A Signature Bridge in a Congested Urban Area

Mohamed Amine Ketfi works mainly on international projects in connection with the production of particular design for bridge and viaducts and, more specifically, for U-profile elevated metro systems among which the Ujembu metro in Korea or that in Mumbai (India).

He also produces design for the rehabilitation of road stay cables bridges, among which the Penang road bridge in Malaysia

1 SUMMARY:

Metro line 1 in Mumbai takes place in the north of Mumbai between Versova and Ghatkopar. This main line named VAG corridor for Versova-Andheri-Ghatkopar is long of 11.443 km in elevated alignment. It runs along the middle of the Andheri Curla Road. The site is near the Oman Sea and for a sustainable design it was imposed to observe a maximum open crack of 0,1mm following the IRS CBC 1997 standard.
The rolling stock includes coaches with a weight of 170 kN per axle, and boggy spaced by 12m.

This crossing was positioned near the WEH station. The constraints of the alignment level (to limit height), the closeness of WEH station on the East, the over crossing of the Western Express Highway imply choosing a very thin deck and a construction method adapted to cross over the WEH (see fig 5). Concerning the construction methods, a scaffolding method was not adapted due to the traffic and the same for the classical cantilever method which needed a thick deck, which would not respect the maximum distance between ground level and top of rail level.
A U shape giving a slim thickness between top of rail and the extreme low fiber was a good compromise. The height of the lateral web, kept equal to the standard spans, has the same visual impact. But the slenderness of the deck was too important (86/2=43) for a railway to use the cantilever method without temporary bends, and bends were not possible with WEH crossing. So, in order to keep the cantilever method and a thin deck, a stay cable bridge was proposed for the design, and it can be looked as a signature bridge. The stay cables could work as temporary supports during construction and gave a significant stiffness during operation.
To support two tracks a wide u shape girder was proposed with two lateral thick webs as shown in Fig. 3.

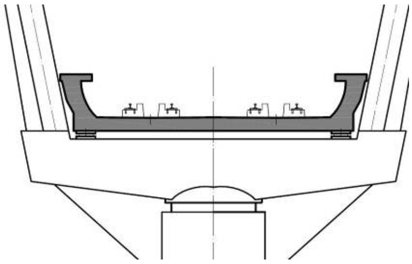


Fig. 1 : Wide u shape supporting 2 tracks

Fig. 2 : WEH congested area view

The elevation of the bridge (see fig 5) shows the distribution of spans: 23m - 23m - 86m - 23m - 23m. As it is symmetrical, only half configuration is shown.

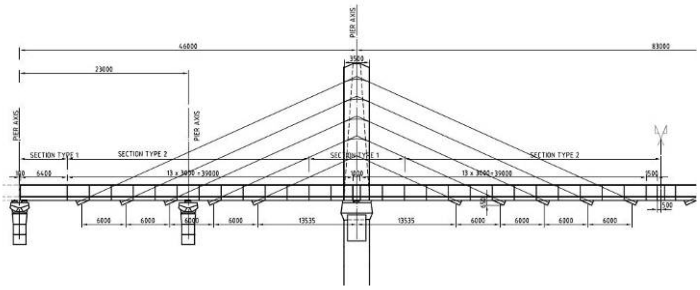


Fig. 3 : ½ elevation and span configuration

Fig. 4 : Crowded site of WEH

Two reinforced concrete pylons were laid out each side of WEH with two legs inclined on the external side of 12.5° with the vertical. Each leg is 18.631m high with a thickness varying from 1.5m at feet to 0.88m at head, and a width varying from 3.9m at feet to 3.5m at head. So the stay cables are disposed in two lateral screens.

The stays layout follows the principle of harp arrangement, with angle varying between 24.1° and 24.2°. The stays run inside saddle offering a scrapping stiffness to avoid any sliding [1]; VSL System SSI 2000 is used, unit 27T15S per cable with maximum stress 0,45 fguts.

The deck passes through the pylon, and it is supported by special pot bearings. The access spans to the main span are made from two spans of 23m long each. The intermediate support is an anti-up-lift pier with a particular layout which is explained below.

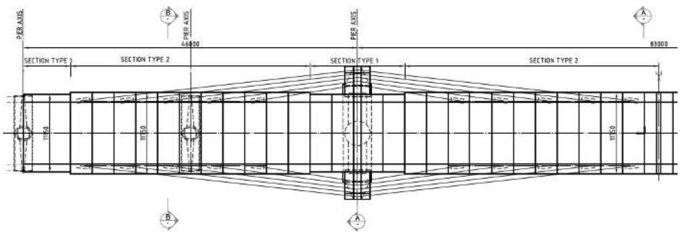


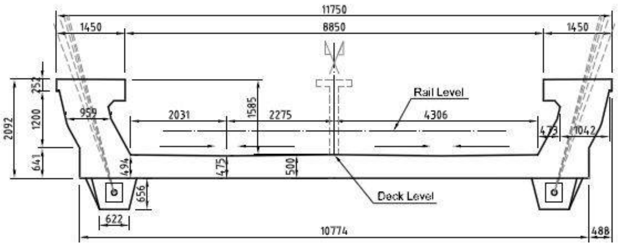
Fig. 5 : ½ plan view with stay cable disposition

The stay cables anchorages are placed under the webs. For tensioning the accessibility is obvious from the street but implies installing scaffolding with a great impact on the traffic. So the design was driven to minimize during the bridge life the stay cable tension adjustments, expecting (based on a common behaviour of concrete) time period of 1, 3, 5 and 10 years after the first adjustment at the operation beginning to take into account the long term effects.

During construction tensioning is realized from the temporary moving scaffolding suspended on the already build part.

The thick webs are 2,09 m deep. Their thickness varies from 0.615m on pylon pier to 0.88m on standard zone and finally 0.47m on abutments. This last layout is to be close from the little u shape cross section. On the pylon pier, the web decrease is only a design consideration to limit the vertical angles of the stay cables to remain closely in a plane (see fig 7).

The bottom slab is 50 cm thick and 10.774m wide on standard zone including the webs (see fig 8 and 9).



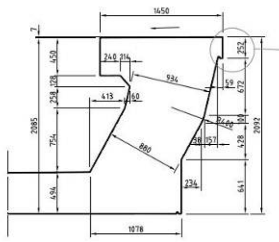
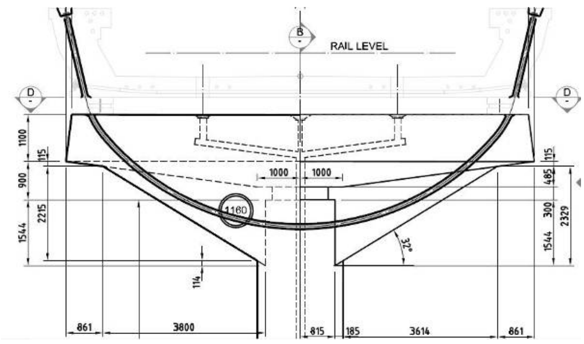


Fig. 6 : Standard cross section

Fig. 7 : Web detail

The circular pier shafts are generally founded on square pile cap supported by 4 piles diameter 1.5m. The bed rock is a volcanic breccia around 15m deep.

The anti-up lift device is realised with external tendons disposed transversally in the lateral piers as shown in the fig. 10 and 11 below, where bearing supports are elastomeric. These tendons work as external tendons. They can be removed and replaced during the bridge life.



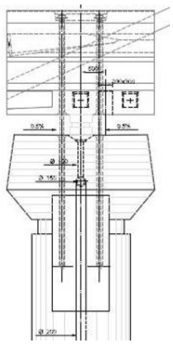


Fig. 8 : Anti uplift device on lateral piers

Fig. 9 : Longitudinal view

2 BEHAVIOUR OF THE BRIDGE

WEH Bridge shows behaviour close to a classical stay cable bridge, far from an extra-dossed bridge, due to the stiff and high pylons. It is confirmed through literature parameters given in [4]. As the scrapping stiffness in the saddles is significant the pylon stiffness filters a part of loads in the main span coming from the stay cables. The deck keeps the other part from stays which loads the pot-bearings on pylon pier-caps. The pylon and its pier cap are significantly loaded during operation and also during significant event as earthquake. Bearings supports and pylon embedment on pier-cap bear significant high global loads, concomitant shear force, torsion and bending moments.

In our design the constraints of the median impacted the design of the pylon piers. A wide pier cap was realised supporting lateral legs. Due to the impact of loads described above, the stability of this element was realized with a transversal prestressing installed in the top cross girder; this one is supported by lateral triangular walls each side of the circular pier shaft of 3.5m wide.

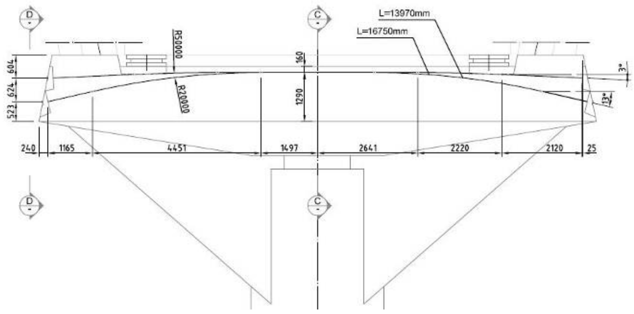
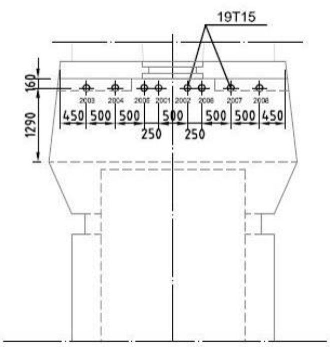


Fig. 10 : Pylon pier cap & deck construction

Fig. 11 : Prestressed cross girder

This cross girder supports 8 cables of 19T15. There are positioned at the top cover in the middle of the pier cap, and avoid the recess for the maintenance jacks near the legs, and also the bottom bolts of the pot bearing embedment which support the deck.



As the long welded rail is used on this metro line, the maximum expansion length governs the layout of fixed point. In this case they are placed on the pylon piercaps, which renders the main span sensitive to imposed strains like thermal strain, shrinkage and creep. The main impact is in the pylon piers supporting the portal effect.

Due to the convergence of deck loadings and internal effect in the structure the reinforcement of pier is significant and more than 3% rate of reinforcement is obtained.

As Mumbai is in zone 3 for the Indian seismic standard IS 1893 [2] the effect is significant in these supports but also in the pylons.

Fig. 12 : Cross section on pier cap

The confinement reinforcement for seismic event was obtained in applying AASHTO standards [3], used to complete the indian standards. It was installed at the bottom of pier shaft with a little part introduced in the pile caps. The same was installed at top of piles.

To stiffen the lateral spans, il was necessary to provide an anti-uplift support. In this aim, a device was instaled in the piers in the lateral spans. This device, as presented upon (fig 10), works to avoid any uplift under live load during operation and also during the ultimate state. This disposition provide a sustainable behavior of this design.

3 DETAILED CALCULATION

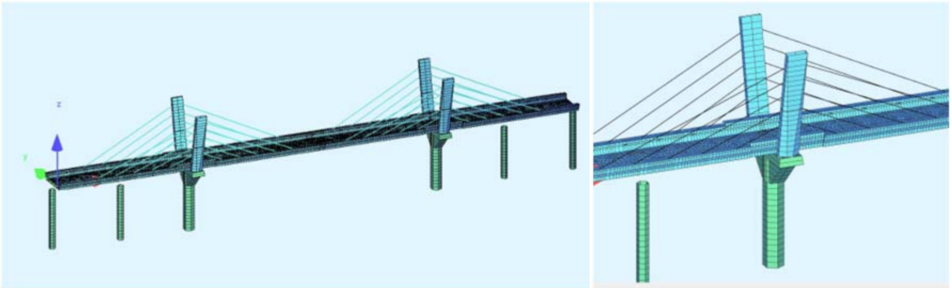
Due to the complexity of this bridge (construction stages, U shape deck) powerful software were used to study all the structure. The main was Sofistik software with its skills in non-linear analysis. The whole structural checks are in accordance with the Indian Standards.

An hybrid model (see fig. 15) with beam elements for the webs and plate elements for the slab was used to create the U-section of the deck. The behavior of this model was checked with a counter calculation on a 3D beam model with ST1 and on a FE model with Robot

The primary objective of the SOFiSTiK calculation was to find stays tensions and to validate the prestressing to ensure that forces in the deck under construction and combinations SLS / ULS meet the stress and strains limits of the standard IRC CBC 97.

The second objective was to validate the device stability of the temporary towers under construction (bents with anti-lift cables) which requested a more refined modelling of non-linear springs with contacts law.

The prestressing and the stay cables were introduced with their behaviour laws. The concrete followed CEB model for creep and shrinkage effects. The duration of each stage was defined with all the process of bridge construction including activation or not of bents and its vertical anti up-lift cables. The Sofistik model is shown below



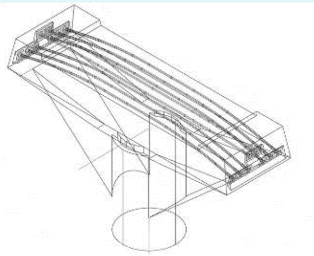
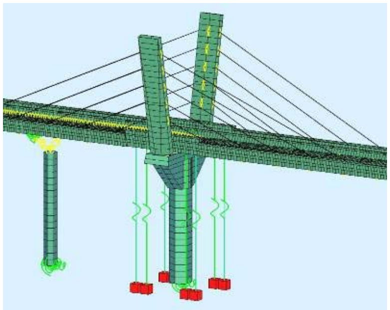


Fig. 13 : Sofistik model

Fig. 14 : Pylon pier cap

The pylon concrete class is M55 as for the deck. For the pier shaft M45 was retained. It was due to the significant loads applied on the pier cap (see fig 16) where the 19T15 are installed.
The pylon is an element with a variable geometry described precisely because any error leads to bad behaviour of the stay cable.
The deviation of the saddles and eccentricities were modelled by rigid elements connecting the tower and stay cables to model no movement between the cables and the saddles.
The stay cables are modelled with “cable elements” and receive their tension during the different phases of construction.



During the first stages of construction, the lateral bents with their vertical active cables (7T15 tensioned at 136t) are necessary. So their behaviour needed to be installed in the model. The temporary towers were modelled with vertical springs of stiffness 13 770 t / ml and they follow a force / deformation law with a contact law to avoid negative reaction. If any uplift only the cables 7T15 participate fixing the deck on the bearings. The stiffness was checked during the works.

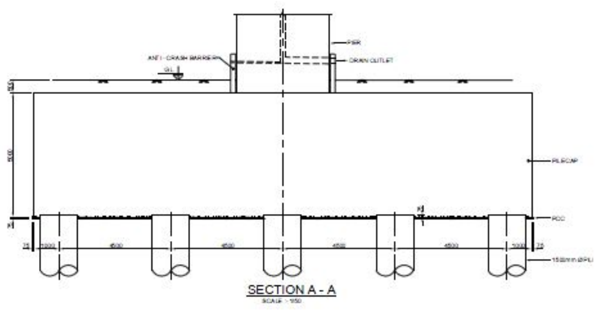
Fig. 15 : Bents in sofistik model

For the seismic analysis the model included the whole structure elements (wells, foundations, piers, deck, pylons and cable stays).
This model was completed with a specific finite element model used to create the U-section of the deck to calculate the transverse flexure of the bridge.

The limit crack condition imposed for the sustainability of the bridge needs a detailed analysis under live loads to check the sagging. To conclude transversal tendons of 4T15 were installed regularly along the deck. They are tensioned after pouring of each segment.
The specific design of pylon pier cap imposed also to realize a particular analysis. For that a strut and tie model was used. It permits to associate an elastic analysis for operation to the ultimate approach with strut and tie to find the reinforcement needs.
The anti-uplift device on lateral piers was studied integrating the effect of fatigue to assure its sustainability. For that French Recommendations for Stay Cables were used [5]. This fatigue occurs with the local strain created by rotation on support under live loads completed by displacement of the deck which can slide on this support under horizontal effect concomitant with the live loads. The recommendations permit the evaluation of the stress state of the cables with estimated traffic loads and the justification with the criteria of $\Delta\sigma$= 80 MPa under 2 million cycles.

4 WORKS

The works realized by SEW, an Indian contractor with a significant experience in a joint venture with VSL began in 2009 with the foundations. To limit impact of the works on the trafic in this congested area the square pile cap under one of pylons was changed in a rectangular thick pile cap 5m high instead 3.5m, 20m long instead 11m, 6.5m wide instead 11m (see fig. 18 and 19). The strut and tie model was used for the justifications. It limited constraints on undergrounded utilities.



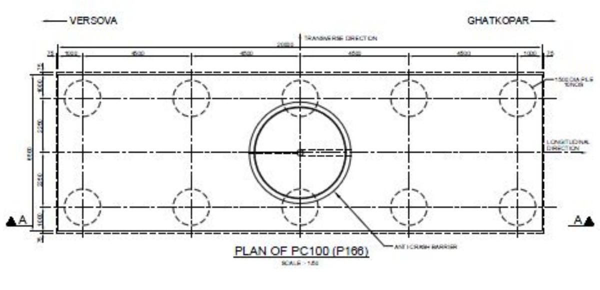


Fig. 16 : Pier cap of P166 under pylon ***Fig. 17 : Plane view of the same***

During works as the quantity of concrete was significant, it was decided to refresh during pouring to limit the concrete heat to 65°C. A water cycle was installed in the pile cap with pipes. But the record of the temperature variation in the concrete showed that it was not necessary and the contractor decided to remove this disposition in the second pile cap. No problem was observed during the completion of the other pile caps.

The pier cap of pylons with two legs offered a significant difficulty due to the quantity of concrete,

the high rate of reinforcement, introduction of few transverse prestressing cables. To complicate more this state, the installation of fixed pot bearings with dispositions to create a fixed point for seismic event, or for breaking and acceleration loads at the top of pier cap, obliged the contractor to work slowly with a great awareness in all stages of pouring, and prestressing.

An important scaffolding was at first installed for the pile cap with few stages for the installation: reinforcement, ducts for the cables, formwork, and so on (see fig. 20).

Fig. 18 : Scaffolding for pier cap

After the realisation of the pier cap and the pouring of the pylons with its incorporated saddles, the works of the deck began with the installation of the lateral scaffolding and temporary tie down as we see in fig. 21: the form traveler is in progress, and the bents with the temporary tie down including an active stay cable to assure the stability of the cantilever construction.

On the mast, VSL saddles were installed with a temporary device to assure a perfect position.

Fig. 19 : Bents and form traveler

The form traveler is a typical under-form which was specially created by VSL to let free the deck for construction.

For the tensioning of stay cables, the form traveler slides behind to let free the anchorage blocks. It is necessary to decrease the bending moment weight effect to avoid significant cracks during construction. The different stages of stay cables installation are usual, showed below (fig 22 to 24).

Fig. 20 : Stay anchorage ***Fig. 21 : Strand in saddle*** ***Fig. 22 : Installation***

The cantilever method is used, and after the third stay cable the form traveler crosses the anti up-lift pier which follows a specific stage of construction to let the sliding of the traveler. This sequence implies another wide scaffolding.

The over crossing of WEH is in progress with security disposition on the highway to protect the form traveler from the traffic. The recent stage was the finalizing of the anti-uplift pier with its pier cap and vertical external prestressing (see fig. 10).

The over crossing of WEH is a technical stage with significant risks. The constractor must check all dispositions of equilibirum during cantilever construction stages and also the safety dispositions on the highway. This challenge is well managed by all the actors in this construction (see fig 25 and 26).

Fig. 25 : Scaffolding for the anti-uplift pier

Fig. 26 : Form travelers above WEH

5 CONCLUSIONS

The construction of Metro line 1 in Mumbai met significant difficulties during design and construction to adapt the pier layout and technical proposals to a very crowded and congested zone with heavy traffic. It was the same for this special bridge designed as a signature bridge in Mumbai, where these difficulties were completed by a complex design, which assumed the first design layout and after, during construction, contractor’s adaptations and methods arrangements. The designers did not account their time to adjust, to verify and to adapt elements to the met constraints. Also the contractors were obliged to propose and to study other details to implement their proposals. During these last stages, the deflection survey was checked by site team and the checker on site with their counter calculation.

6 LITERATURE

[1] VSL- SSI 2000 – STAY CABLE SYSTEM – THE VSL SADDLE – page 12

[2] IS-1893-(part 1) – 2002 – Criteria for Earthquake Resistant Design of Structures – Map of India Showing Seismic Zones – page 5

[3] AASHTO LRFD 2002 – Division 1A section 6 – Design Requirements for Bridges in Seismic Performance Categroy B – pages 459 to 463

[4] Extradossed Bridges In Japan by Atsuo Ogawa and Akio Kasuga, in FIP notes 1994 – pages 11 to 15

[5] SETRA – LCPC – Recommandations of French Interministerial Commission on Prestressing – June 2002 – § 14.2 p. 182 for fatigue, and § 6.2.3 p. 60 for evaluation of bending moment due to imposed rotation.