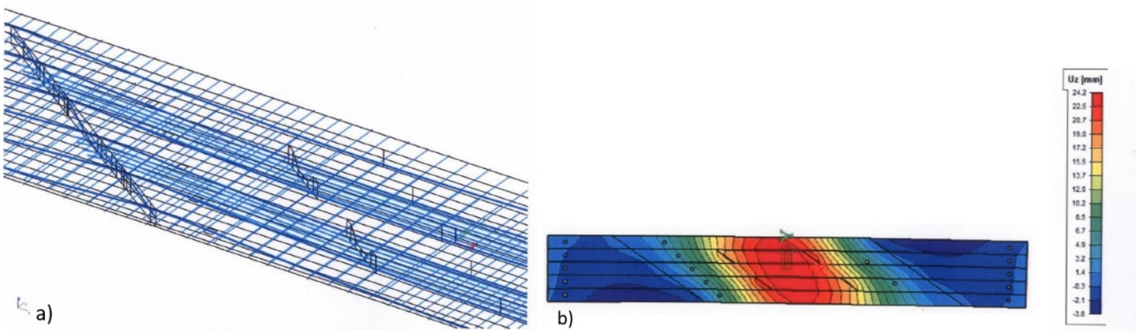

Figure 5. a) Modelling of the reinforced structure. b) Uplift of the lower slab of the skewed main span due to the additional post-tensioning cables.

3.3 *Execution aspects of the maintained rehabilitation and structural upgrading project*

3.3.1 Rehabilitation of the multi-cell box girder of the main span

Prior to reinforcing, all necessary repair works were carried out inside the multi-cellular box girder. All damaged drainage channels were replaced by new ones. The concrete of humid zones, which presented rust due to corrosion of the reinforcement, was replaced. Initially the concrete of these zones at a depth of about 2 to 3 cm was removed by hydrojetting. Where necessary, steel rebars were added and finally a high strength cementitious repair grout was applied that could provide in situ a minimum substrate tensile strength of 1.0 N/mm^2.

3.3.2 Structural upgrade of the shear resistance of the webs through carbon fibre fabrics

Besides the structural deficiency in bending moment, the main span presented also a structural deficiency for shear force. This was due to the increase of heavy traffic and especially of concentrated axle loads in comparison to the heavy traffic and the corresponding axle loads for which the bridge was designed. Moreover, the strengthening concept through implementation of post-tensioning cables with rectilinear geometry results to an introduction of high uplift forces in the new skewed diaphragms (inflection points) and consequently to an increase of the shear force at the webs of the box girder at those areas.

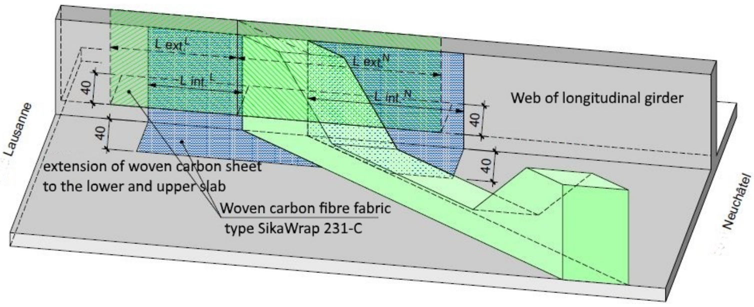

Figure 6. Perspective view of the shear retrofitting of a web of the box girder by woven unidirectional carbon fibre fabrics, in form of continuous jacket.

The structural upgrade of the shear resistance of the webs of the longitudinal multi-cell box girder was realized by applying woven unidirectional carbon fibre fabrics on the surface of the webs of the girder. The product SikaWrap-231 C was used for this purpose, which is provided in packaging rolls of 50 m long, 600 mm width and 0.13 mm thick. The concrete substrate needed to be cleaned or repaired to provide the necessary substrate tensile strength. After cleaning of the concrete substrate, an adhesive primer was applied. Finally, the carbon fabrics (sheets) were positioned on the surface of the webs by using an impregnating and laminating resin. The carbon fiber fabrics covered the whole height of the webs and extended slightly into both the upper and the lower slab, inside the box girder (figure 6). Their end anchoring was achieved by positioning tow additional rolls of a woven unidirectional carbon fibre fabric -one on the lower slab and one on the upper slab- at the longitudinal direction, in such way that its' fibres were perpendicular to the carbon fibre fabrics of the webs.

The shear structural upgrade per meter of web, reinforced by means of one-layer woven unidirectional carbon fibre continuous jacket at both sides of the web, is presented in the following equation obtained from reference (2004). A limiting tensile strain of 0.006 as well as a limiting strain safety factor 1.25, were considered for the design according to the producer's recommendations.

$$\Delta V_d = 2 \cdot V_{sjd} = 2 \cdot \varepsilon_{fd} \cdot E_f \cdot n \cdot t_j \cdot \min(h_w, 0.9\,d)$$
$$= 2 \cdot (0.006/1.25) \cdot 230\,\frac{kN}{mm^2} \cdot 1 \cdot 0.13\,mm \cdot 1420\,mm = 407\,kN$$

3.3.3 Construction of the new skewed cross diaphragms at deviation points of the new added cables

The new additional post-tensioning cables present a rectilinear geometry with two deviation points, at one third of the length from both ends respectively. The initial project foresaw two deviation points for all cables with symmetrical rectilinear geometry. As it will be explained further in section 3.3.4, this configuration had to be changed because of repositioning of some anchorages, and consequently for most cables the rectilinear geometry changed with the angle of deviation in one of the deviation points being reduced.

The deviation of the cables was realized through new skewed cross diaphragms (figure 7). These elements made of self-compacting steel-reinforced concrete and prestressing bars, transfer the deviation forces of the new cables to the existing structure through their connection with the lower slab and the webs of the multi-cellular deck. The casting took place by pumping the self-compacting concrete through the existing openings for the access of the interior of the multi-cellular box deck. The self-compacting concrete was of strength C35/45, exposure class XC4, maximum grain size D8, flowability SF2 (660-750 MM), degree of compactability C3 and chloride content CI 0.10, according to the Swiss Code SIA 262:2013 for concrete structures.

Figure 7. New skewed cross diaphragm, before and after concreting (view from the middle of the span).

3.3.4 Drilling of crossbeams and positioning of the new rectilinear cables

The author had initially proposed to the road authority to proceed to an ultra-sound radar scanning of the two existing skewed crossbeams in order to fix the placement of the anchorages of the additional post-tensioning cables in such way to avoid the conflict with the existing prestressing cables of the skewed crossbeams. It was however decided to proceed without this operation, taking into account only the existing plans, which were indicated that they were in accordance to the as-built. However, the documented information was not sufficiently accurate and at the very beginning of the coring of the skewed crossbeam, such a cable, fortunately not a very important one was damaged. The damage was compensated by adding locally on the skewed crossbeam two post-tensioning bars of grade S670 and diameter 28 mm. Due to this event, it was decided afterwards to operate the suggested ultra-sound radar scanning. This operation resulted in repositioning of the anchorages of most cables. Anchorages of cables n°4, n°5, n°9 and n°10 on the southern existing skewed crossbeam and anchorages of cables n°1, n°2, n°6 and n°7 on the northern existing skewed crossbeam where affected and had to be positioned lower than predicted. Consequently, the rectilinear geometry of these cables changed in comparison to the initial project and had to be adjusted. Besides the presence of the transversal cables, the final and exact positioning of the anchorages was defined considering also symmetry. The reason was to avoid introducing unnecessary torsion to the structure. The geometry of cables n°3 and n°8 was maintained.

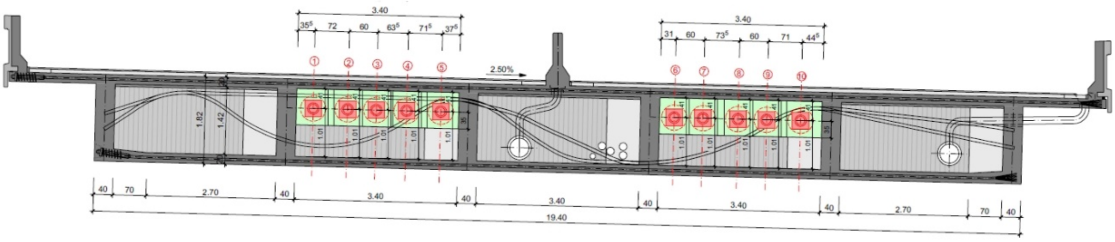

Figure 8: Lateral view of the existing skewed crossbeam (south) and the foreseen anchorage positions.

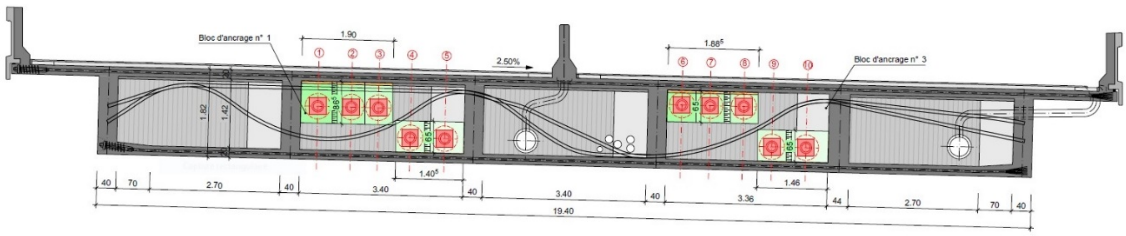

Figure 9: Lateral view of the existing skewed crossbeam (south) and the anchorage positions as executed.

The anchoring of the new cables was realized by cast in place self-compacting concrete blocks monolithically connected to the existing skewed crossbeams, as shown in figures 10 and 11.

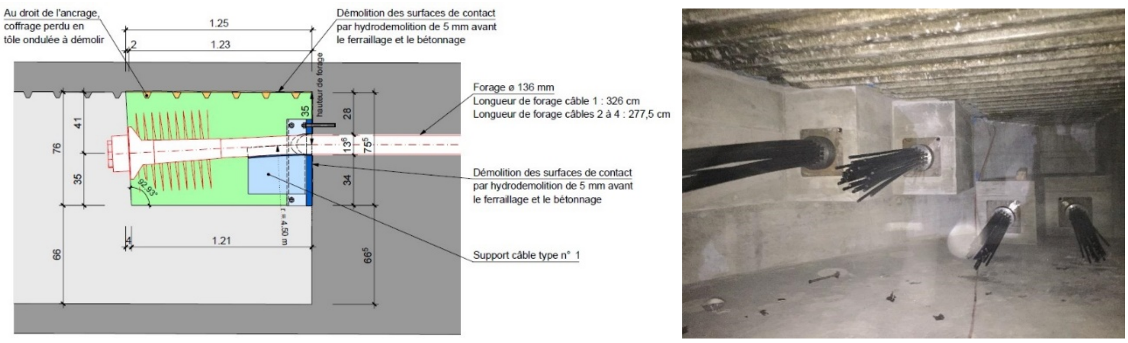

Figure 10. a) Longitudinal section detail of a typical cable anchorage at the existing skewed crossbeam at the edges of the skewed span, b) photo of four out of five anchorages of the additional external post-tensioning cables of type VSL 6-27 Y1860s7-15.7.

Figure 11. a) Unique steel element made of steel type S235 J2, for the positioning of the anchorage of the post-tensioning cables, b) steel reinforcement detail of the cable anchorage.

3.3.5 Post-tensioning procedure of the new cables

The additional post-tensioning of the main span required the installation of ten longitudinal cables of 78 meters each anchored at the pier diaphragms. Each tendons are VSL 6-27 units (27T15S), see figure 12 with the following characteristics

Fig. 12 VSL GC Anchorage

Main tensile element:
27 strands, nominal diameter 15.7 mm, nominal area 150 mm2, tensile strength UTS 1860 N/mm2

PT system components:
Stressing and passive anchorages VSL GC-30 6-27, Duct PE 80 DN 125x6.0 mm, galvanized steel supports at 8m spacing, Injection by cementitious grout type VSL HPI after stressing for long term corrosion protection.

For the thin diaphragms of this project, the trumpet and PE pipes are cast directly into the concrete. To avoid a hard spot at the exit of the PE duct from the diaphragm which could lead to spalling of concrete during stressing and undesirable stresses in the tendon, a neoprene ring is placed around the exit of the pipe. The PE pipe is assembled to the required length by mirror welding to fit between anchorages and / or deviator pipes. It is then installed to the approximate tendon profile and supported every 8 meters with a galvanized steel support. The installation of strands was done by pushing individual strands one-by-one as for conventional internal tendons. The long term corrosion protection is ensured by injecting the tendons after stressing with an optimized tendon grout type VSL HPI.

The main difficulty of the project was the installation of the strands and the stressing operation in a confined space and with very limited space available. The height of the box is only 1.4 meters and only two openings are available to access the anchorages, which are located 30 meters from bridge access points. The stressing is done symmetrically to the centre of the box, each jack is weighing 1’200 kg with a diameter of 570 mm. Due to very difficult access, and limited available space, a special frame is used to handle the jacks inside the box girder, see figure 13.

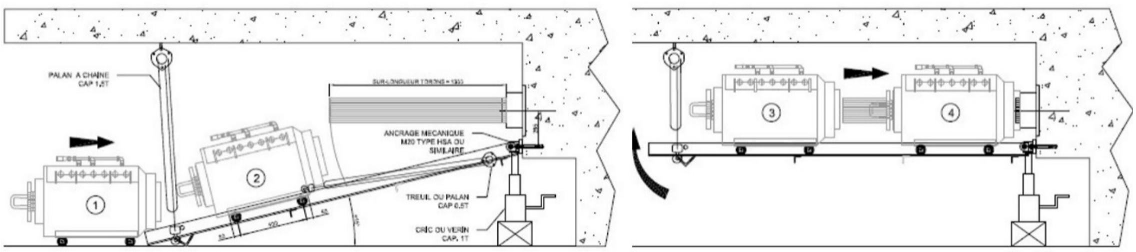

Figure 13: Special tool developed for the project to transport and position the stressing jacks at the anchorage

The post-tensioning was applied by steps at each cable one after the other and right again to the beginning according to a pre-defined sequence so as to avoid introducing torsion in the structure by the generated uplift forces. This procedure takes a lot of time and necessitated the repositioning of the jack several times for each cable. However, this procedure guaranteed the gentle and symmetric uplifting of the deck. When the uplift in the middle of the deck reached 14 mm, it was decided to block the anchorages, despite the fact that not all cables were post-tensioned at the maximal force. The owner authority in accordance with their technical specialist considered that this reserve capacity of the post-tensioned cables was important to be kept for a future intervention, for instance to compensate eventual future sagging by creep or other reasons.

3.3.6 Structural upgrade of the moment flexural resistance of the lower slab by FRP laminates

The generated forces of the additional post-tensioning cables help to overcome the lack of flexural steel reinforcement in longitudinal direction. However, their effect is not enough to cover the lack of flexural steel reinforcement in transverse direction of the lower slab of the skewed span.

Figure 14. Positioning of carbon fibre reinforced polymer strips (80 mm large, 1.4 mm thick). .

This deficit was compensated by adding strips of carbon fibre reinforced polymer (CFRP), of type Sika Carbodur M814 (Elastic Modulus 205 kN/mm^2, tensile strength 2800 N/mm^2, limiting tensile strain 0.065%), in the entire transverse direction of the lower slab for a length of 25 m, about one third of the total length of the skewed main span. This zone extended in the middle of the span and covered totally the area where the tensile cracks were formed with the viaduct in service. Their positioning, three strips per meter, was executed after repairing of the cracked areas and after the prestressing of the new cables. No particular anchorage detail of the bands was needed. The length of the bands is sufficient to activate the needed resisting tensile stress at the required areas.

3.4 Planning and costs

The whole rehabilitation and structural upgrading project of the main skewed span was carried out within a year, during 2015 without interruption of traffic. The construction cost reached an amount of 1.05 million swiss francs without tax, within the cost estimation accuracy of ±10 % according to the swiss regulation SIA 103 (2013).

4 CONCLUSIONS

Bridges designed according to older codes may present today some structural deficiency, even in serviceability limit state, due to the increase of the axial loads of heavy vehicle traffic. In order to model correctly bridges with complex geometry, detailed 3D finite element simulation is needed. Existing data from load tests prior opening to service may be used as basis for the validation of the complex simulation models. Efficient reinforcement solutions can be designed and applied, only if one has a thorough knowledge of the actual structural behavior of a bridge. Interventions on existing structures are more complex than constructing new structures. All potentially useful investigations that can be obtained at the study phase of a rehabilitation-reinforcement project help to avoid surprises and limit adaptations during construction phase. Modern materials and the available technology can help to provide intelligent solutions that are compatible to the structural behavior of the existing structure.

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