Les Cèdres footbridge over the Geneva-Lausanne motorway, in Chavannes-près-Renens, Switzerland

Pasarela de Les Cèdres sobre la autovía Ginebra-Lausana, en Chavannes-près-Renens, Suiza

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ABSTRACT

The bicycle and pedestrian footbridge of Les Cèdres, at Chavannes-près-Renens in the neighbourhood of Lausanne (Switzerland), is intended to connect two districts separated by the A1 motorway, running along the northern shore of the Geneva Lake. The footbridge and its structural concept are aimed at minimising the visual impact and to allow for a fast construction sequence, with the lowest possible interference to the motorway traffic. To achieve these goals, a slender arched-shaped frame was built, with a rise-to-span ratio equal to 1:16. The structure is constituted by a main span and two lateral spans, leading to an integral footbridge with a clear distance equal to 56.50 m between pier bases.

RESUMEN

La pasarela peatonal y ciclista de Les Cèdres, en Chavannes-près-Renens al Oeste de Lausana (Suiza), conecta dos barrios separados por la autopista A1 que discurre a lo largo de la orilla septentrional del lago Lemán. Las ideas base que han guiado el diseño formal de la pasarela han sido la de minimizar el impacto visual y permitir un proceso constructivo con la mínima interferencia posible sobre el tráfico. Para lograr estos dos objetivos, se ha diseñado y construido un arco fuertemente rebajado, con una relación flecha/luz de 1/16. La estructura está constituida por un vano principal y dos laterales, dando lugar a un puente integral con una distancia libre de 56.50 m entre los arranques de las pilas fuertemente inclinadas.

KEYWORDS: footbridge, integral bridge, composite structure, prestressing, vibration

PALABRAS CLAVE: pasarela peatonal, puente integral, estructura mixta, pretensado, vibración

1. Introduction

The bicycle and pedestrian footbridge of Les Cèdres, at Chavannes-près-Renens in the neighbourhood of Lausanne (Switzerland), is intended to connect two highly developing districts located in the west part of Lausanne and separated by the A1 motorway, running along the northern shore of the Geneva Lake.

Figure 1. Site location.

The footbridge aims also at constituting a new landmark and to improving the urban environment and landscape. At its western side, the footbridge develops at approximately 4.5 m above the road level, requiring in addition the creation of an artificial hill with a mild slope.

2. Competition and original layout

In 2012, MC2 Estudio de Ingeniería together with Cano Lasso Architects won the competition for the pedestrian-cycling walkway of Les Cèdres. The solution of the competition consisted of a stress ribbon system, with a total length of 13.15+35.7+13.15 m supported on two piers located on both sides of the motorway. The piers were longitudinally inclined to reduce the central span and incorporated a support chair for the deck (whose curvature minimised the bending moments in the stress ribbon).

The stress ribbon was composed of two steel plates with a 2000.40 mm cross section, in S355 weathering steel, separated 1.10 m from each other and supporting precast concrete segments 0.22 m thick, separated longitudinally by joints of 20 mm through which the deck drainage would happen. The equilibrium of each of the two abutments was ensured by the presence of a foundation system consisting of compression piles concurrent with the piers, tension micropiles installed in the rear part of the abutment, and a horizontal strut under the motorway (executed through a pipe-ramming procedure). In Figures 2 and 3, a render of the competition phase and a longitudinal cross section of the stress ribbon are shown respectively.

Figure **2.** Render of the stress ribbon footbridge, which was awarded the First Prize in the 2012 Competition.

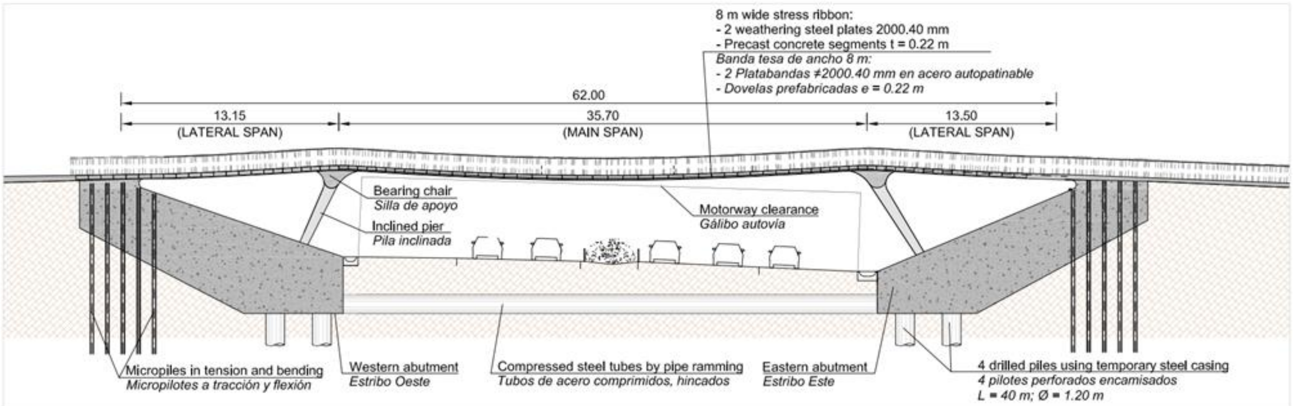

Figure **3.** Elevation of the stress ribbon footbridge.

3. New structural concept

Once the Preliminary Phase of the project was finalised, the stress ribbon typology was rejected by the Swiss Federal Road Authority (ASTRA), which reconsidered its initial approval of the drainage system and required a complete watertightness for the construction. Such demand was not compatible with the stress-ribbon concept as presented in the competition and a new design was conceived.

Following a redesign of the structure, the stress ribbon catenary was reversed into a very slender arched-shaped frame, with a rise-to-span ratio equal to 1:16. The structure is constituted by a main span and two lateral spans, leading to an integral footbridge whose total length is 62 m and with a clear distance equal to 56.50 m between pier bases (Fig. 4).

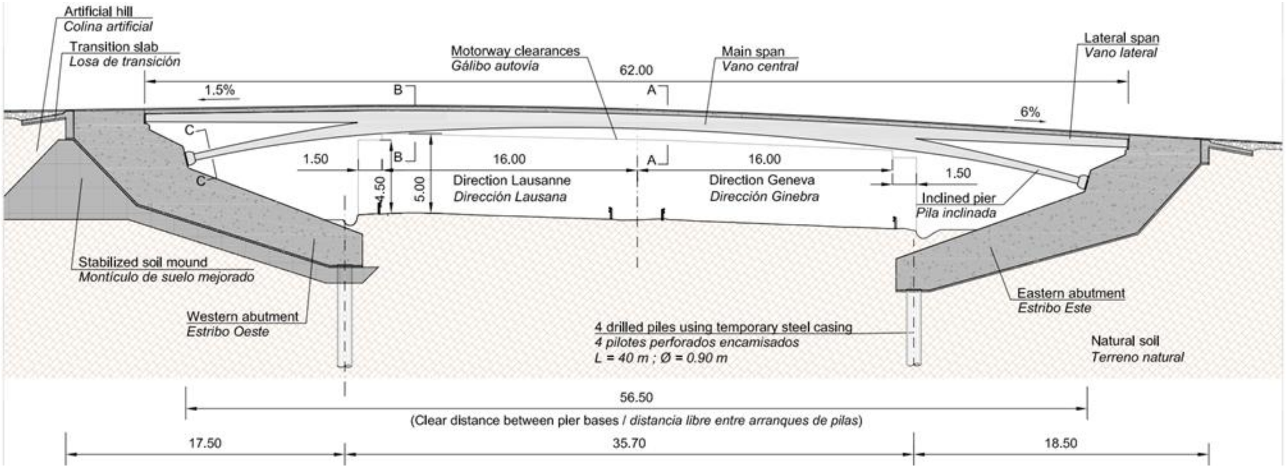

Figure 4. Elevation of the integral arched-shaped frame footbridge.

The composite deck presents a structural height of 1.13 m and 1.66 m at mid-span and over the inclined piers respectively, resulting in slenderness values of L/50 and L/34. The statical concept of the footbridge corresponds to a framed structure whose lintel is rigidly connected to the lateral system composed of inclined piers in compression and lateral spans in tension. The latter elements are connected by means of a prestressed abutment, supported by 4 eccentric piers. Due to the large width of the footbridge (8.00 m), the narrow steel box (2.30 m between webs) and the potential presence of heavy machinery over the footbridge, the concrete slab presents a thickness of 0.23 m.

The 35 m long main span is connected to the lateral structural systems formed by the inclined piers, the deck lateral spans and the prestressed abutments. This system generates a clamping cell as shown in Fig. 5.

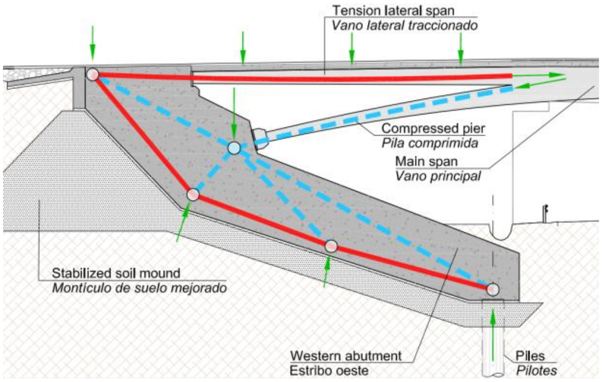

Figure 5. Schematic strut-and-tie model of the abutment

The deck is formed by a trapezoidal steel box girder, of variable height, and an upper 8 m wide concrete slab, constituting a composite structure (Fig. 6 and 7). The deck is of double-composite action, provided with a bottom slab in the areas of hogging bending moments and thus avoiding any local buckling of the steel plates (Fig. 7, section B-B). The box girder webs, with constant inclination, are extended into the piers, giving a minimum width at the start of the latter of 1.10 m.

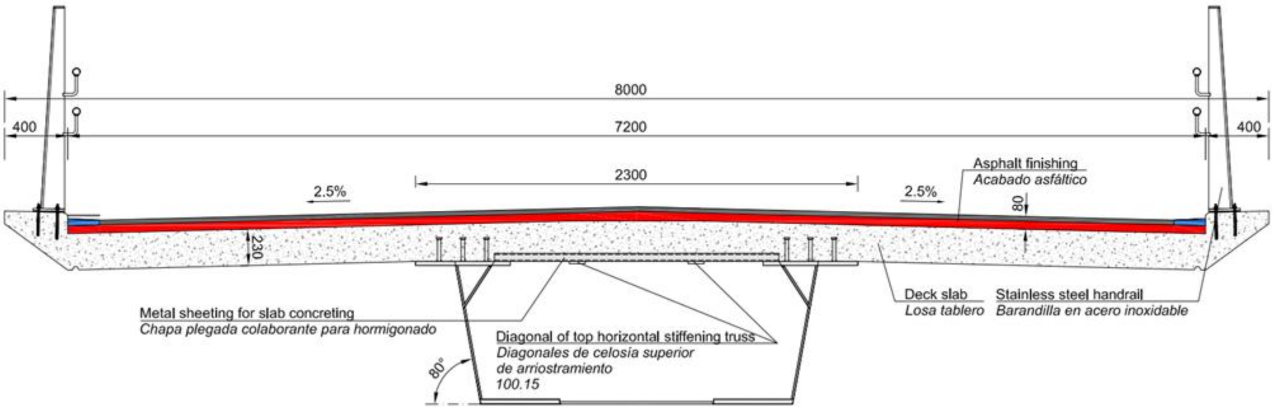

Figure 6. Cross section of the footbridge deck

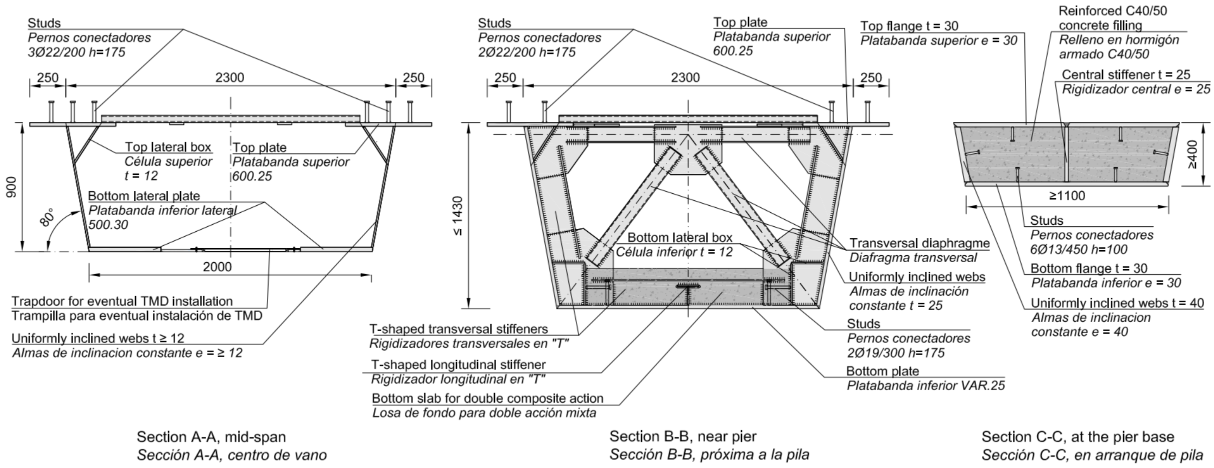

Figure 7. Typical cross sections of the steel deck and of the pier

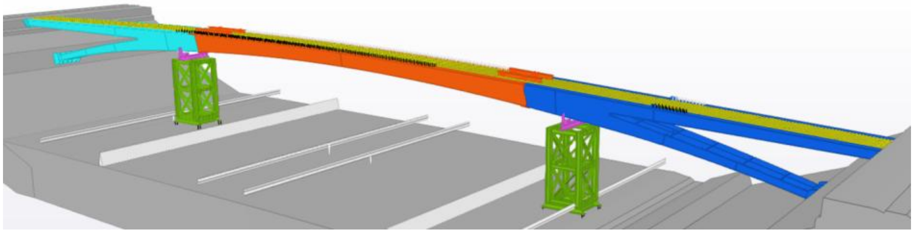

Figure 8. 3D view of the steel structure.

The piers are composed of trapezoidal steel sections, filled with concrete to withstand the significant compression forces (Fig. 7, section C-C).

The slenderness of the steel box girder and the significant width of the deck slab (Fig. 6 and 8), together with the need for a stiff structure to ensure the highest comfort class in terms of vibrations for the users, has led to a relatively high amount of structural steel per square meter of concrete deck, approximately equal to 260 kg/m^2.

The significant tension forces in the lateral spans required high amounts of longitudinal steel reinforcement in the concrete slab. In addition, due to the large cantilevers of the cross section (Fig. 6) and the presence of potentially heavy maintenance vehicles, the concrete slab also required high amounts of reinforcement in the transversal direction, with a total amount of reinforcing steel in the slab equal to 340 kg/m^3.

Each abutment presents a trapezoidal shape in plan and is supported on four Φ0.90 m piles. The abutment is post-tensioned by means of six 19Φ0.62” prestressing tendons, with the active anchors in its upper part and the post-tensioning carried out in two steps, as explained in 6. The concrete volume was 330 m^3 for each abutment, with a steel reinforcement ratio of 110 kg/m^3.

4. Main structure details

4.1. Steel structure connection to the abutment

The pier, subjected to compression (N ≅ - 22 400 kN) and biaxial bending, is connected to the abutment by means of a stiffened 1800.850.70 mm S355 steel plate and 12Φ36 mm prestressed rebars (f_u=1050 MPa), ensuring a compressed interface under all characteristic load combinations and despite the significant bending moments potentially occurring at the pier base (Fig. 9, left). The 4 rebars at the corners of the base plate were not provided with sheathing and were not post-tensioned, as they were used for installation purposes. However, the other 8 rebars are provided with corrugated metal sheathing and they are post-tensioned.

The lateral span, subjected to an axial tension force of 20 700 kN and biaxial bending, is connected to the abutment by means of two 2900.625.70 mm steel plates (front and rear plate) and 12Φ50 and 5Φ32 prestressed rebars between both plates (Fig. 9, right).

The pier base plate and the lateral span front plate are provided with passive rebars uniformly distributed and welded at their back face, to maximise the friction in the steel-to-concrete interface, thus ensuring a suitable transfer of shear forces and torque moments.

The prestressed tendons of the abutment are anchored in the upper part of the latter and thus cross the prestressing rebar region, generating a confined zone. A significant percentage of the tensile force is resisted also by passive rebars located in the central part of the deck slab and bent into the abutment to ensure their anchorage conditions. In order to avoid clashes between the different reinforcements at the same region, a precise 3D modelling of all these rebars was performed.

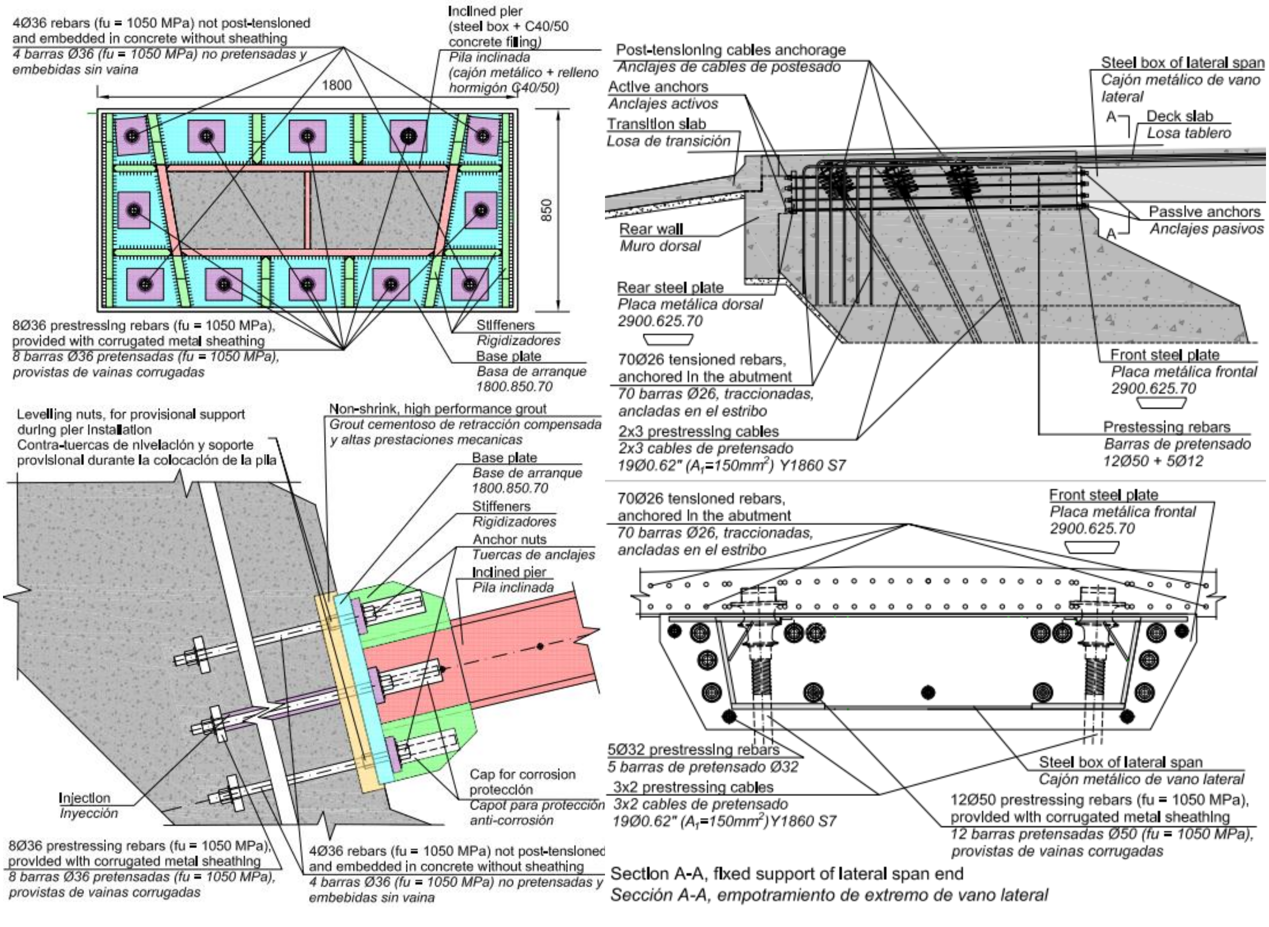

Figure 9. Pier base connection to the abutment (left) and lateral span deck connection to the abutment (right)

4.2 Connection pier-deck

Apart from separately checking each of the structural members concurring at the pier-deck node, the latter was verified by means of a 3D non-linear model to study the load transfer amongst the steel plates. Local buckling of each steel panel was considered only during erection and assembly of the structure, before the concrete was poured into the pier and deck boxes. Regarding the deck slab, only the passive rebars are modelled, given that the concrete is partially cracked at ULS. Fig. 10 shows in purple colour yielded zones, concerning only local areas of the steel structure in the following positions:

[1] Nodes where forces are applied (modelling condition without practical implications).

[2] Lower lateral boxes of the pier: in fact, given that they are connected to the concrete by means

of studs, the stresses are lower than the calculated ones and the actual yield region can be considered as negligeable.

[3] Side webs in correspondence with the connection between the upper flange of the pier and the bottom flange of the lateral span. The plasticized area is again limited and acceptable.

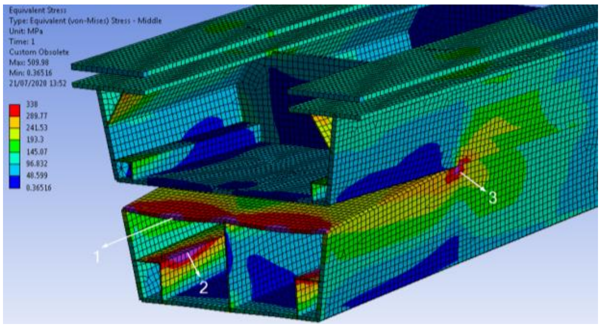

Figure 10. Pier-deck node. Von Mises stress $\leq f_{yd}$

5. Structure dynamics

5.1 Comfort requirements and TMD preliminary design

One of the basic requirements of the owner was the need to ensure the highest degree of comfort (CL1 comfort class according to table 4-4 of [4]), characterised by the following vertical and lateral accelerations limits, under pedestrian excitation (class TC3: 0.5 pers./m^2): $a_{v\ limit} = 0.50$ m/s^2; $a_{h\ limit} = 0.10$ m/s^2. Even though in the design phase, through the FEM model, the arch-shaped frame is assessed to be sufficiently rigid to respect the aforementioned limits, several uncertainties were identified related to the soil stiffness under dynamic excitation and the mobilisation capacity of the foundation mass by the pedestrian load. Consequently, it was decided to design the footbridge allowing for the potential installation of a Tuned-Mass Damper (TMD). The latter was predimensioned in the design phase, for the assumed traffic and acceleration limits (loading models of walking and running were set according to Bachmann et al. [5]).

Structure eigenfrequency and modal mass were not easy to predict due to soil properties uncertainty. However, based on a thorough sensitivity analysis and the designers' experience, the following parameters combinations f_1 / m_1 were considered: 3 Hz / 260 t; 3 Hz / 320 t; 3.3 Hz / 260 t; 3.3 Hz / 320 t. The 2nd scenario (min. eigenfrequency and max. modal mass) and 4th scenario (max. eigenfrequency and max. modal mass) revealed to be irrelevant cases.

The TMD would have been manufactured with main helical springs and a set of secondary tuning springs (Fig. 11). With the multiple goal of optimising the TMD, it was not manufactured during the construction phase, but its fabrication would have started after the structure was erected and the dynamic tests performed. The main helical springs would have been designed to provide the required natural frequency of the TMD, while the tuning springs would have been adjusted on site, after carrying out further dynamic tests, with the TMD installed into the steel deck and participating in the dynamic behaviour of the footbridge.

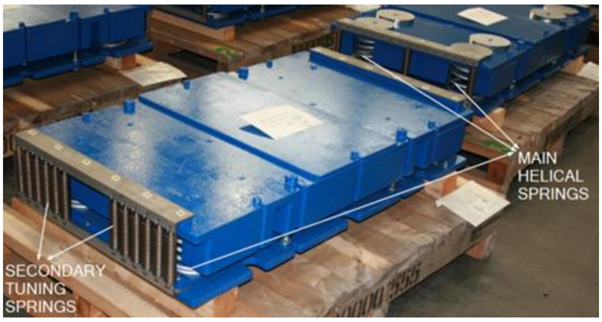

Figure 11. Example of TMD (photo courtesy of Maurer)

For the predimensioning of the TMD, and assuming a structure eigenfrequency of $(f_{1,min} + f_{1,max})/2$ and a modal mass of $(m_{1,min} + m_{1,max})/2$, the following TMD characteristics are obtained:

- Oscillating mass = 5.2 t
- Nominal natural frequency = 3.122 Hz
- Nominal spring stiffness = 2.0011 MN/m
- Nominal static spring deflection = 0.027 m
- Damping ratio = 8.09 %
- Viscous damping coef. = 16.665 kNs/m
- Relative motion = ±16 mm
- Tuning range of TMD natural freq. = ±8 %

5.2. Execution and dynamic test

Dynamic tests were performed once the structure was completed, including handrails and asphalt deck since they participate in the structure mass, stiffness and damping. A dynamic test was carried out by Maurer SA with the double aim of corroborating that the actual structure response reflects the analytical predictions and certifying that CL1 class of maximum comfort for users was respected (without the need to install a TMD into the deck). An accelerometer was placed at mid-span and on the axis of the deck, thus at the antinode of the first vertical flexural mode of the structure. In order to analyse and to corroborate the test results, a synthesis of which is presented below, a FEM model considering only the composite structure was used (as the abutments were hardly mobilized accounting for their high mass).

5.2.1. Structure eigenfrequency

The eigenfrequency of the first vertical bending mode is derived from the PSD (Power Spectrum Density Estimate via Welch Method) of the measured vertical acceleration. It results:

- Eigenfrequency calculated through a model with articulated supports: $f_1 = 3.144$ Hz.
- Eigenfrequency calculated through a model with clamped supports: $f_1 = 3.211$ Hz.
- Structure frequency derived from the test, under ambient excitation: $f_1 = 3.357$ Hz
- Structure frequency derived from the test, under soft walking around the accelerometer (2-3 persons): $f_1 = 3.32$ Hz

The correspondence between the modelled structure and the real one is evident (f_{test} / f_{model} = 1.05÷1.07), the latter being slightly stiffer, as it usually takes place.

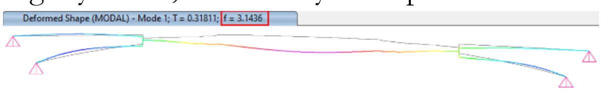

Figure 12. Structure eigenfrequency calculated through a FEM model with articulated supports

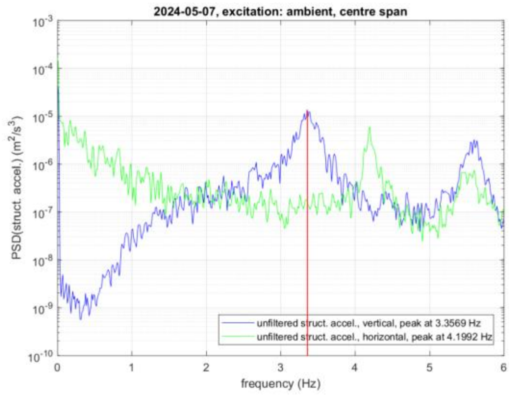

Figure 13. PSD of vertical acceleration due to ambient excitation

5.2.2. Acceleration due to walking

The peak vertical acceleration of the footbridge, subjected to the action of 6 and 10 people walking more or less synchronized with an excitation pace of 2.2-2.5 Hz, is measured, resulting in 0.015 and 0.029 m/s^2 respectively.

In the calculation model, the structure is excited by the load model DLM2 defined in [1], describing the effect of a group with a limited number of unsorted walking persons (8-15 persons). With an excitation frequency of 2.2 Hz and 2.5 Hz, vertical accelerations in the centre of the span are 0.009 and 0.0142 m/s^2 respectively, matching the measured value and being significantly below the acceleration limit for maximum comfort class.

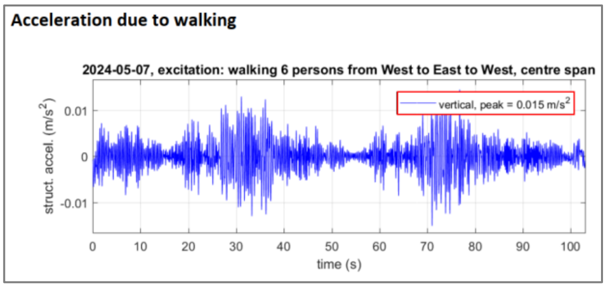

Figure 14. Structure vertical acceleration due to 6 people walking, pace 2.2 – 2.5 Hz

Figure 15. 10 people walking test, pace 2.2-2.5 Hz

5.2.3. Acceleration due to running

The dynamic response of the footbridge, subjected to the action of 6 people running more or less synchronized with an excitation pace of 3 Hz, is assessed, resulting in vertical and horizontal accelerations of 0.169 and 0.023 m/s^2 respectively. These values are again significantly below the acceleration limit for the CL1 class. Due to the significant width of the concrete deck (8 m), the footbridge presents an extremely high horizontal stiffness in the transverse direction, leading to negligible values of horizontal acceleration under environmental and pedestrian excitation.

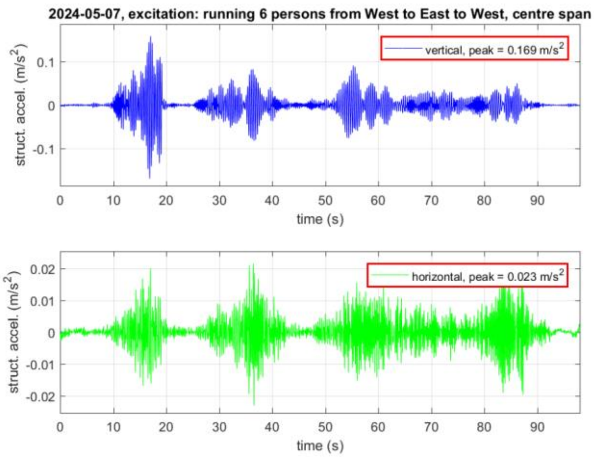

Figure 16. Structure vertical and horizontal acceleration due to 6 people running, pace 3 Hz

6. Construction process

The construction phases are herein summarized:

- Execution of stabilized soil.
- For each abutment, execution of 4 drilled cast-in-situ D0.90m piles, 40 m long, using temporary steel casing of full pile length.
- Execution of sheet pile enclosure around the abutment and lean concrete (Fig. 17).
- Execution of the semi-buried abutments, each one in 7 concreting phases (Fig. 18).
- Partial prestressing (50 %) of the 6 cables.
- Steel structure fabrication at the workshop, composed of 5 pieces: 2×(pier+deck node); 2×(lateral span); 1×(main span) (Fig. 19).
- Transportation and installation of the first structure component (Fig. 20).
- Transportation and installation of the lateral span (Fig. 21).

Figure 17. Sheet pile enclosure, excavation and lean concrete execution (phase III)

Figure 18. Partially buried abutment

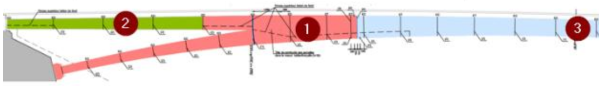

Figure 19. Steel structure mega-pieces

Figure 20. Installation of one pier + deck node

Figure 21. Installation of the lateral span steel deck

- Prestressing of the rebars (Fig. 22) ensuring the connection between the steel deck of the lateral span and the abutment upper part.
- Transportation and installation of the main span (Fig. 23), provided with the falsework.

- Removal of the propping towers.
- Completion of the formwork installation and steel reinforcement placing.
- Concreting of the central part of the deck slab, over a metal sheeting (Fig. 24).
- Completion of the steel reinforcement in the lateral parts of the slab (Fig. 25).
- Concreting of the slab cantilevers (Fig. 26).
- Completion of the prestressing (100 %) of the 6 cables of each abutment (Fig. 27).

Figure 22. Tensioning of the prestressing bars

Figure 23. Installation of the steel deck of the main span

Figure 24. Concreting of the central part of the deck slab

Figure 25. Steel reinforcement of the deck slab edge

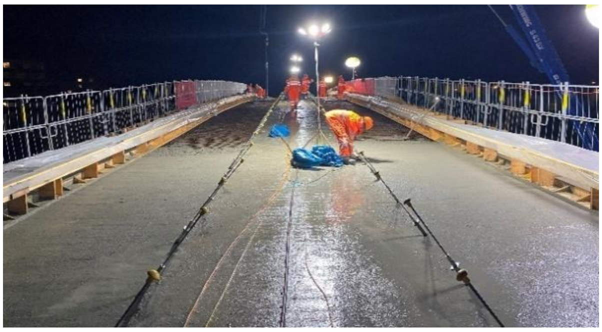

Figure 26. Concreting of the deck slab cantilevers

Figure 27. Prestressing cable tensioning completion

- Placing of the handrails and execution of the asphalt floor finishing (Fig. 28).
- Dynamic testing of the footbridge (Fig. 15).

Figure 28. Main structure completed

7. Final result and conclusion

The Les Cèdres footbridge has resulted into a slender composite structure, satisfactory meeting the requirements of the client in terms of functionality, vibrations and site landmark. The final project is the result of a long process lasting for more than ten years (2013-2024), where multiple changes were incorporated to satisfy modifications in the requirements by the administrations as well as evolutions in the state-of-the-art.

The initial concept, a stress-ribbon footbridge, was a light, elegant and efficient structure with a low material consumption. Nevertheless, such concept had to be abandoned in 2017 due to watertightness requirements by the Swiss Federal Road Administration. To ensure watertightness and to respect the original spirit of the project, dominated by a light structure over the highway, a slender arch-shaped frame was finally built. Such change, in a site dominated by poor geotechnical conditions, was however associated to a relatively high consumption of resources in the abutments and deck.

Finalized in 2024, the footbridge is perceived as an elegant landmark for the users and satisfactorily meeting all requirements. The authors nevertheless acknowledge that current sensitivity in terms of sustainability aspects would probably have led to a potentially different evolution, humbler, in an effort to minimize material consumption and ecological footprint. This project is thus a good example to learn (from its positive and negative aspects) for future projects and to enrich the decision taken during the conceptual design phase.

8. Acknowledgements

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Figure 29. Main structure completed