### THE ALIGNMENT

**OASIS BARAMULLA**
- Commissioned in 2005 (in steps)
- Completion Cost: 3430 CR.
- Cost per KM: 30 CR.

**BAHAL- QAZIGUND (108 km)**
- Commissioned in June 2014
- Completion Cost: 11420 CR.
- Cost per KM: 94 CR.

**KATRA- BANISH (111 km)**
- Target date of completion: 2015
- Anticipated Cost: 21866 CR.
- Cost per KM: 195 CR.

**JAMMU- UDHAMPUR (54 km)**
- Commissioned in 2005
- Completion Cost: 620 CR.
- Cost per KM: 10 CR.

### SALIENT FEATURES OF THE PROJECT

<table>
<thead>
<tr>
<th>ITEM</th>
<th>Route Length (in Kms)</th>
<th>Railing Gradient</th>
<th>Max. Carcass (°)</th>
<th>Bridges</th>
<th>Max-Hoist Bridge (in M)</th>
<th>Length of Bridges (in M)</th>
<th>Length of Spans (in M)</th>
<th>Tunnel Length (in Kms)</th>
<th>Total No. of Tunnels</th>
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FOREWORD

At the outset, I would like to congratulate USBRL team for publication of 11th issue of “Himprabhat”. It gives me a great pleasure to note that USBRL is attempting to record various steps and measures being taken in tackling the venturesome geology of Himalayas for contemporary professionals as well as for posterity.

USBRL is not only about construction of Railway line in J&K State but also bringing about socio-economic change in the far-flung areas and fulfilling the dream of linking Kashmir valley to the Indian Railway network, which is more than a century and half old. Udampur- Katra Railway line (25km), commissioned in 2014 under this project, is lifeline to pilgrims of Mata Vaishno Devi Shrine. Another commissioned section of USBRL connecting Banihal to Baramulla (136 km) traverses through idyllic and ravishing Kashmir valley, piercing the Pir Panjal range through country’s longest transportation tunnel (11.2 km), is hugely popular with locals and tourists.

I am sure, USBRL team will maintain the momentum and spirit and continue to move forward in realization of dreams of connecting Kashmir valley through Railway network by March, 2021 and also continue to chronicle the knowledge and experiences by periodical publication of this magazine.
FOREWORD

I am greatly elated that in perpetuation of their fascinating tradition, USBRL team has come up with 11th issue of Magazine “Himprabhat” to chronicle the unprecedented and unparalleled work being carried out in the most challenging and daunting geology of Himalayas.

I find that this periodical is repository of articles on new technologies, techniques and methodology being executed on project. USBRL team’s initiative and endeavor in publication of this magazine are laudable and deserve to be complimented. The instant issue has inter alia certain captivating articles such as use of curing compound in concreting works, testing and evaluation of slip factor in HSFG bolts etc. I am pleased to learn that Bridge and Tunneling works on all fronts is progressing in full swing. USBRL has picked up momentum in last couple of years and is galloping at rapid pace. The actual expenditure on this project surpassed Rs2000 crores last year for the first time and deserves profound appreciation.

I take this opportunity to instruct USBRL team to continue its glorious momentum and move ahead to share and enrich all readers with knowledge and experiences gained on this rugged terrain, by regular publication of “Himprabhat”.

New Delhi
27th Sept., 2018

(M. K. Gupta)
FOREWORD

USBRL Team deserves to be complimented for taking steps in publication of 11th edition of this wonderful and informative magazine “Himprabhat” to disseminate technical knowledge and challenges faced in execution of this daunting project for adoption in analogous terrain and for updating and brushing up of knowledge of posterity.

I have thrilling reminiscences of my last visit to the Chenab bridge site. Launching of arch segments is unique work being executed and will definitely feature at crest of the list in the annals of construction industry of country. The launched portion of viaduct of Chenab bridge is majestic to behold and deserves appreciation for technical uniqueness in launching the curvilinear portion (2.74 degrees curve) by incremental pushing of welded segments using launching nose. The scintillating memories refuse to fade.

I convey my best wishes to the team USBRL and impel them to keep the momentum and endeavor in the current year to expedite the progress of the work. At the same time, they should continue with their excellent work of publication of this magazine to keep all readers abreast of new ideas, techniques and methodologies adopted on this unprecedented Himalayan project.

[VISHWESH CHAUBE]

New Delhi
24th Sept., 2018
On assuming charge last month, and after having detailed deliberations & interactions with officers as well as inspection of the project sites, I cannot restrain myself in sharing thrill and admiration for the unprecedented & unparalleled work being executed in one of the most challenging and daunting terrain on the globe. USBRL Project traverses through the most difficult geology of the young folded mountains of Himalayas. Negotiating the mighty mountain ranges by burrowing tunnels, hopping canyons through massive bridges, connecting inaccessible project sites through road network etc will go down as landmark achievements in the annals of construction industry of the country. It is fascinating to contemplate on adoption of new technologies of cable anchors, dywidag bars, consolidation grouting etc to stabilize slopes of Chenab bridge, new techniques of butt weld testing by Phased array ultrasonic testing (PAUT), incremental launching by pushing of segments of curvilinear portion of viaduct of Chenab bridge through indigenously designed technique etc.

The iconic Chenab and Anji bridges have a large number of firsts to their credits. While Chenab Bridge is the highest Railway arch bridge, Anji Bridge is the first cable stayed Railway Bridge. There are significantly long tunnels on project. USBRL can take pride in having successfully constructed and commissioned the longest transportation tunnel of country called Pir Panchal tunnel, connecting Jammu region with Kashmir valley. There is another tunnel coming up in Banihal area, designated as T-49, which will surpass our own Pir Panchal Tunnel.

USBRL has taken upon the onus of documenting its fascinating journey by periodical publication of project magazine “Himprabhat” to share and disseminate knowledge and experiences in execution in challenging Railway lines in Himalayas. This periodical publication includes very useful articles and case studies of tunnels and bridges which will definitely inspire Engineers and professionals alike and enrich them with fruitful knowledge and information. The publication of “Himprabhat” moves to 11th edition now and instant issue presents variegated topics on tunneling and bridges.

The alignment of Katra- Banihal traverses through 27 tunnels, most of which are quite long and warrant special measures for safe and expeditious mining. Thorough knowledge of tunneling process is the need of hour in such complex geology, replete with imponderables and surprises. Drilling and Blasting is an important activity to achieve safe & smooth profile and expeditious advance at minimal damage to surrounding rock mass. Sh. Hussain Khan, DyCE Banihal and Sh. Ankur Sharma, XEN/Banihal in their article on “Tunnel Blast Design” have covered the detailed tunnel blast design including drilling pattern, quantity of explosives utilised, explosive types and proper initiation sequence. The authors have also covered a case study of tunnel T-74R on Katra-Banihal section, which is highly relevant and useful to the readers and practicing engineers.

The quality and durability of concrete primarily depend upon the curing technique adopted. Sh. Vinod Kumar, CE/P/KRCL and Sh. Shubannan Chanda, Dy.CE/Br/KRCL has come up with an article on “Review of The Curing Compound and Application Techniques”. The article covers state of the art curing compound used on important bridge no 39 on Katra-Banihal section.
Sh. A.K. Sachan, MD/DFCCIL (Former CAO/USBRL/JAT) on article on “Saga Of Tunneling Construction On Udhampur-Srinagar- Baramulla Rail Link Project (USBRL) - Himalayan Wonder” has given insightful and persuasive account of construction of Railway track in Valley, linking north and south like never before, along with Jammu region piercing mighty Pir Panchal range of Himalayas, Monikered in local parlance as Pir Panchal tunnel and technically as T-80 by Railway men, and 78 km Railway line from Jammu to Katra, marking the culmination of a vision that is over a century old.

Sh. Aqueel Ahmed, Dy.CE/C-II/Banihal and Sh. Ankur Sharma, XEN/C-II/Banihal takes forward this wonderful voyage with their article on Toussaint - Heintzmann (TH) or Top Hat Steel Ribs - A Flexible Support System which is in sharp contrast to the in vogue rigid and jointed arch support. This type of support system has been provided in Tunnel T-74R near Banihal area of Katra- Banihal section.

For the first time in the chronicle of Indian Railways, USBRL has been constructing an aesthetically beautiful cable stayed bridge on Anji River near Reasi District of Jammu and Kashmir. Sh. B.K. Sharma, Dy.CE/C/Anji/USBRL, has given an overview on the proposed cable stayed Anji Bridge. Sh. B.K. Sharma, has also come up with another article on “Testing And Evaluation of Slip Factor” on HSFG Bolted Joints. Nowadays, joints in bridge superstructures are being designed and constructed using state of the art high strength friction grip (HSFG) bolts. The author has covered the functional aspect and utilization of HFGC bolts along with the detailed deterministic methodology of slip factor in these bolts.

As the alignment of USBRL project traverses through difficult geology in Himalayas, in-depth knowledge of behavior of tunnel during execution as well as operational stages is of paramount importance from safety point of view. Dr. Joginder Singh, Consultant Geologist, KRCL and Sh. Amit Sherpuri, Assistant Geologist, KRCL have brought forward the topic on “Geotechnical Assessment Of The Slope And Foundation Conditions for Piers No. A1, P1, P2 & P3 Of Bridge No. 43” on Katra-Banihal section. The authors have thoroughly narrated geological condition of slopes over which the tall piers of Br. No. 43 are being constructed. They have succinctly depicted the various geological features through photographs and sketches in their article.
Sh. Praveen Kumar, XEN/Reasi has interestingly narrated the article on “Consolidation Grouting of Strata underneath Arch Foundation Of iconic Chenab Bridge. The author has presented a case study on consolidation grouting of Arch foundation S-40 of iconic Chenab Bridge. Use of pile foundation on bridge has been in vogue since long on strata having low Bearing capacity, Sh. Radha Mohan Singh, the then Dy.CE has shed light on another facet of pile foundation involving Micro Piles to construct Hybrid foundation, which would prove to be of immense value to the practicing engineers.

Sh. Partyush Sinha, A young energetic IRSE officer posted on Chenab bridge, has covered an overview of Chenab Bridge in his article on “Chenab Bridge- Iconic Bridging Of Mighty River”. The author has narrated details about the Chenab Bridge, its alignment, site selection of bridge, geological parameters and various methods & practices being followed in the construction stage of the iconic bridge.

Another enthralling article on “Installation Of Spherical Bearings on Chenab Bridge” has been presented by Sh. Umesh Koul, Manager Planning, CBPU. The author has given introduction on spherical bearings and procedures adopted for the installation of permanent bearings. Spherical bearings are proposed to be installed on the Approach Span of Chenab Bridge at the pier locations S-180 to S-80.

To meet the challenge of ensuring complete contact between metal surfaces, Sh. Anuraag Srivastava, Manager/Technology of M/s DIAMANT Triumph Metallplastic Pvt. Ltd., India and Mr. Dipl. - Ing. Carsten Kunde Managing Partner - DIAMANT Metallplastic GmbH, Germany have come forward with their article on "Metal Grout System For 100% Force Fit Gap Compensation In Steel Constructions - Application Case On Chenab Bridge Arch Base Plates”. The authors have broadly covered the use of metal grout for achieving gap free surface between the metal contacts at joints. The authors have documented a case study of metal grouting applied on the Chenab Bridge. Sh. P.S. Anudeep Babu, Sr. Engineer - Planning, CBPU has come up with the article Turning Of Deck Segments which has been developed indigenously at site. The author has described various steps involved in turning of segment with sketches and photographs in article.

I am elated and must compliment USBRL team for making arduous endeavors in publication of this document. I am sure that our team will take forward this fascinating journey of publication of magazine to enrich all readers, engineers and professionals interested on the subject.
Bolte Joints / 50
Geotechnical Assessment of the Slope and Foundation Conditions for Piers No. A1, P1, P2 and P3 of Bridge No. 43, Katra-Banihal Rail Line Section, Reasi, J&K / 54
Consolidation Grouting of Strata Underneath Arch Foundation of Chenab Bridge Foundation S-40 location / 64
Piles and Micropiles / 71
Chenab Bridge-Ionic Bridge of Mighty River / 73
Installation of Spherical Bearings in Chenab Bridge Project / 80
Metal Grout System for 100% for cc Fit Gap Compensation in Steel Constructions-Application Case on Chenab Bridge Arch Plates / 89
Turning of Deck Segments / 94

BRIDGES
The Cable Stayed Railway Bridge Crossing the Anji Khad River Along the New B.G. Railway Line Udhampur-Srinagar-Baramulla-J&K State, India / 42
Testing and Evaluation of Slip Factor in HSFG

TUNNELS
Tunnel Blast Design / 17
Review of the Curing Compound And Application Techniques / 27
Saga of Tunnelling Construction on Udhampur-Srinagar-Baramulla Rail Link Project-Himalayan Wonder / 33
Toussaint-Heintzmann (TH) or Top Hat Steel Ribs- A Flexible Support System / 38
PHOTO GALLERY

Pier P4 (89m tall) of Bridge no. 39 on Katra-Banihal section.

Reasi station yard site location.
Banganga Bridge on Katra-Banihal Section

View of Chenab Bridge

Veth Bridge in Kashmir Valley

Steel piers at Chenab Bridge

Arch Fabrication Workshop at Chenab Bridge Site
(Above): Bird’s Eye View of Trial Assembly Of Arch at Chenab Bridge Site
(Below): Panoramic View of Chenab Bridge Site
(Above): Bridge no. 186 (Jhajjar Bridge). (Below): Fixing of Lattice Girder in tunnel
(Above): Viaduct of Tawi Bridge. (Below): Sardan Bridge on Jammu-Udhampur Section
(Above): Main Arch Foundation of Chenab Bridge at Jammu End
(Below): Steel Pylons of 127m for Launching of Arch and Piers of iconic Chenab Bridge of USBRL Project
(Above): DMU approaching Banihal Railway Station, Drilling and charging activities in Tunnel T-13 (9.3 Km length) in progress on USBRL Project. (Below): Drilling Jumbo at Work in Tunnel T48 in Dharam area of USBRL Project, Panoramic View of Portal Area of Tunnel T74 on USBRL Project.
**Introduction**

Boring of tunnels are nowadays a quite common feature in civil engineering and mining projects. A tunnel can be excavated by conventional drilling and blast method or by mechanical method using a tunnel boring machine (TBM) or road headers. An appreciable proportion of world’s annual tunnel advance is still achieved by drilling and blasting method. In drilling and blasting method the tunnel is driven by either resorting to full face or by excavating in parts to its full dimension depending upon the tunnel cross section area and geological conditions encountered. Due to advantages like low investment, easy acceptability to the practicing engineers and wide versatility the drilling and blasting method prevails so far over the mechanical excavation method.

In tunnel blasting, explosives are required to perform in a difficult condition, as single free face (only tunnel face) is available in contrast to bench blasting where at least two free faces exist. Hence more drilling and explosives are required per unit volume of rock to be fragmented in the case of tunnel blasting. A second free face, called ’cut’, is created initially during the blasting process and the efficiency of tunnel blast performance largely depends on the proper development of the cut. The factors influencing the development of the cut and the overall blast results are dependent on a host of factors involving rock mass type, blast pattern and the tunnel configurations. The tunnel blasting mechanics can be conceptualized in two stages. Initially, a few holes called cut holes are blasted to develop a free face or void or cut along the tunnel axis. Once the cut is created, the remaining holes are blasted towards the cut. The results of the tunnel blasting depends primarily on the efficiency of the cut hole blasting. The subsequent section briefly describes various types of cuts.

**Mechanism of Rock breakage:**

Mechanism of rock breakage while release of Explosives energy upon detonation and other relevant points are discussed below:
- When an explosive charge is detonated, chemical reaction occur which, very rapidly changes the solid or liquid explosive mass into a hot gases.
- This reaction starts at the point of initiation where detonator is...
connected with explosives and forms a convex like shock wave (Compressive wave) on its leading edge that acts on the borehole wall and propagates through the explosive column.

- Ahead of the reaction zone are undetonated explosive products and behind the reaction zone are expanding hot gasses.

Understanding theory of detonation of explosives:
The self-sustained shock wave produced by a chemical reaction was described by Chapman and Jouquet as a space. This space of negligible thickness is bounded by two infinite planes - on one side of the wave is the unreacted explosive and on the other, the exploded gases as shown in the Fig. 1. There are three distinct zones: a) The undisturbed medium ahead of the shock wave, b) A rapid pressure at Y leading to a zone in which chemical reaction is generated by the shock, and complete at X, c) A steady state wave where pressure and temperature are maintained. This condition of stability condition for stability exists at hypothetical X, which is commonly referred to the Chapman - Jouquet (C-J) plane. Between the two planes X and Y there is conservation of mass, momentum and energy. Velocity of detonation (VOD) of explosive is function of Heat of reaction of an explosive, density and confinement. The detonation pressure (unit in N/m²) that exists at the C-J plane is function of VOD of explosives. The detonation of explosives in cylindrical columns and in unconfined conditions leads to lateral expansion between the shock and C-J planes resulting in a shorter reaction zone and loss of energy. Thus, it is common to encounter a much lower VOD in unconfined situations than in confined ones.

Rock breakage by Detonation and Interaction of explosive energy with rock:
There are three sources of generation of fragments in mines:
(a) Fragments formed by new fractures created by detonating explosive charge,
(b) In-situ blocks that have simply been liberated from the rock mass without further breakage and
(c) Fragments formed by extending the in-situ fractures in combination with new fractures. Rock fragmentation by blasting is achieved by dynamic loading introduced into the rock mass. The explosive loading of rock can be separated into two phases, the shock wave and gas pressure phase (Fig. 2).

- Rapid the detonation process, the quicker the energy release from explosives mass, in the form of a shockwave followed by gas pressure, is applied to the borehole wall. In other words, faster the detonation velocity of the explosive, quicker is the energy applied to the borehole wall, and for a shorter time period.
- Conversely, with a slower detonation velocity, the energy is applied more slowly, and for a longer time period. The degree of coupling between the explosive and the borehole wall will have an effect on how efficiently the shockwave is transmitted into the rock.
- Pumped or poured explosives will result in better...
transmission of energy than cartridge products with an annular space between the cartridge and the borehole wall.

- Again, the pressure that builds up in the borehole depends not only upon explosive composition, but also the physical characteristics of the rock.
- Strong competent rock will result in higher pressures than weak, compressible rock.
- When the shock wave reaches the borehole wall the fragmentation process begins.
- This shock wave, which starts out at the velocity of the explosive, decreases quite rapidly once it enters the rock and in a short distance is reduced to the sonic velocity of that particular rock.
- Most rock has a compressive strength that is approximately 7 times higher than its tensile strength, i.e. it takes 7 times the amount of energy to crush it as it does to pull it apart.
- When the shockwave first encounters the borehole wall, the compressive strength of the rock is exceeded by the shockwave and the zone immediately surrounding the borehole is crushed.
- As the shockwave radiates outward at declining velocity, its intensity drops below the compressive strength of the rock and compressive crushing stops.
- The radius of this crushed zone strength varies with the compressive of the rock and the intensity of the shock wave, but seldom exceeds twice the diameter of the borehole.
- However, beyond this crushed zone, the intensity is still above the tensile strength of the rock and it causes the surrounding rock mass to expand and fail in tension, resulting in radial cracking.
- The hot gas following the shockwave expands into the radial cracks and extends them further.
- This is the zone where most of the fragmentation process takes place.
- However, if the compressive shockwave pulse radiating outward from the hole encounters a fracture plane, discontinuity or a free face, it is reflected and becomes a tension wave with approximately the same energy as the compressive wave.
- This tension wave can possibly "spall" off a slab of rock (see figure 3).
- This reflection rock breakage mechanism depends heavily upon three important requirements:

a. The compressive wave (and resulting reflected tensile wave) must still be of sufficient intensity to exceed the tensile strength of the rock,
b. The material on opposite sides of the fracture plane or discontinuity must have different impedances,
c. The compressive pulse must arrive parallel to, or nearly parallel to, the fracture plane or free face.

- If carried to extreme, when this reflective breakage or "spalling" process occurs at a free face, it can result in violent throw, a situation that is not desirable.
- This can be overcome by designing blasts with burden and spacing dimensions that are within reasonable limits.
- Once the compressive and tensile stresses caused by the shockwave drop below the tensile strength of the rock, the shock wave becomes a seismic wave that radiates outward at the sonic velocity of the material through which it passes.
- At this point, it is no longer contributing to the fragmentation process.

Technique of the blasting:
Tunneling in rocks is currently performed mainly by blasting, as this method only is capable of providing sufficiently high effectiveness and economics in the construction of tunnel in tough rocks. Tunneling by ‘tunnel borers’ is considered to be less effective especially as regards the construction of tunnels of large cross sectional areas.

The blasts in tunnels and drifts are characterized by lack of adequate free surfaces towards which breakage can occur effectively. Unlike bench blasting, tunnel blasting has only one free face and holes are drilled normal to the...
TUNNELS

Free face surface. In such a situation, the explosives charge will blow out a narrow funnel-shaped crater. But if the hole is drilled at a certain angle to the free face, the result will be better, as the major part of the gasses will break out the rock in the direction of free face (Wedge cut).

Alternatively, if large diameter dummy holes parallel to the blast holes are drilled, the breakage performance is better as the large diameter dummy holes provide additional free face (Burn cut).

Thus, the principle behind tunnel blasting is to create an opening by means of a cut (a set of holes that provide initial free face) and then stoping to enlarge the opening. The cut, usually, has a surface area of 1 m² - 2 m², although with large drilling dia holes it can reach up to 4 m². The different zones in tunnel blasting are shown in Fig-3. The initial opening/cut created either by angled holes or by holes drilled parallel to large diameter dummy holes are widen subsequently by the holes fired after cut holes using proper delays. In other words, the main difference between tunnel blasting and bench blasting is that tunnel blasting is done towards one free surface, while bench blasting is done towards two or more free surfaces. The rock is thus more constricted in the case of tunneling, and a second free face has to be created towards which the rock can break and be thrown away from the surface. This second face is produced by a cut in the tunnel face, which can be a parallel hole cut, a V-cut, or a fan-cut.

After the cut opening is made, the stoping towards the cut begins. Stoping can geometrically be compared to bench blasting although it requires powder factors (The quantity of explosive used per unit of rock blasted) that are four to ten times higher. Such a high explosive consumption is mainly due to drilling error, the demand made by swelling, the absence of hole inclination, the lack of cooperation between adjacent charges and also blasting against gravity in case of lifter holes. The final shape of the cross section is given by trimmers or contour holes with closer spacing and comparatively smaller charge. Contour holes are spaced closely (0.2m to 0.4m apart) and directed outwards to make room for the drill in collaring and advance. The position of the cut has influence on rock projection, fragmentation and also on the number of blast holes. Of the three positions, namely, corner, lower centre and upper centre, the latter is usually chosen as it avoids the free fall of the material, the profile of the broken rock is more extended, less compact and better fragmented.

Fig-3
TYPES OF CUT:
In general there are two types of cuts namely the Burn cut (parallel-hole cut) and the wedge or V-cut (Angled-hole cut).

Burn Cut:
The principle of the burn cut is to drill a number of closely-spaced parallel holes at right angles to the face so as to shatter the rock in the blast and expel it in small fragments to leave a long, roughly cylindrical cavity. It is most important that burn cut holes be drilled as parallel as possible and at the design distance from each other. Burn cuts can be located anywhere in the face and they are often drilled off center. With the aim of maximizing safety, the position of the cut should be varied from round to round to avoid the necessity of drilling the next cut in the bottom of the previous cut. The use of ANFO (Ammonium nitrate fuel oil explosive) should sometimes be avoided in the burn cut blast holes. Blasters who use ANFO for most of the round may require to use a small-diameter cartridge explosive or an ANFO/polystyrene mixture (low density) in burn cut blast holes. Typical arrangements of burn cuts drilled out entirely with small-diameter holes are shown in Figure(4). The 6 variations shown in this figure are ranked in estimated order of effectiveness, that is cut (a) is most effective and cut (f) is least effective.

Wedge Cut:
In angled cuts/ wedge cuts, blast holes are drilled at an angle to the face so as to provide as much freedom of movement for the broken rock as possible. When these blast holes are detonated, a wedge of rock is ejected. Wedge cuts can consist of 2 or more rows of blast holes, and the holes should be angled so that the angle in the earliest-firing wedge is as near as possible to 60°. The toes of the wedge blast holes should be at least 250mm and preferably 300mm. The principal wedge should be
drilled some 150-200mm deeper than other blast holes. Wedge cuts may be horizontal or vertical depending upon which tunnel dimension allows the greater angle. Where long pulls are required, cut should consist of a sequence of wedges symmetrically ranged about a common centre line; each succeeding wedge should break a similar burden of rock. These variations are termed double wedge cuts, triple wedge cuts, etc. Well laminated or fissured rocks respond well to wedge cuts, and where blast holes can be aligned with a large apical angle (as in large-diameter tunnels), the deepest pulls are possible. Once the wedge cut has fired, the remaining blast holes in the round detonate (on subsequent delay periods) in the same way as those in burn cut rounds. Generally, the toe burden for the first easers should not exceed about 0.65m. Different types of wedges used in wedge cut pattern are shown in figure (5).

PARAMETERS INFLUENCING TUNNEL BLAST RESULTS:
The parameters influencing the tunnel blast results may be classified in three groups:
- Non-controllable - Rock mass properties,
- Semi-controllable-(a) Tunnel geometry-(b) Operating factors and
- Controllable-Blast design parameters including the explosive properties. The subsequent section briefly describe the blast design parameters.

BLAST DESIGN PARAMETERS:

Depth of round:
The depth of round is an important parameter in tunnel blasting, as most of the excavation engineers desire a higher rate. There are two options for obtaining a high advance rate. The first one is to go for a deeper round, which may invite more stratacontrol problems unless the ground is competent or smooth blasting practice is followed. The second option is based on pulling shorter rounds with smaller cycle time. This becomes a useful option, particularly when the rock mass is weak. A better pullefficiency is also expected in the second option. However, the drilling resources is the dominating factor in deciding the advance per round in most of the tunnels in India.

A rough guideline on the length of blast holes in the cut and easer holes of a convergent or angled cut is provided by Pokrovsky (1980),

for cut holes, \( l_c = 0.75 \times (A)^{0.5} \) m
for easer holes, \( l_e = 0.5 \times (A)^{0.5} \) m

Where,
- \( A \) = tunnel area, m²,
- \( l_c \) = length of cut hole, m and
- \( l_e \) = length of easer hole, m.

According to Holmberg (1982), the depth of a round in a parallel cut depends on the size of the relief hole (equation (11.8)) as given by the following relation:

\[ A_d = 0.15 + 34.1 \times d \times 39.1 \times d^2 \] m

Where, \( A_d \) is the depth of round (m) and \( d \) is the diameter (m) of the relief hole. In case of more than one relief hole of similar size, the equivalent relief hole diameter \( d_{re} \) (m) should be considered for estimating the round depth. It is obtained by multiplying the relief hole diameter by \( r \), where \( r \) is the number of relief holes (Olofsson, 1988).

Holes per round:
The number of holes per round is decided mainly by the tunnel size and hole diameter. Ziegler (1985) reports that the number of holes per round in a drift reduces by 3 percent with every 0.001m increase in diameter of the explosive cartridge.

Based on US experience, Whittaker and Frith (1990) suggested the number of holes for various tunnel sizes in weak and strong formations are given in table 1 as follows:

<table>
<thead>
<tr>
<th>Tunnel crosssection (m²)</th>
<th>Number of holes per round</th>
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<td></td>
<td>Weak</td>
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<td>10</td>
<td>23-27</td>
</tr>
<tr>
<td>25</td>
<td>45-50</td>
</tr>
<tr>
<td>50</td>
<td>75-85</td>
</tr>
</tbody>
</table>

Explosives and accessories:
Explosive is a compound or a mixture of compounds, which rapidly decompose, releasing large quantities of heated gases at high pressure. When properly initiated, it is very rapidly converted into gases at high temperature and pressure. This process is called detonation. One liter of modern high explosive will expand to around 1000 liters with milliseconds, creating pressure in a blast hole
of the order of 10,000 Mpa. Temperature ranges from 1650-3870°C and the velocity of detonation (VOD) is so high up to the order of 2500-8000 m/s and the power of a single charge is around 25,000 MW. Some of the properties which govern the selection of particular type of explosives are strength, VOD (velocity of detonation), density, water resistance, fume characteristics, sensitivity (impact and friction) and thermal stability. NG (Nitroglycerine) based explosives are one of the most commonly used in tunneling and fragmentation of hard rocks in spite of the fact that large amount of noxious gas is generated from it leading to long defuming time. NG based explosives are being replaced by slurries and emulsion based explosives. Emulsion explosives are the newest form of commercial explosive and have excellent performance characteristics and flexibility of use. Emulsion blasting agent is a water-in-oil emulsion consisting of a super-saturated solution of microscopic AN (ammonium nitrate) droplets suspended in an oil, wax or paraffin fuel and stabilized with emulsifying agents. Entrapped air, in the form of either ultra-fine air bubbles, dispersed throughout the emulsion, acts as a sensitizer. Emulsions are higher energy output, higher reaction rate, better water resistance, better temperature resistance and higher density as compared to slurries. On initiation, the explosive shock wave causes the air bubbles to compress at high speed, thus creating hot spots and causing the emulsion to detonate.

The amount of entrapped air controls the sensitivity and can be varied to create a product that is either a high explosive or a blasting agent. Blasting technology has
achieved a significant development with the introduction of non-electric detonators (initiation system) known as NONEL system which was innovated and developed by Nitro Nobel (Olofsson, 1988). Now a days electronic detonators are also available and have been successfully field tested.

**Charge per hole:**
The explosives consumption increases if the angle of breakage is small. The easier holes in a parallel cut are blasted with small breakage angle against the free face created by the cut. Langefors and Kihlstrom (1973) suggested the following relations to estimate the linear charge concentration in a hole breaking towards a narrow opening or free face (circular or rectangular as depicted in fig. (3):

- **Circular opening** - 
  \[ q_{lco} = 0.55(Dc - W/2)/ \left( \sin \upsilon_a \right)^{3/2} \text{kg/m} \]

- **Rectangular opening** - 
  \[ q_{lro} = 0.35(Dr)/ \left( \sin \upsilon_a \right)^{3/2} \text{kg/m} \]

Where,

- \( q_{lco} \) = linear charge concentration in case of a circular opening, kg/m.
- \( q_{lro} \) = linear charge concentration in case of a rectangular opening, kg/m.
- \( Dc \) = center to center distance of blast hole from circular opening, m.
- \( Dr \) = distance of blast holes from rectangular opening,
- \( W \) = width of the opening, m and
- \( \upsilon_a \) = half of the aperture angle (°) or angle of breakage.

**Type delay and sequence of initiation:**
The delay time must allow the following events to reach completion or at least to be well underway before initiation under subsequent delay.

- Travel of the compressive waves through the burden to face and back to the blast hole.
- A subsequent readjustment of the initial stress field due to the presence of the primary radial cracks and the effect of the reflection of stress waves from the free face
- Acceleration of the broken rock mass, by the action of the explosion gases, to a high velocity to ensure the proper horizontal motion which controls the muck pile profile and the design geometry.
Blasting in T 74R North Portal:

Explosive used:
Explosive used in T-74R North portal is SUPERPOWER 80, it is packaged ammonium nitrate (AN) based emulsion explosive Fig. 4.

Detonater:

Non-electric detonators:
It consists of a shock tube, with a length determined by the blast design, connected to a high power detonator on one end and other end of the shock tube is sealed and has a Cobra type plastic connector with a sticker indicating the number of the delay. This shock tube is small diameter, three-layer plastic tube, coated on the innermost wall with a reactive explosive compound, which, when ignited, propagates a low energy signal, similar to a dust explosion. The reaction travels at approximately 6,500 ft/s (2,000 m/s) along the length of the tubing with minimal disturbance outside of the tube.

Shock tube delivers the firing impulse to the detonator, making it immune to most of the hazards Fig. Detonating cord (Fig. 6) is a thin, flexible plastic tube usually filled with Penta Erythritol Tetra Nitrate (PETN, Pentrite). With the PETN exploding at a rate of approximately 6400 m/s, any common length of detonation cord appears to explode instantaneously. It is a high-speed fuse which explodes, rather than burns.

The velocity of detonation is sufficient to use it for synchronizing multiple charges to detonate almost simultaneously even if the charges are placed at different distances from the point of initiation. It is used to reliably and inexpensively chain together multiple explosive charges. As a transmission medium, it act as a downline between the initiator (usually a trigger) and the blast area, and as a trunkline connecting several different explosive charges. Explosion in detonating cord is triggered by a flame or spark, electrical current, or mechanical shock. In T-74 R North portal detonation is done by electric current.
CONCLUSION:
Underground construction involves different sizes and shapes of tunnel excavation in various rock mass/ geological conditions. Appropriate blast design including drilling pattern, quantity of explosives used, explosive type and proper initiation sequence is essential to achieve safe efficient smooth profile and good advance rate at minimal damage to surrounding rock mass.
1. Introduction
The object of curing is to keep concrete saturated, or nearly saturated as possible, until the originally water filled space in the fresh concrete is filled to the desired extent by by-products of hydration of cement. The necessity for curing arises from the fact that the hydration of cement can take place only in water filled capillaries. That is why a loss of water by evaporation from the capillaries must be prevented. Evaporation of water from concrete, soon after placing depends on the temperature and relatively humidity of the surrounding air and on the velocity of wind over the surface of the concrete. Curing is essential in the production of concrete to have the desired properties. The strength and durability of hardened concrete will be fully developed only if, it is properly cured. The amount of mixing water in the concrete at the time of placement is normally more than the required for hydration & that must be retained for curing. However, excessive loss of water by evaporation may reduce the amount of retained water below what is necessary for hydration to develop the desired properties. The potential harmful effect of evaporation can be prevented either by wet curing or membrane curing. The hydration of cement to continue, the relative humidity inside concrete has to be maintained at a minimum of 80 per cent. If the relative humidity of the ambient air is at least that high, there will be little movement of water between the concrete and the ambient air, and no active curing is needed to ensure continuity of hydration. Active curing can be prevented if no other factors intervene e.g there is no wind, there is no difference in temperature between the concrete and the air, and if the concrete is not exposed to solar radiation. In practice, these ideal conditions of high humidity coupled with minimal temperature difference between concrete and air, can be obtained deep inside the
tunnel, where no active curing, or very little curing may provide suitable environment to prevent evaporation of water from concrete.

1.1 Methods of Curing:
The two systems of maintaining satisfactory moisture content are:
- Continuous or frequent application of water through ponding, sprays, steams, or saturated cover materials such as burlap or cotton mats, rugs, earth, sand, sawdust and straw.
- Prevention of excessive loss of water, from the concrete, by the application of curing compound to the freshly placed concrete.

1.2 Types of curing compounds available in market
Concrete curing compound consists essentially of waxes, natural and synthetic resins, and solvents of high volatility at atmospheric temperatures. The compound forms a moisture retentive impermeable layer shortly after being applied on fresh concrete surface. White or gray pigments are often incorporated to provide heat reflectance, and to make the compound visible on the structure for inspection purpose.

   The compound should be applied at a uniform rate. Curing compound can be applied in two applications at right angles to each other by hand or power sprayer usually at about 0.5 to 0.7 MPa pressure. For small areas, the compound can be applied with a wide, soft-bristled brush or paint roller. For brush or roller application, use equipment recommended by the curing compound manufacturer (para 4.11 of ASTM C 156 -03, Standard test method for Water Retention by Concrete Curing Materials).

   For maximum beneficial effect on open concrete surfaces, compound must be applied after finishing and as soon as the free water on the surface has disappeared and no water is visible, but not so late that the liquid curing compound will be absorbed by the concrete.

   When forms are removed, the exposed concrete surface should be wetted with water immediately and kept moist until the curing compound is applied. Just prior to application, the concrete should be allowed to reach to a uniformly damp appearance with no free water on the surface and then application of the compound should begin at once.

1.2.1 Uses of curing compound
Curing compound can be used with advantage where wet curing is not possible. It is very suitable for large areas of concrete which are directly exposed to sunlight, heavy winds and other environmental influences. It can be used for curing of:
- Concrete pavements, airport runways, bridge decks, industrial floors.
- Canal linings, dams and other irrigation related structures.
- Sport arenas and ice ring.
- Precast concrete components.
- Roof slabs, columns and beams.
- Chimneys, cooling towers and other tall structures.

2. Specifications of Curing Compound
American Society for Testing and Materials (ASTM) C 309 covers specifications for Liquid membrane-forming compounds suitable for application to concrete surface to reduce the loss of water during early- hardening period. White - pigmented membrane forming compound serve the additional purpose of reducing the temperature rise in concrete exposed to radiation from the sun. The membrane forming compounds covered by specification in ASTM 309 are suitable for use as curing media for fresh concrete, and may also be used for further curing of concrete after removal of forms or after initial moist curing.

2.1 Classification of curing compounds as per ASTM
Liquid membrane-forming compounds are classified according to the color of the compound and the type of solid constituent present for forming the membrane. Table 1 shows the classifications for membrane-forming compounds as ATM 309.

<table>
<thead>
<tr>
<th>COLOUR</th>
<th>SOLID CONSTITUENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Description</td>
</tr>
<tr>
<td>1</td>
<td>Clear or Translucent w/out Dye</td>
</tr>
<tr>
<td>1-D</td>
<td>Clear or Translucent w/Fugitive Dye</td>
</tr>
<tr>
<td>2</td>
<td>White Pigmented</td>
</tr>
</tbody>
</table>
3. Curing compound used by AFCONS

AT - Cure WR125W, which is Water based concrete curing compound is being used by AFCONS in Bridge no 39 at Reasi. AFCONS was using this product on DMRC project (Job code CC15) at Sarita Bihar casting yard. AFCON arranged the material from Delhi for this Project. Since the material was already being used on a DMRC Project, therefore, the same material was tried for use on this project. After field trials and laboratory tests, the materials was found suitable, therefore, permitted for use on trial basis on pier no 4, which is 89.15 m high. The performance of the curing compound was found satisfactory, and allowed to be used on other piers also.

3.1 Working Principle of AT - Cure WR125W

After spraying AT - CURE WR125W creates secondary hydration within the concrete which densifies the substrate to form an impermeable layer thereby creating a barrier retaining the water in the concrete. As a result, the waters present in the pores of the concrete remains there and the relative humidity remains almost unchanged providing optimum hydration for strength and durability. It is suitable for all general concrete application.

3.2 Advantages of Curing compound used by AFCONS

- Provides a more durable concrete and ensure achievement of maximum strength.
- Reduce surface cracking and shrinkage.
- Provide a dust free surface.
- If applied as per the manufacturers recommendations, ensure perfect curing of concrete.
- Control of moisture loss, improve the surface quality of concrete.
- Does not affect patching, coatings, paints, lane markers or joint sealants.

3.3 Specifications of Curing compound used on Bridge No. 39 (AT - CURE WR125 W)

- **Type:** Water based liquid (Type 1 - D).
- **Color:** White.
- **Drying time:** Approx 1.5 hrs @ 30°C.
- **Specific Gravity:** 1.09 ± .01 @ 30°C
- **Odour:** No odour.
- **Storage:** Cool & dry place.

- **Toxicity:** Non-toxic, non VOC, contain no solvent.
- **Shelf life:** 1year in unopen condition.
- **Packing:** 200kgs.

3.4 Application Methodology of AT-Cure WR125W

AT - Cure WR 125 W requires no mixing, diluting or agitation. Application should begin as soon as the concrete is free from the surface water and can support foot moment without leaving marks. Two thin layer of curing compound has to be applied to the whole surface using a hand operator low pressure spray gun or roller. Immediately after the first coat is dry apply second coat. For larger areas application can be done by power driven automatic equipment. The concrete surface should not be disturbed until it has achieved sufficient strength. In case of hardened concrete i.e. after demoulding of form work, the surface of concrete should be sprayed with water to saturate it prior to the application of curing compound. After spraying all equipment, shall be cleaned with fresh water.

3.5 Coverage of AT - Cure WR125W

5-6 sq. mtr. surface area is covered per kg of curing compound depending upon texture of surface, wind velocity, humidity and temperature.

3.6 Relevant Codes for quality control of liquid curing compounds

The curing compound should be tested in accordance with following ASTM standards.

1. **ASTM C - 309**
2. **ASTM C - 156**
TUNNELS

Photo 1: Application of Curing compound

Photo 2: Cube with Application of curing compound & traditional wet curing

Photo 3: Curing compound cured finished Surface after drying up
### TABLE-2: CUBE STRENGTHS DURING REGULAR CONCRETING WORK (AVERAGE 7 DAYS CUBE STRENGTH)

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Pier no</th>
<th>Height of pier</th>
<th>Quality of concrete in cum</th>
<th>Date of casting From</th>
<th>Date of casting To</th>
<th>Average cube strength (7 days) Curing compound No</th>
<th>Average cube strength (7 days) Water cured No</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>p1</td>
<td>35.1</td>
<td>608</td>
<td>14.03.18</td>
<td>26.03.18</td>
<td>39 43.06</td>
<td>94 44.45</td>
</tr>
<tr>
<td>2</td>
<td>p4</td>
<td>89.1</td>
<td>1713</td>
<td>01.12.17</td>
<td>23.01.18</td>
<td>- -</td>
<td>- -</td>
</tr>
<tr>
<td>3</td>
<td>P6</td>
<td>74.4</td>
<td>1399</td>
<td>30.01.18</td>
<td>25.02.18</td>
<td>72 44.56</td>
<td>144 49.02</td>
</tr>
</tbody>
</table>

### TABLE-2.1: CUBE STRENGTHS DURING REGULAR CONCRETING WORK (AVERAGE 28 DAYS CUBE STRENGTH)

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Pier no</th>
<th>Height of pier</th>
<th>Quality of concrete in cum</th>
<th>Date of casting From</th>
<th>Date of casting To</th>
<th>Average cube strength (7 days) Curing compound No</th>
<th>Average cube strength (7 days) Water cured No</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>p1</td>
<td>35.1</td>
<td>608</td>
<td>14.03.18</td>
<td>26.03.18</td>
<td>36 51.42</td>
<td>87 51.97</td>
</tr>
<tr>
<td>2</td>
<td>p4</td>
<td>89.1</td>
<td>1713</td>
<td>01.12.17</td>
<td>23.01.18</td>
<td>- -</td>
<td>- -</td>
</tr>
<tr>
<td>3</td>
<td>P6</td>
<td>74.4</td>
<td>1399</td>
<td>30.01.18</td>
<td>25.02.18</td>
<td>69 51.11</td>
<td>171 51.67</td>
</tr>
</tbody>
</table>

3.7 Conformity of material test (Curing compound)
Third party test of AT - CURE WR125 W was carried out at STANDARD TESTING LAB., NEW DELHI. The results obtained are summarized below:

3.8 Trial testing Comparison of wet curing Vs curing compound cured cubes:
Three sets of cubes were casted to check the 28 days strength with different methods of curing and the results obtained are as given below:
Comparing the compressive strength results of the casted cubes, it is observed that compressive strength of concrete cubes cured with curing compound is better, and can be adopted in place of simple water curing.

3.9 Cost of Application of AT - CURE WR125 W:
Cost of curing compound per litre = Rs. 76 (approx).
Coverage per litre of curing compound = 5-6 Sq.m. (based on actual consumption at site).
Approximate cost of application of curing compound = Rs. 15/Sq.m. This is approximate cost and may vary depending upon the type of surface and mode of application.

Note: Curing by water (wet curing) has also cost implication, which includes cost of manpower deputed for sprinkling/ponding water regularly plus cost of electric consumption in pumping water etc. The cost of curing by curing compound may work out to very less or naturalise, if the cost of wet curing (inclusive in the cost of concreting), is deducted from above cost.

4.0 Assessment of performance of curing compound on Bridge no 39
After trial testing, the compound was allowed to be used for curing of concreting piers. The efficiency of the curing compound was found to be satisfactory during regular use, which can be observed from the average cube strength (table 2.0 & 2.1 below), obtained during concreting of piers of Bridge no 39.

**Grade of Concrete:** M40, **WC Ratio:** 0.36, **Admixture:** Master Polyheed 8630, **Dosage:** 1%

### TABLE:

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Tests</th>
<th>Requirement</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Water loss after 72 hrs, Kg/m² (ASTM C-156)</td>
<td>0.55 max</td>
<td>0.41</td>
</tr>
<tr>
<td>2</td>
<td>Drying time, hrs (ASTM C-156)</td>
<td>3 to 4</td>
<td>1.25</td>
</tr>
</tbody>
</table>
5. Conclusions
Based on the experience gained and results of compressive strength of concrete cubes obtained in Bridge no 39, it can be concluded that:
- The concrete curing compounds are effective in preventing evaporation of water from the concrete provided continued and uniform application is ensued by close supervision.
- At most of the construction sites, wet curing is often applied. Wet curing is easy and requires continuous spraying or flooding (ponding), or by covering the concrete with sand or earth, covering with hessian cloth etc. But in high rise vertical structures such as high piers of bridges, where ponding and/or covering the surface with hessian cloths etc. are not practically feasible, use of curing compound provides a practical solution.
- Numbers of curing compound products are available in the market. Selection of curing compound, ensuring quality of supply and proper application of curing compound may lead to better results of hardened concrete.
- Where water curing is inconvenient or potable water for curing is not available, sealing fresh concrete surfaces with curing compound is the best way of curing.

6. Disclaimer
This report has been prepared as an account of work of construction of major Bridges on USBRL Project being executed by M/s. AFCON Infrastructure Limited under supervision of Konkan Railway Corporation Limited (KRCL). This report presents the results of only one product used at site. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise, does not necessarily constitute or imply its endorsement, recommendation, or favouring by Northern Railway or KRCL. The views and opinions of authors expressed herein do not necessarily state or reflect those of the Indian Railway or any agency thereof.

7. Recommendations
The scope of this report is limited to assessment of only one product of curing compound used on Bridge no 39, which is water based liquid. Therefore, the following is recommended:
- Other concrete curing compound consisting essentially of waxes, natural and synthetic resins, and solvents of high volatility at atmospheric temperatures, are available in the market. Before permitting extensive use of concrete curing compounds of RCC structure, and or finalising the guidelines of specification, tests on other products available in the market may also be carried out so that best available products are used for such an important work as curing.
- To ensure application of two coats in the field, pigment of different colour may be used for 2nd coat, if possible.
Down the millennia, Jammu and Kashmir has been the land of legends. A place where mystics and emperors, traders and warriors came, saw and were astounded by its sheer beauty. The state of Jammu and Kashmir has three distinct divisions - Jammu region, Kashmir valley and Ladakh area. All three areas have their own appeal, but it is still Jammu and particularly Kashmir that continues to fascinate the scholar and the layman, the traveler and the poet, not to mention the tourist and the artist. Whatever the etymology of this state, as the crowning glory of the Indian subcontinent, it has been adored and coveted for centuries. All these twists and turns of history has endowed the state with a composite culture that is as unique, varied and mesmerizing as its topography. Though the Himalayan ranges shelter the state from the more bitter winter winds and temperatures, snow does regularly coat its hills and dales in ethereal white.

Little wonder then that India is rejoicing that the Paradise of beauty and plentitude is finally being linked to the rest of the country by twin bands of steel, courtesy the Indian Railways. It will be some time before Jammu and Srinagar are connected directly, but for now the tracks of progress have already been laid within the Valley, linking north and south like never before, along with Jammu region penetrating mighty Pir Panchal.

Bringing the Railways to the fabled Jammu & Kashmir has been one of the most challenging projects that the Indian Railways has ever contemplated and executed. When complete, it will link Jammu to Srinagar and beyond to Baramulla, but even the just-completed segments within the Jammu region and Kashmir valley has been a path breaking task, and a feather in the cap of Northern Railways and particularly of USBRL. Moreover, this Railway line and its progress symbolizes not only the goodwill and determination of a country but also the capability of its human resources.

Though, it is a small fraction of the Indian Railways’ total of more than 67,300 km of rail tracks, the commissioning of the scenic 119 km Broad Gauge line of Kashmir valley, 18 km linking Kashmir valley to Jammu region through country’s longest transportation tunnel (11.25 km) monikered in local parlance as Pir Panchal tunnel and technically as T-80 by Railway men, and 78 km from Jammu to Udhampur, marks the culmination of a vision that is over a century old. It is as imperative now as it was in the 19th century when it was first envisaged, because till a railway link is complete, the National Highway 1A remains the sole surface link to Srinagar in the Kashmir Valley, from the nearest railhead Jammu, 300 km away. Snow cuts off this link in winter and it is vulnerable to landslides and traffic jams.

Though the official starting point of this great engineering adventure was the mention of the 272 km Udhampur-Srinagar Rail link project (USBRL) in the ‘pink book’ of the Railway Budget of 1994-95, it was only in 2002 that it was declared a National Project by the Prime Minister, thereby releasing it from the...
constraints of the Railways budget and bringing the vaster funds of the central government into play. The actual idea, however, germinated much before that in the form of another project linking Jammu to Udhampur of 54 km, termed as Jammu-Udhampur Rail link project (JURL), which was successfully commissioned in 2005, meeting the dreams of local populace and other citizen of Nation. The seriousness of the mission to keep the train chugging in Kashmir is unmatched. The 18-km stretch between Banihal and Qazigund has been supervised intimately by higher echelon of Railways to turn the dreams into reality.

The mighty Himalayas are range of young folded mountain created as a result of collision of Indo-Australian plate with Eurasian plate as recently as about 20 millions years or so and rock formation is a matrix, still in the process of stabilization and metamorphism. Tectonic movements are regular features. Folds and faults due to tectonic movements within the Himalayas have resulted in the region having igneous, metamorphic and sedimentary rock formations. The dominant rocks around the valley are volcanic and are known as ‘Panjal traps’, while in Jammu region, limestone and dolomite of sedimentary genesis predominates in the initial reaches, which changes to other varieties of metamorphic rocks towards Pir Panchal ranges. There are also three major thrusts and faults along the alignment which makes the whole region prone to seismicity. In fact, most of the alignment lies in Seismic Zone IV, with a portion near Srinagar in Zone V.

Working methodology of tunnelling adopted on USBRL is called New Austrian tunnelling method or NATM. It has often been referred to as a “design as you go” approach, by providing an optimized support based on observed ground conditions. More correctly it can be described as a “design as you monitor” approach, based on observed convergence and divergence in the lining and mapping of prevailing rock conditions.

Beyond Katra on one side of the Pir Panjal is Banihal, and on the other side, Qazigund, which forms the 128 km second leg of the USBRL. The first and third legs of Udhampur to Katra (25km) and Qazigund to Baramulla (118km) respectively have been completed and commissioned and serving the aspirations & ambitions of millions of Indian as well bringing glory to Railways. In addition, small but significant chunk of second leg connecting Qazigund-Banihal (18km) have been commissioned through mighty Pir Panchal tunnel.

The 25.6 km Udhampur-Katra rail line, an engineering marvel is not at all about man’s material advancement. Rather it is about being the lifeline of those pilgrims who are hoping to graduate to their higher selves. The ten tunnels across 10.94 km were an engineering challenge what with railway men battling adverse weather conditions, rain-induced seepage, learning from on-site trial and error and sinking in girders in a remote and tough terrain. There were various constraints such as allowable maximum speed, high gradients and sharp curves. Stations had to be readied for optimum utilisation, safety and minimum maintenance in addition to the basic need of the link being in seamless continuity with the rest of the network.

In 2008, tunnels on Katra-Udhampur section faced seepage problems despite seasoning the site. The 3-km long tunnel near Udhampur was redesigned by an Austrian expert team after an intensive geo-technical investigation and use of imported machinery. The problem lay in the swelling soil or layers of clay that acquire volume while absorbing water and contract when they dry out. Imagine finishing almost 10.94 km of tunnelling after testing, mid-course correction and reviews. The Udhampur-Katra section comprised soft strata. So the tunnel roof was first strengthened and then the central rock mass was taken out. This is the heading and benching method, boring a small opening at the top, allowing a stand-in time with supports and then excavating further. For the extreme case of very soft strata, workmen used the multiple-drift method of advance, in which the individual drifts are reduced to a small size that are safe for excavation. Portions of the support are placed in each drift and progressively connected as the drifts are expanded. The central core is left unexcavated until sides and crown are safely supported, thus providing a convenient central buttress for bracing the temporary support in each individual drift. While this obviously slow multi-drift method is an old technique, the Himalayan condition still forced its adoption as a last resort. The curvilinear or elliptical geometry enables a smooth flow of stresses in the ground around the opening, minimising loads acting on the tunnel linings, thereby ensuring stability even during natural disasters. Due to tectonic movements and
thrusting along major faultlines, engineers routinely encountered loose rocks and the water gushing through them. When thick shear zones, comprising crushed rock are encountered, cavities form. So, when water seeps in, its pressure pushes everything out and blocks the portal. The squeezing and swelling rocks sweated out the best engineers. The railway alignment passes through unstable geological formations and undulating terrain of the Shivalik and Trikuta ranges. Mounting the tracks was a mammoth task for engineers as they had to jump rivers, nullahs, canals, channels, gorges and clefts while curling past roads, cart tracks and footpaths. Design engineers faced the daunting and tricky task of erecting earthquake-proof piers and embankments. USBRL engaged RITES, NHPC and WAPCOS for geo-technical investigations on seismic profiles, field and laboratory testing of soils and rocks. At many places, experts resorted to core drilling, boring a hole deep into the earth. They found varying strata. Some comprised pebbles, cobbles and boulders (up to three metres) embedded in a silty, sandy matrix. Some layers were pervious, others impervious. The seismic velocity in some indicated the absence of a solid bedrock. Based on known geological conditions, materials, properties and construction procedure, engineers divided the tunnel support system into five classes - good, fair, poor, very poor, over-burdened. They decided to provide permanent steel support along the length of each tunnel with a 300-mm thick concrete lining. Forty per cent of the Udhampur-Katra route is covered by tunnels. The longest tunnel on the section at 3.1 km. There are ballast less tracks running inside tunnels on this section.

On Katra-Banihal section of 111 km length, about 88% of stretch, which comes to 97 km is in the tunnel that human’s endurance and zeal is tested, despite the best technology from around the world. This stretch will have a 1.3 km long bridge across Chenab River at a height of 359 m with a 467 m steel arch - undoubtedly an engineering marvel and connoted as world highest Railway Arch Bridge. Wearing safety helmets, gum boots and bright orange jackets lined with fluorescent strips, a battery of young men works tirelessly 24x7 to see the light at the end of the 5.96 km “horse shoe” shaped tunnel T5, the toughest in Reasi area. Tunnel T5 takes off from the proposed Reasi railway station, pierces the hilly terrain in the Gran village and is set to emerge at Bakkal village. It is sunk 700 m from the hill top. But despite challenges of the early days, when equipment had to be airlifted to the site. The 3-km Dharam Khand tunnel no. 3 have been completed. Workforce and engineers encountered fragile dolomite rocks intercepted with calcite intrusions and the same is classified as grade IV in rock formation. In short, it is of poor quality and requires engineering ingenuity for tunnel construction. The biggest challenge in these rock layers is that they contain 70 to 80 per cent of the water of Himalayan aquifers which in natural course ooze out through natural blowholes as cascading streams. When you blast through these pervious layers, the water just gushes out. Which is why the tunnels construction have been hazardous and daunting proposition testing the knowledge and skill sets of the best International and National engineers.

At the initial planning stages of construction, there were geological and hydrological tests to find out the nature of the sedimentary rocks and the bands of impervious and pervious layers. In cases of extreme porosity, engineers explored the possibility of putting an impervious concrete layer and diverting the water flow to another channel. The making of a tunnel itself is no less than an adventure in itself, almost like a deep sea or space exploration.

At the time of encountering of loose strata likely to cause rockfall in mining, It all begins with drilling a 15 m pole into the intended rock face. This is called forepoling and is equipped to bring back samples and geological data through sensors. Once the geologists have studied feasibility, a huge drill bore machine punches holes into the wall for explosive sticks to be inserted. A controlled blast takes place to crumble just two metres after which geologists study the rock patterns once again to set the coordinates and direction for the next two metres. This is just the beginning. Holes are bored again to insert long tubes called weep holes from where the runoff can drain off through channels on the sides. Meanwhile the arch is latticed, reinforced and rock bolted. These days there are self-drilling bolts that find their way in and lock themselves up. Then Matrix-like hoses get to work spraying concrete, a process that is called shotcrete, and cover the excavated walls into a uniform façade encrusted with weep holes that are allowed to remain free, the water from which can be
collected and diverted through a channel. This method is based on an Austrian rock engineering template. So what happens to all the debris that has piled up on the ridge, are they just rolled down the slopes? That would be an easy option but the Railway men are constantly reminded of the hazards of tweaking with the geo-sensitivity of the region. So the debris is carried down to a gentler terrain, often piled into a hill where they hope vegetation will take place and become a part of the landscape. They cannot make a mountain but for sure they can make a hill. At T 3, the water once gushed out at 150 litres/minute while in T 5 there is still a veritable waterfall that has flooded the tunnel waist-high. The heavy seepage of water and potential for formation of cavities at the crossings of shear zones poses a very huge challenge in construction.

Tunneling in Sangaldhan area is no meagre challenge for even extraordinary mortal due to varigated geology, perched aquifers, inaccessibility and remotness of sites. Sangaldan sits on the Muree Thrust or faultline and the entire rock mass in the project area is, therefore, deformed. The presence of nullahs and fresh water springs led to water ingress inside tunnels, sometimes at a force of 1,500 litres per second! Transporting heavy machinery to the site was a challenge as the old Dharamkund Bridge could only withstand payloads of 10 tonnes. So a new steel Road bridge with a higher loading capacity was built by Railways in record time to cross the Chenab and facilitate the work at Sangaldan. At the Dharam site in Sangaldan the construction of Tunnel 48 is in progress. T48 has Ramban formation and T47 has Muree formation. So, working strategies on them were different. The closeness between soft clay and foliated rock mass makes tunneling difficult.

Though the Sumber leg of the tunnel T 49, which is going to surpass Pir Panchal as longest transportation tunnel at 12.76 km surpassing Pir Panchal tunnel of USBRL project, is supposed to be 5.1 km. Remaining 7.66 km commencing from Arpinchalla area have two offshoots technically called Adits at Hingni and Kundan areas to provide additionally working fronts for both main tunnel and escape tunnels and subsequently to serve as rescue and restoration arm in disaster management. Ventilation ducts have been put up to get in fresh air for the workers at the construction stage as per international standards. The main tunnel is 8 m wide while the escape chute spans 5 m. According to international standards, any tunnel longer than 3 km requires an escape tunnel for emergency and rescue and restoration operation. After every 375 m, there is a cross passage connecting the two tunnels. This is generally considered a nominal distance for rescue acts. The terrain here is mountainous with V-shaped valleys, deeply incised since the last glaciations. The work on the tunnel is under way at five separate sites. It is quite a spectacle to watch the engineers and geologists working in tandem to earmark the portion to be excavated for another face opening of this tunnel at Arpinchalla station yard. It is almost like a surgical procedure, where the target area is cleared up, in this case pounded flat and weeds are removed. The portion around the arch, which will in time become the mouth of the tunnel, has been reinforced to avoid crumbling of rocks. A continued effort of 24 hours results in carving just a metre due to geological imponderables and surprises associated with young fold mountains of Himalayas. Before cutting into the rockface, the pipe-roofing is quite a challenge. The engineers earmark a semi-circle and drill 114 mm steel pipes deep inside the rocks, each 40 centimetres apart. These pipes hold up the arch, absorb the stresses and equalise the massive overburden when the earth is gouged out from the tunnel. The pipe-roofing process is a tedious task consuming valuable time period to the extend of 3 days. Despite extreme caution to hold the overburden, the rocks sometimes crumble and even smash the pipes.

The 1,140 metres of mountain strata above the Pir Panchal tunnel T 80, technically known as "overburden", is the highest load on any Indian tunnel. Watching the engineers managing the affairs of the tunnel, from the lighting system to the movement of 25 jet fans installed for air circulation, it is worthwhile to recollect the days when this tunnel was under construction and cynics often questioned the possibility of a train ever reaching the Valley through the stubborn mountains. For geo-technical investigations, the engineers drilled the deepest holes, 640 metres down. It is beyond imagination that the tunnel, which has a dry tarmac now, was at one point of time a rivulet of gushing water. During the excavations, the water-soaked bottom of the mountain seeped from every gap, releasing water jets at 150 to 180 litres per second. The engineers could do nothing other
than using boats to drain out the water and resume tunnelling. It took as much as three months to drain the water physically. Maybe facts and figures will enable a better understanding; the tunnel uses 7,500 metric tonnes of steel and 3,28,000 cubic metres of concrete. More than 10 lakh cubic metres of soil has been displaced, enough to build a flyover in the city. But, it would be just a six-minute velvety swish inside the train. Imagine, negotiating the same stretch in a motor vehicle on a serpentine road carved out on a 4,000 metre-high ridge. For years, an army of 2,000 workers, almost each 20 of them supervised by a trained engineer, belonging to railway and private companies, worked round the clock to drill the mountain. Till now, the 11-km tunnel has maintained the pride of place in the record books. This is India’s longest transport tunnel. The straight and flawless tunnel runs parallel to the north-south direction and perhaps is the first to have automatic ventilation and lighting. During the eventful seven years of excavation, the rugged, rocky and mostly uninhabited terrain posed extraordinary challenges to the engineers and the workers. It was a huge task to carry the men and machinery to the places where angels feared to tread. Even the machines encountered diverse kinds of challenges. At one place the drill bits pierced through the rocks of one texture only to encounter even tougher rocks encircling it. Sometimes a blast extended its limit and forced the crew to work on what had not been anticipated. At times, road headers equipped with cutting blades failed to raze through, forcing engineers to employ the drill-and-blast method. The tunnel is nowadays illuminated like a shrine and travelling through it is great fun. Visitors may not believe that during the construction phase, water flooded the chambers to such an extent that the workers had to ferry men and materials inside on a boat.

The 111-km stretch between Katra-Banihal section, where work is in progress is the toughest stretch of the entire project. Unbelievable as it may sound, but 97 km of the track will pass through 27 tunnels and the balance length jump over 37 bridges and station yards.

The project will provide an all-weather means of transport for an area that is snowbound for most part of the year and has already shown signs of boosting the state economy and development. It has generated tremendous opportunities of employment, directly enhancing the economic status of the people. Affected families (three quarters of their land was acquired for the project) have been provided employment. The above efforts have changed the travel scenario totally.
1. Introduction
In contrast to the rigid and jointed arch support, the basic idea of the flexible steel arch support lies in its capability to slide inwards if a high load-bearing capacity is exceeded and not to fail early by plastic deformation. And in so doing the yielding steel arch support maintains or even increases its load-bearing capacity in spite of tunnel deformation. This yielding capability is achieved by the overlapping configuration and position of the associated connections.

In 1932, "Bochumer Eisenhütte Heintzmann", the German mining supply company, was the first to introduce the concept of a yielding tunnel arch support without any joints in the form of the "TH Channel Profile". The paired TH profile (A/B) developed by Heinrich Toussaint and Egmont Heintzmann on the basis of a submarine engineering concept was then introduced into German deep coal mining in 1933. The further development (Figure 1) up to the single profile as well as the continuous constructive optimization of the profiles and their connection technology led to the TH profile as it is used today.

2. Squeezing and yielding principle
The term "squeezing" refers to the phenomenon of large long-term rock deformations triggered by tunnel excavation. Squeezing may lead to the destruction of a temporary lining or even to a complete closure of the tunnel cross section (Figure 2).

Two basic concepts exist for dealing with squeezing conditions. First one is called "resistance principle", in which a practically rigid lining is adopted, which is dimensioned for the expected rock pressure. In the case of high rock pressures this solution is not feasible. The second one is "yielding principle", which
is based upon the observation that rock pressure decreases with increasing deformation. By installing a flexible lining, rock pressure is reduced to a value that is structurally manageable. An adequate over profile and suitable detailing of the temporary lining will permit the non-damaging occurrence of rock deformations, thereby maintaining the desired clearance from the minimum line of excavation.

3. Typology of flexible tunnel support
There are basically two technical options for accommodating deformation without damage to the lining (Figure 3), (a) Arranging a compressible layer between the extrados of a stiff lining and the excavation boundary; (b) Installation of a yielding lining in contact with the rock face. In the first case, the rock may experience considerable convergence, while the clearance profile remains practically constant as the lining’s stiffness limits deformations. Such a solution is therefore advantageous particularly in cases with slow and prolonged deformations during the service period of a tunnel. It is a standard solution for the final support of tunnels crossing highly swelling rock.

In the second solution, the lining deforms with the rock and, consequently, its circumference shortens. This is possible by an appropriate structural detailing involving either steel sets with sliding connections (Fig. 2-b1) or deformable elements inserted into slots left between stiff lining segments (Fig. 2-b2). Thrust transfer occurs via friction in the first case and via compression in the second. The axial force in the lining is controlled by the frictional resistance of the connectors or by the yielding stress of the deformable elements, respectively.

4. Profile construction
The design profile of the support pioneered by Toussaint and Heintzmann was in sharp contrast to all other support profiles of that time inasmuch as it featured a section modulus in the two axes that was as balanced as possible. As is generally known, for a steel roadway arch...
the circle or parabola shape (Figure 4) is the most favorable support form for supporting the rock, as this approaches most readily the natural arch formation. In contrast to a straight beam, e.g. the doorframe form of the same profile and cross-section, this form has a load bearing capacity which is 3 to 6 times higher.

With an application of the arch form, Toussaint believed it can be achieved without an excess section modulus in the y axis such as is featured by the single-web support profiles (I-section). In comparison to the single web profile, their z axis (Figure 5) is only approximately ¼ of the y value.

In TH steel ribs, ratio of section modulus Wzz to Wyy is approximately 1, so they were thus able to take up the actions or external forces such as compression (buckling), bending (normal force), inclined bending (torsion) and naturally also a certain degree of tilting stability across the elastic to plastic deformation range. Based on this design idea, the Toussaint-Heintzmann profile, designated as TH profile or Top Hat profile was created.

The balanced static values of the profiles, easy installation, increased stability during installation even in fissured rock, the high load-bearing capability in connection with yielding at the deformation limit of the segments, the long service duration and the reusability after re-erection led to an ever greater application of the TH support world-wide in all mining activities. A typical yielding element is illustrated in Figure 6 that shows two Toussaint-Heintzmann profile steel ribs nested...
together and clamped to form a frictional sliding joint. Four or Five of these yielding elements are incorporated in each steel rib and they are set to slide a predetermined distance, depending upon the amount of closure to be allowed, before encountering a positive stop welded onto the rib.

5. Lining Stress Controller
Lining Stress Controller (LSC) have been developed as special supporting measure for highly squeezing rock mass conditions. The primary tunnel lining is divided into several segments by longitudinal construction joints. The purpose of this segmentation is the ability to absorb large deformations occurring during tunnel driving in weak ground. LSC steel elements are installed into these deformation joints.

Lining stress controller is mainly set between two arched supports made of flexible TH sections in shotcrete at slots which match corresponding level of sliding joint of TH steel rib.

LSC consists of circular hollow sections divided or rather linked by intermediate plates (Figure 7).

6. Use of TH steel rib in USBRL project
TH 44 steel ribs are prescribed in Tunnel T-74R for rock class D and E (Table 1 & 2). TH-44 (Figure 8) was used in Rock class D encountered in T-74R Adit Main tunnel in a length of 35 meter at 1.25 meter spacing and in length of 40 meter in T-74R south portal Main tunnel at 1 meter spacing in locations where reprofiling was done due to highly squeezing rock mass.

7. Conclusion
Although the physical and chemical processes taking place in the ground around a tunnel in squeezing and in swelling rock differ from each other, there is one fundamental aspect in these two cases: with increasing rock deformation the rock pressure decreases. This is proved both by experience and theoretical investigations. Based on this fact, nowadays a number of design methods are at the disposal of the engineer to control rock pressure even in heavily squeezing and heavily swelling rock. The key element of the design of the temporary rock support was the fulfillment of the requirement to allow controlled radial displacements. The steel support is provided with sliding joints and yielding beam elements are inserted in the shotcrete lining. In this way the lining is capable of providing considerable rock support (so called lining resistance) and at the same time also permitting convergence leading to a reduction of rock pressure for the final lining.
Introduction
Indian Railways is constructing the most challenging Railway line Project to connect Kashmir Valley with the existing rail network in Jammu by negotiating the mighty young Himalayas under two projects viz Jammu-Udhampur, JURL (54 Km) and Udhampur-Srinagar-Baramulla, USBRL (272 Km). The section from Jammu to Katra (79 Km) and Banihal to Baramulla (136 Km) has since been commissioned. The work in the intervening stretch of Katra-Banihal (111 Km) is in progress, which comprises boring of 27 tunnel and construction of 37 Bridges.

Location of Anji Bridge
Anji Bridge is proposed to be constructed between Tunnel T-2 (towards Katra side) and Tunnel T-3 (towards Reasi side) across Anji Khad (tributary of River Chenab). Construction of Anji Bridge is extremely challenging Engineering task in the balance work of project traversing through deep valley.

Although, it is smaller in comparison to Chenab Bridge but it is also an important bridge on this section and after construction, it will be 195m above the river bed and main span across steep slope of Anji Khad River will be more than 290 mtrs.

Importance of Bridge
Since ancient times, bridges have been the most visible testimony in human history. Some bridges embody the spirit and characteristics of a people or a place as: The
Brooklyn Bridge for New York City, the Golden Gate Bridge for San Francisco, the Tower Bridge for London, the Golden Gate Bridge for San Francisco, the Harbor Bridge for Sydney and the Howrah Bridge for Kolkata. Similarly, Chenab and Anji Bridges always figure prominently in the Udhampur-Srinagar-Baramulla Rail Link (USBRL) Project.

**Key considerations in selection of the type of Bridge:**
On the basis of the existing orography and geotechnical characteristics of the site, all the workshops, batching plant and so on, had to be located at Reasi side, little room being available on the other side.

As a consequence of these points, a solution avoiding any cut (or able to minimize the cuts) on the Katra slope side was preferred. Further, keeping in view the schedule of completion of project, the work of Tunnel T2 on Katra side, it was felt to adopt a solution which permits simultaneous tunnel and Bridge construction across Anji, without much bridge construction activity on Katra side.

**Adoption of Type of Bridge**
Considering ease of construction and typical site conditions, bridge was divided into 3 parts:

- a 120 m long approach viaduct (called "ancillary") on Reasi side;
- a main bridge, crossing the deep valley;
- a central embankment, located between the main bridge and an approach (ancillary) viaduct;

In the final configuration, the embankment shall be wider than the bridge deck to have room for auxiliary equipment and assembly workshop for deck components during construction phase. It shall also be used as access road to the main Bridge, coming from the already built service road connecting Reasi to the site of Bridge. The road would continue over cable stayed Bridge along Railway line for maintenance of Bridge as well an alternate access to escape tunnel of T-2 on Katra side for use in emergencies.

Hereafter a short description of the main bridge shall be presented, both the ancillary viaduct and the embankment being of ordinary features.

**The cable-stayed bridge**
Due to the considerations previously explained, an asymmetric scheme of the bridge was compulsory.

Different solutions were compared and finally a cable stayed bridge, with only one tower placed on Reasi side, in a position where the disturbance to the exiting slope is...
reduced, has been adopted. In order to limit the excavations, the foundation of the tower shall be based on a well, which ensures to reach the sound strata of the rock without disturbing the slope.

Main Bridge shall have two steel trusses of constant height, connected by transverse girders that support a concrete slab. The choice of a composite bridge section (steel and concrete) is considered convenient: the deck of the bridge in reinforced concrete assures a high resistance to the environmental actions (wind, rain etc) thereby reducing maintenance interventions; the steel trusses guarantees light dead load combined with a high level of resistance. Besides, the global section of elements of deck is considered a box, so it has a very good torsional stiffness.

The tower shall be in concrete; the lower part (from the foundation up to the deck) is shaped as a large single leg, while the upper part is shaped as inverted Y. In the upper part the stays are anchored in to steel boxes placed inside the concrete, and connected to it: this is tested solution facilitating the positioning for anchorages and eliminates the tension stress in the concrete arising due to the horizontal components of the forces in the stays.

The stays allow an easy construction by cantilevering, without any provisional support and without heavy equipment to carry the segments of the bridge in the final position. A massive concrete abutment (MA2) on Reasi side, based over two large wells, will act as anchorage of the lateral stays and will support all the longitudinal forces transmitted by the deck, both in service (breaking forces, frictional forces) and during an earthquake.

**Merits of adoption of scheme:**
- On Katra side only a small foundation for the abutment is needed.
- All the major works are carried on Reasi side.
- No heavy equipment to build the bridge is required.
- The construction phases are clear and well defined.
- Construction time is reduced and a reliable
**Design Criteria:**
Design has been based on Indian Codes integrated by Eurocodes, where necessary.

The design speed of the line is 100 km/h, limit that does not pose problems for the train-structure interaction.

The area is classified as seismic with a PGA of 0.17g for the Service Limit State analysis, and 0.27g for the Ultimate Limit State.

Site specific Earthquake parameters studies were carried out by Department of Earthquake Engineering, Indian Institute of Technology, Roorkee, to define the seismotectonic framework for the region.

The area is classified in seismic zone v and the maximum ground acceleration corresponding to the maximum considerate earthquake (MCE) recommended by IITR is 0.34 g. PGA of 0.17g for the Service Limit State analysis, and 0.27g for the Ultimate Limit State have been adopted in the design.

Because of the high flexibility of this type of bridges, the seismic analysis was carried on with the elastic spectrum defined by the Indian Code, without any reduction, as prescribed by the Eurocode 8.

**Structural Redundancy**
The following assumptions guarantee the "robustness" of the bridge:

- Two consecutive stays missing: the bridge remains in service for transit of trains at a limited speed (30 km/h) and reduced rail traffic limitations.
- Three consecutive stays missing: no collapse of the bridge under the permanent loads;
- Explosion of 40 kg (TNT equivalent) on the deck: no collapse of the bridge under the permanent loads and possible quick repair with limited cost.

**Construction method and Control of quality:**
The steel trusses shall be prepared in a factory located far from the site, subdivided in ten metres long elements.

These elements shall be transported to the site workshop located on the central embankment.

Here the member of the deck shall be prepared by bolting the transverse beams and horizontal stiffening to form segments.

These segments shall be pushed (pulled) into the final position without stays. The sequential construction will go on by cantilevering using 10 m long assembled...
segments, and suspending the same with the stays to form deck girders. The joints between the segments shall be bolted using HSFG Bolts. The concrete slab shall be cast in situ over in 3 phases to complete the composite deck construction.

Wind tunnel tests:
Because of the long span and the deep valley, tests in the Wind Tunnel were conducted, in order to evaluate the aerodynamic actions (static aerodynamic coefficients) and to investigate about aero elastic phenomena (galloping, flutter and vortex shedding).

Rowan Williams Davies & Irwin Inc. (RWDI), which is a specialty consulting engineering firm in the field of wind Engineering was engaged for conducting the wind tunnel study of the topographical effects on the wind flows and its effects on the design of the proposed Anji Bridge.

The input data derived by RWDI after conducting wind tunnel test on a model of the valley was used to define the base wind action needed for the preliminary design of the cable stayed Bridge. Maximum wind speed at Deck level obtained by RWDI in test is 54.9 m/s, corresponding to 10 minutes mean time, with a return period of 10,000 years.

During detailed design stage, conducting a specific wind tunnel on a sectional model of Bridge was defined in the Design Basis Note (DBN).

The Milan Polytechnic (Italy) was appointed to carry on the further tests, only concerning to the adopted deck, by a sectional model, the vortex shedding for the tower being not considered relevant.

A 1:20 scale sectional model of the deck was prepared; since the chord/length ratio was assumed as 1/5, the overall dimensions of the model were: 3 m long, 0.75 m wide and 0.27 m high.

The first cycle of test showed no vibration problems related to vortex shedding, while the minimum speed for the vertical instability was about 60 m/s. Although, this critical speed is greater than the expected maximum (54.9 m/s). Because of the small width of the deck compared
to its span and the shape of the lift and moment polar curves (negative for positive angle of the wind), the Designer of the Bridge suggested to carry on a second cycle of tests. A number of non-structural elements were added to the main trusses, in order to modify the shape of the section without modifying its structural behavior; the one showing the best performance (Fig. 8) was chosen for adoption. The new polar curves were satisfactory and no instability is expected up to 100 m/s wind speed.
**Structural Health monitoring:**

Due to the importance of the bridge, a large number of sensors shall be placed on it. The monitored quantities shall be:

- Loads on foundations.
- Stress and temperature in the most representative sections of the deck and of the tower as well.
- Forces transmitted by a number of stays and bearings.
- Geometrical data like the angular rotation of the tower and the deflection of the decks.
- Possible movements of the slopes close to the bridge.
- Dynamic behavior during a possible earthquake.

All the data shall be collected inside the cable-stayed deck and from there automatically transmitted to a remote office for monitoring the structural health of the bridge.
Conclusions:
Cable stayed Bridge being adopted for the most of medium/large spans road Bridges. The new Anji Bridge assesses the great potential of the cable-stayed bridges for Railway loading. In the family of the Bridge systems, the cable supported Bridges distinguish their ability to overcome large spans. Actually, cable supported Bridges are competitive for the spans in the range of 250 m to 1500 m (and beyond.) Thanks to the great stiffness of this scheme, railway bridges as long as 500 m for ordinary speed (Oresund link between Denmark and Sweden) and up to 200 m for High. In our case, the span can be classified as "medium" but the presence of a single tower implies performances and design difficulties similar to those of a two towers bridge spanning over 350-400 m.

Finally, in this particular case the universally recognized elegance of a cable-stayed bridge will be enhanced by the location of the site to showcase a perfect dialogue between the nature and human efforts to cross beautiful and pictorious valley.

Acknowledgments:
The work for Detailed Design and Construction Supervision (DDC) of this iconic Bridge has been assigned on to the Italian Company ITALFERR (A company belonging to the Italian State Railways Group "Ferrovie dello Stato Italiane") while the subsequent tender for Proof Checking was assigned to the Company COWI UK.

The author thanks all the personnel of ITALFERR and COWI involved in the Design of this Bridge for their valuable inputs during construction stage. Special thanks to Dr. Mario Petrangeli, Principal Design Engineer for the precious contribution given to the design of the bridge.
2. Basic about HSFG Bolts:
HSFG bolts are tightened such as to induce predefined tension in the bolt shank. Due to the tension in the bolt, the interface between the plies (steel members in a joint) cannot move relatively to each other because of frictional resistance. The bolts act differently than normal bolts or rivets as explained below in Photo 1.

Here, the steel interface between plies which form a joint having HSFG bolts shall have special preparation so that sufficient slip factor is available.

Photo-1: Pictorial depiction of behaviour of HSFG Bolts
3. Definition of Slip factor (IS 4000-1992)
Slip Factor - The ratio of the shear force required to produce slip between two plies to the force (shank tension) clamping the two plies together.

4. Use of HSFG bolts:
M/s. Konkan Railway Corporation Limited (KRCL) has awarded the work of Construction of Major Bridges no. 34, 38, 39, 43, 55, 56, 57, 58, 59, 85, 87 & 88 on Udhampur- Srinagar- Baramulla (USBRL) project to M/S AFCONS Infrastructure Limited. In Bolted joints of steel superstructures of these bridges, HSFG bolts are being used.

To evaluate the slip factor of the HSFG bolted joints of the Bridges, KRCL approached Council of Scientific & Industrial Research (CSIR) -Structural Engineering Research Centre Chennai.

5. SCOPE OF WORK for CSIR
- Evaluation of slip factor of (HSFG bolt M22x110 mm).
- Test as per IS - 4000 - 1992 (Reaffirmed 2003) on 5 number of specimens.

6. SPECIMEN DETAILS
Test specimen prepared as per codal provisions. The photographic view of the test specimen is presented in Photo-2. The bolted joint consists of mild steel plates and High Strength friction grip bolts of property class 8.8 and diameter of 22mm. The surface of the specimens is blasted with shot and sprays metalized with aluminum.
The coating thickness of plate is measured using coating thickness gauge and average value measured is 205µm which meets the codal requirement (>50µm).

### 7. EXPERIMENTAL INVESTIGATIONS

The test was performed on standard specimens as per IS 4000: 1992 provisions using a servo controlled Universal Testing Machine (UTM) of capacity 1000 KN.

Five numbers of specimens are tested in presence of representative of Northern Railway, KRCL, and AFCONs and CEIL (third party inspecting agency). Photo-3 shows a view of specimen after instrumentation and Photo-4 shows the test set-up. Four numbers of Linear Variable Displacement Transducers (LVDT) with a stroke of ±5 mm are used to measure the slip between inner and outer plates. These LVDT’s are placed so as to measure the deformation between the inner plates from the bolt to position of the centre of cover plates. The measurement data are logged using computer controlled data acquisition system with a sampling rate of 2 Hz. Bolt pretension is applied using torque wrench having least count of 10 Nm and the value of torque applied is 600 Nm. Monotonic tensile loading is applied gradually up to remarkable changes in slopes are observed. To measure the induced load in bolts corresponding to an applied torque of 600 Nm, an experimental investigation has been carried out on bolts. Strain induced on the shank of bolt is measured using strain gauges while applying a torque using torque wrench. Photo 5 shows the picture of bolt fastened with strain gauges.

The average strain measured in the bolts was 2386.3 µm/m and corresponding stress is 476.7 MPa. The induced load, calculated by multiplying the cross sectional area at the location of strain gauge and stress at the point, is 168.5 KN. Further, the induced load is also computed using the empirical relation between torque (T) and bolt induced load (R) as given below:

\[ T = k_s d R = 160.42 \text{kN} \]

Where, \( k_s = 0.17 \) (according to manufacturer data) \( d \) = diameter of bolt.

<table>
<thead>
<tr>
<th>S. No.</th>
<th>( R_i ) (KN) ( \text{Measured} )</th>
<th>( \Psi_i ) (KN)</th>
<th>( \mu )</th>
<th>( \mu_i )</th>
<th>( \mu_Ri ) (KN) ( \text{Empirical} )</th>
<th>( \Psi_i ) (KN)</th>
<th>( \mu )</th>
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<td>160.42</td>
<td>174.71</td>
<td>0.55</td>
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8. RESULTS
Table-1 shows the slip factor obtained from the experiment. Figure-2 and 3 shows typical load versus deflection curves for the tested specimens. The slip factor $\mu$ was calculated using the equation, $t = k (\mu m - 1.64 s)$ (ANNEX-B of IS 4000:1992) and found that the calculated value is less than the lowest of all values of $\mu_i$. Hence, slip factor is taken as equal to the lowest value of $\mu_i$ and it is 0.45 and 0.47 corresponding to induced loads of 168.5 KN (based on experiments) and 160.5 KN (based on empirical equation), respectively.

9. CONCLUSIONS
RITES Limited, the engineering consultancy company, has designed the Bridges of M/s. KRCL and assumed the slip factor of 0.4 in the design of HSFG bolted connections. Based on the results it can be concluded that slip factor as obtained from the test results is more than the slip factor assumed in the design of joints by M/s. RITES.

ACKNOWLEDGEMENTS
Udhampur-Srinagar-Baramulla (USBRL) project team wishes to express their gratitude and sincere appreciation to Director, CSIR-SERC, Chennai, for permitting to take up this test in their laboratory. The staff of Advanced Materials Advanced Materials Laboratory and Steel Structures Laboratory of CSIR-SERC deserves special mention and thanks for their valuable cooperation and assistance rendered during experimental works.

REFERENCE
1. IS 4000:1992 (Reaffirmed 2003), High strength bolts in steel structures - code of practice, BIS, New Delhi.
2. BS111-RDSO, Guidelines for use of HSFG bolts.
3. CSIR-SERC, Chennai Report no R & D 05 - CNP 657041 - CR, May 201
GEOTECHNICAL ASSESSMENT OF THE SLOPE AND FOUNDATION CONDITIONS FOR PIERS NO. A1, P1, P2 & P3 OF BRIDGE NO. 43, KATRA-BANIHAL RAIL LINE SECTION REASI, J&K

GENERAL

The Bridge no. 43 is one of the major bridge on Katra-Banihal Rail line section located on the Northern slopes of Reasi inlier in Bakkal area across a valley crossed by a small seasonal nalla. The length of the bridge is 777.00m with fourteen piers including abutment piers. The major part of the bridge is located in gentle to moderate slopes, but the initial four piers i.e. A1, P1, P2 & P3 are located on steep slope. Considering the overall safety of the bridge, besides initial investigations, additional investigations was carried out by detailed geological mapping of the area and sub surface exploration by drilling at the pier locations and across the piers to understand some additional information about the rock mass conditions for safety of the bridge. Triple tube method of drilling was adopted to bring out the better results. Besides shallow pits were also dug, which have came out with excellent information about the nature of slope debris and depth to bed rock. The area in general is dry with no perennial nalla flowing between the piers.

To assess the stability of the slope detailed geological mapping of the area was carried out covering an area of about 50 m on either side of the central line and the area before and after the piers no. A1, P1, P2 & P3 on 1:500 scale. Based on the surface geological studies together with sub surface exploration by drilling bore holes across the pier locations both upstream and downstream and at the centre of the pier location, geological sections were prepared to depict the surface and sub surface geological condition and to understand behaviour of the bedrock profile below the slope debris material.

GEOLOGY OF THE AREA

The surface geological studies by detailed geological mapping have revealed that both up slopes and down slopes area from the central line is occupied by slope wash/slope debris material represented by both large size and small size angular blocks of cherty dolomite/quartzite and sub rounded pebbles of quartzite embedded in soil.

The area also exposes cherty dolomite/quartzite forming isolated blocks and ridges generally along the slopes (Plate-1, Photo-1). Since rock mass is having varied lithology represented by hard and soft rock mass where irregular weathering of the rock mass have been also recorded in some locations showing prominent lithological bands (Photo 2&3). Some shale bands belonging to Kharikot formation forming the top most horizon of Sirban limestone Group were also recorded, besides small out crops of shale along the footpath at the toe of the hill slopes have also been recorded in the area (Photo -4).

The general trend of the rock mass is N200W-S200E and dip varying from 250 to 300 north easterly. The cherty dolomite/quartzite exposed in the up slope forming projected rock mass are generally thickly bedded. These rock units are traversed by number of joint sets where bedding joint is the prominent joint set (Plate-1).
GEOTECHNICAL APPRAISAL

The area both up slope and down slope of this part of the alignment forms moderate to steep slope with some steeps scarps at places both up slope and down slope at central line, besides some overhangs at places (Photo-4&5). The alignment runs almost parallel to the strike direction of the rock mass or making and acute angle with the alignment. Due to folded nature of the rock mass minor variations in the strike as well as dip angle of the rock mass have been recorded in the area.

As mentioned earlier the major part of the area is covered with slope debris material. The thickness of the overburden is not uniform as inferred from the outcrops at places, drill holes and pits. The area around A1, P1 & P2 with comparatively steep also indicate shallow depth to the bed rock as semi-consolidated and un-consolidated material with more thickness cannot stand along steep spurs. Moderate to steeply sloping spur below the abutment pier location A1 also confirms shallow depth of the bedrock, and indicates no immediate threat to the slope below abutment pier A1. The general slope angle along the stretch of the alignment between A1, P2 & P3 varies from 300 to 450 but areas with steep slopes above 600 and 700 have also been recorded in the form of projected rock mass and over hangs particularly up slope of the alignment. The area across pierno. P-3 both up slope and down slope in general is having moderate slope with thick cover of slope debris material with small patches of re-cemented limestone scree and brecciated material which holds the slope as no any recent slope failure have been recorded (Photo-11).

The scanty rock exposures both above and below of
Photo 4: Showing overhangs of the rock mass upslope of the pier no. A-1 (abutment)

Photo 5: Showing overhangs of the rock mass up slope of the area between piers P-1 & P-2
abutment pier A-1 and up slope of the piers P-1&P-2 and their adjoining areas are traversed by at least three joint sets. The cross cut relationship of the joint plans may result into small wedges in case of thinly bedded sequence and large wedges in case of thickly bedded sequence. The water conditions in the area is generally dry. No rock failure is anticipated in the rock mass except in the overhang portion where the rock mass have become blocky in nature due to openness and cross cut relationship of the joint planes and comparatively steeper slopes. The Q-value calculated from the rock mass exposed in the form of overhangs and scanty exposures varies from 5 to 12.5 and the rock can be classified as fair to good. The treatment of the rock mass at the overhang area and in the upslope area is being out carried as per the requirement after assessing the rock mass conditions after excavation. Various joint sets recorded with characteristics are shown in tables 1,2&3 below.

**Joint J-1**

It is a bedding joint, which is most prominent joint set in the area. Due to folded nature of the rock mass some minor variation in the strike as well as dip of the rock mass has been recorded. The general trend of these joints vary from N200W-S200E to N600W-S600E and dip varying from 200 to 450 northeasterly. The joints in general are tight in nature, but opening from 2mm to 4mm has been recorded at places. It is a continuous joint with high persistence varying from 2m to 5m. Because of slope debris cover in large area the continuity of these joint planes cannot be exactly measured. The joints in general have smooth surface with some occasionally

<table>
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<tr>
<th>Joint Set</th>
<th>Strike</th>
<th>Dip</th>
<th>Spacing</th>
<th>Persistence</th>
<th>Nature of Joints</th>
<th>Nature of fillings</th>
<th>Condition of ground water</th>
</tr>
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<tr>
<td>J1</td>
<td>N45°W-S45°E</td>
<td>30°NE</td>
<td>0.10m to 1.50cm</td>
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<td>2mm to 4mm</td>
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<td>2mm to 4mm</td>
<td>Yellowish staining</td>
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<tr>
<td>J1</td>
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<td>45°NE</td>
<td>10cm to 25cm</td>
<td>High</td>
<td>Smooth undulation</td>
<td>2mm</td>
<td>Nil</td>
</tr>
</tbody>
</table>

<table>
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<tr>
<th>Joint Set</th>
<th>Strike</th>
<th>Dip</th>
<th>Spacing</th>
<th>Persistence</th>
<th>Nature of Joints</th>
<th>Nature of fillings</th>
<th>Condition of ground water</th>
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<tr>
<td>J2</td>
<td>N40°W-S40°E</td>
<td>50°SW</td>
<td>10m to 30m</td>
<td>High</td>
<td>Slightly rough, smooth</td>
<td>1-2mm</td>
<td>Yellowish staining</td>
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<tr>
<td>J2</td>
<td>N70°W-S70°E</td>
<td>70°SW</td>
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<td>High</td>
<td>Slightly rough, undulating</td>
<td>1-2mm</td>
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<tr>
<td>J2</td>
<td>N45°W-S45°E</td>
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<td>N35°W-S35°E</td>
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<td>Tight (&lt; 0.5mm)</td>
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<table>
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<th>Dip</th>
<th>Spacing</th>
<th>Persistence</th>
<th>Nature of Joints</th>
<th>Nature of fillings</th>
<th>Condition of ground water</th>
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<td>J3</td>
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<td>Medium</td>
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<td>2mm to 4mm</td>
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</table>
undulating surfaces. The spacing of these joint planes is varying from 5cm to 20cm in thinly bedded sequence to above 1.5m in thickly bedded sequences. Minor clay filling and calcite coating along some of the joint planes have also been recorded.

**Joint J-2**

It is another major joint set cutting almost across the bedding plane of the rock mass and dipping almost in the opposite direction of the J-1 i.e. bedding joint. The general strike of the joint plane varies N350E-S350W to
N700E-S700W and dip 400 to 700 southwesterly. The joint in general is consistence in orientation, but minor swing in the orientation along with bedding plane is recorded in the area which is attributed to the folded nature of the rock mass in the area. The water conditions are generally dry. The joint planes are having slightly rough to undulating surface and generally tight in nature. It has medium to low persistence with spacing ranging from 10cm to over 30cm. Calcite coating and clay filling have also been recorded at places. These joint planes in general are both persistence and non persistence in nature.

**Joint J-3**

It is another prominent joint set in the area with medium persistence. Due to thin cover of the slope debris in surrounding area of the rock outcrops its, complete persistence cannot be recorded. It is generally open in nature with perfuse calcite coating along the joint plane. The surface of the joint plane is rough to undulating. The opening of joint planes varies from 2mm to 50mm. The water conditions in general is dry. Some clayey material due to rain water entering the joints has also been recorded.

**SUBSURFACE EXPLORATION BORE HOLES:**

Sixteen bore holes were drilled across the newly proposed pier locations with four bore holes at each proposed pier locations to establish the subsurface rock condition across the slope. The depth of the bore holes varies from 35m to 55m as per the requirement.

- **Bore hole no. A1/CL** drilled at A1 pier location with RL 861.38m have touched the bed rock at 4.90m below slope debris material represented by angular, sub-angular blocks fragments of quartzite and cherty dolomite embedded in soil (Plate - IV). The material is semi-consolidated in nature. The rock mass below 4.90m is represented by grayish whitish, fractured sheared cherty dolomite/quartzite, with some highly sheared and crushed bands of cherty dolomite. The rock mass in general is slightly weathered with some occasional highly weathered zones. Some intermittent solid bands of rock mass have also been recorded. The core recovery percentage in the rock mass varies from 75% to 100%. The RQD percentage in the rock mass is almost nil in the major drilled depth, except at certain depths where it varies from 20% to 68%. The less percentage of core recovery is due to highly fractured and sheared nature of the rock mass. Since the rock mass is available at reasonable depth the foundation can be kept on rock mass after ascertaining its engineering properties and suitable foundation treatments like grouting and anchoring where ever required.

- **The bore hole no. A1/D-1** with RL 841.230m drilled about 20m (horizontal distance) down slope of the pier location has touched bedrock at 2.50m below thin cover of slope debris. The rock mass is represented by grayish whitish quartzite and cherty dolomite. The core recovery percentage varies from 80% to 100%. The average core recovery percentage is above 85%. The RQD percentage in the bedrock is 20% to 90%. At certain depths due to thinly bedded and fractured nature of the rock mass the RQD is nil. The water loss in the rock mass is partial from 2.50m to 6.95m and 14.00 to 20.80m whereas there is complete water loss between 6.95 to 14.00m and 20.80 to 35.00m. The water loss is due to the openness of the joints. The rock mass is quite hard and competent.

- **The bore hole A1/D-2** with RL 816.030m drilled about 45m (horizontal distance) down slope of the pier no. A1 have touched the bedrock at 5m below the slope debris. The rock mass is represented by cherty dolomite and quartzite. The core recovery percentage varies from 80% to 100%. The RQD percentage varies from 10%-66%. Because of some thinly bedded and fractured bands of the rock mass, the RQD percentage at certain depth is nil. There is complete water loss in the rock mass from 5.00m to 6.20m and 20.80 to 35.00m, whereas there is partial water loss from 6.20m to 20.80m. The water loss is inferred to be due to openings of the joint planes. The rock in general is hard and competent.

- **The bore hole no. A1/U1** with RL 870.300m drilled 20m upslope (horizontal distance) of the pier no. A1 have established the bedrock at 4.50m below slope debris material. The slope debris material is represented by angular, sub angular fragments and blocks of quartzite and cherty dolomite embedded in soil. The material is semi consolidated in nature. The bed rock is represented by both fresh to slightly
weathered well bedded fine grained cherty dolomite/quartzite and highly broken fractured dolomite with some solid bands. Besides some thin shear zones have also been recorded. The core is broken due to highly fractured and jointed nature of the rock mass. Staining and rough surface along joint planes have also been recorded. The core recovery percentage in rock mass in general is above 85% to 100% whereas in some small section it varies from 64% to 75%. The RQD percentage varies from 19% to 72% in some section whereas it is nil in the major section of bore hole. The less percentage of RQD is due to highly jointed and fracture nature of the rock mass. Staining and rough surface along joint planes have also been recorded. The core recovery percentage in general is above 85% to 100% whereas in some small section it varies from 64% to 75%. The RQD percentage varies from 19% to 72% in some section whereas it is nil in the major section of bore hole. The less percentage of RQD is due to highly jointed and fracture nature of the rock mass. Staining and rough surface along joint planes have also been recorded. The core recovery percentage in general is above 85% to 100% whereas in some small section it varies from 64% to 75%. The RQD percentage varies from 19% to 72% in some section whereas it is nil in the major section of bore hole. The less percentage of RQD is due to highly jointed and fracture nature of the rock mass. Staining and rough surface along joint planes have also been recorded.

The bore hole no. P1/CL drilled at P1 pier location with RL 849.228m have touched the bedrock at 11.90m. The slope debris material is represented by angular to sub angular, sub rounded fragments and blocks of dolomite and cherty dolomite embedded in soil (Plate-V). The material is semi consolidated to consolidate in nature. The rock mass is represented by slightly weathered well bedded light grayish moderately strong jointed cherty dolomite. Besides highly fractured fragmented and sheared dolomite with some solid bands of rock mass have been intercepted in bore hole. Also some highly pulverized sheared zones have also been recorded. Major section of the bore hole is represented by highly fractured cherty dolomite. The core recovery percentage in general above 82% except at one section where it is 76%. RQD percentage is generally nil except at certain section where it varies from 20% to 93% in solid rock mass. Water loss in the rock mass is partial from 11.19m to 31.00m whereas there is complete water loss from 31m to 35m indicating openness of joint planes. The rock mass in general is fair to good. The bore hole no. P1/D-1 with RL 828.610m drilled about 20m (horizontal distance) down slope of the pier no. P1 touched the bedrock at 10.80m below slope debris. The rock mass intercepted in the bore hole is represented by quartzite and cherty dolomite. The core recovery percentage varies from 74% to 100% in bed rock. The RQD percentage varies from 10%-100% in the bed rock. At certain depths in the
bed rock the RQD percentage is nil due to thinly bedded and fractured nature of the rock mass. The water loss in the rock mass is partial from 10.80m to 12.70m whereas there is complete water loss from 12.70m to 35.00m. The water loss is attributed to the openness of the joint planes. The overall rock condition is fair to good.

- The bore hole P1/D-2 with RL 810.560m drilled about 40m (horizontal distance) down slope of the pier no. 1 intercepted the bedrock at 1.50m below slope debris. The rock mass represented by well bedded moderately jointed fine grained massive dolomite with some fractured, thinly bedded partially weathered zone. The core recovery percentage varies from 78% to 100% in bed rock, but at certain depth due to sheared and weathered nature of the rock mass the recovery percentage is less. The RQD percentage varies from 20%-78% in the bed rock. At certain depths in the bed rock the RQD percentage is nil due to thinly bedded and fractured nature of the rock mass. The water loss in the bed rock from 1.50m to 7.60m is partial whereas there is complete water loss from 7.60m to 35.00m, which is due to openness of the joint planes. The overall rock condition is fair to good.

- The bore hole P1/U1 with RL 871.40m drilled 20m (horizontal distance) up slope of the P1 pier location have touched the bedrock at 5.15m below slope debris. The rock mass represented by fresh to slightly weathered grayish fine grained moderately strong cherty dolomite with highly sheared and fractured cherty dolomite with some shear zones and bands of solid rocks represented by cherty dolomite. The core recovery percentage in the rock mass varies from 80% to 100% whereas average core recovery percentage is above 90%. RQD percentage varies from 10% to 92 % in certain section in solid and less fractured rock mass and nil in the highly fracture zone. The water loss is partial from 5.15m to 22.65m whereas it is complete from 22.65m to 35m. The overall rock condition is good.

- The bore hole no. P2/CL drilled at P2 pier location

![Photo 10: Showing galena, the ore of lead (small sample) and reddish yellowish gossan after leaching of the lead in the slope cutting upslope of pier p-3](image1)

![Photo 11: Showing re-cemented scree of quartzite and cherty dolomite in the cutting up slope of pier P-3](image2)
BRIDGES

with RL 838.110m have touched the bedrock at 13.70m below slope debris material represented by angular to sub angular boulders blocks and fragments embedded in soil (Plate-VI). The material is semi consolidated in nature. The bed rock is represented by highly sheared pulverized rock mass from 13.70m to 15.30m and fresh well bedded, dark grayish strong brittle cherty dolomite with highly fractured broken grayish whitish cherty dolomite. Thin bands of solid rock mass particularly towards the bottom side have also been intercepted in the bore hole. The core recovery percentage in the rock mass varies from 75% to 100%. The RQD percentage recorded in some sections varies from 10% to 70% in solid rock. The water loss in the entire drill depth is partial indicating slightly tight nature of the joints. The overall rock condition is good.

The bore hole P2/D-1 with RL 824.410m drilled 20m (horizontal distance) down slope of the pier no. P2 touched the bedrock at 10.80m below slope debris. The bed rock is represented by cherty dolomite and quartzite. The percentage of the core recovery in the rock mass varies from 74% to 100%. The RQD percentage varies from 10% to 100% in the bed rock. At certain depths as in case of other bore holes due to thinly bedded and fractured nature of the rock mass the RQD percentage is nil. The Overall rock condition is fair to good.

The bore hole P2/D-2 with RL 808.980 drilled about 40m (horizontal distance) down slope of the pier intercepted bedrock at 3.50m below slope debris. The rock mass is represented by well bedded moderately jointed fine grained massive dolomite with some fractured and thinly bedded sequence at certain depths. The core recovery percentage varies from 43% to 100%. The RQD percentage in the bed rock varies from 53% to 80% and nil in the thinly bedded and fractured rock mass. The water loss in the rock mass is partial from 3.50m to 22.40m whereas there is complete water loss from 22.40m to 35.00m. The water loss is due to the openness of the joints. The overall rock condition is fair to good.

The bore hole no. P2/U1 drilled 20m (horizontal distance) upslope of the P2 pier location with RL 854.89 m have touched the bedrock at 14.70 m below semi consolidated slope debris material represented by angular, sub angular blocks, fragments of gravel size cherty dolomite, quartzite embedded in soil with some blocks of re-cemented brecciated material with clasts of cherty dolomite and quartzite. The material is in semi consolidated in nature, some bands of brecciated dolomite have also been intercepted. The rock mass is represented by hard, thinly to well bedded grey fine grained cherty dolomite moderately broken and highly sheared cherty dolomite. Some micro voids along the joint planes formed by solution of the rock mass have also been recorded. Besides some shear zones with crushed pulverized rock mass have also been intercepted. The rock mass intercepted in the bore hole in general is sheared and broken with some solid bands and less fractured rock towards bottom. The core recovery percentage varies from 58% to 100%. RQD percentage in general is nil, but in some sections of moderately broken and solid rock mass it varies from 17% to 45%. The water loss in the rock mass is partial from 14.70m to 22.40m whereas there is complete water loss from 22.40m to 35.00m. The water loss is due to the openness of the joints. The overall rock condition is fair to good.

The bore hole no. P3/CL with RL 853.110 m drilled at the P3 pier location has not touched the bed rock. The entire bore hole was drilled through the semi consolidated to unconsolidated slope debris material represented by fragments, gravels and boulders of quartzite, cherty dolomite and sandstone embedded in the soil (Plate-VII). Blocks of some re-cemented brecciated material with clasts of siltstone, sandstone, quartzite and cherty dolomite have also been recorded in the drill hole and on the surface adjoining to the bore holes (Photo-9). At certain depths loose sand and clay have also been recorded. The material is semi consolidated to unconsolidated in nature. The material seems to be deposited by some nala or moved along the slope in the past. The rock is very poor in nature.

Bore hole P3D-1 with RL 832.240m drilled at 20m (horizontal distance) down slope of the proposed location of the pier no. P3 touched the bedrock at 58.05m below thick cover of slope debris material.
The slope debris material is represented by boulders, pebbles, clay beds, loose sands with grains of quartzite, dolomite, moderately compact clay grayish loose micaceous sand with some re-cemented small blocks, fragments of dolomite, quartzite, siltstone and sandstone with thickness varying from 4cm to 15cm at different levels mostly in the top horizon. The total thickness of this re-cemented material is around 2.5m in 58.05m thick slope debris material above the bedrock. The recovery percentage in the bedrock varies from 80% to 90% and RQD percentage in the bedrock percentage in the bedrock up to drilled depth of 60m is nil. The overall rock condition is poor.

Bore hole P3D-2 with RL-815.570 drilled 40m (horizontal distance) down slope of the pier have touched the bed rock at 37.50m below slope debris. The slope debris material represented by boulders, pebbles of cherty dolomite, quartzite embedded in sandy matrix. Besides some loose sandy beds with some thin brecciated re-cemented scree bands were also intercepted (Photo-9). The bed rock is represented by well bedded moderately jointed grayish cherty dolomite. Highly weathered and fractured zones have also been recorded in the bedrock. The core recovery percentage varies from 58% to 100% in bed rock. The RQD percentage in bed rock is 15% to 71% in the bed rock and nil in highly fractured rock mass. The overall condition is poor to fair.

The bore hole no. P3/U1 drilled 20 m upslope of the pier no. P3 with RL 861.670 m have also not intercepted the bed rock up to the entire drilled depth of 55m. The material is represented by thin cover of black soil underlain by semi consolidated to unconsolidated slope debris material represented by fragments, blocks, boulders and gravels of cherty dolomite and some re-cemented blocks of brecciated material forming the crust of the area at places with clasts of siltstone, quartzite, dolomite and sandstone in sandy matrix. The overall rock condition is poor. The construction work for the slope stability is in progress. The geological investigation carried out have shown good results.

**SEISMICITY**

Since the area lies in Zone IV of the Seismic zonation map of India and have witnessed moderate to high intensity earthquakes in the past. It is suggested that suitable seismic co-efficient may be adopted in the design of the structure. The most important tectonic features responsible for dissipating the energy to create Earthquakes in the region are Murree thrust and Panjal thrust. The most devastating earthquakes that have occurred in the vicinity of the area are given below.

<table>
<thead>
<tr>
<th>S. NO</th>
<th>DATE</th>
<th>EPICENTER</th>
<th>MAGNITUDE ON RICHTER SCALE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>06-06-1828</td>
<td>Near Srinagar</td>
<td>6</td>
</tr>
<tr>
<td>2.</td>
<td>30-05-1925</td>
<td>Srinagar</td>
<td>7</td>
</tr>
<tr>
<td>3.</td>
<td>06-06-1928</td>
<td>Srinagar</td>
<td>6</td>
</tr>
<tr>
<td>4.</td>
<td>04-04-1905</td>
<td>Kangra (HP)</td>
<td>8.5</td>
</tr>
<tr>
<td>5.</td>
<td>27-06-1945</td>
<td>Kashmir</td>
<td>6.5</td>
</tr>
<tr>
<td>6.</td>
<td>26-06-1963</td>
<td>Kashmi</td>
<td>6</td>
</tr>
<tr>
<td>7.</td>
<td>24-08-1980</td>
<td>Kathua/Basholi</td>
<td>5.5</td>
</tr>
<tr>
<td>8.</td>
<td>19-01-1975</td>
<td>Kanour (HP)</td>
<td>6.5</td>
</tr>
<tr>
<td>9.</td>
<td>08-10-2005</td>
<td>Kashmir</td>
<td>7.4</td>
</tr>
</tbody>
</table>
CONSOLIDATION GROUTING OF STRATA UNDERNEATH ARCH FOUNDATION OF CHENAB BRIDGE NO. 44: A CASE STUDY OF ARCH BRIDGE FOUNDATION S-40 LOCATION

1 INTRODUCTION
Consolidation Grouting is a method of pressure grouting of rock strata to reduce the deformability of jointed or shattered rock. This paper describes the method of conducting the consolidation grouting of the rock strata at the location of arch foundation S40 of Chenab Bridge No. 44. The arch foundation at S40 consists of two rafts (one each at upstream and downstream) each of dimensions 16.5m x 35m which are interconnected at the substructure level by shear beam.

The area subjected to consolidation grouting is 43m x 58m which includes these two rafts. Pressure grouting is done through a grid of drill holes of diameter 76mm and depth 54m provided over the plan area 43m x 58m. These holes are differentiated as peripheral holes, primary holes and secondary holes based on their location and spacing in the grid of drilled holes over the plan area of 43m x 58m and sequence of drilling and grouting. Two exploratory bore holes are drilled at each of these raft locations at the upstream and downstream before the grouting and after the grouting and bore logs studied.

The effect of consolidation grouting is studied on the core recovery, RQD, RMR. Rock samples are extracted from the pregrout and postgrout bore log cores and studied for the specific gravity, density, modulus of elasticity, unconfined compressive strength. The efficacy of the consolidation grouting is determined by conducting water permeability test on 8 numbers of test holes.

2. GEOLOGY
The area along the left bank slope from the Chenab river bed level both thinly and thickly bedded sequence are exposed, which is overlain by thinly bedded reddish dolomite bands followed upslope by grayish whitish thickly bedded cherty dolomite with prominent compositional bends having variable compositional strength and thickness belonging to Sirban group. The general trend of the rock mass varies from (J-1)N 200 W-S 200 E to N450W to S450E with dip varying from 150 to 450 north easterly and with two joint sets. Due to folding in the rock mass change in the strike and angle of dip have been recorded at the places. No folding or faulting on regional scale has been recorded in the rock mass except some local folds and faults with very limited persistence. Except some localized areas no serious adverse feature exactly along the center line of the bridge has been recorded in the area till date. Although some bedding shear planes with thickness of 5 cm to 25 cm have been recorded with persistence of around 5m. In
order to strengthen the rock mass at foundation of the piers and along the bank slopes, as a long term measures various remedial were adopted, which include removal of loose and disturbed blocks along the slopes, cutting and benching, rock bolting, shotcreting. Besides catchment area drainage to divert the surface runoff from the surrounding area entering into the slope along the center line. These remedial measures have shown encouraging results.

3. CONSOLIDATION GROUTING
The objective of doing consolidation grouting is to fill discontinuities, cavities or voids in rock mass by pressure grouting using neat cement grout and also to reduce deformability of jointed rock mass below foundation location S40 at Chenab bridge project.

The need for grouting is determined by conducting water permeability test. Lugeon value in the water permeability test is determined using the formula as given below:

\[ L = \left( \frac{q}{I} \right) \times \left( \frac{P_0}{P} \right) \]

Where,
- \( q \) is water inflow,
- \( I \) is the length of test section,
- \( P_0 \) is the constant pressure (1MPa),
- \( P \) is pressure at collar in MPa.
- \( q \) is calculated from the average of water inflow in last 10 minutes.
- \( P \) is the sum of reading shown in pressure gauge and the pressure along the length from water swivel to the bottom of test section divided by 10.

If water absorption exceeds 3 lugeon then pressure grouting is proposed. Clause 8.3 of IS 5529 Part2 2006 defines Lugeon as the water loss of 1 litre/min/m of the drill hole under a pressure of 10 atmospheres maintained for 10 min in a drill hole of 46 mm to 76 mm diameter.

Pressure grouting will be carried out in the rock mass below the founding level, upto a minimum depth equal to 1.5 times the width of foundation. At the current foundation location S40 this comes out to be 54m.

4. CONSTRUCTION MATERIALS


5. CONSTRUCTION EQUIPMENTS REQUIRED
Following are the construction equipments used:
- (a) Drilling Rig
- (b) Drilling accessories
- (c) Air Compressor
- (d) Grout mixing machine
- (e) Grout injector/grout pump
- (f) Packer pipes
- (g) Single/double packer sets
- (h) Water meter
- (i) Pressure gauge
- (j) Water pump
- (k) Water swivel head
- (l) Other miscellaneous accessories

6. SEQUENCE OF ACTIVITIES
Drilling and grouting for stabilization of rock mass below foundation location S 40 is carried out simultaneously by maintaining distance between such holes.

Pressure grouting is done from river end towards hill end. Grouting is done for alternate holes.

The sequence of drilling, grouting is as given below:
- (a) Drilling and grouting the peripheral holes.
- (b) Drilling and grouting of primary holes.
- (c) Drilling and grouting of secondary holes if required.

The step by step construction procedure for pressure grouting for stabilization of rock mass below foundation was done as per following.
- (a) Drilling and grouting of peripheral hole:
  - (i) Marking of Layout: Peripheral holes at a spacing of 2m c/c along the length and width of foundation are marked over the PCC as shown in Figure 1. Numbering of each hole is done as per convenience for reference.
  - (ii) Drilling of Hole: Staged drilling of grout holes is carried out upto required depth below founding level at all the locations. Packer permeability test is carried out in some of the drilled holes using single packer method.

100mm dia. drill hole is used for carrying out drilling and grouting in initial 1 - 1.5m depth, where 90mm dia. casing pipe is installed. Further, drilling at lower depths is carried out using 76 mm dia. bits.

Stage grouting is carried out for treatment of various...
zones individually, by grouting successively increasing depths after sealing the upper zones i.e. in descending stage. The depth of each drilled section is 3 to 5m.

(iii) **Washing of Hole:** To remove the material deposited on the surface during drilling operation and also to remove erodible material by circulating water till reasonably clean water comes out. The quantity of water flowing into the hole during the period should be adequate and generally not less than 15 l/min. When no return of washing water is observed then the hole should be washed for a reasonable period based on site experience.

(iv) **Water permeability test:** On completion of washing of hole, water permeability test will be conducted. If excessive water loss is found and Leugon value is more than 3, the hole will be grouted by using neat cement grout.

(v) **Grouting:** Grout mix is prepared by using OPC 43/53 Grade cement and water cement ratio of 10:1 to 1:1. Mixing of grout is done by using high speed grout mixer machine. Initially grouting starts with mix of water cement ratio 10:1 and gradually decrease the water cement ratio up to 1:1. Grouting normally starts with a thin mixture which is gradually thickened until about 75% of final desired pressure has been obtained with the pumps operating at normal speed. Grout mix is thickened if there is no increase in pressure after continuous grouting for about 10 min. The grout is injected by connecting grout mixer to the grout pump fitted with pressure gauge, water meter and grout pump to packer pipe by hose pipe. Single packer is fitted on packer pipe and inserted into the desired depth of drilled hole.

(vi) **Grouting Pressure:** Grouting starts with calculated pressure 0.1 to 0.25 kg/sq.cm/m of overburden and the pressure is build up to limiting pressure. Initially the rate of intake of grout may be 20 l/min to 30 l/min and pressure is raised when intake falls below 5 l/min. When surface leaks develop pressure is released immediately. Pressure is controlled using the pressure gauge of least count of 0.1 to 0.2 kg/sq cm if available. The applied pressure is rounded off the nearest value of true pressure considering the least count of pressure gauge. The applied pressure is recalculated and modified as per site requirement/ grouting requirement. The grout pressure calculation is as per IS 6066 - 2004 approved by Dr. T.G. Sitharam.

(vii) **Refusal Criterion:** Grouting is considered complete when the grout intake at the desired limiting pressure is less than 2l/min averaged over a period of 10min. After grouting is completed, the grout hole is closed by means of a valve to maintain the grout pressure for a period of 1 to 2hr to prevent escape of the grout due to back pressure and flow reversal, due to causes like artesian conditions if any.

(viii) **Control of Grout Consumption:** If pressure does not build up even after grouting a thick grout with water - cement ratio less than 1:1 by weight or richer mix, the grouting operation is stopped after the consumption of pre-determined quantity (say 20 bags for 1:1 mix) of grout. The limits of consumption of grout depends on length of stage, size of cavity and open joints and fissures. The limits of grout consumption per grouting operation depends upon site condition and geologist’s decision at site. In such a case, grouting is stopped, a waiting period of 24 hours allowed to elapse and then further grouting resumed.

Grouting in such hole which had to be stopped because the excess of grout consumption, the grouting of same hole is resumed after 24 hours after completing drilling grouting of next holes near vicinity.

(ix) **Extension of hole for further depth:** Grout attains the initial setting (3 to 4hr), drill the grouted portion of the hole and clean the hole as above and conducting the water permeability test and if Leugon value is still more than 03 then repeat the grouting operation to get Leugon value less than 03. After achieving the required Leugon value (i.e. less than 03), next stage of deeper portion of the hole is drilled and repeat the procedure as above till the final depth is attained.

(x) **Precaution while Grouting:** Grout flow should be continuous at desired pressure and grouting equipment should run efficiently throughout the grouting operation. The Site Engineer for grouting should respond quickly and effectively to manipulate the desired changes in the grout mix consistency, rate and pressure of injections etc. as directed by Site in-charge during grouting operation.

Grouting should be stopped whenever pressure gauge noticed sudden drop of pressure or rate of grout intake increases abruptly or there is any indication of upheaval, disturbance or leakage.

(xi) **Efficacy of grouting operation:** The efficacy of grouting operation is estimated using pre-grouting and post-grouting water permeability tests.
(b) **Drilling and grouting of primary hole:** After completion of peripheral holes, primary holes are drilled at a spacing of 3m c/c along the length and width of foundation is marked over PCC as shown in Figure 1. Numbering of each hole is done as per convenience for reference. Drilling and grouting is done in similar manner as above.

(c) **Drilling and grouting of secondary hole:** After completion of drilling and grouting of primary holes, secondary test holes are drilled in between the primary grouted hole at distance of 3m c/c. Water permeability tests are conducted and if values are less than 0.3 lugeon then drilling and grouting of secondary grout holes are not be required. Otherwise, secondary grout holes are to be drilled and grouted in the similar manner as above. For drilling and grouting of secondary hole, the location of hole is in-between the primary hole. The records for the rock/soil drilling, grouting and water permeability test is maintained as per IS 6066:1994 (Reaffirmed 2004).

7. **WATER PERMEABILITY TEST**

Water permeability test is conducted as per IS 5529 (Part 2): 2006 by single packer method. Figure 3 shows the set up for conducting the water permeability test by single packer method.

8. **RESULTS**

- The quantity of cement grout consumed in consolidation grouting in peripheral, primary, secondary holes is shown in Table-1 below:
- To determine the efficacy of consolidation grouting 8 number of test holes were drilled of diameter 76mm and depth 54m after the completion of consolidation grouting. The lugeon value in each stage at all test holes was found to be in the range 0 - 3. A sample calculation of determining the lugeon
value for a stage of test hole no. T2 is shown in Table-2 below:

- To study the rock strata before and after the consolidation grouting, 2 bore holes were drilled before and 2 after the consolidation grouting at the location of the upstream and downstream rafts. Core logging was done in these bore holes and the data studied. Table-3 given below shows the qualitative and quantitative assessment of effects of consolidation grouting on core recovery, RQD, RMR value over the depth of bore hole. For pregrout bore hole 1 at the downstream, during drilling complete water loss was noticed at depths from 20-24m, 51-55m and partial water loss noticed at depths from 0-20m, 24m-51m whereas for pregrout bore hole 2 at the downstream, during drilling no water loss noticed from depths 0-16.5m, complete water loss noticed from depths 16.5m-55m. For pregrout bore hole 1 at the upstream during drilling no water loss noticed from depths 0-6m and complete water loss noticed from 6m-55m whereas for pregrout bore hole 2 at the upstream, during drilling water loss noticed from 0-55m. For post grout bore holes at the upstream and downstream during drilling no water loss observed and permeability of less than 3 lugeons found.

- Rock samples were extracted from core log data of all the bore holes and sent to Test Laboratory Ang Ron Geotechpvt. Ltd. The rock samples analysed for dry density, specific gravity, crushing strength and modulus of elasticity and their range of values at the upstream and downstream are given below in table 4 given below:

### TABLE-1: CONSUMPTION OF CEMENT IN DIFFERENT TYPES OF HOLES IN CONSOLIDATION GROUTING AT S40:

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Type of Hole</th>
<th>Number of Holes</th>
<th>Number of Cement Bags consumed in Consolidation Grouting</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Upstream</td>
</tr>
<tr>
<td>1</td>
<td>Peripheral</td>
<td>100</td>
<td>11769.4</td>
</tr>
<tr>
<td>2</td>
<td>Primary</td>
<td>221</td>
<td>23020.5</td>
</tr>
<tr>
<td>3</td>
<td>Secondary</td>
<td>252</td>
<td>23633.0</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>573</td>
<td>58422.9</td>
</tr>
</tbody>
</table>

### TABLE-2: WATER PENEABILITY TEST RESULT:

<table>
<thead>
<tr>
<th>Location</th>
<th>S40</th>
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<tbody>
<tr>
<td>Drill Hole No.</td>
<td>T2 Test Hole</td>
</tr>
<tr>
<td>Diameter of Hole</td>
<td>76mm</td>
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<tr>
<td>Test Section</td>
<td>From 25m to 30m</td>
</tr>
<tr>
<td>Distance between swivel and bottom of test section</td>
<td>1.68+30</td>
</tr>
<tr>
<td>Height of test zone of Rock mass</td>
<td>30m</td>
</tr>
<tr>
<td>Stage of test</td>
<td>Post Grout</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Pressure Gauge reading in Kg/sq.cm</th>
<th>Water Intake in Ltr/min</th>
<th>Water Pressure at collar in MPa</th>
<th>Water Intake in Ltr/min/mtr</th>
<th>Lugeon Value</th>
<th>Type of Flow</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>0.4</td>
<td>0.717</td>
<td>0.1</td>
<td>0.1</td>
<td>Laminar</td>
<td>Desired</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>0.4</td>
<td>0.917</td>
<td>0.1</td>
<td>0.1</td>
<td>Lugeon</td>
<td>Achieved</td>
</tr>
<tr>
<td>3</td>
<td>8.2</td>
<td>0.8</td>
<td>1.137</td>
<td>0.2</td>
<td>0.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>0.2</td>
<td>0.917</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>0.2</td>
<td>0.717</td>
<td>0.0</td>
<td>0.1</td>
<td></td>
<td>Further drilling</td>
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</tbody>
</table>
### TABLE-3: ANALYSIS OF DRILLED BORE HOLE CORE LOGGING DATA:

<table>
<thead>
<tr>
<th>BORE HOLE</th>
<th>CORE RECOVERY(%)</th>
<th>ROD(%)</th>
<th>RMR VALUE</th>
<th>CUMULATIVE LENGTH OVER WHICH RMR OBSERVED (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Downstream</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bore hole 1 (Pre grout)</td>
<td>40-100</td>
<td>0.0-62.0</td>
<td>55</td>
<td>3.0</td>
</tr>
<tr>
<td>Bore hole 2 (Pre grout)</td>
<td>19-100</td>
<td>0.0-52.0</td>
<td>47-50</td>
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</tr>
<tr>
<td>Bore hole 1 (Post grout)</td>
<td>58-100</td>
<td>0.0-83.0</td>
<td>55-62</td>
<td>6.0</td>
</tr>
<tr>
<td>Bore hole 2 (Post grout)</td>
<td>76-100</td>
<td>0.0-98.0</td>
<td>50-59</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>Upstream</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bore hole 1 (Pre grout)</td>
<td>24-100</td>
<td>0.0-87.0</td>
<td>45-55</td>
<td>5.0</td>
</tr>
<tr>
<td>Bore hole 2 (Pre grout)</td>
<td>27-100</td>
<td>0.0-89.0</td>
<td>56-59</td>
<td>3.0</td>
</tr>
<tr>
<td>Bore hole 1 (Post grout)</td>
<td>80-100</td>
<td>0.0-83.0</td>
<td>45-53</td>
<td>7.5</td>
</tr>
<tr>
<td>Bore hole 2 (Post grout)</td>
<td>92-100</td>
<td>0.0-78.0</td>
<td>48-58</td>
<td>7.5</td>
</tr>
</tbody>
</table>

### TABLE-4: ANALYSIS OF ROCK SAMPLES EXTRACTED FROM DRILLED BORE HOLE CORE LOGS:

<table>
<thead>
<tr>
<th>ROCK SAMPLES</th>
<th>SPECIFIC GRAVITY</th>
<th>DENSITY (gm/cm³)</th>
<th>Modulus of Elasticity (Kg/cm²) x10⁴</th>
<th>UNCONFINED COMPRESSIVE STRENGTH (Kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Downstream</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bore hole 1 (Pre grout)</td>
<td>2.79-2.80</td>
<td>2.67-2.71</td>
<td>7.46-11.6</td>
<td>524-618</td>
</tr>
<tr>
<td>Bore hole 2 (Pre grout)</td>
<td>2.81-2.84</td>
<td>2.79-2.81</td>
<td>4.71-9.91</td>
<td>425-850</td>
</tr>
<tr>
<td>Bore hole 1 (Post grout)</td>
<td>2.77-2.85</td>
<td>2.75-2.83</td>
<td>7.7-20.3</td>
<td>913-1347</td>
</tr>
<tr>
<td>Bore hole 2 (Post grout)</td>
<td>2.77-2.83</td>
<td>2.8-2.83</td>
<td>6.42-16.2</td>
<td>216-1178</td>
</tr>
<tr>
<td></td>
<td>Upstream</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bore hole 1 (Pre grout)</td>
<td>2.75-2.85</td>
<td>2.64-2.84</td>
<td>6.27-11.4</td>
<td>267-786</td>
</tr>
<tr>
<td>Bore hole 2 (Pre grout)</td>
<td>2.74-2.82</td>
<td>2.56-2.79</td>
<td>7.24-8.32</td>
<td>427-959</td>
</tr>
<tr>
<td>Bore hole 1 (Post grout)</td>
<td>2.76-2.84</td>
<td>2.67-2.80</td>
<td>8.81-12.7</td>
<td>278-948</td>
</tr>
<tr>
<td>Bore hole 2 (Post grout)</td>
<td>2.77-2.90</td>
<td>2.72-2.83</td>
<td>7.56-16.7</td>
<td>311-110</td>
</tr>
</tbody>
</table>

Fig-4: Pre grout and Post grout Holes at S40 Arch Bridge Foundation location:
9 CONCLUSIONS:

- During drilling partial and full loss of water was noticed at certain depths in the pregrout bore holes where as for the post grout bore holes no water loss was noticed which indicates the efficacy of consolidation grouting in sealing of joints.
- The analysis of pregrout and postgout bore hole core logging data at the upstream and downstream indicates that at all depths the pregrouting and postgrouting core recovery and RQD value remained more or less same. The reason could be that though the strata is highly fractured in nature, the joints were very tight as evident from the grout consumption.
- The RMR value at the pregrout rock samples at the upstream is varying from 45 to 59. The corresponding net safe bearing pressure would be 178.16 t/sqm - 273.21 t/sqm. The RMR value at the postgout rock samples at the upstream is varying from 45 to 58. The corresponding net safe bearing pressure would be 178.16 t/sqm - 266.42 t/sqm.
- The RMR value at the pregrout rock samples at the downstream is varying from 47 to 55. The corresponding net safe bearing pressure would be 191.74 t/sqm - 246.05 t/sqm. The RMR value at the postgout rock samples at the downstream is varying from 50 to 62. The corresponding net safe bearing pressure would be 212.11 t/sqm - 296.0t/sqm.

REFERENCES:

iii. Method Statement for pressure grouting for stabilization of Rock mass below foundation locations.
iv. Report on the Assessment and Validation of safe bearing capacity for foundation at S40 upstream.
v. Report on the Assessment and Validation of safe bearing capacity for foundation at S40 downstream.
1. Introduction
Piles, the long slender columns, either driven, bored or cast in situ, are a device meant for transferring the structural loads to deeper firm strata. Their use as bridge foundation has been in vogue since long, especially on the locations where the top soil used to be either loose or soft or of a swelling type of very low bearing capacity. Piles can be short (that behaves as a rigid body and rotates as a unit under lateral loads) or long (where the length beyond a particular depth loses its significance under lateral loads, but when subjected to vertical load, the frictional load on the sides of the piles shares a major part to the vertical loads), as far as their length is concerned or may be either vertical (that carry mainly vertical loads and very little lateral loads) or inclined (to take care of lateral loads, and even vertical loads when used in groups) from orientation point of view.

According to the composition of their constituent materials, piles can be classified as timber, concrete or steel piles. As far as their method of installation is concerned, piles are further classified as Driven, Cast-in-situ or Driven and cast in-situ-piles. Driven piles are also known as Displacement piles. On the basis of their mechanism of load transfer, piles are also sometimes classified as frictional, end bearing and uplift piles. Use of all the above piles is primarily confined only to carrying either vertical compressive loads or to resist uplift, horizontal or inclined loads.

A further classification based on the lateral dimension of bored cast in-situ piles do exist. If the diameter of a bored cast-in-situ pile is greater than about 0.75 m, it is called a drilled pier, drilled caisson or drilled shaft. But, if the diameter is equal to or less than 0.300 m, the pile comes under the category of Micropiles.

But, as we will see later in this essay that size is not the only criterion that differentiates a micropile from a conventional bored cast in situ pile. They differ significantly in their constitution and design approach too. Prima facie, micropiles may appear to be a subclass of piles, but in terms of utility and application, most of the piles discussed above can best be explained only as the sub-class of Micro piles. Where the role of conventional piles is confined only to transmitting the loads to a competent stratum, micropiles are utilised for underpinning and slope stability etc. too. In this technical essay, we will discuss in detail the various aspects of micropiles classification, design and their uses.

2. Definition of Micropiles
Federal Highway Administration (US department of transportation) defines micropile as a "small- diameter (typically less than 300mm), drilled and grouted non-displacement pile that that is typically reinforced.”

This definition highlights three major salient features of a Micropile. First of these three are that its lateral dimension should be equal to less than or equal to 300mm. The second one is that it should be grouted instead of being concreted in case of cast-in-situ bored concrete piles. The third one is the most important features of the micropiles and needs to be elaborated in a little detail. While most of the applied load in case of conventional cast-in-situ bored piles is resisted by the reinforced concrete and increased structural capacity is achieved by increased cross-sectional/surface area, whereas the typical steel reinforcement provided in micropiles shares the major portion of loading.

Usually the percentage of steel reinforcement in conventional piles remains very low in comparison to micropiles in which the reinforcement percentage may
go up to 50% or even more. Not only this, reinforcement used in case of micropiles, happens to be of much higher yield strength.

3. Advantages of Micropiles
Any kind of conventional piles require heavy construction equipments for execution of piling works and also the procedure involved is very cumbersome. On the other hand, micropiles are conveniently installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can be installed where the approach is restrictive and in all soil and ground condition. Moreover, they are easily provided at any angle below the horizontal using much simpler equipments that are frequently utilised for installation of ground anchors. Also, as the very small equipments are utilised for the installation of micropiles, they require very minimum working head room and are therefore most suited for working under overhead electrical installations. As their installation, causes minimal vibration and noise to adjacent structures, they are often used to underpin existing structure. Micropiles can be provided in difficult, variable, or unpredictable geologic conditions such as ground with cobbles and boulders, fills with buried utilities and various debris and in heterogeneous layers of weak as well competent formation. They can be successfully utilised in soft clays, running sands, and high water table formation where conventional piling system may not appear an appropriate alternative. Micropiles are commonly used in karstic lime stone formation. They can also be provided.

4. Micropile classification system
Micropiles follow an alpha-numeric classification system in which the alphabets denote the method of construction (grouting) adopted and numbers denote the micropile behaviour (design philosophy). Two numeric (design/behaviour) classifications, known as CASE-1 and CASE-2 respectively are in vogue. In CASE-1, micropiles are loaded directly and the applied load is mostly resisted by the micropile reinforcement. CASE-1 Micropiles always act in isolation, even if provided in groups. On the contrary,CASE-2

Micropiles reinforces the soil circumscribing it and theoretically make a reinforced soil composite that resists applied load.

Four alphabetical classification, based upon the four different methods adopted for grouting, namely Type-A, Type-B, Type-C and Type-D are available.

Details are as under:
- Type-A - Grout is placed under gravity only.
- Type-B - Grout is placed under limited pressure (0.5 to 1MP) to avoid hydro fracturing.
- Type-C - A combination of Type-A and Type-B. Carried out in two stages. In stage 1, neat cement grout is placed under gravity head as in Type-A. In stage-2, prior to the hardening of primary grout (after approximately 15 to 25 minutes), similar grout is placed one time via a sleeved grout pipe, under pressure (Min. Pressure is at least 1MP), without the use of packer.
- Type-D - Almost similar to Type-C with the difference that second stage grouting also called ‘global grouting’ is injected through a sleeved grout pipe at a higher pressure (2 to 8 MP), after the hardening of initially placed grout under gravity. A packer may be used inside the sleeved pipe in this case.

Details of Micropile Classification in a tabular form were given by Pearlman and Wolosick in 1992). Same is reproduced here for better appreciation.

Quite unaware of what the destiny has for him in her cart, he was fighting for the fascist Mussolini in the swampy terrains of Turkey for the cause of destruction during the Second World War. A native of Italy, Fernando Lizzy, got severely wounded and was put behind the bar as a war criminal. Those were the most traumatic and disappointing period of his life. Momentarily, it appeared that everything was finished for him.
With a view to provide an alternative and a reliable transportation system to Jammu & Kashmir, Northern Railway had embarked on venturesome Udhampur-Srinagar-Baramulla Rail link Project (USBRL 272 length) joining Kashmir valley with the Railways network. This is most challenging project being undertaken post independence. The project is a culmination of large number of Tunnels and Bridges Udhampur to Katra and 136 Km length from Banihal to Baramulla. Work on intervening section of Katra- Banihal (111 Km) is in progress. Katra-Banihal section includes 27 tunnels of total length 97 km and 37 major and minor bridges including iconic Chenab bridge. 50% work on Katra-Banihal section has been completed. USBRL had successfully commissioned country longest transportation tunnel of 11.2 Kmin highly rugged and mountainous terrain with most difficult Himalayan Geology. The total length of project from Udhampur to Baramulla is 272 Km, out of which work has been completed on 161 Km, which includes completion of 25 Km of length from length across mighty Pir-Panjal range.

The alignment crosses deep gorges of Chenab River about 11 Km upstream of Salal Hydro Power Dam in Reasi District of J&K, which necessitates construction of mega steel bridge. Site selection of bridge was made on important technical and geological parameters such as narrow valley at site, competent rockmass at banks, favourable Orientation of joints sets, More or less straight reach and Steady river flow without cross-currents.

The length of the bridge is 1,315 m, which consists of 467 m long arch span over Chenab river in tandem with viaduct. Such a mega bridge on the most typical geology was never constructed in country before. After detailed deliberations with eminent consultants and experts, The configuration of steel arch was selected on account of aesthetics, economy, and availability of construction materials. The solution also gives a harmonious appearance to the bridge and an effective structural stiffness. Chenab bridge will be highest Railway bridge in the world. This iconic bridge will be 30 metres higher than the iconic Eiffel Tower in Paris.

This bridge is designed to carry two tracks as per international Standards to withstand the most severe earthquakes and winds of very high speed. There are certain unparalleled features in construction of the Bridge. It is for the first time in India that concrete filled steel arch is being used in the main arch bridge. Concrete filled steel arch ribs helps in improving stability as steel arch in itself is comparatively lighter and would face stability problems against wind. Composite action between the steel arch and filled concrete entails efficient design of bridge.

In view of extremely deep canyon at bridge location, Wind velocity has assumed significance in design considerations for stability and survivability of bridge. Wind tunnel test at 266 Kmph was carried out on the model of the bridge in Denmark and requisite parameters obtained have been used in designing of the bridge.

The Chenab Bridge lies in tectonically active and geologically complex terrain. The region has experienced many earthquakes in past and recent times and also faces the danger of seismic threat. Detailed seismic
hazard analyses by considering site specific geological, seismotectonic and recorded earthquake events in and around the site were carried out by IIT Delhi, IIT Roorkee and IISc Bangalore. These data have been used in designing of super-structure and sub-structure of the bridge. Bridge is located in Zone IV, but taken in zone V for design purpose. Seismic coefficient considered were Horizontal 0.36g and Vertical 0.24g.

The Bridge caters for anti terror features, in consultation with Defence Research and development organization (DRDO), by consideration of blast load to sustain against any miscreant activities involving blasting and explosion. If any of the trestle/ pier gives away, the Deck would not collapse and the Bridge could be restored for normal operation after necessary repairs.

The temperature which falls to sub zero in winters in this area, necessitated the selection of special steel to sustain Minus 20 degree Celsius temperature. E250 grade C, E410 grade C and E410 grade CZ steel are used.

Provision of Bridge and switch expansion joint to cater for large expansions and contractions to the tune of approx. 1.0 m in the continuous girder and LWR is a remarkable feature in this bridge. It is used for the first time on bridge in the country. Internationally also, these have been sparingly used.

The bridge has designed life of 120 years. Following national and international consultants have been engaged:

**Designers:**

1. Viaduct and Foundations: M/s. WSP (Finland)
2. Arch: M/s. Leonhart, Andra and Partners (Germany)
3. Foundation Protection: Indian Institute of Science Bangalore.

**Proof Consultant:**

1. Foundation & Foundation Protection - M/s. URS, UK
2. Superstructure Viaduct & Arch - M/s. Flint & Neil, UK
3. Slope Stability Analysis (Independent Consultant) - M/s. ITASCA, USA
4. Slope Stability Analysis and seismic analysis - IIT Delhi, IIT Roorkee, IISc, Bangalore

The structural detailing of the bridge is done in the most sophisticated Tekla software. The Tekla model provides a walkthrough in the bridge enabling the fabrication engineers to understand the complex details.
Further the Tekla model generates all types of drawings starting from part list drawings to assembly drawings. Fabrication work is being carried out by installing and commissioning of extremely efficient and technically superior workshop at the site. For this purpose, Four Contemporary Workshops at bridge sites and two RSDO approved workshops have been executing fabrication works. Welding Quality is the most important factor in
construction of steel bridges. Welding processes like Submerged Arc Welding, Gas Metal Arc Welding and Flux Core Arc Welding by semi automatic and automatic modes have been adopted. Welding shall be done in accordance with the approved Welding Procedure Specifications and is performed by qualified welder possessing valid Welding Performance Certificate to achieve required mechanical properties of the weld.

Fabrication of about 16000 Mt have been completed. The Deck structure has 164 segments and is being fabricated in segments weighing 60 to 120 MT. Each segment will have a total weld length of about 3 Km. To maximize the extent of down hand welds, the segments are fabricated upside down on specially built platforms. After inspection and clearance, each segment is turned safely to upside by a very specially designed turning arrangement. Steel Fabrication workshop, Surandi at Srinagar end of site has completed pier fabrication work. Steel piers of height 56m on one of the foundations at Katra end has been installed with world longest cable crane. The ongoing works of steel pier erection at other foundations of height more than 100 m are in progress. Height of the steel pier of main arch to be provided on foundation at Katra end is of the tune of 133m, which is much taller than Qutub Minar.

Various types of Non-destructive tests as well as
destructive tests are being carried out on welded segments to assure the quality of the welds before giving clearance. Non-destructive tests such as Visual Inspection, Magnetic Particle Inspection, Ultrasonic test are being done to check the surface, sub-surface imperfections as well as imperfections throughout the thickness as per the relevant standards. Destructive testing involves Tensile test, Impact test, Bend test etc. on the production test coupons in order to check the required mechanical properties as per relevant standards. As a part of non-destructive testing of welds, most advanced phased array ultrasonic testing was adopted for the first time in steel bridge construction in country.

Phased Array Ultrasonic Fabrication work is being carried out by installing and commissioning of extremely efficient and technically superior workshop at the site. For this purpose, Four Contemporary Workshops at bridge sites and two RSDO approved workshops have been executing fabrication works. Welding Quality is the most important factor in construction of steel bridges. Welding processes like Submerged Arc Welding, Gas Metal Arch Welding and Flux Core Arc Welding by semi-automatic and automatic modes have been adopted. Welding shall be done in accordance with the approved Welding Procedure Specifications and is performed by qualified welder possessing valid Welding Performance
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The air tightness of the boxed structure is checked by Air Leak Testing which is carried out by using compressed air at 0.2 Bar pressure. Bridge is painted with exquisite RDSO approved system called polysiloxane painting system having life more than 15 years.

The bridge crosses the river on a very deep gorge with very high vertical slopes. Hence Geo-technical investigations have been given special attention. Bore holes upto 150 m deep have been drilled at the bridge site to know the nature of soil/rock available at site. Two drifts have been made for the first time in Indian Railways at the founding level of abutments having a cross section of 2 m x 2 m and a length of 40 m. These drifts have provided very useful data for determining various parameters for design of foundations. In-situ shear test, plate load test, seismic shear wave velocity test, slake durability test, P&S wave velocity test have been carried out in the drifts. These tests have confirmed the safety of the foundations and have also validated various design parameters. Slopes of the mountain supporting foundations of main arch have been stabilised by state of art technology, by involving national and international consultants. Contemporary state of the art Softwares such as SLIDE 5.0, Slope W, FLAC, UDEC and 3DEC analysis have been leveraged for assessing stability of natural slopes as well as cut slopes.

The results of these analyses have confirmed that the slopes are safe and stable. As a measure of abundant precaution, slope protection and stabilisation measures by way of shortcreting and rock bolting, Dwidag bars, and cable anchors, and grouting of foundations have been carried out. Slopes have been validated as amply stable now. Deck Girder is being designed as a continuous girder resting over the piers. The Bridge of such a marvel definitely has a number of amazing extreme engineering facts. One such amazing fact is the Launching of the curvilinear portion of viaduct on the sharp curve of 2.74 degree by pushing the segments using launching nose. This is the first time this technique has been successfully carried out in country.

The Pylons with cable cranes, as a complete system, for a swift erection and incremental launching of main Arch span has been commissioned. This cable crane having longest span in the world with 34 ton combined lifting capacity deserves special mention. The pylon height is approximately 127 m at the Kauri end and approximately 105 m at the Bakkal end. This technique of erection of structural steel by overhead cable cars is being used for the first time in the country for construction of such a large span of bridge.

The two main arch foundations at Bakkal and Kauri ends are significantly massive having heights of about 47 m and 34 m and volume of concreting works out as about 19000 cum and 14000 cum respectively. Almost all
locations of bridge have been planned to be accessible from inspection and maintenance aspects by provision of accessible inspection gallery, moveable platform and ladders.

Extensive Health Monitoring and warning System at the construction stage/during service have been planned. It includes Anemometer to ensure train regulation if wind speed exceeds 90 kmph, Accelerometer to regulate trains in case of Earthquake, Strain gauging of critical arch components, Inclinometers at critical points.

A team of world class USBRL engineers in November 2017 scripted yet another Golden chapter by successfully commencement of launching of the main arch of this Bridge. It is a noteworthy endeavour as it entails carrying heavy segments from Srinagar end of workshop with the help of the world’s longest cable crane arrangement.

The Chenab bridge will usher in new epoch in J&K state due to Increased employment opportunities for the youth, improved infrastructure due to construction of access road, Better facilities for students to travel to other parts of the country for educational purposes, Boost to tourist industry, connectivity of far flung areas to mainstream of country and overall economic development of the state. All geological challenges have been successfully negotiated and work is going on at war footing on this bridge.

Touted as an engineering marvel the 'Sky bridge' is pitted for completion by June 2019. The progress of the bridge is more than 72%.
INSTALLATION OF SPHERICAL BEARINGS IN CHENAB BRIDGE PROJECT

OVERVIEW
Chenab Railway Bridge, a bridge being constructed across the river Chenab and a vital link of USBRL Project in J&K, is gaining momentum day by day. The bridge is having a total length of 1315 meter consisting of Main Arch Span having a total Length of 467m and Viaduct Approach Spans of 848 meters on both Kouri and Bakkal Side. The Launching of Viaduct Approach from Segment AS65 to AS7 is already completed. The bridge is 359m above the river bed which makes it taller than the Eiffel Tower, a wrought iron lattice tower constructed in 1887 on the Champ de Mars in Paris, France. The Height of Eiffel Tower is 324 m to tip. The bridge alignment is partly curved with circular and transition curves while as it is straight over the balance portion. The Bridge is being supported by 11 Concrete Piers (S170 to S70), 5 Steel Piers (S20, S30, S40, S50 and S60) and 2 abutments S10 & S180. The Bridge is being supported by 22 Spherical Bearings on Concrete Piers from S180 to S 80 in approach Span i.e. from AS7 to AS65.

INTRODUCTION
Spherical bridge bearings are, composed of precision-machined steel plates with spherical concave and convex surfaces, provide flexible movements and rotations between the superstructure and supporting structures to transfer whether horizontal or vertical force safely. SPHERICAL bearings are suitable for use in structures which require the transfer of medium to high loads, and for structures whose bearings must facilitate large cumulative sliding movements - such as suspension bridges which are susceptible to wind forces. The high-strength, high-durability sliding material used at its heart allows the bearing to be designed smaller than would be possible with any other bearing type - a
feature which may be of particular interest where space is limited (Fig-1).

The paper describes the Procedure adopted for the installation of permanent bearings in the Approach Span of Chenab Bridge at the pier locations S-180 to S-80. The Spherical Bearings were designed as per the approved drawing (KR/CHENAB/2612/B/AC, Sheet No-001, REV- H) “Spherical Bearings” and Code BS-5400-Section 9.2 was used for this purpose as per the specification mentioned in the drawing:

![Spherical bridge bearing](image-url)

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<th>-</th>
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<td>170 B2</td>
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<td>±0.005</td>
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</tbody>
</table>
The maximum allowable friction for the Bearing is <10%. Spherical bearings for Chenab Bridge project are designed and manufactured by internationally reputed firm "MAGEBA" as per British standard BS5400: section 9.2:1983.

The precision installation works of these bearings were carried out under the guidance of experienced MAGEBA representative present at site. Bearing installation methodology was checked and reviewed by third parties before starting the job and approved methods as per document no. PR2612QA-09 (FAB) was followed during the process of spherical bearing installation.

**EQUIPMENT DEPLOYED**

Apart from the general erection equipments, following tools, tackles and equipments were mobilized for the job:

- 250T Hydraulic jack (02 no’s) for any single bearing location.
- Power pack of suitable capacity.
- Total station.
- Spirit level.
- Torque wrench of suitable capacity.
- Welding Equipments.

**PREPARATORY WORKS BEFORE INSTALLATION OF BEARING:**

Following stage wise works are to be executed before installation of each bearing:

- Deployment of the necessary equipments on the desired location (jacks, power pack etc).
- Fabrication and erection of necessary platform or additional structure on the Piers in order to facilitate the safe removal of temporary bearing and installation of permanent bearings (Fig-2).

**SPHERICAL BEARING INSTALLATION SEQUENCE IN APPROACH SEGMENTS**

- Designated bearing, as indicated in the drawing for any particular location was shifted to location prior to installation works (Fig-4 & Fig-5).
- The Temporary bearings which were used for the Launching operation were removed with the help of...
Hydraulic Jacks and lifted from Pedestal of Concrete Piers with the help of Wheel Mounted Monorail from top of deck plate. During Lifting of the Superstructure with the help of Hydraulic Jacks for removing the Temporary Bearing, an additional Packing stools were Placed on both sides of the Pedestal for supporting the superstructure in case Jack fails. This was being taken as Precautionary measure.

The spherical bearings were shifted to respective pier location and lowered with the help of wheel mounted monorail crane to the installation location. The superstructure was lifted on the hydraulic jack and packing stools, until the permanent bearing was installed.

Before installing the Bearing, the Sacrificial Plate was cleaned and centre axes(X-X, Y-Y) on the sacrificial plate were marked on the Plate for Proper placement of the Bearing. The Levelling of top surface of sacrificial plate proper level was checked with spirit level and total station.

For drilling of holes on the main girder flange and for fixing of spherical bearing, a special template was made with the dimension taken from the spherical bearing. This template was fixed on main girder flange to do the hole markings and then drilling was carried out after checking and ensuring the exact dimension of the holes. The Wedge Plates were also drilled and then bolted with spherical bearings.

After matching the holes of the bearing and wedge plate assembly with the main girder flange, the specified bolts were inserted and hand tightened. At Last, final torquing of the bolts was done as per given torque value. Shifting and Bolting Tightening of Spherical Bearing with Flange

Finally lowering of the main girder flange and bearing assembly was done on the sacrificial plate.

The alignment of the bearing on the sacrificial plate...
was again re-checked. After ensuring the correct alignment, the welding of the bearing and sacrificial plate as per the proper sequence given by the bearing manufacturer could be done.

- After fixing the spherical bearings with the bottom flange in horizontal position any void space between the wedges shaped plate and the bottom flange could be filled with Multi Metal grout to ensure full contact and uniform load distribution.

**Final View of the Installed Spherical Bearing**

**Features of Spherical Bearings:**
- Complying with BS EN-1337, BS 5400-section 9.1, KS4424, AASHO, ISO or other custom standards.

- Easy installation.
- Low cost maintenance.

**Applications:**
- Ideal for structures with bearings of large turning angles.
- Bridges with big torsions.
- Bridges in low temperatures lower than -30°C.
- Wide and curved bridges.

**Advantages of using Spherical Bearings:**
- Transmit the vertical loads due to permanent and randomly effects; it is possible to cover a wide range of loads about up from 500 to 100000 kN.
Fig-8: Cleaning of Sacrificial Plate
Marking - Using Template

Marking & Punching

Drilling of Bottom Flange

Drilling of Wedge Plate

Insertion of Shim plate, Wedge plate and Permanent Bearing assembly under the main Girdar Flange

Fixing of the Shim plate, Wedge plate and Permanent Bearing assembly with the main Girdar Flange with bolts
- Transmit the horizontal loads with in practise no limitation of the design load.
- Allow rotation as per a spherical hinge. The standard design rotation (±0.02 rad) can be easily increased to compensate structure slopes.
- Suitable for all type of structures like steel and concrete bridges and buildings.
- High durability and no maintenance.

**Materials Used for Manufacturing of Spherical Bearing:**
The following high-quality materials are used in the manufacture of SPHERICAL bearings:
- Steel parts of Grade S355 steel.
- Certified SLIDE sliding material with grease dimples.
- Certified silicone grease as lubricant.
- Hard chromium plating of the calotte’s surface.
- Sliding sheet of polished, certified austenitic stainless steel (grade 1.4404).
- Sliding strips of 3-layer CMI material (DUB).
- Corrosion protection according to environmental conditions and customer requirements.

**PRECAUTIONS TAKEN DURING INSTALLATION:**
- Pre setting of the bearing was not changed without consultation of the bearing expert or the manufacturer.
- During placing of bearings, the surrounding area and bearings was cleaned free from dust, dirt and other foreign contaminants.
- Bearings was placed in such a manner as that there was no gap or void between the bearing and the connecting surfaces.
<table>
<thead>
<tr>
<th>Project Information</th>
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</tr>
<tr>
<td>Structure Type (Steel/Concrete etc.)</td>
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</tr>
<tr>
<td>Client</td>
<td></td>
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<tr>
<td>Contractor</td>
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<tr>
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<tr>
<td>Responsible Person for Bearing Installation</td>
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<td>Brand Name and Type of Mortar Product</td>
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<td>Construction Method of the Mortar Bed</td>
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<tr>
<td>Bearing Type and No.</td>
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<tr>
<td>Applied Load V [kN]</td>
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<td>Pre-setting [mm]</td>
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<td>Movement range [mm]</td>
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<td>Date of Delivery</td>
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<td>Type Plate (right orientation)</td>
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<td>Working Scale (in order)</td>
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<td>Cleanliness and Corrosion Protection</td>
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<td>Cleanliness of Contact Surface to Mortar Bed</td>
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<td>Lifting of Superstructure (Date, Time)</td>
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<td>Pouring of Mortar Bed (Date, Time), Top</td>
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<td>Pre-Setting and Direction [mm]</td>
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<td>Check of hertz. inclination error x/y [mm/m] (Sect. 6)</td>
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<tr>
<td>Displacement x/y [mm] (sliding/deforming bearings)</td>
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<tr>
<td>Sliding, Guidance and Tilting Gaps (for specific types of bearing as detailed)</td>
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Date and Signature: Client
Date and Signature: Main Contractor
Date and Signature: Engineer
2. Introduction
Steel connections are widely seen across any heavy engineering construction, be they bridges, power plants or process plants. Traditionally, these have been connected together with a requirement of 100% contact for complete designed load transfer. However, achieving a 100% contact has been one of the most difficult and expensive processes in the assembly of steel elements.

In recent years, a lot of progress has been made in materials science. New polymeric materials have been developed that allow high load transfer without deformation like creep or shrinkage. These new generation formulations use very high strength fillers in a matrix that boasts of low creep, nearly zero shrinkage, ability to withstand high cyclic loads and have no functional impact / degradation from environmental factors such as salt water, UV Light, heat and rain. Another very important characteristic of these materials is the ability to use them directly on site without the requirement of advanced machinery or special tools.

One such material DIAMANT MM1018 has been used on the Arch base plates of the Chenab Bridge project in India. MM1018 is a special formulation that can withstand 160N/mm² compression loads 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 Federal Institute of Construction Technology (DiBT).

3. Problem Description
The Chenab bridge project 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 design calls for a 2 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 and moisture making corrosion protection a key requirement as well.

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. The arch base plate is embedded in the concrete pillars. Each base plate has an area of approximately 6.3m². The Arch base plates are mated with the Arch base segments and stressed using Dywidag bars that pass into...
the concrete pillars. 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 lead to a situation of structural safety and hence forms a critical requirement.

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

The arch base plates embedded in the concrete are placed at an angle in all 3 planes of axis. This posed a further challenge to meet the 100% load transfer and matched mating face requirements. The actual gap is also dependant on the position of the Dywidag bars protruding from embedded the Arch base plates.

3.2 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 milling machines that will have to be placed to machine the plates in field. This is a very expensive and time consuming process when used on completely horizontal connections. In the case of slanted 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. In the case of the Arch base plate, the Dywidag bars would also hinder the placement of the shims and a possibility of gaps at the centre of the arch base plate cannot be eliminated.

- **Lead sheets:** These are used since they take the shape of the metal plates but have a limitation of failing at higher loads and have poor creep properties.

3.3 Creep:
By definition (sometimes called cold flow) 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[1].

4. Solution using DIAMANT MM1018 for gap compensation:

4.1 Material Description:
DIAMANT MM1018 is a 2 component metal reactive resin system with high filled portions 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 Federal Institute of Construction technology since January 2013 for the "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/ [3].

MM1018 has been proven to be a fast and economical alternative to wedge plates and shim plates and other less resistant bonding materials. Currently it is the only gap filling material of its type with German Government approval.

A number of other organisations world-wide have since done independent testing and made standard guidelines for the use of MM1018 for their applications.
4.2 Material characteristics:
The material has been tested widely for its characteristics. Tests have included standard mechanical properties as well as flow, creep, cyclic fatigue and marine environment compatibility.

5. Application methodology:
There are 2 primary methods for the application of the material i.e. use of MM1018 Paste and MM1018 FL (Fluid). Due to the nature of the application, MM1018 Fluid grade was recommended for use at the Chenab Bridge project.

5.1 Process outline:
MM1018FL is the preferred choice when filling gaps formed after the installation of the mating parts. A pre-tensioned connection is also possible to be filled. Pre-tensioning of the bolts should be to 70% with final tensioning after curing of the MM1018 material. The process for application remains very simple.

- Clean surfaces of any dust, debris and loose particulates This is important to protect against a situation wherein the flow of MM1018 material is blocked at lower gaps.
- Measure the gaps and plan flow of the material - In the case of the Chenab bridge this was a major activity due to the shape and size of the Arch base plate. In the case of the C shaped Arch base plate the pumping was carried out from the lowest points against gravity with the vent points at the top. Engineer’s may also require data on environmental conditions when working in extreme climatic conditions.

- Installation of inject and vent points and flow control valve connections - This is a simple accessory that allows Technicians and Engineers to direct flow of the material across the gap as well as to monitor 100% fill across the surface.

- Sealing of all other open points - This is done using a similar material with faster curing properties called MM1018 Rapid Seal. The material is white in colour and is generally applied along the circumference to a depth of 5mm for small gaps and upto 10mm for larger gaps. The material cures within 2-3 hours @ 20°C in lower humidity conditions. In the case of the Arch base plate, the Dywidag holes were also sealed to ensure no leakage into the Dywidag bar holes.

- Mixing of the MM1018 material - Mixing of the Resin with hardener can be done with a power tool to reduce operator fatigue and to save time. The resin is pre-mixed to loosen any settled fillers and create a uniform consistency. The hardener / activator chemical is mixed prior to the application.

- Filling MM1018 material - Once mixed the material

### 4.2.1 TABLE SUMMARISING A FEW OF THE TESTS CARRIED OUT:

<table>
<thead>
<tr>
<th>S. No.</th>
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<th>Tests Carried out</th>
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<td>1</td>
<td>DiBT, (Federal Institute Construction Technology, Germany)</td>
<td>Physical and Chemical (Density, E- Properties Modulus, Hardness, Compressive Strength, Shrinkage, Viscosity, Pot Life, Creep Coefficient, Thermal Expansion Co-eff., Temperature resistance, Mixing ratio by weight) - Approval No. Z-3.82-2042</td>
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<td>2</td>
<td>American Bureau of Shipping</td>
<td>Validation Tests for use on ABS Class Vessels / facilities. Approval No.16-HG1509022-1-PDA</td>
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<tr>
<td>3</td>
<td>iBAC (Institute for Construction Technology, Aachen, Germany)</td>
<td>Pulsating Pressure load test</td>
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<tr>
<td>4</td>
<td>Eiffel Deutschland Stahltechnologie GmbH</td>
<td>Test of Flow behaviour in enclosed gap</td>
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</tbody>
</table>
is immediately injected into the gap. Due to the viscosity of the material and physical forces the material flow takes the form of a semi-circle. The viscosity of the material is 11,000 mPa·s ±15 % as per DiBT tests and can be compared with Hand creams approx. 8,000 mPa.s. The valves are used to control the flow of material to ensure a 100% fill.

- Post cure finish: The application procedure took on average approx. 1.5 days from start to finish per plate. Post in-situ natural curing, approx. 16 hours per plate after the completion of application the vent points were removed and Dywidag bars were stressed.

- Future trends and Summary Gap compensation between steel elements is a challenge across all constructions and around the world. With advanced materials and technologies that are under continuous development, the possibilities available to engineers are huge. The MM1018 system and method has seen increasing acceptance globally and is the new way of the future. The Chenab arch bridge project once again 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. This protects and improves the longevity of the connection with respect to the elements.
In recent years with increasing focus on reliability and reduced construction timelines, materials such as MM1018 and their variants shall see an increasing demand across the globe. In the case of the Arch base plates at the Chenab bridge, a couple of months were spent in machine-fitting the first joint. With the MM1018 the fitting process was reduced to 2 days. This resulted in significant time savings and as a result significant savings in cost and effort. In India, as in the rest of the world, as Rail systems are refurbished, upgraded or new infrastructure is built, the MM1018 system provides engineers, fabricators and end users an economically viable, fast, proven, globally approved, reliable and high quality solution. The use of MM1018 is seeing increasing use across Bridge bearing applications, flange connections in process plants, marine applications and more recently in quickly refurbishing older structural connections.

7. References
3. DIBt: Allgemeine bauaufsichtliche Zulassung.Zulassungsnummer Z-3.82-2042
TURNING OF DECK SEGMENTS

Working in this kind of humongous project is a dream come true for a dedicated construction engineer. I’m lucky enough to work in the prestigious world’s tallest rail arch bridge project. Coming to the subject matter as given in tag line - The Land of Innovations, which it really demands because every activity that is being taken up has its own significance either due to its designs or typical sections which are never done before anywhere (i.e.), from civil to mechanical fabrication and erection activities (composite project involving many departments/agencies).

Here I would like to bring out one among many innovations which have been practicing at the site, which is Turning of fabricated Deck segments that is developed indigenously at site by CBPU (Afcons Infrastructure Ltd.). This is fabricated upside down for ease of welding avoiding the overhead welding which saves time and increases the production capacity of the workshop. The main governing factor for turning of each segment is the Center of gravity which is pre calculated and fabricated accordingly, below Figure - 1 depicts the schematic drawing of the turning arrangement that is being used at the site.

The step by step procedure involved for rotating the segment is clearly explained below:

Stage - 1: (As shown in Figure - 2):

- Pre arrangements - remove all the locking system with the trolleys after the fabrication.
- Fix the Jaw clamping arrangement to the main girder flange of the deck and lift segment with the help of gantry crane and take to the position of rotating device.
- Lower the segment on the spreader beam arrangement and fix segment intact using bolting, weld all the required clamps with Deck plate and the Spreader beam for the wire rope fixing.
Fix all the wire ropes with the winches.
Later on take the segment towards the rotating pin and fix and lock at the C.G hole provided on the main girder which is pre determined.
Hold the gantry crane and regulate the winches clockwise and the release the gantry which helps to slightly rotate the segment.

Stage - 2: (As shown in Figure - 3):
After fixing the segment to the rotating pins the gantry is removed later on and used in rotating the segment.
Fix two wire ropes one with the gantry and other to the winches.

Initially rotate the segment by 150 with the help of winches allowing the gantry wire rope to be tightened.

Stage - 3: (As shown in Figure - 4):
Hold the Segment same in 150 with the help of winches.
Now all the procedures was in place and the load is gradually transferred to the gantry crane, now release all the wire ropes from the winches and now the complete operation is done by the wire rope from the gantry crane, but the winches are still in place for any emergency backup.
Photographs executed at Chenab Site:
Stage - 4: (As shown in Figure - 5):
- Now the rotating of segment is a continuous process by operating the gantry to loosen the wire rope and automatically the segment rotates.

Stage - 5: (As shown in Figure - 6):
- The rotating of segment is a continuous process by operating the gantry to loosen the wire rope and automatically the segment rotates but care to be taken for the wire rope is in line with the clamping arrangement fixed on bottom flange.

Stage - 6: (As shown in Figure - 7):
- The rotating of segment is a continuous process by operating the gantry to further loosen the wire rope and automatically the segment rotates and the wire rope must pass clamping arrangements.

Stage - 7: (As shown in Figure - 8):
- The rotating of segment is completed and the wire rope is removed from the gantry and the 4 slings are fixed to the arrangement provided in the spreader beam that is fixed with the segment.
- And then the segment is further shifted to the location for further process and then lowered with the help of gantry crane by removing the pins from the rotating device.
Tall Piers of Bridge no. 39 at Reasi The pier are being casted using Slip form Technique

Jhajjar Bridge no.20
Pir Panchal Tunnel

Launching of Main Arch of 467m span of World Highest Railway Bridge.