e-mosty

ISSUE 02/2023 JUNE CHENAB BRIDGE PART II.

FABRICATION AND ASSEMBLY

LIST OF CONTENTS

Front Cover: Installation of the first 60m diagonal. Photo Credit: Frank Bauchspiess

Back Cover: Skytrucks in tandem operation during arch construction. Photo Credit: Afcons Infrastructure Ltd, India

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Dear Readers

*This issue of the e-mosty magazine is the 2nd part of the special series dedicated to the **Chenab Bridge in India**.*

The 1st part was published in March; we brought information about the Project, the Design of the Bridge, Incremental Launching and Arch Construction. You can read it here.

In the May e-BrIM, you can read the article "Use of BIM in Design and Construction of Chenab Bridge in India" by Matti-Esko Järvenpää, Director of WSP Finland.

In the September Edition of e-mosty, we are planning to publish the last part of the series, an article about Wind Engineering.

*In this **special edition of e-mosty, you can find six articles** in which the authors describe in detail the fabrication and assembly of the bridge; the world's largest capacity crossbar cable cranes used for moving and positioning the heaviest parts of the arch and piers; new-generation material which was used on the arch base plates and in bearing top-girder bottom interfaces; and the engineering, supply, installation, and subsequent dismantling of 56 stay cables for temporary use during the arch erection.*

The bridge is equipped with a state-of-the-art structural health monitoring system which ensures safe operations at all times. You can read about it in the last article of this edition.

*I would like to thank **Richard Cooke** for the review and assistance with the content, and all the **authors, people and companies** that have been helping me put the content together.*

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

The next e-mosty magazine will be released on 20th September 2023. We are planning to feature the Douro Arch Bridge in Portugal in cooperation with Arenas y Asociados.

The next e-BrIM magazine will be released on 20th October 2023. We still welcome your articles for this edition. Please contact me at magda@e-mosty.cz to discuss the details.

You can also read a special edition of the e-maritime magazine in which we published the article Caissons of Wind Turbine Generator Foundations, originally written as a book chapter. It also comprises five Case Studies including 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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FABRICATION OF THE CHENAB RAILWAY BRIDGE

Dipl. Ing. Frank Bauchspiess

Chief Engineer, Fabrication and Erection for the Chenab Railway Bridge

Figure 1: A view of the construction site

INTRODUCTION

This article provides a brief overview of the fabrication of the steel segments for the Chenab Railway Bridge – the longest steel arch railway bridge in the world.

The Chenab Railway Bridge with two framework arches, steel piers, a viaduct and a superstructure is one of the world-famous bridges located in the Indian Pre–Himalaya area.

It is a great engineering design from WSP Finland and Leonhardt, Andrä und Partner, Germany (you can read about it in the e-mosty March 2023).

I am sure the Chenab Railway Bridge will get its place as one of the most important and famous bridges in the world.

FABRICATION

Workshops

The fabrication and assembly of steel bridge segments are typically realized in workshops, remote from the site.

After welding and painting, the single segments are transported to the site for their final assembly and installation in the steel bridge.

The site for the Chenab Railway Bridge is located in the Himalayan region, about 50 km from the border with Pakistan, with no convenient connections to the Indian highways.

The roads to access the site where the bridge is located were still being built by the Indian Border Road Organization.

That is why conventional road transport of pre-fabricated steel segments to the site was not possible.

As a result, the company AFCONS has to establish complete new workshops for steel and segment fabrication including all required equipment.

The dimensions of those workshops are around 50 m x 100 m x 40 m in height.

After building the workshops, the fabrication manager established the position of single working stations for cutting, tacking and welding for single segment parts.

Figure 2: Camp on site for engineers and workers

The single parts were assembled with a special method – technology in special areas in the workshop.

It was developed together with welding engineers.

The single workstations in the workshop had to be in line with effective time cycle fabrication and had to cater for the possibility of lifting and transportation with gantry cranes from station to station.

For transportation of the steel plates, single parts of the segments and finished segments, gantry cranes were installed in all workshops.

Figure 3: Building new roads in the mountains on the way to site for the Chenab Railway Bridge

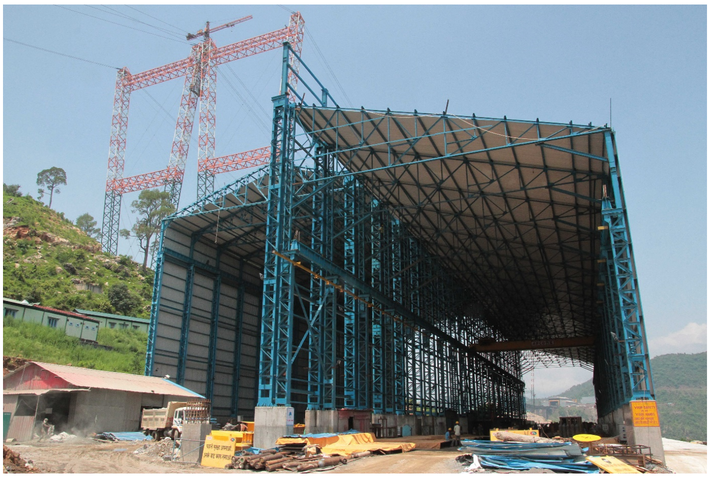


Figure 4: Steel frame construction for the arch workshop

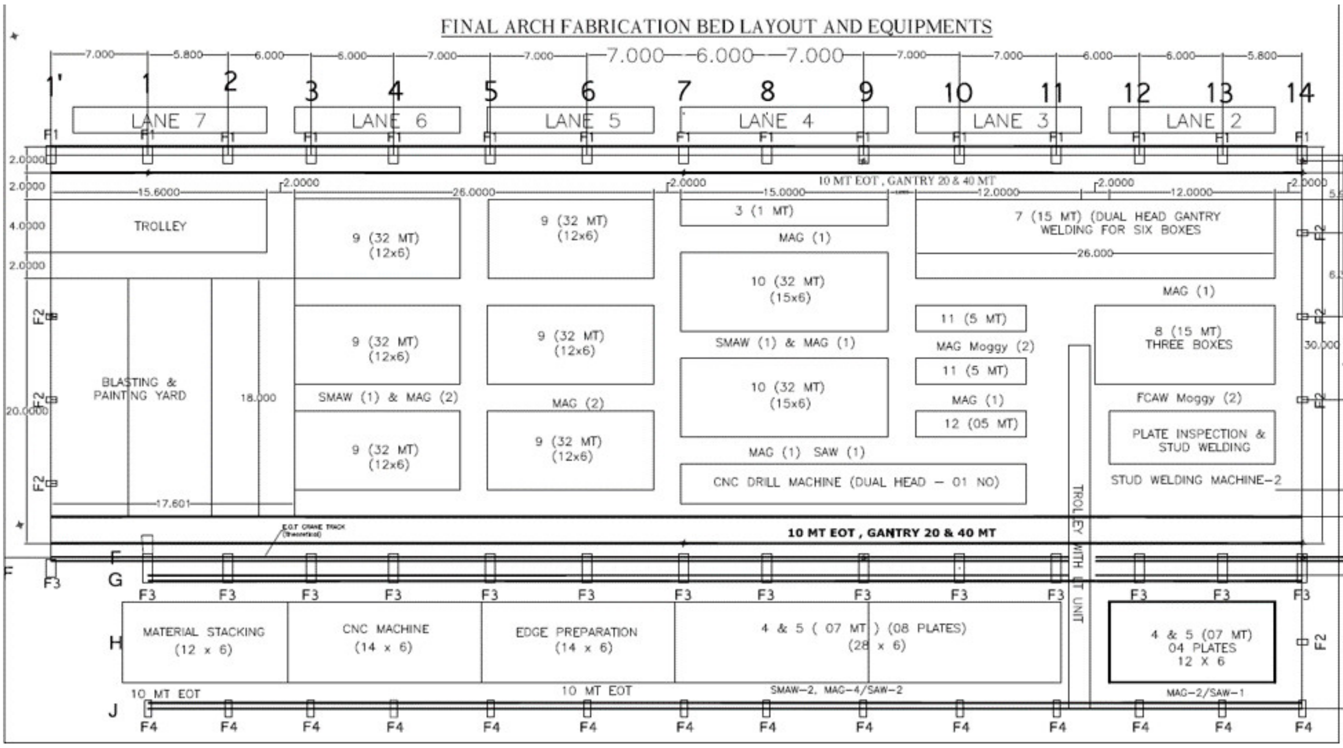


Figure 5: Sketch of the fabrication workshop – plan view

For the rotation of the single segments of the superstructure, a special rotation frame was designed, welded and assembled in the workshop for the superstructure.

All materials and equipment had to be transported to the site and assembled by the workers who lived in houses directly on site.

On site, the following workshops were located:

- Fabrication workshop on the Katra side for the superstructure;
- Fabrication workshop on the Bakkal side for the superstructure;
- Fabrication workshop for steel piers in Surandi (15 km from the site);
- Fabrication workshop for arch segments on the Katra side.

For the fabrication and assembly works, the company AFCONS provided all managerial and technological support in this difficult area.

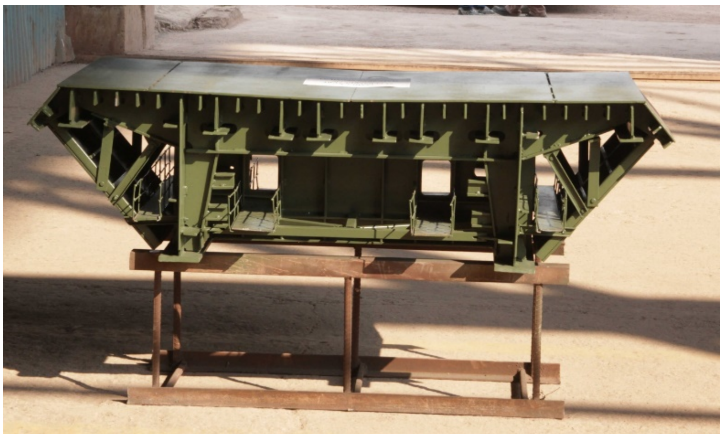


Figure 6: Fabricated steel model for the superstructure

Figure 7: Construction sites

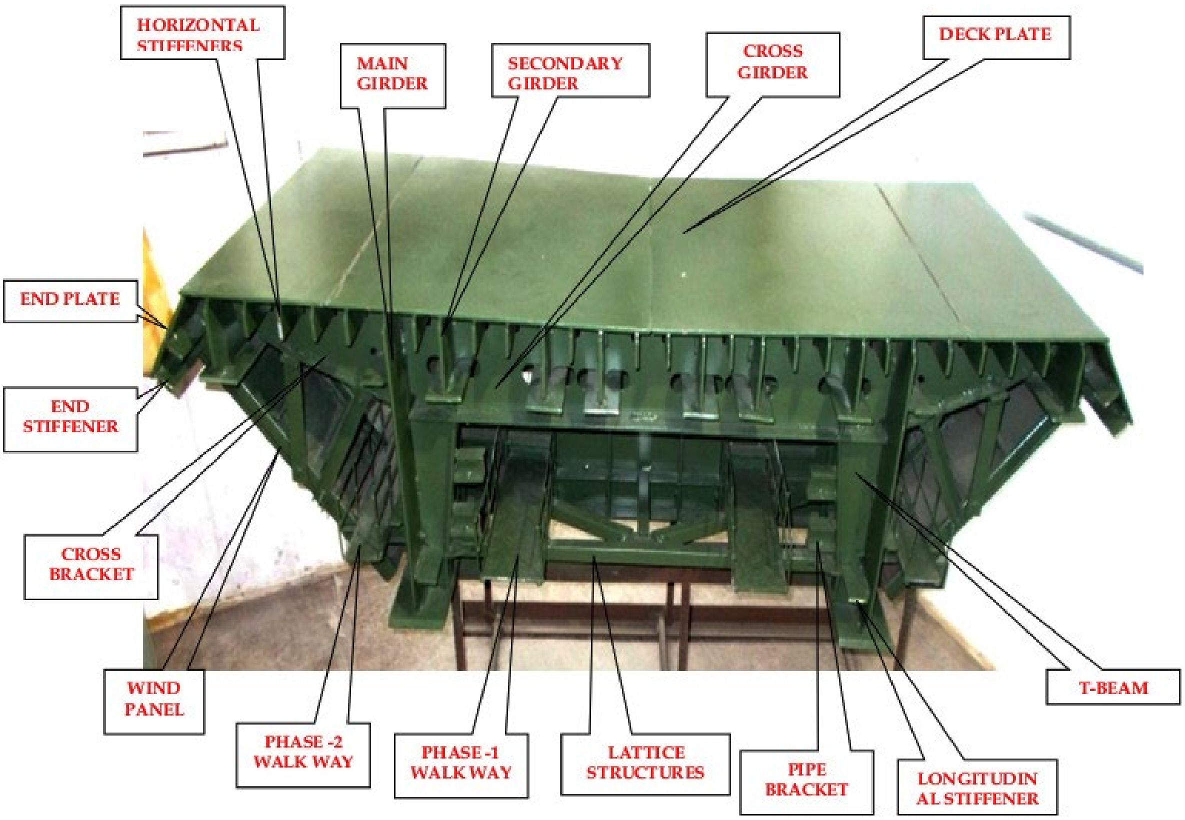


Figure 8: Parts of the segment for the superstructure

Material

The Chenab Railway Bridge was designed according to British Standard BS 5400 (Code of practice for the design and construction of steel, concrete and composite bridges) in all details, however, the load for trains used IRS – Indian Railway Standard.

The steel which was required for the bridge was:

- Viaduct and superstructure arch: E250C according to IS 2062 (Indian Standard for Hot Rolled Medium and High Tensile Structural Steel – Specification);
- Steel piers and arch: E410C according to IS 2062.

Both these steel grades are unknown in Europe.

For example, detailed discussions were held from the start of fabrication and during the fabrication about the permitted thickness of the steel plate.

However, all the differences between the various code requirements had no significant influence on the real bridge structure.

Workshop drawings for fabrication and CNC geometry

The workshop drawings were developed by Indian engineers on site in a special design office using a 3D Tekla model. Every steel plate was taken out from the 3D Model as a 2D single plate drawing in ACAD and then taken over to a responsible Indian CNC engineer.

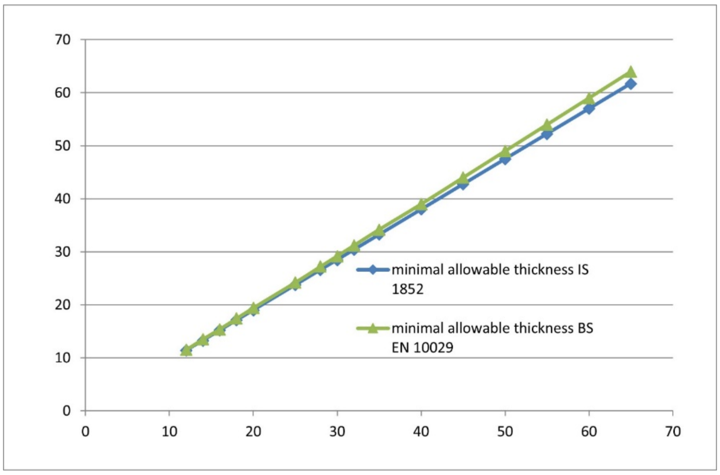


Figure 9: Permitted plate thicknesses in function of plate thicknesses acc. to IS Code and BS EN Code are different

Figure 10: CNC machine – ACAD drawing

Figure 11: Cutting

It was necessary to pay close attention to the fact that the precise cutting geometry could not be taken from the complete 3D computer model.

The first issue to be resolved by the fabrication engineer was the correct cambering of the single bridge segments.

The designer of the 3D Model implemented the geometric outline of the finished bridge under dead load below normal temperature or the geometric outline with the complete camber.

Both geometries are different. It is a fact that is not so well known [1].

The unloaded shape of the bridge for fabrication geometry must be different from the geometric outline under load.

Additionally, the finished camber has nothing to do with the required assembly geometry if the assembly process follows different load situations and different assembly force systems.

In both cases, many misunderstandings can arise among bridge designers.

Generally, using the 3D Model for plate geometry is not the best option.

The responsible fabrication and assembly engineers should have this knowledge and a "feeling" of how big shall be the influence of the finished steel structure as a result of such differences.

They should be alert if the designer of the computer 3D Model explains this shall be the fabrication geometry.

However, for the Chenab Bridge arch, everything went well.

The arch was closed with a geometrically correct closing segment on the top.

For the superstructure, due to its length, this fact was not so much important either.

The responsible engineer on site took over the 2D drawings for the single steel plates and with his experience in CNC cutting and the experience of the welding engineer he made the cutting geometry on both ends longer with:

- 1 mm as a result of CNC flame cut, and
- 1 mm as a result of welding shrinkage.

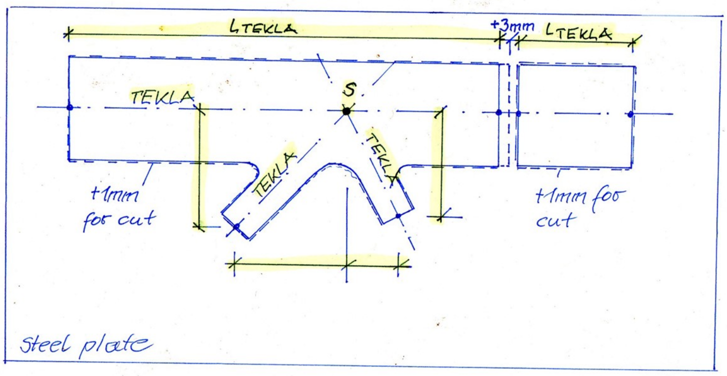


Figure 12: Sketch – web plate for arch segments

Figure 13: Main girder gantry crane

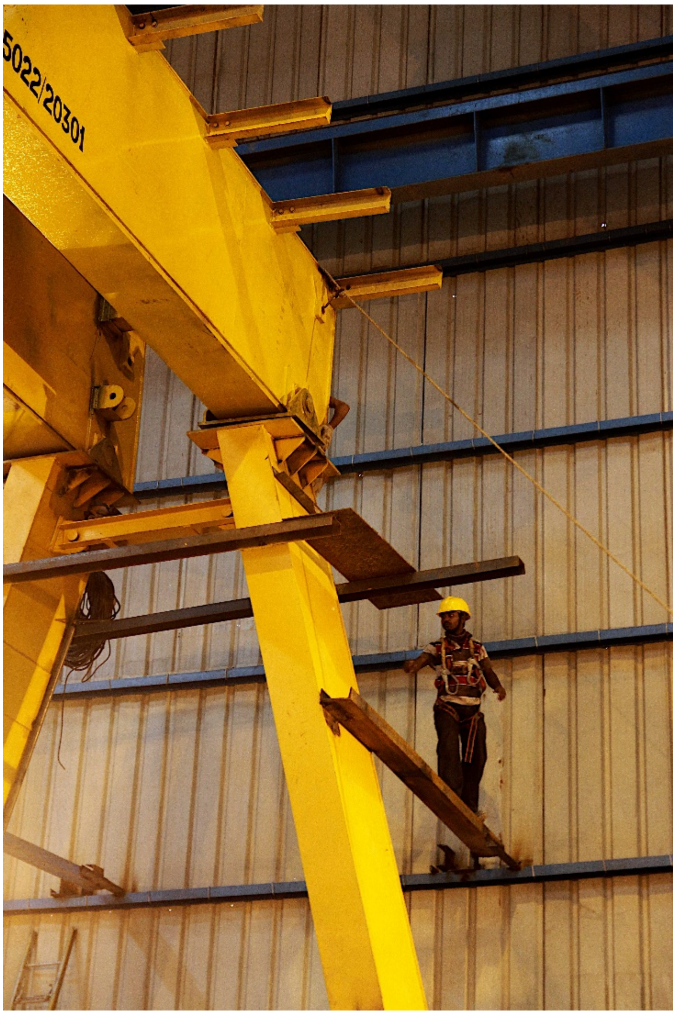


Figure 14: Assembly of gantry crane

Special requirements for workshop equipment on site

The fabrication of the Chenab Railway Bridge required some special equipment that had to be installed by AFCONS on site.

In a common fabrication process for steel bridges, the workshops and equipment are typically located within the fabrication company.

It was a huge challenge to bring all the materials and equipment and install them on the site in a valley in the Pre-Himalaya.

The gantry cranes were not only inside the workshops - for transportation of the steel segments, they were also outside the workshops – for transportation and trial assembly.

The gantry cranes were fabricated, welded and bolted on site by the AFCONS team after concreting the foundations and installation of the rails.

For the welding of the large number of hollow steel sections for the piers, a welding gantry for "up welding" was installed.

The installation of the welding gantry required high-quality welding installations which had to be almost vibration free and in an exact plane position for the successful welds of the welding torch.

The connections of piers and arch during assembly are bolted connections.

For the segments and splice plates drilling holes were required. For the complete Chenab Railway Bridge, there were almost 800,000 bolts. The holes had to be drilled on site.

In the workshop for the piers and arch, a high-capacity drilling machine from Taiwan was installed. It took the 2D ACAD drawings from the computer and drilled the holes in the correct position of the plate edges.

QUALITY MANAGEMENT SYSTEM FOR FABRICATION

General

For supervision of fabrication and welding, AFCONS established a team of Indian QM engineers.

The first task of the QM team was to develop quality criteria for the fabrication of single segments.

The summarization of the defined quality criteria is documented in a Quality Assurance Plan (QAP).

Figure 15: Welding gantry in a workshop for piers

Figure 16: Installed drilling machine in arch workshop

The contract was regulated by the following Codes:

- Indian Code (IS)
- Indian Railway Codes (IRS)
- British Standards, and
- European Codes if these were established in the British Codes

Not all quality standards are the same nor consistent and so differences between the codes had resulted in long discussions between the client and the fabrication team on site.

The following regulations are given by QAP (see the list at the end of the article):

- Material: IS 2062, BS EN 10025
- Bolts: IS 3757, IS 6623ff
- Welding consumables: AWS codes (American welding society)
- Steel segments: BS 5400-6-1999
- Welds, Welding procedure: BS EN 15609-2004
- Edge preparation: BS 5400-6

Those different codes have different expertise and engineering theories as basics. Considerable engineering flexibility shall be required to handle this situation on site between client, supervision and fabrication.

<u>“Production test plates” according to BS 5400-6, 1999</u>

Chapter 5.5 BS 5400-6 defines the numbers of production test plates in tension zones of structure required.

This shall ensure the safety for an equal quality level during the complete fabrication. In contrast, EN 1090 (Structural Steel and Aluminium CE Marking) as a fabrication code such investigations are not required anymore.

For testing the production test plates, AFCONS established a testing laboratory on site. After two years and a long process of certification, this laboratory was accredited by RDSO India (Research Designs and Standards Organization).

Directly on site the team could make all tests for steel and welding which are required for continuous supervision of the fabrication quality – especially the welds.

<u>Testing of welds</u>

The following Codes are defined in the QAP for welds:

- Visual inspection: BS EN ISO 5817
- Ultrasonic Test (UT): BS EN 1714 in combination with BS EN ISO 5817
- Radiographic Test (RT): BS EN 1435 in combination with BS EN 5817

The acceptance criteria for welds in BS EN 5817 have different acceptance levels for weld imperfections.

However, the Indian Steel Bridges Code and the design and fabrication code of British standard BS do not have a definition or implementation of which acceptance level shall be required.

This necessitated research and investigation and together with the designers WSP and LAP, a decision for the document was agreed. Generally, steel structures under almost compression stress do not require the highest quality level for welds.

If the quality level is unnecessarily high, the fabrication costs are increased and in some cases some repairs of welds are made, which may not have been necessary. Not every weld repair makes the welds better, especially in the Heat Affected Zone (HAZ).

The following acceptance levels according to BS EN ISO 5817 were used:

- Viaduct and superstructure: level B
- Steel piers: level C
- Arch: level B

RT (Radiographic test) inspection as a non-destructive test required a stop of fabrication and needed very special safety conditions. In field fabrication, RT shall not be used.

The client and AFCONS QM Team changed the RT inspection in 3D Phases array inspection, called PAUT. With PAUT you get a 3D colour scan of the weld in the computer and the procedure is quick and convenient.

The main issue is that an experienced PAUT inspector for the interpretation of the scans is needed and the definition of which welds are imperfect must be given.

During the fabrication of the Chenab Railway Bridge, no special acceptance code for PAUT existed. That is why as an acceptance code for PAUT, BS EN 1714 (for normal UT – 2D) was defined.

However, this may be too conservative and may require too many welding repairs and it gives almost no possibility and time for an engineering discussion with the client.

As far as I know, the use of 3D Phased array inspection for steel bridges in fabrication and its application on the Chenab Railway Bridge was the first time in the world.

WELDERS

It was not easy to find the required number of qualified welders due to the remoteness of the area near the Pakistan border.

It was necessary to educate and train a large number of welders directly on site with special check procedures under the supervision of the Indian RDSO and the test result from the laboratory on site. Every welder got a certificate according to DIN EN 281-1.

Figure 17: Welder working on welding pieces

Figure 18: Welder qualification according to DIN EN 281-1

Figure 19: Inspection of the welding pieces

DISTORTION AND RESIDUAL STRESS FROM WELDING

For a successful assembly of steel components, the work performance and welding sequence are important. It is also important to have sufficient knowledge and experience.

Every welded structure has distortion and residual stress from welding, see Figure 20.

The knowledge of crack development in fracture mechanic theory shows the important impact of residual stress in structures.

The typical well-known criteria for the design of welded steel structures are sometimes not correctly used, especially with regard to weld thickness based on real stress requirements.

The FEM calculation of cross sections with steel plates is another critical fact. For a good weld calculation, the stresses according to the technical bending theory are required. FEM forces must be transformed into such inner forces.

There are more and more requirements for fully welded steel structures with bigger distortion or/and specifications of residual stress in the structure.

At the same time, pre-heating does not remove the complete residual stress in the structure. The residual stress from welding can only be decreased to a hot yielding point of the steel material.

On site we developed and implemented welding sequences in the function of the stiffness of single welded parts during the welding of the steel components. Those welded sequences were developed under the leading discussion of Dr. Suresh – an international welding expert.

Implemented residual stresses decreased the buckling stability of columns and steel plates.

Fracture mechanics facts are well-known: residual stresses under crack development have an important impact on life cycles.

Fabrication Tolerances

At the start of fabrication the tolerances according to the BS 5400-6; 1999 were defined. Knowledge about permitted tolerances is important. The permitted tolerances shall be given by the designer, especially for connection points by welding joints on site and/or erection and assembly of single steel segments to the steel structure.

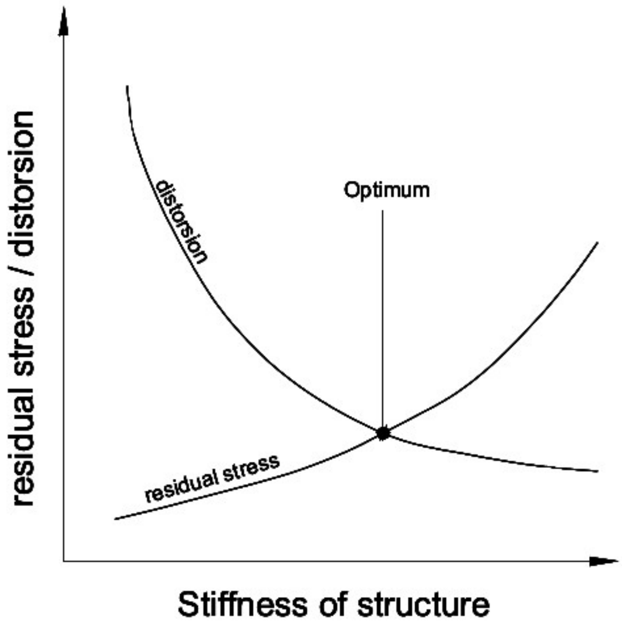


Figure 20: Combination of distortion and residual stress in a welded structure

The code from 1999 was not sufficient for a few tolerances in fabrication and that is why we fabricated and checked the steel segment dimensions in accordance with the tolerances defined by DIN EN 1090-2.

Generally, all fabricated distortions of steel segments of the Chenab Railway Bridge are below the permitted tolerances.

Documentation

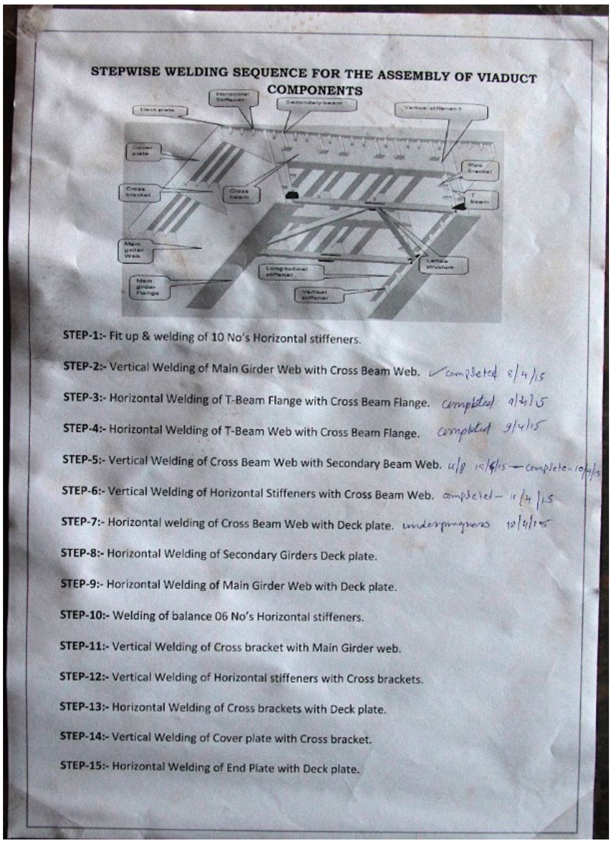
After finishing the fabrication, all required documents were prepared, filed in a folder both in digital and paper forms and handed over to the client:

- Certificate of materials
- Certificate of welding consumables
- Certificates of welders and WPS
- Welding inspections visual, UT and PAUT
- Test results of production test plates
- Survey of single segments
- Workshop drawings
- Fabrication procedures

Fabrication of segments for the viaduct and the superstructure

The segments weigh between 50 to 100 tons. Their lengths are between 8 to 10 m. Every segment had 1,000 m of weld length as a single layer.

Over 6 months the welding imperfections detected were below 2%.


STEPWISE WELDING SEQUENCE FOR THE ASSEMBLY OF VIADUCT COMPONENTS

STEP-1:- Fit up & welding of 10 No's Horizontal stiffeners.

STEP-2:- Vertical Welding of Main Girder Web with Cross Beam Web. ✓completed 8/4/15

STEP-3:- Horizontal Welding of T-Beam Flange with Cross Beam Flange. completed 9/4/15

STEP-4:- Horizontal Welding of T-Beam Web with Cross Beam Flange. completed 9/4/15

STEP-5:- Vertical Welding of Cross Beam Web with Secondary Beam Web. u/p 10/4/15 — complete - 10/4/15

STEP-6:- Vertical Welding of Horizontal Stiffeners with Cross Beam Web. completed – 11/4/15

STEP-7:- Horizontal welding of Cross Beam Web with Deck plate. underprogress 12/4/15

STEP-8:- Horizontal Welding of Secondary Girders Deck plate.

STEP-9:- Horizontal Welding of Main Girder Web with Deck plate.

STEP-10:- Welding of balance 06 No's Horizontal stiffeners.

STEP-11:- Vertical Welding of Cross bracket with Main Girder web.

STEP-12:- Vertical Welding of Horizontal stiffeners with Cross brackets.

STEP-13:- Horizontal Welding of Cross brackets with Deck plate.

STEP-14:- Vertical Welding of Cover plate with Cross bracket.

STEP-15:- Horizontal Welding of End Plate with Deck plate.

Figure 21: On every trolley station a plan for the fabrication sequence was fixed

Figure 23: Welding ribs on deck plate

Figure 24: Tacking of cross girder and LT beams

Figure 22: Welding deck plate

Figure 25: Welding of web plate

Figure 26: Distortion of web plate – post heating

Figure 29: Tacking web on deckplate

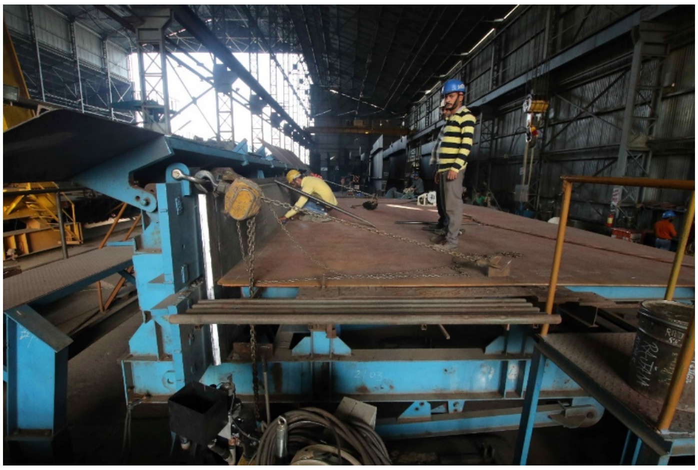


Figure 27: Installation of bottom flange – web plate

Figure 30: Installation of bracing in the bottom flange level

Figure 28: Welding bottom flange – web plate after rotation

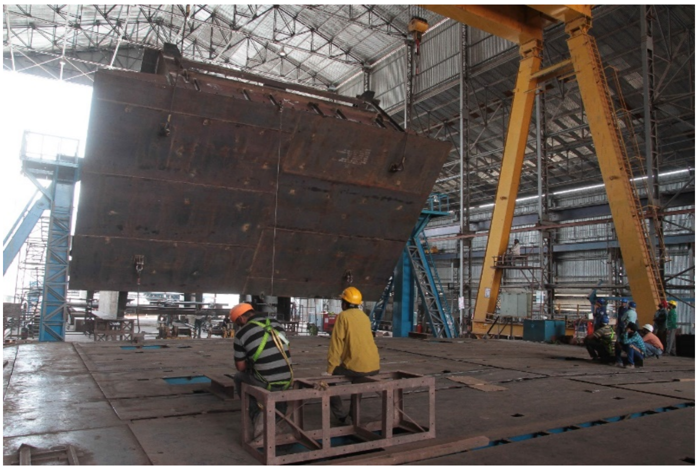


Figure 31: Rotation of segment

Figure 32: Final position for finishing welds on deck plate

Figure 33: View in the workshop with at least five fabrication trolleys in parallel

Figure 35: Heating during welding with gas

Figure 34: Welding of part 1 for the hollow section with welding gantry

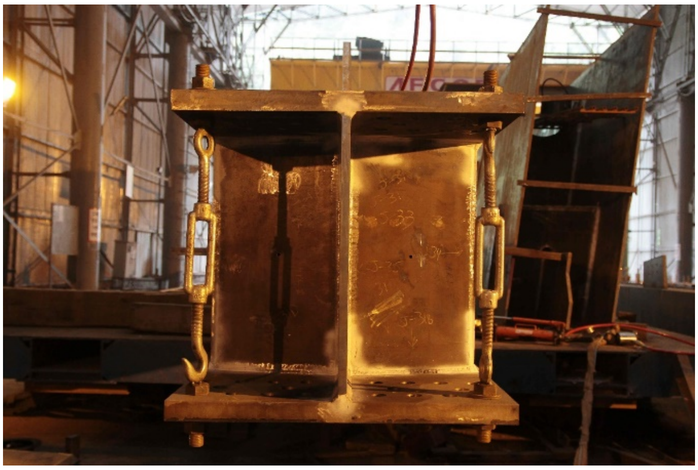


Figure 36: Temporary fittings for reducing distortion from welding

Figure 37: Checking heating temperature during welding

Figure 38: Tacking of web plates for the arch base segment

Fabrication of main hollow segments for steel piers

The fabrication of the hollow sections for the steel piers was not as difficult as the viaduct segments.

Fabrication of hollow sections arch segments

The fabrication of the hollow sections for the arch segments followed the same procedure as the one for the steel piers.

The fabrication started around one year after the piers and all the experience gained from the pier fabrication was implemented in it.

Only the fabrication of the arch base segment was new and required some heavy welding.

← Figure 39: Heavy stiffeners shall be welded with very small place

↙ Figure 40: General issue of fabrication: scallops too small

↓ Figure 41: Required radius of scallop as a function of plate thickness and weld throat

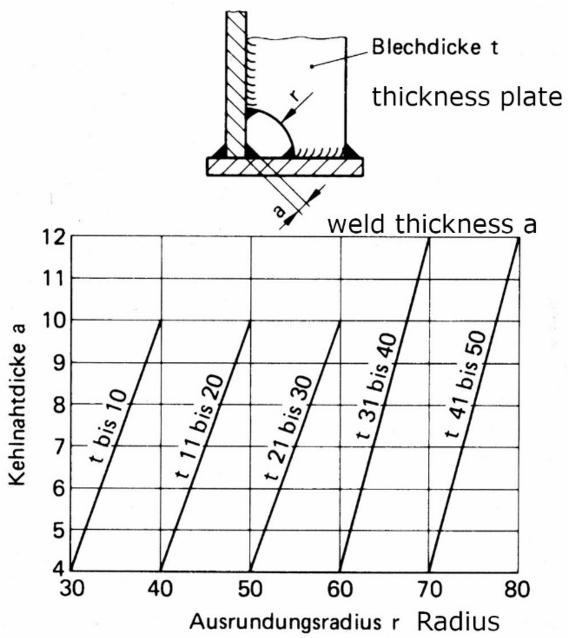


Figure 42: Finished base segment

Figure 43: Lifting of the base segment

CONCLUSION

The fabrication of all segments for the Chenab Railway Bridge on site with complete responsibilities of the company AFCONS was a significant challenge.

In many analytical meetings and discussions on site between all the parties to the project agreement was reached to resolve a wide range of technical issues.

All segments were fabricated to the specified quality levels according to EN 1090, Part 1 and Part 2.

In a geographically remote area, with extreme environmental conditions and restricted social life over an extended period, the fabrication of the Chenab Railway Bridge became a part of the personal and professional lives of all those involved in the valley of the River Chenab in the Pre – Himalaya in front of the mountains of Pir Panjal.

ACKNOWLEDGEMENT

I would especially like to dedicate this article to:

Mr. Palash Bhaduri, Fabrication Manager for Viaduct and Superstructure
Mr. P. Subramaniam, Fabrication Manager for Steel Piers
Mr. Mukesh, and Mr. Rao, Fabrication Managers for Arch Segments
Mr. Yogesh Ohja, Head and Manager of the QA Department on site
Mr. Dr. Suresh, SFI, Consultant for welding

and all the Indian engineers, welders, fitters, and grinders in the workshops who were involved.

References

[1] Wai-Fah Chen, Lian Duan: Bridge Engineering Handbook. CRC Press LLC, 2000. ISBN 0-8493-7434-0

[2] IS 2062 – Hot rolled medium and tensile structural steel

[3] BS EN 10025 – Hot rolled products of structural steel

[4] IS 3757- Specification for high strength structural bolts

[5] BS 5400-6 – Steel, concrete and composite bridges – specification materials

[6] BS EN ISO 5817 – Welding – Fusion welded joints – quality levels

[7] BS EN 1714 – Non-destructive testing of welded joints – Ultra sonic testing

[8] BS EN 1435 – Non-destructive testing of welded joints – Radiographic examination of welded joints

ASSEMBLY OF THE CHENAB RAILWAY BRIDGE – PIERS AND ARCH

Dipl. Ing. Frank Bauchspiess

Chief Engineer, Fabrication and Erection for the Chenab Railway Bridge

Figure 1: Chenab Valley before bridge assembly

INTRODUCTION

The article gives an overview of the assembly of the steel structures of the Chenab Railway Bridge.

For a successful assembly, it is necessary to understand stress and geometry. The bridge designer is responsible for erection steps that have an impact on the stress distribution and the final required geometry. The responsibility for assembly rests with the contractor of the bridge.

Detailed assembly procedures in planning and calculation must be developed by the bridge contractor with experienced engineers in steel bridge erection.

For the Chenab Bridge, the design team on site and AFCONS developed the erection steps in detail for the necessary temporary structures.

A substantial part of the bridge assembly is the correct survey, handling of wind, and temperature impact on site. Special lifting operations with the two-track cable crane on site needed detailed thinking for the safety of workers and the steel segments.

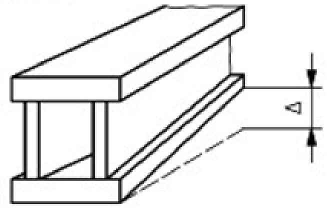
2	Twist: 	Overall deviation Δ in a piece of length L	$\Delta = \pm L/700$ but $4\ mm \leq \lvert\Delta\rvert \leq 10\ mm$	$\Delta = \pm L/1\,000$ But $3\ mm \leq \lvert\Delta\rvert \leq 8\ mm$

Figure 2: Permitted twist according to EN 1090-2

BOLTS

One of the main challenges was the investigation for correct installation and handling of the heavy bolted joints with HV 10.9 bolts in large numbers on every assembly joint.

Bridge books and bridge literature do not give a detailed and sufficient explanation of the assembly of such joints.

The design and required construction rules in codes have a rather theoretical background for such bolted joints.

The required tolerances for HV 10.9 bolted connections are stricter than the permitted deformations of welded steel segments according to EN 1090-2 Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures, see Figure 2.

For HV connections the permitted gap on edges after tightening must be smaller than 2 mm as a requirement for corrosion. Stiff splice plates will never be deformed by the tension force in bolts.

At the same time, there is a risk of over-tightening the bolts.

For HV bolts (developed in Germany), the tension forces in the bolts are a result of the first 3 - 5 threads below the nut in plasticization.

For safe tightening is the combined torque method required. For sufficient tightening, the torque sequence must go from stiff to not so stiff in the joint.

HR bolts (developed in the UK) and the next development - HRC bolts - are more convenient for installation.

torque sequence

requirement — start at the most rigid part of the connection and progressing to the free edges

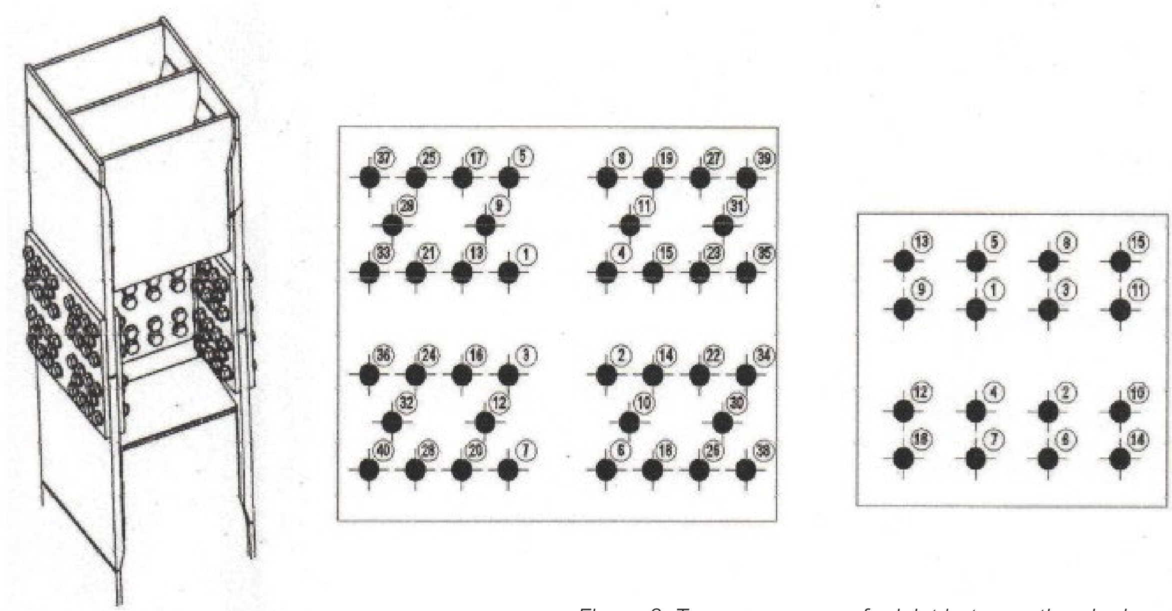


Figure 3: Torque sequence for joint between the pier legs

ASSEMBLY – STEEL PIERS

Erection Base Segment

The piers have an inclination from bottom to top. The correct installation of the four base legs of the pier is important for the correct alignment of the piers.

Supporting plates with threads in the required height on the anchors were installed.

With this, the correct inclination was achieved.

The survey on site during erection is a core part of achieving correct assembly.

For this, precise engineering performance was required.

Figure 6: Leading the base segment to match the drilling holes

Figure 4: Plates for correct installation of the base segment

Figure 5: Transport of base segment cable crane with an inclination

Figure 7: Installed base leg with correct inclination

Always use calibrated, precision survey equipment as in Figure 8 (1-degree difference between the two instruments).

<u>Temperature Impact</u>

A knowledge of the temperature development during the assembly time shall be an important item for a correct assembly geometry.

Before starting the assembly the author calculated the impact of temperature for the assembly on site with SOFiSTiK in a 3D model.

During real assembly situations and investigation of the sun circle over the Chenab Valley the following experience was gained:

1. If the temperature distribution in the steel structure was 20 degrees Kelvin between the bottom and top cross-section there was a deflection in the node points more than ±30 mm. With this value, the drilling holes for the HV connections never match.
2. The temperature has only an impact on the deflection of non-stiff structures – at the time when all bracings were installed, the temperature changes caused only internal forces (basic mechanical fact).
3. Similarly to the experience by Mr. Amman during the erection of the Verrazzano Narrows bridge: a survey for the assembled structure is only possible in the eary morning hours without sun impact.

Figure 8: Measurement of inclination

4. The real temperature distribution is different from the temperature distribution written in codes – especially in Eurocodes (see Figure 11 with different shadow conditions).
5. During erection: the computer calculated deflections were never encountered on site as a result of the short duration of heating the steel surface during the sun's travel over the day.

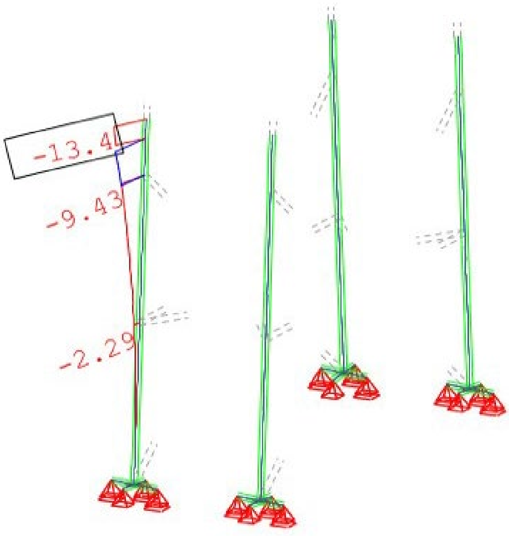


Figure 9: Horizontal deflection base leg (linear temperature gradient 10 degrees Kelvin)

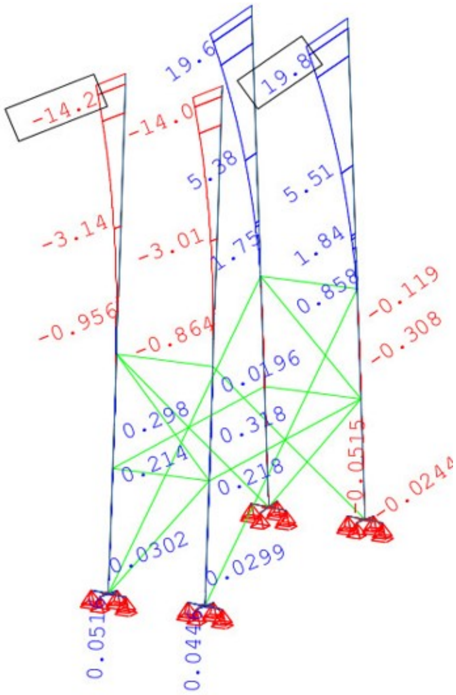


Figure 10: Horizontal deflection of legs by pier legs step 2 (linear temperature gradient over cross height of 10 degrees Kelvin)

Figure 11: The pier below the sun with shadows

The calculated deflections shall only give an indication of how the structure can react – but in reality, it does not always respond like this.

With small forces using a chain hoist, it is possible to achieve a match between the drilling holes of the bolted joints in the node – and the splice plates.

Safety for Assembly at Height

The safety of workers and assembly personnel was the primary consideration on site.

All assembly crews were educated and trained in special climbing techniques and safe working procedures at height.

A special teacher with experience in climbing and working on height structures trained all assembly workers and engineers.

As a structure for training, the 3-D tower of the cable crane was used, see Figure 12.

Installation of the vertical diagonal cross-bracing

The diagonal cross-bracing was pre-assembled on the ground and lifted with a cable crane in position, lead with slings in position and with temporary pins fixed in the drilled node plates of the pier leg.

Pre-calculation was made with SOFiSTiK to study the deformations to get the information for matching the splice plates with drilled node plates.

The drilling holes had only a 2mm tolerance for matching.

Figure 12: Training in climbing on site

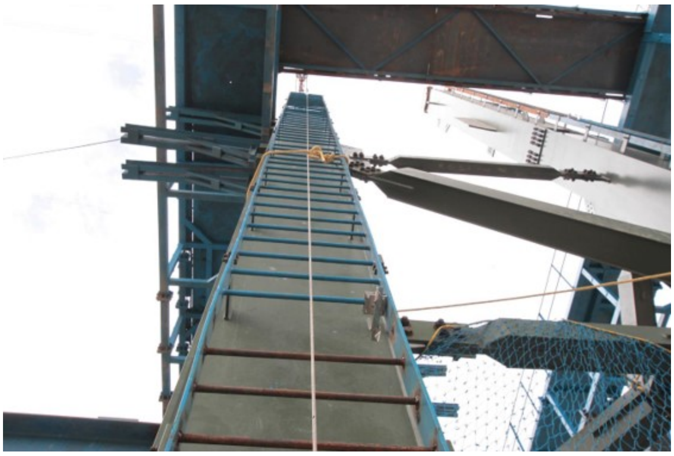


Figure 13: Installed life line systems on pier leg

The software calculation is rarely representative of the structural stiffnesses to get the correct deformation results.

The stiffness of the pre-installed erection bolts for the cross-bracing point is difficult to correctly formulate in the computer.

Figure 14: Vertical cross-bracing installation

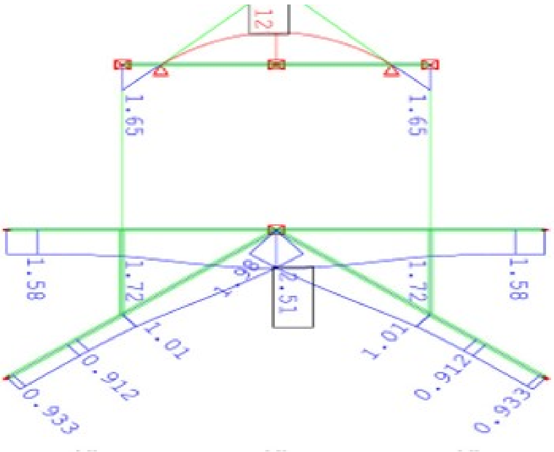


Figure 15: SOFiSTiK model

The software results give only an indication in which direction matching problems can occur.

WSP Finland defined the permitted tolerance of the horizontal deflection for every pier after assembly of ±100 mm.

The first Pier 20 when fully assembled had a difference compared to the design alignment of 37 mm.

→ Figure 16: Detail of matching the drilling holes and fixing node point

Figure 17: Pier assembly in progress

Figure 18: Trial assembly of a part of the arch on ground – installation of 3D wind bracing

ASSEMBLY – SINGLE ARCH

Trial Assembly

The necessity of a trial assembly shall have a good justification. Special areas and special equipment and assembly time are required. The objectivess for a trial assembly shall be clearly defined.

Many bridge designers think the trial assembly simulates the real assembly – or erection process. For such a target, the trial assembly must follow the same conditions as the real assembly, namely:

- in geometrical form,
- lifting operations,
- the static systems,
- temperature,
- joint stiffness.

See Figure 18 above – the centre of the wind bracing had a supporting steel column. In the real installation of the wind bracing this would not be possible at a height of around 200 m above the ground.

The cantilever erection steps of the arch are very rarely simulated.

In fabrication with correct CNC cutting from good computer workshop drawings a trial assembly is not always necessary.

A trial assembly which does not simulate the same stress situation as in the real assembly cannot give a correct impression of the real assembly. At the same time, the real assembly situation is totally different from the trial assembly.

Figure 19: Part of the base plate fixed on reinforcement with pre-installed anchor bars

Base Segment

The base of the arch is fixed in the concrete arch foundation in the slope.

The correct position of the base segment fixes the complete arch geometry for all the next steps.

First, it is necessary to install the steel base plate embedded in the concrete foundation of the arch.

The tolerance of such huge plates embedded in concrete is not clearly defined in codes. But every assembly has tolerances.

The measurement after concreting gave a difference in plane of +5 mm and in maximum height difference of +11 mm.

Values vary below permitted tolerances for smaller plates according to the code.

To protect the concrete under the plate during hydration of concrete from cracking and temperature elongation of the steel plate, the plate was cooled during hydration with water.

Successful concreting was checked with a hammer test.

In total, all tolerances from the hot rolling surface, installation of the anchor plate and deformation of welding shall be a gap of ±26 mm between the anchor plate and arch base segment permitted.

The bridge designer shall know about fabrication and assembly tolerances on site for developing comfortable and successful detail solutions.

Figure 20: Lifting and lowering the base segment

Figure 21: Lowering the base segment in the correct position to match the drilling holes for the pre-installed anchor – heavy work on site around 200 m above the ground and 50-degree inclination

The maximum real gap achieved was +17 mm.

The gap was filled with a special metal polymer material from the German company Diamant (*for more information read their article in this edition of e-mosty*).

<u>Assembly of a single triangle as Ground Structure of the Steel Frame Arch</u>

The designer (Leonhardt, Andrä und Partner, Germany) developed and calculated the global erection sequence:

- Installing the arch up to the first re-anchorage point;
- Re-anchoraging the cantilever arch;
- Installing the next arch structure up to the second re-anchorage point;
- Re-anchoraging the structure.

The lifting capacity of the cable crane was limited. Consequently all single segments of the arch had to be installed as a single element.

The steps to install the triangle arch triangle were:

- Lifting and lowering the bottom chord in an inclined position;
- Installing the bolts according to the given procedure;
- Re-anchoraging the bottom chord;
- Unloading the cable crane;
- Lifting and lowering the diagonals in an inclined position;

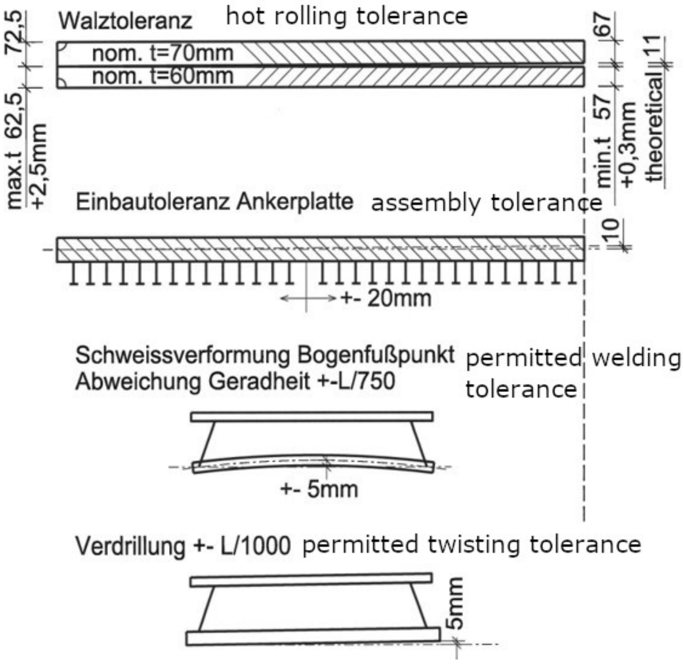


Figure 22: Sketch of permitted tolerances for base segment installation

Figure 23: Installed injection holes for filling the gap

- Installing the bolts according to the given procedure;
- Unloading the cable crane;
- Lifting and lowering the top chord in an inclined position;
- Installing the bolts according to the given procedure;
- Unloading the cable crane.

Figure 24: Installation of the bottom chord

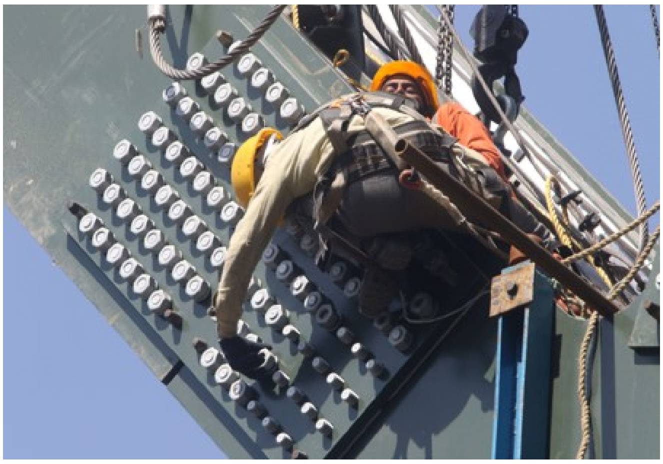


Figure 25: Detailed view of the bolted joint

The critical point is the time when the load is on the cable crane. In strong windy conditions the time must be short.

After long research, American erection companies with experience in the correct handling and installation of such bolting joints were identifed.

The principles were as follows:

- Bringing the structure to the final position;
- Preparing installation pins (diameter +1 mm more than the drilling hole) for fixing and trying to get a match with the drilled holes;
- Installing (without special calculation) 50% erection bolts;
- Removing the load from the crane;
- Installing the other 50% with the final bolts;
- Replacing the erection bolts with the final bolts;
- Torqueing the joint.

A torque of the complete joint with a load on the cable crane was never possible. Independently of the required manpower for the torque of one joint, the torquing of 300 bolts is finished in 12 hours.

Erection pins in steel grade material 10.9 were fabricated on site.

The installation procedure was modifed to minimize the time.

With re-anchorage it is not required to install 50% erection bolts.

Figure 26: Installation of the diagonal

Figure 27: Installation of top chord

Figure 28: Step by step the triangles of the arch were installed

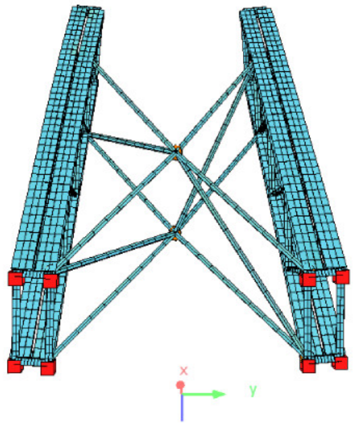


Figure 29: SOFiSTiK model for investigation of the installation sequence on site

Assembly of 3D wind bracings

The installation of the special 3D wind bracings was the next challenge in the erection of the arch structure.

Initially, it was very difficult to determine how the wind bracing could be assembled between the three single arches.

Only two weeks before starting the assembly was a methodology established as to how this could be done.

Figure 30: Installation of the first 60m diagonal

→ Figure 31: Installation of the cross-diagonal

The process was as follows:

- Lifting the pipe and bringing it into position between the arches by two single cable crane tracks;
- Lowering the end of the pipe in different height positions;
- Installing the supporting slings for the pipe;
- Pinning the splice plates on the nodes between pipe and arch;
- Installing 50% bolts on every node connection;
- Removing the load from cable crane;
- Installing the remaining 50% of the bolts in every node connection.

It was a big effort with a successful installation of the special 3D wind bracing.

Figure 32: Matching the drilling holes for the bolting joints in height

Figure 33: Assembly situation after installation of the second anchorage cable; the assembly process was stable and safe

CONCLUSION

The assembly of the Chenab Railway bridge was a big challenge for all involved workers and engineers.

Many steps for erection and assembly on site for such a big structure were completely new for many people.

Starting from investigations and developing very special procedures, discussions and meetings a successful way was found for building the biggest railway steel arch bridge of its kind in the world.

ACKNOWLEDGEMENT

I would especially like to thank:

Mr. Saroji Hore, Assembly Manager on site

The crew of the cable crane
The complete assembly crew from Punjab
Mr. Anukul Halder and the survey team
Mr. Samvit Trisal and Mr. Appar – design on site

CASE STUDY ON CHENAB BRIDGE ARCH BASE PLATES AND BEARINGS: METAL-TO-METAL GROUT SYSTEM

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DIAMANT Triumph Metallplastic Pvt Ltd, India

Carsten Kunde, Managing Partner
DIAMANT Polymer GmbH, Germany

INTRODUCTION

Steel-to-steel connections are widely prevalent in bridge construction. Traditionally, all steel bridges have specifications that require 100% contact for complete designed load transfer.

Ensuring such 100% contact between the mating surfaces has been a challenge faced by design and fabrication engineers worldwide.

Achieving it may be one of the most difficult and expensive processes in larger and more complex steel constructions.

Improper or non-100% contact severely affects the ability to transfer design loads and leads to a shorter life of the structure and its components such as bearings.

Advancements in materials science allow for a range of new metal grouting systems that are assisting engineers and steel fabricators around the world in solving this issue with minimum effort.

In recent years, a lot of progress has been made in materials science. New polymeric metals have been developed that allow for a 100% gap compensation through non-traditional methods.

These new-generation materials use very high-strength fillers in a matrix that boasts of low creep, nearly zero shrinkage and the ability to withstand high cyclic loads.

They do not have functional impact or degradation from environmental factors such as salt water, UV Light, heat and rain.

Another very useful characteristic of these materials is the ability to use these directly on-site without the use of advanced machinery / special tools.

In India, one of the fastest growing bridge construction markets, a number of steel bridges are under various stages of construction - the goal being speed and quality as the country upgrades its infrastructure.

In this case study we present the usage of one such new-generation material which was used on the arch base plates and in bearing top-girder bottom interfaces of the Chenab Bridge project in India.

DIAMANT MM1018 is a special formulation that can withstand 160N/mm^2 compression stress and has been proven to withstand environmental degradation with no impact on performance.

The material is suitable for use in gaps as small as 0.1mm and has been tested for performance up to 140mm height by the German Institute of Construction Technology (DiBT) along with cyclic fatigue to withstand heavy rail traffic.

PROBLEM DESCRIPTION

The Chenab Bridge is the highest Railway arch bridge in the world at a height of 359m from the bed level and a 469m main arch span.

The bridge was designed as a two-ribbed arch with steel trusses made of concrete-filled steel box segments.

The bridge is located in an area that is exposed to temperature changes between -10°C and +40°C and moisture making corrosion protection among key requirements.

The concrete arch base foundations support the entire arch. The steel arch is seated upon 8 arch base plates, 4 on each side of the river Chenab, see Figure 1.

The arch base plate is embedded in the concrete pillars. Each base plate has an area of approx. 6.3m^2.

The arch base plates are mated with the arch base segments and stressed using Dywidag bars that pass into the concrete pillars, see Figure 2.

The arch base segments are steel box segments that are field fabricated and are prone to minor deflections due to manufacturing limitations.

A non-full contact connection can therefore lead to the selective transfer of loads. This may compromise the level of structural safety and hence is a critical requirement.

Figure 1: Arch base mounted on the arch base plate

Gap creation

The arch base segment is a box structure with stiffeners. During the extensive welding process, steel is prone to heat-related distortions.

Even when conforming with existing fabrication standards, an acceptable warp at fabrication will yield a very high distortion on structures of this size and complexity.

In the case of the base segments, due to the extensive use of stiffeners over a large surface area, a 100% flat surface was not possible to achieve.

The arch base plates embedded in the concrete are placed at an angle in all 3 planes of the bridge's 3-dimensional 'X-Y-Z' geometry.

This posed a further challenge to meet the 100% load transfer and matched mating face requirement.

Figure 2: Arch base plate with Dywidag bars

Traditional methods of gap compensation

A number of techniques have been used traditionally to overcome these situations with each having a limitation for a project of this nature and scale.

- **Machining:** This requires the use of large specialized milling machines that have to be placed at the site of installation to machine the plates in the final resting position. This is a very expensive and time-consuming process when used on completely horizontal connections. In the case of inclined connections, this would be a major challenge since both faces would need machining after studying final alignments.
- **Steel shims:** This method makes use of custom steel plates prepared based on the gaps observed, however with no certainty of full contact. Most gaps have a varying profile which limits the use of these plates. The use of embedded Dywidag bars also complicates the installation of custom shims.
- **Lead sheets:** These are used since they take the shape of metal plates but they can fail at higher loads and have very poor creep properties.

Creep

By definition (sometimes called cold flow) creep is the tendency of a solid material to move slowly or deform permanently under the influence of mechanical stresses.

It can occur as a result of long-term exposure to high levels of stress that are still below the yield strength of the material.

This is an important factor when using products and materials to fill gaps, especially for preloaded connections as creep may lead to a loss in tension force which in turn reduces the load capacity of the construction.

COMPENSATION OF GAPS USING DIAMANT METAL-TO-METAL GROUTING

Material Description

DIAMANT MM1018 is a 2-component metal reactive resin system based on epoxy resins with high-filled proportions of diverse, mainly metallic powders.

The product is available in a few different versions based on application requirements.

The fluid and paste versions have been tested and acknowledged by the German Institute of Construction Technology since January 2013 for the holohedral, 100% force-fit gap compensation with respect to filling unevenness and roughness between metal elements in face plates, bridge bearings, railroads and steel elements as per general Approval Z3.822042/1/.

A number of other tests have been carried out by authorities across the world proving all the parameters required for its suitability for the case of the gap compensation at the arch base.

The material has seen use across other projects in India and has proven its use in very aggressive operating environments.

Usage on the Chenab Bridge

Engineers at the Bridge contractor upon intense discussions with the bridge designers and DIAMANT engineers decided upon the use of the liquid grade of the material.

A decision was taken to place the arch base on the base plates in their final position and then proceed with pressure grouting, see Figure 3.

One of the concerns with such a project is always to prove to the client that the grouting is providing 100% fill and the resultant transfer of full load at cure.

Figure 3: Injection of MM1018 between the arch base plate and first arch segment

Figure 4: Steel elements that are not evenly supported lead to uneven application of force and pose a problem for corrosion protection

Figure 5: Typical gap situation observed in bearing installation

The material is designed to self-cure naturally without the use of accessory agents or machinery.

Concerns related to the post-life (operational life of the compound after mixing) of the compound being sufficient to grout such a large area were also addressed by way of the application methodology.

BRIDGE BEARINGS

Traditionally, in this interface construction companies have used shaped wedges and cement and cement epoxy grouts.

While these are good when used with concrete interfaces they have shown to severely fall short in steel connections and also the stresses transmitted through the metal segments.

These have led to crushed or shattered grouting and in turn expensive bearing loss and replacements.

The success of the metal-to-metal grouting at the arch base helped the installation of the bridge bearings interfacing with the bottom of the girder.

An approach similar to arch-based grouting was followed to solve the issue of gaps due to the bridge camber and bearing top plate.

As steel fabricators learn and implement new possibilities using the advanced metal grouting material, new possibilities are available to them to ensure even the smallest of gaps are reliably filled.

Figure 6: Bearing installation with gap between upper bearing plate and bridge girder

Figure 7: MM1018 application between upper bearing plate and bridge girder to ensure 100% connection of the bearing to the bridge

FUTURE TRENDS AND SUMMARY

Gap compensation between steel elements is a challenge across all constructions. With advanced materials and technologies that are under continuous development, the possibilities available to engineers are huge.

The MM1018 system and method have seen increasing acceptance globally and are the new way of the future. The Chenab arch bridge project proved the utility and effectiveness of the MM1018 system.

While the gap was closed to a 100% force fit, the material also exhibits permanent corrosion protection on account of its formulation.

This protects and improves the longevity of the connection with respect to the elements.

In recent years with an increasing focus on reliability and reduced construction timelines, materials such as Diamant MM1018 and their variants shall see an increasing demand across the globe.

In India, as the Indian Railway upgrades its infrastructure, with major constructions in process, the metal-to-metal grouting system provides its engineers, fabricators and end users an economically viable, fast, proven, globally approved, reliable and high-quality solution.

The use of a liquid shim is seeing increasing use across bridge bearing applications, flange connections in process plants, marine applications and more recently in quickly refurbishing older structural connections.

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SKYTRUCK 200 – CROSSBAR CABLE CRANES IN USE AT CHENAB BRIDGE

Hans J. Gmeiner, SEIK GmbH/Srl, Italy

Figure 1: Skytruck working at Chenab Bridge construction

INTRODUCTION AND HISTORY

In 2005 SEIK was asked to submit a transport solution for the construction of the main part of the Chenab Bridge situated in the State of Jammu and Kashmir in India.

This bridge crossing the Chenab River at 359 m above the bed level is an important part of the Udhampur-Srinagar-Baramulla Rail Link Project. The distance to be covered between bridge ends in Kauri and Bakkal was nearly 1,000 m.

To meet the requirements provided by AFCONS, SEIK offered a system of two crossbar cable cranes with 20 tons of payload each.

These cable cranes operate on a cable system consisting of four 54 mm diameter suspension cables for each crane and pylons with heights between 105 and 127 m on each side. The cable cranes can work separately as well as in tandem providing a maximum payload of 36 tons.

After SEIK was awarded the supply of the two cable cranes the first site visit was made in November 2005. Both cable cranes were supplied in 2006, and rope pulling and assembly of the cable cranes started in 2010 and ended in 2013.

The project had been delayed in 2008 while site specific data was collected which is why crane rope assembly operations only began four years after delivery.

After commissioning in August 2013, the two cable cranes were carrying in total a load of 21,000 tons of steel and concrete until the construction of the bridge was completed in 2022.

The cable crane system is intended to stay in place for maintenance of the Chenab Bridge for at least 5 more years.

PROJECT REQUIREMENTS AND CRANE FEATURES

For an optimal workflow of bridge construction, the cable crane system must be able to cover a length of 915 m and a width of 2 x 40 m at a full payload of 20 tons.

Near the ground, the use of a gantry crane would be a solution, but for building a bridge crossing a river at this length and height the only technical solution is a cable crane system.

Two crossbars each with 40 m width operate on the four No. 54 mm diameter suspension cables.

Figure 2: Skytruck 200 in operation

Each crossbar can carry a payload of 20 tons and in tandem operation, with the balance beam, the system can lift 36 tons.This solution made it possible to cover an area of more than 73,000 m² with 20 ton loads and more than 36,000 m² with 36 ton loads.

The world's largest capacity crossbar cable crane was the solution for moving and positioning the heaviest parts of the arch and piers.

The cable crane system is completed with pylons on each side with 127 m height at Kauri and 105 m at Bakkal. The pylons were designed by VCE in Austria and are anchored on each side with eight 58 mm diameter cables.

Figure 3: Skytruck 200 ready for operation

Figure 4: Skytruck 200 in operation

Figure 5: Skytruck 200 preparing for tandem operation

FEATURES OF EACH SKYTRUCK

- Own weight: 50 tons
- Payload: 20 tons
- Power source: Diesel engine with 440 kW
- Travelling speed: up to 3 m/s
- Hoisting of loads: up to 180 m at max. 2 m/s
- Lateral movement of load: up to 40 m at max 1 m/s
- Fixed traction rope 30 mm diameter
- Operator's cabin with A/C attached to the crossbar
- Tandem operation with the second crossbar using a balance beam, payload of 36 tons
- Area served: more than 73,000 m² with 20 tons payload, more than 36,000 m² in tandem operation with 36 tons payload
- Total weight of cable crane system at full payload: 160 tons

SPECIAL CONSIDERATIONS

The advantages of this system are:

- It can reach any position on the site with its lifting hook
- Only one system used for the complete site
- No transfer of loads from one crane to the other
- High loads and access to any place
- The system can be operated with one operator in the cabin and up to two ground operators
- The ground operators can lower and take up the load by remote control exactly at the desired place.
- The site area is free of tower cranes and towers
- Wind resistant
- Economic
- Remote maintenance via Internet access

The remote location of the site allowed shipment of the equipment only in 20 ft cargo containers (5.90 m in length, 2.35 m in width and 2.39 m in height). This required more work for assembly on site.

SUMMARY

In a time span of nearly 10 years, the challenge of building the highest railway bridge in the world was completed by using the world's largest capacity crossbar cable crane system.

After the bridge is completed, the crossbar system will stay in place for about five more years for maintenance purposes.

→ *Figure 6: Skytruck 200 in tandem operation*

↓ *Figure 7: Situation in October 2022*

TEMPORARY STAY CABLES FOR ARCH ERECTION OF CHENAB BRIDGE, INDIA

Jakub Szreder, Marcus Schraml

DYWIDAG-Systems International GmbH, Germany

INTRODUCTION

In Autumn 2017, DYWIDAG was awarded the contract for engineering, supply, installation, and subsequent dismantling of 56 stay cables for temporary use during the arch erection of the Chenab Railway Bridge.

The project was executed by the technical team from DYWIDAG's head office in Munich, Germany.

The erection design required the following cable sizes:

- 8x Type DYNA Grip® DG-P 19 (with up to 19 strands) for stays S41.1 and S49.1
- 16x Type DYNA Grip® DG-P 31 (with up to 29 strands) for stays S41.2, S42, S48 and S49.2
- 24x Type DYNA Grip® DG-P 43 (with up to 42 strands) for stays S10, S20, S43, S44, S47 and S70
- 8x Type DYNA Grip® DG-P 55 (with 49 strands) for stays S46 and S80

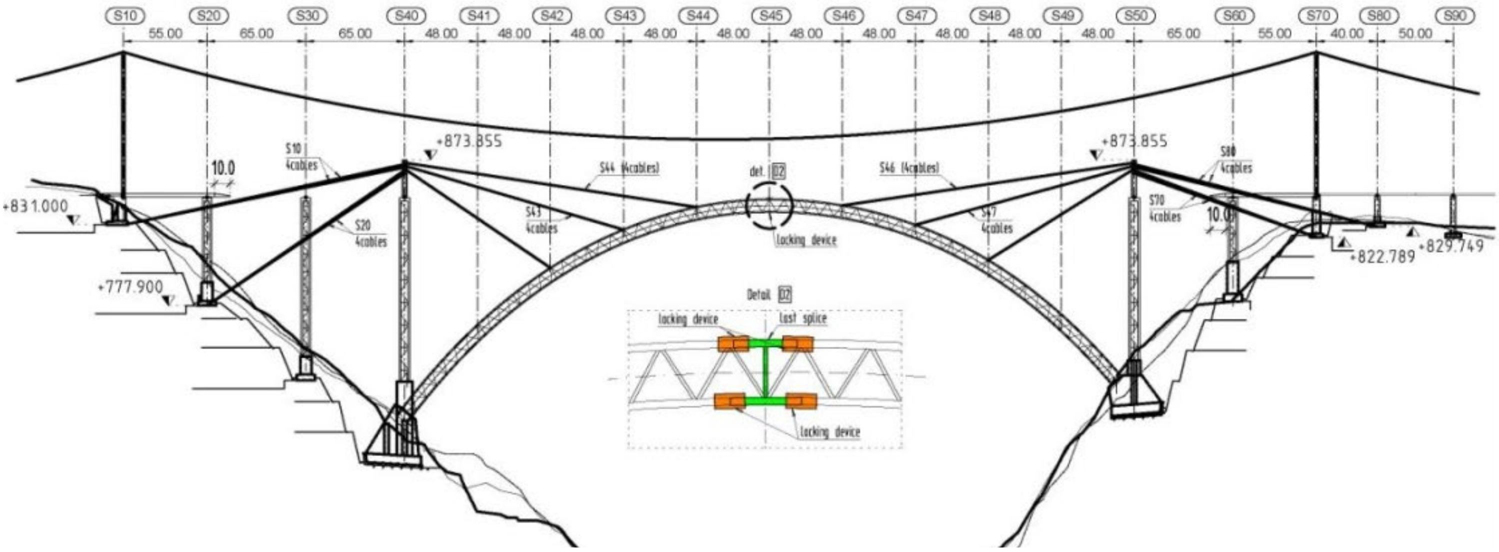


Figure 1: Snapshot of the installation Stage 8 with cable numbers

Although the stay cable system was meant for temporary use, due to the complexity and scale of the project, DYWIDAG decided to use anchor details identical to those used for permanent applications.

With this, a bending filter including a sealing unit at the rear side of every anchorage was placed which assured proper performance and corrosion protection of the system even in the case that the construction period of the arch had been extended, and with this, a longer service life of the cables would have been required.

Each of the supplied and installed cables utilized only fixed-type (non-adjustable) anchorages.

This was sufficient since in none of the construction stages the design required force increase or decrease via an adjustable locking nut along an anchorage having an outer thread.

Strand protrusions and the front side of the anchors were protected by temporary protection covers, whereas the free length of the strand bundle was protected by an outer HDPE pipe.

At the upper end of free length, an HDPE sleeve was attached to the temporary tower allowing for unconstrained pipe length changes resulting from temperature gradients.

The use of an HDPE pipe was not only advantageous for strand-by-strand installation of the cables with lengths between 105m and 193m but it also protected the cables from extensive vibrations due to wind which may have resulted because of the irregular shape of the multiplicity of single strands.

With this, the need for tying strands together at a certain distance to avoid strand rattling effects became unnecessary, too.

Figure 2: Arch – stay pipe with 1st strand installed; another stay pipe being installed

As the main tension element for the cables, 7-wire strands according to prEN 10138-3:2012 (Prestressing Steels Part 3 – Strand) with the following characteristics were used:

- Nominal strand diameter Ø = 15.7 mm (0.62”)
- Cross-sectional area S_n = 150 mm^2
- Ultimate tensile strength R_m = 1860 N/mm^2 (270ksi)
- Modulus of elasticity E = 195000 N/mm^2

Due to their temporary nature, the supplied strands were neither used with additional galvanization nor were they required to be waxed and PE-sheathed as would be typical for permanent stay cable applications with a design life of at least 100 years.

CABLE INSTALLATION

Installation followed a cable method statement which was agreed and approved in cooperation with the Designer and the Main Contractor before.

The exact sequence of installation was specified by the Designer, and it was always comprised a group of four cables fixed at the temporary tower (on top of piers S40 and S50) and going down to either the piers concrete foundations S10, S20, S70 and S80 (backstays) or to both steel arch cantilevers (forestays).

Anchors were preassembled including their sealing unit and bending filter and afterward placed onto the bearing plates on the arch structure as well as at the temporary towers.

The outer HDPE stay pipe which was delivered to the site in lengths suitable for transport, was mirror-welded to the required length.

Together with the first strand already inside, the pipe was then lifted by means of the cable crane and moved to the required position.

After threading the first strand into the lower anchorage at the pier respective to the arch side of the pier and setting it with a wedge, the opposite strand end was placed into the anchorage at the temporary tower and there it was stressed to its required force with the help of a monojack.

With the stay pipe supported by the first strand, further strands were installed and immediately stressed using DYWIDAG’s ConTen method (Controlled Tensioning).

Figure 3: Stressing anchors at the temporary towers

This method provides equalized forces amongst strands and considers the alteration of forces in already stressed strands while adding new strands in a cable.

After installing all strands, wedges at the passive side (pier with respect to the arch) were post-wedged and protected against corrosion by applying a corrosion protection compound and activating the sealing units.

The same operations were repeated at the active side anchors after applying the last stressing step and before the cables were put in service for progressing with arch erection.

The anchorages front side and strand details were encapsulated by using layers of tarpaulin.

While the arch erection progressed and additional forestays were added, it was also required to restress certain backstays.

Figure 4: Arch before closure; shortest forestays already dismantled

Figure 5: Arch after closure

CABLE DISMANTLING

After arch closure and before deck launching, the remaining 40 temporary cables needed to be dismantled and the temporary towers removed. During the arch erection, the 16 shortest forestays had already been dismantled.

In principle, cable removal followed the installation approach, but in reverse. Strands were de-stressed one by one and removed from the lower anchorage and subsequently from the upper anchorage and sheathing.

A very simple but quick approach was used by letting single strands slide downwards through a small plastic pipe. This continuous pipe was attached along the existing arch with its ending close to the foundation area of pier S40 or S50.

While strands safely moved downwards through the pipe simply by gravity, they left the pipe at the foundation area. The area was widely cordoned off for safety reasons.

Once the stay pipe was only in place with the last two strands remaining, the pipe was temporarily fixed at its lower end to the arch and to the cable crane at the upper end.

Subsequently, the last two strands were de-stressed and lowered downwards by gravity through the plastic pipe as well.

Then by means of the cable crane, the stay pipe assembly was removed from the temporary piers towards the riverside slope and dismantled there. The remainder of the strands could later be partly used for the assembly of the temporary horizontal ties required for deck launching.

HORIZONTAL TIES FOR DECK LAUNCHING

After arch closure and installation of all piers, 18 temporary horizontal tie cables connecting piers during the launching of the superstructure had to be installed.

Those ties served the purpose of reducing the bending moments in the piers when superstructure was launched over.

The system utilized consisted of PT-wedge plates accommodating 12 bare strands each, running through an outer HDPE pipe. Strands and pipe material were taken from the previously dismantled stay cables.

Figure 6: Stay pipe removal

Figure 7: Horizontal ties partly in place

As in the temporary stay cable application, the outer HDPE pipe facilitated the strand-by-strand erection method. Stressing was achieved by using a monojack and the anchorage areas were protected by layers of tarpaulin afterward.

After the finalization of the deck launching process, horizontal ties were removed in reverse to the installation process.

Figure 8: Horizontal ties during installation

Figure 9: Stressing of horizontal ties

Erection and dismantling of horizontal ties followed the specifications given in the detailed design structural analysis and were coordinated closely with all participants on the construction site.

CONCLUSION

DYWIDAG was involved in the project for the erection of the Chenab Bridge over several years after being awarded the supply contract in 2017 until the latest cable works on the construction site in late 2022.

During that period, collaboration between all stakeholders was affected by a very constructive spirit.

Close communication in this process resulted in continuous and efficient support especially during the construction of the arch when the installation and dismantling of temporary stay cables was a crucial part in the erection schedule.

CHENAB BRIDGE STRUCTURAL HEALTH MONITORING SYSTEM

Umesh Koul, Manager - Planning & Monitoring

Afcons Infrastructure Limited, India

INTRODUCTION

The iconic Chenab Rail Bridge is a steel and concrete arch bridge between Bakkal and Kauri situated in the Reasi district of Jammu and Kashmir, India.

The bridge spans the Chenab River at a height of 359 m above the river, making it the world's highest rail bridge.

This bridge is equipped with a state-of-the-art structural health monitoring system which ensures safe operations at all times.

As a part of safe practices, it was required contractually that trains are stopped before entering the bridge section when wind velocity exceeds 25m/s, on receiving a warning.

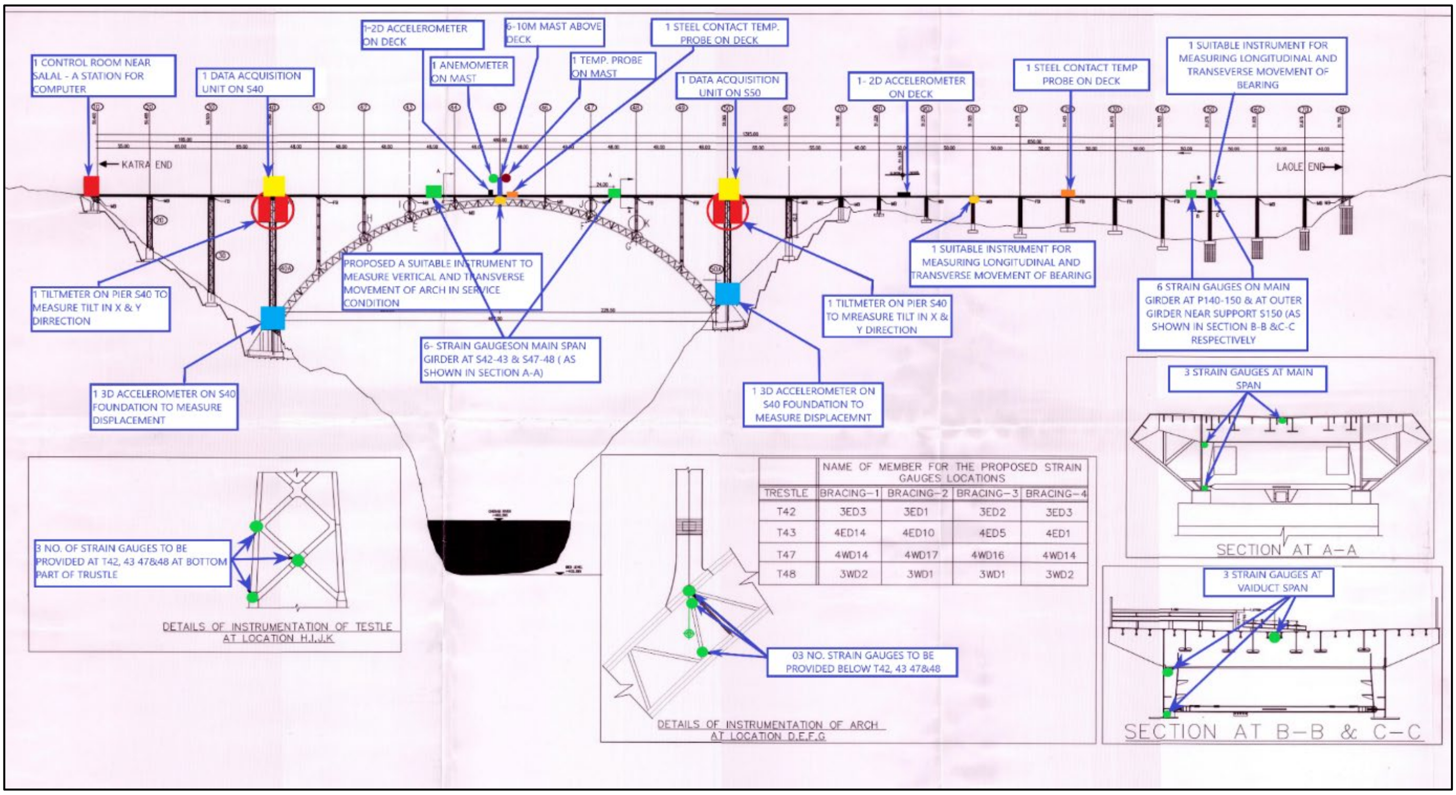


Figure 1: The Warning system is implemented by means of the Bridge Monitoring System

The Structural Health Monitoring System (SHMS) on the Chenab Bridge is a complex monitoring system designed to capture the environmental and live loads on the bridge, in addition to the structural response of the bridge and its foundations.

Through a unique design of multiple dynamic data loggers on a fibre Local Area Network (LAN), immediate trigger events, alarms and post-processing analysis can take place automatically to detect damage (if any), and report the condition of the bridge in real-time.

The SHMS comprises a combination of ground and bridge instruments such as dynamic strain gauges, temperature sensors, tiltmeters, accelerometers, anemometer, GNSS RTK rovers, linear variable displacement transducers (LVDT), load cells, extensometers and customised Smart Asset Management System (SAMS™).

The dynamic data loggers on the bridge are designed to manage dynamic data locally, and synchronising with SAMS™, located on the local server within the Control Room which is located 1.25 km towards the Bakkal side of the bridge.

The instrumentation scheme has been designed considering various parameters on each segment, see Table 1.

SENSOR TYPE	PURPOSE & LOCATION
Tiltmeter	Measure tilt at the S40 and S50 pier caps
Strain Gauges	To measure the strain and hence stress at pre-determined locations on the arch, trestle, and deck segments.
Linear Variable displacement sensors (LVDT)	Longitudinal and transverse movement of the deck in relation to the pier at bearing locations
Anemometer	To measure wind speed and direction at the highest point at S45, and used to trigger post-processing analysis of the bridge response
Air Temperature	To measure air temperature
Accelerometer	To measure acceleration at the deck crown and near S120
Steel Temperature	To measure steel temperature for correlation to deck position
Seismic Sensor	To measure seismic tremors within the foundations, to trigger post-processing analysis of the structure
Extensometer	To measure settlements at different soil/rock profiles
Load cells	To measure the loads within the ground anchors.
GNSS	To measure the position and deflections of the bridge deck
Data Loggers	Data Acquisition Units (DAU) at various locations on the bridge to convert raw analog data into engineering units, and pre-process for the server to trigger alarms and post-processing

Table 1: Hardware Summary

DETAILS OF THE SENSORS USED IN BRIDGE INSTRUMENTATION

TILTMETER

The bi-axial tilt sensors shall output useful information on the movement of the bridge piers. Being such tall and slender piers, the angle is very important to keep the movement within the allowable limits.

The requirements are for four uniaxial tilt sensors in total, measuring both longitudinal and transverse inclination. The sensors are held in place using an L-Bracket, secured to the structure using high-strength magnets.

STRAIN GAUGES

The strain gauges used on the Chenab Bridge are micro spot-welded 350ohm electrical resistance strain gauges. A total of 72 strain gauges are installed on the bridge.

The strain gauges are capable of sampling data in 1,000 samples per second. The strain gauge is protected using a special three-layer waterproofing technique, developed by Strainstall over many years.

The strain gauges are connected by multicore cable to local strain gauge amplifiers to maximise the accuracy and resolution of the sensors.

The strain gauge by itself measures linear strain, which is converted to stress, using the material properties of the steel.

The strain gauge arrangement is however designed to provide further outputs such as bending, by making use of multiple strain gauges around a detail.

The strain data is used in the post-processing analysis of the structure, triggered by train, wind and seismic events.

LVDT

Linear Variable Displacement Transducers shall be used to measure the current position and cumulative movement of the bridge deck and hence bearings.

Four LVDTs were installed. This quantity shall be split between the bearings at the expansion joints.

The displacement sensor shall be used to confirm the bearings are functioning well.

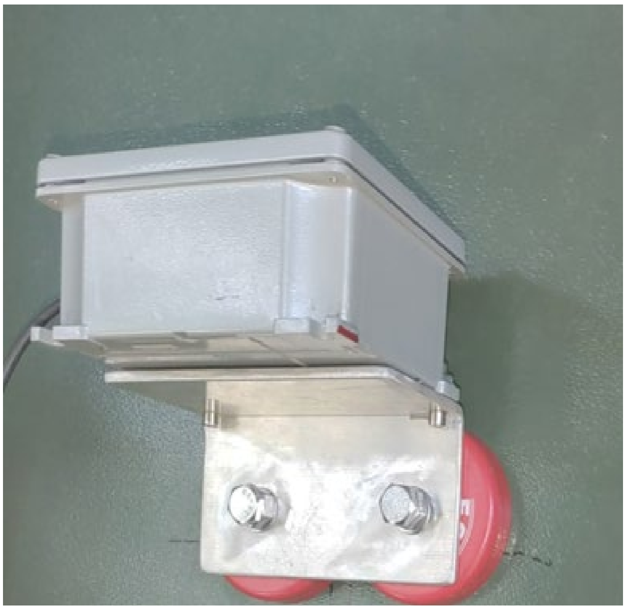


Figure 2: Installed tiltmeter

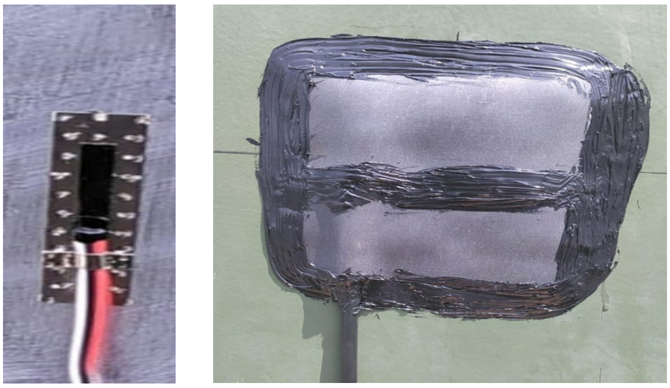


Figure 3: Strain gauge installation and protection

Data is continuously monitored, and in the event of reduced movement detected, a warning notice shall be sent to propose an inspection of the bearings for failure.

Secondly, the sensors shall add the cumulative movement each day and therefore keep track of the total bearing distance travelled.

This information can be used to plan replacement and maintenance or report an underperforming bearing to the manufacturer.

The LVDTs are held in place through L-brackets, secured to the structure by bolt or high-strength magnets.

Figure 4: Installed longitudinal and transverse LVDTs

ANEMOMETER

Due to the exposed location of the bridge, high wind speeds are expected through the valley upto 266 km/h (74 m/s).

In order to monitor these loads on the bridge one ultrasonic anemometer shall be installed at the central point of the main span to measure wind speeds and directions.

The anemometer is installed on a tall pole/mast to ensure interference from wind refraction from the bridge is not measured by the sensor.

An ultrasonic style of anemometer has been selected because it does not have any moving parts, which improves the resilience of the sensor and reduces the requirement for maintenance on the sensor.

The anemometer wind speed input is directly linked to the automated post-processing scripts; any high wind speed event triggers an immediate alarm, and instructs all sensors to sample at their fastest speed, with the dynamic data used in a post-processing analysis report.

AIR TEMPERATURE

The air temperature sensors are installed on the same mast as the anemometer to monitor the ambient temperature.

The sensor is enclosed within a shield to avoid direct sunlight readings which could influence the measurement.

The general ambient air temperature is used as a good correlation tool to the bridge's articulation and position, and for comparison of seasonal changes.

Figure 5: Anemometer installed on the mast

Figure 6: Air temperature sensors

ACCELEROMETER

A 3D MEMS accelerometer has been installed on the bridge deck at S45 and S80 to calculate the bridge accelerations in the vertical and two orthogonal horizontal planes.

The sensors are connected to the Data Acquisition Units (DAU) before passing through Anti-Aliasing Filters (AAF) to remove unwanted high-frequency data from the signal.

Each channel is sampled at 100Hz, with a 5Hz Low Pass Filter (LPF). The acceleration data is further processed to give an indication of stiffness and natural frequency.

With reasonable accuracy, displacement can be derived from the accelerations. The acceleration data is used in the post-processing script to report on the mode shape of the structure and modes of vibration caused by live load events.

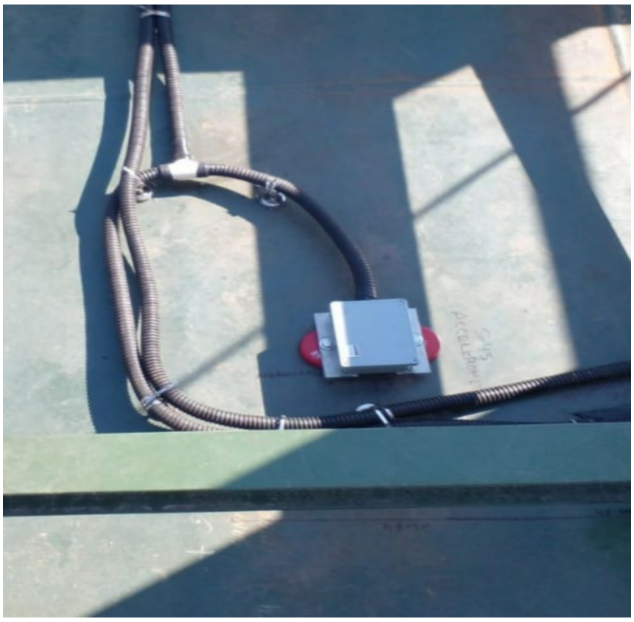


Figure 7: Accelerometer installed on the deck

STEEL TEMPERATURE

The steel temperature sensors have been installed on the bridge deck at S45 and S120 expansion joints.

The extreme temperature variation in the area causes large changes in the bridge articulation and the sensor becomes an important correlation tool to monitor, map and predict the bridge movement with thermal expansion and contraction.

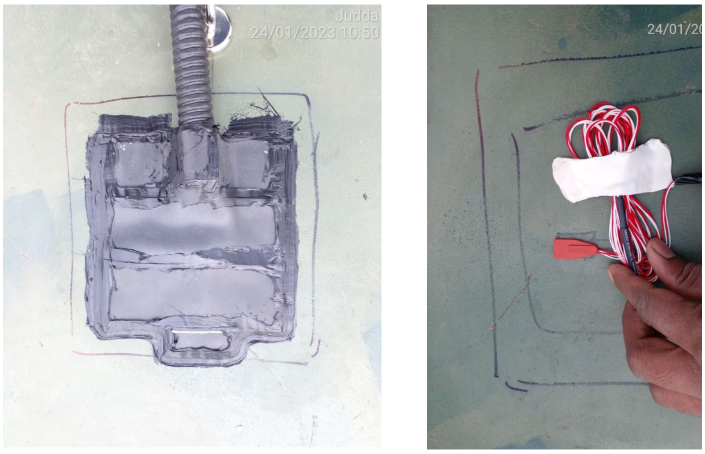


Figure 8: Steel temperature sensor

SEISMIC SENSOR

Seismic sensors are used for seismic measurements; they are robustly protected in a die-cast enclosure.

Seismic sensors have been installed at the S40 and S50 foundations. These sensors shall be secured into the surface of the concrete foundations of the arch springing points, and the cable would be run to the nearest DAU.

The accelerometer shall be sampled >1,000Hz with specific in-built filters to detect ground tremors. The seismic sensor signal is a key input to trigger the post-processing scripts of the bridge response when any tremor is detected.

Any tremor will also generate alarms for the operators of the system, whereby the user can quickly gauge all sensors on the bridge and look for any potentially dangerous changes.

Figure 9: Seismic sensor

Figure 10: Extensometer installations

EXTENSOMETER

The borehole extensometers help measure longitudinal displacements in the rock or soil at predetermined depths.

The system has multiple anchor points with rods extending to the surface. The head assembly at the surface measures the respective movement of each of these rods to provide information on the ground movement. The extensometers have been installed at S30, 40, 50 and 60 foundations.

LOAD CELLS

Through-hole load cells have been used to measure the loads within the ground anchors and S40 and S50. The through-hole load cell makes use of a series of vibrating wire strain gauges internally, to derive the average axial strain, which is relatable to the load, through the individual calibration certificate of each load cell. The load cell is held in place by two bearing plates and measures compressive load. Any change in the load measurement might indicate movement or a loss of strength in the ground anchor, so the sensor becomes a critical instrument in ensuring the ground anchors are performing as per their purpose.

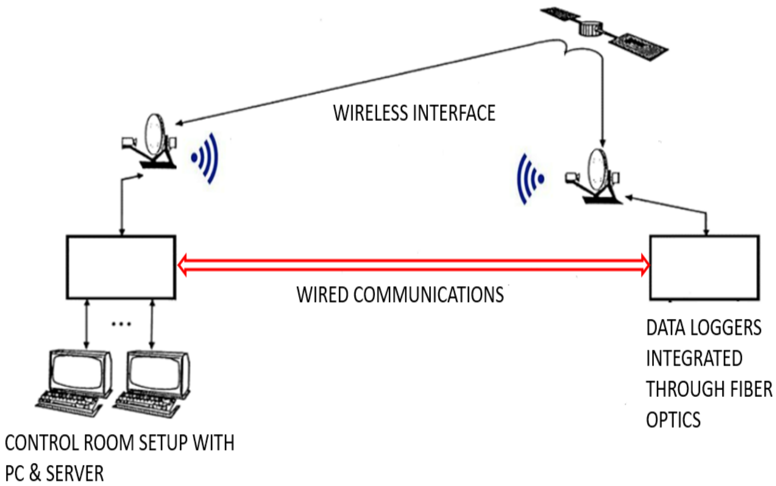
Figure 11: Ground anchor load cell

GNSS

GNSS (Global Navigation Satellite System) is the latest acronym for the current satellite system used for position tracking. The GNSS rover is installed upon a post at the midspan of the bridge. The rover obtains its position through satellite communication, in common easting, northing and level (or X, Y, Z).

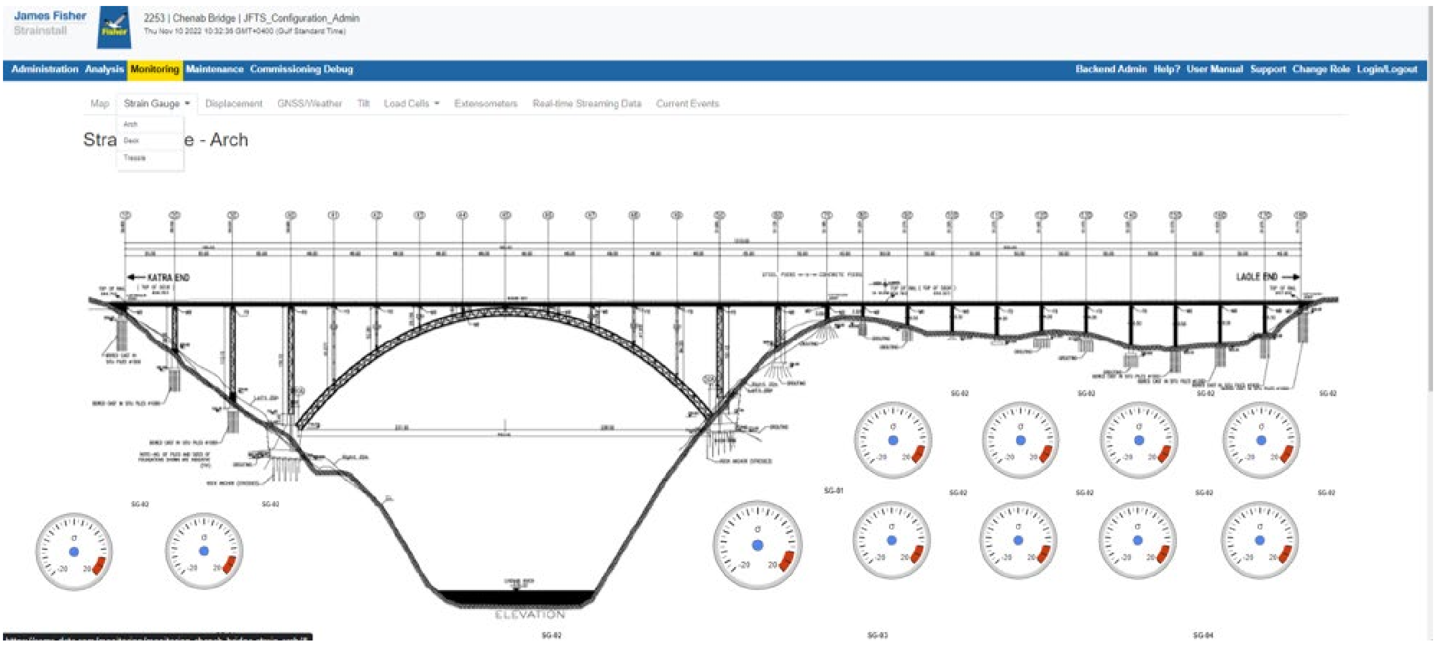
To enhance and improve the accuracy of this measurement, a reference antenna is located at a stable location on the roof of the control room nearby. Using this reference station and enhanced algorithms the positional system can achieve sub centimetre accuracies from 24 satellites in orbit around the Earth.

→ Figure 12: GNSS Bridge rover and reference station



← Figure 13: Communicaton Layout

↓ Figure 14: Typical Software Display



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References:
- Bahia de Cadiz, Spain
- Hochmoselübergang, Germany
- Osman Gazi Bridge, Izmit, Turkey
- Mainbrücke Randersacker, Germany
- Millau Viaduct, France
- Rheinbrücke Schierstein, Germany
- Rion Antirion, Greece
- Russky Island Bridge, Vladivostok, Russia
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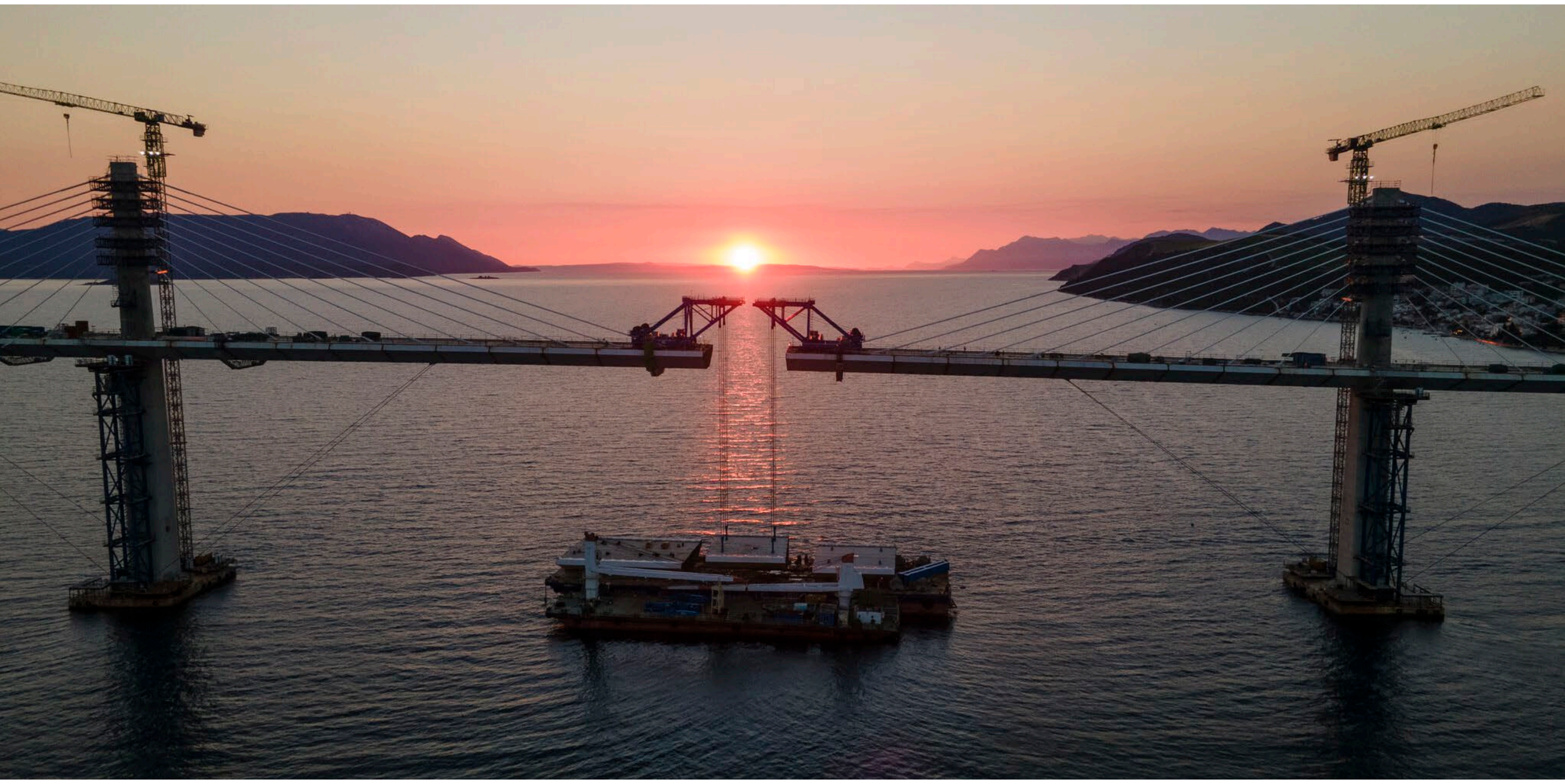

Pelješac Bridge, Croatia

Conceptual/Preliminary/Final design
Joint Venture Faculty of Civil Engineering, University of Zagreb; Ponting; Pipenbaher Consulting Engineers

Ada Bridge over Sava in Belgrade, Serbia

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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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FABRICATION AND ASSEMBLY