e-mosty

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Photos on the Front and Back Covers: Chenab Bridge, India   Credit: Afcons Infrastructure Limited, India

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Dear Readers

This issue of the e-mosty magazine is dedicated to the ***Chenab Bridge in India.*** *It is the 1st part of the series dedicated to this bridge.*

The arch bridge across the river Chenab is part of a 272 km long Railway Line from Udhampur to Baramulla joining the Kashmir valley with the Indian Railways network called the Udhampur-Srinagar-Baramulla Rail Link Project (USBRL).

The Chenab Bridge has a central span of 467 m and it is built at a height of 359 m from the bed level. This is the highest railway bridge in the world being constructed to date.

The presentation in this e-mosty edition comprises five articles describing the project and its background, design and construction of the bridge – incremental launching and arch construction. It also comprises drawings and a construction photo gallery.

In the e-BrIM magazine in May and in the e-mosty 2nd special edition in June, we will focus on BIM, Wind Engineering, various technical aspects of the construction, operation and maintenance, and other topics connected to the design and construction of the Chenab Bridge.

If you do not want to miss them, you can subscribe. It is free of charge – the magazines are Open Access. We will notify you of the newly released edition.

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I would like to thank ***Richard Cooke*** *for the review and assistance with the content;* ***Kilian Karius*** *(LAP) for valuable advice and assistance, and* ***all authors, people and companies*** *that have been helping me to put the content together.*

We also ***thank our partners*** *for their continuous support.*

We have established cooperation with ***NBI - Nowoczesne Budownictwo Inżynieryjne*** *(Modern Building Engineering) which is a nationwide branch magazine in Poland. The magazine is printed, released bimonthly, with a circulation of 10 000 copies/issues. Selected articles that have been issued in e-mosty & e-BrIM magazines are translated into Polish by Professor Marek Salamak, Kamil Korus and their colleagues at the Silesian University of Technology, and published in NBI. With the kind permission of the authors, three articles have already been shared this way:*

721 Sky Bridge | *Padma Bridge* | *Cyarera Bridge*

Thank you all very much for your cooperation.

The next e-mosty magazine will be released on 20th June 2023, and the next e-BrIM magazine will be released on 20th May 2023. We are also preparing a special edition of the e-maritime magazine, www.e-maritime.cz that will be released on 30th May. It will comprise one article originally intended as a book chapter; it will bring lessons learnt from bridge caisson foundations in Türkyie and their application to offshore wind projects.

Magdaléna Sobotková

Chief Editor

INTERNATIONAL ONLINE PEER-REVIEWED MAGAZINE
ABOUT BRIDGES

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e-BrIM

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20 February, 20 May and 20 October.
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CHENAB BRIDGE PROJECT

Umesh Koul, Manager - Planning & Monitoring
Afcons Infrastructure Limited, India

Figure 1: View of the complete Chenab Bridge

1. INTRODUCTION

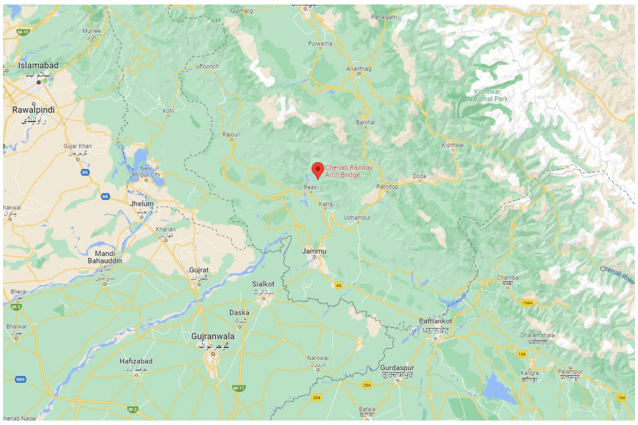
To provide an alternative and reliable transportation system to Jammu and Kashmir, the Government of India planned a 272 km long Railway Line from Udhampur to Baramulla joining the Kashmir valley with the Indian Railways network called the Udhampur-Srinagar-Baramulla Rail Link Project (USBRL).

This project has been the most challenging works undertaken since 1947 by Indian Railways. In view of the importance of the USBRL project in providing seamless and hassle-free connectivity, the Project was declared a "National Project" in 2002.

The alignment of USBRL involves the construction of a large number of Tunnels and Bridges in highly rugged and mountainous terrain with the most difficult and complex Himalayan geology.

These bridges include the iconic Chenab Bridge Project which is being constructed in the Reasi district of the Union territory of Jammu and Kashmir.

The bridge is about 111 km by Road from Jammu on the ongoing Katra-Banihal section, see Figures 2 and 3.



Figures 2 and 3: Chenab Bridge location on the map. Source: google maps

The bridge across the river Chenab having a central span of 467 m is being built at a height of 359 m from the bed level. For comparison, the height of the Qutab Minar Monument in New Dehli, India, is 72 m and of the Eiffel Tower in Paris, France, is 324 m.

This is the highest railway bridge in the world being constructed to date. For the construction of the Arch portion of the bridge over the river, a novel method of construction using the crossbar cable crane was designed and commissioned.

Two cross bars having a capacity of 20 MT (metric tonnes) each and 36 MT in tandem run on 54mm cables laid across the river valley and are connected through a 127m high pylon (tower) on the Kauri side and 105 m on the Bakkal side of the river.

The Chenab Bridge will usher in a fresh era in Jammu and Kashmir thanks to increased employment opportunities for the young population, improved infrastructure by virtue of the construction of access roads, and better facilities for students to travel to other parts of the country for educational purposes.

It will also boost the tourist industry, connectivity to distant areas to the mainstream of the country and overall economic development of the state.

Remotely located villagers at the Kauri and Bakkal ends who until now have no vehicular means to travel to the Reasi district and other places have started enjoying the fruits of development brought about by the construction of black-topped approach roads in the region.

In fact, a window to a world full of opportunities has opened up to the local population of the region.

2. KEY STAKEHOLDERS

Client	Northern Railway and Konkan Railway Corporation Limited, India
Contractor	Chenab Bridge Project Undertaking U/o Afcons Infrastructure Limited, India
Design Consultant	WSP Finland and Leonhardt, Andrä und Partner, Germany
Proof Consultant	Scott Wilson Kirkpatrick & Co. Ltd, UK (now AECOM) Flint & Neill Partnership, UK (now COWI)
Slope Stabilization	IISC, Bangalore IIT, Delhi ITASCA (USA)
Construction Engineering	Afcons Infrastructure Limited, India
Cross bar cranes	SEIK, Italy

3. PROJECT SCOPE

Summary of principal quantities:

Excavations	1,002,658 m^3
Concreting	70,193 m^3
Shotcreting	76,280 m^2
Rock Bolting	66,684 m
Fabrication and Erection of Structural Steel	31,062 MT (metric tonnes)
Installation of HSFG Bolts	306,312 pieces

4. UNIQUE FEATURES OF THIS BRIDGE

- Bridge designed to withstand maximum wind speeds of up to 266 km/h (74 m/s).
- Bridge designed for blast load in consultation with DRDO (Defence Research and Development Organisation) for the first time in India.
- Bridge designed to resist earthquake forces of highest intensity zone-V in India.
- First time on Indian Railways, the Phased Array Ultrasonic Testing machine used for testing of welds.
- First time on Indian Railways, National Accreditation Board for Testing and Calibration.
- Laboratories (NABL) accredited lab established at site for weld testing.
- Provision of long welded rail (LWR) over the bridges and resulting force calculation as per UIC – 774-3R (Code for Track Rail Interaction) guidelines.
- Design life of 120 years.
- Incremental launching of deck structure on combined circular and transition curves was done for the first time in the Indian Railway.
- World's largest capacity crossbar cable crane used for the erection of piers, trestles and arch segments.
- Extensive wind tunnel testing - Topographic Model Study, Section Model Study and Aero-elastic Model Study for the first time in India.
- Installation of the double corrosion-protected bar and cable anchors for the first time in India in Indian Railway.
- 10.9 grade M36 HSFG Bolts with Geomet Coating.
- Blast Resistant Design.

5. SPECIAL CONSIDERATIONS IN DESIGN

Geographical Location: (In the vicinity of snow-clad Himalayas Mountains)

- The Bridge is designed for temperature -10^0 C to 40^o C
- The adoption of special structural steel, e.g., E410C, E410C+Z15, E410C+Z25, E410C+Z35
- Seismically highly active area - Zone V – Site Specific Response Spectrum, Dynamic Analysis of Structure and Ductile Detailing

Geological Condition (Unstable slopes and Erratic Geology)

- Very detailed Geotechnical and Geological Investigations
- Extensive slope stability analyses

High Wind Speeds

- Stopping of trains if wind speeds exceed 90 km/h (25 m/s)
- Design based on wind tunnel tests

Sensitive Border Area

- Even after the removal of one pier, the bridge will not collapse under self-weight.
- Redundancy: Even after removal of one critical member of the arch, the bridge will still be able to carry the traffic at a restricted speed.

  Even after removal of one critical member of a pier and arch, the bridge will still be able to carry the traffic at a restricted speed.

6. GEOLOGY OF THE AREA

The State of Jammu and Kashmir is one of the largest Union Territories of the Indian Union and is situated at the base of the Himalayas. It lies between 32° 15' to 37° 05' latitude north and 72° 35' to 80° 20' longitude east.

Jammu and Kashmir are home to several valleys such as the Kashmir Valley, Tawi Valley, Chenab Valley, Poonch Valley, Sind Valley and Lidder Valley. The main Kashmir valley is 100 km wide and 15,520 km^2 in area.

The Himalayas divide the Kashmir valley from Ladakh while the Pir Panjal range, which encloses the valley from the west and the south, separates it from the Great Plains of northern India.

Along the NorthEastern flank of the Valley runs the main range of the Himalayas. This densely settled and beautiful valley has an average height of 1,850 m above sea level, but the surrounding Pir Panjal range has an average elevation of 5,000 m.

The Reasi thrust separates the youngest Siwalik rocks from the overlaying oldest rocks.

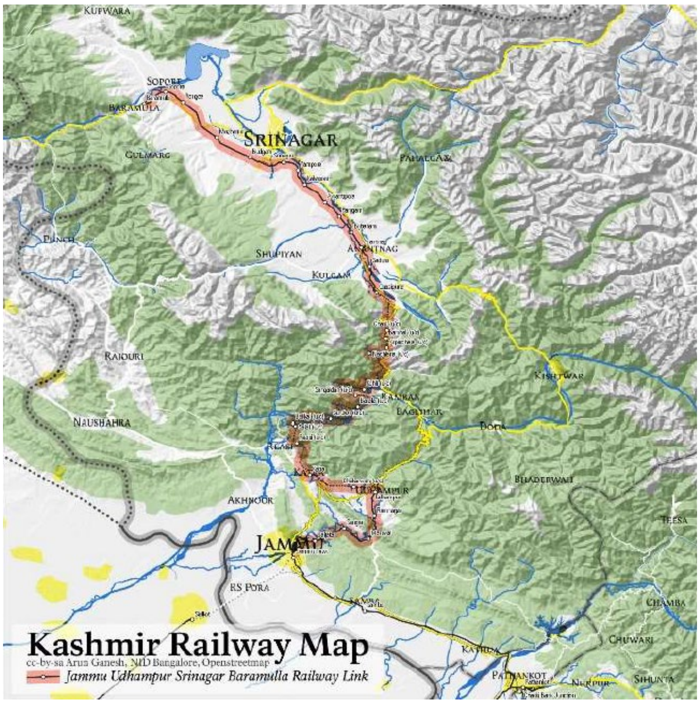


Figure 4: Chenab Bridge location within the USBRL Alignment

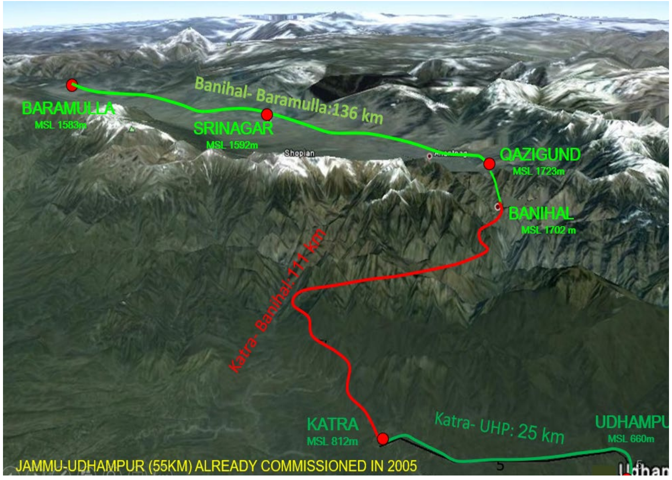


Figure 5: Alignment of the railway line

The railway line passes through the Himalayan Mountains i.e., outer Himalaya (sub-Himalaya), lesser Himalaya and great Himalaya, see Figures 4 and 5.

The Chenab Bridge is located at Chainage km 51.800 between Bakkal village (left bank) and Kauri village (right bank).

Geology along the Chenab Bridge from the left to right bank consists of three formations, namely Sirban limestone group, Jangalgali/Kheri formation and Subathu formation.

The slope stabilization measures for the Chenab Bridge are based on geological logging and using DIPS software for kinematic analysis and SWEDGE software for wedge failure analysis.

For the first time in India Railways, 33.5m long double corrosion-protected pre-stressed Dywidag bars anchors were installed for slope stabilization work on the right bank.

The major rock types encountered and exposed are Jointed dolomite, Brecciated dolomite, and Cherty Dolomite with different degrees of weathering and fracturing.

Besides dolomite near the S-70 foundation, quartzite with a shale band was also observed at the Kauri end.

The dolomite encountered is mainly dark grey to grey in colour. It is mostly weathered on the surface.

Figure 6: The area is highly undulating and with rugged topography

Figure 7: Chenab River Canyon

In some places, it occurs as prominent warping at different slopes.

The dolomite falls under Sirban series of dolomite. It is fresh, hard and competent. The rock units have undergone tectonic movement and local folding/warping was noticed.

Rock unit generally strikes NW-SE and dip at 30° to 50° towards NE direction.

However, rock units on both sides, i.e., left as well as right banks, are locally folded in the form of warping.

The entire area comprises hilly terrain traversed by numerous small and large nallas. Terrace farming or step-wise farming is done on hill slopes.

The area is highly undulating and with rugged topography, see Figures 6 and 7.

It is characterized by strike ridges, dip slopes with scarps and drainage patterns controlled by bedding joints and other joint planes.

a) Topography on Bakkal Side

The area on the left bank has a maximum elevation of ±845m near Bakkal village. In general, hill slope areas are very steep with slope angles varying between 50º – 60º.

The Bakkal end hill slope is covered with debris/soil, the thickness of which varies from 1 to 3 m but in some places rock exposures are encountered.

Geological mapping of the left abutment has revealed that right from the S10 location to the S40 location i.e., up to R.L. 843.016 m to 671.100 m, the slope is made up of highly jointed to blocky dolomite, brecciated dolomite/brecciated quartzite.

b) Topography on Kauri Side

The area on the right bank has a maximum elevation of ± 850 m near Kauri village. The slopes are very steep with slope angles varying between 50º – 75º.

The Kauri end hill slope is covered with debris/soil and vegetation.

Their thickness varies from 1 m to 3.5 m but in some places rock exposures are encountered.

Geological mapping of the right abutment revealed that right from the S80 location to the S50 location i.e., up to R.L. 833.739 m to 703.749 m, the slope is made up of highly jointed to blocky dolomite, brecciated dolomite/brecciated quartzite, quartzite and shale.

7. ESTABLISHMENT AND LOGISTICS AT SITE

Managing major construction projects like the Chenab Bridge requires an integrated approach to logistics with respect to the mobilization and establishment of workshops for the execution of work at site.

Sophisticated workshops were developed at the Bakkal and Kauri sides for carrying out the fabrication of deck segments, arch segments and pier segments.

These workshops were equipped with sophisticated CNC Cutting M/C, CNC Drilling M/C and welding equipment.

Internal approach roads 3 – 4 km long were constructed for the mobilization of material, tools, and equipment which was one of the biggest challenges due to the hilly terrain, narrow and sharp curves, and with land sliding-prone areas though out the stretch, see Figures 8 and 9.

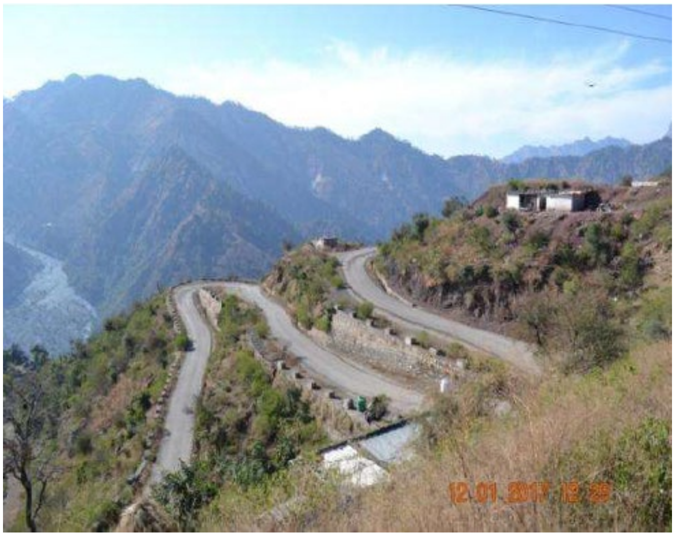



Figure 8 and 9: Challenging conditions for transport

The plates and the outsourced segments for trestles and the arch were planned to be stacked at the central yard of Reasi, 35 km away from the site, as the 12m trailers on which these segments were being transported could not travel directly to the site location due to turns/sharp curves and with insufficient width for the movement of such trailers.

The material from the central yard was then transported to the site using 9,8m trailers – all material was supplied to the site by these trailers only.

Other Challenges Faced During Logistics

There were many other challenges encountered and some of them are as follows:

a) Land Slides: Logistics of the material, workforce and equipment through these hilly terrain roads during the heavy rains was very difficult as there were always slides of unstable rocks from the mountains. The vehicles were often delayed for many days during clearance of the road, Figure 10.

b) Remote Location of the Site: Due to the remote location of the site and the bad condition of roads, logistics of material, workforce and equipment could not be carried out at night which impacted the pace of the project. A fleet of trailers was escorted from the Reasi yard to the site location. Supervision of road conditions was monitored prior to the actual movement of trailers.

c) Widening of Roads: The transportation of the segments on the other side of Chenab valley was obstructed because there was insufficient width for the transportation of the segments on the Bailey bridge constructed by the BRO (Border Roads Organisation). The site management took the decision to change the alignment of the road by constructing a side-by-road to the existing bridge for the transportation of segments.

Figure 10: One of the slides after heavy rains

Challenges faced during the Erection

a) Dependency on Cable crane: The cable crane was a critical construction machine and the only source for erection activity in this project due to its location and height. However, cable crane operations could be affected by heavy rains, gale-force winds, thunderstorms, and lightning which affected the arch erection productivity. Therefore, detailed planning and time-bound activity were of utmost importance.

b) Alignment of Erected Segments: Alignment of arch segments was very important for controlling the geometry of the arch. This involved meticulous and regular surveys during the erection of arch segments. Temperature and wind monitoring were crucial during the erection and survey. Surveys were done early in the morning to avoid temperature variation. Arch erection was stopped if wind speed exceeded 15m/s. Temperature played a crucial role during the erection.

c) Trial Assembly of Arch Segments: Before the arch erection, the geometry at ground level was checked so that any modification or alteration could be done on the ground before erection, see Figure 11. It could have been extremely difficult and risky to rectify errors at such precarious heights after erection.

d) Long Wind Bracings of 40m in length: Transportation and lifting of wind bracings to the erection location was a challenging task due to their length and weight, see Figure 12. The uneven mountainous terrain made transportation of the wind bracings extremely difficult. Before the erection of wind bracings, platforms were provided at the location to ease the erection with proper safety measures. Pre-assembly was done with the required degree of inclination to erect as per the required geometry. Accordingly, arrangements were being made for the lifting of wind bracings. After erection, in-situ welding was done at the required locations which was a difficult activity under windy conditions.

e) Torquing of HSFG Bolts: Torquing plays a vital role in the erection of the arch. Shifting the equipment for torquing at such height and location is challenging and consumes time. Working at height is very risky and needs specialized teams and platforms with safety measures in place. It is impossible to retrieve any HSFG bolt if it, unfortunately, slips and falls into the river from such height.

Figure 11: Trial assembly of the arch on the ground

Figure 12: Transportation of wind bracings

Figure 13: HSFG Bolt

CONCLUSION

The Jammu-Udhampur-Katra-Banihal-Srinagar-Baramulla Rail project is a vital national project which has a major bearing on national security and nation building. It will help in the integration of Jammu and Kashmir with the rest of the country, will help strategically in many ways and will help to generate the job opportunities for the local population.

GENERAL CONCEPT AND DESIGN OF THE CHENAB BRIDGE

Pekka Pulkkinen, WSP Finland Ltd.

1. BACKGROUND

It is almost 20 years since the project of building the Chenab Bridge started. Indian Railway Company KONKAN RAILWAY CORPORATION LIMITED organized a tender for Design and Construction of the Chenab Bridge which will be the major bridge of the new railway line from Udhampur to Baramulla in the State of Jammu and Kashmir in India.

History

Indian Contractor Afcons Infrastructure Ltd hired two European engineering companies WSP Finland Ltd. as the main consultant and Leonhardt, Andrä und Partner AG as subconsultant to prepare the tender design.

The new railway will pass tunnels and bridges constructed in difficult Himalayan mountains. The crossing of the Chenab River between Bakkal and Kauri is the most challenging part of the project. The height from the river to the bridge deck is more than 350 m which leads to the huge span of the main crossing.

In the tender, an arch bridge type was preferred by the Railway Company.

Consequently, the design team prepared several arch options in order to find the most efficient arch concept with a reasonable span arrangement.

Although the arch bridge option was preferred the design team studied a concept of a stay cable bridge type as well.

Finally, a steel arch bridge with a main span of 460 m was chosen for tender.

At the tender evaluation stage, there were meetings between the Client, Contractor and Consultant where critical design criteria and details were discussed. Issues dealt with included the use of steel pipes in the arch structure, design parameters, welding conditions at site, etc.

Finally, the Client accepted the offer made by Afcons Infrastructure, and the design contract was signed between the Main Consultant and Contractor in December 2004.

The design and construction started immediately after signing the contract. At first, the consultant started with the design of the Approach Bridge which is a conventional steel girder bridge.

Wind tunnel tests for terrain models and section models of arch and deck cross sections were also carried out in the very early stage. Construction of foundations of the Approach Bridge started.

Figure 1: The first visit to the bridge site in April 2005

Unfortunately, the project was halted in 2008. This was due to the uncertainty of basic data from the bridge site and the new railway line.

The project restarted in 2010, when the construction of the foundations of the Approach Bridge and slope excavations in the main gorge continued.

The bridge concept was evaluated and updated regarding the span length of the arch span, the erection method of the arch and the number of technical design parameters approved by the Indian Railway Authorities.

The main span was lengthened from 460 meters to 467 m which was caused by new contour and soil investigation results from the area at arch foundation S40.

The erection method of the arch was changed to be executed totally by cable crane with a capacity of 34 tonnes.

The erection method of the superstructure in the arch span was also changed. Originally deck segments were planned to be lifted by derrick lift and bolted together.

In the new method, the superstructure was planned to be welded continuously behind the abutments and launched over the steel piers.

After the restart, a comprehensive Appreciation Report about design changes and agreed parameters was prepared and published in 2013. This was the real restart for the design.

2. DESIGN BASIS AND EXCEPTIONAL DESIGN PARAMETERS

Design Basis and Standards

Design Basis was already established by Indian Railways at the tender stage in 2004.

This document included standards, load definitions, load parameters and load combinations, etc.

It is noteworthy that the design method for steel structures was the India Service Load Design Method (Allowable Stress Design) and for concrete structures the Ultimate Limit State Design Method.

After signing the contract, the Design Basis was updated, but it took a long time for the final Design Basis was agreed upon. The final Design Basis was signed in 2010.

In the final design work the National Codes of India, Indian Railway Standards (IRS), Indian Road Congress (IRC) recommendations and Indian Standards (IS) had to be used but also international standards for instance for steel structures British Standards (BS) and even AASHTO and Eurocodes could be used.

Although international codes could be considered, BS:5400 Part 3 – Design of Steel Bridges - was preferred for steel design.

Also fatigue assessment was done as per BS: 5400 Part 10 – Code of Practice for Fatigue.

Figure 2: Photomontage of Arch Bridge in tender

Figure 3: The Arch Bridge fully erected

Structural deformation limits had to be taken from UIC 776-3R – Deformation of Bridges. Ductile detailing of reinforced concrete structures RCC should be done as per Eurocode.

Main Loads

Here are listed the main loads from the Design Basis.

Basic rail loading is as per MBG: 1987 from the Indian Codes (Modified Broad Gauge Loading specification). Dynamic Augment (CDA) was taken from IRS Bridge Rules for the deck and piers, for arch there was no CDA.

The deck was designed for two traffic arrangements: in the beginning, one track will be installed in the middle of the deck, but in the future, the track can be removed and the deck can be furnished for two tracks.

Braking and Acceleration loads were taken from IRS Bridge Rules.

The bridge was designed for seismic loads according to IS 1893, Part 1, 2002 – Criteria for Earthquake Resistant Design of Structures - Zone V and site-specific spectral studies as carried out by IIT, Roorkee.

A challenging 50% of this seismic loading had to be considered in the erection stages.

Because of the high altitude of the deck, complex terrain and anticipated lateral movement of the superstructure caused by wind the following wind tunnel models and tests were carried out in the very early stage:

a) Terrain model
b) Static/aero-elastic section model of the deck and static section model of the arch
c) Full aeroelastic model.

The analytical wind response computation was done by multimode frequency-domain analysis by taking into account the results of the terrain and section model tests.

Static equivalent wind loads were sent to structural designers to use in the design of the in-service bridge and construction stages.

Finally, the analysis and wind resistance of the completed bridge were confirmed with a full aeroelastic model test.

Partial safety factors and load combinations for reinforced concrete structures were taken from the Indian Railway Standard, but for steel structures, these were taken from British Standards slightly modified.

Some Exceptional Load Configurations

The Design Basis included some exceptional load configurations which were basically quite demanding for some structural components.

Blast Load had to be considered in the design. There were two scenarios of blast taking place on the deck or in close proximity to the foundations:

- Blast occurring at ground level at a near distance of the face of the pier/abutment.
- Contact blast occurring at any point on the steel deck with a train running on the centre track of the bridge deck.

No damage to the arch trusses and no collapse for the bridge span under the above scenarios were allowed.

Any damage to the structure has to be repairable so that it can be restored to its original serviceability requirement.

In the deck, this led to the solution where a sacrificial steel net was designed on the deck to ensure minimum damage to the main superstructure.

Another special design requirement was the **Structural Redundancy** of major elements.

In this configuration structural redundancy of structures was assessed by removing critical bridge elements one by one as follows:

- A single element of one of the arch trusses was removed one by one from the structural model.

  The effect of the removal of one chord of the arch truss (one box of eight boxes) or one diagonal member of the arch truss had to be studied.

  One train passing at low speed had to be considered at the time of redundancy.

- The effect of a collapse of piers one by one should be checked using the ULS method.

3. CONCEPT OF CHENAB BRIDGE

In the following section, the bridge concept is explained in brief. More details and background to the design and construction are presented in other articles in this and the next e-mosty publication.

The Chenab Bridge is a steel railway arch bridge with a total length of 1,315 m. It is composed of an Approach Bridge, which is 530 m in length and an Arch Bridge, which is 785 m. The main arch span is 467 m, making it one of the longest arches in the world and probably the longest arch for railway traffic. The deck is 13.5 m wide and it will carry two tracks in its final arrangement.

The superstructure has been constructed 350 m above the surface of the river flowing in the valley.

Challenges

When the bridge concept was developed in the tender stage, we recognized topographical challenges which had to be solved for the bridge construction. The major challenge was the huge river gorge with very deep slopes on both sides.

The railway line was already investigated and designed at the bridge location by the Railway Authorities, so we did not have any scope to change the site location.

All this led to the conclusion that there should be one long span bridge over the river gorge and a separate approach bridge before the main bridge.

Before tender, the Client had already studied bridge options at the site and it was stated they preferred an arch bridge. This led to the decision to design a steel arch with a steel deck to the main bridge.

The erection of long span arch over the gorge was considered to be executed mainly by a huge cable crane and derrick lift, which was also accepted by the Client.

The stability of slopes in earthquake incidents was also a demanding design task. The design of slopes was not in the bridge designer's scope. The Client and Contractor used Indian and international experts for that task.

There were plenty of other challenges as well. The horizontal geometry of the approach bridge is composed of a transition curve, constant radius and straight portions.

Due to poor road conditions, logistics of transportation of materials to the bridge site were a big challenge. This led to the decision that the major portion of steel structures had to be manufactured in workshops at the site. The steel plates and segments could be transported by trucks with a maximum length of 12 m.

The design code definitions in the tender documents were challenging as the Indian Railway Authorities did not have specific requirements and code clauses for long span bridges. Typically spans in railway bridges are less than 100 m.

Also, welding of demanding steel structures in bridges was not common in India, they typically used structures with bolted joints.

The Bridge Concept

The Arch bridge is composed of a 467 m long steel truss arch and steel piers with a steel girder deck. The Arch consists of two inclined arch elements which are made of four steel boxes filled with concrete. In order to improve horizontal stability, the two outer arch elements taper inside. All steel piers are also tapered with the same inclination. This provides a balanced combination of lateral stiffness.

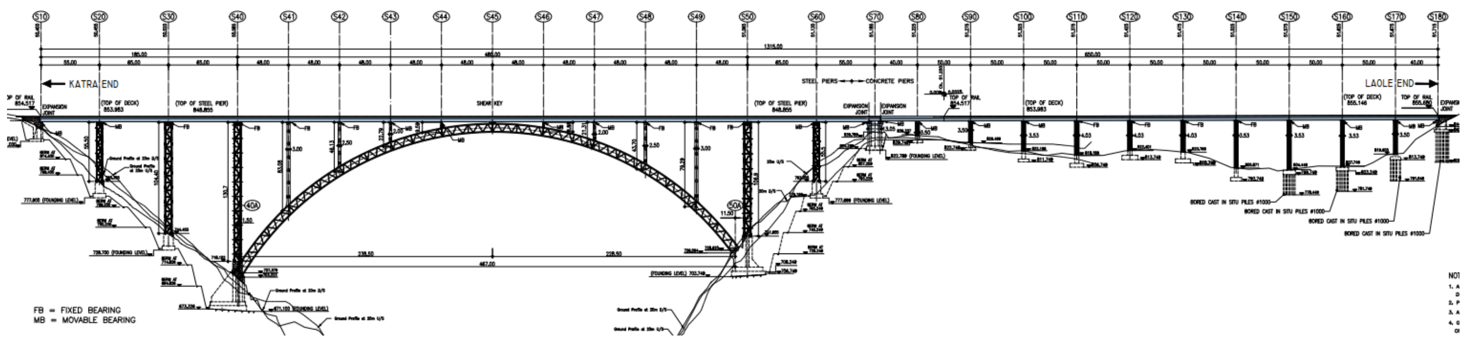


Figure 4: The elevation of final bridge concept. Click on the image to open it in a higher resolution

The major arch elements have been transversely connected to each other using a steel pipe space truss. Arch and steel piers have bolted joints.

Trestle piers over the arch divide the arch into ten 48m long spans in the superstructure. The lengths of the side spans in the arch bridge were optimized structurally and economically taking into account the height of the steel girder and the number of high steel piers.

The bridge is longitudinally fixed at the arch crown by shear keys. Transversely the deck has shear keys at each pier top. Vertically the deck is supported by spherical bearings.

The cross-section in the arch spans has inclined side wind panels. This system was studied in a wind laboratory to improve the wind behaviour of the deck.

At track level, the deck has six longitudinal secondary beams which are supported by cross beams.

At first, the trains operate in the middle only. Later the deck can be furnished with two railway tracks. The superstructure is continuous and welded together.

Arch foundations are huge concrete structures. The height of the left arch support at S40 from the bottom to the pier foot is about 47 m high. It includes several concrete casting segments in stages. All steel piers and arch feet are fixed by grout and stressed bolts to concrete foundations.

In the Approach Bridge, the superstructure is very similar to the main bridge. The deck is 13.5 m wide; in the main bridge, it is 17 m. Also, the depth of 3.8 m is smaller; in the main bridge, the depth is 4.5 m.

The approach bridge has concrete box piers, which was the requirement of the Client. The deck is longitudinally fixed at four middle piers, which are the highest piers. At each pier, transverse shear keys are installed against earthquakes.

Expansion joints are located at end supports S70, S180 and S10. At these joints rail movement joints are also located.

At support S70 where the decks of the Arch Bridge and Approach Bridge are separated, we had to divide the rail movement joint into two parts. This was due to large longitudinal rail movements caused by earthquakes and temperature changes.

The bridge will be equipped with a health monitoring system focusing on railway safety and earthquake control.

A comprehensive Bridge Maintenance Manual has been prepared. The Manual is based on UIC Code 778-4 R (2009) – Defects in Railway Bridges and Procedures for Maintenance - published by the International Union of Railways. It is meant to be read together with BRIDGE MANUAL/Indian Railways 1998.

4. INTERACTION BETWEEN DESIGN AND CONSTRUCTION

At the very beginning of the project, it was understood by all parties that this will be a long-term project which will include big challenges in design and construction.

The design team of the Indian contractor came from Europe. Also, the Independent Checker of design consisted partly of international experts.

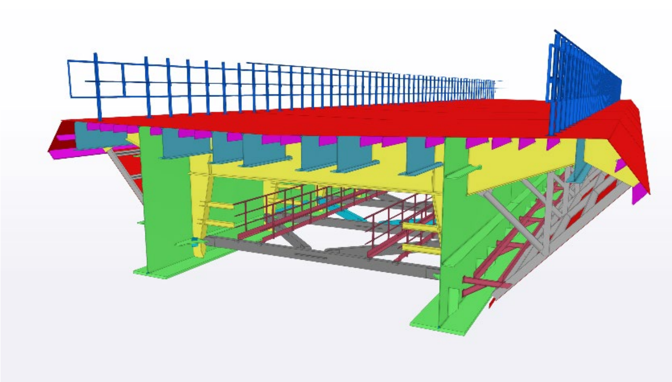


Figure 5: The cross section in main span

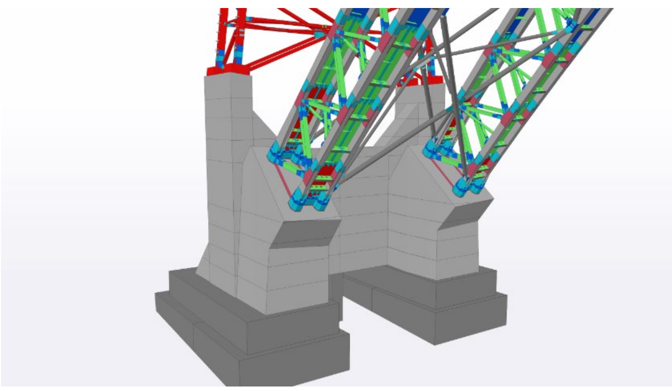


Figure 6: Arch foundation at S40

The Client is a big and important Indian Railway Organisation with a large number of railway experts and specialists.

Cooperation between parties was seen as most important. Face-to-face meetings took place quite regularly, particularly in the early stages of the project.

Digitalization was the major tool in the interaction of design information change. Although the site is located in an underdeveloped area, it was possible to build a working internet communication environment.

This enabled information exchange and utilization of design models and documents in a workshop production, survey, construction and supervision.

In the beginning, the designer established an FTP server for the project. Access was given to the Designers, the Contractor, the Checker and the Client.

All relevant and up-to-date documents were available via the Internet.

The design contract also included preparing workshop drawings. Preparation of all design documents from huge steel arch, superstructure and piers with a total quantity of steel components of 31,000 tonnes was an enormous task.

It would not be possible to do this kind of work without 3D models, which ensured the accuracy of the design of steel components for production.

During the project, the Contractor recruited its own engineers who were able to handle TEKLA models and do small updates and support to workshops at the site.

During construction stage the Designer was supporting the Contractor all the time.

All key designers in their own offices were available. This worked well and the need to visit the site was reduced.

5. CONCLUSIONS

It has been a privilege to have been involved in the design and construction of the Chenab Bridge.

The experience during the years has given me a good perspective in which to express some conclusions.

Figures 7 and 8: Arch in erection stage

The Design Basis is the most important document for designers. In order for a design to proceed smoothly a well prepared and agreed Design Basis is essential.

The possible modifications to the Design Basis should be done and agreed upon with parties at the beginning of the project.

The Design and Build contract method is advantageous for this kind of project. Innovations in construction and design can be developed only in this kind of contract type, where close cooperation between the parties leads to improvements in quality, construction time and cost.

The time schedule agreed upon should take into consideration challenges in design, site circumstances and logistics.

The best way is to prepare at first preliminary design concept with chosen bridge type and construction method.

This work should be done in close cooperation with the Client's organization. At that time the Contractor can mobilize and establish a construction site which always takes a certain amount of time.

After agreed preliminary design solutions a final design can proceed fluently and the design checkers can be made aware of critical details.

It is important that the latest developments in information sharing, 3D modelling and communication are adopted..

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Figure 9: Approach Bridge

Figure 10: Chenab Bridge, almost completed

CHENAB BRIDGE – DRAWINGS

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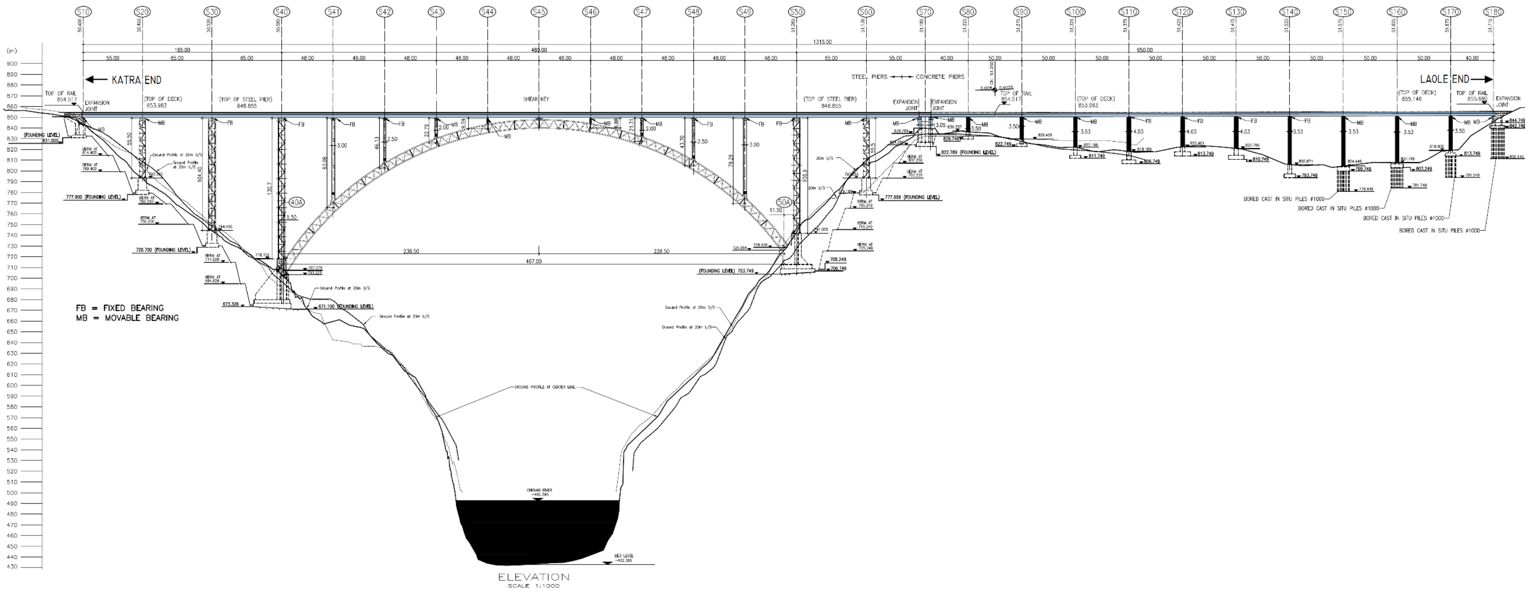

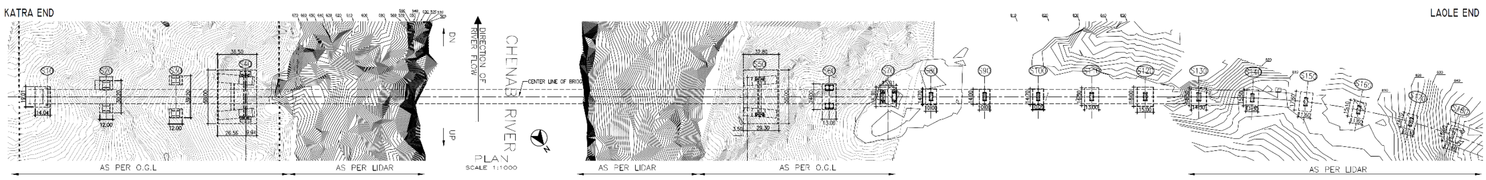

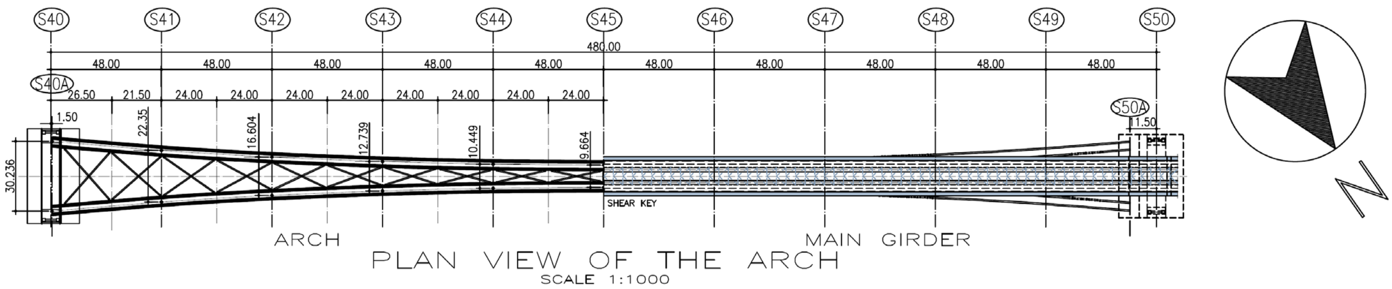

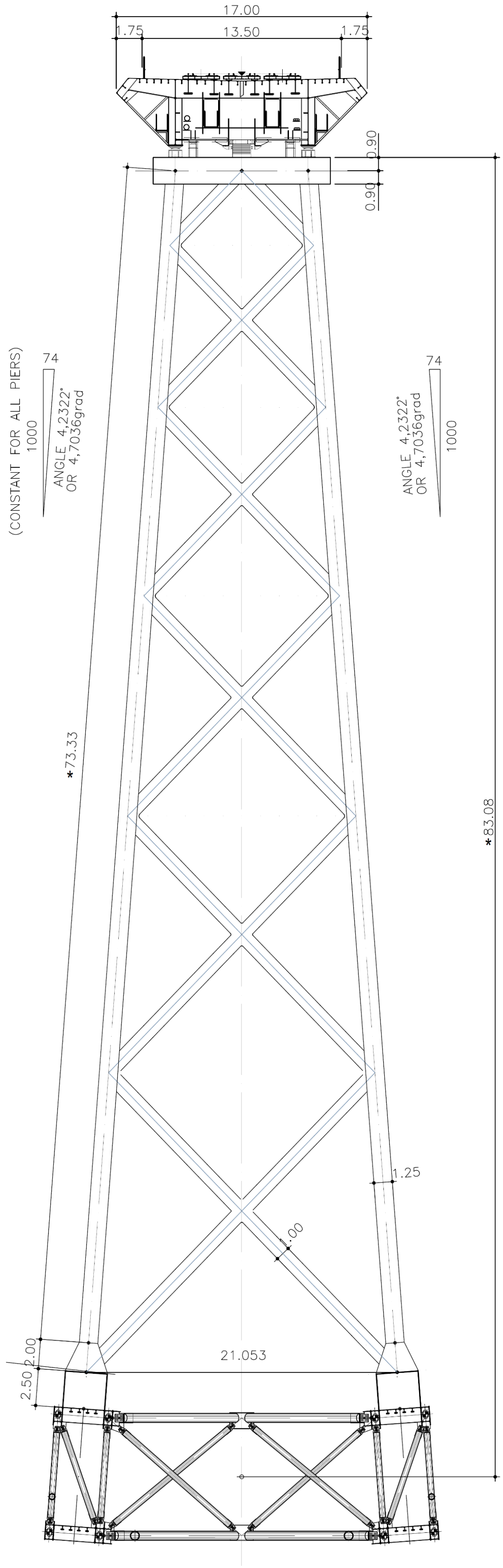
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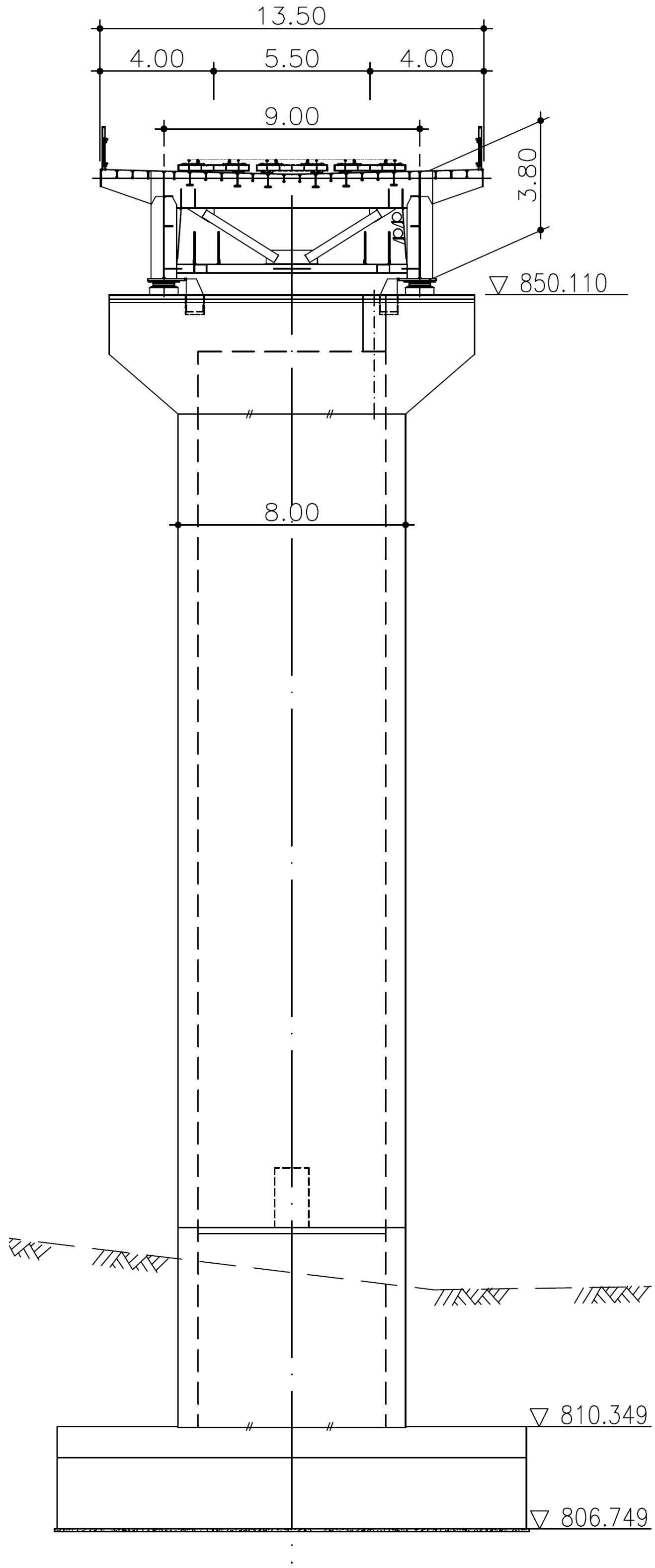
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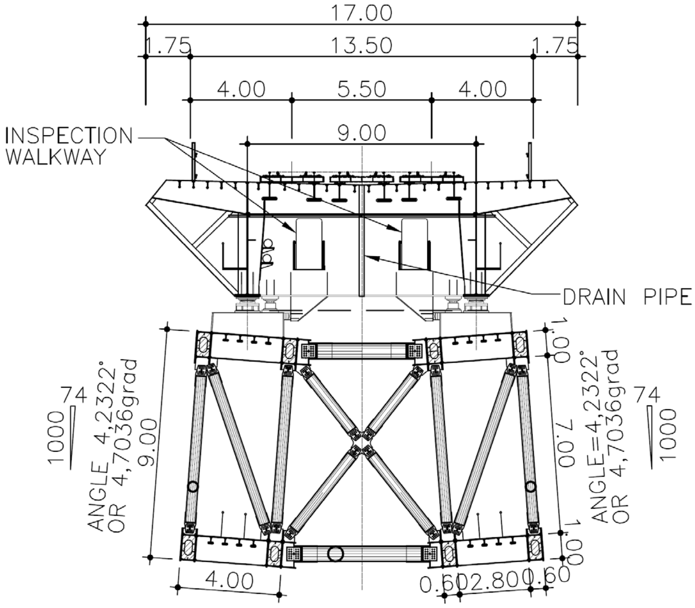
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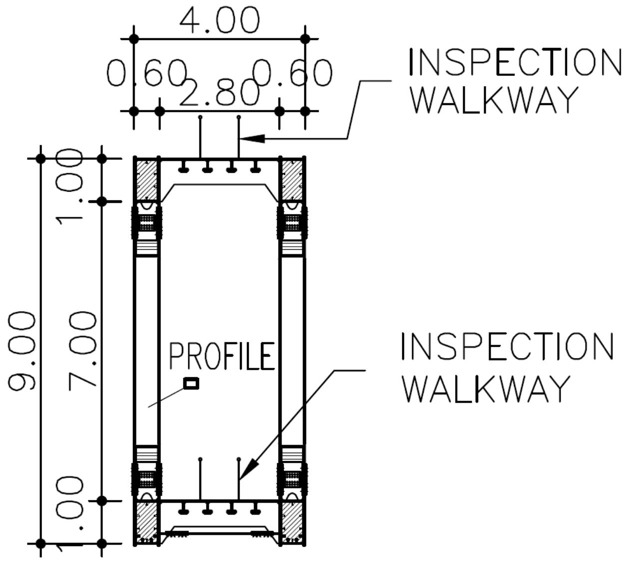
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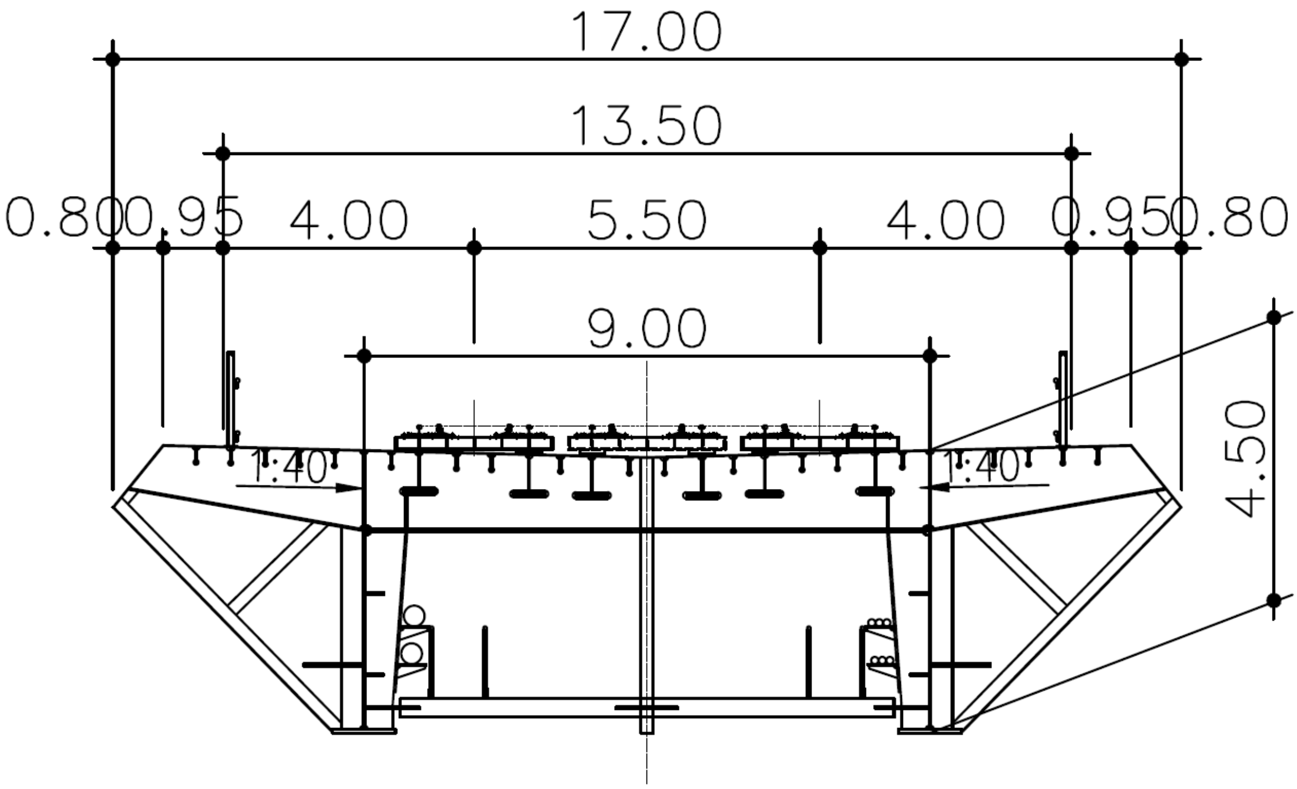
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ANALYSIS AND ENGINEERING OF THE CHENAB BRIDGE

Kilian Karius

Leonhardt, Andrä und Partner Beratende Ingenieure VBI AG, Germany

1. INTRODUCTION

The mighty Chenab Bridge in the Himalayan mountains is about to be commissioned and will soon connect remote regions to the Indian railway network. With an arch span of 467 m, Figure 1, and a height of 359 m above the Chenab River, it is undoubtedly one of the most impressive railway bridges worldwide.

This paper describes the challenges overcome by the design team and in fact by the overall project team since the tender period, a journey that started almost 20 years ago. It focuses on the design and analysis of the arch including the erection methods and construction stage analysis.

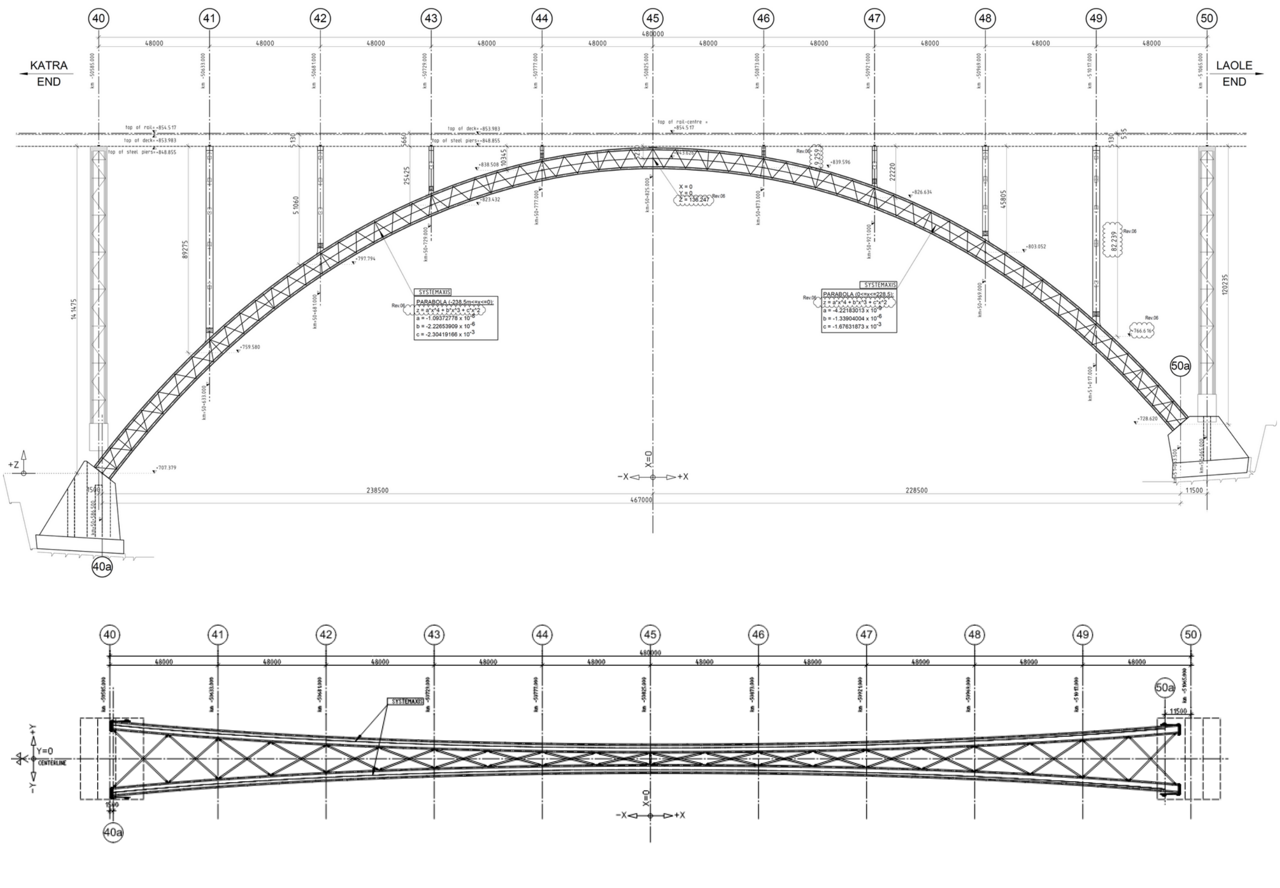


Figure1: General layout of the Arch Bridge. Click on the image to open it in a higher resolution

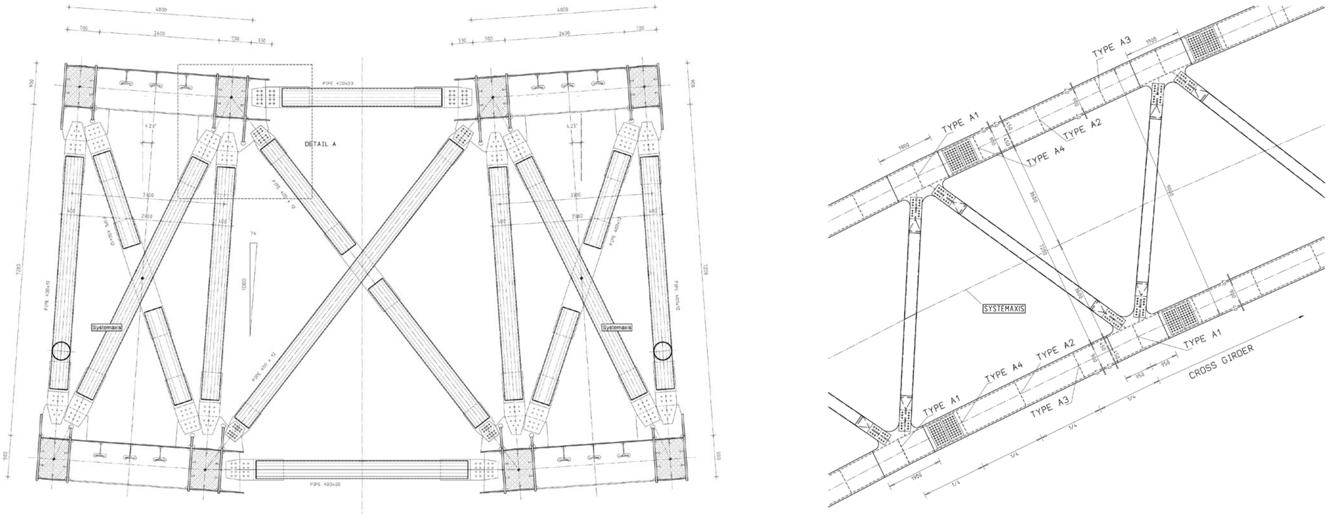


Figure 2: Arch principal composition. Click on the image to open it in a higher resolution

The arch structure itself is a steel truss composed of elements sized to be transportable by a cable crane that spans 915 m between support pylons and provides a payload of 30 tons (at 36-ton capacity on a single hook when operated in tandem).

Arch piers or trestles support the superstructure by standard spherical bearings at 48 m centres. The longitudinal fix point is located at the centre of the arch.

The overall truss is composed of two main arches, each 4 m wide and 9 m high, that are leaning against each other at an inclination of 74/1000 against vertical, Figure 2.

A 3D truss girder is connecting the two arches and bracing the structure against environmental actions from wind and earthquake, Figure 3.

Diagonals connect the top and bottom chords of each arch in its plane.

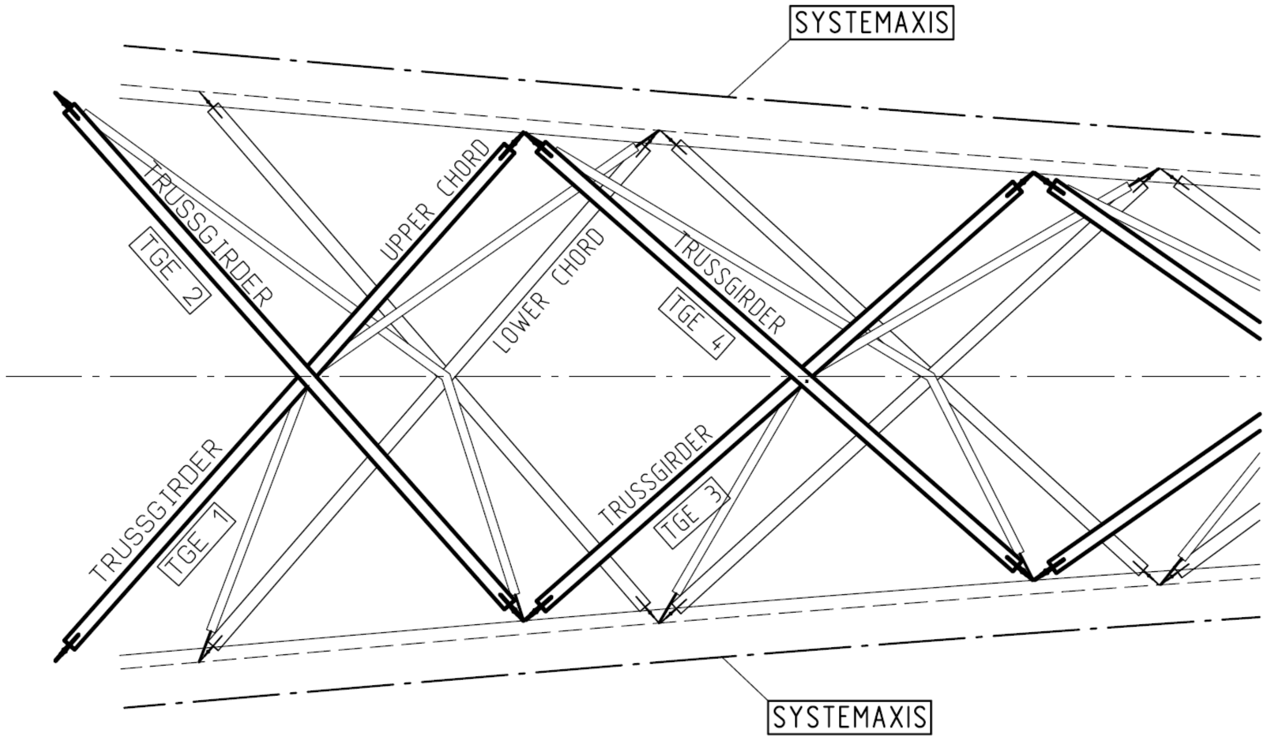


Figure 3: Top view of truss girder

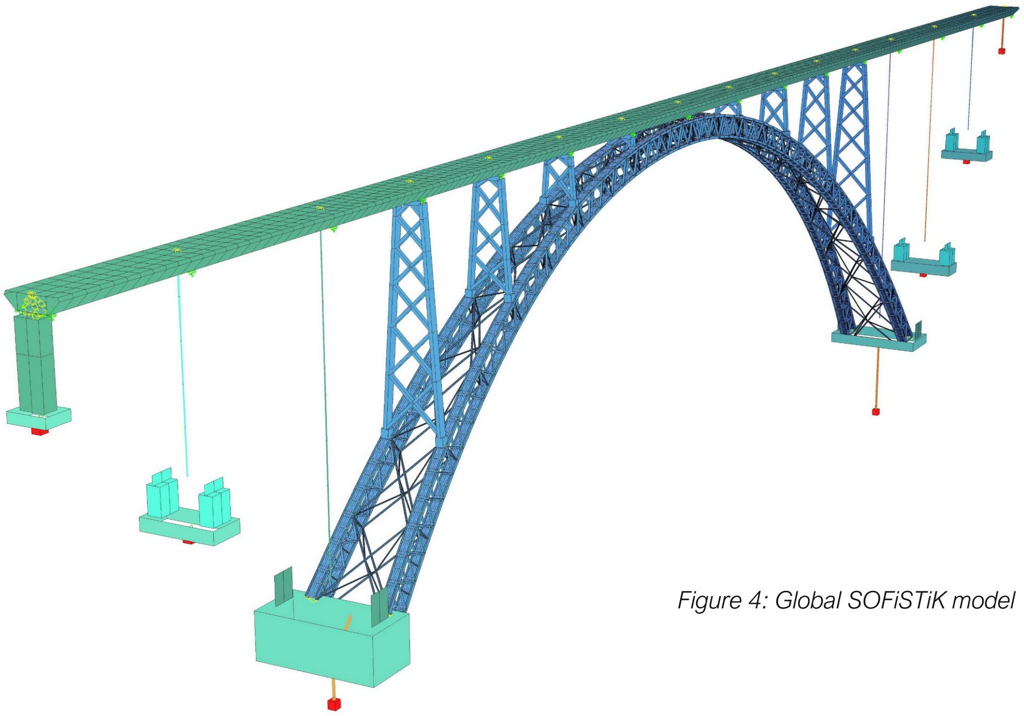


Figure 4: Global SOFiSTiK model

2. ANALYSIS MODELS

Chenab Bridge has been modelled in multiple ways at various design phases.

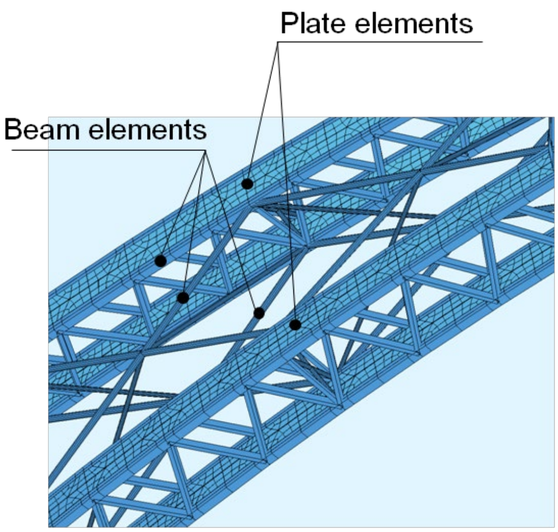
The main calculation model used for the global analysis and erection stage analysis is set up in SOFiSTiK (previously in RM7) and was continuously updated to the latest software version during the course of the project, Figure 4.

The purpose of this model is to obtain global action forces and displacements as well as detailed design forces for the arch and trestles.

Beam elements are applied as a standard. The chords of the arch are modelled by two separate longitudinal spines each, which are connected by K-bracing elements to replicate the true stiffness for lateral loads and torsion.

For the analysis of specific load conditions such as the loss of members, the K-bracing elements are replaced by actual plate elements allowing a study of the local behavior and residual capacities, Figure 5.

This more complex model has not been employed in the analysis of other load conditions as no necessity existed and for the benefit of faster computation, especially during the erection phase where reaction time by the designers was of the essence.



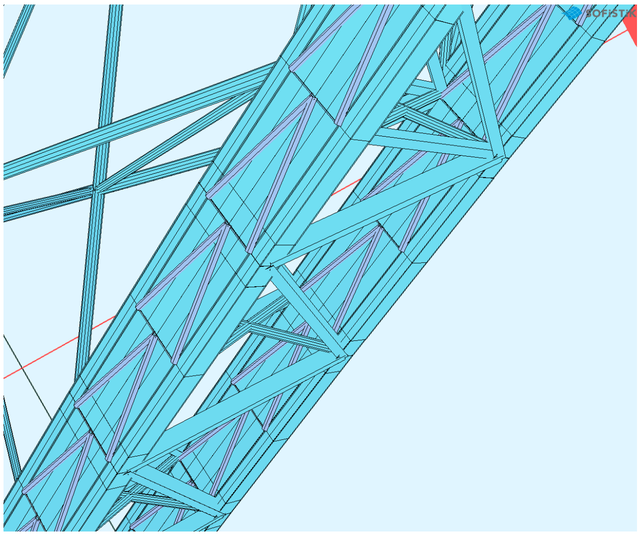


Figure 5: Modelling of the arch chord (a) with plate elements, (b) with K-bracings

As the main arch is being filled with concrete after its closure, the beam elements of the arch chords are modelled with composite cross sections, Figure 6.

The partial materials are activated in line with the actual erection procedure.

All structural elements in the models are split as a minimum according to the fabrication units to enable their activation in accordance with the erection process or for any side studies.

Most elements are further subdivided at intervals sufficient to understand the development of action forces along the overall member.

Hence, the same basic model could be used for the global analysis at the service stage and for the detailed erection stage analysis.

The global analysis model was supplemented by local FE models for the design of particular elements such as the central shear key and the wind bracing connection details.

For the erection stage analysis, temporary structural elements were implemented in the global model, namely the temporary stays and the temporary towers on top of the main pillars, Figure 7.

Some critical erection operations involved a number of small steps, in particular the installation of the first set of temporary stays and even more so the closure of the arch.

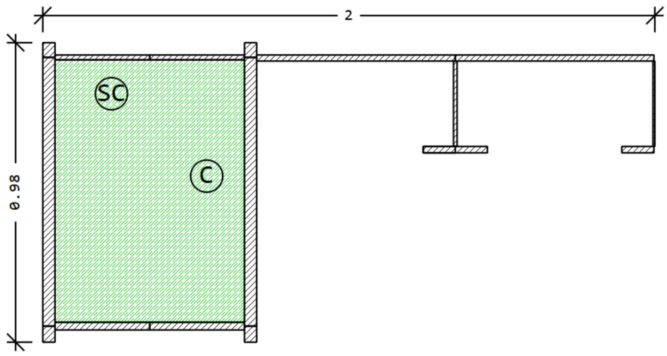


Figure 6: Cross section model of arch chord

In order to provide the site with data for the various sub-steps of these operations, models were split off the overall erection stage analysis and supplemented with further detail.

Following the division of work between the designers, a second analysis model was set up for the detailed design of the land piers and the superstructure.

This model was set up in the software LUSAS. The availability of two entirely separate models within the design team permitted a valuable comparison of results.

Apart from such internal verifications and corresponding design checks, a full independent proof check of the design and analysis has been carried out on this National Project.

For BIM modelling, refer to the upcoming e-BrIM magazine.

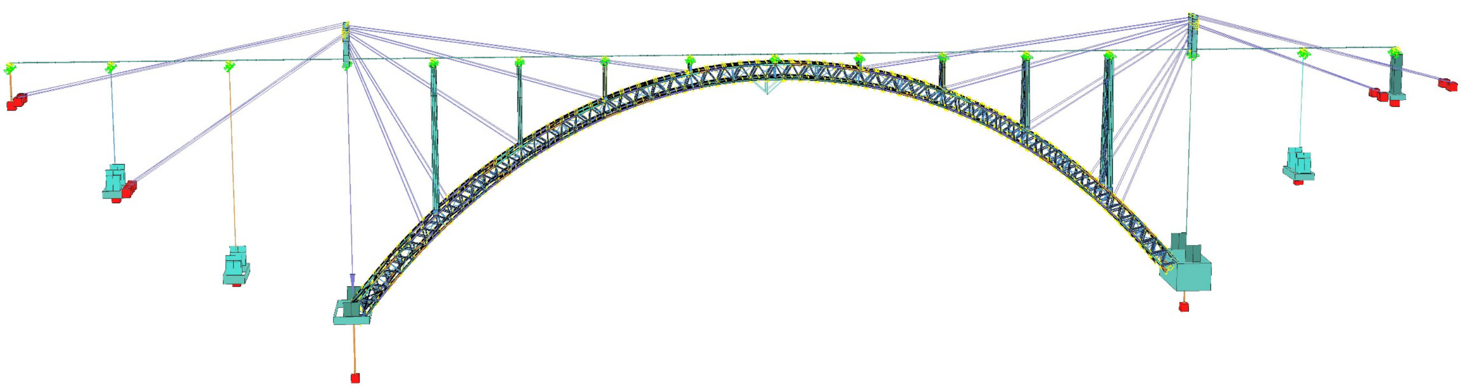


Figure 7: Analysis model for Erection Stage Analysis

3. EXTRAORDINARY LOAD CONDITIONS

Being situated in Himalayan terrain on the steep slopes of the Chenab River gorge, the bridge is exposed to extreme environmental conditions.

The high hills and the deep gorge cause the gust wind velocities and turbulence characteristics to vary strongly with the wind direction.

Equivalent static wind loads were developed on the basis of wind tunnel testing; it will be described by Dr. Risto Kiviluoma in the next upcoming edition of e-mosty.

With a 3-second gust wind speed up to 87 m/s at 120 year return period, extreme wind loads are decisive for the design of lateral members, in particular the wind bracings.

At extreme wind speeds, the trains cannot operate. The service wind load to be combined with MBG train live loading is based on a 3 s gust velocity of 49 m/s as implied by the Indian Railway Standard Bridge Rules. This load combination is decisive for the design of further structural members in the arch and trestles.

Separate wind load studies and testing were carried out for the design at the erection stages.

Specific critical operations such as the installation of the first few strands of the temporary stays and some sub-stages of the closure required a time window of low wind speeds, which was ensured by obtaining reliable weather forecasts from neighbouring weather stations.

The site location is classified as the highest seismic zone of India. The seismic design was carried out force based and governed the design of those lateral members in which the seismic demand exceeds the demands from wind loads.

Site-specific response spectra were developed by IIT Roorkee. Modal damping was conservatively considered as 2% for the entire structure including the composite main chords.

One of the advantages of this truss structure is its redundancy performance. It was verified by analysis that single elements of the arch – one at a time – can be rendered ineffective by external damage while the bridge would still be operational at a lower level of efficiency.

It has also been verified that entire piers can be removed from the system without collapsing the structure.

The deck girder design incorporates further features and strengthening elements to ensure redundant behaviour at major local damage scenarios.

Global and local imperfections are considered in the design. While the fabrication tolerances are clearly specified by the codes, global tolerances were set out project-specific, owed to the scale of the structure.

The main global imperfections are the verticality of the piers and the tolerances of the arch which are described as a series of shape imperfections, Figure 8.

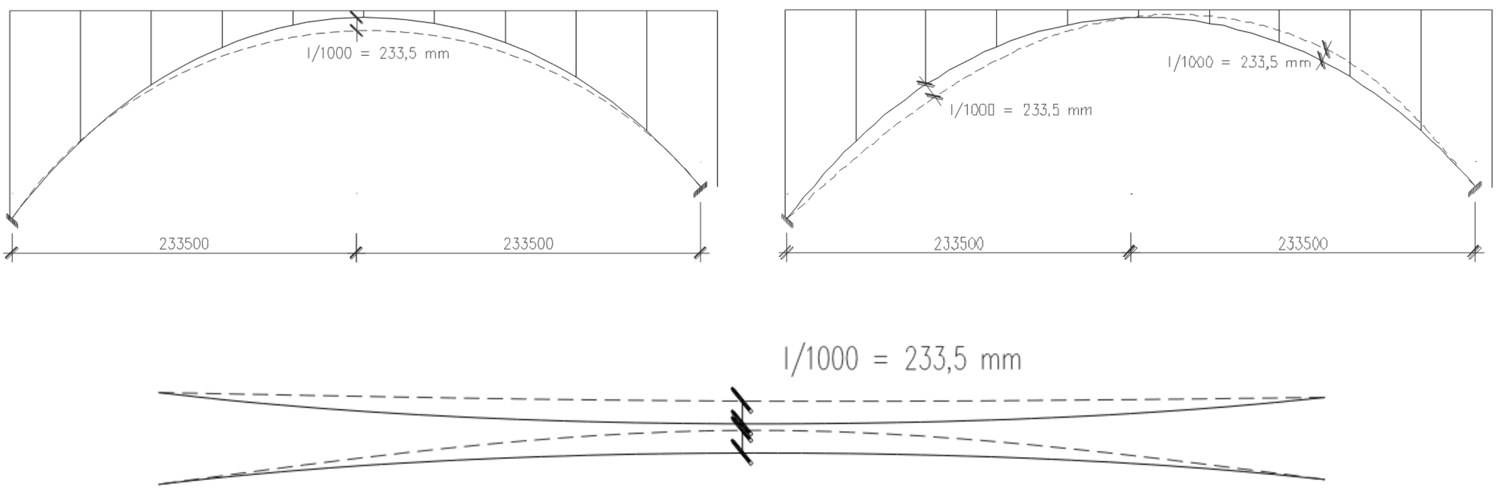


Figure 8: Arch design tolerances

The allowable erection tolerances are by safety factor 1.50 smaller than the design tolerances. The working tolerances on site were even tighter.

At certain trigger limits, the designers would be consulted to explore the reasons for deviations and to instruct countermeasures with a target to avoid the error being projected and causing any tolerance exceedance at a future stage.

Theory second-order calculations with pre-deformation were carried out as part of the detailed design.

4. STRUCTURAL DESIGN

Chenab Bridge is designed for a service life of 120 years. Steel grades used in the structure are E 410 C according to IS 2062 for the arch and piers (f_y = 410~380 MPa depending on plate thickness), and E 250 C (f_y = 250~230 MPa) for the superstructure.

The superstructure is an open girder section, while elements of the arch and piers are typically box sections to facilitate bolted splices as site welding was not allowed on the project, Figure 9.

Plate thicknesses are specified between 12 mm and 40 mm as a standard, and up to 70 mm for highly stressed members and gusset plates.

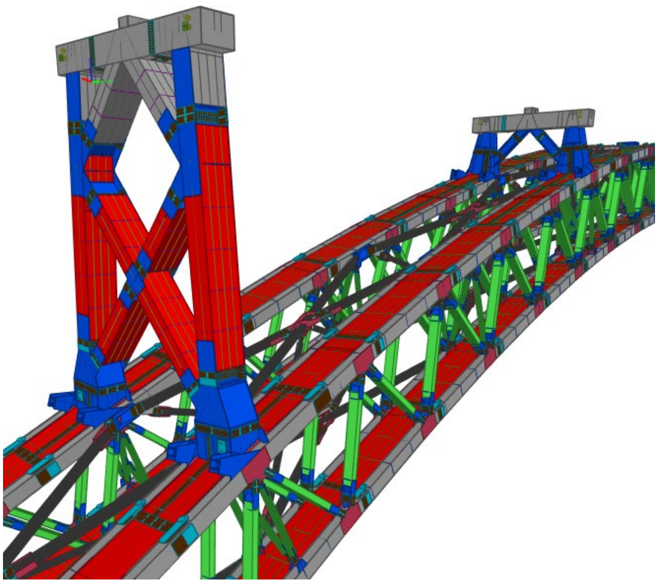
Bolted connections are designed to be non-slipping at SLS and acting in bearing at ULS. The bolts are high-strength friction grip (HSFG) bolts grade 10.9 HSFG according to BS 4395, with typical sizes M30 and M36.

The main boxes of the arch are filled with self-compacting concrete. This decision was taken in order to improve the robustness and the dynamic performance of the structure.

Composite action is ensured by providing shear studs and small-diameter rebar stirrups inside the steel boxes, Figure 10.

An added benefit of the infill concrete is the prevention of local buckling, thus allowing higher utilization of the arch main chord. Casting was tested on a mock-up of an arch section, with a focus to ensure the concrete quality and avoid any air entrapments.

Temporary bracing of the arch boxes for fresh concrete pressure was foreseen but eventually avoided by the contractor's refined procedures, concrete mix, and associated analysis. Only the wind bracings of the arch and a small quantity of secondary members are formed using pipe sections.



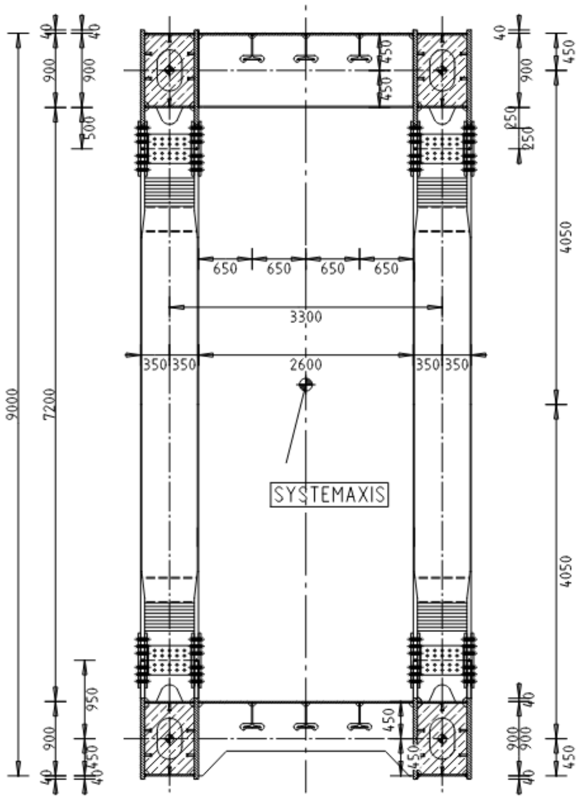


Figure 9: Arch snapshot (TEKLA) and cross section

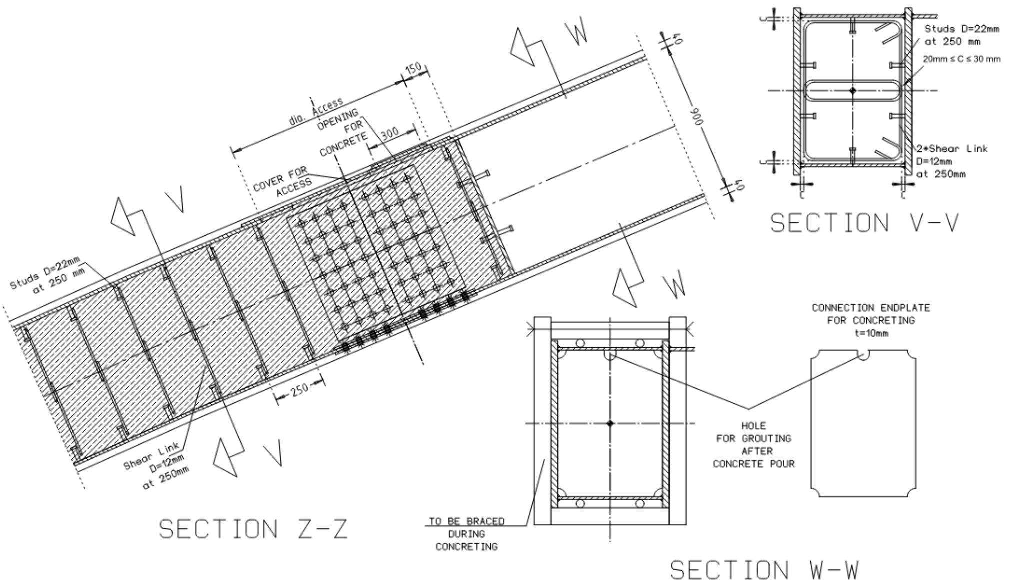


Figure 10: Arch concrete fill. Click on the image to open it in a higher resolution

Pipe sections were not preferred on the project as they are difficult to source in India, and not ideal to connect by bolts. However, the complex 3D geometry of the wind bracings was easier to achieve by using circular members.

5. ERECTION METHOD

Development and consideration of erection methods were an integral part of the design from the earliest stages and were key to the success of the project.

At the early project stages, the arch and superstructure were planned to be erected by cable crane in combination with a derrick lift [5].

At the restart of design in the year 2013 [1], the first and foremost decision was to erect the arch and piers by cable crane alone, Figure 11, and to launch the superstructure.

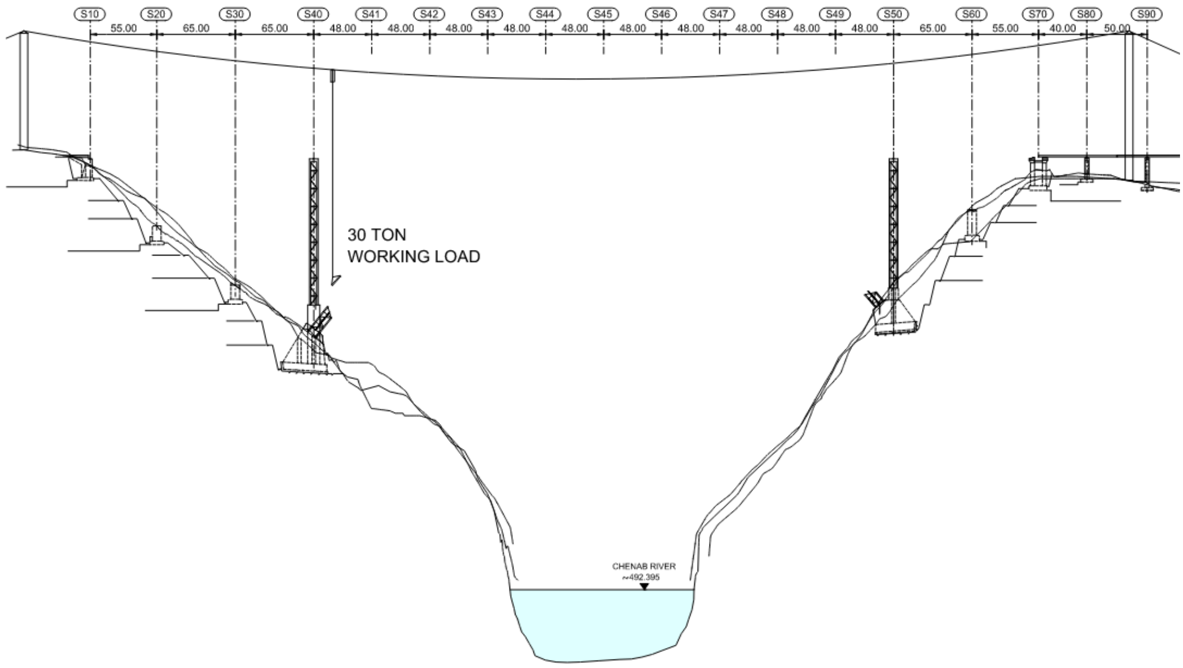


Figure 11: Arch Erection Stage 1, maximum payload of cable crane. Click on the image to open it in a higher resolution

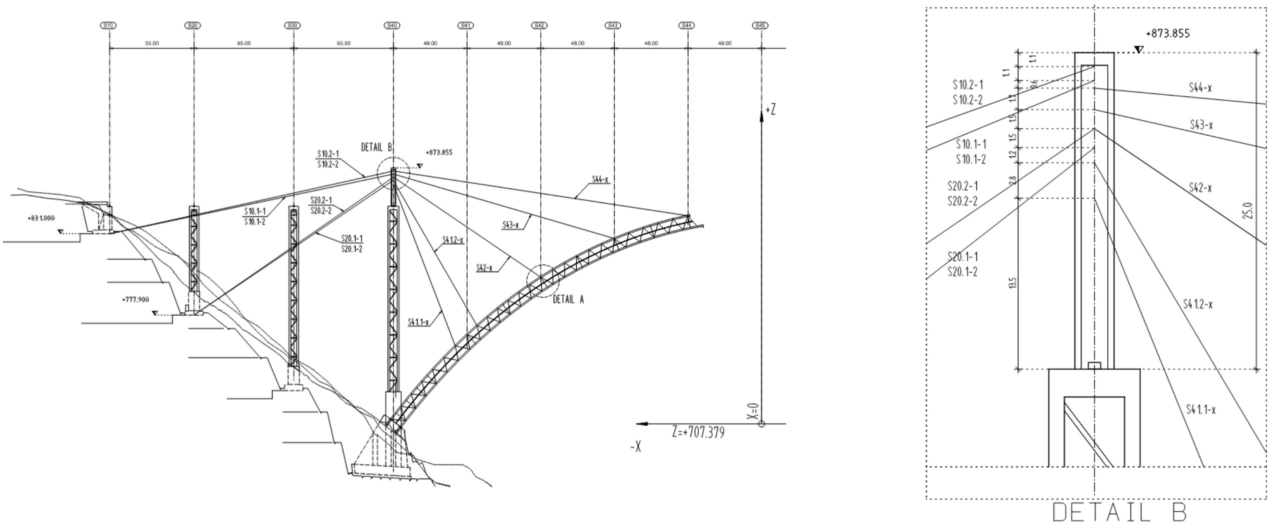


Figure 12: Arrangement of temporary stays (Bakkal side). Click on the image to open it in a higher resolution

This decision and the available lifting capacity of the already commissioned cable crane prescribed the final maximum weight of the arch and pier segments and were therefore a key parameter in composing the structural elements.

Subsequently, the optimum support of the partially erected arch by temporary stays was developed. The most straightforward solution was to anchor the temporary stays on a temporary pier with the maximum possible height in order to obtain the steepest possible angles and thus the highest possible efficiency of the stays, Figure 12. This height was dictated by the clearance of the cable crane operating above it.

Based on this principle, arch erection started with the foot pieces being installed onto the concrete sockets of the foundation by means of tie bars, Figure 13.

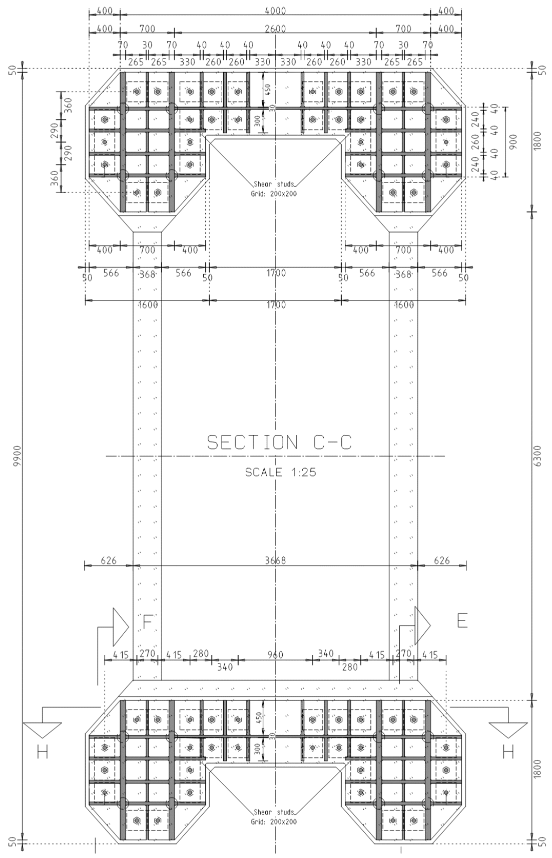


Figure 13: Arch foot layout and installation

Click on the image (left) to open it in a higher resolution

This operation had to be prepared and carried out to the highest possible accuracy as any errors in setting out the foot piece would be difficult to correct and in the worst case project along the whole arch.

The erection continued with a typical local installation cycle of:

- Installing a bottom chord segment;
- Installing a diagonal element;
- Final setting out and torquing of bolts in the bottom assembly;
- Installing a diagonal element (in assembly with top chord element if weight permits);
- Installing a top chord element;
- Final setting out and torquing of bolts in the top assembly.

The vertical adjustment of the bottom and top chord was controlled by tangent values f_b and f_t derived from the overall geometry and any local deflection.

As the arch members are straight within each truss triangle, f_b and f_t are essentially used to control the kink angles of the polygonal shape, Figure 14.

Again in this local cycle, the permissible tolerances were tight because errors in the kink angle cause deviation forces that are not accounted for.

When the arch cantilever had proceeded up to the first temporary stay cable anchorage, the initial set of stays was installed by a highly refined strand-by-strand installation sequence.

The reason for this detailed sequencing is that the main piers with mounted temporary towers holding the stays are completely freestanding in this initial stage.

With heights on each side of 140 m (S40) and 125 m (S50), these support pillars can only take minor differential horizontal forces, hence the horizontal component of any strand stressing force had to be compensated by installing and stressing single strands at the corresponding backstay or forestay cable.

A time window of low wind speeds had to be awaited before starting the operation. The stay-stressing operations were controlled, among other parameters, by live monitoring of the support pier verticality.

With an increasing number of strands and stays, the system stiffness increased and the support piers were less sensitive to the installation of further strands.

On the other side, the allowable force eccentricity decreased with increasing pier axial forces, hence different tolerance limits were specified for every stay installation.

As the arch erection and stay installation proceeded, the most vertical forestay cables were unloaded as a result of the arch developing an upward camber shape around Piers S41 and S49, and were uninstalled, Figure 15.

The most detailed procedure in arch erection was the closure sequence. The target of the sequence was to close the arch with the correct forces in the top and bottom chord (= arch global bending moments), and diagonals (= arch global shear).

There are several possible options to achieve this. In the case of the Chenab Bridge, the closure was achieved by a series of jacking operations, Figure 16.

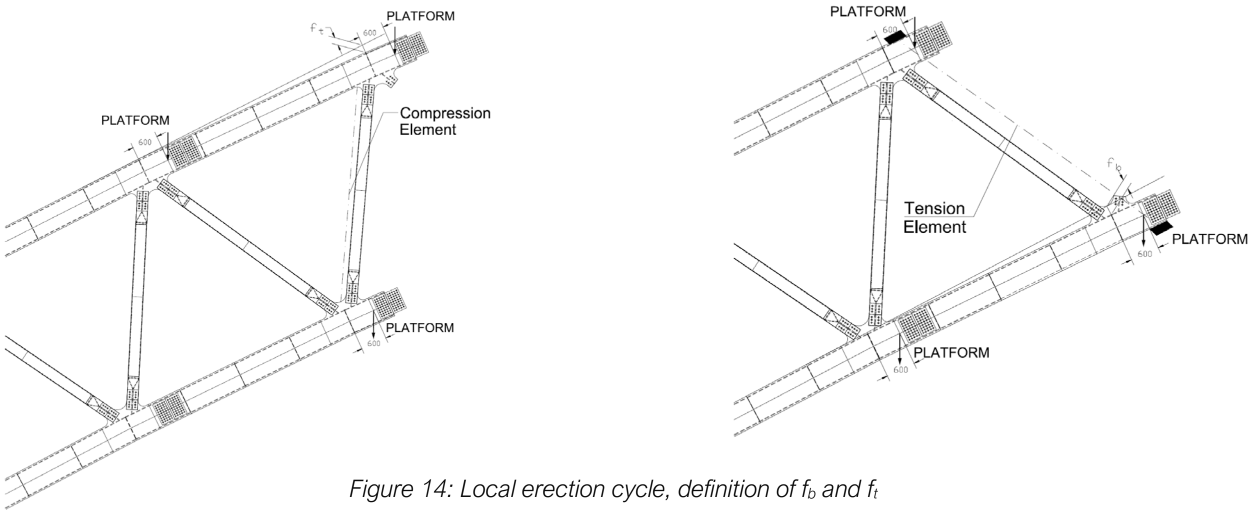


Figure 14: Local erection cycle, definition of f_b and f_t

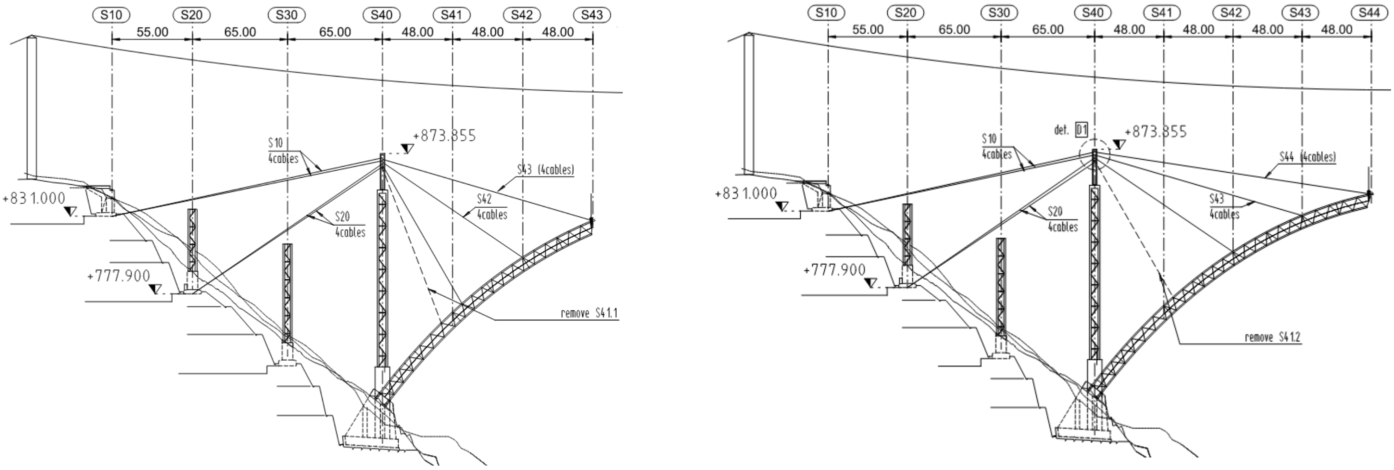


Figure15: Uninstalling temporary stays S41.1 and S41.2 (similarly S49.1 and S49.2 on Kauri side)

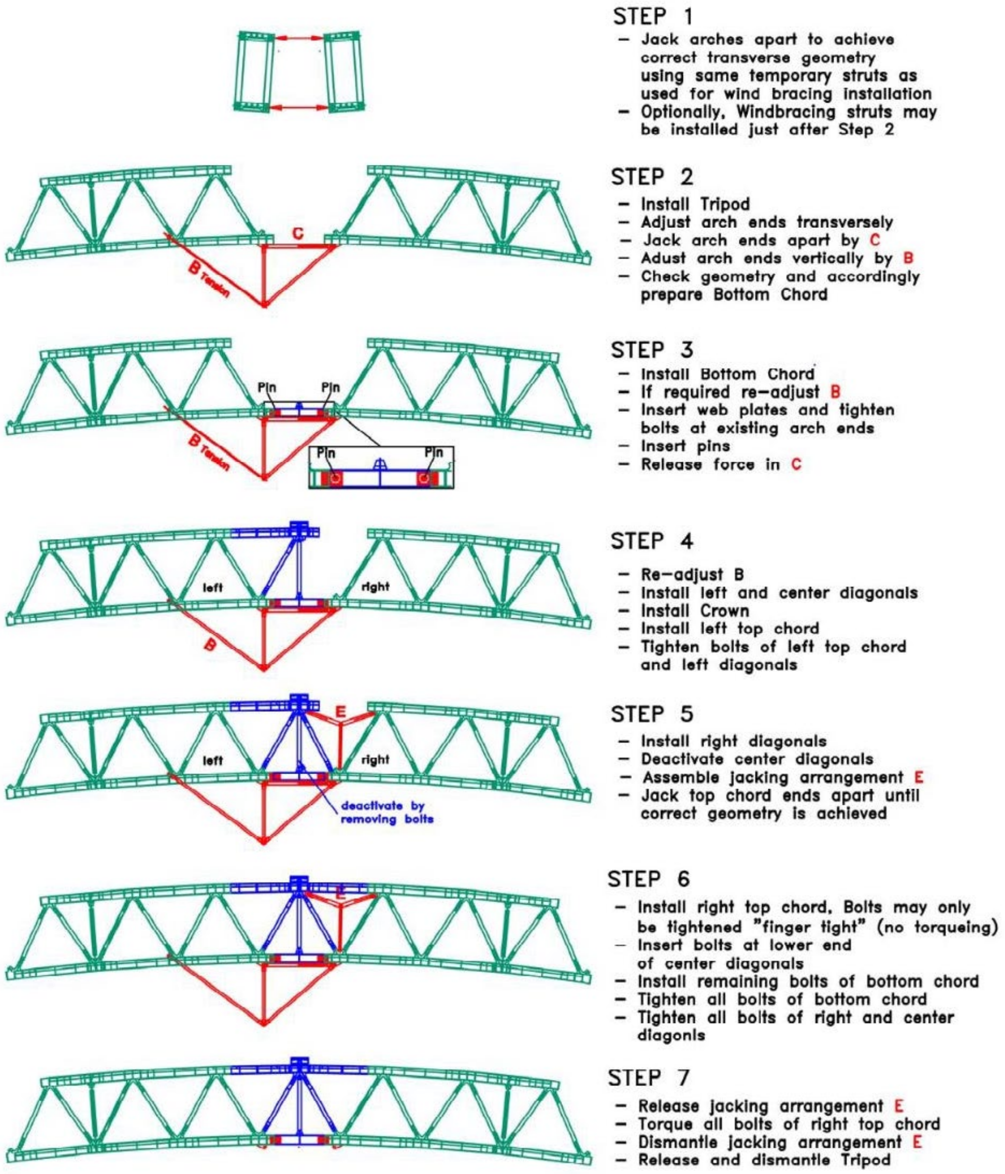


Figure 16: Arch closure sequence, Concept and temporary works by Andreas Faessler

A comprehensive Arch Erection Manual prepared by the designers was in place to summarize the design assumptions and to specify load limits, permissible tolerances, global and local erection cycles and special operations such as the temporary stay installation and removal, the arch closure, the concreting of the arch.

In particular, the contractor was supplied with data sheets to allow an own 3D geometry control at any moment in time, including calculation tools for temperature correction and consideration of construction live load.

Only at certain predetermined stages, the data was exchanged with the designers and subsequent steps were modified when needed to achieve the correct final stage condition.

After the arch had been closed, the temporary stays were uninstalled and the arch boxes filled with concrete.

Thereafter, the arch piers were erected based on camber data sheets developed to ensure verticality at the final stage.

The launching design of the superstructure was similarly challenging and is presented in [2].

6. CONCLUSION

Throughout the long course of the project, analysis, design and construction methods went hand in hand and were constantly updated to the latest technologies.

The continuous, productive interaction between the designers, the contractor, and the proof consultant made this landmark project possible and a success.

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ARCH CONSTRUCTION OF CHENAB BRIDGE, INDIA

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1. INTRODUCTION

The infrastructure industry encompasses design and construction of many elements whether small or big. Engineering advances and a high level of technical culture have brought about a revolution in this industry leading to the construction of some massive record-breaking structures.

Precast and Steel construction is the norm of the day since it saves a lot of time, effort and cost as compared to cast in-situ construction while giving an added advantage of superior quality of finished structural members.

On the other hand, it demands a lot of research for the erection of these elements in their final desired location at the site by transporting them from the shop (casting yard or workshops).

The Chenab Bridge is one such project involving the fabrication of massive steel structural elements of tall Piers and the arch in workshops and subsequently placing them in their final position in the valley of the river with the help of the Cable Crane standing on a pylon at both ends of the valley.

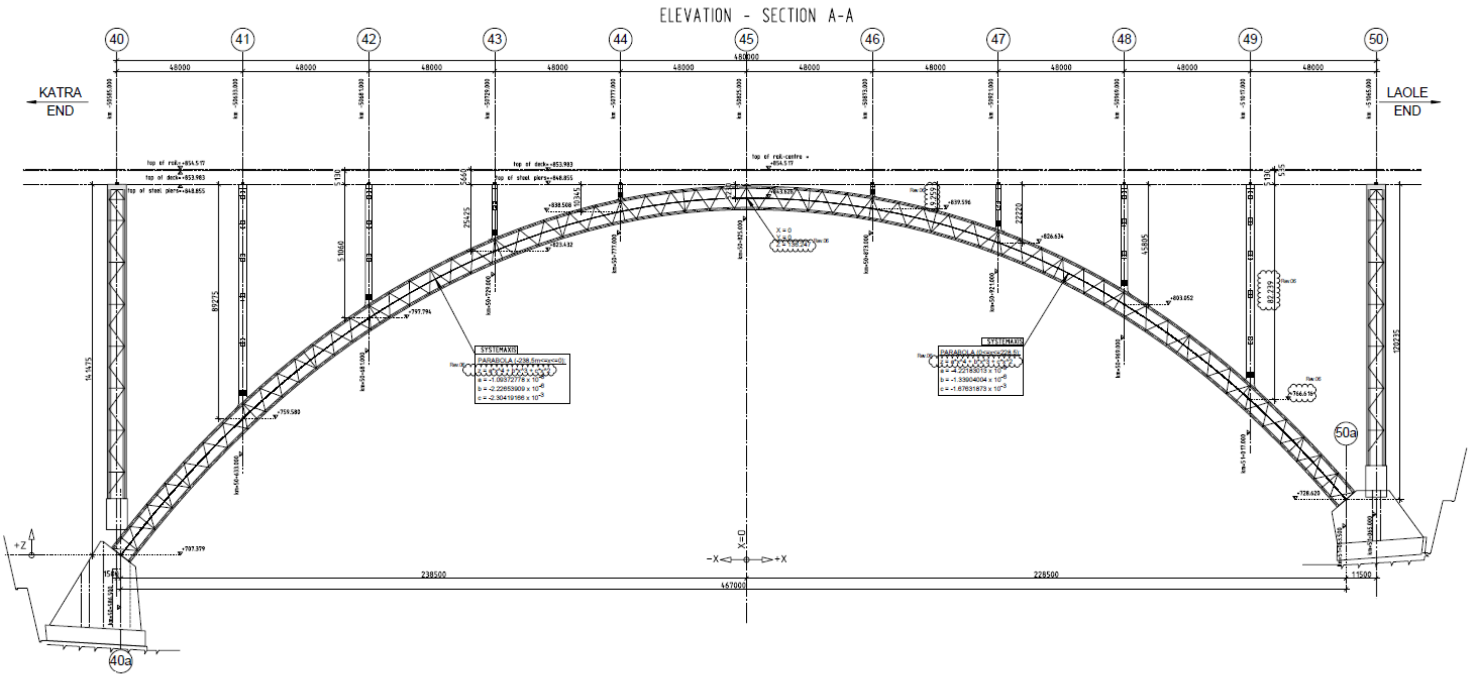


Figure 1: General arrangement of the arch bridge showing elements of the steel arch

Click on the image to open it in a higher resolution

Erection of arch elements and their bracings was done by first lifting, aligning and placing the elements to the desired location, inserting bolts to free the cable crane for lifting other elements and then installing working platforms on the individual members so that all the splicing geometry could be made accessible for that section.

The connections were fixed so that the erection cycle could proceed but only after checking the required geometry of the member.

2. CONSTRUCTION CHALLENGES AND THEIR SOLUTION

2.1. CONSTRUCTION OF ARCH ELEMENTS BY CABLE CRANE

The steel arch girder has a main span of 467 m and abuts on the two ends of the valley on two massive foundations S40A and S50A.

Each arch segment is made up of box sections of 10 m to 12 m in length and bracings which are connected to the top and bottom chords of the arch, both of which are lifted with the help of a cable crane for placing in the final position.

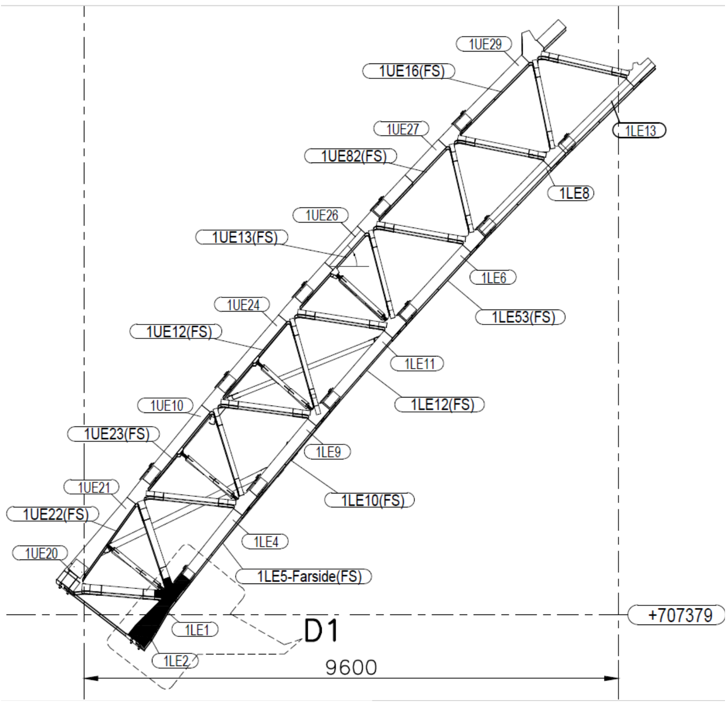


Figure 2: Upstream elevation of the arch in span S40A - S41showing top and bottom chord with box bracings

Figure 3: Crossbar cable crane

Even though the construction of these elements was done by cable crane due to difficult terrain, the actual in-place erection and geometry matching was quite a challenge, Figure 2.

All steelwork on the slopes of the river gorge, including the iconic arch itself, was erected by the World's largest crossbar cable crane supported by 127 m tall pylons, Figure 3, spaced at a distance of 915 m.

The crossbar cable crane is composed of two units with a lifting capacity of 20 tonnes each.

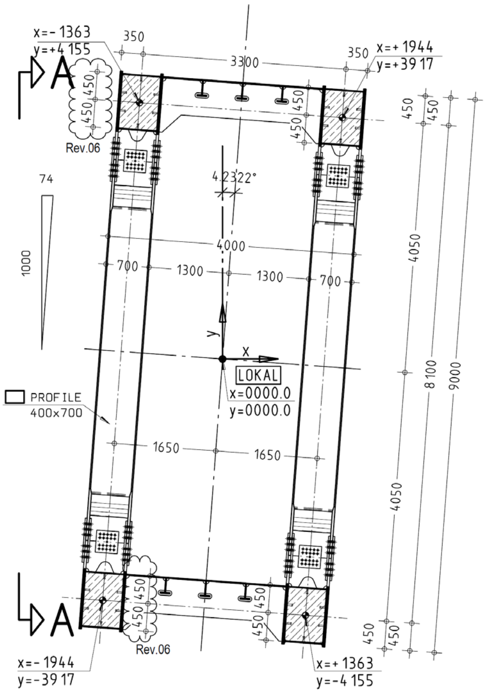
The same can be increased to 36 tonnes on a single hook when operated in tandem.

This erection method and the available lifting capacity were the driving factors for the sizing of all truss elements of the arch and piers.

Since site welding is not permitted on this project, the single components are connected by bolted splices, a fact that spurred the decision for using rectangular steel members.

Only the wind bracing of the arch was designed with steel pipes as this simplified the accommodation of varying connection angles.

The overall complex geometry of the bridge required detailed procedures and fine adjustment to lift all assemblies at the correct inclination in two planes considering their exact gravity centre.



TYPICAL SECTION
SCALE 1:50

Figure 4a: Typical cross section of the arch and photo of fabricated single boxes in shop

Click on the image (left) to open it in a higher resolution

2.2. LIFTING OF INDIVIDUAL ARCH SEGMENTS – SPLIT AND NON-SPLIT

Lifting in engineering terms is the process of lifting an object from one position and placing it in another desired position where it can be rendered for its intended use.

The term lifting is many times an understatement concerning the accuracy and safety of work required in it.

There are two types of segments of arch: split segment and non-split segment.

Split segments are members that are split into two and consist of longitudinal splices throughout their length, Figure 4.

They are designed as such due to the limitation of the lifting capacity of the cable crane.

Each segment (split or non–split) has varying weights and weighs up to a maximum of 30T.

The lifting scheme had to be devised in such a way that the arch segments be inclined at angles varying from 49° (base of the arch) to 0° (arch crown) in elevation and at a constant angle of 4.23° in cross section.

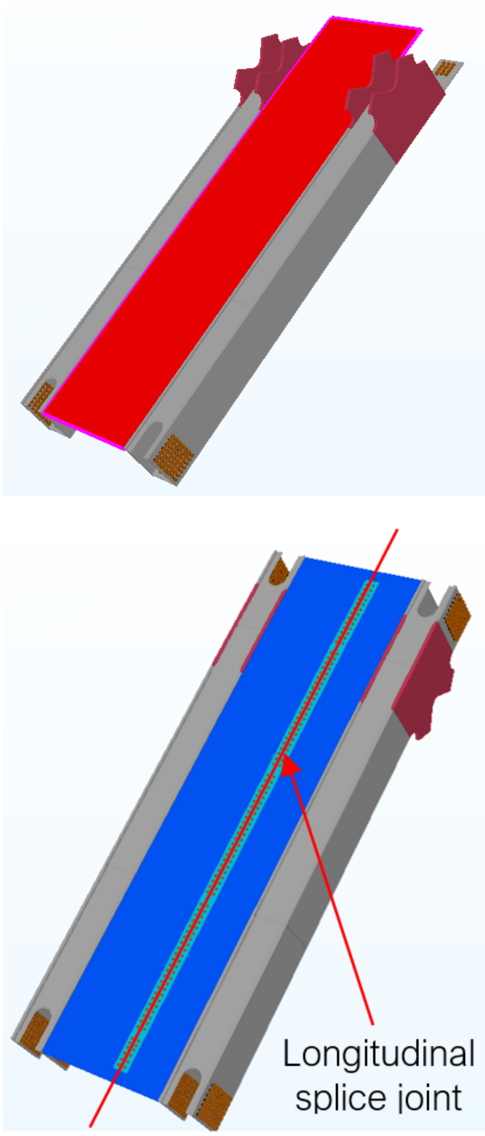


Figure 4b: Bottom chord non-split segment and top chord split segment with longitudinal splice joint

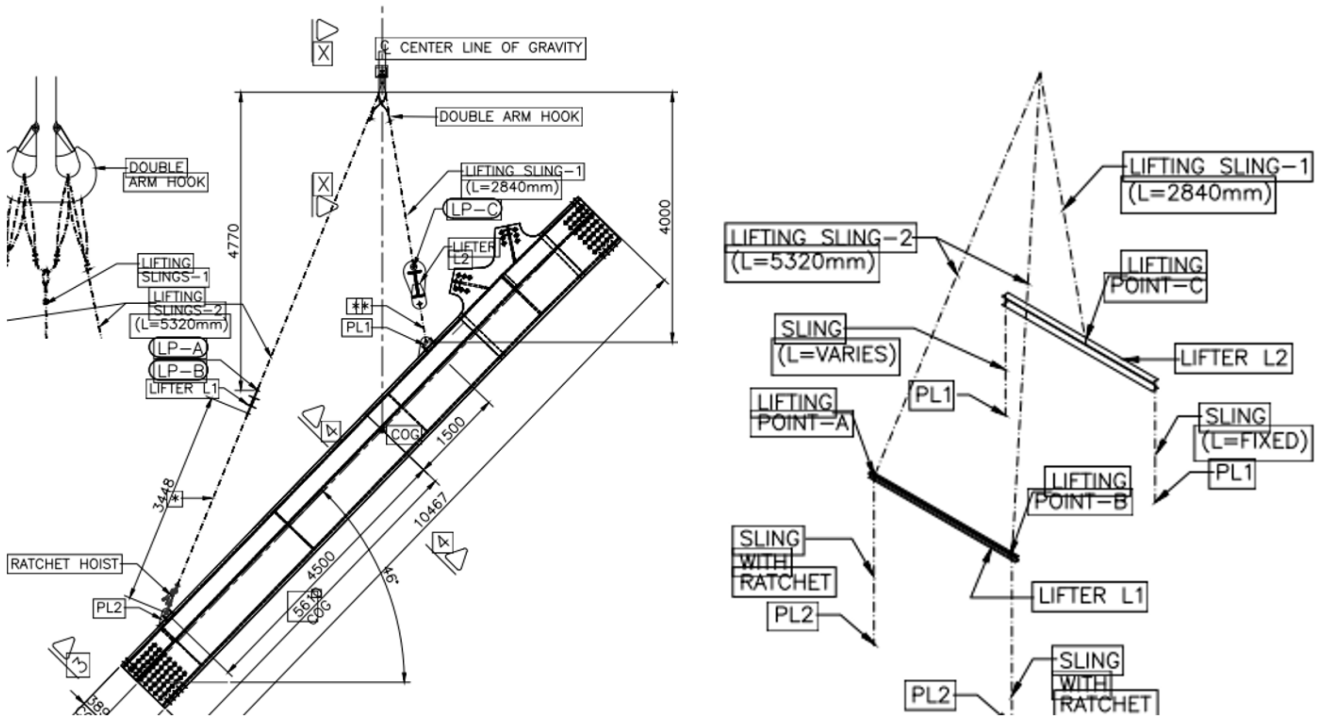


Figure 5: Typical lifting arrangement of the arch segments

For this purpose, the location of centre of gravity of the element is of utmost importance.

Assembly drawings, converted in AutoCAD consisting of centre of gravity (CoG), are obtained from TEKLA software.

The arch segment is checked whether the load is within the capacity of the cable crane and if not a technical query was raised with the consultant to provide a splice joint to accommodate the assembly weight within the capacity of the cable crane.

The lifting scheme was devised to cater to the varying inclination of the arch in elevation.

It is basically a 3-point lifting system that uses two spreader beams and is connected to the arch segments at four hook points.

Two hook points are above the level of the centre of gravity and two are below it.

All hooks are welded to the inner web of the arch which ensures there is neither any prying force action nor irregular deflection of the arch assembly.

All points are further connected by means of slings.

The provision of a 6T capacity ratchet hoist is made with slings connecting the bottom hooks for fine adjustment of the segment.

Two slings in the same plane are provided with varying lengths and a chain hoist system of suitable capacity to cater to the orientation of 4.23° in cross section, Figure 5.

The two hook points on the upper half of the segment are located so that they share the maximum load of the lifted assembly whereas the hook points on the lower half of the segments are located so that they share a minimal amount of the weight, in this case, governed by the capacity of the ratchet hoist.

As can be seen from Figure 5, the spreader beam at the top has one lifting point and the beam at the bottom has two lifting points thus rendering it a 3-point lifting system.

The advantage of such a system is that it ensures the design load distribution in all the slings.

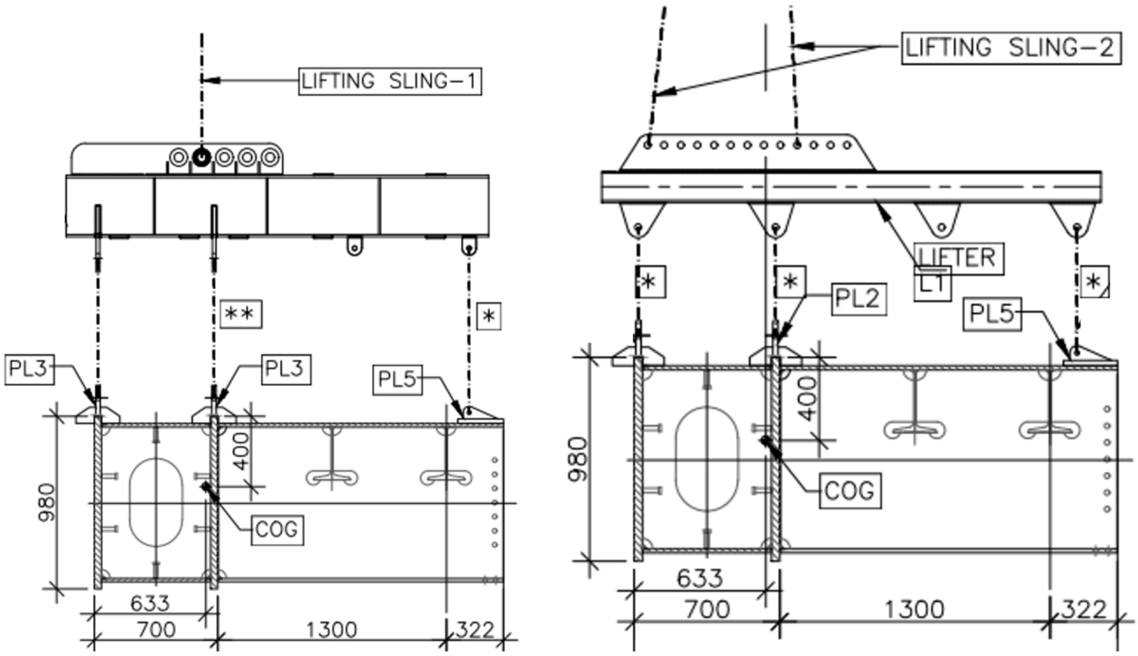


Figure 6: Typical lifting arrangement of the split arch segments in cross section

A similar concept is used for lifting the split arch segment with few modifications to the system, Figure 6.

It remains a 3-point lifting system but the hooks are welded to both the webs of the arch (inner and outer).

The spreader beams used consist of a number of holes due to the variation in the CoG of split segments and are smaller in length. Both systems, once fabricated, are capable of erecting the entire arch elements (approximately 300 segments).

Figure 7: Arch bottom chord erection in progress

2.3. ERECTION OF ARCH SEGMENTS – CONNECTIONS AND GEOMETRY CONTROL

Once the segments are lifted into their desired position, they are to be connected to the already erected member.

This connection involves the insertion of a required number of drift pins into the splice plate before the cable crane can be released and further torquing can be completed after local geometry is achieved.

For this purpose, platforms are required to be placed on the previous member for access of bolting wherever required. Three types of platforms are provided for different types of connections including main box joint, longitudinal joint and cross joint.

A C-type platform was developed and fixed to the arch segment, Figure 8, hence the C-type platform has been lifted with the Arch segment which enabled the ease of erection of the platform and its weight was optimized to a great extent.

Further coordination with the site team resulted in modifications and additions to the platform so that the system can be a robust layout for all arch segments.

Simultaneously, another lightweight platform was developed that was seen as a substitute to the above-mentioned system, both of which were suitable to execute the job, but can be used by the team as per the actual site condition, Figure 9.

2.3.1. Erection Challenge due to Configuration

The wind bracings of the arch form a very important element in the arch design since they control all the wind-induced vibrations during cantilever construction of the arch as well as the lateral design wind loads.

The arch has a width of around 30m at the abutment and reduces to almost 9.5m at the arch crown.

The maximum length of wind bracing is 40m, consisting of pipe sections and has a gusset connection inclined in all three directions, and it has connections to the edge of the arch as well as at its centre.

Figure 10 shows the wind bracings with customized access platforms that were necessary to cater to changing inclinations of the arch.

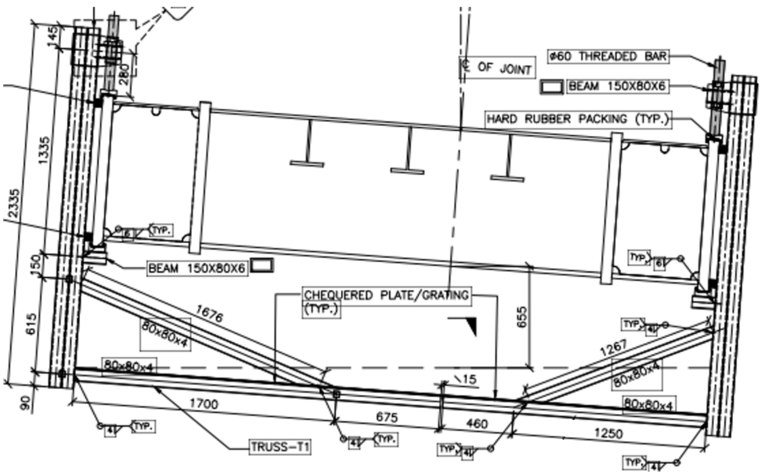
Taking into account the slenderness of the pipe sections to be erected, it was foreseen that the wind bracings should have considerable sag during lifting which could cause a mismatch during connection at ends.

2.3.2. Long Access Platforms for Connections and Jacking

The wind bracing assemblies have to be connected with the arch and also to one another after they are lifted and placed in position.

Hence the requirement for access to the desired location is a must for any minor or major adjustment so that the bolts are properly inserted in splice plates and have to be torqued after achieving the final geometry.

↑ *Figure 8: C-Platform connected to bottom chord for longitudinal splice connection*



↑ → *Figure 9: Adjustable access working platform for joining split segment of arch*

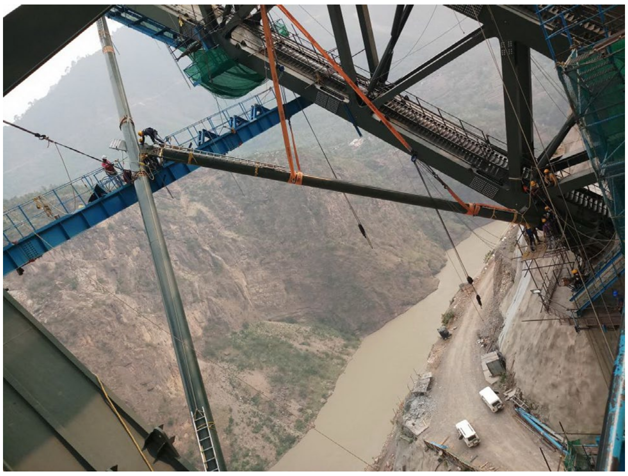


Figure 10a: Erected wind bracing 40m pipe with connecting pipe at bottom chord level

Few important parameters here are the requirement of jacking arrangement to correct the sag of the slender pipe bracings and also the various numbers of locations where platforms have to be provided considering the changing inclination of the arch at the top and bottom chord.

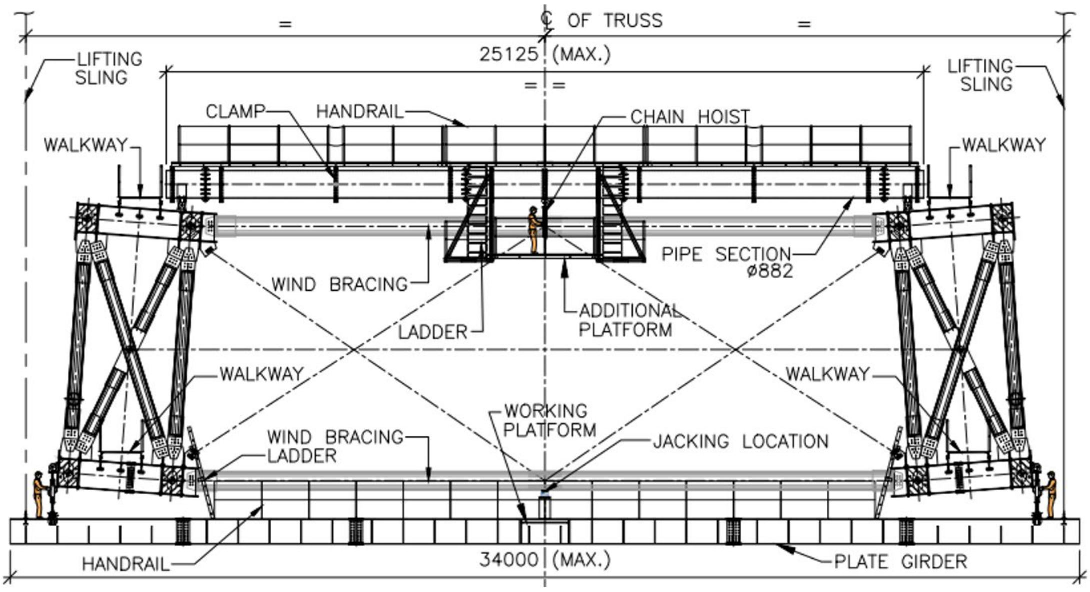
The in-house designed wind bracing working platform enables all the activities mentioned above in one single combination.

There are two platforms: a 35m-long plate girder to access the bolting at the bottom chord level that requires jacking, and a 25-m long pipe section at the top chord level for bolting access, Figure 11.

Considering the number of wind bracings (truss girder sections) in the entire geometry of the arch

↑ *Figures 10b and 10c: Erected 1st set of wind bracings with access platforms in span S40A-S41 & S50A-S49*

↓ *Figure 11: Typical cross section of arch at wind bracing connection location with platforms at top and bottom chord levels*



and the varying inclinations of each wind bracing in elevation, it was decided to design platforms that are customized to suit the entire arch geometry.

The platforms can be broadly classified as:

- Wind bracing top chord platform;
- Wind bracing bottom chord platform.

As the arch erection moved away from the abutments, the width of the arch reduced and so did the length of the truss required. The platforms were hence provided with splice joints as required for the curtailed length.

All the locations on the arch where these platforms were going to be required were discussed in close coordination with the site team and the design was taken up once the locations were finalized.

Maximum jacking force was calculated by the known deflection of the pipe section in dead weight and the platform was designed accordingly.

Although the platforms were designed to rest on the arch, the main access requirement was at their centre.

Special cantilever platforms were provided as fixations on the main platform. The bottom chord additional platform was in the form of a cantilever bracket projection on the plate girder whereas the top chord additional platform was a hanging bracket on the pipe truss girder.

The top chord platform rests on eye plates welded to the arch and the bottom platform is fixed to an assembly of turnbuckles.

As these platforms are to be reused and shifted to various locations, this type of fixing arrangement proves very effective for removal as well as installation.

3. ARCH TRIAL ASSEMBLY

Arch trial assembly is the setup of all arch elements on land to minimize fabrication errors during final erection.

Although it was not a mandatory requirement in the contract, however, being cautious of geometric control during the erection of the arch it was decided to go for full-scale trial assembly.

The trial assembly was done in two parts:

- Stage 1: Horizontal Trial Assembly by keeping a straight plane;
- Stage 2: Vertical Trial Assembly by actual inclination.

For conducting the arch trial assembly, an area of 54m x 100m with a reinforced concrete base was developed near the arch fabrication shed.

The major equipment used was two gantries of 100MT and 60MT capacity used for handling arch components (i.e. arch bottom chord, diagonal bracings and top chord), a telescopic crane of 60MT capacity used for handling wind bracing components in a trial assembly area, hydraulic jacks of 50MT capacity used for supporting/aligning arch chords above trestle.

To ensure elevation geometry check of the arch in trial assembly, the work points obtained from the Tekla model at node points of each segment of the arch on the inner and outer side of both the upstream and downstream arch were measured using a Total Station.

The members of the arch were erected in accordance with the data measured from the Tekla model.

After one span was assembled in the yard, the first half portion of the span was dismantled.

Figure 12: Arch trial assembly progress and completion for one typical span

The remaining portion of the span was moved back and the next span was assembled at the front. The process continued till the completion of the arch.

To ensure the plan geometry check of the arch in trial assembly, the centre line of the bridge is marked in the trial assembly area. All readings were taken with these benchmarks as reference.

The horizontal distance between the centre line of the bridge and the inner working points were measured. This check confirmed whether the arch was in the required inclination of 4.23° and placed with respect to the centre line of the bridge.

4. ARCH CONSTRUCTION

4.1. ERECTION STAGE ANALYSIS (ESA)

The complex task of the Erection Stage Analysis (ESA) is different from that of the detailed design of the arch in the permanent condition.

ESA is an integral part of the project since it determines many parameters required to be monitored during the arch erection.

ESA was supplemented by the analysis of critical erection stages to ensure the structural capacity of all elements does not exceed the individual erection stages.

A sophisticated step-by-step analysis has been performed to achieve a clear understanding keeping in view the following parameters.

a) Detailed erection sequences.
   - When to install and release the temporary stay cables
   - When stay cables need to be re-stressed.

b) Forces and deflections of all parts during all erection stages
   - Cable forces,
   - Camber curves of the arch,
   - Local and global deflections,
   - Support reactions

c) Adjustment values to account for variations in
   - Temperature,
   - Construction live loads i.e., equipment, local access platforms, walkways, etc.
   - Cable force

Special care was taken to address the exact dead weight of the steel segments, which was based on the final Tekla Model.

During the erection, the load and climatic situation may deviate from the assumptions made for the erection stage analysis (ESA). Hence to consider these deviations, the theoretical deflections and forces needed to be adjusted on-site.

Therefore, unit temperature and other unit load cases were developed and applied to the static system at various erection stages.

To determine the values for adjustment linear interpolations were to be used with the unit load values.

This was, however, a complex procedure and the adjusted values of stressing force were extracted and confirmed with the designer to achieve the required geometry.

The construction analysis is based on a 5-year return period wind level. The design basis means wind velocity at deck level is 42 m/s and a gust wind velocity of around 50 m/s.

4.2. TEMPORARY TOWER FOR SUSPENSION OF TEMPORARY STAYS

On top of Piers S40 and S50, temporary towers were installed. They serve for holding the forestay and backstay cables, Figure 13.

With increasing inclination of the forestay towers, the support of the arch becomes more effective. For this reason, the temporary towers had to be as high as possible.

However, their height was limited to 25 m above deck level in order to avoid clashes with segments running over the top under a deformed cable crane.

Towers were assembled from three pieces which are bolted together to limit the erection weight of each segment to 30 tonnes. The main actions on the towers are the stay cable forces and wind loads.

4.3. GLOBAL ERECTION CYCLE

The arch was erected simultaneously from the Bakkal and Kauri sides by cantilever construction technique.

Figure 13: Temporary tower above pier during erection (left) and after erection completed (top)

The cantilevers were temporarily supported by stay cables of the strand system at the future trestle locations. As a result, the cantilevers were freestanding for approximately 50 m before installation of the next set of cables.

Two types of temporary cables were used:

- The forestay cables spanning between the arch front end and a temporary tower.
- Backstay cables that were connected between the temporary tower and stiff supports at S10 and S20 foundations on Bakkal side, and S70 and S80 foundations on the Kauri side.

Figure14: Photo showing erected Arch girders and the first set of stay cables

When the first set of forestay cables was installed to support the cantilever arms near S41 and S49 locations, backstay cables were simultaneously tied to rigid anchor points at S20 and S70, Figure 14.

All cables were stressed from the temporary tower. The installation and stressing of the first cables in particular were planned and executed in a very refined sequence since the freestanding, 131m tall support piers could accommodate only limited differential horizontal forces. Weather forecasts and wind speed limits had to be observed to enable this operation.

Further forestay cables were added as arch erection progressed, and backstay cables added or restressed in such a way that the support pier was leaning towards the hillside immediately after stay stressing, as planned and predicted by the construction stage analysis.

With the weight of the increasing free cantilever, the pier inclination changed towards the riverside until the next set of cables was installed and stressed.

This procedure was essentially followed until just before the closure.

The cable forces at each stage were calculated using SOFiSTiK software and if required the forces of the cables were adjusted by the designer accordingly at each stage to adjust the geometry of the arch as per theoretical condition, Figure 15.

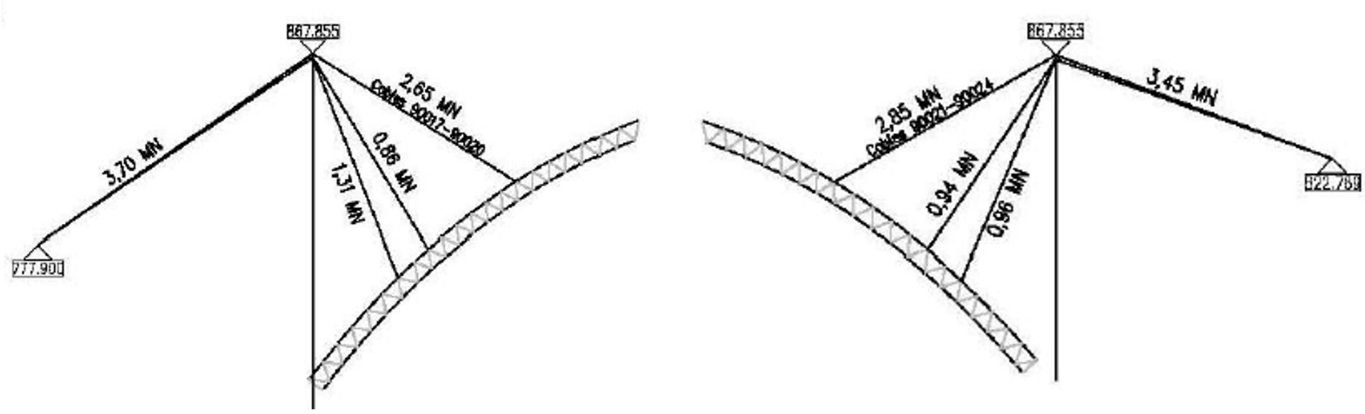


Figure 15: Snapshot of Computer-aided analysis of Erection stage calculations

The stay cables were stressed to the force at each stage so that geometry or as-built camber was used as control data.

The cable forces were given by the designer, however, consent was given in the erection manual that adjustment of the cable forces (within ± 5 %) to adjust the global elevation of the arch cantilever was acceptable.

Hence site engineers were having an important task to keep the camber and deflection within the limits so that the required adjustment in cable forces was within the limit specified by the designer.

After all required adjustments of the theoretical values had been performed (load and temperature adjustments), the location of the work points in global coordinates was to be met with desired requirements for each individual erection step.

The working tolerances on site were tighter than the allowable tolerances so that corrective action was taken before the allowable limits were reached.

Erection sequences were established so that the arch cables were stressed symmetrically and local stresses in any of the members were not developed. The sequence of stressing is described in Figure 17 below.

The symmetry of stressing was achieved by the inner cables S2 and S3 of the arch stressed at 50% and next the outer cables S1 and S4 were stressed at 100% and subsequently the remaining inner cables S2 and S3 completed the 100% stressing.

$S_{x,2}$ & $S_{x,3}$ (50%) → $S_{x,1}$ & $S_{x,4}$ (100%) → $S_{x,2}$ & $S_{x,3}$ (100%)

The arch inclination angle varies from 51 degrees at the arch base to almost 0 degrees at the arch crown.

Figure 16: Arch Erection in progress from both sides

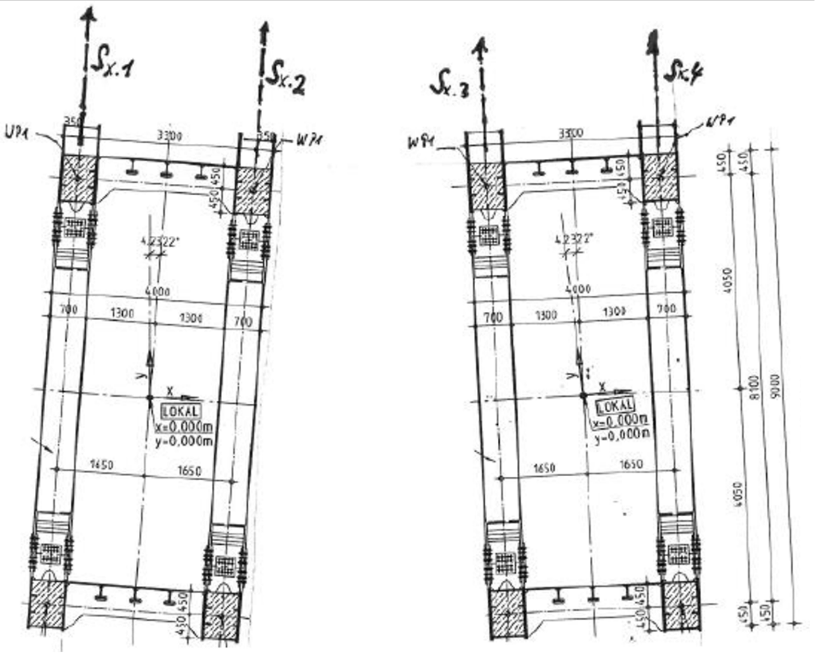


Figure 17: Photo showing cross section of Arch and sequence of stressing cables

Hence the angle of the forestay cables also reduces from a steep slope to a gentle slope to almost horizontal at the trestle base location over the arch at S44 and S46, Figures 18 and 19.

Subsequently, the arch progressed towards the centreline of the bridge location during erection and hence another set of forestay and backstay cables was added and stressed till the arch reached the closure at the crown.

Figure 18: Snapshot of Computer-aided analysis of Erection stage calculations

Figure 19: Photo showing arch erection from both sides and last thee cables just before arch closure

While this construction process was carried out, the inclement and unpredictable weather in this region of Jammu and Kashmir was the biggest and toughest challenge. Wind, temperature variation and rain were the three major elements that played a significant role and main challenge to the planners, designers and the construction team.

5. ARCH CLOSURE PROCESS

The ach closure process began when the arch arms erected from opposite banks were approximately 6m apart at the bottom chord level and 15m apart at the top chord level. The key segment had to be fitted into the structure with a certain force specified by the designer.

This was needed to assure the correct force distribution within the arch and a tangential alignment of the elements. A locking device was needed to fix the two opposite ends during the bolt installation process.

The overall arch closure process ensured the following three aspects were bundled carefully and systematically.

a) The reduction of the built-in construction stage stresses induced in the arch components due to cantilever construction;
b) Lifting the tips of both the arches thereby reducing deflection of the arch portion extending beyond the previous forestay cable;
c) Bringing the arch shape to the final expected theoretical curve by creating the necessary space to fit the final few segments into the position as per their theoretical fabricated lengths.

To employ the arch closure process as planned, the following enabling work components were designed and put into action:

- Strut “C”
- Tripod “B”
- Strut “E”

The Strut ‘C’ and its components were primarily used to bring both the arch arms in perfect horizontal alignment in plan and to push the arch tips apart in elevation to create space for installing the bottom chord segment, Figure 21. It also lifted the arch tips slightly by pushing them against each other.

The Tripod ‘B’ was used for doing level adjustment of the arch tips which were at different elevations.

By putting Strut C and Tripod B in action, the final bottom chord segment, bracings above them and two out of the last three remaining top chord segments could be put in place.

The central crown segment is also placed in this step.

The Y-shaped Strut ‘E’ was then fitted in place. It was used to apply the final jacking force on the top chord of the arch and push them further up and apart to fit in the final top chord segment, Figure 22.

After placing this segment, the remaining 23 pieces of the wind bracings were fitted in their positions.

After the placement of all the wind bracings, the arch closure process was completed. This was followed by the dismantling of the remaining temporary works.

Figure 20: Arch closure in progress

Figure 21: Photo showing Tripod "B" and Strut "C"

For the construction teams at site, the planning of the arch closure process was undertaken by thoroughly checking the weather forecast from authentic meteorological labs.

Though the enabling work components were designed for a wind speed going up to 39m/s, the overall lifting operations and the execution process were planned to be carried out only in the period when wind speed was below 15 m/s.

6. ARCH BOX CONCRETING

There was a requirement for concrete pouring inside the arch segments (boxes) after erection to keep the arch box section still in the lateral direction to cater to the onerous wind loads.

There was a possibility that the stresses in the plates of the boxes might increase due to the lateral pressure of the concrete, and it may result in damage to joints between the plates of the box as well as the failure of the plate.

Design documents suggested that the stresses in any of the plates of box segments should not exceed 25 N/mm^2 during concrete pouring.

Special analyses were carried out in Staad Pro software for the lateral concrete pressure loads and performed the 3D Finite Element Analysis (FEA) on arch segments to find out the behaviour of stresses in plates due to concrete lateral pressure.

Analysis was performed for each of the mentioned groups; extracted the analysis results from Staad Software.

Bending Moment along the arch axis and across the arch axis, the stresses were calculated using these bending moments for all the group of segments.

Figure 22: Photo showing Arch closure in the process: Strut C, Tripod B and Strut E (Blue colour)

Figure 23: View of the completed arch

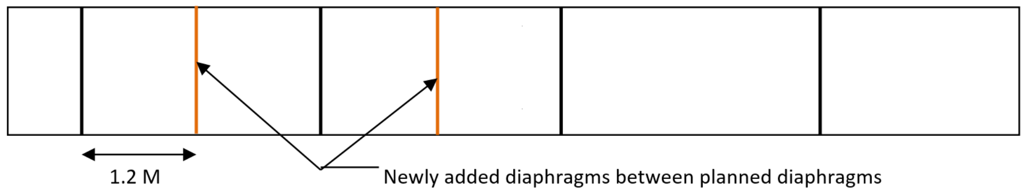


Figure 24: Modified arrangement of Diaphragms

The following remedial measures were suggested after the analysis was performed:

a) Additional intermediate diaphragms were provided in the members between the existing designed diaphragms, Figure 25;
b) Lateral pressure during the concrete pour was calculated according to the CIRIA guide. The rate of pouring of concrete was controlled according to the limiting stress requirement.

7. CONCLUSION

The magnitude of fabrication and the level of accuracy were very high hence methods were adopted in such a way that the given system would work for the entire geometry of the arch.

Accuracy, safety, time and cost savings required are always high considering the challenges of construction and transportation in tough terrain such as these.

Co-ordination between all the stakeholders – from the designer up to the site execution team, through the communication channels of support services have been of utmost importance.

The cable crane operator, the erection team, the planning, the design team and all the support function teams have played a crucial role in helping and accomplishing the arch erection process as planned.

Arch design and construction required complex erection stage analysis (ESA) which was carried out for every stage of the cantilever construction of the arch and incremental launching.

Constant exchange of monitoring and survey data between the site teams and designers enabled full control and immediate corrections, resulting in an imposing final structure within all tolerances.

The pandemic has brought with it numerous uncertainties and we are slowly but steadily overcoming this challenge. Remote working, keeping the morale of the site team high and seeing the light at the end of the tunnel, learning new techniques of online communications while delivering projects, use of technology for day-to-day design and drawing activities.

The inclement and unpredictable weather in this region of Jammu has been one of the toughest challenges on this project.

Figure 25: Photo showing celebration on arch closure completion with Indian Tricolour

INCREMENTAL LAUNCHING OF SUPERSTRUCTURE FOR THE CHENAB BRIDGE

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1. INTRODUCTION

Udhampur-Srinagar-Baramulla Rail Link (USBRL) Project is a project of National importance of the Government of India for providing rail connectivity from the Kashmir Valley to the mainland of India. The project comprises complex engineering structures in the seismically active Himalaya terrain.

The Chenab Railway Bridge is the most critical and toughest project in the Udhampur Srinagar Baramulla Rail line project.

The challenge to construct this bridge superstructure of 1.315 km in length and at a maximum height of 359 m above the river water level in an isolated terrain was an equally difficult task.

From Reasi to the Chenab Bridge site, the approximately 50 km long road was predominantly single lane with steep gradients and sharp curves making access to the site very difficult.

It was therefore decided to establish state-of-the-art fabrication facilities at both the Kauri and Bakkal sides of the bridge to specifically fabricate the deck segments.

Transportation of deck segments of maximum sizes 17.5 m x 8.33 m x 4.5 m (100 tonnes) was carried out using a special multiaxial and hydraulically steered trailer, from the fabrication shops to the launching pads.

Superstructure orthotropic deck on the bridge erection was done by incremental launching method. The method was divided into Incremental Launching on Viaduct Spans and over Arch spans.

Launching platforms were constructed at site positions P180, P70 and P10 to launch the segments by an incremental method. Launching platforms were equipped with segment welding and painting facilities.

The arrangement included a pulling device erected on a stiff pier/abutment, a pulling arm and backing beam behind every end welded segment.

The methodology for the launching of segments included in-depth planning for the lifting of segments from ground level and placing them on the launching platform situated 39 m above ground, maintaining the launching reactions within the limit which would occur due to varying camber of segments, controlling deflection of the tall steel piers and method of launching to complete with Safety, Quality and saving Cost and time.

The launching of Arch Span segments of 17 m width above the tall steel piers ranging from 55 m to 131 m amid the hostile weather at the site was a major challenge. The viaduct span deck was launched incrementally on a combined circular and transition curve for the possibly first time in the world.

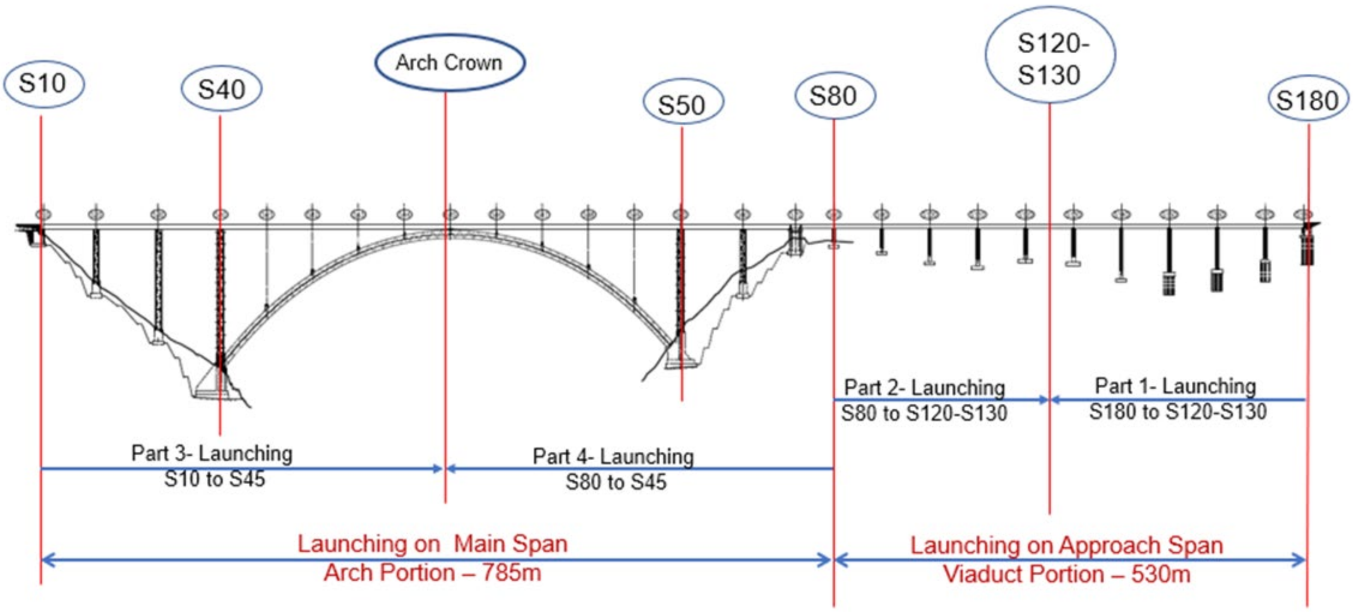

Figure 1: Incremental Launching Scheme in Viaduct and Arch Bridges

In this article, we would like to highlight the methods and arrangements made to overcome the challenges during the curved and transition launching from Abutment S180 and the launching of main span segments over the Central Arch Portion from Abutment S10 and pier S70.

2. SUPERSTRUCTURE

The expansion joint at support S70 divides the steel superstructure into two separated bridges i.e. Viaduct and Arch bridges, see Figure 1.

The deck is continuous in each portion and special rail and bridge expansion joints are provided at S10, S70 and S180 locations.

The cross-sections are slightly varied. The deck of the viaduct bridge is a conventional two I-girders cross-section where the deck plate works as an upper flange, and the depth of the deck is 3.8 m.

The cross-section of the arch bridge is similar, but the depth is 4.5m and the deck plate is 3.5 m wider than the viaduct bridge.

The cross-section of the arch bridge includes wind plates on both sides. They improve deck behaviour against the wind. Both cross-sections have lateral bracing at the level of the bottom flange.

Span lengths in the viaduct bridge vary from 40 m to 50 m and in the arch bridge from 48 m to 65 m.

The cross sections are designed for two railway lines. In the beginning, the railway is placed in the middle of the deck and later when two lines will be operating, the railway lines will be installed with a 5.5 m transverse spacing.

3. INCREMENTAL LAUNCHING

Superstructure erection was done by the incremental launching method. The method was divided into incremental launching on viaduct spans and over the arch spans.

The viaduct portion construction of super-structures was done in parallel with the arch erection since those were independent activities.

The incremental launching of the main span segments over the arch commenced after completing the erection of the arch, long steel piers and arch piers.

The incremental launching scheme was divided as follows, see also Figure 1.

1. Incremental Launching over the Viaduct Spans. The segment incremental launching is further divided as:

   Part 1 - Curved incremental launching from S180 to mid of S120 and S130.

   Part 2 - Straight Incremental launching from S80 to mid of S120 and S130

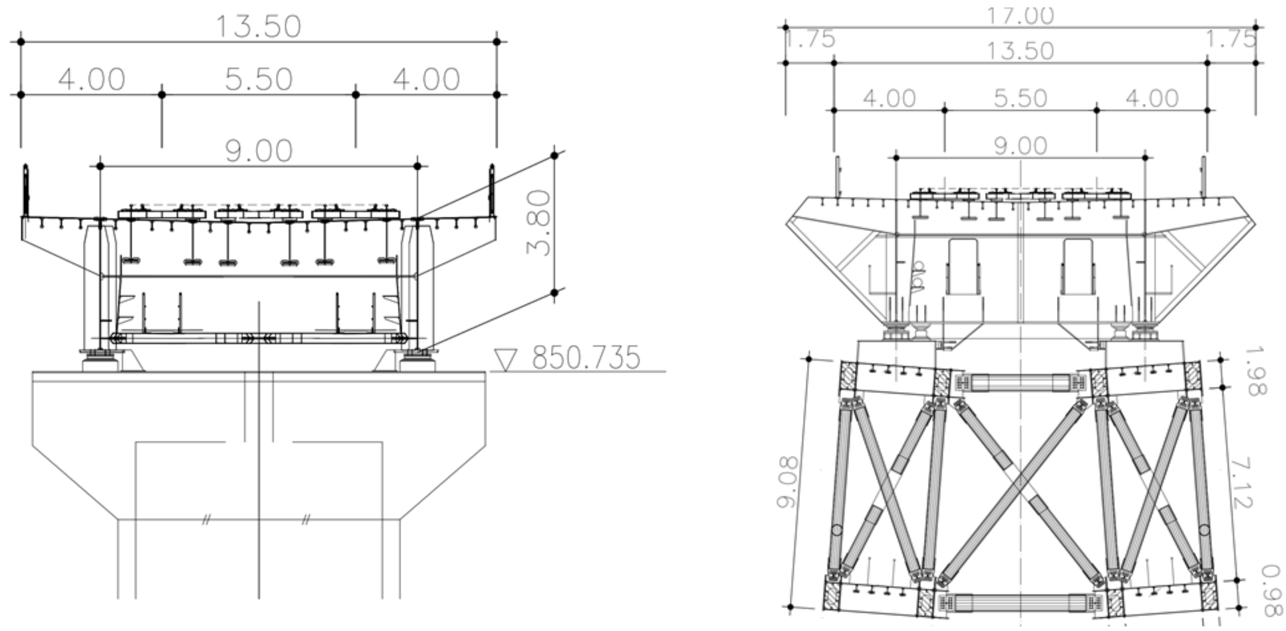

Figure: 2 Cross sections, Viaduct bridge on left and Arch bridge on right

2. Incremental Launching over the Arch. The segment incremental launching is further divided as:

   Part 3 - Straight Incremental launching over the arch section from abutment S10 to S45

   Part 4 - Straight Incremental launching over the arch section from pier S80 to S45

Both the deck on the viaduct and arch sections were separately incrementally launched from both sides.

INCREMENTAL LAUNCHING OVER VIADUCT SPANS

The viaduct portion of the bridge comprises a bridge axis circular curve (Radius 638.686 m), a transition curve and a straight section. Since the project is in hilly terrain and the deck is designed as a continuous deck over supports, it was decided to adopt incremental launching for the construction of the deck.

The incremental launching was carried out in two stages from both ends. In the first stage, the deck

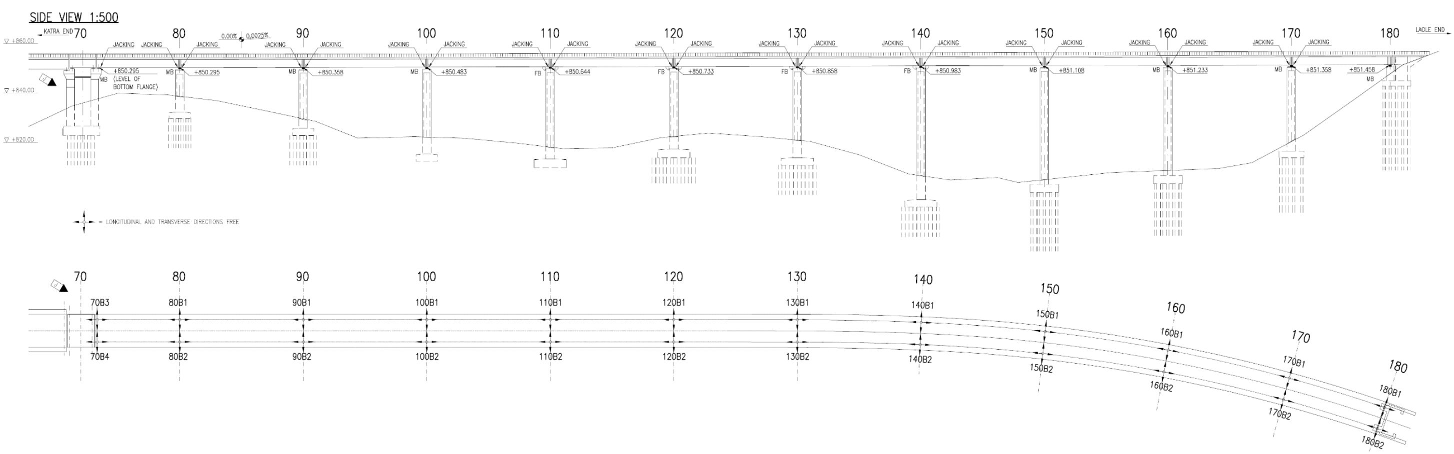

Figure 3: Viaduct Span Arrangement
Click on the image to open it in a higher resolution

was launched from the S180 abutment on a combined circular and transition curve and in the second stage, the deck was launched from the S70 pier on the straight portion as shown in Figure 3.

The segments from both ends reached the middle of the S130 - S120 span, and the deflection and camber values from both ends at several intervals were measured. Subsequently, the viaduct section was connected with the HSFG bolts and the spans were lowered from the temporary sliding bearing onto permanent spherical bearings.

Part 1 - Curved incremental launching from S180 to middle of S120 &andS130

In the curved spans, incremental launching of the deck was carried out on a combined circular and transition curve by fabricating the deck following the alignment of the bridge and launching the deck along a predefined launching curve using central guidance and wide temporary bearings.

This innovative method was developed by the contractor and had probably never been used before.

The deck was fabricated and joined as per the designed horizontal alignment.

However, the launching was carried out on a theoretical launching curve of radius 720.894 m which had been determined by trial and error.

The central guide beam was connected to the lateral bracing structure under the segment along the theoretical curve as shown in Figure 4.

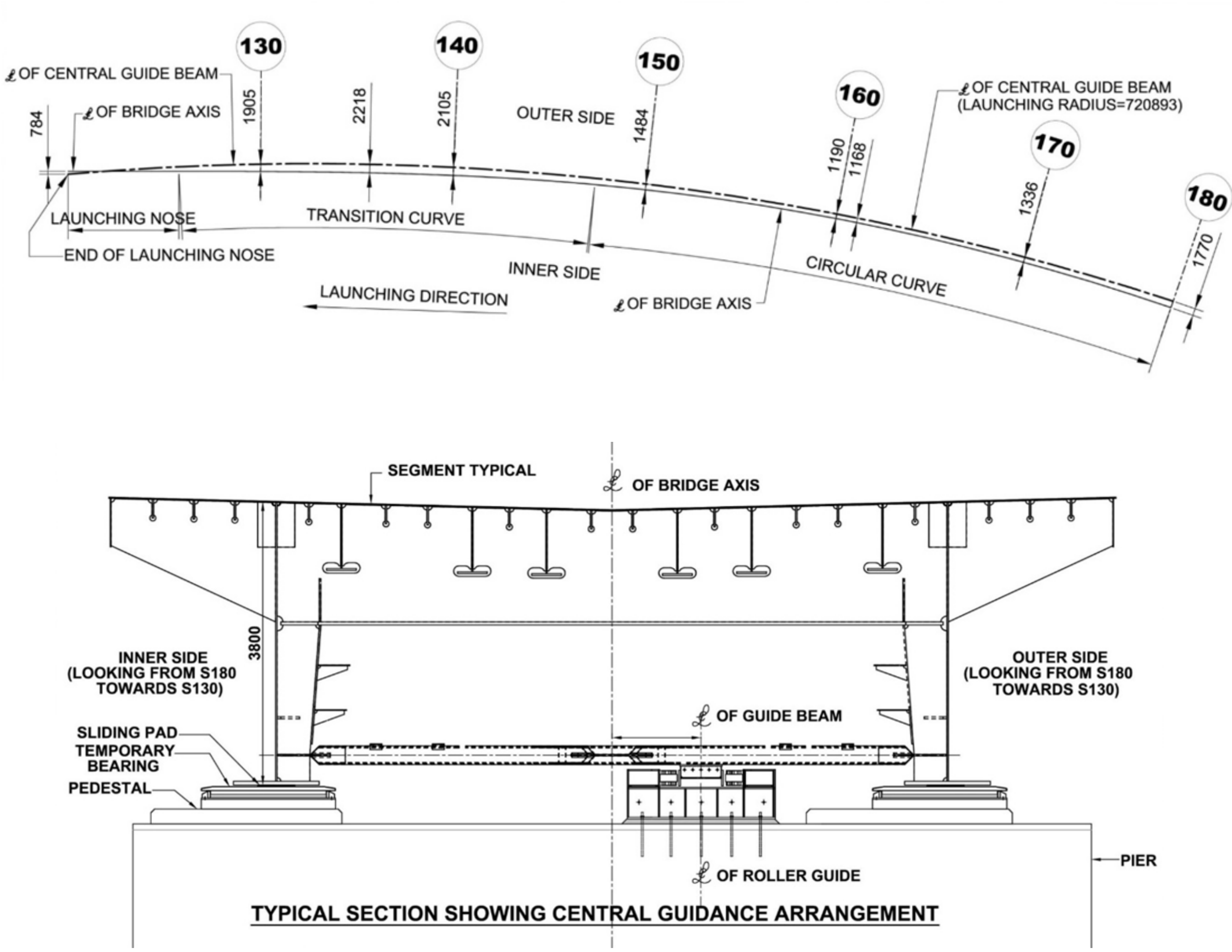

Figure 4: Designed bridge axis curve and Theoretical launching curve offsets to central Guidance

Figure 5: Launching Nose Jack (left) and Curved Launching (right)

The segments were pushed using jacks as the central guidance system following the launching radius as per the design.

At supports, the lateral position of the deck was changing all the time. This was possible because of the wide sliding pads.

The completed launched deck is shown in Figure 5 (right).

Launching noses were connected at the front of the segments to reduce the cantilever moments of the viaduct segments during the launching operation. For the viaduct section, three lightweight launching noses 9m long each were designed.

The three launching noses were split as marked LN1, LN2, and LN3 of 9 m x 9 m with a tapered height from 1.5 m at front of LN1 to 3.8 m at end of LN3 for the viaduct section and special jacking arrangement was provided at the tip of LN1 to correct the vertical alignment, Figure 5 (left).

In cantilever construction during launching, the launching nose dips below the pier cap level hence the jack provided at the tip of LN1 helped to lift the launching nose front and placed it on the temporary bearing of the pier to enable smooth progress of the launching sequence.

The stability of the deck segment during maximum cantilever conditions was considered in all spans and reactions on every support were assessed.

The tabulated summary was prepared which helped the site to monitor the launching activities and keep the track of launching steps, segment weights at each stage, pulling force required in jacks, and maximum pressure required in the jack for the respective launching step.

A fabrication yard was set up behind the S180 abutment where the segments were fabricated, aligned and arranged in line for the incremental push launching.

The fabrication yard was fully equipped with a 120 MT (metric tonnes) EOT crane for the segment handling facility. Fabricated segments of 3 m – 3.5 m lengths were arranged in a line and welded together.

Touch-up painting required at the edge locations was carried out and placed on the launching bed.

Push launching was done once the segments were aligned as per the bridge axis and another segment was added behind.

A temporary backing beam was placed behind the segment and push launching was carried out using the two 125 MT hydraulic jacks placed on the launching bed as shown in Figure 6.

Next segments were added and welded and pushed using hydraulic jacks.

Figure 6: Push Launching arrangement behind the Abutment S180

Part 2 - Straight Incremental launching from S80 to middle of S120 and S130

The segments were launched from the top level of the S80 pier which was 26 m above ground level. The temporary platform with a segment lifting capacity of 120 MT was designed for placing the segments at the S80 location.

This temporary platform was used for other works such as a gantry track for the movement of the 120 MT gantry, placing the viaduct segments, welding of segments, sandblasting/painting of segments and launching of segments, Figure 7.

The platform is designed so that the same platform can be used for launching the main span segments from S70 to S45 over the Arch.

The erection of this 120 MT capacity portal gantry above the launching platform at a height of 26 m from the ground was a major task.

A special crib system was designed to support the components of the 120 MT portal gantry during erection, Figure 8.

A total of 33 viaduct segments varying from 80 MT to 100 MT were launched from the S80 platform.

The platform was designed for lifting segments from the trailer and placing them in alignment on the platform.

The segments were welded to a previously erected segment on the platform. Weld testing and touch-up painting can also be done on the platform.

Segments were placed on the Teflon sliding arrangement supported on the platform columns.

The platform was designed to carry the load of four segments which were placed in-line welded together and pulled from the arrangement placed on the S90 pier.

Segments launched from the S80 platforms were fabricated at the yard behind the S180 abutment.

The deck segments of maximum size 17.5 m X 8.33 m X 4.5 m (100 tonnes) posed the greatest transportation challenge which was mastered by special multiaxial and hydraulically steered trailers, Figure 9.

The segment was loaded on a trailer in the segment fabrication workshop at the S180 location.

Figure 7: Launching platform at S80 location

Figure 8: Gantry erection above launching platform

Figure 9: Transportation of segments

The trailer route was checked for the movement of the Tractor Unit connected with the trailer up to the S80 location.

The arrangement of the launching platform was designed so that it would not obstruct the movement of the trailer.

The segments were transported from the S180 location and brought under the 120MT EOT gantry placed on the launching platform at S80.

There was a limitation of space availability under the launching platform hence segments were lifted in a direction of minimum width (cross direction) to avoid clashing with the columns of the platform, Figure 10, thereafter the segments were rotated using the swivel hook arrangement of gantry above platform level, Figure 11, and placed in launching direction according to the bridge axis.

PULLING ARRANGEMENTS FOR INCREMENTAL LAUNCHING

This arrangement includes a pulling arm and a backing beam. The pulling arm was fixed to the pier through Dywidag bars.

The hydraulic jacks were mounted on this pulling arm. The backing beam was behind every last segment of the respective launching stage.

The strands on the other side were locked behind this backing beam.

When strands were pulled due to jack operation, this backing beam pushed the segment in the forward direction.

The pulling device was erected on the S90 pier for launching segments from S80 to S120, Figure 12.

Figure 10: Segment lifting in cross direction

Figure 11: Rotation of segment

Figure 12: Pulling frame at S90 pier and S10 Abutment

Subsequently, each new segment was added behind the previously launched segment and the backing beam was placed behind the new segment for pulling from the S90 pier. Lateral guidance was placed on every pier hence the segments were launched in a straight line.

The total weight of the segments launched including the launching nose weight equals 2005MT. Considering the 7% friction resisting force, a jacking force of 140MT was required. Hence two 200MT jacks allowing for a factor of safety were mounted on the S90 pier for the pulling operation.

DISMANTLING OF LAUNCHING NOSES BETWEEN S130-S120

The launching noses were dismantled in parts when they reached the already launched segments of the span from S180 to S130.

The dismantling was done using the 60MT capacity crane.

First, the bracings were dismantled, then the near side girder of the launching nose was dismantled and then the far side girder of the launching nose was dismantled.

Figure 13: LN dismantling in progress

CLOSURE OF SEGMENTS BETWEEN S130-S120

After removing the launching noses, the launching of the remaining segments was completed. The first segment of the already launched span from S180 to S130.

AS34 between pier S130 and S120 was joined with the first segment of launching from S80 to S120. AS33. Both ends of segments were aligned for matching of splicing holes, Figure 14.

Splice plate connection was carried out through HSFG bolts connecting the segments.

LAUNCHING OF STRAIGHT ARCH MAIN SPANS

In the original tender design concept the erection of steel deck and arch members was considered to be carried out by using a derrick with a capacity of 90 tonnes.

This concept was based on the idea to avoid cutting arch elements into small pieces, since the capacity of the cable crane was limited to 30 tonnes, see Figure 15.

In the final design stage, the erection concept was reconsidered and the erection method was changed.

It was decided to carry out the erection of the arch by the cable crane and the launching of the deck from two sides over the arch, see Figure 20.

Figure 14: Splicing of segment AS33 - AS34

INCREMENTAL LAUNCHING OVER THE ARCH

The segment incremental launching was further divided as:

Part 3 - Straight Incremental launching over the Arch portion from Abutment S10 to S45

Part 4 - Straight Incremental launching over the Arch portion from pier S80 to S45

From Abutment S10 to mid-length of the Arch Span (Part 3) eight spans with 51 segments were launched. The main span segments were pulled from the specifically designed pulling frame mounted on the S10 abutment.

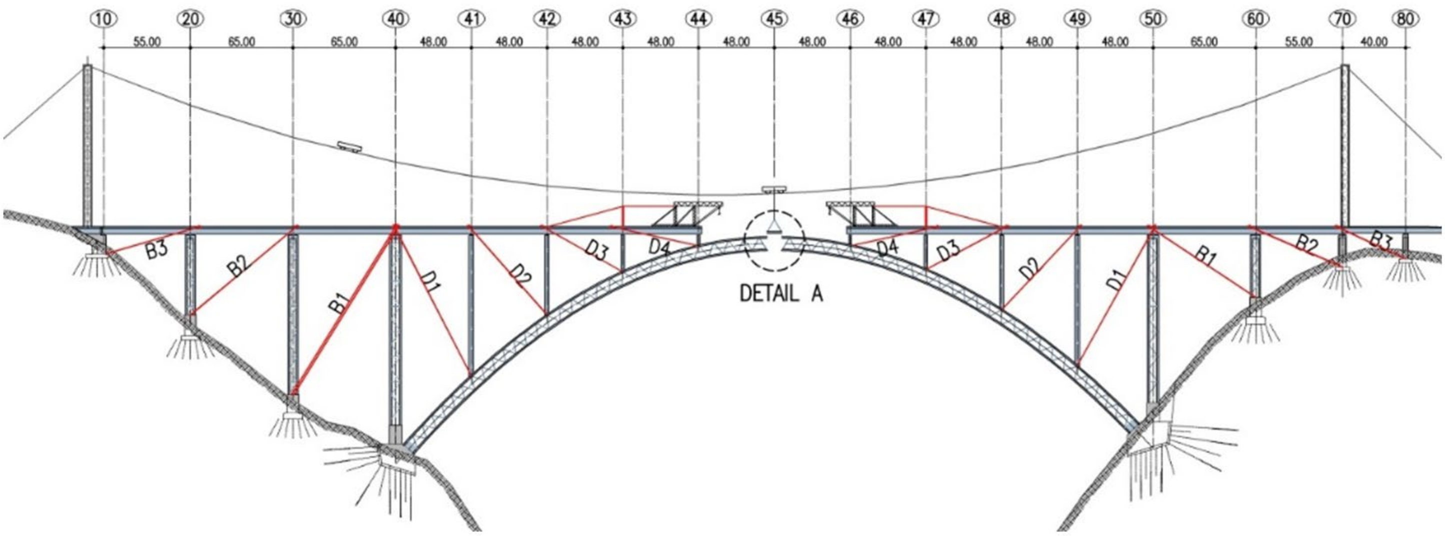

Figure 15: Original Erection procedure by Derrick lift

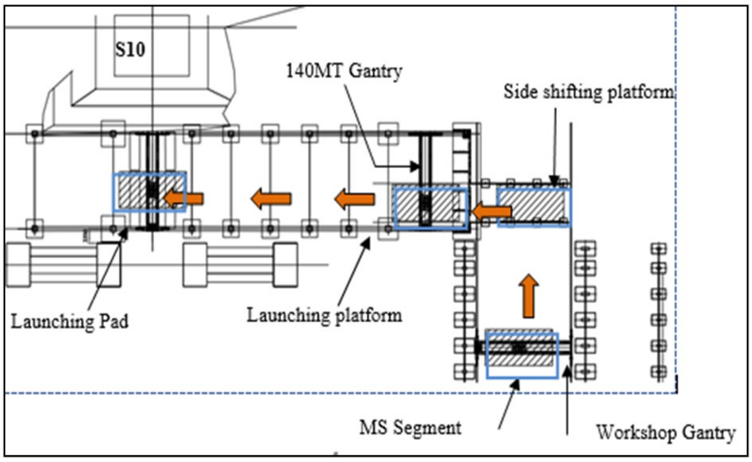

Figure 16: Movement of segment from workshop to launching pad and segment transportation using 140MT gantry

The segments were shifted out of the fabrication workshop using the workshop gantry and then placed on the moving trolley kept on the segment side shifting platform.

After that, they were shifted to reach the 140MT gantry placed on the launching platform.

The segments were then lifted from ground level by the gantry and moved on the segment transportation platform to the launching pad level and moved forward to be placed on the assembly pads using the gantry, Figure 16.

The segments were launched from the top level of the S10 abutment, which is approximately 39m above the fabrication shop level.

The temporary platform with a 140-tonne EOT gantry was designed at S10 for moving and placing 120-tonne segments, Figure 17.

Welding of segments and launching of segments was carried out on the launching platform.

Figure 17: Temporary Platform at S10 Location

Figure 18: Establishment of Main launching pad with parallel assembly pads

The location of the launching platform was decided in such a way that the segment could be transported from the workshop to the launching pad without using a trailer for carrying the segment.

To expedite the launching process and increase the launching production, two parallel welding assembly pads were established for segment-to-segment fit up and welding on both sides of the launching pad at the S10 location, Figure 18.

Two segments were placed on assembly Pad 1 and two segments were placed on assembly Pad 2. The fit up and welding between the two segments were carried in parallel on respective assembly pads.

Hence, four segments on two Pads were completed on the assembly Pad in parallel with the welding/launching of additional segments already placed/side shifted on the main launching Pad.

After all piers were erected, the pier heads were connected to each other by stressing horizontal temporary ties between them. Tendons were anchored to end supports and the arch crown.

In this way, friction forces created by launching were transferred to stiff supports and the bending of piers was minimized.

The proposed launching method was employed with specially designed temporary launching bearings called “Tandem Bearings” which were fixed at all pier locations for smooth and precise Launching.

The tandem bearings would distribute vertical loadings smoothly to a sufficiently wide length of the web of segments.

This prevents yielding and/or local buckling failure of the web of the main bridge segments and also provides a more stable and larger bearing area during launching. Bearings were provided with adequate rotation capability.

Low kinetic friction was achieved using Teflon pads inserted between the stainless steel slide plates of the bearing and the bottom flange of the segment.

Figure 19: Tandem Bearing

Figure 20: Launching on the arch bridge

Smooth stress distribution was achieved using 50mm thick elastomeric bearing pads between the top flange of the bearing and the stainless-steel plate, Figure 19.

After finalizing the stressing of horizontal temporary ties, the allowable deformation at the pier's head was maintained within the limit given by the designer and readjustment of the force was done in the horizontal temporary ties if required.

Tall piers over the arch at S41/49 and S42/48 were braced with a temporary strut arrangement to keep the end connections of trestles safe during tie cable stressing and launching, Figure 20.

Launching of the superstructure was done simultaneously from both ends, i.e. S10 from the Bakkal side (Part 3) and S70 from the Kauri side (Part 4). One side was permitted to be ahead of the other by a maximum of two spans to limit the asymmetric bending of the arch.

Finally, after joining the superstructure from both sides by the “Golden Joint”, the bridge was structurally complete and paved the way for the construction of the track laying and associated activities.

Figure 21: Deck launching over arch span

CHENAB BRIDGE – CONSTRUCTION PHOTOS


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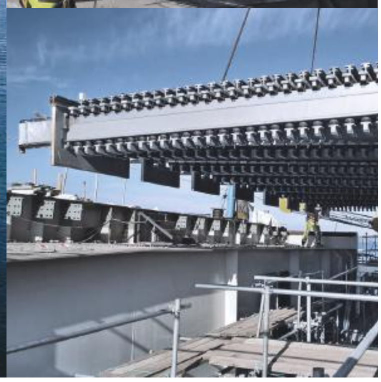

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References:

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- Hochmoselübergang, Germany
- Osman Gazi Bridge, Izmit, Turkey
- Mainbrücke Randersacker, Germany
- Millau Viaduct, France
- Rheinbrücke Schierstein, Germany
- Rion Antirion, Greece
- Russky Island Brigde, Vladivostok, Russia
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forces in motion

Helgeland Bridge, Norway

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*Wyatt Brooks and Kevin Donovan - "Eliminating Uncertainty in Market Access: The Impact of New Bridges in Rural Nicaragua," 2017.

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DESIGN & CONSTRUCTION PART I.