DESIGN FEATURES OF JAMMU-UDHAMPUR-SRINAGAR-BARAMULLA RAIL LINK PROJECT

Presented By

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INTRODUCTION

Construction of Jammu-Udhampur-Katra-Quazigund – Srinagar-Baramulla new rail link is the biggest project undertaken by the Indian Railways in the mountainous terrain since independence.

Challenges in the construction of a Railway line through the hilly terrain start right from the conception stag itself. There are various constraints such as allowable maximum speed, high gradients, sharp curves, stations to be kept for optimum utilization, safety and minimum maintenance need in future in addition to the basic need for providing the link with the rest of the network. Projects in mountainous regions are associated with special features such as deep cuttings, high embankments, tall piers and long span bridges across deep gorges and fast flowing flash flood rivers with big boulders and unusually long tunnels etc. These challenges are enhanced in view of the terrain in young Himalayas, where geology is poor and changes occur frequently.

Surveys undertaken in the region have been a fascinating experience. The territory from Salal to Quazigund with virtually no habitation, no approach roads or even rudimentary pathways through dense jungles without any light or water connections, is a survey storey in itself. In this part of the project, the engineers are expected to tackle tunnels for over 50 % of the length with the longest being about 10 km across Pir Panjal range. The tallest bridge is about 360 m above bed level and of a 505 m in length (Single Span) is also to be tackled in this reach over river Chenab. The project is a challenge to the Engineers of India in general and to the Railway Engineers in particular.

THE LINK

Indian railways are linking the Kashmir Valley with rest of the country by a rail link between Jammu and Baramulla. This project is perhaps the most difficult new railway line project undertaken on Indian subcontinent. The terrain passes through young Himalayas, which are full of geological surprises and continous changes , due to lying in the thrust region. The alignment of the project is shown in figure 1. For execution purpose, project has been divided into 4 sub-section. Construction activities are complete on Jammu-Udhampur section (54 km) and are in progress in the balance 3 sub sections (287 km).

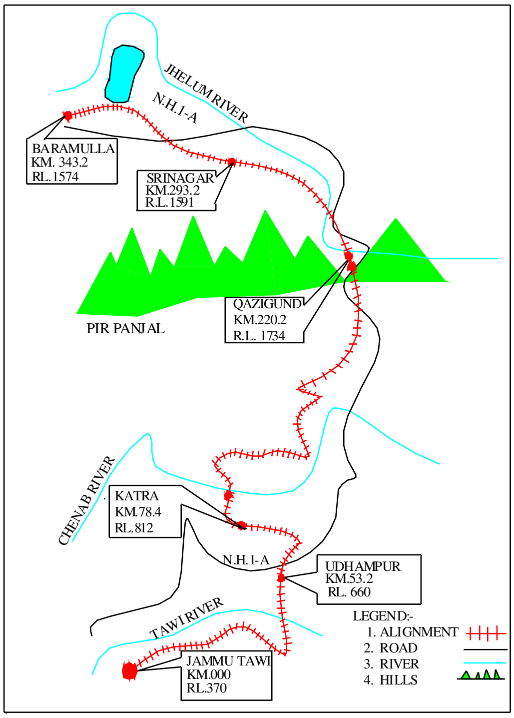


Figure 1 Alignment of Project

The new broad gauge single line is being constructed with a ruling gradient of 1 in 100, degree of curvature is restricted to 2.75° (5° in Jammu-Udhampur Section), so as to achieve a speed potential of 100 kmph. The total route length is 340 kms. involving 413 lacs cum of earthwork, 228 Km of Access Roads, 738 nos. bridges (out of which 180 are major bridges) and 129 kms. of tunneling. This presentation brings out the various design features being taken in to consideration for this project, through improvisation and innovations, depending upon the specific site requirements. This has resulted so far in the successful construction of 42 m. high embankment, 75 m tall pier, 102 m long PSC girder with cantilever construction and more than 15 km of tunneling work in bouldery and flowing soil conditions, where free standing time is even less than an hour.

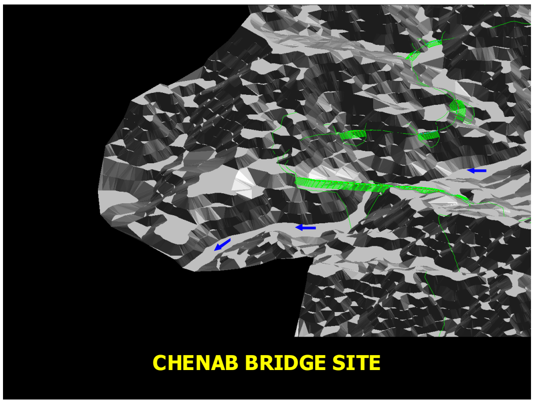
1) SURVEY

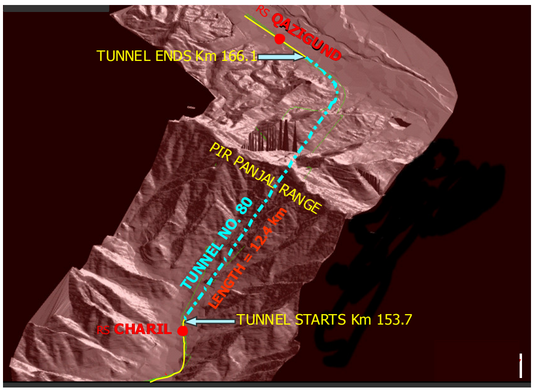
The alignment of Jammu - Udhampur – Katra – Quazigund – Baramulla rail link project passes through highly undulating terrain, especially in Katra – Quazigund section. Construction activities are already in full swing in Udhampur - Katra and Quazigund - Baramulla section. Beyond Katra (Km 30) up to Quazigund (Km 167) a part of alignment from km 50 to 120 falls in thick forest cover with no habitation and absence of even trekking paths. Due to difficult site conditions, initial alignment proposed was marked on the topo sheet of 1 in 25000 scale with 20 m contour interval, based on satellite imagery and aerial photographs.

To study the alignment and terrain in detail before proceeding any further, this was converted into a Digital Terrain Model (DTM) and proposed alignment was transferred on it. This Model is being used for:-

a) Checking and refinement of the alignment i.e. at a location from model it could be visualized that a stretch where earthwork was being planned, earthwork is not feasible as embankment slope of 2(H) : 1(V) will block the drainage.

b) Planning of project logistics before venturing into the actual execution in the field.





As the alignment was drawn on topo sheets with 20 m contour interval based on satellite imagery, which is feature specific with an accuracy of 100 m in XY direction & 200 m in Z direction, it was planned to go in for Aerial Survey using GPS. It was also planned to fix control points along the alignment during aerial survey.

The Aerial Survey will provide maps at 1:5000 scale with contour interval of 2.5 m at an accuracy of 15 cm in XYZ direction, except in the thickly wooded reach from Km 50 to 120. For Km 50-120, aerial survey was planned to be supplemented with Airborne Laser Terrain Mapper (ALTM), which will give topo maps with 1 m contour interval. Thus, through aerial survey, maps on a bigger scale of 1:5000 with 2.5 m / 1 m contour interval and co-ordinates of control point became available which greatly reduced the efforts in transferring the actual alignment on ground using total station and/or theodolite /level.

In addition to the above, intensive geophysical, geotechnical and hydrological studies have been conducted to look in to various aspects of the design.

2) EARTH WORK

The project involves 413 lacs cum of earthwork. Such a huge quantum of earthwork posed numerous technical and logistical challenges. These challenges have been met by going in for:-

1. Construction of 42 m high embankment

2. Reinforced Soil to reduce embankment slopes, wherever stable slope of 2(H) : 1(V) for the embankment was not available or obstructing / blocking natural drainage or even national highway in a case.

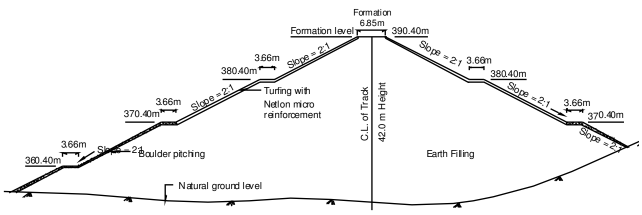
3. Construction of viaduct where good quality earth was not available, called as preferred fill materials in railway parlance.

4. Construction of cut & cover at locations where cut slopes were not stable.

5. Provision of blankets to solve the formation problem.

2.1) CONSTRUCTION OF 42 MTRS HIGH EMBANKMENT

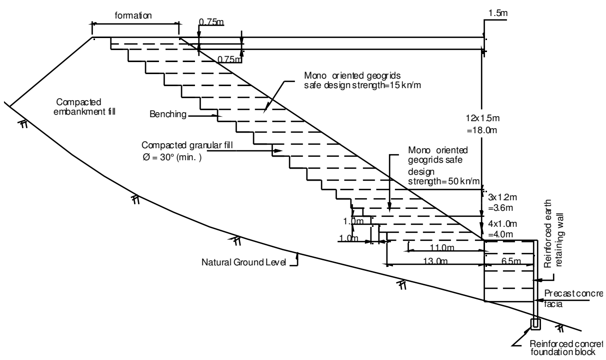
Cross section of bank is shown in fig. The bank was designed by carrying out slope stability analysis based on Bishop's Modified Method. The minimum factor of safety for circular slip surfaces is calculated and slopes changed to have adequate factor of safety. Effective stress analysis was used.



A slope of 2 H : 1(V) was provided, with berm of 3.66 m at every 10 m height. All the fill area is demarcated with the help of toe lines and reference lines. All vegetations was removed and proper benching done as the natural ground is in slope. Earth is brought in tippers/dumpers from cutting /borrow area and is spread in layers of 400 mm (loose) with the help of bulldozers. Boulders more than 200 mm in size are separated out. The compaction was done with vibratory roller with the moisture contents within allowable moisture contents at MDD so that minimum 98% of maximum dry density corresponding to modified proctors compaction is achieved. The fill area is divided in 3 parts. In first part earth is being dumped, in the second rolling is in progress and in third part testing of density is done. Further, to check erosion at some locations, turfing has been done with netlon micro reinforcement provided on top & sides and grouted pitching below it.

2.2) REINFORCED SOIL

Reinforced soil is compacted soil reinforced with geogrids. Geogrids are stretched and oriented grid structures made of HDPE polymer. The geo grid reinforced embankment has been designed by "Limit Equilibrium Method" using Jewell's Chart. The safe design strength of mono-oriented geogrids used in the design is 15 KN/m and 50 KN/m. The length of geogrid is 11m with total of 21 layers with spacing varying from 0.75m to 1.5m.



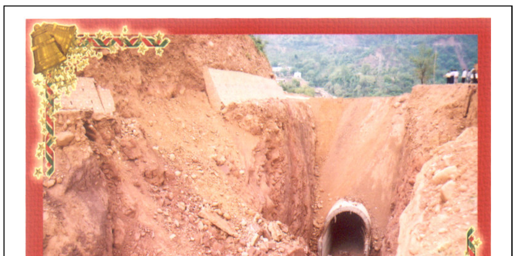
As per the construction scheme, granular fill material is laid and compacted in layers of 200mm thickness. Then mono oriented geogrid is placed at the designed spacing and then further 200 mm thick fill material is laid over the geogrid layer and compacted. It is ensured that geogrid alignment is horizontal. Polymeric micro reinforcement grids duly anchored are used for vegetation for erosion control of the slopes.

2.3) NON-AVAILABILITY OF GOOD QUALITY EARTH

Good quality earth is defined to be the preferred fill material, whose technical requirement are laid by GE Directorate of RDSO. Normally, the fill material should be course grained soils with fines less than 50%, liquid limit less than 35% and plasticity index less than 15%.

At some locations on Jammu – Udhampur section, It was found that good quality earth is not available and leading of earth from distance would have been un economical. Thus, instead of embankment, construction of viaduct was planned in that stretch.

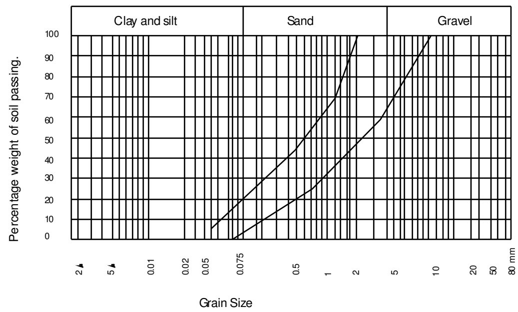
2.4) CUT & COVER CONSTRUCTION



At many locations where it was apprehended / found after Geo- technical investigation that cut slopes will not be stable or site was not permitting to go in for cutting with 1(H):1(V) slope, construction of cut & cover in small stretch was resorted to. This has not only made construction activity safe but also eliminated the future safety concerns.

PROVISION OF BLANKET

The stresses caused by traffic are transferred from sleepers to sub grade soil through ballast and sub ballast/blanket. Formation like any other structure deforms under the traffic loads. If the deformations are small and terminating the formation is stable. If the induced stresses exceed the “threshold stress value” the formation fails. Large and permanent deformations in sub grade soils disturb track geometry and accumulate rain water. Stresses at the bottom of ballast may exceed the strength of subgrade soil resulting in puncturing of ballast and formation of ballast pockets.



Notes :-

1. No skip grading to be there.
2. Plastic fines should not exceed 2-5%
3. Non plastic fines allowed upo 8-12%.
4. Uniformity coefficient (D_{60}/D_{10}) in no case less than 4 peferably more than 7.
5. The coefficient of curvature ($D_{30}/D_{60}XD_{10}$) to be within1 and 3.

Numerical analysis using Finite Element Modeling carried out in RDSO, Lucknow, in collaboration with IIT Kanpur, have shown that sub grade stresses reduce only marginally (4% to 6%) with the increase in rail section or sleeper density. However, the induced stresses reduce very drastically with the increase in depth of construction which is the sum total of depth of ballast and depth of sub ballast/blanket. Even experience gained on leading world railways show that provision of blanket layer of adequate depth is the most effective and economical way of reducing sub grade stresses to an acceptable level. Hence provision of blanketing as per requirement has been made.

3) BRIDGES

In the hilly terrain, the construction of the bridges has been a difficult task and posed numerous challenges. Apart from the complexity of design, the construction of these bridges requires great amount of planning and special techniques. The topography of the area resulted into long girders, combined with large pier heights. This part of paper presents various Design features and technical solutions adopted in construction of these bridges, some of which are being adopted for the first time on Indian Railways.

3.1 PLANNING FOR BRIDGES

A careful selection of alignment is being done to ensure shortest possible height and length of bridges, keeping in view the ruling gradient of 1 in 100 (compensated). The choice of alignment is most important for planning of bridges in hills. Detailed geological investigations were carried out. Geological features consisting of variable strata of sand rock, soft and hard shale, boulder-studded soil, etc. have also influenced the bridge lengths and span arrangements. The long spans were necessitated as a result of fixing pier locations in the middle of the gorges/streams so as to avoid constructing piers on sloping banks. This aspect itself called for cantilever method of bridge construction in some bridges. This method has the added advantage of elimination of costly centering and false work and reduced requirement of shuttering and fast pace of construction. The design of the bridges in question has been fairly complex and it was an elaborate task. Detailed Design criteria were developed which were bridge specific. The long spans and tall piers associated with highly seismic characteristics of the area have made the designs cumbersome and tricky. The bridges have been designed for Modified Broad Gauge (MBG) loading - 1987 as per Indian Railways Bridge Rules. The design complexities were further compounded by the stringent requirement of maintaining 5% residual compression in superstructure at all stages of construction, which was finally relaxed to 'no tension' condition.

3.2 Seismic Design considerations

The bridge sites lie in the Seismic Zone IV & V as per the current Seismic Zoning Map of India contained in IS:1893-1984. The data show that seismic events having Richter's magnitude greater than five occur at frequent intervals in this area. The design of bridges with pier height up to 30 m has been done by using seismic co-efficient method as given in IS 1893-1984. The values given by this method have stood the test of recent earthquake (year 2005) of 7.6 on Richter scale having epicenter near Muzzafarabad. For the tall piers, site-specific spectrum has been adopted. The work has been entrusted to earthquake engineering department of IIT Roorkee.

The following additional seismic related measures have been adopted to reduce the impact of earthquake: -

a) Bridges have been mainly provided with POT-PTFE bearings and elastomeric pads attached to the vertical surface of the concrete projections on top of the pier caps for seismic restraint devices.
b) Rigid structures absorb more seismic energy requiring a design for larger seismic forces than a comparatively flexible structure. Some innovative shapes of abutments have been adopted to make them substantially flexible in order to achieve desired results. Abutments were conceptualized as consisting of a RCC tank with 3 walls and a base and separate pier .The presence of a large tank with soil along with a base shear key gives effective resistance to longitudinal sliding. Size of the pier has been kept to a minimum by providing large amount of reinforcement in order to keep them more flexible. Additionally the piers have been tapered in order to further reduce the stiffness, thereby reducing the seismic forces experienced by the bridge.
c) The structures have been blended with the incorporation of Ductile Detailing. Special confining reinforcement in the form of closely spaced stirrups/ties is

expected to impart reserve strength to the joints and connections where formation of plastic hinges are anticipated.

d) STAAD III software has been used for dynamic analysis for the idealized structure consisting of springs and member end release after a few simplifications. The longitudinal and transverse behaviour has been analyzed separately so as to reduce the amount of computations and margins of errors.

3.3 Geological Investigations

Trial bore holes using NX size heavy-duty diamond rotary core drills were carried out at each foundation location up to a depth of about 1.5 times the width of foundation below the founding level. The soil samples collected were tested for bulk density, specific gravity, uni-axial compressive strength of rock and chemical analysis. The standard penetration test were carried out at every 30 cm depth. The founding strata consisted mostly of alternate bands of shale, sandstones, & boulder studded soil matrices. Hence, in most of the cases, open raft foundations were adopted.

3.4 Construction of foundations

Excavation for foundations was done by drilling and blasting and mucking was done by hydraulic excavators. Well foundation were chosen for bridge No. 26 where aquifer was encountered during excavation and for Tawi Bridge because of heavy floods encountered in river Tawi and scouring expected in the upper strata of the river bed which consist of boulders. Pile foundations are not suitable for the boulder studded soils of the hilly region and hence were not used.

The cutting edge was fabricated in parts and then placed on the location and welded. Thereafter well curb was erected and concrete placed. Steining in M-25 grade of concrete was cast with the help of crane and concrete buckets and then sinking started. Blasting had to be resorted to dredge out conglomerate and sandstone which was done very cautiously so as to avoid damage to the cutting edge and steining. Dredging was started with crane and grab bucket but later JCB loader (with a small back hoe) was lowered with the help of crane in the Tawi well which made dredging very fast.

3.5 Design of Foundations

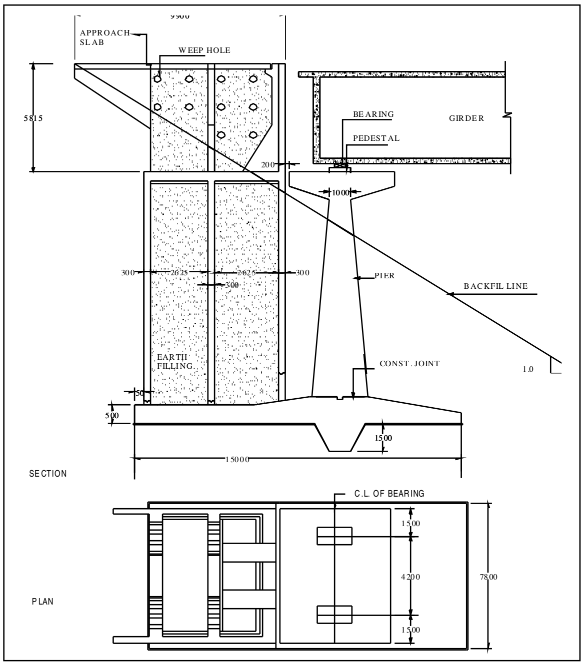
Open foundations have been designed in the usual manner. Some of them have become abnormally large due to the added problem of uplift of foundations owing to the large seismic moments. Minimum 75% contact area at the base has been ensured as per the provisions of IRS Codes for rocky strata.

The well foundations have been designed by and large as per the provisions of IRC:78. The thickness of steining has been restricted to 1.25 (D/8 + H/100) subject to a minimum of 1.2m, wherein D = external dia of well and H = height from bed level to founding level. Stability analysis for the well has also been done.

3.6 Construction of Piers

To avoid construction joints piers are being cast using slipform construction. Slipform construction essentially consist of hanging shutters supported by yoke legs which in turn are supported on the radial beams. This whole assembly continuously move upwards with the help of the jacks taking reaction from the jack rods en sleeved in pier wall and resting on the bottom of pier. For tapering the radius is reduced by turn buckles and sliding shutters. After completion of the piers the pier caps in M-40 grade of concrete were cast.

3.7 Design of Hollow RCC Piers



The hollow RCC piers with continuously decreasing diameter and taper at uniform rate have been constructed. These are additionally checked for a temperature gradient of 20^0 C between the inside and outside faces of the pier shafts. Ventilation holes covered with GI wire mesh are being provided at regular intervals to reduce the temperature gradients.

3.8 Design and Detailing of Abutments

A unique design of abutment has been adopted. These innovative abutments are provided with a small size pier to take the vertical load and a reinforced cement **concrete wall tank filled with earth** for counter acting the horizontal forces, both supported by a common raft foundation with a provision of shear key at the base. This tank filled with earth adds to the weight and helps in overcoming the problem of sliding and the slender pier which is quite flexible reduces the seismic forces.

3.9 Cantilever construction of Dudhar , Tawi, Ringhal and Sardan bridges

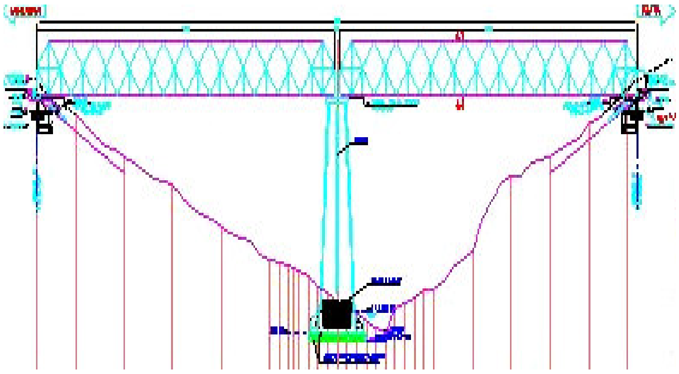
As per the cantilever construction sequence, first of all pier head units about 10.5m long are cast over the pier cap and after attaining of sufficient strength the pier head segment is prestressed longitudinally. Then the cantilever construction equipment is erected over pier head unit and construction of cantilever segments starts. After casting of cantilever segments is complete, end span on either side is cast on staging and after concrete attains sufficient strength the end span prestressed continuity cables are stressed. Then the vertical holding down pre stress cables are cut off and packing plates removed so as to transfer the loads to the permanent bearings. Thereafter central segment for closure pour in the centre is cast on shuttering supported from the two cantilever tips and after concrete gains strength the central span prestressed continuity cables are stressed.

DUDHAR BRIDGE

3.10 FEATURES OF BRIDGE NO.20 IN UDHAMPUR- KATRA

(Largest single simply supported span & tallest pier on Indian Railways)

Br. No. 20 is situated across Jhajjar Khad at 20 Km from Udhampur on Udhampur-Katra section. This bridge consists of 2 spans of triangulated truss girders of span 153.4 m each. It consists of one Central pier and two abutments at ends. Central pier is 90 m high and is resting on open raft foundation. Both the abutments are resting on well foundations. This bridge is crossing a local khad named Jhajjar Khad, which is approx 125 m deep gorge.

Bridge No. 20 (General Elevation) View of Pier during construction

3.10.1 GEOLOGY

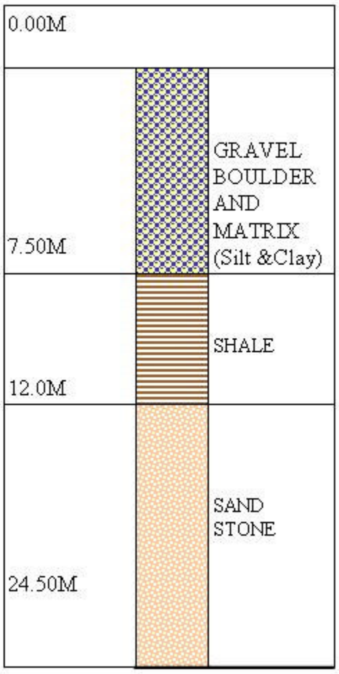
The terrain at site is hilly. The slopes of both the approaches are gentle to steep. The material constituting the steep bank slope comprises unconsolidated sediments. Bed rock is nowhere exposed and lies buried under thick cover of alluvial deposits comprising pebbles, cobbles and boulders set in sandy matrix with occasional thin pockets of silty matrix. The nallah bed is covered with boulders, gravels and sand.

3.10.2 CENTRAL PIER

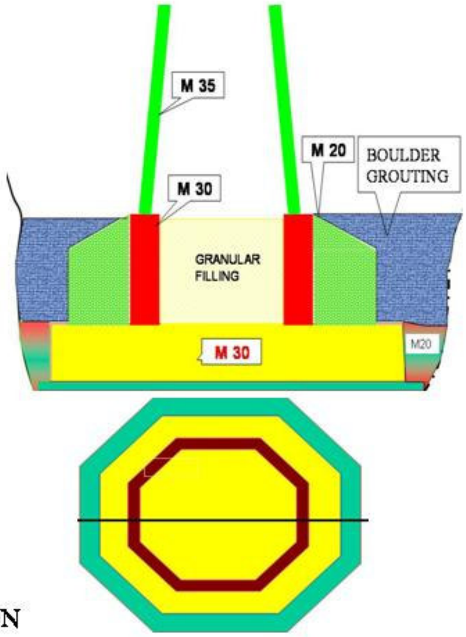
Strata at pier location is sand stone at founding level. Foundation of central pier is 27m wide RCC (M-30) open raft foundation. Strata at pier location is sand stone at founding level (Shown in bore log). It has been designed in octagonal shape for better stability in all directions.

Raft foundation 2.6 m thick is resting on 1.0 m thick PCC. Entire over-excavated portion between foundation/PCC and adjacent rock has been filled with concrete M-20 for better anchorage of foundation with adjacent rock. In order to provide better stability to pier, extra weight in terms of counter weight concrete all round the pier over foundation (up to 6.0 m height) has been provided.

Space inside the pier has been filled with granular material with cement grout unto approx 9.0 m height for still better stability. To enhance further anchorage of pier and to provide stability to cutting slope annular space has been filled with boulders & grouted with concrete. About 2.5-3.0 m of depth of foundation has been embedded in good rock. Pier of 93m height is resting on 6m high pedestals emerging from the foundation.



BORE LOG AT PIER LOCATION



6.0 M HIGH PEDESTALS OVER RCC RAFT

The outer diameter of pier at bottom is 18m, whereas at top it is 10m. Wall thickness is 500mm, uniform except in top 5m. In top 5m wall thickness increases from 500mm to 1750mm. Pier has been constructed in M-35 concrete. Foundation has been designed for safe bearing capacity of 100 T/sqm. Foundation has been laid on sand stone at about 10m below riverbed.

Foundation has been constructed in M-30 grade concrete. Construction of foundation 2.6 m thick has been done without any cold joint. Two horizontal construction joints were given in preplanned way. Pouring sequence was decided well in advance to avoid the cold joint and concreting process was done in such a way that next layer was laid before the previous layer was set.

3.10.3 Abutments

Both the abutments rest on well foundation and the pitching level of cutting edge is at a depth of 12 to 25m from original ground level. Through soil investigation it has been observed that strata at the locations of both the abutments is gravel boulder matrix (sand, silt and clay).

At Udhampur approach of bridge there is tunnel, however at Katra end approach there is deep cutting, the depth of cutting being of the order of 18-20 meters.

LOCATION OF ABUTMENT A1 AT UDHAMPUR END

LOCATION OF ABUTMENT A2 AT KATRA END & ASSEMBLY AREA

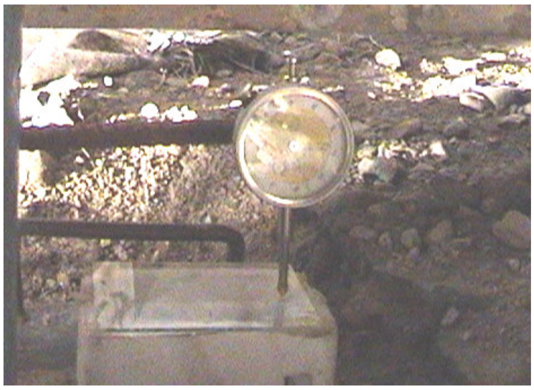
Total depth of abutments including well foundation is 14-25m. Strata is conglomerate unto 100m from the ground level. Well foundation has been provided to reduce the area of cutting and to transfer the load at greater depth so that pressure line starting from bottom of foundation should remain at much below the slope which is very steep. Well foundation of abutment have been designed for end bearing only without considering any wall friction. The bottom plug of this well foundation has been designed as RCC raft. The well is double-D type rectangular in shape with over all dimensions 8.5mx10.5m and 16m deep.

CUTTING EDGE OF ABUTMENT WELL

STEINING OF ABUTMENT WELL

While designing the well, safe bearing capacity of soil has been assumed as 50 T/sqm. This bearing capacity has been checked by plate load test inside the well.

REACTION LOADIND FROM CENTRAL PORTION OF CUTTING EDGE



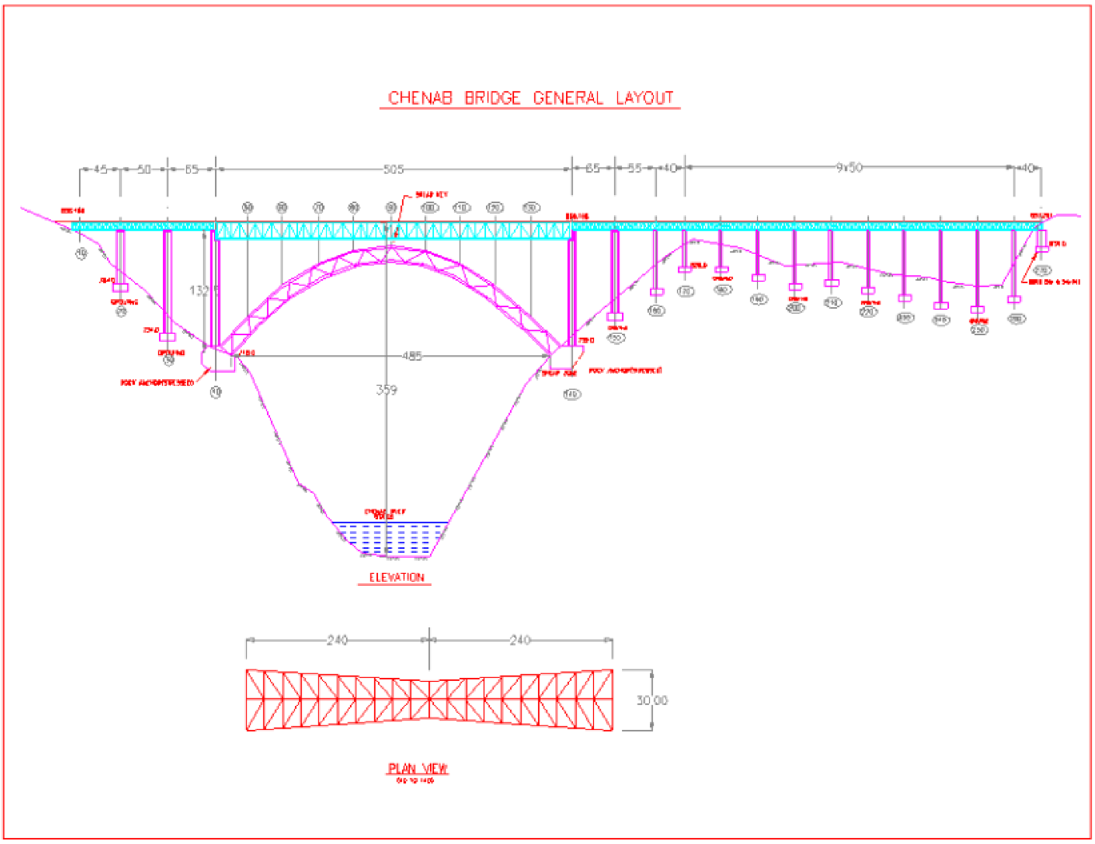
GAUGE TO MEASURE SETTLEMENT

Above bottom plug, well has been filled with clean sand and covered with CC top plug. At top, 1m thick well cap has been constructed. Abutment is emerging out from well cap in the shape of counter fort of walls connected together. At top there is abutment cap of 1m thickness. Thus, total height of abutment is 22m.

Both the approaches are very steep and highly erodible in nature. To reduce the possibility of erosion catch water drains has been planned in such a way so as to drain of all the water away from bridge location. Catch water drains have been provided with steeper slopes to ensure to self-cleaning. Some breast walls have been constructed across the bridge on slopes to retain the earth. Area up to about 30m from abutment is proposed for boulder pitching. Weep holes have been provided at every 1 m interval to drain off water and to avoid any hydro static pressure from behind the pitching and breast walls. Further to reduce the erosion, seeds of *dedonia viscosa* have been planted on the slopes which are showing good results. At bottom, riverbed shall be canalized with the help of retaining walls backed by wire net trungers.

The 2x154 m open web through girders are to be launched by Cantilever method. A launching girder of 68 m length will be attached in front of first span. Both the main girders will be connected by a stitch girder. The complete assembly will be pushed together with hydraulic jacks .The forces have been worked out for different stages of launching. 200 MT of steel has been provided extra to strengthen the first span to cater for the erection forces.

Special Design Features of Anji & Chenab bridge



Bridges over river Chenab and Anji Khad are being constructed in the reach from km 30 to km 120 (total = 90 km) between Katra-Laole involving almost 70 kms of tunneling work along another 15 km of bridges.

Chenab bridge will be the highest steel arch railway bridge, at 359 m (from bed level) higher than Eiffel Tower by 60 m**.** The other one at Anji will also have a height of 189m from bed level. These are situated in Seismic Zone V with extremely high wind speeds. The designed wind speeds have been taken as 220 kmph. The slopes at location of the bridges are of the order of 45-50 degrees on one side and vertical to sub-vertical on the other side such that the placing of any piers would have been rendered impossible, added by the existing deep gorge.

These bridges are designed in the shape of trussed steel Arch with span of 505m & 260m respectively. For Chenab Bridge, the arch proposed is a two- rib arch made from large steel boxes. The chords of the trusses will be sealed steel boxes, filled with concrete to assist in controlling wind-induced forces.

The various special features related to the construction of these bridges are as follows:-

- Highest structural steel arch Railway bridge
- Since no precedence of design and drawing for such bridge exists, railways have gone in for a turnkey contract.
- Listing of various national and international codes applicable for design and construction of the bridges (IS,BS,UIC,ASTM,AREMA)
- Special provision for wind tunnel testing
- Bridge is being designed with adequate redundancy
- Designed for 120 years life with mechanized provisions of instrumentation
- Site-specific spectra done by IIT, Roorkee considering various modes of vibration for the bridge is falling in Seismic Zone V.
- In view of the extra ordinary high designed wind speeds of 220 kmph, physical topographic models of the site are tested in a wind tunnel laboratory being carried out by FORCE Technology, Norway
- Designed for fatigue consideration in consonance with requirement of BS 5400.
- Bridge to be designed and checked adequacy for various stages of construction so that partially completed structure is sturdy enough to resist the effect of wind/earthquake and unforeseen forces.
- Designed for blast loading as per IS 4991-1968.
- Provision of permanent mechanical cages for carrying out necessary inspection of trestles/piers.
- Provision of 3-year maintenance period along with further 3 years as defect liability period.
- Provision of auxiliary/cable way bridge for launching the bridge segments
- Various instruments such as anemometers, accelerometers, temperature controls and central monitors to monitor effects of wind and seismic loads in producing strains and loads and automatic comparisons with pre-fixed limits.

Structural System

The bridges at Chenab and Anji are two-rib/three-rib arches, made from large steel trusses. The chords will be sealed steel boxes, internally stiffened by filling with concrete, which will help in resisting wind induced forces. No internal access to the boxes will be available. Aesthetic merit of the bridge has been taken in to consideration for a design, which will be in consonance with the nearby environment.

Codes and Design Loads

IRS standards will have the priority as regards both applicability and loads consideration. Concrete bridge code and bridge rules also shall be applicable in their area of concern. Adequate supplementary help shall be taken from UIC, BS and other international standards.

- Wind loads taken after testing carried out in the wind tunnels using models.

- Fatigue assessment of arch members done using trainload spectra specified as per BS 5400.
- Load combinations taken as per the provisions of IRS bridge rules.

Design Standards

Most of the parameters are tested as per the BS 5400 as against IS codes since BS 5400 provides comprehensive coverage on structural element types being used in the design of these 2 bridges such as box girders having torsional and distortional stresses and high strength grip connections.

BS 5400 also offers more coverage on highly stiffened steel panels such as being used in these bridges, which have related issues of redistribution of stresses at ULS and design of stiffeners and web panels including panel and sub panel buckling. It also provides an integrated set of design and workmanship standards.

BS 5400 also provides more robust wind loading provisions, greater reliability for combination of loading actions, fatigue action and it is also actively updated and monitored in accordance with the latest theories and research. Since IRS is primarily intended for simply supported bridges with spans less than 100 m and in the proposed bridges, continuous spans are to be used where BS 5400 has been extensively used and calibrated, BS 5400 is much more relevant to the design.

Design Methods

Limit state design specifications of BS 5400 Part III have been adopted for the whole bridge including smaller supported deck spans. Design is being carried out by large displacement analysis using RM7/GT-STRUDEL, internationally recognized software, appropriate for such a complex structure.

- Wind Tunnel Testing – for Chenab bridge, is being carried out by FORCE technology, Norway to assess topographic effects of the site on he design wind speed, occurrence of vortex shedding, flutter and effects of gust-buffeting on the structure.
- Wind tunnel testing- on the Anji Khad bridge is carried out by RWDI, Canada.

Structural deformation limits are calculated using UIC codes for lateral motion. Checks are also carried out with regard to passenger comfort. Vertical movement under service loads will be checked by IRS provisions.

Materials – M35 concrete is being used as per durability considerations. Grout specification shall be decided after completing geo-technical investigations.

Bearings- Minimum bearings are used on the approach viaduct by using continuous construction, which will help in maintenance and inspection effort.

Other Features

- Deck fencing with provision of pathway has been proposed.
- Construction tolerances will be as per BS 5400:Part VI and IRS Specifications.
- Preset and pre-camber for the deck will be specified on the fabrication drawings.
- Maintenance by ladder access for all elements is mandatory. The under side of the arches will be accessible by means of riding cradle.
- Lightening protection will be available to all the parts of the structure.
- Location of sensors for continuous monitoring of the bridges will be linked to the overall railway monitoring system. IRS provisions will be given priority over BS 5400 in case they are more restrictive in nature.

Specific Loading Provisions

Rail – IRS loading incorporated in to BS 5400 for fatigue assessment.

Thermal- BS 5400 duly adopted for Indian climatic conditions.

Seismic – IS 13920 and other appropriate provisions for RCC seismic detailing design along with review AASHTO, Euro codes and published guidelines to achieve best practical results. Plastic hinges are to be avoided in the main legs of the arch.

Blast- Partial load factor for this has been taken as fe=1.It contains redundancy of critical members, operating at reduced levels of efficiency and possible events and repair time schedules.

In case of Chenab Bridge having arches composed of two truss ribs with each rib containing 4 chords, redundancy level is being able to sustain the loss of one or two chords. In case of Anji khad, where arch is made up of three solid box ribs, redundancy has been provided by sustaining the loss of an entire rib.

4) TUNNELS

Tunneling through Siwalik ranges of Himalayas is an extremely arduous and hazardous task. This part of paper gives an overview of the planning, investigation and design, procedure for alignment control, methods of tunneling and major difficulties faced so far along with their solutions.

4.1 **Planning, investigation and design of support system**

Railways engaged RITES, NHPC and WAPCOS for geo-technical investigation and provide consultancy on various aspect related to const. methodology design of tunnel support system, for Jammu – Udhampur – Katra section. Geo – physical survey by seismic profile, field and laboratory testing of soils and rocks were carried out. Further, geological profile were confirmed through core drilling. At many locations, geo–technical investigation revealed that strata comprises of pebbles cobles and boulders (up to 2 -3 m.) embedded in silty, sandy matrix, complete loss of return water indicating pervious nature of strata, seismic velocity indicated that there is no firm bed rock, co recovery very poor and RQD has nil. Based on known geological condition, materials properties and construction procedure, tunnel support system was divided into five classes i.e. good, fair, poor, very poor, over burden. Due to weak geology, it was decided to provide permanent steel support all along the length of tunnel along with provision of 300 mm thick concrete lining.

4.2 Procedure for alignment control.

Before commencing the tunneling work, reference points were marked on ground and protected against damage from any construction activity. Relative position of reference points at both ends were determined in term of northing, easting and reduce levels. For this purpose triangulation is done and also closed traverses are run along the alignment. The large overburden with thick forests and steep hills made this quite difficult. All permanent reference point are being maintained and check for time to time for accuracy. In day-to-day checking, one second theodolite has been used where as periodical checking of alignment as well as permanent reference point has been done by Total Station. A slight difference between centre line of track and centre line of tunnel is kept in tunnels in curves to account for the tilt of the rail vehicle.

4.3 Method of tunneling

Key factors, determining the method of tunneling, are the size and shape of tunnels, equipments available, conditions of geological formations and extent of supports needed. So far on Jammu – Udhampur – Katra section, use of Tunnel Boring Machine (TBM) was not preferred because of, high initial cost of the equipment, small lengths of tunnels, long commissioning and decommissioning time of TBMs, the requirement of D shaped cross section for railway tunnels and non-availability of indigenous technology. Thus, the conventional method of tunneling by drill and blast has been used. At locations where soft ground was encountered the heading and benching method/ multi drift method using single / double ribs along with fore polling, grouting was adopted.

4.4 Problems faced and their solutions

a) **SLIPS ENCOUNTERED**

Work at Udhampur end of T.No. 1 on Udhampur – Katra section was started with following scheme.

1. Open excavations from Ch.2180 – 2220 i.e. 40 m long.

2. RCC cut & cover from Ch. 2220 – 2290 i.e. 70 m long.

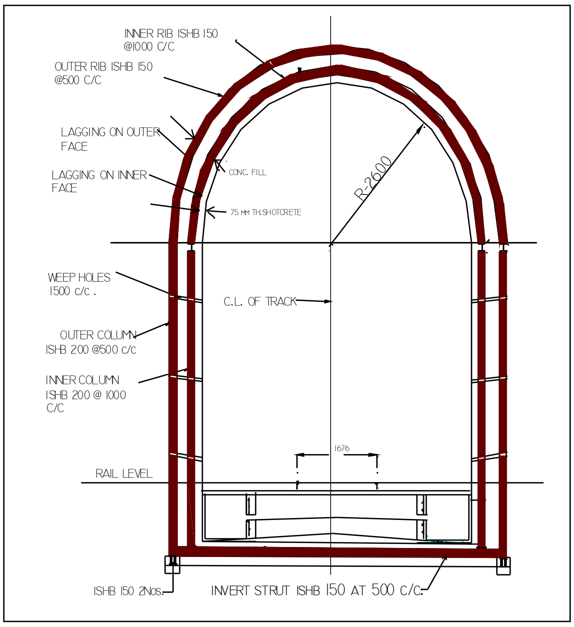
3. Tunneling beyond Ch.2290 (Depth of cover at Ch.2290 = 22m)

Accordingly excavation was started between Ch.2180–2290. However, during rains the overlying loose strata on the LHS of alignment flowed over the underlying hard strata & the flowing material filled up the already excavated stretch between Ch.2240 – 2280. Slip got extended up to 80m on LHS of alignment. Due to this slip const. scheme was revised as under:-

1. Cut & cover using single/double rib from ch. 2180 – 2270 i.e 90 m long.

2. Tunneling beyond ch.2270.

However, while doing excavation beyond Ch. 2218 it was seen that slope on LHS were so unstable that virtually it was impossible to concrete for cut and cover, due to continuous falling of loose material. In view of this double rib construction methodology i.e providing outer rib @ 500m c/c, inner rib at @ 1000mm c/c & filling M-20 concrete in between the ribs was adopted. Lagging was placed on outer face of outer rib & inner face of inner rib. This methodology made construction of cut & cover possible in this stretch & accordingly cut & cover with double rib was done upto Ch.2250 (i.e. for 32m). This scheme was executed taking 2m/5m stretch at a time. Fig. 4 depicts X-section followed with double rib in cut & cover



b. **Tunneling through bouldery strata.**

On Udhampur end of T.No. 1 , Udhampur – Katra section , in a part length , the strata comprises of boulders embedded in sandy / sility matrix. Tunneling through this strata posed serious problem, though tunneling was planned with heading and benching along with fore polling. As the strata contains boulder, great deal of difficulty was faced in driving fore poles. Also activity of fore poling was consuming a huge time and in 1^{st} cycle 22 nos fore poles of 28 mm dia rod, 3 m long took 3 days. Since the fore poling was to be done in a considerable length, necessity of finding alternative scheme for doing fore poling was felt. Accordingly, "fore poling adopter" was fabricated which was attached to jackhammer. “Fore polling adopter” is a small improvised device fabricated with 40mm dia G.I pipe 100-150mm long with 10mm steel plate welded at one end. To enable placing of fore pole rods, side restraints are welded to 10mm steel plate. The adopter was then placed over a jackhammer fitted with a broken drill rod. Fore poles were then driven through a combination of jerky rotation & pushing action of the jackhammers. Due to this improvised scheme, tunneling could be possible through bouldery strata soils.

Accordingly cut & cover construction was started from ch. 5280 to 5235 (i.e for 45 m) & tunneling was started from ch. 5/235 with full face conventional drilling & blasting method. The strata encountered in the beginning was pebbles in sandy/silty matrix in a totally dry state. As tunneling progressed a patch of conglomerate started appearing on RHS of alignment. No difficulty was faced in carrying out tunneling up to ch. 5195. However, as a precautionary measure no un-supported stretch was left at any stage during tunneling i.e next cycle of drilling and blasting of 2 m was carried out only when the earlier cycle of excavation was totally supported and backfilled. Beyond ch. 5195 some seepage was noticed and also loose fall started occurring from the conglomerate patch on R.H.S and the crown portion. Tunneling was continued beyond ch. 5195 up to ch. 5186.5 although the progress got slower due to difficult condition. At ch. 5186.5 the gap between the last backfilled rib (rib No. 97) and the face was only 30cm on RHS and Nil on LHS. However, due to loose strata, heavy loose fall and started. In this stretch tunneling could be made possible by adopting following measures.

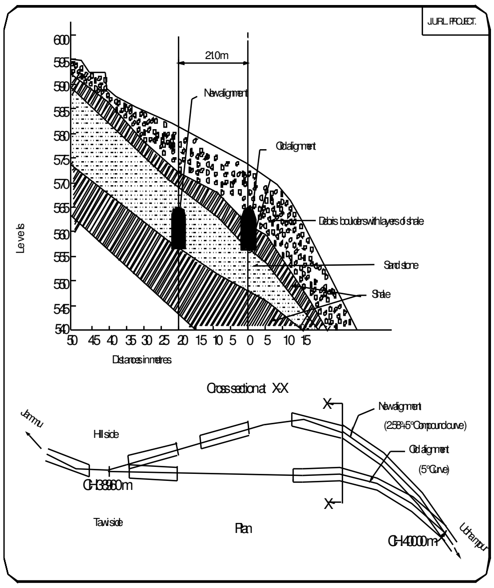
Inside the tunnel cement bags filled with earth were placed to control the loose fall from rib no. 96- 97, touching the tunnel fore poling was done using girder ISHB 150 x 75 and umbrella was formed. Stiffening of already erected ribs were also carried out. Grouting from inside the tunnel and from the top by drilling a hole was carried out. Further ribs were placed by manual excavation following multi drifting methodology.

After completion of boring the tunnel, while lining was being done, sequeezing has taken place along with up heaving of invert to the extent of 1m in certain portion of the tunnel. The work of detailed investigation and rectification is in progress.

c. Excessive flow of water from Tunnel No 3:

The work of boring 2.75 km tunnel was taken up from both ends. While the tunnel was bored for approx 2.5 km,& 500 m was remaining, very heavy flow of water approx 10000 liters/min started coming. This was from a under ground stream. The work had to be abandoned for about a year and has been taken up along with heavy Fore polling.

d. Alignment Shift :-



On Jammu – Udhampur section , tunnel No. 10 –E, F is in close proximity of Dhar-Udhampur Road. This alignment at some points passes across the hill with lateral ledge of about 20 m only. When the work was in progress some movement of ground was noticed on the hill slope. The detailed geo-technical investigations were carried out by boring 5 bore holes on the alignment and up hill side. The over burden in this reach of the alignment consisted of loose boulders/debris in the upper strata of about 10-12 m thickness and hence was unsafe to undertake tunneling. It was decided to shift the alignment towards the up hill side by about 21m which ensured safe tunneling. The over burden along this revised alignment was of loose boulders/debris in the upper strata of about 10-12 m thickness and about 8 m thick cover of sand rock and shale which ensured safe tunneling. This also gave additional lateral stability to the tunnel.

The work of Vibration signature analysis has been undertaken on the tunnels already completed. This is being done by IIT/Kanpur

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