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From the Desk of Editor-in-Chief

Him Prabhat moves into its 3rd issue. The USBRL team is grateful for all the letters of encouragement and compliments they have been receiving from our readers from across the country.

In this issue we go further ahead with the endeavor. The article of Mr Sandeep Gupta, Chief Engineer /South / USBRL on “Segmental Construction of Continuous Spans of Banganga Bridge on Katra – Reasi Section” discusses the experience gained while constructing its superstructure by cantilever construction method using Norwegian Rail System (NRS) type gantries by progressive construction of a cast in situ span in segments and stitching them with the already completed segments by pre-stressing under constraints offered by the site.

Unstable slopes present one of the major challenges being faced by the engineers on USBRL project. Professor K.S.Rao of IIT Delhi, Mr Varughese, A, Senior Research Officer and Mr Rathod, G.W., Research Scholar walk the readers though numerical modeling code for advanced geotechnical analysis of soil, rock, and structural support in three dimensions by 3DEC in their paper “3D Stability Analysis of Chenab Bridge Abutments”. 3DEC simulates the response of discontinuous media (such as jointed rock) that is subject to either static or dynamic loading. Its application on possibility the most wonderful slopes of Himalayas would be exciting for engineers and geologists alike.

In June 2013, Indian Railways commissioned their 11.2 kilometer long tunnel through Pir Panjal Range. Safe operation of trains through a transportation tunnel entails synchronization of different systems. Execution of ‘Supervisory Control and Data Acquisition’ (SCADA) System to enable coordination of multiple systems at multiple locations and automation by processing the output of different systems is in continuation of an earlier article on tunnel ventilation system. Paper by Sh. R.K.Chaudhary, Chief Electrical Engineer, USBRL Project, on SCADA System for Tunnels - In Particular reference to Pir Panjal Tunnel (T-80) does justice to this aspect of USBRL project.

The New Austrian Tunneling Method (NATM) of modern tunnel design and construction which integrates the principles of the behavior of rock masses under load and monitoring the performance of underground construction during construction is introduced deftly by Sameer Singh, Executive Engineer/C, USBRL Project/ Sangaldan.

Mr Bajrang Goyal, Executive Engineer/USBRL, familiarizes the readers with portion of project area of which displays deeply incised, narrow, V-shaped valleys with steep flanks, due to the significant uplift rates of the Himalaya range within the most recent geological periods and the action of rivers in his article “Overview of Sumber – Arpinchila Section of USBRL Project”. And Mr Nem Singh Baghel, XEN / USBRL Project shares technical experiences on rehabilitation of tunnel when lateral deformation and partial collapse of the tunnel have been taken place after its construction in his article “Rehabilitation of Tunnel T1”. Mr.Vinod Kumar, Dy. Chief Engineer/ Banihal / USBRL introduces “Rockbolts and Dowels” in his article on the subject. Shailendra Kumar, XEN / USBRL Project at Banihal in his paper on “Formulation of shift of a circular curve with unequal transition length” presents the formulation of co-ordinates for the alignment having straight lines and circular curves and in quest of that presents formulae for calculation of shift of a circular curve provided with unequal transition length. Entire fabrication work of the Chenab Bridge is planned to be taken up as fabrication of segments for the steel deck structure. Mr Syed Rashid Mahmood, of KRCL explains this planning in his first part of paper “Fabrication Work for the Construction of Chenab Bridge.” Mr. Baldev Singh, SSE / Drawing / BAHL / USBRL Project brings out the importance of the drawing office in his article on the subject “Drawing Office - Heart of the Project”.

Then after rigorous engineering and technical text is the breeze of worldly wisdom on Sri Sri Paramhansa Yogananda – a compilation of whose thoughts are brought to the readers by Ms Rajwansi Koul, OS/P, USBRL / Project. Nissar, SSE/C/Srinagar/ USBRL Project gives another armchair journey through paradise on earth in his article “Kashmir Valley”. Mr Eajaz Ahmad Kawoosa, SSE/Works/Banihal/ USBRL Project introduces the Mughal Gardens in his piece on the subject. Dr Pankaj Singh, Sr DMO, Jammu, Northern Railway gives an in depth introduction to the causes, symptoms and what to do in the event of chocking. Knowledge of these may help you save a life.

Editor-in-Chief
Sh. Manohar Lal

Sh. Manohar Lal, SSE/P.Way/construction, Northern Railway/Udhampur was born on 15.03.1962, at Kathua in J&K(India). He did his diploma in Mechanical Engineering from Govt. Polytechnic, Jammu and B. tech (civil) through Distance education. He joined Railways in 19.07.1985. Initially he was posted in Allahabad division and worked as PWI Gr-III in PQRS unit and Track relaying Train in Allahabad Division. He has been with laying of PQRS Base Depots at Bharwari & Malwa stations of the Allahabad Division in Indian Railway. He has been awarded for his meritorious services and was given DRM’Award in 1990 by DRM/Allahabad. In Nov-1991, he came on transfer to Jammu for opening of 1st Block of Jammu-Bajalta section in JURL project. He was Junior Engineer/P.Way and held the responsibilities of execution and supervision of all Track linking works in Jammu-Bajalta Block section (Jammu-Udhampur Rail Link project). From 1994-July 1999, he had the responsibilities of main store of Jammu-Udhampur rail link project. Than from July 1999-July 2005, he was posted as senior section Engineer/P.way. His responsibility was estimation of P.way material requirement of Manwal-Udhampur section of Jammu-Udhampur Rail Link project including Ballast. He supervised work of Track machines deployed during opening of Jammu-Udhampur Section & preparation of LWR plans. He was given CAO’Award in 2004. From July-2005 to June-2012 he was posted as Senior section Engineer/Srinagar. He ensured quality of all P.way works of Qazigund–Baramulla Rail link project. Than he was posted as senior section engineer /P.way/Banihal. He supervised all P.way works carried out in connection with commissioning of Banihal–Qazigund section. He had worked at different places and carried out of P.way work efficiently and subsequently completed targeted works in project of National importance. He has worked with great zeal in Kashmir link despite having law & order problems and tough weather conditions. Besides all these he has additional computer knowledge and is very simple and can walk on track many kilometers in one go with no tiredness on his face.

Sh. Saroop Lal, driver is always ready and punctual despite all odds while driving in remote location and tough weather conditions. His serenity, calm nature and sincerity towards his work is inculcated by his fellow officials. His favourite food is simple Dal & Roti and colour is brown. Sh. Saroop Lal says “It makes feel great to be associated with commissioning of Jammu-Udhampur rail link and now Udhampur-Katra rail link project”.

Sh. Manohar Lal, SSE/P.Way/says, “I feel fortunate & great to to be associated with commissioning of Jammu-Udhampur rail link and now Udhampur-Katra rail link”.

Likes
Favourite food
Favourite colour
Best moment

Light music hindi songs
Rajmah Chawal
Sky Blue
Opening of Jammu-Udhampur Section

Sh. Saroop Lal

Sh. Saroop Lal s/o Datt Lal was born on 1ST October 1955 at Dhurburgee Distt, Gurdaspur, Punjab. He had his early education in his village and joined Railways as C/Labour on 02.6.1977. He was given temporary status as Khalasi w.e.f 1.1.1982. He came on transfer and was posted as Khallasi in JURL project on 27.4.88. After his performance, he had been promoted to officiate as driver w.e.f 12.4.91 in JURL Project. He has successfully passed trade test for the post of driver from Ferozpur division. He has awarded many times for his meritorious services while working in JURL project. He is hard worker, dedicated and sincere in his work while rendering his services during completion of Jammu-Udhampur, opening of Rail link in valley and now in commissioning of Udhampur –Katra section. Sh. Saroop Lal, driver is always ready and punctual despite all odds while driving in remote location and tough weather conditions. His serenity, calm nature and sincerity towards his work is inculcated by his fellow officials. His favourite food is simple Dal & Roti and colour is brown. Sh. Saroop Lal says “It makes feel great to be associated with commissioning of Jammu-Udhampur rail link and now Udhampur-Katra rail link project”.

Employee of the Month
PROJECT NEWS
Udhampur Srinagar Baramulla Rail Link Project

1. Porch of Shri Mata Vaishno Devi Katra Station Building, Pic 1 & 2

2. Member Electrical Visited USBL Project on 13.4.2014. Discussion with Team USBRL during presentation at Katra. Pic 3

3. Work in progress for installation of escalator at katra station. Pic 4

4. Presentation was given by Electrosteel steel Ltd on TMT bars and Ductile Iron Pipes on 07/05/2014. Pic 5 and 6
Presentation was given by Innovative Enterprises on use of construction and demolition waste for making bricks and solving problem of muck disposal, stabilization of roads and micro surfacing of roads, anti corrosion coating for bridges and rail track, water proofing of Tunnels and Dams and solar signaling systems and other application on 07/05/2014. Pic 7 & 8

View of Bridge no 20 (now Bridge no 186) over Jajhar Khad River.
Presentation on methodology of launching of superstructure for Viaduct Portion of Chenab Bridge held on 3/6/2014

Presentation by Dr. V.K.Raina on History and Evolution of Bridges on 24/6/2014

Udhampur station is located on Banks of Tawi River which meanders along foothills of Shivalik mountain range. This early morning photograph shows a thick layer of mist over the river. Udhampur Railway Station is seen at bottom left corner.
1. Pic 16- Chenab Bridge Site
2. Sh Inder Kumar Gujral then Prime Minister of India during foundation stone laying ceremony of Qazigund-Srinagar-Baramulla Rail Link Project.
3. Viaduct Pier of Chenab Bridge Site.
4. Chenab River Valley near Chenab Bridge Site
5. Bird’s eye view of Sangaldan Station Yard.
1.0 Introduction

On Katra – Reasi section of USBRL project, Br. No. 32 on Banganga River is under construction. This work is being directly executed by Northern Railway. The superstructure of this bridge is of pre-stressed concrete. The span arrangement of this bridge is 34m + 64m + 92m + 64m. The first span of 34m is simply supported PSC box girder span. 64 + 92 + 64 is the continuous portion of superstructure. Owing to great height of this bridge, the superstructure had to be constructed by cantilever construction method using Norwegian Rail System (NRS) type gantries. Segmental construction/cantilever construction is a technique of progressive construction of a cast in situ span in segments and stitching them with the already completed segments by pre stressing. The cantilever construction is very much convenient where longer spans are to be constructed and where erection of staging/scaffolding is infeasible.

The planning & finalisation of GAD of this bridge posed numerous complexities and engineering challenges. After lot of deliberations, the GAD & design of Dudhar Bridge, already constructed and in operation on JURL project, was adopted with suitable modifications to suit the site conditions. The GAD is shown in Fig.1. Design features in brief and methodology adopted for segmental construction for the continuous portion of superstructure of Banganga bridge is described in the following paras :-

2.0 Special features of design.

The bridges constructed using balanced cantilever methods are not only complex to construct, they also call for equally complex design methodology. In this method of construction, the bridge deck is built by a succession of segment concreted in situ in travelling forms. Each segment, after it has attained sufficient strength, is stitched with the already cast segment with the help of pre-stressed cables, set in upper portion of the deck and thereafter carries the weight of the formwork and next segment.
Segmental construction of continuous spans of Banganga Bridge (Br. No. 32) on Katra – Reasi section

The volume of calculations involved is huge as there are large number of sections to be checked and the development of static diagram of the work during this construction. Due to the fact that there are various stages of construction, the effects of creep of the concrete, relaxation of the steel and continuous redistribution of the hyperstatic stresses in the structure have to be accounted for and hence design is dependent on the construction sequence and methodology. The evaluation of pre-stressing losses is also required to be done at each stage of construction. For this purpose an American software, ADAPT was used. This program takes care of time dependent stresses like creep, shrinkage and relaxation losses. The second order effects like creep redistribution and hyper static moments are calculated by the software itself.

The complete structure was idealized for design on ADAPT software. The end portions which were to be cast on temporary staging, were idealized based on cable profile (tendon geometry). The construction sequence was given in detail explicitly in “ADAPT- ABI” input file, and also the pre-stressing sequence explained.

Pre-stressing was planned to be done after four days of casting of concrete by which time it attains sufficient strength. Based on these assumptions, the “ADAPT-ABI” software checks the adequacy of pre-stressing profile chosen. At every stage of construction, it is ensured that the concrete stresses on either side of the anchorage remain within permissible limits.

To control the deflection during construction and consequent final profile of the bridge, pre camber calculations were also carried out.

In the service stage, some amount of uplift occurs at the abutment locations caused by unequal placement of live load and seismic cases for which uplift elastomeric bearings have been provided on U-type portal to hold the superstructure down.

The entire superstructure is provided with large size pre-stressing cables of 19T13 using low relaxation strands conforming to IS:14268-1995.

In order to satisfy all the stringent conditions of contract and Railway codes, regarding concrete stresses at various stages of construction, the depth of box girder required over the piers and at the centre of middle span is about 1/11 and 1/17 of central span length respectively, for 3-span cantilever construction bridges. To achieve economy in a 3-span cantilever construction bridge, the end spans should be about 70% of the central span.

3.0 Segmental construction methodology:

The continuous span of Banganga Bridge is being constructed using segmental cantilever construction methodology. The sequence of casting of various portions of superstructure is as under:

i) Casting of Pier Head Unit (PHU).
ii) Erection of Cantilever Construction Equipment (CCE) on PHU for casting of segments.
iii) Casting of successive segments using CCE.
iv) Casting of end spans.
v) Casting of central closure span.

The stages of construction are depicted in Fig. 2.
3.1 Casting of Pier Head Unit (PHU):
After completion of Substructure upto pier cap/abutment level, Pier Head Unit (PHU) on pier P1 & pier P2 of length 10.4m were cast on the staging, supported on the pier caps. The depth of pier head at the centre was 8750mm. POT PTFE bearing were placed on the bearings pedestals before casting the pier head units.
The main bearing pedestals were cast on top of the pier cap with pockets left for the anchor sleeves of the bearings. The bearings were placed in position with the sleeves properly fitting inside the sleeve pockets already left in the bearing pedestals. The bearings were brought in exact alignment by matching the centre line of pedestals and bearings.

3.1.1 Lifting of PHU:
Temporary pedestals (4 Nos.) were already cast before casting of PHU on the pier caps. The temporary pedestals are an important part of segmental construction. The PHU is lifted @ 10mm by lifting jacks. 10mm thick steel plates are inserted between the temporary pedestals and bottom of PHU in the gap created by 10mm lifting of PHU.

Now the total weight of PHU transferred on the temporary pedestals and the contact of main bearing with PHU is also discontinued.

Pre-stressing strands, 08 Nos. (2 in each pedestals) are provided for vertical holding of PHU. One end of these strands were already anchored in pier caps and other end coming out inside the PHU through temporary pedestals. These cables are then stressed to bind the PHU with the pier caps and hence all the weight of cantilever segments is transferred to pier cap through temporary pedestals.

3.1.2 Sequences of whole operation in brief:
   a. Placing of bearing in position.
   b. Casting of PHU on bearing and temporary pedestal.
   c. 10mm lifting of PHU and inserting 10mm thick steel plate over temporary pedestals.
   d. Stressing of 08 nos vertical hold down cables.
   e. Erection of gantry for casting the segments on PHU.

3.2 Erection of Cantilever Construction Equipment (CCE):
3.2.1 Now a pair of gantry system is erected on the top of PHU, one on either side. The gantries projects beyond the PHU to support the hanging shuttering required for casting of segments on either side. The external shuttering of the box section is supported directly from the gantry members. Each travelling gantry system is provided with counter weighing system. The reaction required to transfer the weight and construction loads of the unit/segment to be cast by cantilever shuttering is realized by means of suspenders (stress bars) passing through the decking and soffit slabs and anchored at the base of previous unit.

Cantilever construction equipment consists of truss frame structure placed on bogies. These bogies are supported on rails. The rails are anchored with PHU with the help of high yield strength bars (stress bars). These bars have threads in whole length for tightening of nuts at any required position. For casting of first segment, the gantries are anchored with the PHU through stress bars. Holes are provided at the designated places in the deck and Soffit slab. The stress bars are taken through these holes and bolted at the bottom of deck & Soffit slab. Schematic diagram of gantry is depicted in Fig. 3.
3.2.2 **Sequence of operations:-**
- First of all center line of bridge & position of rail for CCE on deck slab of PHU is marked with paint.
- Main rails and secondary rails are then placed in position with all accessories and marking of position for jacks, rear bogie & rail bogie is done.
- Steel packing & jacks are placed in position & erection of front bogie and vertical post of main frame is done.
- Erection of main frame, front frame, rear frame is then done.
- Stress bars of rear bogie & rail bogie are placed.
- Erection of outer stress bars for rear twin beam & twin beam of bottom slab form work are suspended with the help of these stress bars.
- Erection of stress bars for front twin beams of bottom slab form work and suspension twin beam for erection of bottom formwork.
- Erection of cantilever frame work and wallers in both directions.
- Erection of outer frame work and wallers in both directions.
- Erection of suspenders for inner rails.
- Erection of inner frame work, wallers & tie rods.
- For erecting second CCE on the same PHU :- repetition of above three steps 1 to 3 is done.
- Erection of ancillary frames & repetitions of step 4 as above.
- Removing auxiliary frame & repetition of steps 5 to 11 above.

3.2.3 **Precautions taken:-**
- All stress bars were kept straight & protected from rusting by application of grease etc.
- No welding or gas cutting is allowed.
- All locking plates & quarter frames were placed in position.
- Proper seating of spherical nuts in the bearing plates is to be ensured and all nuts are fully tightened before concreting.
- At the time of casting, the reactions at the location of front bogie was kept on hydraulic jacks & not on wheels. Lock nuts of these jacks were fully tightened.
- Wheels of rear bogie were kept free & without any load at the time of casting of segments. Negative reaction at the location or rear bogie was held back with the help of stress bars at location of rear bogie.

3.2.4 Before concreting of first segment, the web vertical portion of PHU from were the concrete of 1st segment starts, was chipped off with the help of chipping hammer of (BOSCH Co.) electrically operated & then cleaned with wire brush & wetting was done continuously for 12 to 24 hours. Wetting is stopped two Hrs. before starting of concrete. Bonding agent is applied to the old surface so that there is proper bond in old & new concrete. Concrete was done in two stages i.e., soffit and web upto 90cm below deck top on both sides, in first stage and deck in second stage.

3.2.5 **Movement of CCE from the one segment to next segment:-**
This is done sequentially as under :-
- Release inside form work & retract inner vertical form work.
- Release outer form work, remove all tie rods and retract outer form work.
- Release the rail bogie and free the main rail and move main rail forward under launching system moving rail as per approved drawing.
- After moving rail to the location of next segment lock the rail bogie and tighten the stress bars.
- Release the jack at front bogie location so that the front wheel rests on the rail.
- Removal of stress bars of the rear bogie to be done in the following steps :-
  - Pull the rear end of the CCE with the help of pull down cylinder so that the stress bars are free.
  - Unfasten the stress bars of rear bogie.
  - Release the jack of pull down cylinder slowly so that the wheels of the rear bogie touches the cantilever slab connected to already cast segment.
- Remove the stress bars at rear twin beam connected to the soffit slab.
- Remove the rear bars of cantilever slab connected to already cast segment.
- Move CCE as shown in drawing under launching system – moving CCE, with the help of chain pulley block.
- After moving up to next segment location, connect stress bars of rear twin beam to the soffit slab and rear cantilever bogie to already cast segment.
- For adjustment of levels of top of segment, front twin beam is to be lifted with the help of two chain pulley blocks hanging from front frame and opposite level to be achieved as indicated in pre-camber drawing. The final adjustment of this level shall be achieved with the help of pull down cylinder and then rear bogie stress bar to be locked.
3.2.6 On an average, a cycle time of 15 days could be achieved for casting of segments. The details of activities are as under:

<table>
<thead>
<tr>
<th>Day</th>
<th>Activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>Dismantling of web stopper, lifting of reinforcement and cutting of HTS wires.</td>
</tr>
<tr>
<td>2nd</td>
<td>Loosening of Tie rod units, dismantling of inner shuttering, web ballaster and lifting of steel.</td>
</tr>
<tr>
<td>3rd</td>
<td>Dismantling of inner shuttering, HTS cable threading, lifting of steel for other segments.</td>
</tr>
<tr>
<td>4th</td>
<td>Threading of strands, cleaning &amp; lifting of bearing plate of stressing jacks.</td>
</tr>
<tr>
<td>5th</td>
<td>Stressing of cables.</td>
</tr>
<tr>
<td>6th</td>
<td>Loosening of top &amp; bottom deck, rail shifting and locking.</td>
</tr>
<tr>
<td>7th</td>
<td>Moving gantry half portion, leveling &amp; cleaning etc.</td>
</tr>
<tr>
<td>8th</td>
<td>Moving other half gantry portion, leveling and cleaning etc.</td>
</tr>
<tr>
<td>9th to 12th</td>
<td>Reinforcement fixing of both sides, sheathing pipes &amp; profiling.</td>
</tr>
<tr>
<td>13th</td>
<td>Side stopper fixing, tie rod fixing, shuttering, vertically checked.</td>
</tr>
<tr>
<td>14th</td>
<td>Check by Railway &amp; removal of taints.</td>
</tr>
<tr>
<td>15th</td>
<td>Concreting work.</td>
</tr>
</tbody>
</table>

3.2.7 Provision of Pre-camber, before casting of segments:
The segments undergo appreciable deflection during construction owing to cantilever action. The components of vertical deflection of segment can be divided in following parts:-

(i) Deflection of CCE.
(ii) Extension of suspender.
(iii) Deflection due to dead load.
(iv) Deflection due to pre-stressing.

To compensate for these deflections, the segments are cast with some pre-camber. These pre-cambers were provided in segments & the form work were erected likewise. Levels were taken at top of segment on a plate embedded on the top of initial segment expressly for this purpose and on the CCE. These levels were continuously monitored during the whole operation of concreting and pre-stressing. Schedule of recording levels is as under:

(i) After finalizing the shuttering of segment.
(ii) Just before concreting.
(iii) After concreting.
(iv) After stressing of segment.
(v) After removing the CCE support.

3.3 Casting of end spans:-

3.3.1 After the completion of casting of cantilever segments, the end spans are cast on staging supported from the ground. The end span staging and reinforcement is tied prior to completion of casting of cantilever segments. After the concrete of end spans attains strength of 35 Mpa, continuity cables, 36 numbers 19T13, are stressed to make the various parts of the girder continuous.

3.3.2 Before casting of central closure segment, vertical stability cables provided inside PHU are de-stressed and whole of the superstructure is lifted by about 15mm by means of 8 numbers 500t capacity hydraulic jacks, introduced between the soffit of the box girder and the pier cap at predetermined jacking points, using reinforced cement concrete stools and 20mm thick steel packing inserted over the temporary pedestal is removed and then the jacks are released slowly and steadily so that whole of the dead load of the girders is transferred on to the main bearings. The process is repeated for both PHUs.

3.4 Casting of Central Closure Span:

3.4.1 The central continuity pour of 3.6m length is to be cast on shuttering resting over cantilever tips on either side. Any mismatch in the levels of adjacent arms of cantilever reflected is to be corrected by counterweights.
3.4.2 After the central key segment concrete attains a strength of 35MPA, the continuity cables total 44 numbers will be stressed to make whole of the bridge continuous.

3.4.3 After this ballast retainers, wearing coat etc. will be cast. Thereafter load testing will be done.

4.0 Construction Parameters:

The continuous span involves 13 segment @ 3.00 on either side of PHU and than end span of 19.8 m and a closing span of 3.6 m.

5.0 Design Mix and Concreting:

- M-45 concrete has been used for casting of continuous span by segmental construction.
- Semi-automatic batching plant having 15 cum/hour capacity has been installed @ 300 m away from the site.
- Concrete is conveyed at site by transit mixers and at desired location through concrete pumps.
- Compaction is done by electrically operated needle vibrators of 40 & 60 mm needle size.
- Casting of segments is being done in two stages.
- Exposed surface of segments where construction joints are provided, are chipped by an electrically operated chipping hammer and all laitance of concrete are properly cleaned.
- To make the shuttering leak proof, foam sheets are used between the shuttering plates and then all joints are sealed with putty.
- To prevent blockage of sheathing by leakage of concrete, due to risk of puncturing of sheathing, PVC pipes of lesser dia are inserted in the sheathing. These PVC pipe are moved inward and outward periodically during concreting.
- The cables in the cantilever segments are stressed after 04 days of casting, when concrete has achieved a minimum 35MPA compressive strength.
- The stressing of cables is being done by professionals of “Dynamic Prestressers”.

6.0 Specifications of Tendons are as below:

Pre-stressing Tendons = 19T – 13 ( in continuous span)

i. 12.7mm dia of 7 ply confirming to clause 2 of IS : 4268.
ii. Minimum breaking strength = 18750 Kg.
iii. E = 1.95 x 10^6 Kg/cm^2
iv. Cross sectional area 98.68 mm^2
v. Tolerance for location of strands + 5mm.
vi. Stressing is being done simultaneously from both the ends.

7.0 Grouting of pre-stressing cable ducts:

- The grouting of cables, is being done using pure cement mortar having water cement ratio of 0.45, with addition of admixture CUX – 100.
- Grouting is commenced initially with a low pressure of injection.
- The grout is allowed to flow freely from the other end until the consistency of the grout at both ends is same (i.e injection end and other end).
- When grout flows from other end, the end is plugged and built pressure is built up.
- Full injection pressure of 5kg/cm^2 is maintained for atleast one minute before closing the injection pipe.
ABSTRACT

The three-dimensional distinct element program, 3DEC was used to investigate the stability of the bridge abutments which crosses the river Chenab along the proposed railway line between Katra and Qazigund of Northern Railways. In this case, the distinct element code 3DEC was selected to observe movement of the model well into post-peak behavior. Since 3DEC is a three-dimensional numerical code, utilizes a Lagrangian calculation scheme to model large movements and deformations of a blocky system, allows for modeling of large movements and rotations, including complete detachment, of rigid or deformable discrete blocks. The analyses have been carried out in two stages for predicting the behavior of the jointed rock slope. First, an initial static loading is applied in the numerical model to simulate the prevailing rock mass conditions at the site by introducing the joint sets in the rock slope. Second the pier loads were applied on the existing model. Various monitoring points were introduced in the model for critical observations. The displacements, stress, velocity values observed confirms the stability of the rock slope. This paper presents the details of the methodology adopted, properties selected and the results obtained in the analysis.

1. INTRODUCTION

Slope failures and landslides are responsible for millions of rupees of damage to public and private property every year. The primary factors driving this trend include aging slopes constructed for major transportation systems in the country and the ever-increasing need to develop land on steep natural slopes and fills for public and private purposes. Because slopes consist of native or transported earth materials, engineering properties and behaviors are quite variable and unpredictable to precise limits. This variability is compounded by the frequent presence and influence of surface water runoff and groundwater infiltration that often trigger landslide movements.

Slope stability analyses and stabilization require an understanding and evaluation of the processes that govern the behavior of slopes (Hoek and Bray, 1997). Today, the analysis and solution of landslide problems as well as the prevention of landslide problems requires an understanding of geology, hydrology, seismology, geotechnical exploration and engineering, computerized analytical methods, and practical and constructible engineering solutions. The fundamentals of these subjects must be understood as well as the methods for obtaining the data necessary for input to reliable slope stability analyses. Once this data is obtained and analyses carried out using methods ranging between rules of thumb and sophisticated computer methods, the results must be interpreted correctly and actions taken to stabilize the slopes if instability is suspected (Cundall, 1988; Fairhurst, 1993).

The slope stability analysis has many applications in the engineering projects such as the dams, the roads, the open pits structures, the railways and bridges etc. Today due to the fast infrastructure development railways are reaching places which were considered as impossible or inaccessible in early days and through landslide prone areas. Indian railways have made an ambitious plan to connect Kashmir valley with rest of India. The railway project linking Kashmir is a matter of pride and prestige for the Indian railways. This railway line has to cross Chenab river in Reasi district of Jammu and Kashmir state. To understand the stability of bridge abutments a study was carried out on both abutments of the proposed Chenab bridge in Katra-Dharam Section of USBRL Project, J&K using the 3 Dimensional Distinct Element Code (3DEC).

2. SLOPE STABILITY ANALYSIS OF CHENAB ABUTMENTS

The proposed railway line between Katra and Qazigund of Northern Railways cross the river Chenab and Anjikhad, which is a tributary to the Chenab River near Reasi. Two railway bridges are proposed to be constructed, one between Chainage 50.4 and ground level 846.008 m on left abutment near Balckal village and Chainage 51.715 and ground level 848.457 m on right abutments near Kauri Village and the second between Chainage 38.430 Km on left abutment to ICatra side and Chainage 39.087 Km on right abutment to Reasi side. The Chenab bridge alignment is N120°E towards the left abutment to N300° towards the right abutment and the Chenab River flows in SW direction. The Chenab bridge, once it is constructed, it would be the Highest Bridge in the World at height of 359m from the river bed level and the abutments of the proposed bridge are shown in Fig. 1. The 1,263m long bridge consists of 950m span steel arch over the river, in tandem with a 313m long viaduct upto Salal road ‘13’ station. This bridge consists of total 18 piers resting on ground. Among them 4 piers designated as P10, P20, P30, P40 are on left abutment and the other 14 piers from P50 to P160 resting on right abutment and sloping at 50-60°.
3. Geological and Geotechnical Features:

The railway alignment passes through the Shiwaliks and Pre-Tertiary rocks overlain by unconsolidated sediments of recent to sub-recent ages. The primary lithological units are dolomitic limestone with different degree of fracturing and occasional weathering. Cherty, boulders brecciated and massive dolomite or dolomitic limestones of Sirban formations are present in the area. The top layers are moderate to highly weathered but invariably the dolomite is fractured resulting into blocky mass. No major shear zones or solution cavities were found in both the abutments. Thin bands of quartzite and shales are also present in the open pits dug at the pier locations. The fresh dolomitic blocky mass looks competent and stable except on the surface. Mapping with reference to the alignment on the left and right abutments (Chainage 50.383 Km 51.200Km) yielded that the strata are characterized by prominent sub-horizontal foliation joint and two sub vertical joint sets. Few random joint sets are also present. The attitude of the formations of left and right are shown in Table 1.

<table>
<thead>
<tr>
<th>Feature</th>
<th>Strike</th>
<th>Dip</th>
<th>Dip Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rail Line Alignment</td>
<td>N120° E</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foliation Joint</td>
<td>N140° E</td>
<td>21°</td>
<td>N050° E</td>
</tr>
<tr>
<td>Joint-1</td>
<td>N150° E</td>
<td>65°</td>
<td>N240° E</td>
</tr>
<tr>
<td>Joint-2</td>
<td>N075° E</td>
<td>80°</td>
<td>N165° E</td>
</tr>
</tbody>
</table>

Table 1: Attitude of Discontinuities on the Left and Right Abutment

The three joint sets make the rock broken, forming cubical structure. Though the spacing of foliation and other joints is very close, the rock mass is highly interlocked and dry. The joints are very rough or irregular, planner and unaltered with occasional infilling and very minor surface staining. No apparent indications for the presence of any solution cavities in the abutments. The lithological conditions of rock and discontinuity pattern on either side of abutment are more or less similar. The rock mass was characterized through RQD, RMR, Q, and Geological Strength Index (GSI) Classifications by several agencies. The suggested ratings are as follows:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>49 (Fair-Poor)</td>
</tr>
<tr>
<td>RMR</td>
<td>48 (Fair)</td>
</tr>
<tr>
<td>GSI</td>
<td>6.13 (Fair)</td>
</tr>
</tbody>
</table>

These ratings are also used for the prediction of rock mass behavior. The surfacial appearance is very deceptive due to natural dependency of dolomitic rock to break into blocks and associates by points. Normally rockmass ratings will improve with depth. Several plate load tests and insitu direct shear tests were conducted by CSMRS (2009) for ascertaining the rock mass properties. Extensive laboratory studies for obtaining intact rock and joint properties were conducted by NIRM (2005). The geotechnical characteristics of the left and the right abutments of the proposed Chenab bridge are presented in Tables 2 and 3.

4. Analysis using 3DEC

Analysis was carried out using 3DEC (Itasca Manual, 2008) on an equivalent model of size with a width of 200 m (100m on both sides from the proposed railway alignment). Since the actual topographical model is not accepted by 3DEC so equivalent model depicting all the major topographic features were used in 3DEC. The actual topographical model and the equivalent model used in 3DEC analysis is given in Figures 2 & 3. Basically, two random joint sets and one foliation joint set are represented in the model.

Table 2: Representative Geotechnical Characteristics of Chenab Intact Dolomite

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density g/cc</td>
<td>2.762</td>
</tr>
<tr>
<td>Sp. Gravity</td>
<td>2.81</td>
</tr>
<tr>
<td>Porosity, %</td>
<td>1.30</td>
</tr>
<tr>
<td>Hoek-Brown Constant 'm' &amp; 's'</td>
<td>40.1</td>
</tr>
<tr>
<td>UCS (dry) MPa</td>
<td>160.50</td>
</tr>
<tr>
<td>UCS (sat) MPa</td>
<td>160.50</td>
</tr>
<tr>
<td>Point Load Index, MPa</td>
<td>14.12</td>
</tr>
<tr>
<td>Sonic Wave Velocity dry km/sec</td>
<td>4.60</td>
</tr>
<tr>
<td>Ei (50), MPa</td>
<td>4.41 x 10⁴</td>
</tr>
<tr>
<td>Poisson ratio v</td>
<td>0.22</td>
</tr>
<tr>
<td>Cohesion c, MPa</td>
<td>22.50</td>
</tr>
<tr>
<td>Friction, φ°</td>
<td>58°</td>
</tr>
<tr>
<td>Degree Miller Classification</td>
<td>MC/MB</td>
</tr>
<tr>
<td>Vp &amp; Vs, m/sec</td>
<td>6350, 3580</td>
</tr>
<tr>
<td>σ, MPa</td>
<td>16.86</td>
</tr>
</tbody>
</table>

Table 3: Geotechnical Characteristics of Rockmass and Rock Joints for Chenab Abutments

Fig. 2: Actual Topographical Model of Left Abutment
Roller boundary conditions are assumed along the lateral sides of the model such that no displacement is allowed in the x-direction. At the base of the numerical model, the boundary is fixed such that no movement is allowed in the y and z direction. The monitoring points and the locations of piers along the proposed railway alignment for both abutments are shown in Figures 4 & 5. Here displacements, shear stress and shear velocities are recorded throughout the numerical simulation. The stresses in X, Y, Z, directions, shear stresses in XY, YZ, XZ planes, X displacement, Y displacement, Z displacement contours, X velocity, Y velocity, Z velocity contours, maximum and minimum principal stresses at Z=-100 (along the proposed railway alignment), Z=-50 and Z=150 (50m on the either side of the proposed railway alignment) were also recorded.

5. RESULTS AND DISCUSSIONS
Chenab Left Bank with Pier Loads
After the application of pier loads it was found that the X displacements at all monitoring points varies between 4.0 to 7.0 mm, Y displacements varies between -9 to -15 mm and the Z displacements varies between -0.6 to 0.25 mm. The XX, YY and ZZ stresses at P40 vary from -0.9 x 10^6 Pa to -0.46 x 10^6 Pa respectively. The XX YY ZZ XYZ and YZ stresses at Z = -100m (along the railway alignment) vary from 0.5 x 10^6 Pa to 8.5 x 10^6 Pa respectively. The XX YY ZZ XYZ and YZ stresses at Z = -50m and Z=-150m (50 m from the railway alignment) vary from 1 x 10^6 Pa to 9 x 10^6 Pa respectively. The velocities in X Y and Z direction converging to zero at all measured locations. The displacements, stresses and velocities at Z = -50m, Z=-100 and Z=150m values confirm that the shear stresses and displacement values are very low which indicates that there is no appreciable movement of blocks due to the piers and the slope was found to be stable.

Chenab Right Bank with Pier Loads
After the application of pier loads it was found that the X displacements at all monitoring points varies between 1 to 2.5 mm. The Y displacements at all monitoring points varies between -7 to -10 mm. The Z displacements at all monitoring points varies between -1.3 to -1.9 mm. The XX YY and ZZ stresses at P50 are calculated as -0.18 x 10^6 Pa and -0.72 x 10^6 Pa respectively. The XX YY ZZ XYZ and YZ stresses at Z = -100m (along the railway alignment) varies from 0 Pa to 8.5 x 10^6 Pa respectively. The XX YY ZZ XYZ and YZ stresses at Z = -50m and Z=-150m (50 m from the railway alignment) are calculated as 0 Pa and 6.5 x 10^6 Pa respectively. The velocities in X Y and Z direction converged to zero at all measured locations. The displacements, stresses and velocities at Z = -50m, Z=-100 and Z=-150m values confirm that the shear stresses and displacement values are very low which indicates that there is no appreciable movement of blocks due to pier loads, so the slope was inferred to be stable.

6. CONCLUSIONS
Numerical analysis of natural 3DEC considering all joint sets, rock properties and stiffness characteristics with different loading conditions were carried out. Horizontal (arch) loads on the central sections improved the stability. 3DEC analysis revealed left and right slopes are stable under different loading conditions. The displacements, shear velocities, shear stresses, XX and YY and ZZ stresses at all monitoring points and at Z=-50m, Z=-100m and Z=-150m for both left and right abutments are within the acceptable limits and shows that the slopes are stable in static condition. No considerable changes are observed at foundations of the arches. No differential values are observed at monitoring points installed 10 m below the foundation level of the proposed pier locations.

REFERENCES:
CSMRS (2009). Report on deformability characteristics and shear strength parameters of rockmass in the right bank drift (reasi end) of Anjikhad Bridge project, Reasi dist., J & K.
1.0 Background:-

2.0 Introduction:-
2.1 Supervisory Control and Data Acquisition (SCADA) System have been introduced to enable coordination of multiple systems at multiple locations and automation by processing the output of different systems. SCADA Systems are being installed for automatic control, remote supervision and remote control of the safety-critical systems of applications on transportation routes. Commonly SCADA Systems are being used in every industry where automation is required and the main goal is to achieve coordinated automation of a variety of instruments. It is a very old system for Electric Traction Distribution system in Railways which came with electro mechanical relays from SNCF Railway, and mainly used for operation of switchgear of Traction Substation, Sectioning Post and Sub Sectioning Posts remotely, from Centralized Control Centre.

2.2 Primary systems such as Lighting System to provide better visibility on the route, Signaling System to direct vehicle or passenger traffic, Fire Detection System to detect fire cases, Ventilation System to provide constant air quality, suppression and intervention of fire events, CCTV System to monitor the route, Video Detection System to automatically detect incidents by CCTV images, Public Address System, Emergency Telephone System and Radio System to communicate with people on the route and Power Systems to supply various systems along with other application specific systems have been evolved and being deployed to work together and maintain the safety of the passengers on the route.

2.3 On USBRL Project, for the first time on Indian Railway, SCADA system has been used for Pir Panjal Tunnel (T-80) having a length of 11.215 kM for operation, control and monitoring the Power Supply, illumination, Ventilation and Safety systems from Tunnel Control Centre located at Banihal Railway Station. Need for SCADA system arises, because of various Safety systems, Power Supply systems, Lighting systems and various type of sensors are spread over the entire length of tunnel which cannot be operated, controlled & monitored manually. For their manual operations a huge manpower is required to be deployed which even may not provide required action promptly and in correct sequence.

2.4 To curtail the manpower requirement and minimize the human intervention, SCADA system has been used for monitoring the Tunnel Environmental Conditions, health of various switchgears, ventilation fans, safety systems etc. and operation of various above systems remotely from Tunnel Control Centre through SCADA. A view of SCADA Control room is shown.

3.0 System Design Adequacy:-
The Electromechanical and Tunnel Ventilation System with SCADA have been designed by M/s Geo Consultant RITES appointed by M/s IRCON. The adequacy of design was got validated by an independent Proof Consultant M/s HBI, Germany who has certified as “HBI certifies an adequate and state-of-the-art design of the tunnel ventilation, fire escape, emergency rescue and other E&M systems for Tunnel No- T-80 across Pir Panjal. The design is adequate for the specified modes of operation and the specified weather conditions.”

By:-
Sh. R.K.Chaudhary,
Chief Electrical Engineer,
USBRL Project
4.0 SCADA SYSTEM: Consists of Software and Hardware.

4.1 SCADA SOFTWARE SYSTEM:-

4.1.1 The goal of SCADA is to get fast response time and reliability. In safety critical systems, the most important issues are reliability, fast response and ease of use. SCADA software can communicate with the devices in the tunnel in less than three seconds. SCADA System along with all the infrastructure and supporting systems are built with optimization towards these priorities.

4.1.2 SCADA system runs on Redundant Server System. This enables redundancy of data archiving and operations. System run operations via either one of the servers located at Service Buildings at North Portal and South Portal of Tunnel T-80. In case, that server is disabled due to any reason, other server is activated automatically. This sustain 24X7 control without any interruptions. While the system is running on an active server, all the data is archived on both servers. Redundancy of servers and redundancy of controlling computers gives the system flexibility and is fail safe for even the worst case scenarios.

For physical SCADA System, three workstation computers are located at i.e. Tunnel Control Centre at Banihal Station, Equipment Building at South Portal and Equipment Building at North Portal for visualization of software. Users are able to access, search, filter, manipulate and print all records, alarms and logs from both servers through any one of these 3 locations.

SCADA system software architecture has 4 password protected user types i.e. Service engineer, Administrator, Operator and Monitoring only which can be given as per Railway requirement.

4.1.3 Basically Installations in the tunnel has 3 levels for control mode:

a. Local mode: Control through commending panels and directly through equipment. This mode is out of SCADA Software range and the software has no control on it.

b. Remote manual mode: Controlling remotely through SCADA, manually by human operator commands.

c. Remote automatic mode: Controlling remotely through SCADA, automatically by pre-programmed software with operator selected parameters.

Operator can switch between remote automatic mode and remote manual mode. Local mode is activated by maintenance personal from the MVS-Niches or Equipment Room Buildings. A priority system is used for controlling system.

4.1.4 SCADA system software has 4 control levels location-wise:

a. Automatic remote control from Control Center in case of fire: System operates automatically depending on sensor values or scenarios


c. Manual remote control from Control Center: System operates manually using single remote commands from the operators.

d. Automatic remote control from Control Center: System operates automatically depending on sensor values or scenarios for normal operations.

Whatever control is actively running, in case of unexpected situations, SCADA Software will warn operator via alarming system. In case of a failure, fault or unexpected reading, the system generates an alarm with visual and audible indicators to warn operator.

In fire cases, the system can even override the operator to make sure that necessary action is taken in time or operator can take over on manual mode to ensure running of ventilation system in right direction after having a confirmation from train crew.

4.1.5 Depending on the urgency of the action required, alarm signals from the monitoring systems are classified into 4 types as below:-

(i) FIRE: This type of alarm signal includes all signals from the fire detection system. Alarms from the linear fire detection system is given top priority and all other fire alarms including emergency push button alarms and fire extinguisher removal is given lesser priority than linear fire detection system alarms.

(ii) URGENT: This type of alarm signal includes all events or circumstances, which involve an actual or potential hazard/danger requiring immediate action, for example alarms which indicate a breach / lack of security.

(iii) ALERT: This type of alarm signals includes all alarms signals which do not require immediate action but repairing action need to be taken. This type of alarms is stayed on the alarm list until the needed repairing action is done.

(iv) RECORD: This type of alarm signal includes events such as plant switching events and the entry or exit of personnel attending the service’s buildings, i.e. actions resulting from normal plant operation. Although these alarms require no action, they provide an historical record which is useful for fault investigation.
When systems receive an alarm signal, there is a flashing alarm banner at the top of the screen which gives the detailed information about the alarm, such as alarm time, the device name or location. Also, the corresponding device changes its visual state on the SCADA screen to alarming state.

Alarm banner continue flashing and audible signals won’t be silenced until the alarm is acknowledged by the operator. After the alarm is acknowledged, the alarm banner disappears and audible signals silenced but alarm will stay in an active alarm list until the input is repaired or rectified.

All types of alarms and acknowledging of these alarms are logged into the server storage. Users with the “Administrator” privilege are able to search, filter and list the alarms retrospectively.

### 4.2 SCADA HARDWARE SYSTEM

#### 4.2.1 SYSTEM HARDWARE ARCHITECTURE

System is equipped with 7 redundant complete Programmable Logic Controllers (PLC) and Remote Terminal Units (RTUs).

For the usage of these electronic devices, there are a specific panel that contains this PLC, RTU and other automation and communication devices. These panels are:

(i) **UPS PLC Panels**: 14 UPS PLCs manage the equipment (alarms, faults, measures, settings) and are capable of carrying out the processes and actions of the reflex action system in its Niche Sub-station and Portal Sub-station zones, UPS PLC Panels contain 2 CPUs for redundancy.

(ii) **11KV RTU Panels**: A RTU panel is responsible from the 11KV Cubicles, circuit breakers and disconnectors that are located at each Portal Sub-Station and MVS Niche.

#### 4.2.2 SYSTEM HARDWARE COMPONENTS

- **Distributed RTU Panels**: These RTUs are not classified location based instead of system based. There are 38 Distributed RTU Panels located in 38 MNs inside the tunnel.

All of these devices create 7 main systems. Each system has the following properties:

(i) One Active CPU & One Redundant CPU
(ii) 2 Redundant power supplies
(iii) 2 Industrial Ethernet Adapter
(iv) 2 Field Network Adapter(PROFIBUS)
(v) 2 Field BUS network ring topology
(vi) Some RTUs distributed to site
(vii) Active & Passive network components used at installation of network.

5.0 Control and Monitoring of Various Systems through SCADA

#### 5.1 VENTILATION SYSTEM:

Status, faults and alarms of all ventilation equipment (jet fans and measurement sensors) and readings from the measurement sensors are displayed in Ventilation Interface. Working hours of each jet fan are also being displayed. Carbon monoxide, Dust Particle Density (Visibility), velocity of air and direction of the air flow inside the tunnel are measured and displayed.

Operator can change the status of a jet fan or operation mode of the jet fans from this interface. There are three operation modes for jet fans:

- **Automatic Remote**: Jet fans operates automatically depending on air quality measurement sensor readings
- **Manual Remote**: Jet fans operate manually using single remote commands from the operators.
- **Local Mode**: Jet fans are controlled by the maintenance personal from the MVS-Niches or Equipment Room Buildings.

Operator can switch between automatic remote mode and manual remote mode. Local mode will be triggered automatically whenever maintenance personal controls jet fans from the MVS-Niches or Equipment Room Buildings. A priority control system is used for controlling ventilation system.

1st (highest) priority: manual control at the MVS-Niches
2nd priority: automatic control in case of fire (fire emergency operation)

### 3rd priority: manual control from Equipment Room Buildings
4th priority:  manual control from Control Centre
5th (lowest) priority: automatic control via air quality
In order to control jet fans automatically via air quality measurements, SCADA system takes the readings from 21 points inside the tunnel and compares the most critical value (it will be the highest or lowest reading depending on the sensor type) with the pre-defined thresholds. After the comparison, if values are higher/lower than threshold value for a period of time (five minutes by default), SCADA change the ventilation level and start/stop the jet fans if necessary. Operator is able to change the ventilation level manually from the Ventilation interface. All threshold values and waiting time period can be changed from the SCADA settings menu.

In case of an incident detection in tunnel (such as fire or an accident), SCADA system controls the jet fans appropriately depending on the predefined scenarios. These scenarios and actions are described in the software. There are 16 scenarios for Passenger train and 16 scenarios for Goods train.

5.2 LIGHTING SYSTEM:-
A global view of the tunnel lighting is displayed in the Lighting Interface. Main tunnel lighting, access tunnel lighting and emergency lighting

5.3 EROS :-
Escape route orientation sign (EROS) systems can be controlled through lighting interface.

5.4 MEDIUM VOLTAGE SYSTEM:-
From MV interface, user can observe all electric equipment installed in Service Buildings, MVS-Niches and Maintenance Niches. Users are able to see the on/off status, operation modes and faults of circuit breakers, current, voltage, effective power, and frequency measurements, fuse breakdowns and compensation errors, status and fault information of diesel generators and UPSs.

In addition to measurements, reports and trends about these information (such as total power consumptions for the preceding and current months, cumulative real, reactive consumptions) is displayed at the reports.

In case of a fault or alarm situation, the SCADA system warns the operator from this interface by generating visual and audible alarms on the screen.

5.5 LINEAR FIRE DETECTION SYSTEM:-
SCADA system will take the readings from 44 zones of temperature values through the sensors inside the tunnel and if the temperature increases according to fire, the location of fire is displayed on fire detection screen and by the separated zones, location of fire can be easily seen.

5.6 FIRE DETECTION SYSTEM :-
Alarms, status and faults of door contacts installed on maintenance cabinets, accesses and air locks, fire extinguisher contacts and status of fire alarm button is displayed in this interface. In case of a violation, SCADA generates alarms and show the exact location of the incident through sabotage screen. Also, SCADA automatically displays the corresponding camera on the emergency screen.

5.7 CCTV SYSTEM :-
Alarms, status and faults of all CCTV equipment are displayed in the CCTV interface. A synoptic view of locations and of the CCTV cameras is displayed. And also, alarms coming from the Video Detection Unit are displayed in this interface.

Whenever Video Detection Unit detects an incident, software switches on automatically to the CCTV interface and shows the location of the alarmed camera with a symbol over it. This symbol changes depending on the type of the incident. Also, the SCADA software is automatically display the corresponding camera image on the emergency screen.

5.8 RADIO SYSTEM:-
Alarms, status and faults of all Radio equipments (cabinets, antennas, etc.) are displayed in the Radio interface.

Radio equipments installed inside tunnel.
5.9 PA SYSTEM: -
A location of Loudspeaker which is used for Public Address System, is displayed in this interface.
Loudspeaker installed inside the tunnel.

5.10 EMERGENCY CALL SYSTEM: -
Alarms, status and faults of all Emergency call system equipment are displayed in the SOS interface. In case of a call, SCADA generates visual and audible alarms and nearest camera is shown on the emergency screens

6.0 SCADA SOFTWARE SCREENS are as under: -

6.1 MAIN SCREEN
This screen is also called Welcome Screen and it is displayed when users log into the SCADA System. On this screen sub-systems, alarms, active server, date time are displayed. Tools button show system reports and trends and change SCADA software settings.

Various Tool Buttons and their functions are given in the table below:

<table>
<thead>
<tr>
<th>BUTTON</th>
<th>FUNCTION</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCADA SETTINGS</td>
<td></td>
<td>SCADA settings with this button. Only administrators can access settings window.</td>
</tr>
<tr>
<td>SCADA MANUAL</td>
<td>MANUAL/AUTOMATIC</td>
<td>Can choose SCADA screens to manual or automatic.</td>
</tr>
<tr>
<td>SCREENSHOT</td>
<td></td>
<td>Can capture SCADA screen with this button. Can also print SCADA screen to printer.</td>
</tr>
<tr>
<td>TRENDS</td>
<td></td>
<td>Can see graphical system data from this section.</td>
</tr>
<tr>
<td>REPORTS</td>
<td></td>
<td>Can see tunnel report from this section.</td>
</tr>
<tr>
<td>SYSTEM LOGS</td>
<td></td>
<td>Can see system log from this section. Only administrators can access system log section.</td>
</tr>
</tbody>
</table>

Table 1 – Tool Menu Buttons Functions and Descriptions

6.2 MV SCREEN: -
With the information in MV Interface it is possible to observe and alter the MV Supply by controlling the Circuit Breakers.

Image 1 – MV Screen

Image-1
1. View the MV screen.
2. Status and faults of Circuit Breakers of 11KV GIS Panels
3. Status and faults of Circuit Breakers of Net Input Panels
4. Monitory information provided from 11 KV GIS Panels
5. popup screen for controlling the Circuit Breakers manually.

6.3 LIGHTING SCREEN: -
In lighting screen all lightings and EROS signs are controlled.

Image 2 – Lighting Screen

Image-2
1. View the lighting screen.
2. To control HPS or UPS & NET lighting which you want to control from the screen.
3. ON/OFF control screen for lightings.
4. EROS
5. control screen for EROS to choose direction.
6. to turn all UPS&NET lights off for power saving mode.
7. for multi controlling for EROS.

Manually switching lights on/off;
- 1- open the lighting screen.
- 2- choose the lights which you want to control and click HPS or NET & UPS to switch it on / off.

Manually switching EROS right / left:
- 4- choose EROS which you want to control.
- 5- click direction image to change direction and if you want to flash it click “flashing”.

6.4 VENTILATION SCREEN:

On this screen, air flow meter, carbon monoxide and visibility values are shown. Also for each portal, control options are available. By these fields, ventilation system can be operated manually or automatically by the user. In manual operation, the user makes any jet fans work by clicking the image of it. In automatic operation the jet fans are working automatically according to ventilation sensor values. Also various fault and communication error information about ventilation system equipments can be seen on this screen.

Image 3 – Ventilation Screen

Image 3;
1. View the ventilation screen.
2. air flow meter, carbon monoxide values are shown instantly.
3. For controlling of automatic to SCADA and vice versa.
4. Jet fans image to be chosen for controlling.
5. control options screen for jet fans.
6. stop all running jet fans.

Taking control of ventilation system by SCADA or Automatic;
- 1- opens the ventilation screen.
- 3- select control type from the image that over the jet fans to control jet fans automatically or manually.

Running jet fans RIGHT / LEFT directions;
- 1- opens the ventilation system screen.
- Be sure that SCADA button is clicked on image (3) for the zone that the jet fan that is desired to be run belongs.
- Click the jet fan that is to be controlled.
- 5 choosing direction of jet fan and click APPLY button.
- After counting to zero, animation appears. Jet fan works for the direction that is chosen.

Stopping Jet fan;
- 1- opens the ventilation system screen.
- Be sure that SCADA button is clicked on image (3) over the jet fan that is to be stopped.
- Click the jet fan that is to be stopped.
- Click “STOP” button and click “APPLY” button.

6.5 CLOSE CIRCUIT TELEVISION (CCTV) SYSTEM SCREEN:

6.6 LINEAR FIRE DETECTION SCREEN:

On this screen inside tunnel, fire alarm that is activated by the values of detector that is sensitive to temperature is shown on SCADA interface. In this system the tunnel is divided into specified zones. If the alarm is activated in any zone, that zone produce video and audible alarm on SCADA and the process of related scenarios about fire alarm occurs.

Image 5 – Linear Fire Detection Screen

Image 5;
1. View the linear fire detection screen.
2. indicates the detectors.
3. shows the separator for the zones.
4. shows the heat data from the OFC.

6.7 FIRE DETECTION SCREEN:

Sabotage detection sensors distributed inside the tunnel, that belongs to fire detection system and the alarms that are received from these systems can be viewed. The detectors that are activated when the fire extinguisher is removed inside the tunnel and the camera that is related with the location of this situation can be seen on the sabotage screen. Also fault status of detectors can be seen from the SCADA screen.

Image 6 – Fire Detection Sabotage Screen

Image 6;
1. View the fire detection screen.
2. When fire extinguisher is removed, alarm is activated and the image of the fire extinguisher which is removed is flashing.
3. when the fire alarm is activated from inside the tunnel, you can see the corresponding button in this screen is flashing.
6.8 EMERGENCY CALL SCREEN :-
Inside the tunnel, there are emergency phones that are attached to telephone niches and their status can be monitored by SCADA interface. If there is call from control center to the tunnel or any call inside the tunnel is being monitored on SCADA instantly with audible and video alarm. When the call ends, log information that includes caller id and call duration about the call can be seen. The operator can take its soft copy if he wants.

Image-7; Emergency Call Screen

1. View the emergency call screen.
2. the image becomes red in which the emergency phone is used and video alarm banner appears at the upper part of the screen with audible alarm.

6.9 RADIO SCREEN:-

Image-8; Radio Screen

1. View the radio screen.
2. the image becomes red in which the radio system failures and video alarm banner appears at the upper part of the screen with audible alarm.

7.0 REPORTS :-
The data from the equipment that is inside the tunnel is gathered via PLCs’ and stored in a relational database (RDBS). These data are shown on SCADA software as it is gathered. But also for maintenance, statistical use and enhancement purposes this data is stored. These data is accessible from SCADA software at anytime. The following reports can be generated:-

7.1 DAILY REPORTS:-
Daily reports are generated in various formats for the following records:-
(iii) Lighting runtime record.
(iv) Tunnel air quality monitoring and sensors record.
(v) Linear heat detection temperature record.
(vi) Jet fan runtime record.
(i) Power consumption record.
(ii) System-wise fault alarm count record

7.2 MONTHLY REPORTING:-
It is the same format as daily reports; total results of the values are calculated monthly.

7.3 YEARLY VENTILATION REPORT:-
Monthly measured values for above records are summarized on yearly basis.

8.0 TRENDS IN GRAPHICAL FORM:-
Trends in the SCADA Software are graphs that show the change of value for a data over time. These displays provide easily understandable summaries of data. Using trends visualizing the changes of environment in the tunnel is much easier.

8.1 AIR FLOW VELOCITY VALUES TREND:-

The graph shows the air velocity measured by Air Flow Monitoring (AFM) sensors at selected location(s) on time axis and summarizes air velocity measurements at any given time.

8.2 CARBON MONOXIDE AND DUST PARTICLE (VISIBILITY) VALUES TREND:-

It is possible to generate graphs of the data from the detectors such as dust particle and carbon monoxide detectors that are located on various locations throughout the tunnel. These graphs are useful for the optimizing the use of ventilation system. Multiple graphics choice for multiple detectors is available for this trend. All of the color codes and graphic names of each graph are written on the trend. To resolve optical illusion problem, take the graphs by searching the details of vision.
8.3 NUMBER OF RUNNING JET FANS TREND:

The number of running jet fans are also observed and stored by the system. This trend is useful to understand the relation between tunnel ventilation system and air pollution. Display for this trend can be done in comparison with Carbon Monoxide readings and DP (Visibility Readings).

![Image 11 – No of Running Jet Fans with CO Values Trend](image11)

![Image 12 – No of Running Jet Fans with DP (Visibility) Values Trend](image12)

9.0 ALARMS:

SCADA software generates visual and audible alarms that are relative to the system according to the information that are coming from the equipment inside tunnel. These alarms are classified on the base of systems. The information about these alarms like time, equipment, location is recorded along with the operator’s reaction to this alarm. If there is an alarm that becomes extinct without being taken by operator is also recorded as a different group. To view the alarms, click ALARM LIST on the upside of SCADA screen. The alarms that are listed on that alarm list are active alarms and their alarm status is still fresh to inform operator.

![Image 13 – Alarm List Screen](image13)

10.0 SCENERIOS:

The SCADA System Server provides the centralized control of the scenarios. Evaluating the data from equipment, SCADA Software residing in the Server catches any alerting predefined case and takes necessary actions.

For example in case a fire detector gives fire signal, the SCADA Software will automatically control lighting and ventilation depending on all the other data coming from the tunnel and predefined procedures.

In such a case a button appears on the upper right corner of the SCADA software that resides on the Workstations. In case the operator confirms the scenario, all the actions required will be handled by SCADA Software with only one click.

![Image 14 – Scenarios Action Button](image14)

11.0 Conclusion:

11.1 As an automation system, the main goal of SCADA System is to create autonomy and handle the coordination of multiple systems. SCADA Systems are controlled and observed by Human Operators through Human Machine Interfaces of different sources such as computers, tablet computers, smart phones and etc. The Operator can observe the status of the systems and individual plants, process data gathered from the plant, perform an operation or override automatic operations. One of the common features of the SCADA Systems is generation of alarms depending on plant data, in case of a fault, malfunction or unexpected event. SCADA Systems are also equipped with data logging features; all relevant data such as sensor readings, alarms and performed actions are stored within the system for later reference.

8.2 System has been designed, installed and commissioned as per the world Standard Practices for Tunnel T-80. The system was provided for first time on Indian Railways which has been approved by Railway Board vide letter No-2010/W-2/NR/J&K/06 dated 22/24.06.2013.

8.3 The electronic data available in Tunnel Control Centre can be hooked through GPRS to ASMs of adjoining stations and Divisional centralized traffic control in future for better information sharing and monitoring the condition of the Tunnel environment and the equipments.

8.4 For Tunnels under construction on Katra-Banihal section, Consultants of International repute are required to be appointed to suggest uniform systems for illumination, ventilation, Power supply and safety systems for all type of tunnels in shape and sizes and frame the guidelines for future works accordingly.
New Austrian Tunneling Method (NATM)

When we go back to the origin of NATM, Prof. L.V. Rabcewicz (1964), the principal inventor, explains the method as: "...A new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring –called an "auxiliary arch"– the deformation of which is measured as a function of time until equilibrium is obtained."

The definition given above has then been redefined by the Austrian National Committee on Underground Construction of the International Tunneling Association (ITA) in 1980– "The New Austrian Tunneling Method (NATM) is based on a concept whereby the ground (rock or soil) surrounding an underground opening becomes a load bearing structural component through activation of a ring like body of supporting ground".

Another recent definition on NATM given by Sauer (1988) states that NATM is: "...A method of producing underground space by using all available means to develop the maximum self-supporting capacity of the rock or soil itself to provide the stability of the underground opening."

As a result of the above statements, it is clearly agreed by the Austrian proponents that NATM is an approach to tunnelling or philosophy rather than a set of excavation and support techniques.

In summary, the following major principles/characteristic features, which constitute the NATM, are as follows:

1. The inherent strength of the soil or rock around the tunnel domain should be preserved and deliberately mobilised to the maximum extent possible.
2. The mobilisation can be achieved by controlled deformation of the ground. Excessive deformation which will result in loss of strength or high surface settlements must be avoided.
3. Initial and primary support systems consisting of systematic rock bolting or anchoring and thin semi-flexible sprayed concrete lining are used to achieve the particular purposes mentioned in the earlier points. Permanent support works are usually carried out at a later stage.
4. The closure of the ring should be adjusted with an appropriate timing that can vary dependent on the soil or rock conditions.
5. Laboratory tests and monitoring of the deformation of supports and ground should be carried out.
6. Those who are involved in the execution, design and supervising of NATM construction must understand and accept the NATM approach and react co-operatively on resolving any problems.
7. The length of the unsupported span should be left as short as possible.

Understanding NATM approach or philosophy:

The Rabcewicz shear failure theory around an opening-

Recalling his failure theory when a cavity is made in rock, the stress rearrangement occurs in three stages as seen in Figure 1.

At first, wedge-shaped bodies on either side of the tunnel are sheared off along the Mohr surfaces and move towards the cavity (I). In stage two, the increase in the span leads to convergence of the roof and floor. The deformation at the crown and the floor of the cavity increases more and the rock buckles into the cavity under the constant lateral pressure (III). The pressures that arise in stage (III) are termed “squeezing pressures” and rarely occur in civil engineering activities due to shallow depth of excavations.

Based on above shear failure theory, proposed NATM support systems by Rabcewicz fall into two main groups: -

"The first is a flexible outer arch-or protective support-design to stabilize the structure accordingly, and consists of a systematically anchored rock arch with surface protection mostly by shotcrete, possibly reinforced by additional ribs and closed by the invert…

The second means of support is an inner arch consisting of concrete and is generally not carried out before the outer arch reached equilibrium…".

To be able to design the load bearing capacity of the lining for different types of rock or soil, the phenomena of shear failure, explained earlier, should be interpreted accordingly. The relationship between the disturbed ground around the cavity, “protective zone” and the bearing capacity of the support, “skin resistance” is required to be established.

The ground response curve (Figure 2) shows the rock/support interaction and deformations in time.

Figure 2: Ground-support interaction curves (quoted by Rabcewicz 1973)
It provides a tool to idealise support stiffness and time of installation. When a stiffer support (shown as '2') is chosen, it will carry a larger load because the rock mass around the opening has not deformed enough to bring stresses into equilibrium. Thus, the safety factor will sharply decrease. After point C, ground behavior becomes non-linear. If the support (1) is installed after a certain displacement has taken place (point A), then the system reaches equilibrium with a lower load on the support. Thus, Rabcewicz (1973) concluded, "It is a particular feature of NATM that the intersections always take place at the descending branch of the curve". This implies a less stiff support which causes the required deformation as in the case of a NATM application.

Moreover, he stressed that rock support should be neither too stiff nor too flexible. After the point B “detrimental loosening” starts and the required support pressure to stop the loosening increases greatly. However, if the support is applied at the right time for the correct deformation, the support pressure takes the minimum value at this point.

The above discussions lead to NATM definitions merging in a sense that
i. Utilisation of ground as a part of support is the main concern.
ii. Application of the primary lining to reach equilibrium at the optimum deformation with possible additional support elements, such as rock bolts, steel arches, ribs etc.
iii. Closing the ring at an appropriate time by using the ground support interaction curve and monitoring the ground response with systematic measuring systems.
iv. Stabilisation of the tunnel by use of a secondary lining.
v. Dimensioning the excavation portions of the tunnel dependent on the ground conditions.

Design criteria and features of NATM:-
The principles for an appropriate design methodology for NATM can be divided in two main design groups. The first could be considered as a function of NATM technical requirements with the application in soft ground or rock regarding support system. The second is dependency on the external constraints, such as settlement problems, environmental impacts, safety, engineering technology, and contractual and financial constraints.

After determination of the geometry and size in respect to its application in soft ground and/or rock mass, NATM design is mainly related to its support characteristics:

Primary and final support design:-
For shotcrete and secondary lining design the following should be considered:

1. **Ground characteristics**, such as strength and stand up time must be determined. The ground support interaction curve obtained accordingly.
2. **Ground water** must be taken into consideration and required drainage or sealing should be maintained
   a) If drainage is considered, the long-term stability of the drainage holes must be preserved and the quantity of these holes in respect to the water intake must be determined.
   b) When sealing is considered, water pressure must be taken into account in the design to calculate the loads on the lining. The long-term stability of the waterproof membrane should also be considered.
3. **Additional support elements** such as rock bolts, spiling, lattice girders, steel welded mesh or steel fibre reinforcement should be used to increase the strength of the shotcrete. Shotcrete materials must be considered in the lining design to optimise time-dependent behaviour to answer the necessary flexibility and load bearing capacity.
4. **Monitoring** of the stresses in/on the lining and the deformation must be provided.
5. **Preliminary design of the initial lining** should be conducted using available means of analysis such as empirical methods based on stochastic and/or observations, computational methods and small or full-scale physical models.
6. **The secondary lining** is placed after shotcrete has been applied. These concrete slabs are generally connected to each other with joints, which may be plane, or helical joints, concave/convex joints, convex/convex joints, and tongue and groove joints.
Primary and Final lining along with Waterproofing membrane, for C-1 Rock class

Geotechnical design criteria: -
Recalling NATM’s main principle, the surrounding body of an opening is the main load-carrying component in its application. For optimisation of the load bearing capacity of the medium the characteristic ground-support reaction curve needs to be established. Therefore, the possible ground conditions should be interpreted from site and laboratory tests. It is also believed that the main cause of failure is unexpected ground conditions. Therefore, the ground investigation must be conducted thoroughly to ensure that there is no possibility of meeting any unexpected ground conditions. The strength of the ground, stand-up time, pore water and drainage conditions, homogeneity and non-linearity of the ground, heave potential, time dependency or creep behaviour, discontinuities, the earth pressure at rest, magnitude of overburden pressure must be taken into account during these investigations. As a result, appropriate geotechnical design parameters must be chosen to fulfil analytical or computational preliminary design for eligible excavation patterns and geometry, and face advance in each round, as well as optimum support design.

Safety measures and design criteria before, during, and after a construction of NATM tunnel, as summarised in a report on Safety of NATM tunnels are as follows:
- Ground investigation: to reduce the likelihood of encountering unexpected geological conditions.
- Engineering technology: The technological improvement in tunnelling equipment must be considered and new technological progress should be employed to take advantage of them.
- A risk-based approach to NATM design: In tunnel design and construction, there has always been some degree of uncertainty. This issue is significantly related to the NATM. Thus, a risk-based approach to design and management is required.
- Monitoring: There are two essential objectives of monitoring; design monitoring and construction monitoring. Monitoring should be undertaken to ensure safety of design and construction. Data assessment and interpretation must be done by the geological/geotechnical specialists, tunnel designers, construction managers (including quality and safety managers).

- Stability of the tunnel heading: The tunnel heading is the part of the tunnel that is excavated ahead of the completed support ring. Most failures occur during or soon after excavation of this part of the tunnel. Therefore, stability of the face must be maintained using additional supports such as forepoles, faster excavation, draining ground water and reducing the face size or advance per round.
- Ground settlement control measures: To reduce the risk of damage to surface buildings, settlement due to tunnel excavation must be controlled by proper construction of the tunnel heading, under-pinning existing structures, and compensation grouting.
- Sprayed concrete lining design: The physical properties of the shotcrete such as thickness, additional reinforcement, must be designed according to the project requirements. Necessary computational design as well as small-scale trial works and past experiences should be considered.

Construction of Tunnel T-48 (Dharam-Quazigund section), based on NATM methodology: -
The construction of Tunnel T-48 in Dharam-Quazigund section is also based on NATM Methodology or also the Controlled Convergence Method. The concept of controlled deformation is applied. The choice of primary support measures to be applied over a certain stretch of the tunnel is continuously verified by means of convergence measurements. In case of signs of excessive or uncontrollable deformation, the support measures are adjusted, improving them or changing the section type to a heavier one. In the contrary case, for nearly no deformation at all and favourable rock mass conditions, the primary support can be enlightened or the section type can be changed to a lighter one.

The basic design methodology adopted for Tunnel T-48 can be summarised as below:
The design methodology for the tunnel T-48 is also based on Risk-Reduction process :-

Tunnel excavation and installation of Primary Support:-

The excavation cycle, for Drill & Blast method employed in T-48, can be subdivided as follows:
1. Setting / Marking at the face
2. Drilling
3. Charging
4. Blasting
5. Mucking
6. Scaling
7. Application of 1st layer of shotcrete (if required by the local rock mass conditions)
8. Installation of wire-mesh
9. Installation of steel ribs / lattice girders (if required by the local rock mass conditions)
10. Installation of rock bolts
11. Application of 2nd layer of shotcrete
12. Application of 3rd layer of shotcrete (if required by the total thickness of the shotcrete)
13. Once the excavation is ahead and no longer perturbed by further activities on the profile, the smoothening layer can be applied for creating the due background for the water-tightening membrane.

Final Lining:-

Tunnel lining shall be executed by means of movable formworks, for the crown and sidewall portion of the tunnel lining, which normally runs of steel rails installed on previously casted concrete shoulders.

Before start of the actual casting activities, some preparation work will be required. Specifically:
1. Removal of mucking material from invert (applicable when concrete invert is required in the particular stretch of the tunnel)
2. Application of smoothening layer of shotcrete
3. Waterproofing system installation, including longitudinal drainages at the sidewalls.

Subsequent activities will be:
a) Casting of invert and/or shoulders
b) Crown and sidewalls steel reinforcement installation (where applicable)
c) Casting of concrete.

Threshold radial convergence values adopted for T-48:-

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>Section Type</th>
<th>Alert (cm)</th>
<th>Alarm (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MT/Adits</td>
<td>A1/A2</td>
<td>Negligible</td>
<td>Negligible</td>
</tr>
<tr>
<td></td>
<td>B1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>3</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>C1</td>
<td>6</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>6</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>Negligible</td>
<td>Negligible</td>
</tr>
<tr>
<td></td>
<td>C2b</td>
<td>15</td>
<td>22</td>
</tr>
<tr>
<td>ET</td>
<td>A1/A2</td>
<td>Negligible</td>
<td>Negligible</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>3</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>Db</td>
<td>10</td>
<td>15</td>
</tr>
</tbody>
</table>

The whole stability and safety of the excavation is not deputed to the monitoring system alone but also to a suitable control of the support measures installed and a continuous and ubiquitous visual check of the conditions along the tunnel and the several ancillary pars of the projects.

The instruments produce a continuous flow and a large amount of data. These are managed by means of database software, specific for the project governance, named SISO. This software acts as the general database for the whole project, summoning the information regarding construction register, design documents, MOMs, Site Visit reports, etc. and, specifically for the present purpose, the module dedicated to the monitoring data management. The software also acts as an active control on the registered data. By the definition of limits for progressive states of alarm, it is possible to verify these ones and to communicate to all the selected responsible staff to monitor the situation "in real time" (as soon as all the data are available and the evaluation possible).
Monitoring for the Main and Escape Tunnel and for the Adits of T-48 shall require the following instruments:
- Optical targets Bi-reflex type
- Multipoint Extensometers
- Pressure Cells
- Strain meters

The measurement instruments underground are organised according to three different measurement sections. The lighter measurement section is provided by only optical targets while the complete one includes radial pressure cells, multipoint extensometers and strain gauges in the final lining. The frequency of measurement will be adjusted to the advance and the expected stabilisation of the measured values, so it will be progressively reduced. In case long term measurements show renovated activation of the measured value, the first action will be the reinstatement of a daily frequency. Adjustment and optimisation during construction remain possible.

In case of particular cross sections, the targets will be fixed on the final lining and the measurement continued once a month.

Monitoring of the Portals shall require the following instruments:
- Vertical Inclinometers
- Piezometers
- Load cells for Rock bolts
- Multipoint Extensometers
- Optical Targets prism type.

Procedure for Support Measures Selection and Optimisation

Process: -
The process for selecting the support measures begins with the design, selecting the hazard conditions to be expected during the excavation. During construction, the process is continued and starts with the evaluation of the rock mass conditions at the excavation face and considers the experience of the already excavated sections. The general process is subdivided in four parts:

- The face mapping allows for the initial definition of the rock mass type. In the present design for T-48, following Rock mass Types (RT) have been identified:
  - RT1: quartzitic phyllite and phyllitic quartzite
  - RT2: phyllite and carbonaceous phyllite with shear occurrence along the foliation
  - RT3: sheared zone and highly fractured rock.
  The analyses and computations in the design stage defined the suitable selection of the support measures in the several analysed conditions. These ones were differentiated on the base of the GSI (Geological Strength Index) according to Hoek & Brown and Hoek & Marinos, generating the Rock mass Behaviour Types (RBT). During face mapping, the RBT are defined with the help of RMR and GSI evaluation for focusing the RBT.

- The support measures are summoned into several different combinations, named Section Types, which are the result of the analysis of the local condition at the face. These are selected according to the relevant RBT.

- The monitoring system allows for the verification of the suitability of this selection in terms of actual equilibrium or gives indications for the adjustment of the support measures at the very place and for future similar conditions.

The selection of support measures as per process identified above can be best understood with the help of one such example/table as given below:

<table>
<thead>
<tr>
<th>MT and Adits</th>
<th>RT2</th>
<th>Behaviour Types RBT (Fazetti)</th>
<th>&lt; 400</th>
<th>400-600</th>
<th>600-800</th>
<th>800-1000</th>
<th>&gt; 1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>RT1</td>
<td>1-2</td>
<td>A1</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT2</td>
<td>1-2</td>
<td>A2</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT3</td>
<td>1-2</td>
<td>B1</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT4</td>
<td>1-2</td>
<td>B2</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT5</td>
<td>1-2</td>
<td>C1</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT6</td>
<td>1-2</td>
<td>C2</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT7</td>
<td>1-2</td>
<td>C3</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT8</td>
<td>1-2</td>
<td>C4</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT9</td>
<td>1-2</td>
<td>C5</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
<tr>
<td>RT10</td>
<td>1-2</td>
<td>C6</td>
<td>G1</td>
<td>G2</td>
<td>G3</td>
<td>G4</td>
<td>5</td>
</tr>
</tbody>
</table>

Adjustments according to the monitoring results

Along the tunnel the monitoring system allows for controlling the efficiency of the support measures. The alert and alarm levels, defining the situations when it will be required an active intervention for adjusting the sequence or the support measures.

- In case the monitoring confirms the respect of the alert level, nothing will be changed in the support measures definition and distribution.
- In case the measured deformations are clearly lower than the alert for the corresponding conditions, the support measures could be reduced.
- In case the measured deformations are higher than the alert for the corresponding conditions, the tendency has to be verified.
  - In case the tendency is to a clear steady state, the alert and corresponding alarm level can be revised if the final level is compatible with the geometry and the exigency for the realisation of the future final lining.
  - In same case but incompatible geometry exigency for the final lining, the support measures has to be increased in order to reduce the deformation or the excavation pay line should be revised in order to assure the required geometry exigency.
  - In case the tendency is not to a steady state, emergency measures for improving the support have to be implemented at the place and the support measures have to be revised improving them for future application in analogue geological and geotechnical conditions.
- In case the support measures have been increased after an incompatible deformation, the residual thickness of the final lining has to be verified and remedial measures have to be implemented.
All the above discussions clearly reflect the **advantages** of following NATM approach and can briefly be listed as follows:

1. Flexibility to adopt different excavation geometries and very large cross sections.
2. Flexibility to install additional support measures, rock bolts, dowels, steel ribs if required.
3. Flexibility to monitor deformation and stress redistribution so that necessary precautions can be taken.
4. Less overall support cost by ensuring that support is sufficient for the loadings and ground conditions without being excessive.
5. Providing a good contact surface between support and ground by using shotcrete.
6. Easy to install primary support, i.e. shotcrete.
7. Flexibility to use in various ground conditions.

# ELECTRIFICATION ON UDHAMPUR - KATRA

Electrification work on section Udhampur – Katra has been taken up by RE unit at Ambala for 25 RKM / 36 TKM section at an anticipated of Rs 31.19 Cr. The work is apart of CAO USBRL estimate.

The electrification work on the section has challenge of construction of OHE on bridges and tunnels which happen to be over 45% of the section length. At present the works is in progress between Tunnel number 24 and 25 near Chakrakhwal station.

The work on Udhampur – Katra has been planned and is being aligned to be completed more or less in sync with the opening of Udhampur – Katra section of the Indian Railways. The contract to this extent was awarded in Nov 2013 and the work physically started in Jan 2014. So far the work on 6 RKM has been completed. Such a section when energized on 25 kV shall be a unique section for Indian Railway where in one of the longest tunnels 3.12kms shall get energized on 25 kV.

Broadly the progress of the works is as below:

<table>
<thead>
<tr>
<th>Unit</th>
<th>Total</th>
<th>Cumm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation</td>
<td>No’s 592</td>
<td>115</td>
</tr>
<tr>
<td>Mast Err</td>
<td>No’s 468</td>
<td>76</td>
</tr>
<tr>
<td>BOOM</td>
<td>No’s 85</td>
<td>18</td>
</tr>
<tr>
<td>Brackets</td>
<td>No’s 1150</td>
<td>65</td>
</tr>
<tr>
<td>Wiring</td>
<td>Shot 120</td>
<td>8</td>
</tr>
</tbody>
</table>

**Electric Traction an efficient transportation system for sustainable development having**:

1. **Economy in fuel consumption:**
   As against a rupee invested in diesel traction fuel, the electric traction costs around 56 paisa. This helps in improving operating ratio of Indian Railways thereby resulting in lower per unit cost of transportation for Passengers and Freight traffic.

2. **Reduced travel time**
   Electric traction enables haulage of heavier freight and longer passenger trains at higher speed. It enables faster and cleaner intercity service as also helps I.R. to generate additional line capacity.

3. **Environment friendly**
   Electric Traction is environmental friendly green initiative without smoke resulting in cleaner and pollution free environment.

4. **Capable of regeneration**
   The system supports regenerative braking, thus returning some energy from the train to the system instead of wasting in friction brakes, thereby improving the system efficiency further.

5. **Sustainable development**
   Electric Traction is right step towards sustainable development. It creates infrastructure to facilitate running of trains on Electric Traction without differentiating the source of power generation which can be renewable or non-renewable source of energy.

It reduces nation’s dependence on imported petroleum based energy and enhances energy security.

By:-
Deepak Gangu
Chief Project Manager,
Railway Electrification,
Ambala
Overview of Sumber – Arpinchila Section of USBRL Project

1.0 Introduction:

Sumber – Arpinchila section is an approx. 15 km sub-section of the alignment of Prestigious Udhampur – Srinagar – Baramulla Rail Link Project (USBRL Project) which runs from km 110 to Km 125. The project area is located at the vicinity of the Pir Panjal Range of the lower Himalayas. The morphology of the project section area displays deeply incised, narrow, V-shaped valleys with steep flanks, due to the significant uplift rates of the Himalaya range within the most recent geological periods and the action of rivers. Mountain peaks are at an elevation of approx. 4000 m in this section.

The dominant valleys in this area of project are the Chenab River Valley and its tributaries the Bishlari Valley and the Mohu Mangat Valley. The tunnel portals and adits itself are accessible via the tributaries Sumber Nala, Urnihal Valley, Bhata Valley, Badarkot Valley, Higni Nala, Kundan Nala and Koda Nala. The terrain in the project area is mountainous with V-shaped valleys, deeply incised since the last glaciation. The present state of the erosional retreat of the river beds since the last glacial maximum leads to the important fact that the Sumber Station at the southern end of the project area towers about 1000 m above the bed of the main valley of the Chenab River, whereas the Arpinchala Station is located just 30 m above the Mohu Mangat River. This has severe implications on the accessibility of the tunnel portals and adits. The project area is located within the tectonically active area of mountain-building processes in the High Himalayas and falls in Seismic Zone V. The presence of a major fault in the area indicates regular intense earthquakes in the past and also probably continuing in the future.

The key data of the optimized alignment in this section of the project is as given below:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alignment start / elevation</td>
<td>km 110.235 / 1415.54 m</td>
</tr>
<tr>
<td>Alignment end / elevation</td>
<td>km 125.100 (match line with subsequent lot designated as km 125.000) / 1559.79 m</td>
</tr>
<tr>
<td>Total length</td>
<td>14.790 km</td>
</tr>
<tr>
<td>Difference in elevation</td>
<td>144.25 m</td>
</tr>
</tbody>
</table>

By:-
Bajrang Goyal
Executive Engineer/PM Cell/USBRL
Northern Railway, Kashmere Gate, Delhi
The structures along the optimized alignment are shown in the schematic map below and are also listed in the table-

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>START (KM)</th>
<th>END (KM)</th>
<th>STRUCTURE LENGTH (M)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge BR-2</td>
<td>110.263</td>
<td>110.704</td>
<td>441</td>
</tr>
<tr>
<td>Sumber Yard</td>
<td>110.282</td>
<td>111.482</td>
<td>1200</td>
</tr>
<tr>
<td>Bridge BR-3</td>
<td>111.097</td>
<td>111.482</td>
<td>341</td>
</tr>
<tr>
<td>Tunnel T-49</td>
<td>111.500</td>
<td>124.258</td>
<td>12758</td>
</tr>
<tr>
<td>Bridge BR-4</td>
<td>124.266</td>
<td>124.301</td>
<td>35</td>
</tr>
<tr>
<td>Tunnel T-50</td>
<td>124.310</td>
<td>124.565</td>
<td>255</td>
</tr>
<tr>
<td>Bridge BR-5</td>
<td>124.637</td>
<td>124.703</td>
<td>145</td>
</tr>
<tr>
<td>Arpinchila Yard</td>
<td>123.926</td>
<td>125.126</td>
<td>1200</td>
</tr>
</tbody>
</table>

2.3 Sumber Station and Yard
The Sumber station area is located just 50 m above the foot of a relatively long hill slope with approx. 30-35° steepness. Above the station area the slope extends approx. 100 m in height before its steepness reduces. The slope is partially covered with deposits of rockfall events of small to medium intensity. The Sumber Yard will contain two tracks and will have an approximate length of 1200m as required by railway standards. Due to the length, it will extend over both Bridges 2 and 3. The yard will be located along a steep hillside and the main line of yard is located to the greatest possible extent on cut and/or on bridges. The fill will sit on competent ground after topsoil; agricultural terraces are removed and eventual seepages from the hillside are drained by permanent and reliable measures. The maximum depth of the cut is approx. 45 m, whilst the maximum vertical height of the fill for any point on the fill does not exceed 20 m. The station buildings are located on the valley side of the alignment in cut. Platforms are located in cut on both sides. Tunnel excavation material, rather than slope excavation material is proposed for the construction of fill sections.

2.0 Details of Key Structures of Section –

2.1 Bridge 2: km 110+263 – 110+704
Bridge 2 spans the valley between Tunnel T-48 in the South and Sumber Station in the North.

2.1.1 Bridge 2 Design Considerations

<table>
<thead>
<tr>
<th>DESIGN CONSIDERATION</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design speed</td>
<td>100 km /h</td>
</tr>
<tr>
<td>No. of tracks</td>
<td>2</td>
</tr>
<tr>
<td>Distance between tracks</td>
<td>5.40 m</td>
</tr>
<tr>
<td>Load Model</td>
<td>IRIS MBG 1987</td>
</tr>
<tr>
<td>Layout straight/curved:</td>
<td>straight</td>
</tr>
</tbody>
</table>

2.2.2 Bridge 3 Construction Parameters

<table>
<thead>
<tr>
<th>Construction Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Length</td>
<td>Approx. 341m</td>
</tr>
<tr>
<td>Bridge Width</td>
<td>Approx. 16m</td>
</tr>
<tr>
<td>No. of spans</td>
<td>14x24.4</td>
</tr>
<tr>
<td>Structural systems of superstructures</td>
<td>Open web girder</td>
</tr>
<tr>
<td>Structural systems of substructures</td>
<td>Piers in solid section, abutments with sing walls</td>
</tr>
<tr>
<td>Max. height of pier</td>
<td>Approx. 19m</td>
</tr>
<tr>
<td>Max. height of abutment</td>
<td>Approx. 4.5m</td>
</tr>
<tr>
<td>Material of substructure</td>
<td>Reinforced concrete: M30 (unless a higher grade of concrete is required according to detailed design, calculations, structural analysis or exposition class conditions)</td>
</tr>
<tr>
<td>Foundation of Pillars</td>
<td>Heavy foundation with scour protection</td>
</tr>
<tr>
<td>Foundation of abutments</td>
<td>Foundation system based on geotechnical results</td>
</tr>
</tbody>
</table>

2.4 Tunnel T-49
T-49 will starts from Sumber station and it will ends at bridge no 4. A part of Arpinchila station will also run through this tunnel. The total length of tunnel is 12.75 KM which makes it longest tunnel of the project.
A parallel escape tunnel is provided which has a separate portal in the Sumber area, as well as the access to the surface at Hingini. The portal of the escape tunnel at Sumber shall be located approx. 120 m downstream of the main tunnel. Cross-passages are provided between the two tubes. Every third cross passage has an enlarged (drivable) profile. Tunnel T-49 has 2 additional approaches beside its portals, namely Kundan Nala Adit and Hingini Nala adit. Due to additional escape paths through these adits, the escape tunnel ends at hingini adit.

2.4.1 Details of salient Features of Tunnel T-49

<table>
<thead>
<tr>
<th>Component</th>
<th>Chainage (Km)</th>
<th>Elevation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-49 Portal P-1</td>
<td>111.500</td>
<td>1417.24</td>
</tr>
<tr>
<td>T-49 P-1 Escape</td>
<td>111.500</td>
<td>1418.60</td>
</tr>
<tr>
<td>Hingini Adit</td>
<td>120.619</td>
<td>1512</td>
</tr>
<tr>
<td>Kundan Nala Adit</td>
<td>122.794</td>
<td>1526.5</td>
</tr>
<tr>
<td>T-49 Portal P-2</td>
<td>124.258</td>
<td>1557.84</td>
</tr>
</tbody>
</table>

The form of main tunnel is horseshoe shape, with or without a rounded invert as determined by the encountered geological conditions. The size is determined by the clearance profile of the train and the necessity of accommodating a 1.2 m wide walkway on both sides of the track.

The key components of the main tunnel cross-section are:
- Sprayed concrete outer-lining
- Waterproofing membrane
- Concrete inner-lining
- Longitudinal drainage on both sides of the tunnel
- Track drainage
- Walkways on both sides of the tunnel
- Cable ducts underneath walkways
- Provision of water for fire-fighting

2.5 Bridge 4: km 124+266 – 124+301

2.5.1 Bridge 4 Design Considerations

2.5.2 Bridge 4 Construction Parameters

<table>
<thead>
<tr>
<th>CONSTRUCTION PARAMETERS</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge length:</td>
<td>Approx. 35 m</td>
</tr>
<tr>
<td>Bridge width:</td>
<td>Approx. 15.0 m</td>
</tr>
<tr>
<td>No. of spans:</td>
<td>1</td>
</tr>
<tr>
<td>Length of spans:</td>
<td>Approx. 35 m</td>
</tr>
<tr>
<td>Structural systems of superstructure:</td>
<td>Hollow box section with 11.40 m clear width and clear height 7.00 m (from TOL) With up side clamping plate system in reinforced concrete on damping layer</td>
</tr>
<tr>
<td>Material of superstructure:</td>
<td>Reinforced concrete M40 (unless a higher grade of concrete is required according to detailed design, calculations structural analysis or excavation class conditions)</td>
</tr>
<tr>
<td>Bearing concept:</td>
<td>One side horizontal tie, one side longitudinal movable</td>
</tr>
<tr>
<td>Type of bearings:</td>
<td>PGT Bearings, steel bearings with concrete lappets</td>
</tr>
<tr>
<td>Structural systems of substructures:</td>
<td>abutments with wing walls, integrated in tunnel-portal</td>
</tr>
<tr>
<td>Max height of pier:</td>
<td>–</td>
</tr>
<tr>
<td>Max height of abutment:</td>
<td>Approx. 10 m</td>
</tr>
<tr>
<td>Material of abutments:</td>
<td>Reinforced concrete M30 (unless a higher grade of concrete is required according to detailed design, calculations structural analysis or excavation class conditions)</td>
</tr>
<tr>
<td>Foundation of Piers:</td>
<td>Foundation system based on geotechnical results, integrated in tunnel-system</td>
</tr>
<tr>
<td>Foundation of abutments:</td>
<td>Foundation system based on geotechnical results</td>
</tr>
</tbody>
</table>

2.6 Tunnel T-50

T-50 is a 255m long double-track cross-section tunnel with centreline distances of 5.30 m due to Arpinchala Station extending into the tunnel. However, the platforms do not extend into the tunnel and the station building is located outside of the tunnel. The station continues onto the Rockfall Protection Bridge no 4 and into T-50. Due to the short length of T-50 a safety tunnel is not required.

2.7 Bridge 5: km 124.6+37.325 – 124.7+82.675

2.7.1 Bridge 5 Design Considerations

2.7.2 Bridge 5 Construction Parameters

<table>
<thead>
<tr>
<th>Construction parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge length:</td>
<td>Approx. 145m</td>
</tr>
<tr>
<td>Bridge width:</td>
<td>Approx. 16.90m</td>
</tr>
<tr>
<td>Railway deck width:</td>
<td>Approx. 11.25m</td>
</tr>
<tr>
<td>Footway platform width:</td>
<td>Approx. 5.65m</td>
</tr>
<tr>
<td>No. of spans:</td>
<td>6 x 24.2</td>
</tr>
<tr>
<td>Structural systems of superstructures:</td>
<td>PSC Girder</td>
</tr>
</tbody>
</table>
2.8 Arpinchila Station and yard-

The Arpinchila Station is located in an area with generally smooth hill slopes and an elevated old riverbank. This riverbank is in the lee section of a river bend and therefore quite well protected from erosion. Above this riverbank a heavily vegetated hill slope with inclinations of 30° to 45° continues. Due to the required length of the yards, the Arpinchala Yard extends through the tunnel T-49, tunnel T-50 and over bridge no 4 and 5. The yard is located partly in cut and partly on fill. Suitable tunnel excavation material for the purpose of accommodating the facilities of the station at rail level will be used. The fill will sit on competent ground after topsoil and agricultural terraces are removed and eventual seepage from the hillside is drained by permanent and reliable measures.
Rehabilitation of Tunnel T1

1.0 Introduction

The Udhampur-Katra section is the most prestigious new single line Rail link between two cities Udhampur and Katra. This single track rail link of length 24.80 km comprises seven tunnels (Tunnel No. 1 to 7) from Udhampur to Katra. This Udhampur -Katra rail link is the part of the Udhampur-Srinagar-Barramulla-Rail Link (USBRL), the most important National Project of the Indian Railway connecting to Kashmir valley. The present work focuses on the rehabilitation of tunnel no.1, one of the longest tunnels of the Udhampur-Katra section. The length of tunnel was 3.1 km and the alignment goes almost from east (Udhampur End) to west (Katra End) with the constant gradient of 1 in 100 rising towards Katra. The original cross section of the tunnel constructed has D-shape profile consisting of a semi circular heading with an inner diameter of 5.2 m and rectangular benching portion with a height of 5.5 m.

The tunnel was constructed earlier in 2001-05 by the conventional tunnelling method in heading benching with vertical support system of steel ribs and precast RCC laggings. The contractor faced massive problems like bottom heaving and lateral deformations during construction of tunnel. However, it could not withstand the developing stresses and subsequent huge bottom heaving; lateral deformation and partial collapse of the tunnel have been taken place after construction.

Northern Railway appointed consultant M/S Geoconsult-Rites JV in 2007 to analyse the situation of tunnel T-1 and to develop a design for the remedial measures and preparation of tender document regarding the required remedial measures. Accordingly, it was decided to construct a bypass tunnel (realigned tunnel) to circumvent the badly damaged/collapsed section of the existing tunnel. The construction of 1.8 kms long tunnel on the realigned path in the existing tunnel of length 3.1 km and reprofiling/repair works in the balance length of the existing tunnel was awarded to M/S Tantia-CCIL JV in January 2010. The new tunnel on the realigned path was constructed with the New Australian Tunnelling Method (NATM). During the construction of the tunnel on the realigned path the massive problems of lateral deformation face but the Northern Railway Engineers in consultation of the consultant became successful to control the deformations. The final concrete lining of the tunnel completed in May 2013 and thereafter the repair work in the existing tunnel and lying of Blast Less Track structure was done in the complete length of the tunnel.

2.0 Geological & Geotechnical investigations

Geological & Geotechnical investigation is the important part for the construction of Railway track in the hilly terrain because the Geology affects the alignment of the project. Geology and hydrogeology in tunnelling works plays very important role in planning, design and construction of tunnel. Geology not only influence the cost of the tunnelling project but it also affects the completion period, behaviour of the structure and long term durability.

The Udhampur region falls in the Lesser Himalayan belt of the Himalayan organic system. In the Lesser Himalayan lithologic succession comprises Tertiary-Quarternary sediments, which has been further classified into Palaeogene (Murree Group) and Neogene (Shiwalik Group) deposits separated by Main Boundary Thrust (MBT). The Murree group composed of brown to reddish fine to medium grained sandstone, siltstone, claystone and shale alternations deposited in shallow marine environment and shiwalik group contains fine to medium grained sandstone, siltstone, claystone, coarse to very coarse grained sandstone, conglomerate to boulder beds deposited in fluvial environment.

The lithology along the alignment of the Tunnel T-1 of the Udhampur-Katra section comprises dominantly rocks of Murree (Udhampur End) and Shiwalik (Katra End) groups with subordinate recent sediments. Two major shear zones along the tunnel are also present. The geotechnical parameter obtained along the path of the tunnel also referred that the tunnelling will be through “soft ground” the shiwalik rocks. Due to presence of swelling minerals and weak rock formation with high rock cover the alignment was encountered with the swelling and squeezing ground conditions.

The rock mass was directly investigated only in areas, where windows in the primary support were opened or on the other hand where no backfill has been installed. During the earlier construction of the tunnel rock and soil samples have been collected and stored. These samples have also been classified and shall be presented through the following pictures.

By:-
Nem Singh Baghel,
XEN/C/S&C-I/UHP
USBRL Project

Picture 2.1: Shiwalik Formation - mainly gravels, boulder and pebbles in a sandy matrix, well rounded and poorly graded

Picture 2.2: Siltstone at Chainage-2240
3. Tunnel Construction and Design:

3.1 Status of Existing Tunnel

The construction and design of the existing Tunnel T-1 has been done according to the standard design for single track railway tunnels. The regular section of the tunnel consist of vertical sidewalls and a circular roof section. The invert was also designed and constructed straight. The excavation has been performed as a top heading & bench/invert excavation, whereas the circular roof section constitutes the top heading and the area with the straight sidewalls forms the bench/invert section of the profile. The support system of the existing tunnel consist of steel ribs type IHSB 150 and 200, which were installed at every round. After excavation of a top heading round, the corresponding steel rib was installed and concrete lagging were placed to serve as shutter. Once the lagging were installed, concrete was filled behind these prefabricated elements to serve as primary support in combination with the steel ribs. A horizontal steel rib was place at the heading footing to serve as connection between top heading and benching. The invert consists of horizontal steel ribs which were placed between the steel ribs of the vertical sidewalls, which were embedded and covered by approx. 10 cm in concrete of grade M15.

After finishing the primary support a steel plate frame was installed to cast the inner lining with the thickness of 15 cm and grade of lining concrete was M20. The casting length of the segment was 6m. Due to the thickness, the lack of reinforcement and the shape of the cross section, the inner lining structural importance were negligible.
3.2 Existing Tunnel Behaviour

There is no assumption of initial rock mass pressure, water pressure, swelling or squeezing pressure available with regards to the design. Both primary support and inner lining have suffered excessive deformations which cannot be verified or estimated due to the lack of material behaviour values of both rock mass and structure. The deformation monitoring in the existing tunnel was executed with hand held laser reflection meters. The closure was measured between targets consisting of steel bars installed in the lining (tape extensometer measurements, giving only relative values).

The collapse of the earlier constructed tunnel occurred between ch 4800 and 4840 at geological boundary between two geological units that was shivalik formation and recent sediments. The over burden above the tunnel crown at the collapsed section of the tunnel was approx. 40m. Out of this 40m over burden the upper layer with the thickness of approx 25m consists of overburden soil and debris. In the area of collapse, this layer of over burden soil and debris forms some kind of basin at on top of the shivalik formation. The collapse occurred approx 500 m inside of the tunnel from the katra end portal.

The heaving occurred in the existing tunnel and was measured from bottom to crown without fixed targets. On long stretches of the tunnel from chainage 3590 to 3970, chainage 4390 & chainage 4658 the bottom heaving took place even upto 100 cm.

The bottom ground lifted breaking the concrete invert and bending and some location even breaking of the invert steel struts took place. The connection between the invert strut and the wall structure was intact. No bending in the footer of the wall structure was visible.

3.3 Construction of New Tunnel on Realigned path

The design of the tunnel on the realigned path of Tunnel T-1 has been formulated upon the concept of New Austrian Tunnelling Method (NATM). The concept of NATM is based on the understanding of the behaviour of the ground as it reacts to the creation of an underground opening. NATM attempts to mobilize the self-supporting capability of the ground. Geological and geotechnical investigation has been undertaken by RITES (2008) to derive a ground response classification and subsequently assess tunnel support needs. With the data from the ground investigation, field mapping and estimation of overburden in combination with the laboratory test results of the rocks, the ground response to tunnelling has been assessed and based on that different excavation and support classes have been defined. The excavation round length depends on the class of rock and especially its behaviour during the time span of the excavation. Although, the Individual classes of rock contain clear specifications for excavation round length and initial support and pre-support measures to be installed and the sequence of excavation and support installation. However, Required Excavation & Support Sheets (RESS) have been used for initial or additional support or local support or pre-support measures to deal with local ground conditions. The excavation round length of 0.80 to 1.20 m was achieved in the top heading section of the tunnel. The forepoling was installed as support measures ahead of the face to improve the roof stability and reduce the probability of the ground failure and overbreak. Depending on the ground conditions two types of forepoling were used, one steel rods and the other self drilling anchors (SDA) with different overlapping length.

The shape of the tunnel cross section has been chosen to activate the self supporting arch in the surrounding ground for which cross section geometry is elliptical, consisting of compound curve in both arch and invert. The curvilinear (or elliptical) geometry will enable a smooth flow of stresses in the ground around the opening, minimizing loads acting on the tunnel linings. The excavation of the cross section of the tunnel has been subdivided into top-heading, benching and invert. The tunnel excavation causes disturbance of the initial state of stress in the ground. The extent of the stress disturbance depends mainly on ground conditions, size of the excavation and excavation...
round length. As the excavation of the tunnel advances the shotcrete hardens from an initially “green” shotcrete and becomes fully loaded at a distance of about one to two tunnel diameters from the face. Such sequencing combined with early support installation contributes to the development of the self-supporting capability of the ground. The excavation round length varies according to the rock-mass conditions and support classes. Due to the poor geological conditions and expected surprises, the excavation method adopted here is mechanical breaking instead of drilling and blasting.

The basic support system consists of dual lining comprises of an initial shotcrete lining and a final cast-in-place concrete lining, instead of traditional thick and stiff single lining. The dual lining method is based on the principle of controlled deformation for permitting partial stress relaxation. Soon after excavation, the ground structures remain in unsterilized state; the stabilization may only achieved by transforming the stresses caused by overburden load into controlled deformation and into the establishment of a new equilibrium stage. The final lining of the cast-in-place concrete was installed after complete stabilization.

3.4 Deformation

The squeezing and swelling in Murree rocks are highly predictable while tunnelling. Expected conditions have been encountered at the udhampur end. It has been observed that the primary shotcrete lining has been deformed significantly. Not only the cracks along the crown observed but noteworthy buckling of the lattice girders also occurred within few days of the installation of shotcrete lining. However, in the concept of NATM, controlled deformation is allowed. By deforming, it enables the inherent strength and self-supporting properties of the ground to be mobilized as well as to re-distribute stresses between the lining and ground. During deformation, stresses acting within the shotcrete...
lining are transferred into the surrounding ground. This process generates subgrade reaction of the ground that provides support for the lining.

The proper in-situ monitoring of geological conditions and reaction of the ground to the excavation and the support system during construction is an integral part of the NATM for verification of the design assumptions made regarding the interaction between the ground and initial support as a response to the excavation process. For this purpose, 3-D monitoring program have been laid out for the detail and systematic measurement of deflection of the primary shotcrete lining. The main focus of the 3-D monitoring was to monitor the deflection of the shotcrete lining and accordingly to access and install the additional support system to control the rate of deflection within the permissible limits. To monitor this deformation bireflex targets as Deformation Monitoring Points were installed in the tunnel roof and at selected points along the tunnel walls. The vertical, horizontal and longitudinal deformation components were measured with Total Station at 5 to 7 targets in a each section. The convergence action or the decrease of cross-sectional area was very prominent and maximum settlement has been recorded from the targets on the crown of the tunnel. At udhampur end, the vertical and horizontal displacements of the targets were measured to be usually around 50-150 mm, and at some places the displacement reached upto 200-300 mm (i.e.at the sheared zones). However, the shotcrete lining in Shivalik rock at Katra end does not have undergone much deformation. The amount of convergence at Katra end ranges between 40 and 50 mm.

4. Conclusion

The mudstone and the sheared rocks of the Murree group have undergone swelling and squeezing due to the presence of the higher amount of clay minerals and larger overburden pressure on such weak rock mass respectively. The basic idea of the NATM is derived from the steady observation of the behaviour of the rock mass and deformation. NATM being the observation approach needs proper and thorough monitoring as well as quick documentation and communication. Hence unusual occurrence at the construction stage can easily be monitored and design can be modified accordingly.

Monitoring of the deformation on the primary shotcrete lining indicates significant convergence of the tunnel cross section and it also confirms that the deformation usually takes place in the initial stages of emplacement of the shotcrete lining. This point to the fact that due to excavation, the rock mass becomes disturbed and unstabilized. Therefore, the rock mass trends to come in equilibrium by deforming the shotcrete lining. Gradually the lining attends stability which suggest gaining of new equilibrium state by the surrounding ground. The ground then acts as self-sustaining load bearing structure. The another importance feature in the NATM is the adaption of dual lining system. The deformation of the primary shotcrete lining not becomes a matter concern, rather if follows the idea of the controlled deformation. Early ring closure in inevitable while tunnelling though a poor rock condition. Complete ring closure by benching and invert or partial closure by the temporary invert played important role in controlling the rate of deformation. Before final concrete lining whatever deformation the tunnel can have that already had. The final concrete lining of the new tunnel on realigned path was executed with the CIFA shutter formwork in the elliptical cross section. The existing tunnel at the both ends retained in the original D-shape profile.
ROCKBOLTS and DOWELS

1.0 INTRODUCTION:
Rockbolts and dowels have been used for many years for the support of underground excavations and a wide variety of bolt and dowel types have been developed to meet different needs which arise in mining and civil engineering. Rockbolts generally consist of plain steel rods with a mechanical or chemical anchor at one end and a face plate and nut at the other. They are always tensioned after installation. For short term applications the bolts are generally left ungrouted. For more permanent applications or in rock in which corrosive groundwater is present, the space between the bolt and the rock can be filled with cement or resin grout. Dowels or anchor bars generally consist of deformed steel bars which are grouted into the rock. Tensioning is not possible and the load in the dowels is generated by movements in the rock mass. In order to be effective, dowels have to be installed before significant movement in the rock mass has taken place. Figure 2.9 illustrates a number of typical rockbolt and dowel applications that can be used to control different types of failure that occur in rock masses around underground openings.

2.0 ORIGIN:
The origin of the rock bolt is unknown, but it is believed to have originated with mining companies during the late 1800s. The first documented use of rock bolts was at the Saint Joseph Lead Mine in the United States in the 1940s. Since that time, these bolts have gone on to worldwide acclaim and their use has developed into sophisticated techniques for tunnel building and more.

3.0 USES OF ROCKBOLTS/ANCHORS/DOWELS
3.1 Underground Excavations: The uses of Rockbolts in underground excavations are as under:
(i) To support discrete wedges or blocks or rocks that would otherwise be free to fall or slide;
(ii) To reinforce the crown or side walls of a tunnel;
(iii) In older designs rockbolts were used as part of temporary support, but more recently as part of the permanent support system.

Typical support schemes are shown in figures 2.1 to 2.4 and in detail in fig 2.5.

Fig. 2.1 Use of rock bolts to support wedges in underground excavation.

Fig. 2.2 Arrangement of rock bolts to support roof excavation.

Fig. 2.3 Rock bolting to generate “beam effect” in layered strata.

By:-
Sh. Vinod Kumar,
Dy. Chief Engineer/ Banihal USBRL Project
3.2 Rock excavations, slopes and faces:

(i) For highway works rockbolts are predominantly used to stabilize relatively small instabilities.

(ii) Rockbolts can give support to discrete unstable blocks bounded by discontinuities of various types.

(iii) Where there is widespread instability, a gridage of rockbolts is used to improve the overall integrity and stability of rock mass, sometimes in combination with netting where rockbolt/dowels have been used to hold rock fall protection (i.e. at crest and toe). Common situations are shown in fig 2.6.

(iv) For maintenance and improvements schemes i.e rock slope protection. A typical scheme for highway cutting is shown in fig 2.7. As shown here an integrated approach is commonly used in such works combining ground anchorages and rock bolts with small scale buttressing and dental concrete.

3.3 Other applications:

(i) Rockbolts have been used to restrain light structures such as gantry signs which are subjected to overturning or tension forces.

(ii) Rockbolts have also been used to strengthen or repair earth retaining walls see for example fig 2.8.
3. Use of tensioned GFRC bars for masonry repairs

4.0 MECHANISM OF ROCKBOLTS

The mechanism of working of Rock bolts is explained with the help of a model which is based on a version of Tom Lang’s model constructed by the U.S. Army Corps of Engineers Waterways Experiment Station in Vicksburg, Mississippi. The model consists of rock pieces/gravels of average piece diameter as ‘a’. These rock pieces are then bound together with the help of miniature rockbolts length ‘L’ with spacing ‘s’ and placed on a temporary base. The temporary base is then removed. This model is explained.

<table>
<thead>
<tr>
<th>Low stress levels</th>
<th>High stress levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive rock</td>
<td>Massive rock subjected to low in situ stress levels. No permanent support. Light support may be required for construction safety.</td>
</tr>
<tr>
<td>Jointed rock</td>
<td>Massive rock with relatively few discontinuities subjected to high in situ stress conditions. Epic bolts located to prevent failure of individual blocks and wedges. Bolts must be braced.</td>
</tr>
<tr>
<td>Heavily jointed rock</td>
<td>Heavily jointed rock subjected to low in situ stress conditions. Light pattern bolts with mesh/steel plate/steel plate with sliding joints may be required. Invert arches or concrete floor slabs may be required to control floor heave.</td>
</tr>
<tr>
<td></td>
<td>Massive rock with relatively few discontinuities subjected to high in situ stress conditions. Heavy rockbolts or dowels, instead of cross rock structure, with mesh or steel fibre reinforced concrete on roof and sidewall.</td>
</tr>
</tbody>
</table>

Figure 2.8: Use of tensioned GFRC bars for masonry repairs

Figure 2.9: Typical rockbolt and dowel applications to control different types of rock mass failure during tunnel driving.

Photo 1: The empty frame of the rockbolt plate model

Photo 2: Miniature rockbolts ready for installation

Photo 3: Uniformly sized clean gravel for the plate

Photo 4: Attachment of the temporary base to the model

Photo 5: Positioning the rockbolts in holes drilled into the temporary

Photo 6: Placing the gravel in the frame
A zone of compression is induced in the region shown in red and this will provide effective reinforcement to the rock mass when the rockbolt spacing ‘s’ is less than 3 times the average rock piece diameter ‘a’. The rockbolt length ‘L’ should be approximately ‘2s’. Note there is no support between the washers (unless mesh or shotcrete is applied) and the rock pieces will fall out of these zones on the underside of the beam.

5.0 TYPES OF ROCKBOLTS/DOWELS

Rockbolts and dowels can be classified into following three broad categories, according to how they are anchored into the rock mass: -

(i) **Mechanical Anchored**: - In Mechanical Anchored Rockbolts, the load is transferred to the rock mass through some form of mechanical device.

(ii) **Cement Grouted**: - In Cement grouted rockbolts/dowels, a cementitious grout is used to anchor the bolt into the rock.

(iii) **Resin Grouted**: - Typically these employ polyester resins to anchor the bolt into the rock, but epoxy resins have also been used. In most cases, for convenience, the grout is supplied in pre packed sausage like capsules which contain the resin and hardener in separate compartments. The action of rotating the tendon during installation ruptures the capsules and mixes their contents. Resinous grouts have been used in pumped or poured forms but these are less common.

5.1 Mechanically Anchored Rock bolts: -Mechanically Anchored rock bolts are the most common type of rock bolts. These long rods actually expand within the hole when they are installed/ twisted in order to activate a mechanism. This mechanism increases its size, assuring a snug fit that can be counted on to remove pressure from the surface of the rock. The various types of Mechanical Anchored Rock bolts/ Anchors are as under: -

(i) Expansion Shell Anchors;
(ii) Full length expansion anchors;
(iii) Split set stabilizer;
(iv) Swellex Frictional anchor;
(v) Slotted bolt and wedge.

5.1.1 Expansion Shell Anchors: - Expansion Shell Rockbolts are examples of Mechanically Anchored Rockbolts. Expansion shell rockbolt anchors come in a wide variety of styles but the basic principle of operation is the same in all of these anchors. As shown in Figure 4, the components of a typical expansion shell anchor are a tapered cone with an internal thread and a pair of wedges held in place by a bail. The cone is screwed onto the threaded end of the bolt and the entire assembly is inserted into the hole that has been drilled to receive the rockbolt. The length of the hole should be at least 100 mm longer than the bolt otherwise the bail will be dislodged by being forced against the end of the hole. Once the assembly is in place, a sharp pull on the end of the bolt will seat the anchor. Tightening the bolt will force the cone further into the wedge thereby increasing the anchor force. This type of bolt can be tensioned immediately after installation and grouted at later stage when short term movements have been ceased.
Figure 4: Components of a mechanically anchored rockbolt with provision for grouting

These expansion shell anchors work well in hard rock but they are not very effective in closely jointed rocks and in soft rocks, because of deformation and failure of the rock in contact with the wedge grips. Mechanically anchored rockbolts have a tendency to work loose when subjected to vibrations due to nearby blasting or when anchored in weak rock. In such rocks, the use of cement grouted/resin cartridge rockbolts/anchors is recommended. A range of expansion shell systems is shown in fig 5.

Often such bolts are used as permanent support and in such cases secondary grouting would be employed to provide the tendon with some protection against corrosion. Various means have been developed to achieve this, but typically a rubber bung is inserted in the collar of the drill hole to centralize the hole and act as a seal against the grout leakage. Alternatively, a rapid set mortar can be used to seal the collar and often in such cases the mortar is extended to bed down the face plate. Grout can be injected into the drill hole by various arrangements. For upward facing holes, the grout is injected into the collar end and the return pipe is extended to the base of the hole; grout injection is stopped when all the air has been displaced and grout flows from the return tube. For downward facing holes, grout is pumped to its base through a full length injection pipe and exits at its collar.

5.1.2 Full Length Expansion Anchors: - A form of this type of anchor, developed by Worley Co. of Philadelphia, is shown in Figure 6. As the nut is tightened against the washer, the ramps along the tendon expand the anchor against the sides of the drill hole. Undoing the nut and hammering on the end of the bolt reverses the process and allows the anchor to be loosened and removed; provided that it is not too deformed or corroded the device may be reused. Because it cannot be grouted, and hence protected from deterioration, this device is only suitable for short-term applications. Furthermore the device cannot be tensioned so that load is transferred to its distal end. It is therefore usually installed as soon as possible after the excavation of the drill hole, i.e. before any movement of rock has occurred due to relaxation.

Pattin Bolt
Goldenberg Bolt
Bail Bolt

(a) Standard Types

(b) Shells for large diameter drillholes or for use in soft rock

Figure 5: Details of typical expansion shell rock bolts

5.1.3 Split set stabilizer: - Split-set friction rock bolts are placed inside predrilled holes. They are made of collapsed steel tubing, which is placed within the hole and twisted. This twisting causes the tubing to expand, which secures the bolt to the hole’s wall. These bolts are simple to install, but they lack tension and the bolts can’t be anymore than 3 meters long.

Split Set stabilizers were originally developed by Scott (1976, 1983) and are manufactured and distributed by Ingersoll-Rand. The system, illustrated in Figure 7, consists of a slotted high strength steel tube and a face plate. It is installed by pushing it into a slightly undersized hole and the radial spring force generated, by the compression of the C shaped tube, provides the frictional anchorage along the entire length of the hole.

These are mainly used for mining applications and seldom used for civil engineering applications.

Pattin Bolt
Goldenberg Bolt
Bail Bolt

(a) Standard Types

(b) Shells for large diameter drillholes or for use in soft rock

Figure 6: Details of “Worley” Mechanical Rock Bolt (Mine Roof Systems)

Figure 7: Split Set stabilizer; Ingersoll-Rand Co. Ltd.
5.1.4 Swellex Frictional anchor: - Swellex friction rock bolts are similar to split-set friction bolts. They are also made of collapsed tubes except they expand through the use of water pressure. They are extremely simple to install. The main problem with them is their lack of durability.

Developed and marketed by Atlas Copco Construction and Mining Ltd. in Sweden, the ‘Swellex’ system is illustrated in Figure 8.1. The dowel, which may be up to 12 m long, consists of a 42 mm diameter tube which is folded during manufacture to create a 25 to 28 mm diameter unit which can be inserted into a 32 to 39 mm diameter hole. No pushing force is required during insertion and the dowel is activated by injection of high pressure water (approximately 30 MPa or 4,300 psi) which inflates the folded tube into intimate contact with the walls of the borehole.

Corrosion of Swellex dowels is a matter of concern since the outer surface of the tube is in direct contact with the rock. Atlas Copco has worked with coating manufacturers to overcome this problem and claim to have developed effective corrosion resistant coatings.

Speed of installation is the principal advantage of the Swellex system as compared with conventional rockbolts and cement grouted dowels. In fact, the total installation cost of Swellex dowels or Split Set stabilizers tends to be less than that of alternative reinforcement systems, when installation time is taken into account. Both systems are ideal for use with automated rockbolters.

Swelllex Rock bolts exists in two typical versions: -

(a) Swellex Premium Line (Pm): - It is a relatively stiff rock bolt for tunneling and mining in moderate stress conditions. The Premium line is a typical tunneling bolt, with a high yield load and good deformability. Pm rock bolts can also be used in mining when low to medium stress conditions require a stiff Swellex rock bolt with a high yield load.

(b) Swellex Manganese Line (Mn): - It is a highly deformable rock bolt when ground movement is expected. The Manganese line was developed to suit large stress changes occurring in some mining and tunneling projects. Made from a very specific steel type, Mn rock bolts undergo a heat treatment to improve their mechanical properties. Mn Swellex rock bolts have a unique yielding behavior and provide a high ultimate load and a large deformation capability.

Atlas Copco provides two versions of corrosion protected Swellex bolts: -

(a) Bitumen coated Swellex: - Bitumen coated Swellex is coated with a high build rubber bitumen coating to provide medium term corrosion protection against most corrosive conditions.

(b) Plastic coated Swellex: - Plastic coated Swellex is providing its long term corrosion protection through a thick plastic coating that is impervious to water and current. The impact resistant coating is very effective in extremely corrosive environments and its resistance and effectiveness is proven over years of application in highly acidic conditions.

These two products extend the normal life of a Swellex significantly.
Rockbolts and Dowels

Swellex Rock bolts are also available for some special applications as follows:

(a) **Swellex Pm24C – a connectable bolt:** The Swellex Pm24C offers the possibility of deeper reinforcement without grouting operation. It is a connectable rock bolt, which consists of individual sections which are joined together via "R" thread connections. The required bolt length is achieved by adding one or more sections. Swellex Pm24C bolts are highly competitive alternatives to short and medium length cable bolts.

(b) **Swellex Pm24H – a hanger:** The Swellex Pm24H provides a high anchorage capacity hanger with a flanged head with a female M36 thread. The rock bolt has a static axial load capacity of 200 kN. After the Pm24H bolt has been installed, a forged eye bolt with M36 thread is screwed on. Utilities can then be suspended directly from the eye bolt. As the Pm24H is not designed for eccentric loading, the design load should always be coaxial to the bolt profile.

5.2 Cement Grouted Rockbolts/Dowels:- When conditions are such that installation of support can be carried out very close to an advancing face, or in anticipation of stress changes that will occur at a later excavation stage, dowels can be used in place of rockbolts. The essential difference between these systems is that tensioned rockbolts apply a positive force to the rock, while dowels depend upon movement in the rock to activate the reinforcing action.

The simplest form of dowel in use today is the cement grouted dowel as illustrated in Figure 10. A thick grout (typically a 0.3 to 0.35 water/cement ratio grout) is pumped into the hole by inserting the grout tube to the end of the hole and slowly withdrawing the tube as the grout is pumped in. Provided that a sufficiently viscous grout is used, it will not run out of the hole. The dowel is pushed into the hole about half way and then given a slight bend before pushing it fully into the hole. This bend will serve to keep the dowel firmly lodged in the hole while the grout sets. Once the grout has set, a face plate and nut can be fitted onto the end of the dowel and pulled up tight. Placing this face plate is important since; if the dowel is called on to react to displacements in the rock mass, the rock close to the borehole collar will tend to pull away from the dowel unless restrained by a faceplate.

Early devices used in the mining industry were manufactured from timber, but these have been superseded by ones manufactured from fiberglass and from steel – details of these have been given by Hoek and Brown (1980).
5.3 **Resin Grouted Rockbolts:** Resin anchored rock bolts are sealed using a resin and a catalyst. A cartridge full of the resin is placed at the end of the hole, and the bolt is stuck in the hole after it. The rebar is then "drilled" through the hole, puncturing the cartridge and causing the resin to dry and seal the bolt in the hole. The resin is then released into the hole, and it slowly hardens and keeps the bolt in place. This type of rock bolt is very common because it is very simple to install.

A typical resin product is made up of two component cartridges containing a resin and a catalyst in separate compartments, as shown in Figure 11.1. The cartridges are pushed to the end of the drill hole ahead of the bolt rod that is then spun into the resin cartridges by the drill. The plastic sheath of the cartridges is broken and the resin and catalyst mixed by this spinning action. Setting of the resin occurs within a few minutes (depending upon the specifications of the resin mix) and a very strong anchor is created. This type of anchor will work in most rocks, including the weak shales and mudstones in which expansion shell anchors are not suitable.

For 'permanent' applications, consideration should be given to the use of fully resin-grouted rockbolts as illustrated in Figure 11.2. Resin grouting involves placing slow-setting resin cartridges behind the fast-setting anchor cartridges and spinning the bolt rod through them all to mix the resin and catalyst. Spinning the bolt rod through all of these cartridges initiates the chemical reaction in all of the resins but, because the slow-setting 'grout' cartridges are timed to set in up to 30 minutes, the bolt can be tensioned within two or three minutes of installation (after the fast anchor resin has set). This tension is then locked in by the later-setting grout cartridges and the resulting installation is a fully tensioned, fully grouted rockbolt.

Most resin/catalyst systems have a limited shelf life which, depending upon storage temperatures and conditions may be as short as six months. Purchase of the resin cartridges should be limited to the quantities to be used within the shelf life. Care should be taken to store the boxes under conditions that conform to the manufacturer's recommendations. In critical applications, it is good practice to test the activity of the resin by sacrificing one cartridge from each box, before the contents are used underground. This can be done by breaking the compartment separating the resin and catalyst by hand and, after mixing the components, measuring the set time to check whether this is within the manufacturer's specifications.

Breaking the plastic sheath of the cartridges and mixing the resins effectively can also present practical problems. Cutting the end of the bolt rod at an angle to form a sharp tapered point will help in this process, but the user should also be prepared to do some experimentation to achieve the best results. Note that the length of time or the number of rotations for spinning the resins is limited. Once the setting process has been initiated, the structure of the resin can be damaged and the overall installation weakened by additional spinning. Most manufacturers supply instructions on the number of rotations or the length of time for spinning.

In some weak argillaceous rocks, the drill hole surfaces become clay-coated during drilling. This causes slipping of the resin cartridges during rotation, resulting in incomplete mixing and an unsatisfactory bond. In highly fractured rock masses, the resin may seep into the surrounding rock before setting, leaving voids in the resin column surrounding the rockbolt. In both of these cases, the use of cement grouting rather than resin grouting may provide a more effective solution.

There is some uncertainty about the long-term corrosion protection offered by resin grouts and also about the reaction of some of these resins with aggressive groundwater. For temporary applications, these concerns are probably not an issue because of the limited design life for most rockbolt installations. However, where very long service life is required, current wisdom suggests that cement grouted bolts may provide better long term protection.

The great advantages of resin-based systems are that they are simple to use and they set relatively quickly which maintains a rapid cycle time in a construction sequence where the cost of the material may be a relatively inexpensive part of the process. Depending upon the quality of the rock, this type of bolt can mobilize high lock-off loads. With appropriate setting times, a one-shot installation can produce a fully grouted and tensioned rockbolt, and such bolts are widely used for permanent works.

6.0 **The Rock bolts can be classified into following two categories based on materials type:**

(i) **Steel Rock Bolts:** These are made up of Steel of different grades;

(ii) **Glass fibre Reinforced Composite (GFRC)/ Glass Reinforced Polymers.**
6.1 Steel Rock bolts: - These Rock bolts are made up of various grades of steel.

6.2 Glass Fibres Reinforced Composite Rockbolts: - Glass fibre reinforced composite materials (GFRC) are used for a range of structural applications, particularly where its non-corrodible property is valued. GFRC and GRP (glass reinforced plastic) rock bolts were originally developed for the coal mining industry to meet the need for a strong but temporary reinforcement to an advancing face or a sidewall, which could be subsequently excavated by tunnelling and cutting machines without damage to the cutting teeth.

A composite bar consists of thousands of continuous glass fibres laid parallel to one another and encased in a matrix of polyester or epoxy resin. Typically the composite is manufactured by a continuous pultrusion process which produces bars having diameters ranging from 1mm to in excess of 20mm. As an example, a 7.5mm diameter bar might contain about 64,000 individual fibres which have a mean diameter of 25 microns (i.e. 25x10^{-3} mm). The volume of fibre varies from between 45% to greater than 75% of the composite: the percentage varying according to the manufacturer and application.

In tunnelling, Fiber glass rock bolts (Figure 12) of similar ultimate load as steel bolts may be used for face bolting. Bolts may be either threaded rods or self drilling bolts. It is lighter in weight and is highly resistant to corrosion.

**Table 2:** - Technical details of GRP Bolts

<table>
<thead>
<tr>
<th>Outer diameter</th>
<th>16A-25</th>
<th>16A-29</th>
<th>K60-20</th>
<th>K60-22</th>
<th>K60-25</th>
<th>K60-27</th>
<th>K60-30</th>
<th>K60-32</th>
<th>K60-38</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>25/12</td>
<td>28/12</td>
<td>20</td>
<td>22</td>
<td>25</td>
<td>27</td>
<td>29</td>
<td>22</td>
<td>28</td>
</tr>
</tbody>
</table>

- **Tensile stress area** (mm²): 250, 357, 200, 250, 346, 430, 510, 580, 830
- **Ultimate load** (kN): 250, 350, 200, 250, 350, 400, 490, 580, 750
- **Breaking load strength (kN)**: 150, 260, 80, 100, 180, 200, 200, 320
- **GRP n.r. kN**: 70
- **Power (kN)**: 150, 180, 80, 100, 180, 180, - , -
- **Ultimate Strength (N/mm²)**: 1,000, 1,000, 1,000, 1,000, 1,000, 1,000, 950, 950, 950
- **Tensile resistance (N)**: 100, 110, 60, 70, 120, 130, 180, 230, -
- **Shear resistance (N/mm²)**: 460, 460, 460, 460, 460, 490, 430, 430, 460
- **Tensile P-Max (N/mm²)**: 50,000, 50,000, 50,000, 50,000, 50,000, 50,000, 50,000, 50,000, 50,000

**Figure 12:** GRP Bolts

The advantages of the GFRC/GRP Rockbolts are as under:

(a) High corrosion resistant;
(b) Cuttability;
(c) Continuous threaded bar;
(d) High Tensile Strength;
(e) Flexibility;
(f) Light Weight;
(g) Easy Handling.

The technical details of two types of GRP bolts manufactured by “MINOVA & FiReP” Switzerland with brand names “POWERTHREAD FiReP Rock bolt System” Model K60 Solid Rods & Model J64 Tubular Rods are as under (Table 2) for appreciation:

7.0 Self Drilling Anchor:- Self Drilling Anchor (SDA) consists of a hollow rod with sacrificial bit attached to it. After drilling operation is completed and anchor is placed in position, the cement/resin grouting is done through the hollow rod which surrounds the cavity between the rock and rod. These are very useful and effective for collapsible soil or loose rocks.

Two types of Self Drilling Anchors are mainly used in USBRL Project these days:

(i) MAI Self Drilling Anchors from Atlas Copco.
(ii) SupAnchor Self Drilling Hollow Anchor (Chinese made).

7.1 MAI Self Drilling Anchors:- A MAI SDA self-drilling anchor consists of:

(i) A hexagonal nut;
(ii) A bearing plate;
(iii) Extension couplings, if the anchor consists of several anchor rod sections;
(iv) Hollow anchor rod(s);
(v) A sacrificial drill bit.
Technical Data of MAI Self Drilling Anchors is under:

Table 3:

<table>
<thead>
<tr>
<th>Anchor Rods</th>
<th>R22 L</th>
<th>R22 N</th>
<th>R22 S</th>
<th>R38N</th>
<th>R51L</th>
<th>R51 N</th>
<th>T76 N</th>
<th>T76 S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Outer diameter</td>
<td>mm</td>
<td>32</td>
<td>32</td>
<td>32</td>
<td>38</td>
<td>51</td>
<td>51</td>
<td>76</td>
</tr>
<tr>
<td>Internal diameter</td>
<td>mm</td>
<td>21.5</td>
<td>18.5</td>
<td>15.0</td>
<td>19.0</td>
<td>36.0</td>
<td>33.0</td>
<td>51.0</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>mm²</td>
<td>304</td>
<td>396</td>
<td>488</td>
<td>771</td>
<td>776</td>
<td>939</td>
<td>1835</td>
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<tr>
<td>Ultimate tensile load</td>
<td>kN</td>
<td>210</td>
<td>280</td>
<td>360</td>
<td>500</td>
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<td>800</td>
<td>1600</td>
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<tr>
<td>Yield load</td>
<td>kN</td>
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<td>250</td>
<td>280</td>
<td>400</td>
<td>450</td>
<td>630</td>
<td>1200</td>
</tr>
<tr>
<td>Weight per m</td>
<td>kg</td>
<td>2.9</td>
<td>3.4</td>
<td>4.1</td>
<td>6.0</td>
<td>7.0</td>
<td>8.4</td>
<td>15.0</td>
</tr>
</tbody>
</table>

Extension couplings:

| Diameter | mm | 22 | 22 | 22 | 22 | 22 | 22 | 22 |
| Length | mm | 145 | 145 | 190 | 220 | 140 | 190 | 200 |
| Weight | kg | 0.8 | 0.8 | 1.0 | 1.7 | 1.2 | 1.9 | 6.4 |

Morten Nuts:

| Key size | mm | 46 | 46 | 46 | 50 | 50 | 50 | 75 |
| Length | mm | 45 | 45 | 45 | 50 | 50 | 70 | 70 |
| Weight | kg | 0.4 | 0.4 | 0.4 | 0.4 | 1.6 | 1.6 | 3.6 |

Anchor plates:

| Dimensions | mm | 150x150 | 200x200 | 200x200 | 200x200 | 200x200 | 250x250 | 250x250 | 250x250 |
| Thickness | mm | 8 | 10 | 12 | 12 | 30 | 40 | 40 | 60 |
| Hole diameter | mm | 35 | 35 | 35 | 41 | 60 | 60 | 60 | 80 |
| Weight | kg | 1.4 | 3.1 | 3.7 | 3.6 | 8.8 | 18.5 | 18.5 | 26.9 |

Table 3a: Technical data of SupAnchor Self Drilling Hollow Anchor (Chinese made)

8.0 NATIONAL AND INTERNATIONAL CODES/STANDARDS ON ROCKBOLTS

8.1 ASTM D4435-08 Standard test method for rock bolt anchor pull test

The objective of test method is to measure the working and ultimate capacities of a rock bolt anchor. This method does not measure the entire roof support system. This method also does not include tests for pre-tensioned bolts or mine roof support system evaluation. This test method is applicable to mechanical, cement grout, resin, (epoxy, polyester, and the like), or similar anchor systems. The bolt is pulled hydraulically and the displacement of the bolt head is measured concurrently. The bolt is pulled until the anchor system of rock fails. The ultimate and working capacities of bolt are calculated from plot of load versus displacement.

Such pull test may be used to provide a quantitative measure of relative performance of different anchor systems in the same rock type. Anchor systems may be different mechanical anchors or different bond materials or lengths for grouted anchors. Such data can be used to choose an anchor type and determine bolt length, spacing and size.
8.3 ASTM D4436-08 Standard test method for rock bolt long-term load retention

The objective of this test method is to determine the time over which rock bolt tension decreases from the installed value to the designated minimum value. This test method is applicable to any anchor system which is not fully encapsulated immediately upon installation, including mechanical, cement grout, resin (epoxy polyester and the like) or other similar systems. In this test, the rock bolt is installed in the same manner and in the same material as its intended support use. The load on the bolt is monitored over a period of time, usually several weeks. After installation of the rock bolt, the load generally decreases over the lime due to deterioration of the borehole wall, creep, and the other factors. This process may be arrested by fully encapsulating the bolt shortly after installation. This is generally done by pumping the bolt hole with full of cement grout, through synthetic resins may also be used. The rate of load loss determines the interval during which the bolt must be encapsulated during construction.

8.4 ISRM document; Suggested Methods of Rockbolt Testing

Part 1; Suggested Method for Determining the strength of a rockbolt anchor (Pull Test): - This test is intended to measure the short-term strength of rockbolt anchor installed in the field conditions. Strength is measured by a pull test in which bolt head displacement is measured as a function of the applied bolt load to give a load-displacement curve. The test is usually employed for selection of bolts and also for control on the quality of materials and installation methods.

Part 2; Suggested Method of determining the Rockbolt Tension using a Torque Wrench: - This method may be used to applied a specified tension during rockbolt installation, or to estimate loss of tension in a previously installed bolt. The same method may be used to verify that anchor strength is greater than a specified value consistent with the maximum tension that can be applied with the wrench.

Part 3; Suggested Method for Monitoring Rockbolt Tension using Load Cells: - This method is used for monitoring changes in tension that occur in a rockbolt over an extended period of time following installation.

8.5 ASTM F432-10 Standard Specification for Roof and Rock bolts and Accessories

This specification covers the chemical, mechanical, and dimensional requirements for roof and rock bolts and accessories. Addressed in this specification are double-end threaded and slotted steel bars; fully grouted bolts and threaded bars; mechanical anchorage devices used for point anchorage applications; roof truss systems; partially grouted deformed bolts; formable anchorage devices; and other frictional anchorage devices. All of these products represent various designs used for ground support systems.

8.6 ASTM D7401-08 Standard test methods for laboratory determination of rock anchor capacities by pull and drop tests

For a support system to be fully effective, the support system must be able to contain the movement of rock material due to excavation stress release, slabbing, etc. Data from the load tests are used by engineers to design the appropriate support system to improve safety and stability of underground support systems. Test Methods D 4435 and D 4436 are used for in-situ load tests. The local characteristics of the rock, such as roughness and induced fractures, are significant factors in the anchor strength. The material used to simulate the borehole surface should be sufficiently roughened so that failure occurs in the rock anchor and not at the simulated anchor-rock surface. In the case of steel pipe, internal threading using different spacing and depth is accomplished using a machinist’s lathe to simulate roughness. These test methods cover the quantitative determination of the working and ultimate static or dynamic capacities of full scale rock anchors. Dynamic capacities are determined to simulate rockburst and blasting conditions. The rock anchors are installed in steel pipe to simulate standard boreholes sizes. Rock anchor capacities are determined as a function of resin to steel bolt bond strength and steel bolt yield strength. These tests are not intended to determine rock anchor to borehole rock surface shear strength. These test methods are applicable to mechanical, resin, or other similar anchor systems. Two methods are provided to determine the capacities of rock anchors as follows:

(i) Method ‘A’: - Using a horizontal hydraulically loaded pull test system. In the Pull test, the rock anchor is hydraulically pulled horizontally and the displacement of the bolt head is measured concurrently. The bolt is pulled until the anchor system fails (or to the ultimate stroke of the ram). The ultimate and working capacity of the rock anchor is calculated from the plot of load versus displacement.

(ii) Method ‘B’: - Using a vertical dynamically loaded drop test system. In the Drop test, a known mass is released vertically impacting on a plate at a preset distance that is in turn affixed to the end of a rock anchor. The maximum energy is expressed in kJ.
Vinod Kumar, a young IRSE officer, is currently posted as Deputy Chief Engineer/North/USBRL Project at Jammu and also looking after the duties of Dy. Chief Engineer (Construction)/Banihal. He belongs to a small village (Chackbakhtawar) of Jammu District of Jammu & Kashmir & has done his schooling from Jammu itself. He did his BE (Civil Engineering) from Moti Lal Nehru Regional Engineering College, Allahabad (presently known as Moti Lal Nehru National Institute of Technology) in 1998 and joined Indian Railway Service of Engineers in Sep’1999 (i.e.1998 Exam Batch).

Vinod joined USBRL Project in June’2009 as Deputy Chief Engineer (Construction)/Banihal and served there for almost four and half years. The jurisdiction of Dy. Chief Engineer (Construction)/ Banihal extends from Dharam in Ramban District of Jammu Division and Baramullah in Baramullah District of Kashmir Division of Jammu & Kashmir out of which Dharam-Qazigund is geographically and geologically most difficult terrain having total ten tunnels out of which three tunnels are more than 10.00 Km long and one is 8.60 Km long.

Vinod has previously served in Northeast Frontier Railway in different capacities before joining USBRL Project such as Assistant Divisional Engineer/Barpeta Road, Executive Engineer (C)/Lumding, Deputy Chief Engineer (C)/ Planning, Maligaon and Deputy Chief Engineer (C)/ Rangia. Out of these postings he was involved for three and half years in construction of tunnels in Lumding-Silchar Gauge Conversion Project which is also a National Project. Vinod counts his eight and half years of service out of thirteen years (excluding initial training of one and half years) being involved in construction of tunnels in hilly areas of Northeast and Jammu & Kashmir.

Services of Sh. Vinod Kumar have been outstanding throughout his carrier. He was awarded General Manager’s Awards in year 2002-03 & 2006-07 while working as Assistant Divisional Engineer/Barpeta Road and Executive Engineer (Construction)/Lumding. He was awarded Director General’s Medal for standing first with Distinction in Management Development Course No. MDP-09/2007 at Railway Staff College, Vadodra.

He has also received highest award of railway i.e. MR award during 58th Railway Week function’2013 for outstanding services. Vinod has also written and presented various technical papers. He was conferred the K.C.Sood Memorial Award in January’2014 at National Technical Seminar of IPWE for the best paper on “Laying of Ballastless Track System in Pir Panjal Tunnel”. He has also attended various training courses on tunneling and recently visited National Institute of Technology, Oslo Norway in connection with training on Tunneling Technology.

Vinod feels proud of having associated with USBRL Project despite all odds such as difficult terrain, remote and backward locations, non-family posting stations, adverse weather conditions, severe winters, frequent road blockades. Some words from Vinod are: -

“When I was posted at Banihal in June’2009 there were inaccessible and remote project sites like Sumber for which 8 to 10 hrs. on foot were required to reach there. I still remember the day, 4th May’2011, when I fainted on hill slope of deep nullah while going to Sumber. Thanks to Sh. Jagdish Kumar, then AXEN/C/Sangaldan & Sh. Avinash Kumar of J&K Police, PSO with me for their timely action and shifting me to a nearby suitable place for saving my life. I still cannot forget the days from 6.01.2012 to 12.01.2012 when Banihal was totally cut off from the rest of world due to heavy snowfall and I was stuck up with my family with one and half year old daughter and other railway officers and staff. The conditions might have been tougher before my posting as lot of infrastructure was already in place when I joined in the Project. However, such incidents have become things of past now. With the construction of the access road to Sumber by Railway, it has been connected to the mainstream. Also with the opening of Banihal-Qazigund section by the Railway, Banihal always remains connected to Srinagar and other major town of the Kashmir valley even during heavy snowfalls. In this way Railway Officers and Staff posted in USBRL Project are working relentlessly under adverse working conditions and doing commendable job to provide connectivity to the remote areas, connecting the people with the mainstream, providing them access to basic amenities etc. not only by constructing the railway line, but also by constructing the access roads. Smile on the people of these areas gives immense satisfaction and proud feeling of having associated with this Project”.

Vinod Kumar

Profile

Name: Vinod Kumar, IRSE
Place of Birth: Jammu
DOB: 13.03.1973
Hobbies: Reading books, listening music
Favorite food: Vegetarian food
Quake Tremor or Temblor

The dynamic loading that act on structural system may result from wide range of input mechanism. One of the many sources of external loading that must be considered in fixed structures, the most important by far in terms of its potential for disastrous consequences is the earthquake. The degree of importance of earthquake loading in any region is related to its probable intensity and likelihood of occurrence that is to the seismicity of the region. It is evident that the design of economic and attractive structures which can successfully with stand the forces induced by a severe ground motion is a challenge demanding the best in structural engineering, art and science.

This, third part of the series is devoted to understanding the various methods used in quantification and simplification of an earthquake event.

Measure of earthquake size

Magnitude

For design the most important aspect of an earthquake is the effect it will have on structure. The damage potential of an earthquake is at least dependent on the size of an earthquake and a number of measures are used for different purposes. The most important measure of size from seismological point of view is the amount of strain energy released at the source and this is indicated qualitatively as the magnitude. By definition, Richter magnitude, was developed by Charles F. Richter in 1934, is the (base10) logarithm of the maximum amplitude, measured in micrometer (10^-6 m) of the earthquake record obtained by a seismograph, corrected to a distance of 100 km. This magnitude rating has been related empirically to the amount of earthquake energy released \( E \) by the formula:

\[
\log E = 11.8 + 1.5 M
\]

by this formula, the energy increases by a factor of 32 for each unit increase of magnitude. More important to engineers, however, is the empirical observation that earthquakes of magnitude less than 5 are not expected to cause structural damage, whereas for magnitude greater than 5, potentially damaging ground motion will be produced.

Since this M scale was simple to use and corresponded well with the damage which was observed, it was extremely useful for engineering earthquake-resistant structures, and gained common acceptance in early days.

Intensity

The magnitude of an earthquake by itself is not sufficient to indicate whether structural damage can be expected. This is a measure of size of earthquake at its source, but the distance of the structure from the source has an equally important effect.

![Earthquake frequency and destructive power](image)

By: Sangrah Maurya  
Deputy Chief Engineer/Design  
USBRL Project
The **moment magnitude scale** (abbreviated as MMS; denoted as $M_W$ or $M$)

Unfortunately, the Richter scale, do not provide accurate estimates for large magnitude earthquakes. Today the **moment magnitude scale**, abbreviated $M_W$, is preferred because it works over a wider range of earthquake sizes and is applicable globally. The moment magnitude scale is based on the total moment release of the earthquake. Moment is a product of the distance a fault moved and the force required to move it. It is derived from modeling recordings of the earthquake at multiple stations. Moment magnitude estimates are about the same as Richter magnitudes for small to large earthquakes. But only the moment magnitude scale is capable of measuring $M_8$ (read ‘magnitude 8’) and greater events accurately is used by seismologist to measure the size of earthquake in terms of the energy released . The magnitude is based on the seismic moment of the earthquake which is equal to the rigidity of the Earthmultiplied by the average amount of slip on theand the size of the area that slipped. The scale was developed in the 1970 of slip on the fault and the size of the area that slipped. The scale was developed in the 1970s to succeed the 1930s-

<table>
<thead>
<tr>
<th>Magnitude</th>
<th>Description</th>
<th>Mercalli intensity</th>
<th>Average earthquake effects</th>
<th>Average frequency of occurrence (estimated)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 2.0</td>
<td>Micro</td>
<td>I</td>
<td>Microearthquakes, not felt, or felt rarely by sensitive people. Recorded by seismographs</td>
<td>Continual/several million per year</td>
</tr>
<tr>
<td>2.0–2.9</td>
<td>Minor</td>
<td>I to II</td>
<td>Felt slightly by some people. No damage to buildings.</td>
<td>Over one million per year</td>
</tr>
<tr>
<td>3.0–3.9</td>
<td>Light</td>
<td>II to IV</td>
<td>Often felt by people, but very rarely causes damage. Shaking of indoor objects can be noticeable. Noticeable shaking of indoor objects and rattling noises. Felt by most people in the affected area. Slightly felt outside. Generally causes none to minimal damage. Moderate to significant damage very unlikely. Some objects may fall off shelves or be knocked over.</td>
<td>Over 100,000 per year</td>
</tr>
<tr>
<td>4.0–4.9</td>
<td>Moderate</td>
<td>IV to VI</td>
<td>Can cause damage of varying severity to poorly constructed buildings. At most, none to slight damage to all other buildings. Felt by everyone. Casualties range from none to a few.</td>
<td>1,000 to 1,500 per year</td>
</tr>
<tr>
<td>5.0–5.9</td>
<td>Strong</td>
<td>VI to VIII</td>
<td>Damage to a moderate number of well-built structures in populated areas. Earthquake-resistant structures survive with slight to moderate damage. Poorly designed structures receive moderate to severe damage. Felt in wider areas; up to hundreds of miles/kilometers from the epicenter. Strong to violent shaking in epicentral area. Death toll ranges from none to 25,000.</td>
<td>100 to 150 per year</td>
</tr>
<tr>
<td>6.0–6.9</td>
<td>Major</td>
<td>VII to X</td>
<td>Causes damage to most buildings, some to partially or completely collapse or receive severe damage. Well-designed structures are likely to receive damage. Felt across great distances with major damage mostly limited to 250 km from epicenter. Death toll ranges from none to 250,000. Major damage to buildings, structures likely to be destroyed. Will cause moderate to heavy damage to sturdy or earthquake-resistant buildings. Damaging in large areas. Felt in extremely large regions. Death toll ranges from 1,000 to 1 million.</td>
<td>10 to 20 per year</td>
</tr>
<tr>
<td>7.0–7.9</td>
<td>Great</td>
<td>VIII or greater (^{[16]})</td>
<td>Near or at total destruction - severe damage or collapse to all buildings. Heavy damage and shaking extends to distant locations. Permanent changes in ground topography. Death toll usually over 50,000.</td>
<td>One per 10 to 50 years</td>
</tr>
<tr>
<td>8.0–8.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0 and greater</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Peak Ground Acceleration

Peak ground acceleration (PGA) represents the maximum acceleration of ground measured during an earthquake and is an important parameter in the design of structures. Richter and moment magnitude represent the maximum energy released at the fault plane during an earthquake but PGA represents how hard the ground has accelerated at the site where recording of an event has been done. As deliberate in the previous article PGA is measured by instruments, such as accelerographs. During an earthquake energy is dissipated in all directions in the form of wave, causing ground movement in two horizontal directions and one vertical direction. Acceleration value (rate of change of speed) of these motions is recorded as PGA and PGV (peak ground velocity) is the maximum speed (rate of change of movement) of the motion experienced by the ground whereas PGD (peak ground displacement) is the maximum value of distance moved. The structures are directly affected by the ground acceleration, velocity or displacement, hence they have important place in structural design then the magnitude and intensity value.

These values vary in different earthquakes, and in differing sites within one earthquake event, depending on a number of factors. These include the length of the fault, magnitude, the depth of the quake, the distance from the epicentre, the duration (length of the shake cycle), and the geology of the ground (subsurface). Shallow-focused earthquakes generate stronger shaking (acceleration) than intermediate and deep quakes, since the energy is released closer to the surface.

Peak ground acceleration is expressed in g (the acceleration due to Earth's gravity, equivalent to g-force) as either a decimal or percentage; in m/s² (1 g = 9.81 m/s²) or in Gal, where 1 Gal is equal to 0.01 m/s² (1 g = 981 Gal).

PGA value recorded for some notable earthquakes is given below.

### NOTABLE EARTHQUAKES

<table>
<thead>
<tr>
<th>PGA single direction (max recorded)</th>
<th>PGA vector sum (H1, H2, V) (max recorded)</th>
<th>Mag</th>
<th>Depth</th>
<th>Fatalities</th>
<th>Earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.7 g</td>
<td>2.99 g</td>
<td>9.0</td>
<td>30km</td>
<td>&gt;1500</td>
<td>2011 Tohoku earthquake and tsunami</td>
</tr>
<tr>
<td>2.2 g</td>
<td>6.3</td>
<td>5km</td>
<td>185</td>
<td></td>
<td>February 2011 Christchurch earthquake</td>
</tr>
<tr>
<td>2.13 g</td>
<td>6.4</td>
<td>6km</td>
<td>1</td>
<td></td>
<td>June 2011 Christchurch earthquake</td>
</tr>
<tr>
<td>4.36 g</td>
<td>6.9/7.2</td>
<td>8km</td>
<td>12</td>
<td></td>
<td>2008 Iwate-Miyagi Nairiku earthquake</td>
</tr>
</tbody>
</table>

### Response spectra

Consider a SDOF (single degree of freedom system) as shown in figure below having stiffness k and mass m. SDOF is defined as a system that is free to translate in one direction (horizontal displacement in the present case) only, translation and rotation in other direction being restrained. The stiffness of the horizontal members is much much larger than the stiffness of the vertical members so that rotation of horizontal members is negligible.
When such a system is subjected to support excitation equivalent to ground acceleration time history generated during an earthquake as shown in the figure on left, the system sets into motion and produces its own acceleration response. The displacement response of the SDOF system will depend on its stiffness and natural period of vibration. The response of the SDOF system could be something like as shown in the figure below for different stiffness (hence varying time period of oscillation).

A. For stiffness tending to infinity i.e. very rigid systems acceleration response will be the same as the ground/support excitation.

![Acceleration vs Time](image1)

B. For slightly stiffer system say for time period of oscillation of 0.5 sec acceleration response will be like

![Acceleration vs Time](image2)

C. For less stiffer system say for time period of oscillation of 1.0 sec acceleration response will be like

![Acceleration vs Time](image3)

D. For even more less stiffer system say for time period of oscillation of 2.0 sec acceleration response will be like

![Acceleration vs Time](image4)

If highest value of acceleration recorded in each case is plotted corresponding to the natural period of oscillation the curve so drawn is called the acceleration response spectra. The response spectra can be drawn for acceleration, velocity or displacement. The maximum value of steady state response (acceleration, velocity or displacement) during the entire time history of loading (ground/support acceleration) represents the maximum displacement response for the linear system. When these maximum response value are calculated for different natural period of SDOF system and plotted, the plot is termed as response spectra. The resulting plot can then be used to pick up the response of any linear system, given its natural frequency/time period of oscillation.

The concept of response spectra was first incorporated into the United States building codes in the late 1950’s by means of the coefficient C in the lateral force equation \( V = K C W \) by the Structural Engineers Association of California (SEAOC, 1960), where \( V \) is the total lateral force, \( K \) is a structural systems coefficient of 1.33, 1.0 or 0.67, and \( W \) is the total dead load. Over the decades, response spectra have been playing an increasing role in the development of earthquake design criteria.

Response spectra provide a very handy tool for engineers to quantify the demands of earthquake ground motion on the capacity of buildings to resist earthquakes. Data on past earthquake ground motion is generally in the form of time-history recordings obtained from instruments placed at various sites that are activated by sensing the initial ground motion of an earthquake. The amplitudes of motion can be expressed in terms of acceleration, velocity and displacement. Although useful to express the relative intensity of the ground motion (i.e., small, moderate or large), the PGA does not give any information regarding the frequency (or period) content that influences the amplification of building motion due to the cyclic ground motion. In other words, tall bridge piers with long fundamental periods of vibration will respond differently than short bridge piers with short periods of vibration. Response spectra provide these characteristics.

As already defined the peak acceleration (Sa) of each of these SDOF systems, when subjected to an earthquake ground motion, is calculated and plotted with the corresponding period of vibration (T), the locus of points will form a response spectrum for the subject ground motion. Thus, if the period of vibration of a structure or bridge pier is known, the maximum acceleration can be determined from the plotted curve for that particular ground history for which the response spectra has been plotted.

Consider the ground motion history for PGA in two horizontal direction for El Centro Earthquake (first strong motion earthquake to be recorded)

![El Centro Earthquake](image5)
In physics, attenuation (also called extinction) is the gradual loss in intensity of any kind of flux through a medium. For instance, sunlight is attenuated by dark glasses, X-rays are attenuated by land and light and sound are attenuated by water.

The amplitude of a seismic pulse in an idealized, purely elastic earth is controlled by the reflection and transmission of energy at the boundaries and by geometric spreading. These seismic waves can propagate indefinitely once they are excited. But this would be true if the earth was perfectly elastic. It is known that the real earth is not perfectly elastic. This causes the waves that are propagating to attenuate with time as they travel. This attenuation in the propagating waves is caused due to various energy loss mechanisms. A seismic wave loses energy as it propagates through the earth this mechanism is called attenuation.

The energy with which an earthquake affects a location depends on the running distance. The attenuation in the signal of ground motion intensity plays an important role in the assessment of possible strong ground shaking.

Annulation relations present the results of analyzing strong motion data in showing how large the ground motions are expected to be for a certain earthquake magnitude and a certain distance from the earthquake. Usually the attenuation relations are obtained by a statistical process called regression. Given a specified mathematical equation, regression determines parameters for that equation. In some cases, regression is used to determine the remaining parameters when the other parameters are given by geophysical attenuation models. Then, given a magnitude, a distance, and a geologic site condition, the equation gives the average value of the ground motion expected.

Using the attenuation equation thus developed ground motion for a future earthquake of higher magnitude can be estimated. However, the ground motion so predicted will not be the actual average value given by the attenuation equation, but rather a value in some uncertainty range around that average value. The regression also gives an estimate of that uncertainty range.

First attenuation equation for PGA (Peak Ground Acceleration) was proposed by Milne & Davenport in 1969 followed by another equation by Esteva in 1970. Similarly first attenuation equation for spectral ordinates was proposed by Johnson in 1973. Till the end of year 2000 total 121 numbers of attenuation equations for PGA and 76 numbers for spectral ordinate have been proposed by various researchers. Compilation of all the equation are available in Engineering Seismology and Earthquake Engineering Journal, Imperial College of Science, Technology and Medicine, Civil Engineering Department, London.

Attenuation equation proposed by Milne & Davenport and Esteva are given below, both the equation were for W.USA region:

1.1 Milne & Davenport (1969)

- Ground motion model is:
  \[ A = \left( \frac{g(10M)^{12}}{v} \right)^2 \]
  where \( A \) is in percentage, \( v = 1.05 \), \( v = 1.17 \), and \( v = 1.09 \) (not given).

- Use data from Esteva & Rosenblatt (1964).

1.2 Esteva (1970)

- Ground motion model is:
  \[ a = c_1 c_2 (R + c_3)^{-a} \]
  where \( a \) is in cm/s², \( c_1 = 1200 \), \( c_2 = 0.8 \), \( c_3 = 25 \), \( c_4 = 2 \) and \( a = 1.02 \) (in term of natural logarithm).

- Records from soft soils comparable to stiff clay or compact conglomerate.

- Records from earthquakes of moderate duration.

Few of the other important attenuation Equation proposed by other scientist are given below:-

12 Donovan & Borein (1978)

- Ground motion model is:
  \[ g = b_1 b_2 M (R + 25)^{-b_2} \]
  where
  \[ b_1 = c_1 R^{a/2} \]
  \[ b_2 = \frac{a}{1 + 2 \log R} \]

where \( g \) is in gal, \( c_1 = 2.154 \), \( c_2 = 2.8 \), \( d_1 = 0.046 \), \( d_2 = 0.436 \), \( c_3 = 2.515 \), \( c_2 = -0.486 \). For \( g = 0.01 \), \( g = 0.03 \), \( g = 0.05 \), \( g = 0.08 \), \( g = 0.10 \), \( g = 0.46 \) and \( g = 0.15 \), \( a = 0.41 \) (in term of natural logarithm).
Above equation considers two site conditions viz Rock (21 records) and stiff soil (38 records), most records considered were from within 50 km radius and with magnitude of about 6.5.

2.16 Cornell et al. (1979)

- Ground motion model is:
  \[
  \ln A_p = a + bM_L + c \ln(R + 25)
  \]
  where \( A_p \) is in cm/s², \( a = 6.74, b = 0.859, c = -1.80 \) and \( \sigma = 0.57 \).
  - No more than 7 records from one earthquake to avoid biasing results.
  - Records from basements of buildings or free field.

2.21 Campbell (1981)

- Ground motion model is:
  \[
  PGA = a \exp(bM)[R + c \exp(dM)]^{-d}
  \]
  where PGA is in g, for unconstrained model \( a = 0.0109, b = 0.968, c = 0.0006, d = 0.700 \), \( R = 1.00 \) and \( \sigma = 0.632 \) (for natural logarithm) and for constrained model \( a = 0.0185, b = 1.28 \), \( c = 0.114, d = 1.75 \) and \( \sigma = 0.384 \) (in terms of natural logarithm).

2.57 Jacob et al. (1990)

- Ground motion model is:
  \[
  A = 10^{a_1 + a_2M + a_3 \log(d + a_4d)}
  \]
  where \( A \) is in g, \( a_1 = -1.43, a_2 = 0.31, a_3 = -0.62 \) and \( a_4 = -0.0026 \) (\( \sigma \) not given)
  - Note equation only for hard rock sites.

2.72 Campbell (1993)

- Ground motion model is:
  \[
  \ln(Y) = \beta_0 + \beta_1 M + \beta_2 \log[\exp(0.47) - \ln(R)] + \ln(0.01 \exp(0.17))^{1.17}
  \]
  where \( Y \) is in g, \( \beta_0 = -2.35, \beta_1 = -0.0, \beta_2 = 1.6 \) and \( \beta_3 = -0.0130, \beta_4 = -0.000096, a_1 = -0.685, a_2 = -0.017, a_3 = 0.0596, a_4 = 0.27, a_5 = -0.405, a_6 = 0.620 \) and \( \sigma = 0.56 \).
  - Uses two site categories:
    - S=0 (Quaternary deposits (soil)).
    - S=1 (Tertiary or older sedimentary, metamorphic, and igneous deposits (rock)).
    - Also includes depth to basement rock (ftm). \( D \).
  - Uses two fault mechanisms:
    - F=0 (Strike-slip).
    - F=1 (Reverse, reverse-oblique, thrust, and thrust-oblique).
  - Recommends use \( F = 0.5 \) for normal or unknown mechanisms.

New Generation attenuation equation.

Since 2000, due to increase in number of installation of sensitive and modern instruments to record earthquake induced ground motion the database of ground histories has increased many folds. Modern day practice is also to install the recording instruments in array form to have spatial recordings of the ground motion measuring separately the three component of motion. Similarly recording of shallow crustal ground motion has increased confidence of researchers and given insight to predict these motion more vividly as crustal deformation are more catastrophic to structures.

In view of above, a project was initiated by Pacific Earthquake Engineering Research center (PEER), with headquarters at the University of California, Berkeley. The objective of the project is to develop a new ground motion characterization (GMC) model for the western, Central and Eastern North-American (CENA) region.

The GMC model consists in a set of new ground motion prediction equations (GMPEs) for median and standard deviation of ground motions (GMs) for use in probabilistic seismic hazard analyses (PSHA). NGA-East was originally developed as a science-based research project originally designed as a follow-up to the previous NGA project (referred to as NGA-West for clarity) that focused on the development of GMPEs for shallow crustal earthquakes in active tectonic regions.

AS already deliberated ground motion prediction equations (GMPEs), giving ground motion intensity measures such as peak ground motions or response spectra as a function of earthquake magnitude and distance, are important tools in the analysis of seismic hazard. These equations are typically developed empirically by a regression of recorded strong-motion amplitude data versus magnitude, distance, and possibly other predictive variables. The amount of data used in regression analysis is an important issue, as it bears heavily on the reliability of the results, especially in magnitude and distance ranges that are important for seismic hazard analysis.

In the instant project developers of five pre-existing and widely used empirical ground motion models participated in the concurrent development of the NGA models. These developers, along with references to their pre-existing models, are as follows:


To meet the needs of earthquake engineering design practice, all NGA models were required to be applicable to the following conditions (Power et al., 2006):

1. They should include the ground motion parameters of peak ground acceleration, velocity, and displacement (PGA, PGV, PGD) and 5%-damped elastic pseudo-absolute response spectral acceleration (PSA) for a minimum set of periods ranging from 0–10 s;
2. They should model the average horizontal motion as well as motions in the strike-normal (SN) and strike-parallel (SP) directions, although this latter requirement was eventually postponed to a later phase of the project;
3. They should be valid for shallow crustal earthquakes with strike-slip, reverse, and normal mechanisms in the western United States;
4. They should be valid for moment magnitudes ranging from 5.0–8.5;
5. They should be valid for distances ranging from 0–200 km; and

The work and models for GMPE is available on web site of Pacific Earthquake Engineering Research Centre (http://peer.berkeley.edu/) and can be referred for more information.

To be concluded…..
Formulation of shift of a circular curve with unequal transition length

Quite often rail or road alignment moves through different curvatures. It may consists of straight lines, circular curves and compound curves or may have some other different compositions. This paper presents the formulation of co-ordinates for the alignment having straight lines and circular curves and in quest of that the article presents formulae for calculation of shift of a circular curve provided with unequal transition length.

First of all the relations will be drawn in local co-ordinate system which can be very easily converted into WGS-84 or any other type of global co-ordinate system. Global co-ordinate does not mean only standard co-ordinates system like WGS-84 but a co-ordinate system adopted for a

**Straight Line:**

For given two number of points other points between them can easily be determined as the relationship between them will remain linear.

**Circular Curve:**

Let us consider a circular curve of radius ‘R’ between the tangents $T_1O$ and $T_2O$. The deviation angle between the tangents are $\delta$. The centre of the circular curve is $c_1$. The circular curve touches the left and right hand tangents at $A$ and $B$ respectively.

Take a case when the circular curve is shifted in such a way that the centre of the curve assigns a new position $c_2$ from its old position $c_1$. Absolute distance between $c_1$ and $c_2$ is $s$. $c_2M$ and $c_2N$ are perpendicular to the tangents from $c_2$. (Refer figure 1)

$$x = ay^3 + k$$

(1)

Where, $a$ and $k$ is a constant

$$\frac{1}{R} = \frac{\frac{d^2x}{dy^2}}{[1 + \left(\frac{dx}{dy}\right)^2]^{3/2}}$$

Assuming as $\frac{dx}{dy} = 0$ within transition length and this assumption fairly accurate if $L \leq R$, then

$$\frac{1}{R} = \frac{d^2x}{dy^2}$$

At point $P_1$, where transition curve touches circular curve, curvature of both curves will be same,

At $P_1$, $y=L_1$ and $x=0$, therefore $\frac{1}{R} = \frac{6aL_1}{24R}$ where $a = \frac{1}{6L_1R}$

At $T_1$, $x=0, y=0$, therefore, $k = 0$

Assuming again,

At $M, x = \frac{s \cos(\phi_1)}{2}, y = \frac{L_1}{2}$ where $\angle(c_1c_2M) = \phi_1$

Therefore, $a \left(\frac{L_1}{2}\right)^3 = \left(\frac{1}{6L_1R}\right) \left(\frac{L_1}{2}\right)^3$

$$s \cos(\phi_1) = \frac{L_1^2}{24R}$$

(2)

This way the above mentioned assumption, basically serves as a boundary condition to find out the value of $s$. Refer figure no. 2 and 3. Similarly, to the other side of the transition curve where the length of the transition curve is $L_2$, $\angle(c_1c_2N) = \phi_2$

$$s \cos(\phi_2) = \frac{L_2^2}{24R}$$

(3)

Also, $AM = s \sin(\phi_1) = \frac{L_1^2}{24R} \tan(\phi_1)$

Similarly, $BN = \frac{L_2^2}{24R} \tan(\phi_2)$

Say, $S_1 = \frac{L_1^2}{24R} & S_2 = \frac{L_2^2}{24R}$, hence

$AM = S_1 \tan(\phi_1)$ & $BN = S_2 \tan(\phi_2)$ where $S_1 = \frac{L_1^2}{24R}$ & $S_2 = \frac{L_2^2}{24R}$

**Figure 1**: Curve shown with shift

**Figure 2**: A zoom portion of curve

**Figure 3**: A zoom portion of curve

By:-

Shailendra Kumar,
XEN/C-II, Banihal
USBRL Project
Finding co-ordinate of different portions of a circular curve in a given co-ordinate system:-

Let us take co-ordinate system \((X, Y)\) as shown in figure 1, as our global co-ordinate system. Presented below a typical way to find out the co-ordinates of different portions of a circular curve as represented in figure no. 1. Here, the case has been represented as right hand curve.

1. First Transition Curve: The length of the transition curve is \(L_1\), which extends from \(T_1\) to \(P_1\). The chosen form of transition curve is a cubical parabola as explained above,

\[
x = ay^3; a = \frac{1}{6L_1}\delta 0 \leq y \leq L_1
\]

2. Second Transition Curve: This transition curve which is of length \(L_2\) and extends between \(T_2\) and \(P_2\) as shown in figure 1. Let us first define the co-ordinates in local co-ordinate as shown in figure no 1 as \(X', Y'\)

\[
x' = ay^3; a = \frac{1}{6L_1}\delta 0 \leq y' \leq L_2
\]

Now, the co-ordinates found out in local co-ordinate \((X', Y')\) could be transferred in global co-ordinate \((X, Y)\) by using transformation equations as explained below,

\[
x = x_0 + x \cos(\theta) - y' \sin(\theta) \quad (4)
\]

\[
y = y_0 + x' \sin(\theta) + y' \cos(\theta) \quad (5)
\]

\((x_0, y_0)\)are the co-ordinate of origin of \((X', Y')\)with respect to \((X, Y)\) co-ordinate system.Here in this case, \(\theta = (180 - \delta)\), if \(\delta\) is the deviation angle in degree and replace \(x' - x\), so that the orientation of \((X', Y')\) be similar to that of \((X, Y)\)

Now, consider triangle \(T_1OT_2\), where \(z = 180 - \delta\)

\[
T_1O = 0.5L_1 + S_1 \tan(\phi_1) + R \tan\left(\frac{\delta}{2}\right) \quad (6)
\]

\[
T_2O = 0.5L_2 + S_2 \tan(\phi_2) + R \tan\left(\frac{\delta}{2}\right) \quad (7)
\]

Also, \(x_0 = T_1O \sin(\delta)\) and \(y_0 = T_1O \cos(\delta)\)

\[
\phi_1 + \phi_2 = \delta, \text{ Using equations 2and 3}
\]

\[
\frac{\cos(\phi_1)}{\cos(\phi_2)} = \frac{L_1^2}{L_2^2}
\]

\[
\phi_1 = \tan^{-1}\left[\frac{\frac{L_2^2 - \cos(\delta)}{L_1^2 - \cos(\delta)}}{\sin(\delta)}\right] \quad (8)
\]

Therefore, the equation 4 and 5 can be evaluated.

3. Circular Curve: A curve of Radius \(R\) has to fit between two transition curves. Refer figure no. 4. The circular curve \(AVB\) and its shifted portion cuts the bisector at point \(V\) and \(v\).

\[\text{If} \, \text{v} = s', \text{then} \, c_1v = R - s', c_2v = R \quad \text{and} \quad c_1c_2 = s. \]

From \(\Delta(vc_1c_2), \quad \Delta(vv_1c_2) = 180 - \left(\phi_1 - \frac{\delta}{2}\right)\)

\[
\cos(\Delta vc_1c_2) = \frac{(vc_1)^2 + (c_1c_2)^2 - (vc_2)^2}{2vc_1c_1c_2}
\]

\[
\cos \left[180 - \left(\phi_1 - \frac{\delta}{2}\right)\right] = \frac{(R - s')^2 + (s)^2 - (R)^2}{2(R - s')s}
\]

Figure 4: Curve showing shift

Co-ordinate of centre \(c_2\) will be \(\left[R + s \cos(\phi_1), \frac{L_1}{2}\right]\). If \(c_2 = (c_x, c_y)\), then \(c_x = R + s \cos(\phi_1)\) and \(c_y = \frac{L_1}{2}\). The equation of circle will be,

\[
(x - c_x)^2 + (y - c_y)^2 = R^2.
\]

\[
x = R + s \cos(\phi_1) - \sqrt{R^2 - (y - \frac{L_1}{2})^2}
\]

Where \(L_1 \leq y \leq \frac{L_1}{2} + s \sin(\phi_1) + R \sin(\frac{\delta}{2}) - s' \sin(\frac{\delta}{2})\)

Similarly, with respect to \((x', y')\) also remaining portion of circular curve could be drawn suitably by transforming to global co-ordinate system \((x, y)\) by using transformation equations as explained in equations 4and 5

**Conclusions:**

1. Shift of circular curve of radius \(R\) and unequal transition length \(L_1, L_2\) can be calculated as

\[
s' = (R + s \cdot K) - \sqrt{R^2 - s^2(1 - K^2)} \cdot \delta s = \frac{L_1}{24R \cos(\phi_1)} = \frac{L_1}{24R \cos(\phi_2)} \text{ where, } K = \cos(\phi_1 - \frac{\delta}{2})
\]

\[
\phi_1 = \tan^{-1}\left[\frac{\frac{L_1^2 - \cos(\delta)}{L_1^2 - \cos(\delta)}}{\sin(\delta)}\right]
\]

2. Formulation in local co-ordinate system and its consequent transformation into global co-ordinate system is quite easy and can be easily implemented in computer programing.
1. **INTRODUCTION**

Entire fabrication work of the deck slab of viaduct portion of Chenab bridge is planned to be taken up in segments. There are total 164 segments. Works will be done in two phases from S10 to S180 in two phases i.e. 65 segments from S70 to S180 will be taken up first & subsequently 99 Segments from S10 to S70. 6 Steel Piers on Supports S10, S20, S30, S40, S50 & S60, 9 steel trestles (S41 to S49) on arch between S40 to S50 and fabrication of Steel Arch. The fabrication of 65 segments of viaduct deck structure from S70 to S180 is in progress, initially with 3

2. **FABRICATION OF VIADUCT DECK SUPERSTRUCTURE FROM S70-S180**

The fabrication of deck structure from S180 - S70 makes it the most critical and challenging job, as the alignment being a combination of straight, transition curve and circular curve along its longitudinal profile. Added to this, the orthotropic fully welded deck structure, comprising of two monolithic deck structures - one combining 33 Assembly Segments (AS1-AS33) and the other combining 32 Assembly Segments (AS34-AS65, curve portion with rising gradient of 1:400), brands it to be a vibrant steel fabricated deck structure ever fabricated for the Indian Railways or may be elsewhere.

To bring the fabrication work to its desired quality, in terms of geometry, shape, size, dimensions, camber etc. in general and standard of weldment in particular, the complete process of fabrication to produce an ideal segment, can be visualised from the procedures adopted such as a dedicated Quality Management Procedure (QMP), Quality Assurance Plan (QAP), Inspection and Testing Plan (ITP) for the Steel Plates at manufacturer’s end, Welding Procedure Specifications (WPSs) for each joint, position and process, and Welder Qualification Record (WQR) for each Welder after a vigorous Welder’s Qualification Test (WQT), Weld Test Plan (WTP). In-house engineered Welding Positioners & Fixtures, Fabrication Methodology & Welding Sequences, to take care of the minimum distortion and stresses generated due to welding have been formulated with the help of expert from WRI. These activities, followed by adequate Non-Destructive Tests (NDTs) and Destructive Tests (DTs), would undoubtly be a hallmark in producing the components to be assembled to a segment and then segments welded together to form a continuous welded deck structure first of its kind.

For better understanding, though each assembly segment has its unique design and geometry w.r.t its Assembly Drawing and a set of Single Part Drawing (varying from 65 to 90 nos.) generated out of a three dimensional TEKLA software model, designed by WSP Finland Ltd., Proof Checked by FLIENT & NEILL Limited, London (presently URS-Scott Wilson India) and validated by Konkan Railway Corporation Ltd., through “TEKLA MODEL”, typically a segment constitutes (a) **Main Girders** with **Tee-Beams**, Vertical & Longitudinal Stiffeners, (b) **Secondary Beams**, (c) **Cross Beams**, (d) **Lattice Structures** (e) **Deck Plate etc.**, all assembled together to form a single unit of assembly segment (AS), as per the 3D sketch shown thus, using Butt welds with different edge preparations like Double ‘V’, Single ‘V’, Single Bevelled & Double Bevelled, and fillet welds with equal Leg Length of size varying from 8mm to 18mm. Approximately a segment assembly is fabricated with 2.5 KM butt weld and 2.1 KM fillet weld. After the fabrication of a component, total weld deposit in the segments will vary from 1120 Kg to 1880 Kg which amounts to maximum 2% of the weight of a segment including wastage.

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Leaving aside the above, in our strive towards fabricating quality products, in a stipulated time frame, with the limited space available for the workshops, presently, two workshops are in place at Kauri End of the bridge to cater for different activities in different work stations for smooth production, inspection, testing and passing of the product. A self explanatory fabrication methodology has been developed in defining the various work stations as per the succeeding flowchart.
3. **PRE & POST WELD INSPECTION & TESTING**

In so far as, quality control and quality assurance program for the welded structure is concerned, a four tier system of inspection is in place for both pre and post weld inspections, which comprise the Quality Assurance and Quality Control Team of Contractor at first stage, the Third Party Agency for Quality Assurance (TPAQA) in the second stage, thirdly by the Inspecting Team from KRCL and finally followed by NR/RDSO, as per the Quality Assurance Plan (QAP). The pre-weld inspection activities at all Stations are more of physical in nature i.e. Flatness, Dimensions, Eccentricity, Verticality & Horizontality of the parts/components. The post-weld inspections comprise Non-Destructive Tests (NDTs) such as Physical, Visual Check, Magnetic Particle Inspection (MPI) & Ultrasonic Test (UT) and Destructive Tests (DTs) such as V-notch Impact Test, Transverse Bend Test, Transverse Tensile Test, Hardness Test, & Macro Etching for Butt Weld and Fracture Test, Macro Etching & Hardness test for Fillet Weld. The NDTs are conducted at the respective stations of the job and DTs are performed in a well-equipped site Testing Lab.

4. **CONCLUSION**

The fabrication of the deck superstructure, initially taken up for segment assemblies AS34, 35 & 36, after completion, shall be a symbol of the latest state of the art welding engineering excellence, using plates in thicknesses varying from 10mm to 63mm, with the contemporary & mechanised welding processes and techniques like Submerged Arc Welding (SAW), Gas Metal Arc Welding (GMAW) and Flux Cored Arc Welding (FCAW), giving weldment the ultimate strength more than the parent metal. KRCL is duty bound and obliged to meet the engineering excellence & challenges for the fabrication of welded Deck Structure and will leave no stone unturned in completing this mission.
Though all the sections of the any engineering department are equally important to run the working of the office but the drawing section is most important. I don’t hesitate to say that It is the Heart of the Project which collects blood of rough data from field staff purify it in the form of planning & designing on papers and send to different organs namely field staff offices for execution after approval of competent senior officials of the project. The final outcome of this heart of the project is the all-round development and beauty of structures seen around us which is called marvellous sometimes.

To day high rise buildings, towers, long span bridges, tunnels, roads & other architectural structures which are seen now, were some time earlier planned & designed by a team of some expert officers and engineers, in drawing office. Some beautiful and wonderful structures once planned and designed so in drawing offices like The Taj in India, Tower Bridge of London and recently constructed Pir Panjal Tunnel on USBRL Project became history.

**How it works:**

If Field data is blood of this heart then The Team of JE, SE & SSE’S are the arteries, vessels and ventricles of it. All these components work day and night under the Commendable direction of AXEN, XEN, DY. Chief Engineer, honourable Chief Engineer and his Excellency CAO of USBRL Project of northern railway. Some times this team of JE, SE & SSE etc commits mistakes which are rectified with the help of seniors.

This team of drawing office collects field data like measurements and levels, prepare drawings, estimates, tender schedule, agreements etc. as advised by the senior officials of the project. Final bills and measurement books are technically checked in drawing office which is a check on excess and wrong payments to contractors by mistake. Completion drawings, Land Plans and Land Indents are also prepared in drawing office and finally sent to divisional office of Railways. Though these days Some PSU and Private reputed agencies have been engaged in construction work in Railways. The drawings and other related works are now being done by these agencies even then drawing office is as important as earlier.

**Future of Drawing Office:**

As I earlier said that drawing office is the Heart of the project, it is universal phenomena. As long as this heart beats this body of Project survives when it cease to beat the body of the Project cease to exist. No project can survive without it. It is equally true that Its location can be changed at any time any where but its existence is vital for any project.

Finally I would like to say that contribution of drawing section for erecting these beautiful structures for ultimate comfortable living of human being is as important as that of the rest of the engineering wings of the Project. Thus drawing Office is The Heart of the Project so keep it healthy.

**SOME QUOTES OF SRI SRI PARMAHANSA YOGANANDA**

If anyone acts on these thoughts, I thinks one will never face any problem in one’s life.

1. Learn to be calm and you will always be happy.
2. Do little things in an extraordinary way; be the best one in your line.
3. Whenever you mix with people, mix respectfully with love and sincerity.
4. Analyze what you are, what you wish to become and what shortcomings are impeding you.
5. Greater than the destructive force of hate is the compassionate power of love.
6. Understanding is the balance of calm intelligence and purity of heart.
7. Follow fearlessly the truth whenever you perceive it.
8. Constructive thought will absolutely, like a great hidden search light, show you the pathway to success.
9. He who masters his moods, becomes a more balanced individual.

By:-
RAJWANSI KOUL
OS/P
USBRL Project

By:-
BALDEV SINGH
SSE/DRAWING/ BAHL
USBRL Project
MUGHAL GARDEN VERINAG

The present name Verinag is probably the deformed version of VIRAH – NAG in Sanskrit. It is located about 26 Km from Anantnag and 78 Km South – East from Srinagar at the toe hill of Pir Panchal Range. The gushing spring of Verinag is the source of river Jehlum in Kashmir Valley. The distance from nearest Railway Station i.e. Hillar Halt Station to Verinag is just 6 Km.

Mughal Emperor “Jahangir” was highly impressed by the natural beauty of this irregular spring and its surroundings and built an Octagonal tank of sculptured stone around it in the year 1620 A.D. Seven years later his son “Shahjahan” constructed cascades and foundation in straight lines in front of spring and also hot and cold baths (Hamams) of which only ruins are now left. Its bountiful gushing water, open lawns, Mighty Chinars, Colour and fragrance of flowers against the back drop of a green forest are the bounties of nature one can be proud of. The garden covers an area of 5.30 ha. The spring is 54 feet deep in Centre having three steps of 18 feet height. The plate has been provided in the centre of the spring to prevent the excess/gushing flow of water. The spring having a discharge of 400 gallons/second, this spring is known to never dry up or over flow. The carvers for construction were brought from Iran. The main Engineer of the spring was Hyder Ali of Mysore. The out coming water stream in front of the spring has been constructed same on the pattern of Historic Taj Mahal Stream.

There are two stone slabs built into the Western and Southern walls of Verinag spring, on which prose in Persian language, in praise of the spring and the dates of construction of the tank and aqueduct, are inscribed.

By:-
Eajaz Ahmad Kawoosa
SSE/Works/Banihal
USBRL Project

The translation of prose in Persian language written on stone slab built into the Southern wall of spring is as follows:-

Lord of his age………
God be praised ……. made the Cascade and aqueduct flow.
This aqueduct reminds one of the aqueduct of Paradise.
By this Cascade Kashmir attained glory
The unseen Angel declared the date of Aqueduct:-
The aqueduct has issued from the heavenly Spring (1037 Hijri)
A Hindu shrine with Shivling is set up in one of the arches where the marigolds and rosebuds wreath the drab plaster walls. Pink indigo bushes and lilac wild-flowers flourish on the earthen roofs, and grow between the grey cornice stones; behind which the giant poplars whisper restlessly in the lightest breeze; while over the close, delicate, northern harmonies the pine woods brood sombre and remote. Then with a sudden burst of sound and colour, a band of newly-arrived pilgrims flock in to make their puja at the shrine.

Avantipora or Awantipur is a town in Pulwama district of Jammu and Kashmir. It is situated mid-way between Anantnag and Srinagar cities on National Highway NH1A, about 30 KM from Srinagar. The town is situated on the banks of river Jehlum. The name Awantipora has been driven from the king Awanti Varman’s name.

MUGHAL GARDEN VERINAG

Avantishwar temple located at Jawbhara in the Centre of Court Yard by a colonnaded peristyle is dedicated to Shiva on the banks of the river Jehlum (Vitasta). Less than a Kilometer away is Avantiswamin temple dedicated to Vishnu. The Vaikunta Vishnu illustrated as frontispiece is said to be found in this temple. The two temples are quite similar structurally. The walls of the entrance are ornamented with sculptured beliefs both internally and externally.

The ruins of temples constructed by Lalitaditya, the Brahmin emperor of Kashmir, are also located in Awantipora. The ruins of temples are protected and maintained by Archaeological Survey of India.

Railway Station, Awantipora
Railway Station, Awantipora is situated near Awantipora at Malangpora village about 3 KM from Awantipora Town.

Air Force Station, Awantipora
Indian Air Force Station, Awantipora is situated near Awantipora at Koil about 5 KM from Awantipora Town.

AWANTIPORA

Awantipora has a number of ancient Hindu Temples built by King Awanti Verman (AD 855-883), when he chose the site as his Capital.

THOUGHT

“If poverty is the mother of crime then want of sense is its father”

“The best laws should be constructed as to leave As little as possible to the decision of the judge”

“You can find all the reasons for not doing a thing or you can find some reasons for doing it. If the reasons for doing it are good then you have go to have the courage to try it, and work out the problems as they come up.”

“Two weapons in particular are important; prayer and knowledge. Prayer binds us to the goal of heaven, and knowledge fortifies the intellect with salutary opinions; each complements and guides the other.”

“Beauty in vain their pretty eyes may roll; Charms strike the sight, but merit wins the soul.”

“I thought a thought. But the thought I thought wasn’t the thought I thought I thought.’

By:-
Jameel Ahmed,
OS/P
USBRL Project
CHOKING

Few months back, I got a call from Satyam, the USBRL project office, Jammu for an emergency (suspected heart attack). In spite of doing my best, I think it must have taken me 20 minutes to be on the spot. On being there, I realized that it was a case of CHOKING; fortunately things were normal by the time. We were lucky, it could have been fatal. Nature's intervention this time was on our side. Therefore it is important to put few facts about choking for people to help them understand & react positively to any such event.

Choking in simple words is caused by blockage of our windpipe (respiratory tract) by some thing, mostly food; leading to difficulty in breathing & if it is not relieved quickly, either by itself or by intervention, it will cause brain damage & death.

The problem with bystanders is that many a times, they fail to make out the cause of patient’s discomfort & even if they do they don’t know what to do. Let us see the symptoms, so that we don’t waste the precious seconds.

**Symptoms include:**

- Panicked and distressed behavior.
- Inability to talk in complete sentences or at full volume.
- Frantic coughing.
- Unusual breathing sounds, such as wheezing or whistling.
- Clutching at the throat.
- Watery eyes.
- Red face.

**What to do?**

If the person is coughing forcefully, do nothing, coughing is likely to clear the airway; ask the person if he is fine? If he is able to speak, it is partial obstruction. Never give him water, as it is likely to go to airway causing further problem.

If the person is not pregnant or too obese, do abdominal thrusts:

- Stand behind the person and wrap your arms around the waist.
- Place your clenched fist just above the person's navel.
- Grab your fist with your other hand.
- Quickly pull inward and upward.
- Continue cycles of 5 back blows and 5 abdominal thrusts until the object is coughed up or the person starts to breathe or cough.
- Take the object out of his mouth only if you can see it. Never do a finger sweep unless you can see the object in the person's mouth.

If the person is obese or pregnant, do high abdominal thrusts:

- Stand behind the person, wrap your arms them, and position your hands at the base of the breast bone.
- Quickly pull inward and upward.
- Repeat until the object is dislodged.

**If the Person Is Conscious but Not Able to Breathe or Talk:**

1. **Give Back Blows**
   
   Give up to 5 blows between the shoulder blades with the heel of your hand.
3. **Give CPR, if Necessary:** if the obstruction comes out, but the person is not breathing or if the person becomes unconscious.

   Place fist above navel while grasping fist with other hand.

   Leaning over a chair or counter-top, drive your fist towards yourself with an upward thrust.

   **Helping one self, if no one is around.**

   Place the infant stomach-down across your forearm and give five thumps on the infant’s back with heel of your hand

---

**Prevention is better than cure**

In back of our mouth there are two openings, one for food pipe & one for windpipe. While we are eating windpipe is covered by a flap called epiglottis, thus preventing the food particles from going into the airway, if the closure is not proper the food will enter the airway & obstruct it.

**Causes of choking:**

In children choking is caused by incomplete chewing, attempting to eat large pieces, eating too much at time, eating hard food, candies, nuts etc; children can also be choked when playing with coins, marbles etc, which they tend to keep in mouth & it accidently slips into the airway. Adults can choke while talking or laughing at time of eating, eating in undue haste & eating while laying down. Alcohol is another big cause of choking, many people have died in pub or while partying, alcohol slows down our reflexes epiglottis fails to act in time.

Therefore remember the words of our elderlies, who always asked us not to talk or joke while eating, eat with ease, chew properly & be in right posture.

**Choking can be fatal.**

---

**TONGUE TWISTER**

You’ve No Need To Light A Night-Light On A Light Night Like Tonight,
For A Night-Light’s Light A Slight Light,
And Tonight’s A Night That’s Light,
When A Night’s Light, Like Tonight’s Light,
It Is Really Not Quite Light To Light Night-Lights With Their Slight Lights On A Light Night Like Tonight.

How Much Wood Would A Woodchuck Chuck If A Woodchuck Could Chuck Wood?
He Would Chuck, He Would, As Much As He Could, And Chuck As Much Wood As A Woodchuck Would If A Woodchuck Could Chuck Wood.

Mr. See Owned A Saw
And Mr Soar Owned A Seesaw
Now See’s Saw Sawed Soar’s Seesaw
Before Oar Saw See,
Which Made Soar Sore.
Had Soar Seen See’s Saw
Before See Sawed Soar’s Seesaw,
See’s Saw Would Not Have Sawed Soar’s Seesaw.
So See’s Saw Sawed Soar’s Seesaw.
But It Was Sad To See Soar So Sore
Just Because See’s Saw Saved Soar’s Seesaw!!

By:-
Usman ul Haq,
Nephew of Jameel Ahmed
OS/P
USBRL Project

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**Note:** In all conditions medical help should be called for, in case a higher medical centre is nearby one should go for it without wasting time, if the above procedure(which is called Heimlich maneuver) fails to relieve the patient, surgical intervention in form of tracheostomy may be needed.
Check List for Tunnel alignment

✓ Does the tunnel passes through the young mountains?
✓ Is there any intra thrust zone?
✓ Are there active and inactive fault/thrust zones?
✓ Where are the thick shear zone?
✓ Is rock cover excessive?
✓ Is pillar width between tunnels adequate?
✓ Are there thermic zones of ground temperature that are too high?
✓ What is the least rock cover or shallow tunnel beneath the gullies/river/oceans?
✓ Are there water-charges rock masses?
✓ Are there swelling rocks?
✓ Are Joints oriented unfavorably or is the strike parallel to the tunnel axis? Is the tunnel along an anticline(favorable) or syncline(unfavorable)?
✓ Mark expected tunneling condition and corresponding methods of excavation along all alignment according to chapter 7
✓ In which reaches, open/single-shield-double-shield, should TBMs be used in very long tunnels?
✓ In which reaches are conventional drill and blast methods recommended?
✓ Is it likely that a landslide-dam will be formed and lake water will enter the tailrace tunnel and powerhouse cavern, and so forth?
✓ What are the expected cost of tunneling for different alignment along their period of completion?
✓ What is the possible surveying error, especially in the hilly terrain?

Seepage during excavation of Tunnels

It may so happen that during tunneling alternate inclined beds of impervious (shale, phyllite, schist etc.) rock and pervious rock (crushed quartzite, sandstone, limestone, fault etc.) are encountered. Heavy rain/snow charge the beds of pervious rocks with water like an aquifer. While tunneling through the impervious beds into a pervious bed, seepage water may suddenly gush out. This flooding problem become dangerous where the pervious rock mass is squeezing ground due to excessive overburden. In two projects in the Himalayas, the machine and Tunnel Boring Machine (TBMs) are partially buried.

Seepage should be monitored near the portal regularly. The discharge of water should be plotted along the chainage of the face of the tunnel. If the peak discharge is found to increase with the tunneling, it is very likely that sudden flooding of the tunnel may take place with further tunneling.

Chimney formation.

There may be local thick shear zones dipping towards a tunnel face. The soil/gouge may fall down rapidly unless it is carefully supported immediately after excavation. There are chances of formation of a high cavity/chimney along the thick shear zone. The chimney may be very high in water-charged rock mass. This cavity should be completely backfilled by lean concrete.
Project Alignment

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