LAUNCHING ON SUPPLE SKATES

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Abstract

The paper presents the push launching of a large steel railway viaduct in Caracas, Venezuela. The bridge with a total length of 458m and a weight of 5500 ton is made of 4 simply supported 60m girders and a continuous one of 3 spans the central one of 110m. The bridge is launched from the abutment joining together the 5 girders one to another by prestressing bars to be released and removed once the bridge is in place. Given the hogging profile of the bridge intrados, the cusps formed by joining the adjacent simply supported spans and the necessity to place two plus two skates on each pier to reduce the contact pressure, an innovative design of the skates has been devised to minimize the force localization and insure an evenly distribution of the reactions among the different skates on each pier and among subsequent piers.

This has been obtained by introducing a significant flexibility in the skate response using rubber pads supporting a telescopic mechanism capable of translating and rotating in the vertical plane. The skate design is specifically addressed with technical details, numerical simulations and results of laboratory tests.

1. Introduction

Venezuela is currently carrying out a significant improvement of its road and railway infrastructures. Among these projects a new railway line connecting Caracas to its suburbs and satellite cities is currently under construction. The viaduct under consideration (Viaducto 1.1) is placed along this line, few miles away from Caracas. The bridge spans a shallow valley while crossing over the motorway connecting Caracas with its International Airport "La Guaira". The viaduct is part of a contract managed by a JV of Italian companies (Impregilo, Ghella, Astaldi) with a local sub-contractor, Preacero Pellizzari, in charge of the fabrication and erection of the steel deck.

This paper is part of the 7th International Conference on Short and Medium Span Bridges, Montreal, Canada, 2006.

From the beginning all parties agreed on the advantages of launching the bridge from the South abutment would given the local topography and the opportunity of this abutment being located in within a military area with restricted access and excellent security conditions for the working personnel and equipment. Furthermore, the incremental launching would allow crossing the above mentioned motorway without interruption to traffic.
The authors have been in charge of the whole launching project including the design of all the necessary equipments and temporary structures, the verification of the bridge in the different launching phases and the on-site technical assistance.

2. Viaducto

The structure consists of a continuous beam with 3 span of 54-110-55m respectively and 4 simply supported beams of 60m span each (see Figure 1).

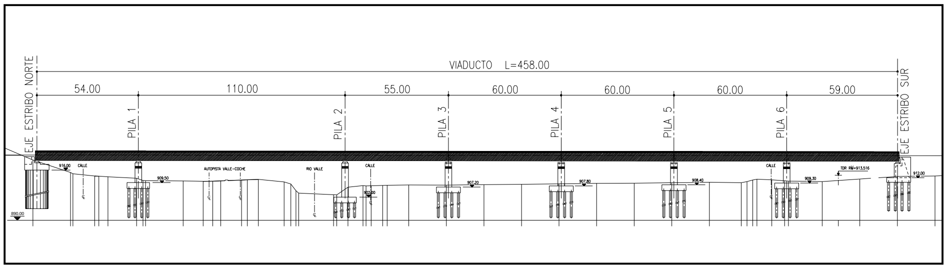

Figure 1: Bridge profile


Photo 1: Bridge cross-section

The bridge cross-section is made of two plated steel girders 5m high connected with lower transverse beams carrying an orthotropic steel deck (see Photo 1). Average structural weight is 11 ton/m, with the heaviest segments of the continuous deck at nearly 15 ton/m.

When deciding the most suitable launching method a number of factors had to be taken into account, most of them unfortunately adverse.

- The bridge intrados sport significant hogging under self weight. This is to counterbalance the sagging under dead load but is also used as an imposed distortion to transfer the vertical reactions of the continuous girder towards the end supports and avoid uplift under live load. Undeformed hogging therefore varies from 52 mm for the simply supported beams to 430 mm for the continuous one as showed in Figure 2.
- Jointing together the 5 girders with this significant hogging creates 4 severe cusps hardly smoothed by the 30cm transition steel keys added between the 5 girders so as to launch the bridge with is final geometry (length).
- Use of intermediate temporary supports was not advisable for security and structural reason. Only one temporary support was allowed under the main continuous span (in a gas station!) to reduce the maximum launching free length to 75m circa.

The three factors together made it practically impossible to launch without using adaptive skates to redistribute and spread the load (reactions) so as not to damage the steel beams or the underneath pier caps. Furthermore, the bridge is located in a highly seismic areas (PGA^{475}=0.67g) and therefore makes use of numerous seismic devices (isolator, shear keys, stoppers, etcc) making the pier caps particularly crowded and hardly ideal for placing the equipment required for launching.

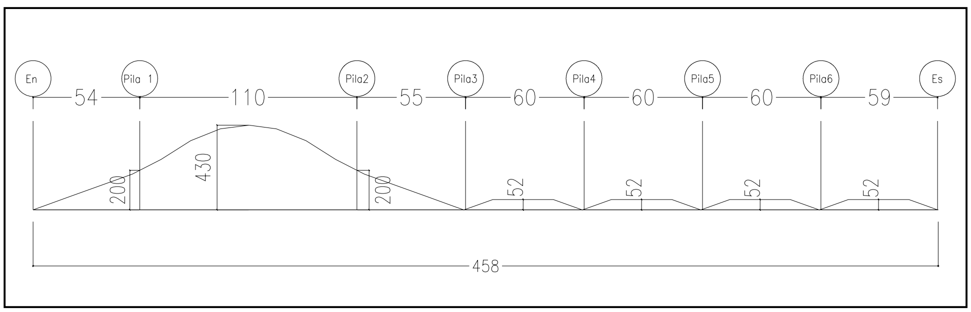

Figure 2: Bridge undeformed hogging

3. The push launching

From the very beginning it was clear that a push launching would work best as the total launching length would make pulling uneconomical and dangerous and with an overall bad handling because of the intrinsic flexibility of the method [1] .

Since the structure has significant strength reserves it was also decided that no intermediate supports were needed except from one under the main continuous span of 110 m. For the same reason, the launching nose length was fixed so as to keep the maximum reactions on the front skates when cantilevering out of the same order of magnitude of the maximum reactions in trail. The nose is therefore made of a 32.5m long truss girder with welded T section for the top and bottom chords and standard UPN for diagonals.

Once the overall layout was fixed, the study had to concentrate on the resulting skate reactions and local stresses in the bridge steel beams. Finite element simulation of the launching phases showed the maximum reactions to be in excess of 1200t under the continuous girder, well above the 800t of a continuous beam of the same weight over 60m span. The maximum reaction per unit length that could be transferred by the fillet welding between the webs and bottom plates of the main steel girders was estimated at 3 ton/cm therefore requiring a total skate length of 4m minimum on each pier, assuming an evenly distribution of the reaction itself. This value was also compatible with other two designing factors:

1. Maximum unit load causing local buckling of the main girder webs. The above value allowed for a safety factor not less then 1.4 except from few spots where additional vertical stiffeners were required .
2. Maximum contact pressure on the sliding surface of the skates.

The latter also designed the minimum spacing between the butt straps along the bottom flanges of the main girders.

4. The temporary joints

The temporary joints between the 5 girders are to be released once the bridge is in its final position. At that point these joints, located above the piers, are under severe hogging moments. To facilitate the task, the joints are made of steel keys prestressed between the beam ends with high strength bars. These bars can be released stepwise without the risk of the joint opening and shearing off a traditional bolted connection. During the push launching these joints are subjected to alternate bending moments and shear forces. Maximum bending being 7800 ton*m and maximum shear 720 ton per deck or half these values for each girder. The final design of the joints uses 4 steel inserts for each beam, two connecting the upper and lower flanges plus two shear keys positioned along the web. This configuration obviously requires a certain attention to detailing since the 4 inserts may not work simultaneously if misaligned.

To this extent the shear keys where designed to minimize this risk therefore allowing for construction tolerances as shown in Photo 2. This was obtained by inclining the contact surfaces at 45^{o} along the principal (shear) force directions.

Another issue to be addressed at the design stage was the clearance above the pier when the prestressing bars and the steel shoes used to connect them to the main girders had to pass over the skates .

Photo 2: Shear key

In the final design 36 bars D40 were used to join the continuous girder and the simply supported one. 12 bars each for the top flanges and 6 for the bottom one since sagging moments during launching are lower. Similarly 28 bars were required between the simply supported spans (3 joints).

Photo 3: Top Flange connection

5. The supple skates

With a total skate length in excess of 2m on each side of the pier it is very likely the vertical reaction localizes dangerously increasing the contact pressure on the sliding surface and the shear stresses on the fillet welds. This is even much so as the required 2 metre sliding length had to be obtained with two 1.2m skates positioned 1.7m to 2.3m apart to allow for other subsequent operations to be carried out.

In order to avoid localization either an active control of the skates was to be used or the skates are flexible enough to redistribute the reaction. Redistribution should be achieved at three different length scales:

1. The smaller scale is that of the cusps formed when joining two adjacent hogging spans. The angle formed by the bottom plate is 0.2° that means 3.5mm/m.
2. The intermediate one is that between the two skates that are 2 metres apart. Lack of horizontality of the beam intrados caused by rotation of the girder when cantilevering out during launching and underormed hogging may add up to a maximum of 17mm in 2 metres.
3. The larger scale would be that of redistributing the reactions among subsequent piers counteracting the undeformed girder hogging.

The latter scale was clearly not achievable in full as these would require relative displacement of the skates on adjacent piers of up to 430mm for the heavier continuous girder. The other two scales were tackled by inserting a 170mm rubber pad into a steel telescopic mechanism made of simple welded plates, shown in Figure 3 and Photo 4.

A prototype was built and tested at the FIP facilities in Selvazzano (PD) Italy. The force displacement response of the skate is depicted in Figure 4, while on Figure 5 the response under eccentric compression is reported.

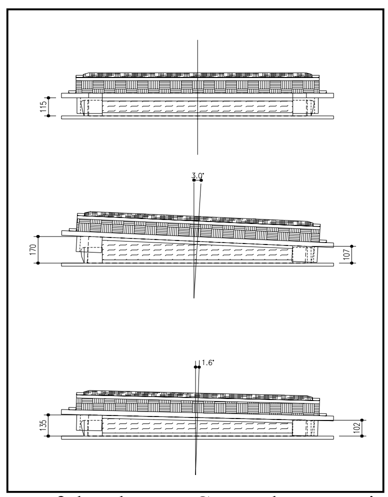

Figure 4 and 5: Force displacement response of the skates. Centred, eccentric.

The design of the skates had stringent geometrical constraints in term of height and width having fixed the total skate length at 1.2m. These geometrical constraints were partly due to the reduced clearance over the piers and partly due to static necessities.

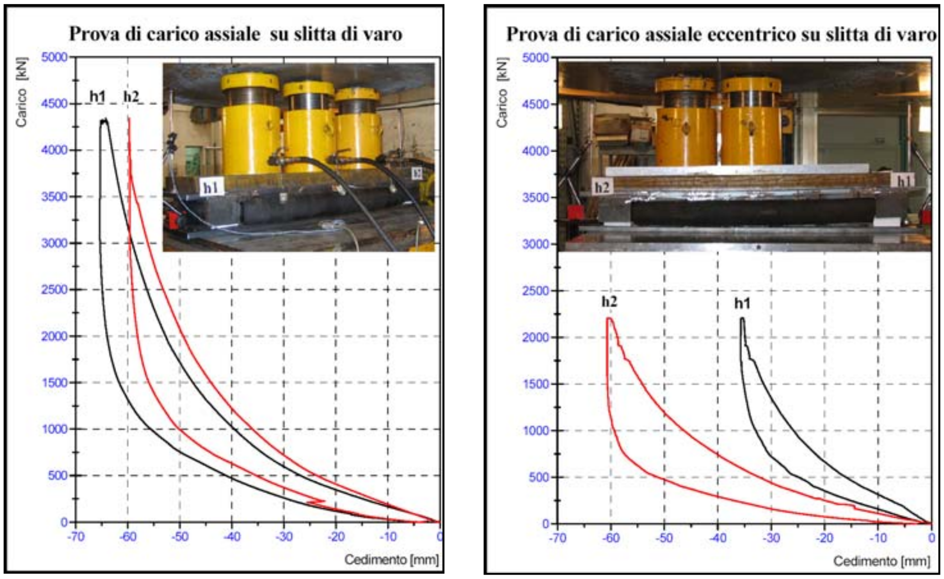

Photo 4 and Figure 3: The supple skates

A minimum height was required for the necessary compliance of the skates and a minimum width to keep the compressive stresses in the rubber pad below 10MPa. These dimensions had also to satisfy the stability verifications as rubber is prone to instability under compression when certain geometric ratios are trespassed (a total encasement of the rubber into a steel box would have caused a too stiff response).

5. The hydraulic equipment and launching yard

The launching yard has been equipped with two concrete launching girders resting on a concrete slab 63m long and 15m wide. The whole area is protected by an industrial shed where the deck assembly take place. In each launching phase roughly 45m of deck are assembled and pushed. The driving force is provided by two hydraulic jacks with a maximum capacity of 400t each and 2m drive. Cylinder specifications as follows:

- 14” Bore; 8” Rod dia.; 78.74” working stroke; 82.74” total stroke;
- Operative pressure 5000 P.S.I. max.

The reaction forces are provided by the two launching concrete girders via two steel saddles and massive steel pins acting as shear keys, as shown in Photo 5. The launching girders are heavily reinforced (see Photo 6) as their width, once again, was limited by the same geometrical constraints designing the equipment over the piers.

Photo 5: The launching yard

Photo 6: The launching concrete girders steel reinforcements

6. The numerical simulations

The launching phases have been numerically simulated every 30 m circa for a total number of 25 steps. For each span 3 analyses were carried out namely with the launching nose just resting on the first pair of skates, with the girder on the 4 skates and the launching nose cantilevering out and with the nose just short of the next support. Given the non repetitive bridge geometry (heading continuous girder), none of the steps could be omitted. The time history of the reaction at Pier 6 is plotted in Fig. 6. The graphs clearly shows two peaks due to the passage of the continuous girder and a trailing history of smaller reactions due to the simply supported spans with the hogging in "phase" with the supports (piers).

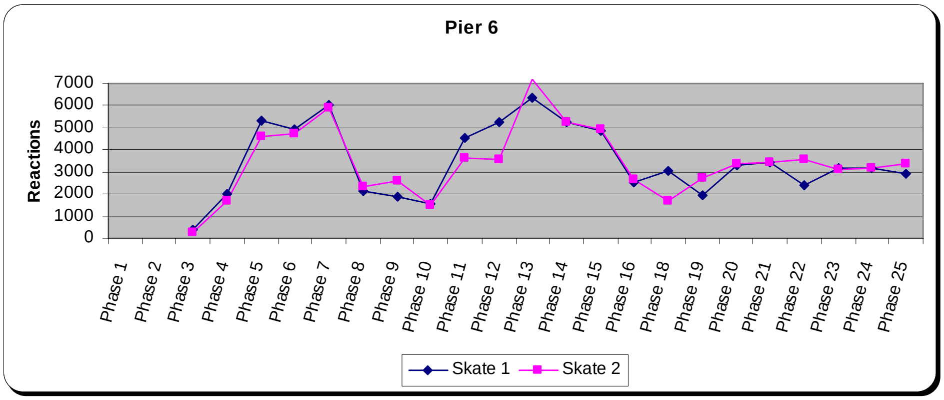

Figure 6: Reaction time history at Pier 6

The design of the launching nose required a number of intermediate steps, one every 4m circa, that were carried out with respect to the heaviest load case only, taking place on reaching the temporary support after the 75m fly over the motorway. With these reactions, a full 3D simulation of this structure was carried out, taking also into account seismic and wind loading.

Finally, a seismic analysis of the bridge during the launching operation had to be carried out in order to designing the lateral guides to be placed at each pier. The Ultimate State design earthquake (475 years return period) for the bridge, has been fixed, by the client, at PGA=0.67g. The scheduled 6 months launching operation was conservatively extended to 1 year. Accepting a trespassing probability, during 1 year time of 10%, a 10 year return period was used consequently. Using well established scaling rules (see [2]), this gives a PGA=0.15g. It was then decided to limit to 30mm the maximum bridge lateral displacement at the pier (skates). With a displacement based approach [3] the lateral retaining steel frames were finally dimensioned. These frames are made of twin HEA340 tied against the pier caps by 2 prestressing bars D40 each, as shown in Fig. 7. A telescopic mechanism on the top allowed for the bottom flange width variation of the various girder segments. The same lateral frames were also to be used as the bridge lateral guides during launching. It's worth noticing that seismic forces exceeded those required to laterally redirect the bridge during launching (both under static and dynamic friction).

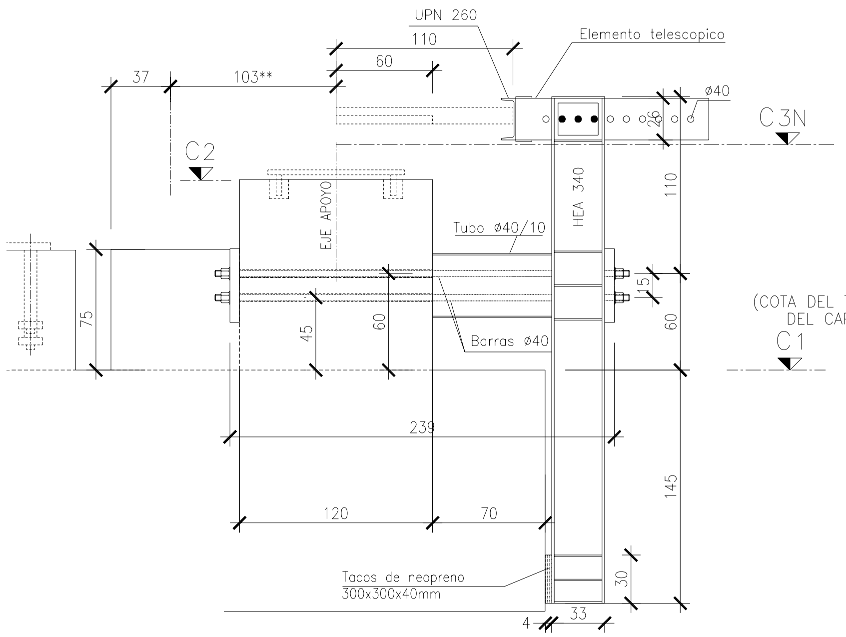

Figure 7: The lateral retaining stopper scheme

7. Feedback from site operations

At the time of writing the bridge has arrived into its final position and lowered onto the bearings. Launching has taken roughly the 6 months scheduled at the design stage. Each 45 m segment has taken 2 weeks to be assembled and push launched. A small delay was caused by the feeding chain ahead of lunching since the steel plates were shipped from Europe to South America and then manufactured in the Pelizzari yard, before being delivered by truck on site. These delays could hardly be made up for on site since assembly by bolting and welding of the bridge segments could not be squeezed below the above said 2 weeks and each fly (45m) required a whole day with the deck fully out on 8 supports and 30 skates.

As far as the behavior of the supple skates is concerned, this was in line with the theoretical calculations, and the experimental tests carried out in Italy. A minor setback was caused by using local rubber instead of the specified one (Shore A3=50; G≥0.7 N/mm^2) being delivered late on site. The former did not have the same strength and modulus and therefore squashed until part of the load was directly transferred by the skates steel guides. These rubber pads were subsequently substituted with the design ones.

The driving force required (i.e. the equivalent friction coefficient) was in line with the value generally obtained in similar cases. This means that the push equipment was substantially redundant having been designed for an 8% friction and then further increased to 400+400ton. The measured static friction rarely exceeded 5% dropping to 2.5% under motion with the skates properly lubricated.

Photo 7: The launching phase over the motorway

8. References

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