हिमप्रभात

HIM PRABHAT

USBRL Technical News Magazine

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Vol.II issue V, February 2015

Contents

From the Desk of Editor-in-Chief

Indian Railways after opening of Udhampur to Shri Mata Vaishno Devi Katra section in 2014-15 continues in its journey through the Himalayas on Shri Mata Vaishno Devi Katra to Banihal. The route shall traverse the remaining distance passing through high bridges and long tunnels. The experience of engineers, consultants, geologists etc working on the railway line is also mounting and this issue of Him Prabhat brings to the readers newer subjects.

The Udhampur-Srinagar-Baramulla Rail Link (USBRL) between Katra and Banihal 11 km (10%) over the bridges and viaducts. It includes the famous bridge on River Chanab. When completed it would be a longest and the highest single span Railway Bridge in the world. Article by Shri P.S.Gupta, CE/D/KRCL and Shri R.K.Singh, Dy.CE/D/KRCL shares experience in the areas that were handled during the course of Geotechnical Investigation and Parameters for foundation of Chenab Bridge. This will certainly be interesting for engineers on Indian Railways and would certainly be available. The article by Shri Dr T.G.Sitharam, Prof IISc Bangalore on Slope Stability Analyses for Chenab Bridge along USBRL Project and similar another article by Branco Damajanac, Loren Lorig and Varun from ITASCA discusses use of numerical modeling for the range of design static and dynamic loadings for examining stability of the critical slopes and foundations for Chenab River Bridge and Anji Bridge. The papers brings out numerical analyses to determine the maximum transient and permanent displacements of the arch abutments and the pillar foundations, as well as the safety factors of the slopes before and after construction of the bridges under static loads and for the design seismic loading. The paper *inter alia* discusses three-dimensional distinct element modeling software *3DEC,* which is a numerical modeling code for advanced geotechnical analysis of soil, rock, and structural support in three dimensions which shall certainly excite the readers. This region being in zone IV/V (IS 1893-2002) has experienced many earthquakes in the past and recent times. And also faces the danger of seismic threats from the central Himalayan seismic gap. The goal of earthquake-resistant design is to produce a structure or facility that can withstand a certain level of shaking without excessive damage. Seismic hazard analysis involves the quantitative estimation of ground-shaking hazards at the site. Prof. K.S.Rao, Department of Civil Engineering, Indian Institute of Technology, New Delhi in his paper on "The Site Specific Seismic Hazard Analyses Of Chenab Bridge Location" on USBRL project engages the attention on issues involved in taking a railway link in the tectonically active and geologically complex Himalayan Mountains. While carrying out discussions on issues of slope stability, the discussion will be incomplete if the slope stability on portals of tunnels is not taken up. The article byShri P.S.Gupta, CE/D/KRCL and Shri R.R.Mallick, Dy.CE/D/KRCL onValidation of slope of Chenab Bridge abutment and on Slope Stability Of Sangaldan Area Portals brings out experience gained on this aspect. As a case study the fourth berm of Bakkal slope of Chenab Bridge is considered in former article and portal T42P1 has been considered in the later for discussion.

Most of the new railway line that is being built passes through tunnels. The technology that is being adopted for tunneling is NATM – for New Austrian Tunneling Method. The philosophy and techniques of NATM have been deftly discussed by Shri S.Maurya, Dy.CE/Anji/NR in article on New Austrian Tunneling Method. The use of Shotcrete for the support of underground excavation was pioneered by the civil engineering industry. Shotcrete, plays a very important role in the New Austrian Tunneling Method particularly when working in weak ground. Simultaneous working of multiple headings, difficulty of access and unusual loading conditions are some of the problems which are peculiar to tunneling and which require new and innovative applications of shotcrete technogy. Shri Sunil Bhasker, Dy.CE/Dl in their paper on Shotcrete In Tunnelling- Concept, Precaution & Methodology bring to the readers all this and much more. It is well known that concrete is commonly used in construction industry. While different civilizations had started using crude concrete in construction of their buildings but modern concrete stated developing when John Smeaton in 1793 produced hydraulic lime and Joseph Aspdin in 1824 invented Portland cement. Since then the science of concreting has developed manifold. Modern buildings, structures would be impossible without use of concrete and civil constructions in USBRL are no exception. In this project, construction is being carried out in extremely cold weather particularly during the months of December, January and February when mercury dips even below zero. This throws yet another becomes a challenge for the engineers. Article by Shri B.Salwan, XEN/SINA on Concreting in Cold Water analyses and presents pragmatic strategies to get a sound concrete at these temperatures. Then from stabilizing the slopes to NATM use of rock bolts is a technical imperative. Article by of ShriVinod Kumar, Dy.CE/N/USBRL the use of rockbolts in practice will be useful to engineers. Rock mass classification and rock support system – case study for tunnel T-74 R is another interesting reading by Sh Shailendra Kumar, Secy/CAO/USBRL discussing various aspects and systems for rock classification and the method adopted for tunnel T-74 R.

The compilation of Shri Hussain KhanXEN/D-II/USBRL brings to us the statistics, photographs and other essential information on "Some Of The Longest Railway Tunnels Of World". The list of famous tunnels includes Gotthard Base Tunnel, Swiss Alps, Seikan Tunnel, Japan, Channel Tunnel, (English channel strait of Dover),UK, The LÖtschberg base Tunnel,Swizerland.Guadarrama (Tünel de Guadarrama) Tunnel, Spain and would interest both the technically inclined and the inquisitive.

Rehabilitaion of tunnel T-10 by Sh Sandeep Gupta, CE/S/USBRL, discusses various issues involved in rehabilitation work of tunnel T-10 between Udhampur Katra Section. Slope failure and its Kinematics is another interesting reading by Sh P.S.Gupta, CE/D/KRCL and Sh R.R.Mallick, Dy.CE/D/KRCL on various types of rock slope failure and kinematic studies and its application. Geology of Chenab Bridge Alignment on Katra –Quazigund Section by Sh Joginder Singh,Rtd. Director/GSI covering geology of the area around Chenab bridge.

As a Green Initiative announced by Hon'ble Prime Minister of India, at the time of opening of Shri Mata Vaishno Devi Katra – Udhampur section, a 1 MW Solar Energy Power Plant at Shri Mata Vaishno Devi Katra Station is under construction. Article by Shri R.K.Choudhury, Chief Electrical Engineer takes the readers in a walk through the Solar Energy technologies.

Amarnath is considered as one of the most sacred of Hindu Temples, dedicated to Lord Shiva. Although it is not included in list of "twelve jyotirlingams" of India but even then large number of devotees undertakes yatra every year in tough terrains and harsh weather of Kashmir. It speaks manifolds about their immense devotion and staunch faith in pilgrimage of Amarnath. This issue carries an article by Shri B. Salwan, XEN/SINA on Amarnath Yatra. In our series introducing the readers with the human geography and socio economic and other features of the section, this time Shri M.P.Singh, Dy.CE/Reasi/USBRL brings Reasi sharply into focus in his article on Stations Along USBRL Project-Reasi. Then being a member of the team which builds this formidable railway line, I thought about bringing to readers something more about the royal history of spice of Kashmir – the Saffron. Bringing out along with its deep colour and aroma, its historic roots and mythology. Hope all this shall make it enjoyable for readers.

Mohit Sinha
EDITOR - IN -CHIEF

P R O J E C T N E W S

Presentation on feasibility of using TBM on Himalayan Geology was held on 07.01.2015. Presentation was given by Shri Puranchand Bhave, General Manager, EPC, HCC (Pics 1 to 3)

From L to R
Shri G. C. Agarwal, retired General Manager, Eastern Railway,
Sh S.C. Gupta, Retd IRSE,
Dr E.Sreedharan, former Chairman of Delhi Metro Rail Corporation, Chairman of the committee,
Sh Brij Mohan Khera, Redt. IRSE,

On the directions of Hon'ble high court order, Railway board constituted a committee of experts to examine the comparative merits of the two systems i.e. an alignment with a grade of 1 in 80 and another alignment with grade of 1 in 44. Committee comprises of following members Dr E.Sreedharan, former Chairman of Delhi Metro Rail Corporation, Chairman of the committee, Sh S.C. Gupta Retd IRSE, Sh Brij Mohan Khera Redt. IRSE, Shri G. C. Agarwal, retired General Manager, Eastern Railway. Committee held its meeting on 6th Dec, , 23rd and 24th , Dec, 2014 and 14th and 15th Jan, 2015. In the Pictures from Pic4 to Pic 12 are various proceedings of the committee.

Team USBRL during one of the presentation to Expert Committee

From L to R Sh Hukum Singh Chooudhary, Dir/Rly/URS, Sh Ankush Kisan, Adv Rly/URS, Sh A.A Khandey, Astt..Dir. Rly/URS, Sh Alok Verma, SAG/NR

PROJECT NEWS

From L to R Prof K.S.Rao ,IIT-D , Sh. P.S.Gupta CE/D/KRCL, Chrysanthos Alexandrou, Project Consultancy Head, Geodata (DDC of KRCL). Dimos Papantonis, Senior Engineering Geologist, Geodata (DDC of KRCL).

From L to R
Dimos Papantonis, Senior Engineering Geologist, Geodata (DDC of KRCL), Sh. P.S.Gupta CE/D/KRCL, Dr Joginder Singh, Ex Dir/GSI, Chrysanthos Alexandrou, Project Consultancy Head, Geodata (DDC

From L to R
Shri Achal Jain, Executive Director(L&A)I/Rly. Bd., Secretary to the Expert Committee
Shri G. C. Agarwal, retired General Manager, Eastern Railway,
Sh S.C. Gupta, Retd IRSE,
Dr E.Sreedharan, former Chairman of Delhi Metro Rail Corporation, Chairman of the committee,
Sh Brij Mohan Khera, Redt. IRSE,

From Lto R Mr Pekka Pulkkinen, PDE/WSP Sh Giridhar R Dir/CBPU, Prof Sitharam IISC/Bangalore and Associate Prof. Madhavi Lata IISC/Bangalore

From L ro R
Mr Siegfried Hopf PDE/LAP, Mr Pekka Pulkkinen, PDE/WSP Giridhar R Dir/CBPU

Team USBRL during Expert Committees meeting

PROJECT NEWS

Presentation on use of RHEDA sleepers for BLT on USBRL Project was held on 31-10-2014. Presentation was given by Patel Group and experts from RHEDA ,GMB

Presentation on National Pension Scheme was held on 06-02-2015 Shri Veer Singh ,Dy. FA & CAO/USBRL explained the features of NPS to officers and staff of USBRL Project.

Presentation by Designers and Proof Consultants engaged for Chenab Bridge was organised by KRCL on 11/11/14 and 12/11/14 at New Delhi. In the Pic team USBRL Designers and Proof Consultants for Chenab Bridge during the presentation

P R O J E C T N E W S

Presentation by Sh. Kumud Goyal CMD/Trivestar on implementation of MIS for USBRL Project .Presentation was attended by Team USBRL.

Celebration of Republic day at USBRL Office Jammu

Employees of the Month

Sh. P.L.Dhar

Sh .P. L. Dhar OS is an extremely obedient , sincere hard working and intelligent worker .He is young and Dynamic worker .At present he is posted as Office Supdt. in Personnel Branch. He is capable of handling all types of ministerial works, court cases, and other policy matters. He is discharging his duties with full devotion and dedication .In recognition of his meritorious service, he has been given many awards and also been given the CE Level award in the year 1999 besides other group awards .

Favourite Food: Curry Chawal
Favourite Colour: Pink
Best moment: Opening of Udhampur- Katra Section

Sh. Arun Kumar Kak

Sh. Arun Kumar Kak Sr. Section Engineer is an extremely obedient, sincere and intelligent worker. He is young and Dynamic worker .At present he is posted as Sr. Section Engineer (Const) at Katra. He is entrusted the job of Construction of Single line Bridge No.32 over river Banganga with span (64+92+64)m of PSC continuous at Katra on Udhampur –Srinagar-Baramulla Rail Link Project .He is discharging his duties with full devotion and dedication .Due to extra ordinary efforts most of the construction work of Bridge No 32 is being completed .In recognition of his meritorious service ,his name has been recommended for grant of G.M level Railway week award - 2015.

Favourite Food: Rajmah Chawal
Favourite Colour: Sky Blue
Best Moment: Opening of Udhampur – Katra section

Employees of the Month

Sh. Prakarmi Ram

Sh.Prakarmi Ram SSE/Drg. under Dy. CE/S & C –I/UHP was born on 25.11.1965 in District Chamba of Himachal Pradesh. He Joined the Railways as Draughtsman (Civil) in 11.12.1987 and initially posted in the JURL Project at Jammu Tawi up to October 1998. He has also worked as SSE/G Baroda house New Delhi from November 1998 to Aug' 1999 and dealt in compiling the progress of 08 Division of N.R. Further, he was posted in USBRL, Project at Udhampur. He has worked in various capacities such as preparation of the working drawings of all types of structures, scheduled estimates, tender documents, land acquisition matters such as approval of land indents, placing of land indents to State Authority, chasing and maintaining the updated position of forest, private & govt. Land along with red accounts time to time. Presently he is working in the unit of Dy. CE/S & C –I/UHP and prepared the CRS papers and all completion plans on AUTO CAD. He has contributed immensely in the submission of CRS papers and associated in the CRS inspection with complete records of Udhampur- Katra section in connection with USBRL Project. He is sincere, hardworking and dedicated towards his duties.

Geotechnical Investigation and Parameters for foundation of Chenab Bridge

1.0 Introduction

Chenab bridge is proposed between stations Salal Road 'A' (towards Jammu side) and Salal Road 'B' (towards Shrinagar side). The proposed bridge is under construction between Ch. 50.400 on left abutment near Bakkal Village (Jammu side) and Ch. 51.715 on right abutment near Kauri village (Shrinagar side). The bridge is having a total length of 1315m, consisting of a) Approach span – 185m long (From 50/400 to 50/585), b) Steel Arch span – 480m long (From 50/585 to 51/065), c) Viaduct portion – 650m long (From 51/065 to 51/715). The railway track will be passing over the Chenab river at a height of about 359m. The geographical coordinates of the centre point of bridge is Latitude 33°04'75'' North of Equator, Longitude 74°54'57'' East of Greenwich. At this location the river Chenab is flowing in a direction towards SW. The proposed railway line alignment at the bridge crossing site is in the direction of N120° towards left abutment (Jammu side) to N300° towards right abutment (Shrinagar side). The Chenab river forms about 350m deep gorge in a V shaped valley in this area. The width of river bed level is about 150m at river bed level and river bed level is 488 m, while the rail track level on the proposed bridge will be 856.188m.

The area on left bank is having a maximum elevation of 906m near bakkal village (Jammu Side). The area on right bank is having a maximum elevation of 860m near Salal Road 'B' station. The entire area comprise of hilly terrain traversed by numerous small and large nallas. In general hill slopes are very steep, with slope angle varying between 50°-60°. The area is highly undulating and with a rugged topography. It is characterized by strike ridges, dip slopes with steep scraps and drainage pattern controlled by foliation and joint planes.

2.0 Several Geological and Geotechnical studies have been carried at chenab bridge site for the design of foundation, its protection work and slope stabilities. Initially, Geotechnical Investigation carried out by following agencies:

a) Geophysical survey for subsurface stratigraphy at bridge site carried out by NIRM (National Institute of Rock Mechanics), March'2003.
b) Geotchnical mapping at bridge site carried out by NIRM, March'2004.
c) Geotechnical Investigation, April'2004.
d) Geotechnical Investigation, April'2005
e) Geotechnical Investigation at each foundation location, July'2005.
f) Tests (Cyclic Plate Load Test & Insitu Direct Shear Test) for Foundation S40 & S50, June'2009
g) Geophysical Survey (MASW) Inside Drift at Chenab Bridge by NGRI, July'2009
h) Strength Parameters for Rock Spalls Excavated from S70-S80 by IITR , Dec'2009
i) Geotechnical Report Pier foundation location S40 - S60, April'2010
j) Results of Joint Filling Material Testing of Drifts S50 by ANGRON, Dec.'2013
k) Results of Joint Filling Material Testing of Drifts S50 by IIT Roorkee, Dec.'2013
l) Results of Joint Filling Material Testing of Drifts S40 & S50 by CSMRS, May'2014

2.1 NIRM (March'2003) carried out mapping of subsurface stratigraphy up to 40m depth by Seismic Refraction Survey (SRS) and Vertical Electrical Sounding (VES) study at site and given following recommondations:

- ***Bakkal End (At Ch. 48.930)***

  SRS Study:
  - Thick soil layer - up to 20m depth
  - Weathered and joint rock mass - up to 40m *depth.*

  VES Study:
  - Overburden - up to 10m followed by weathered rock.
  - No indication of presence of any water bearing strata.

- ***Kauri End (At Ch. 51.920)***

  SRS Study:
  - Thick but poor rock strata up to 27-28m depth.
  - Weathered rock up to a depth of 40m of (20-40MPa).

  VES Study:
  - Overburden up to 15m depth.
  - Weathered rock / fractured rock after 15m.
  - No indication of water – bearing strata.

2.2 GeoTechnical mapping at bridge site location has been carried out by NIRM, (March'2004) and concluded the following:

a) The major lithologies are dolomitic limestone with different degrees of weathering and fracturing; boulders of dolomitic limestone, brecciated limestone and quartzite with silt and clay material, poorly consolidated conglomerate bands, nodules and minor bands of chert and siliceous limestone. The strata (dolomitic limestone and limestone) are exposed at higher elevations and at the foothills along the riverbed.

b) The strata are characterized by prominent one sub-horizontal foliation joint and two sub –vertical joints and summary of the orientations of these features are tabulated as below

Features	Strike	Dip amount	Dip direction
Railway line alignment	N 120° - N 300°	-	-
Foliation Joint	N 140° - N 320°	27°	N 050°
Joint -1	N 150° - N 330°	65°	N 240°
Joint -2	N 075° - N 255°	80°	N 165°
Random Joint (1)	N 014° - N 194°	Vertical	-
Random Joint (2)	N 037° - N 217°	77°	N 127°
Random Joint (3)	E - W	88°	N 179°

Prem Sagar Gupta
CE/Design/KRCL

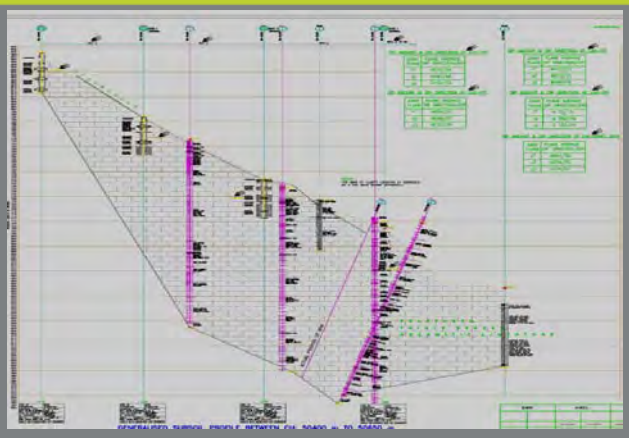
Rajesh Kr. Singh
Dy.CE/Planning/KRCL

c) The surface weathered and fractured dolomitic limestone are classified using various standad approaches and the rating are given below:

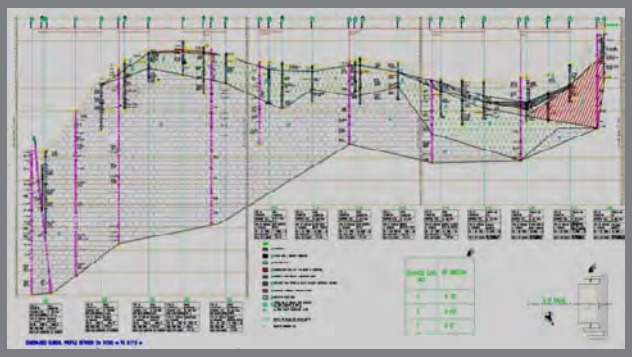
Parameters	Surface weathered dolomitic limestone	Fractured dolomitic limestone
Rock Quality Designation Index (RQD)	49	10
Rock Mass Quality (Q)	6.12	1.25
Rock Mass Rating (RMR)	48	25
Geological Strength Iindex (GSI)	43	27

Based on these values, the surface weatherd dolomitic limestone was classified as "Fair'' rock mass, while the fractured dolomitic limestone as "Poor" rock mass.

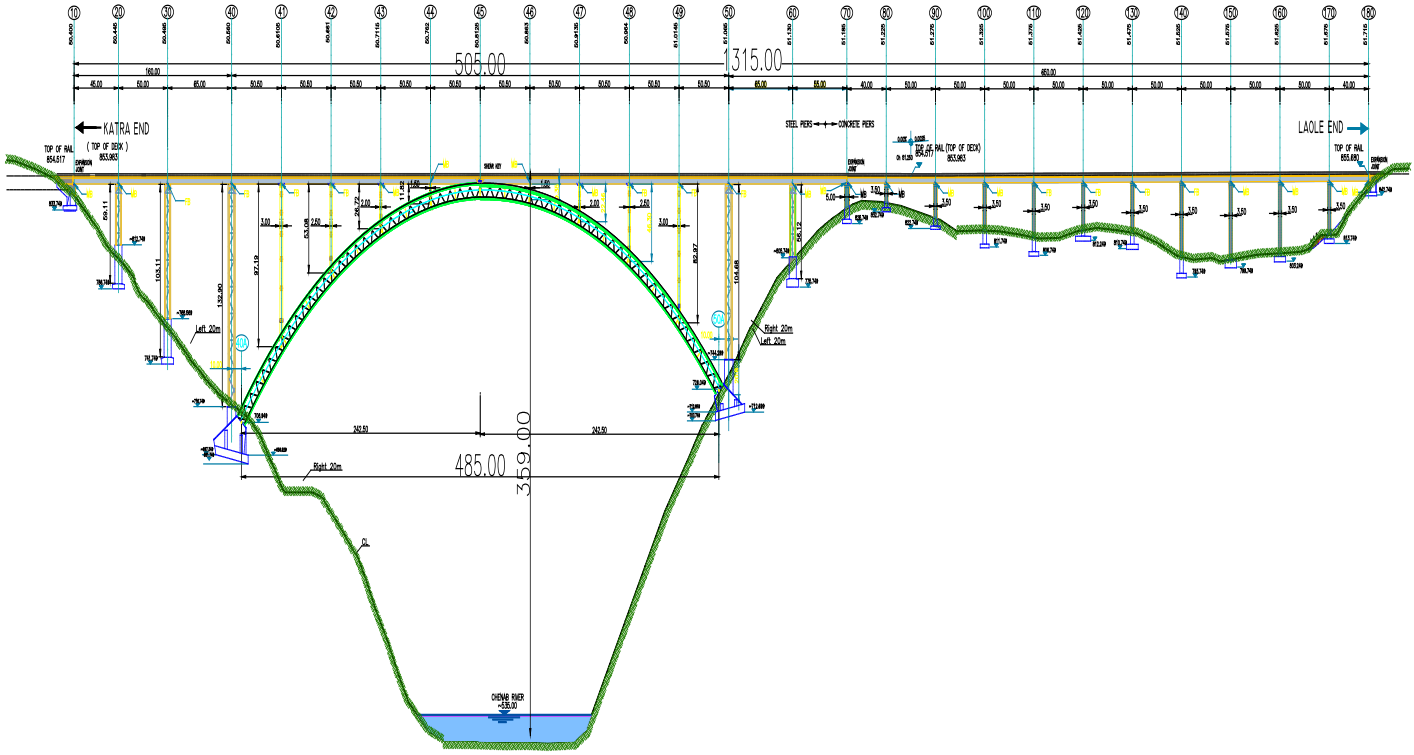
2.3 Geotechnical Investigation (March'2004, April'2005 & July'2005) Several Geological and Geotechnical studies have been carried at chenab bridge site for the preparation of design of foundation and its protection work / slopes. Total 40 no. of borehole have been drilled to asses the geotechnical parameters of bridge site. Nine (09) boreholes upto depth of 30m & 40m have been carried out in April'2004 and the physical, mechanical and joint properties of rocks have been tested at NIRM & CSMRS laboratory. Fourteen (14) no. of boreholes upto 150m depth in main bridge & 70 to 80m in viaduct portion have been carried out in March'2005 and physical properties of rocks were tested in the laboratory of NIRM. Seventeen (17) borehoes upto depth of 30m, 40m & 64m have been drilled at each specified foundation location by the Contractor. The physical & chemical properties of rock materials were tested in the laboratory of IIT Mumbai, ANGRON and R&D Unit of Structwel Consultant in July'2005 for the foundation design of Bridge.



Location boreholes (13 Nos.) on Bakkal Side (S10 – S40)

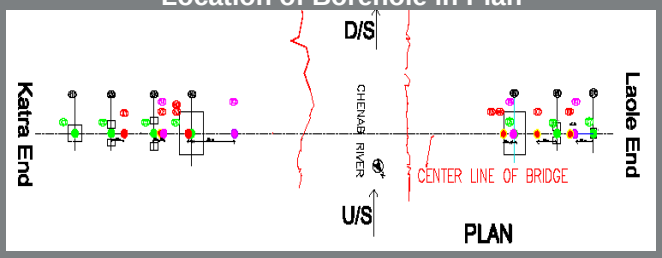


Location boreholes (27 Nos.) on Kauri Side (S50 –S180)



GENERAL ARRANGEMENT OF THE BRIDGE

Location of Borehole in Plan



2.3.1 Methodology of field work at site:

a) **Methodology of locating the borehole:**

All the boreholes on land were located within +/- 100 cm. away from alignment and their specified location. At some foundation locations 1 & 2 more boreholes had been drilled from 10 -15 m away from centre line of foundation.

b) **Drilling Equipment:**

3 drilling rigs VOLTAS-35, VOLTAS-90 anf JOY-7 type hydraulic and mechanical drilling rigs were mobilised and 3 more light weight, skid mounted, SWENSKA make drilling rigs at site. SWENSKA rig is a rotary drilling rig of screw feed type, capable of drilling upto a depth of 250m, using BW type drill rods. SWENSKA machine is powered by 24.5 HP, 2YDA diesel engine and drilling has been carried out at very slow drilling speed of 100-150 rotations per minutes.

c) **Drilling in Soil:**

In the initial 2-3 m of drilling in soil, or till encountering weathered rock drill hole was advanced by carrying out rotary drilling and subsequnt insertion of casting to prevent collapse of sides into the borehole. 4-1/2"Ø and 3"Ø casings were telescopically installed upto required depth.

Drilling in overburden / initial weathered rock was carried out using 100mm Ø core barrel fitted with Tungsten Carbide bits. Each TC bit was provided with 12 – 16 cutting teeth and waterway grooves.

Bentonite slurry was used as circulaing / cooling medium to bring up the cuttings / drill sludge from the bottom of borehole upto ground level. Standard Penetration Tests (SPT) were carried out at certain specified epths, in overburden, in some boreholes, till N≥100 was enco untered or full 450 mm penetration was achieved in the specific test. Split spoon sample, confirming to IS 9640-1980, was used for carrying out Standard Penetration Test. SPT tests were conducted, as per procedure laid down in IS 2131 – 1981. Disturbed samples obtained from SPT tests, were collected, packed in polythene bags, labled properly and sent to laboratry for carrying out visual examination identification and classifications tests. Drilling information and description of soil stratum as per IS-1892 -1979, has been prepared and indicated on borehole logs.

d) Drilling in Rock:

Except some initial portion in weathered rock, where drilling was done using HX size single tube core barrels, all the drilling, in moderately weathered to fresh rock, was done using NX size double tube core barrels.

In most of the HX size drilling, Tunsten Carbide (TC) bits were used in Highly / Moderately Weathered Rock. At some boreholes, specially made 100 mm Ø diamond bits were also used to carry out HX size drilling in moderately weathered to fresh rock.

Appropriate type of diamond bits and reaming shells, compatible with series of double tube core barrels, were used in carrying out drilling in moderately weathered, slightly weathered, fresh rock.

Suitable type of core catcher springs, were used in all the core barrels to ensure maximum core recovery and to minimize slippage of cores from core barrel, at the time of taking out core barrels from borehole. At the end of each drilling run, of generally 1.5 to 3.0 m length, rock cores were systematically recovered from top to bottom, in the same order, as they were recovered from core barrel. These rock cores were properly arranged in the core boxes.

The ends of all core pieces were carefully examined for presence of machine fractures, or breakage during handling and / or natural fracture on account of natural joint.

e) Field Tests:

As a part of this investigation, following field tests were carried out in this boreholes:

i) Standard Penetration Test (SPT)
ii) Packer Permeability Test

i) Standard Penetration Test (SPT):

SPT were conducted in the overburden portion, in some boreholes, at specified depth. The borehole was flushed clean and all loose cutting soil, drill sludge which had settled at the bottom of borhole was removed. Astandard split spoon sampler of 50.4 mm dia, 450 mm length with detachable cutting shoe at bottom and driving head attachment at top, which could be connected to drill rods was used. The split spoon sampler was carefully lowered to the required depth and held in a vertical position. This sampler was driven, in the section of soil by a 65Kg, weight (monkey) falling through a height of 75 cm in each blow.

The blows required to drive the first 15 cm. of sampler, were neglected as seating blows. The number of blows required to drive the next 30 cms were recorded, as Standard Penetration Resistance value (N value).

For very hard stratum, where penetration of the sampler was small, even after giving 100 blows of the monkey, the test was stopped after 100 blows. In such typical cases, corresponding penetrations achieved were noted. After completing the test, the sample collected in split spoon sampler, was carefully examined for identification of soil stratum, labeled properly and stored in polythene bags. The results of the SPT are noted on the borehole logs, at appropriate depths, at which these tests have been carried out.

ii) **Packer Permeability Test :**

Packer permeabilty tests were carried out in rocks stratum, in some boreholes, at specified depths, generally in accordance with IS 5529 – Part II-1987. In this test, normally two types of tests are carried out:

Single Ppacker Permeability Test
Double Packer Permeability Test

In this work area, single packer permeability tests were carried out preferably. Generally, single packer permeability tests are preferred, as the were carried out as the borehole progresses and results are considered more reliable.

Double packer tests are generally carried out to reestablish permeability values, in certain zones of special interests, where very high permeability values are observed or abnormal permeability values are observed or permeability values need to be reconfirmed in view of general observations.

In these tests, section of borehole already drilled upto required depth, is isolated with help of a packer, in such a way, that during the test water flows only through the section to be tested. Water is injected into the borehoe at specified increments of pressure such as 1 Kg/cm^2,1.5 Kg/cm^2, 2 Kg/cm^2,2.5 Kg/cm^2, 3 Kg/cm^2. Each pressures increment is maintained for a period of atleast 15 minutes. Intake of water by borehole side walls or 'Water loss' is observed at interval of 5 minutes. Pressure increaments are sequentially increased upto maximum pressure in ascending stage and systematically decreased upto lowest value in descending stage. Permeability values is calculated in Lugeons. The results of permeability tests have been indicated seprately for specific boreholes where such tests were carried out.

2.3.2 Determination of **physical, mechanical and joint properties** of the collected sample were carried out in laboratory of NIRM, CSMRS, ANGRON, IIT Mumbai, IIT Roorkee and R&D Unit of Structwel Consultant.

A. Physical Properties

- Density
- Specific Gravity
- Water Absorption
- P&S Wave Velocity

B. Mechanical Properties

- Uniaxial copressive strength
- Elastic constants under uniaxial stress conditions
- Tensile Strength
- Triaxial compression test – C, Ø and m (constant in Hoek – Brown failure criterion)

C. Rock Joint Properties

- Joint wall compressive strength (JCS)
- Basic friction angle
- Normal Stiffness
- Shear Stiffness

2.3.2.1 Sample Preparation

Lithologically, all the rock cores are fine grained dolomite. On examination of the cores, it was found that most of smples are having tight joints oriented at random direction. Dimensions and their tolerances were maintained as per the IRSM suggested method. The following machine were used for preparing the samples for various tests:

Rock core cutting machine
Rock core surface grinder
Rock core drilling machine

Length to diameter ratio for the samples is maintained for various tests as detailed below:

Test	Length /Diameter ratio
Density	0.4 to 0.5
Water absorption	0.4 to 0.5
Porosity	0.4 to 0.5
Compression wave velocity	0.5 to 0.6
Uniaxial compressive strength	2.0
Modulus of Elasticity	2.0
Poisson's ratio	2.0
Tensile strength	0.5
Tri-axial compressive strength	2.0

Triaxial compression tests were carried out using Hoek Triaxial cell. Laboratory was having BX (-42 mm) and NX (-54.4 mm) size triaxial cell. Cel can not accommodate samples if there is a variation in diameter of the test samples by about ± 0.5 mm. In case of some of the samples received, the diameter was more than the actual requirement, such samples were recorded to BX or NX size using rock core drilling machine. The samples were cut to the required length using rock core cutting machine and the loading faces of the samples were polished using an automatic surface grinder. For the tests under dry condition, prepared samples were kept in the atmosphere for about a week, and were tested therafter.

For the tests under water saturated condition, the prepared samples were saturated under vaccum in water for about 4 hours. Thereafter, they were removed from the vacuum and immersed in water for 24 hours, and were tested immediately after removing from water.

- The following testing machines were used to carryout different tests.
  - 150 Ton MTS Servo-hydraulic Compression testing machine
  - 300 Ton Compression testing machine
  - Hoek Triaxiaal cell
  - MTS axial & lateral extensometers for measuring the strain during the compression testing
  - Shear testing machine

2.3.2.2 Testing Methodology

All the tests were carried out as per the ISRM suggested method and ASTM Standard. ASTM Standard C-97 gives the procedure for determining water absorption and specific gravity from the same sample. Due to limited number of core samples, test samples were selected randomly from different samples for the determination of properties.

A. Physical Properties

The methodology for determination of properties; Density, Specific gravity, Water absorption, Porosity and Ultrasonic velocity (P&S wave), are as follows:

i) Density

Density was determined by measurement method. The volume of the sample was calculated by measuring its dimensions using vernier caliper and the mass was determined using weighing machine. Density was calculated by dividing the mass by volume. Based on the length and diameter of the sample, volume of the sample was calculated using the following formula:

Volume $= \pi r^2$ h

Where r is the radius of the sample and h is the height of sample

Weight of the sample was measured using the electronic balance up to 3rd decimal

Density of the sample was calculated using the following formula:

Density = Weight / Volume and expressed in kg/m^3

ii) Water absorption

Prepared samples were saturated with water under vaccum for about 24 hours. At the end of 24 hours, the sample was removed from the water. The surface was wiped with damp cloth and weighed using an electronic balance. The sample were kept in the oven at 100°C, to expel water for 24 hours. At the end of 24 hours, sample were removed from the oven, cooled for about 6 hours and weihed gain. Water absorption was calculated using the following formula:

Water absorption, % = (B-A)/A x 100

Where ,

A = Weight of the dried specimen

B = Weight of the specimen after immersion in water

iii) Specific Gravity

The same sample used for determing the water absorption was also used for determing the specific gravity. After determing the water absorption, the weight of the same sample was measured by suspending in water. For this purpose, a chemical balance was used. Abeaker containing water was supported above the balance pan using a small wooden stand. The sample was suspended by means of a thread using the arm of the balance, and then immersed in the water. Weight of the sample in water was determined. Bulk specific gravity was calculated using the following formula:

Bulk specific gravity = a/(b-c)

Where

A = Wieght of the dried sample

B = Weight of the water saturated sample in air

C = Weight of the sample in water

iv) Porosity

Porosity was determined by water saturation technique. Prepared samples were kept in the vaccum desiccator, and saturated with water for about 24 hours. Then they were removed from the desiccator and weighed using an electronic balance. These samples were kept in the oven at 100°C to expel water for 24 hours. At the end of 24 hours, the samples were removed from the oven, cooled and weightd again. Porosity was calculted using the following formula:

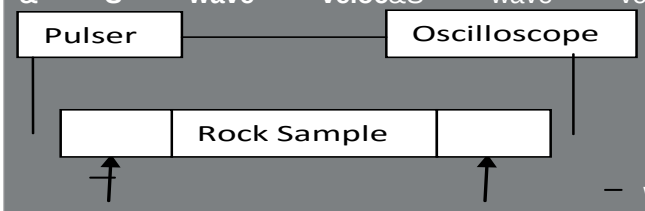
Weight of sample + water : Msat
Weight of dried sample : Ms
Pore Volume (Vv) = (Msat – Ms) / ρw
Porosity = 100 Vv%/V

Where V is the volume of the sample determined by caliper method

v) P & S wave velocity

P&S wave velocity was determined by pulse transmission method. Velocty was measured using 2 MHz frequency probe. Initially both in faces of the probes were in contact, travel time was noted on the oscilloscope screen. Grease was applied to both the sides of the sample. Probe was placed on both the sides of the sample, again travel time was noted. Actual travel time through the sample was obtained by subtracting the travel time due to probe – probe contact. Velocity was determined by dividing the thickness of the sample by travel time. Equipment set up shown as below.

& S wave veloc&S wave velocity



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B. Mechanical Properties

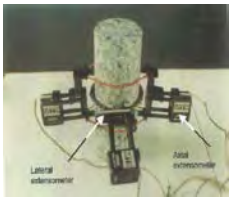
Due to availability of limited number of samoles, a few representative samples were selected from each bore hole for testing. Test were conducted on dry and water saturated sample.

i) Uniaxial Compression test & Elastic constants

Uniaxial compression tests were carried out on cylindrical rock core sample to determine the compression strength and elastic constants (Young Modulus and Poisson's ratio). Cylindrical core samples were prepared asper the recommendation of ISRM. Compression test was carried out using 300 ton compression testing machine. Test specimens were kept inside the load frame of the compression testing machine, and loaded at a constant load rate of about 2 tons/minute such that the failure occurs within 5 to 10 minutes. The Uniaxial compressive strength was calculated by dividing the maximum load by the cross sectional area of the specimen.

For measuring the axial and lateral strain, direct contact MTS extensometers were used. These extensometers were mounted directly on the sample. The extensometers have precision resistance type foil strain gages, bonded to a metallic element to form a wheatstone bridge circuit. The axial extensometer consists of two arms and two knife-edges mounted on them to have firm contact with the specimen. Elongation or compression of the specimen causes the movement of the arms on which the knife edged are mounted. The arm movement causes the metallic elements to bend, thus changing the resistance of the starin gages. The resultant change in the balance of the wheatstone bridge produces an electrical output that is proportional to the displacement of the extensometer arms. The circumferential extensometer comprises of a roller – link chain wrapped around the specimen, which monitors the relative motion of the ends of the chain. This extensometer measures the average change in specimen diameter by determining the change in specimen circumference.

Testing was carried out using MTS compression testing machine. Sample fitted with extensometers was kept inside the MTS load frame and the axial load was applied at a constant rate of 2.0 tons/min. The stress, axial strain and lateral strain were recorded by the computer using the data acquisition system. The Young's modulus and Poisson's ratio were calculated from the linear portion of the stress – strain curve.



ii) Tensile test (Brazillian test)

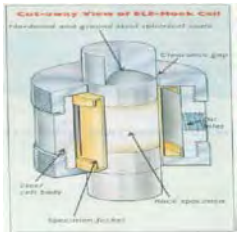
This is an indirect method of determining the tensile strength of rock samples. In this test, a circular solid disc is compressed to failure across the diameter. Specimens were prepared to a thickness equal to 0.5 times the diameter (L/D = 0.5) anf mounted in the Brazillian apparatus, which consists of two loading jaws designed to contact the disc shaped rock sample at diametrically opposed surfaces over an arc. This loading arrangement induces a tensile stress perpendicular to the loaded diametrical plane. Specimen was compressed in a 20 ton Universal Testing Machine at constant rate of 20 Kg/sec up to the point of failure. Tensile strength was calculated from the failure load.

iii) Triaxial compression test

Triaxial compression test was carried out to determine the following parameters:

- **Cohesion (C), Internal angle of friction (Ø) in Mohr Coulamb failure criterian**
- **'m' constant in Hoek – Brown failure criterian**

Due to limited number samples multistage triaxial compression test were carried out using Hoek triaxial cell (ELE make) as per the ISRM suggested method. The cell consists of a steel body, a urethane rubber jacket to prevent the hydraulic fluid from entering the specimen and steel plates of diameter equal to the diameter of the test specimen with a spherical seating arrangement . The specimen was placed inside the cell along with the rubber jacket. Axial load was applied using 815 MTS Compression testing machine, and lateral confining pressure by the hand pump. MTS system was operated under displacement control at a rate of 0.12mm/min. Both the axial load and confining pressure was increased. Initially the confining pressure was maintained and confining pressure was increased. Initially the confining pressure was maintained at 5 MPa or 8 MPa. Load was increased and load vs. Displacement curve was observed on the computer monitor screen. Loading was then continued until the peak strength of the specimen was reached. At this time , the confining pressure was increased to the next higher value. The peak strength of the specimen was identified by a reduction in the loading rate or by the bending of the load – displacemnt curve towards x-axis on the screen. Ttriaxial compression tests were carried out at confining pressure varying from 5 to 30 MPa. Mohr – Coulomb & Hoek – Brown parameters were calculated using Rock – Data software.



C. Joint Properties

Joint are fractured in rocks along which there has been little or no displacement or very slight movement perpendicular to the joint surface. Fresh rock joints are essentially in the same physical condition as when they were formed while altered joints have undergone significant physical modification subsequent to their formation. The most important properties of joint are its roughness, friction angle, and joint wall compressive strength. These properties in turn affect the peak shear strength, normal and shear stiffness. Numerical models using joint elements require the knowledge of the normal (Kn) and shear (Ks) stiffness parameters. The following joint properties were determined in the laboratory.

- Basic friction angle (Φb) and residual friction angle (Φr)
- Joint wall compressive strength (JCS)
- Peak shear strength
- Normal stiffness
- Shear stiffness

Brief methodology of determining these properties are descibed below:

i) Joint Wall Compressive Strength (JCS)

The compressive strength of the rock comprising the walls of a discountinuity is very important component of shear and deformability of joints. Schmidt hammer (L type) is used for determining the joint wall compressive strength. Selected joint sample was held in a vice and hammer was kept on its surface and was pressed and the rebound number 'r' was noted. Each surface was tested number of times to get the representative set of results, and the mean value of 'r' was used to calculate the JCS (in MPa) from the following fromula:

$$JCS = 10^{(0.00088\rho r+1.01)}$$

Where ρ = density of the material (KN/m^3)

r = rebound number

i) Basic & Residual Friction Angle (Φb &Φr)

The basic friction angle is related to the friction between two parallel and plane surfaces of the same material. It is estimated by conducting simple tilting tests using smooth cores. Three cores were used for determining the tilt angle. Two pieces of core are placed on the horizontal base in contact with one another. Third piece pf core is placed on their top, and is allowed to slide. The angle of tilt, ɑ, is recorded. Basic friction angle is calculated from the following formula:

$$\Phi b = \tan^{-1}(1.155 \tan\alpha)$$

The residual friction angle (Φr) of weatherd rock joints is very difficult to determine experimentally due to the large shear displacements required. The following empirical relation is used to estimate Φr from Φb based on rebound values using a Schmidit hammer. It is based on rebound tests on both un-weathered rock (rebound R) and on the weathered joint wall (rebound r).

$$\Phi r = (\Phi b - 20) + 20r/R$$

iii) Normal and Shear Stiffness

When a rock block containing a single discontinuity is loaded under compression, the two contacting surfaces are forced into closer proximity. If a shear load is also applied incrementally, the surface will be forced to displace relative to each other at the instant the shear load overcomes their resistence to sliding. Discontinuity closure and shear displacement are the two main components of rock mass deformability.

The deformability properties of individual joints are described by normal (Kn) and shear (Ks) stiffness. Joint normal stiffness can be measured in the laboratory by simple compression tests. Shear stiffness can be measured from the pre-peak portion of the shear stress – displacement curve.

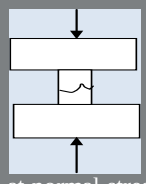
- **Determination of Normal stiffness (Kn)**

Normal stiffness is determined by the compressive loading of joints. Two joints halves, which are having proper matching, are selected . MTS extensometers is fixed between the joints to measure the closure of joints.

Sample with the extensometers was kept between the platens of the MTS compression testing machine. Joint plane was shown by color line on the sample. Load was applied at a rate of 1 ton/min to a maximum load of 5 tons. Load and displacement was recorded in the computer. From the normal stress vs displacement curves, stiffness was calculated. In general, normal stiffness can not be defined by a single value as the normal stress vs. Displacement is nonlinear. In the present study, I nitial tangential stiffness was calculated.

- **Shear behaviour – shear stiffness**

The determination of shearing characteristics of a rock is carried out in the laboratory by means of direct shear test on sample. Test was carried out at a constant normal stress (σn), and the shearing force and shear displacement were measured throughout the experiment. Adirect shear testing machine with constant normal loading facility was used to carryout the tests. Matched joints were molded using concrete mixture, and were cured for about 10 days in water. The mould with the joint is shown as below:



Tests were carried out at normal stress of 2, 4, and 6 MPa. Molded samples were kept inside the shear box and the desired normal stress was applied. Sample was sheared at a rate of 0.1 mm / second. Shear load and displacement was recorded in the computer. Shear stiffness was calculated from the peak shear stress and peak shear displacement.

2.3.3 Detailed summary of ***subsoil profile*** at each foundation locations (S10-S180) are described as below:

a) S180 Borehole

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.755	855.680	844.749	811.40	44.28	40	1

Depth	Description of Stratum
0.0 m - 9.50 m	Soil overburden comprising of Brownish Clayey silt with fragments of rock / boulders of limestone.
9.50 m - 15.00 m	Brownish Hard Silty clay derived from the total decomposition of highly weathered shale. Interpreted as residual soil.
15.00 m - 23.90 m	Rock is Quartzite in form of thin bands within thick beds of Shale. Shale material is washed away during drilling.
23.90 m - 34.00 m	Rock is Quartzite in form of thin bands within thick beds of shale.
34.00 m – 40.00 m	Rock is Fractured Quartzite.

- During drilling 100% water loss observed at 12.5 - 23.9m & 34.0 – 40.0m depth.
- From 0.00 -15.00m depth, N value more than 100.
- From 15.00 - 23.90m depth, core recovery very poor.
- From 23.90 – 40.00m depth, core recovery varied from 20% to 43% and RQD - NIL to 20%.
- Rock Mass Rating (RMR) Value, 34 between 23.90 – 34.00m & 42 between 23.90 – 34.00m depth.
- Discontinuities in this portion, mainly foliation joints and two sets of joints.
- Joint surface, smooth to rough and filled up with the weathered material.

b) S170 Borehole

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.675	855.580	819.600	791.549	28.051	80.000	2

Depth	Description of Stratum
0.0 m - 15.50 m	Brownish to black decomposed material consists of hard Silty Clay with fragments / boulders of Limestone.
15.50 m - 17.00 m	Brownish Hard Silty clay derived from decomposition of shale.
17.00 m – 18.50 m	Brownish Hard Silty clay derived from decomposition of shale.
18.50 m - 20.00 m	Sandstone
20.00 m – 30.00 m	Grey to black carbonaceous shale with occasional thin bands of sandstone
30.00 m – 54.50 m	Rock is limestone to shalely limestone.
54.50 m – 72.00 m	Rock is coaly shale to decomposed coaly shale
72.00 m – 80.00 m	Rock is jointed and fractured dolomite

- 100% water loss at 12.5 – 23.0m depth.
- Single packer test conducted between 8-11m, permeability value - 26 Lugeons.
- From 0.00 – 15.50m depth, the core recovery varies from 13 to 38% and RQD - NIL.
- From 15.50 – 24.00m depth, the core recovery varies from 13 to 41% and RQD - NIL.
- From 24.00 – 30.00m depth, the core recovery varying from 33% to 86% & RQD as NIL.
- From 21.00 - 27.00m depth, Joints surface - smooth to rough & stained with ferruginous material with closed and tight joints.
- From 30.00 – 54.50m depth, the core recovery varies between 16% to 45% and RQD - NIL.
- From 54.50 – 80.00m depth, the core recovery varies from 13 to 37% and RQD - NIL.
- Rock Mass Rating (RMR) Value, 30 & 42 between 20.00 – 23.00m depth

c) S160 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.625	855.455	807.749	784.000	23.749	70.000	2

Depth	Description of Stratum
0.0 m - 7.50 m	Overburden consists of Brownish Hard Clayey Silt with boulders.
15.50 m - 19.50 m	Black hard silty clay mainly derived from the decomposition of Carbonaceous Shale.
15.00 m – 21.00 m	Grey shale
18.50 m - 26.00 m	Altered rock or bauxite formation.
26.00 m - 31.00 m	Grey shale
31.00 m – 38.00 m	Small pieces of broken sandstone
38.00 m – 54.50 m	Sandstone
54.50 m – 64.50 m	Rock is reddish, brecciated rock
64.50 m – 70.00 m	Rock is jointed and fractured dolomite

- From 0.00 – 7.50m, SPT conducted N = 30.
- From 7.50 -19.50m depth, SPT conducted N>100.
- From 19.50 – 21.00m depth, the core recovery 36% and RQD - NIL.
- From 21.00 – 24.00m depth, the core recovery between 33% to 43% & RQD- NIL.
- From 24.00 – 27.00m depth, the core recovery varying from 57% to 88% & RQD as NIL.
- From 21.00 - 27.00m depth, Joints surface - smooth to rough & stained with ferruginous material with closed and tight joints.
- From 27.00 – 31.50m depth, the core recovery varies between 37% to 53% and RQD - NIL.
- From 31.50 – 38.00m depth, the core recovery varies from 27 to 65% and RQD - NIL.
- From 38.00 – 70.00m depth, the core recovery varies from 16 to 36% and RQD - NIL.
- Altered rock or bauxite formation, countered at depth of 22.5m below existing GL.
- From 27.00 - 38.00m depth, Joints surface - smooth to rough & stained with ferruginous material.

d) S150 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.575	855.330	804.449	780.7	23.749	38.000	2

Depth	Description of Stratum
0.0 m - 7.50 m	Brownish Black, Hard ,Clayey Silt , N =30
7.50 m - 10.50 m	Totally decomposed Carbonaceous Shale. N > 100.
10.50 m – 16.50 m	Black hard shale with small pieces of sand stone
16.50 m - 18.00 m	Crushed grey shale
18.00 m - 21.00 m	Grey shale
21.00 m – 25.50 m	Crushed quartzite pieces (suspected shear zone)
25.50 m – 38.00 m	Quartzite – Chert - Breccia

- From 0.00 -7.50m, SPT conducted N = 30.
- From 7.50 -10.50m depth, SPT conducted N>100.
- From 10.50 -16.50m depth, the core pieces mainly broken in small pieces.
- From 18.00 -21.00m depth, the core recovery 80% and RQD varies between 0- 16%.
- From 21.00 – 25.00m depth have crushed quartzite pieces.
- From 25.00 – 38.00m depth, the core recovery varies between 39% to 78% and RQD varies between 0 to 60%.
- Joints surface, smooth to rough and filled up with weathered material and are stained.

e) S140 Borehole:

Location of foundation with borelog /Stratum details:

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.525	855.205	808.122	796.000	12.122	70.000	3

Depth	Description of Stratum
0.00 – 1.50 m	Overburden from decomposition of altered rock (bauxite) and shale
1.50 – 7.50 m	Altered rock (Bauxite) and grey shale
7.50 – 18.00 m	Small pieces of Quartzite,
18.00 – 25.50 m	Sandstone
25.50 – 45.50 m	Small pieces of Quartzite, Bauxite and brecciated altered rock
45.50 – 64.50 m	Thinly bedded and fractured dolomite with shear zone
64.50 – 70.00 m	Rock is fractured dolomite

- From 1.50 – 7.50m depth, the core recovery varies 20% to 27% and RQD - NIL.
- From 10.50 – 12.00m depth have quartzite, altered rock and breccia. The core recovery - 24% and RQD - NIL.
- Joints surface, smooth to rough and stained between 1.50– 12.0m depth
- From 12.00 -18.00m depth, the core recovery varies from 17 to 30% and RQD - Nil.
- From 18.00 – 25.50m depth of quartzite rock, the core recovery varies between 16 to 28% and RQD - Nil. Core pieces - broken along the joints at various angles in to small pieces.
- From 25.50 – 38.00m depth of quartzite rock, the core recovery varies from 29 to 67% and RQD varies from 15 to 38.
- From 38.00 – 70.00m depth, the core recovery varies from 16% to 43% and RQD – Nil. Joints surface - smooth to rough and stained.

f) S130 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.475	855.080	823.017	813.000	10.017	30	1

Depth	Description of Stratum
0.00 – 1.50 m	Clayey silt with gravel i.e. overburden
1.50 – 6.50 m	Broken, grey coloured, brecciated rock
6.50 – 17.00 m	Rock is reddish, brecciated rock
17.00 – 19.00 m	Rock is reddish, brecciated rock
19.00 – 21.50 m	Rock is brocken and shattered dolomite
21.50 – 30.00 m	Rock is brocken and shattered dolomite

- From 6.50 – 17.00m depth, the core recovery varies from 22% to 100% and RQD varies NIL to 70%.
- From 17.00-19.00m depth of reddish, brecciated rock. Core recovery varies from 96% to 98% and RQD varies 53% to 71%.
- From 19.00 – 20.50m depth, the core recovery varies from nil to 80% and RQD - Nil.
- From 20.50 – 30.00m depth, the core recovery varies from 28% to 90% and RQD varies Nil to 40%. Core pieces recovered in this portion, broken along the joints at various angle.
- Joints surface, smooth to rough and stained between 6.50-30.0m depth.

g) S120 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.425	854.955	824.255	816.000	8.255	70.000	3

Depth	Description of Stratum
0.00 – 1.50 m	Clayey silt with gravel i.e. overburden
1.50 – 9.00 m	Fractured, chert – quartzite breccia
9.00 – 10.50 m	Greyish, fractured, chert – quartzite breccia
10.50 – 25.50 m	Small pieces of sandstone, suspected shear zone
25.50 – 30.00 m	Broken Sandstone
30.00 – 37.00 m	Bauxite rock
37.00 – 44.50 m	Rock is jointed to massive dolomite
44.50 – 55.00 m	Thinly bedded and fractured dolomite
55.00 – 70.00 m	Fractured and sheared dolomite

- From 0.00 -18.00m depth, the core recovery varies from 17% to 42% and RQD - Nil.
- From 18.00- 44.50 m depth, the core recovery varies from 20% to 48% and RQD - Nil.
- From 44.50 – 70.00m depth, the core recovery varies from 18% to 39% and RQD - Nil.
- Joint surface, smooth to rough and filled up with weathered rock with suspected shear zone between 10.50 -15.00m depth.

h) S110 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.375	854.830	820.440	809.000	11.440	30.000	1

Depth	Description of Stratum
0.00 – 1.50 m	Clayey silt with gravel i.e. overburden
1.50 – 6.00 m	Boulders of altered rock (Bauxite Formation)
6.00 – 18.00 m	Greyish, altered rock (Bauxite Formation)
18.00 – 30.00 m	Sheared dolomite, small pieces with rock powder

- From 6.00 -18.00m depth, the core recovery varies from 16% to 42% and RQD - NIL.
- Core pieces recovered in this portion, small in length and mostly broken along the joints at various angle.
- From 18.00- 30.00 m depth, the core recovery varies from 12% to 32% & RQD –NIL and joints surface, smooth to rough

i) S100 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.325	854.705	824.447	814.000	10.447	70.000	3

Depth	Description of Stratum
0.00 – 1.50 m	Clayey silt with gravel i.e. overburden
1.50 – 8.00 m	Boulders of chert quartzite breccia
8.00 – 11.00 m	Rock is chert quartzite breccia
11.00 – 12.50 m	Massive, chert quartzite 10reccias
12.50 – 21.50 m	Massive, chert quartzite 10reccias
21.50 – 27.50 m	Massive, chert quartzite 10reccias.
27.50 – 38.00 m	Massive, chert quartzite 10reccias
38.00 - 56.00 m	Massive fractured, jointed dolomite
56.00 – 70.00 M	Thinly bedded jointed and fractured dolomite

- From 0.00 -14.00m depth, the core recovery varies from 19% to 52% & RQD -NIL
- From 14.00 -24.50 m depth, the core recovery varies from 40% to 54% and RQD - 14% to 56%.
- From 24.50- 38.00m depth, the core recovery varies from 40% to 83% & RQD - NIL .
- From 38.00 – 55.00m depth, the core recovery varies from 20% to 45% and RQD - NIL.
- From 55.00 - 70.00m depth, the core recovery varies from 13% to 57% and RQD - NIL.
- Joint surface, smooth to rough and stained between 8.0- 35.0m depth.

j) S90 Borehole

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.275	854.580	831.752	825.000	6.752	26.000	1

Depth	Description of Stratum
0.00 – 1.50 m	Clayey silt with fragments of boulders
1.50 – 4.50 m	Greyish white, boulders, embedded in clay silt
4.50 – 9.50 m	Fractured altered, quartzite sandstone breccia
9.50 - 20.00 m	Altered rock, quartzite sandstone breccia
20.00 - 26.00 m	Altered rock, quartzite sandstone breccia

- From 4.50 - 9.50m depth, the core recovery varies from 20% to 44% & RQD – NIL and smooth to rough, stained joint surface.
- From 9.50 -20.00 m depth, the core pieces recovered in this portion, small in length and broken along the joints in various angles.
- From 20.00-26.00m depth, the core pieces recovered in this portion, small in length and broken along the joints in various angles and smooth to rough, stained joint surface.

k) S80 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.225	854.517	839.225	829.749	9.476	150.000	3

Depth	Description of Stratum
0.00 – 2.00 m	Yellowish brown, Sandy silt, with fragments of boulders i.e. overburden
2.00 – 4.60 m	Greyish white sandstone
4.60 – 18.20 m	Greyish, thin beds of shale in thick beds of quartzite
18.20 - 21.00 m	Crushed .Broken Quartzite
21.00 - 33.00 m	Slightly Altered quartzite, Breccia and Dolomite
33.00 – 45.00 m	Jointed dolomite with shear zone
45.00 – 54.00 m	Massive dolomite with shear zone
54.00 – 56.00 m	Jointed dolomite with shear zone
56.00 – 64.50 m	Jointed dolomite
64.50 – 67.50 m	Jointed dolomite with shear zone
67.50 - 103.00 m	Jointed and Massive dolomite
103.00 - 125.00m	Jointed dolomite joints with dip 30°,20° & 70°
125.00 – 150.00m	Massive and jointed dolomite

- From 0.00 – 33.00m depth, the core recovery varies from 10 to 33% & RQD – NIL
- From 33.00 – 53.50m depth, the core recovery varies from 20 to 50% & RQD varies from 12 to 39%.
- From 53.50 – 72.00m depth, the core recovery varies from 30 to 67% & RQD as NIL.
- From 72.00 – 103.00m depth, the core recovery varies from 39 to 95% & RQD varies from 19 to 74%.
- From 103.00 – 134.00m depth, the core recovery varies from 28 to 90% & RQD varies from 10 to 73%.
- From 134.00 – 150.00m depth, the core recovery varies from 32 to 92% & RQD varies from 14 to 58%.

l) S70 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found.Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.185	854.517	839.789	822.789	17.000	150.000	3

Depth	Description of Stratum
0.00 – 4.50 m	Yellowish brown, completely weathered dolomite in form of boulders in hard sandy silt
4.50 – 7.15 m	Pinkish, thin bands of quartzitein thick beds of shale
7.15 – 18.00 m	Greyish, thin bands of shale in thick beds of Quartzite
18.00 - 27.00 m	Greyish, brecciated, Quartzite
27.00 - 31.00 m	Crushed / broken jointed dolomite with joint dips 10°,20°,30° and 70°
31.00 – 32.50 m	Suspected Shear Zone
32.50 – 38.50 m	Jointed dolomite with jointed dips 10°,20°,30° and 70°
38.50 – 57.50 m	Jointed dolomite with jointed dips 20°,40° and 70°
57.50 – 65.00 m	Jointed dolomite with dips 30° to 50°
65.00 – 69.50 m	Crushed dolomite with shear zone with dips 30° to 50°
69.50 – 75.50 m	Jointed dolomite with dips 30° to 50°
75.50 – 91.00 m	Thinly bedded and fractured dolomite with shear zones
91.00 – 93.00 m	Jointed dolomite with dips 30° to 50°
93.00 -100.00 m	Thinly bedded and sheared dolomite with dips 30° to 50°
100.00 -135.00 m	Jointed dolomite with dips 30° to 50°
135.00 -137.00 m	Shear Zone
137.00 -145.00 m	Jointed Dolomite with dips 30° to 50°
145.00 -147.50 m	Shear Zone
147.50 -150.00 m	Dolomite with dips 30° to 50°

- From 0.00 -6.00m depth, the core recovery varies from 60 to 88% & RQD varies from 13 to 47%.
- From 6.00 -15.50m depth, the core recovery varies from 16 to 50% & RQD as NIL.
- From 15.50 -31.00m depth, the core recovery varies from 40 to 90% & RQD varies from 27 to 71%.
- From 31.00 - 150.00m depth, the core recovery varies from 27 to 90% & RQD as nil.
- Core pieces recovered are small in length and broken along the joints.
- Sloping ground between P80 and P50 to be cut in a number of small cuts, coupled with 5-6 m wide berms to increase the factor of safety of existing sloping ground to a value of 1.5m.

m) S60 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.130	854.517	806.854	777.699	29.155	150.000	3

Depth	Description of Stratum
1.50 – 6.00 m	Greyish, Fractured Dolomitic Limestone
6.00 – 7.50 m	Greyish, Dolomitic Limestone
7.50 – 15.50 m	Fractured Dolomitic Limestone
15.50 - 18.50 m	Crushed Dolomitic Limestone
18.50 - 23.00 m	Brecciated Dolomitic Limestone
23.00 - 27.50 m	Crushed Dolomite
27.50 - 32.00 m	Fractured Dolomitic Limestone
32.00 - 42.50 m	Fractured Dolomite with intermittent layers of crushed Dolomitic Limestone between 32.5 to 33.0m and 38.0m to 39.50m
42.50 – 54.50 m	Jointed dolomite
54.50 – 59.00 m	Fractured and jointed dolomite
59.00 – 80.00 m	Thinly bedded, fractured and sheared dolomite
80.00 -105.00 m	Massive dolomite
105.00 – 115.00 m	Thinly bedded, fractured and sheared dolomite
115.00 – 124.50 m	Jointed dolomite
124.50 – 127.50 m	Fractured dolomite
127.50 – 144.00 m	Thinly bedded, fractured and sheared dolomite
144.00 – 150.00 m	Fractured dolomite

- 100% water loss, during the course of drilling.
- From 1.50 - 10.00m depth, the core recovery -Nil & RQD – 13 to 48%
- From 10.00 - 25.00m depth, the core recovery varies from 33 to 67% and RQD - 10 to 33%.
- From 25.00 - 80.00m depth, the core recovery varies from 23 to 73% and RQD -NIL
- from 80.00-105.00m depth, the core recovery varies from 39 to 100% and RQD – 20 to 85%.
- From 105.00 - 150.00m depth, the core recovery varies from 10 to 33% and RQD - NIL.

n) S50 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
51.065	854.517	726.064	703.749	22.315	160.000	3

Depth	Description of Stratum
0.00 – 5.00 m	Fractured, Dolomitic Limestone
5.00 – 6.00 m	Crushed ,Dolomitic Limestone
6.00 – 10.50 m	Fractured, Dolomitic Limestone
10.50 - 24.00 m	Fractured, Dolomitic Limestone
24.00 – 28.50 m	Brecciated, Dolomitic Limestone
28.50 – 30.00 m	Shear Zone
30.00 - 38.50 m	Jointed dolomite
38.50 – 42.50 m	Massive to blocky dolomite
42.50 – 51.50 m	Joint dolomite
51.50 – 53.00 m	Shear Zone
53.00 – 60.00 m	Jointed dolomite

60.00 – 62.00 m	Shear Zone
62.00 – 65.00 m	Jointed dolomite
65.00 – 69.50 m	Massive dolomite
69.50 – 73.00 m	Jointed dolomite
73.00 – 76.00 m	Massive dolomite
76.00 – 77.50 m	Jointed dolomite
77.50 – 87.00 m	Shear zone is suspected
87.00 – 91.50 m	Jointed dolomite
91.50 – 110.00 m	Thinly bedded and fractured dolomite with shear zone
110.00 – 116.00 m	Jointed dolomite
116.00 – 134.50 m	Jointed dolomite irregular quartz veins
134.50 – 145.50 m	Fractured and sheared dolomite
145.50 – 150.00 m	Jointed dark grey dolomite
150.00 – 152.50 m	Fractured dolomite
152.50 - 160.00 m	Jointed dark grey dolomite with shear seams

- Bore hole located on a very steep slope and hill is slopping towards south of an angle of about 50 -55 Degree.
- Down the hill, slope is more or less continuing at same angle.
- 100% water loss during the course of drilling.
- From 0.00 – 7.50m depth, the core recovery 21% to 50% and RQD - NIL.
- From 7.50 – 20.00m depth, the core recovery varies 20 to 90% and the RQD -NIL.
- From 20.00 – 42.50m depth, the core recovery varies 27 to 73% and the RQD – 10 to 27%.
- From 42.50- 150.00m depth, the core recovery varies 10 to 23% and RQD - NIL.
- Core pieces recovered between 0.00-70.0m, small in length and broken along the joints.

o) S40 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
50.585	854.517	703.223	671.100	32.123	150.000	3

Depth	Description of Stratum
0.00 – 10.50 m	Greyish, Thinly bedded Fractured Dolomitic Limestone
10.50 – 12.00 m	Crushed Dolomitic Limestone
12.00 – 22.50 m	Greyish, Thinly bedded Dolomitic Limestone
22.50 - 27.00 m	Crushed Dolomitic Limestone
27.00 - 37.50 m	Brecciated Dolomitic Limestone
23.00 - 27.50 m	Crushed Dolomite Limestone
27.50 – 37.50 m	Thinly bedded Fractured Dolomitic Limestone
37.50 - 40.50 m	Jointed and Fractured /Crushed Dolomitic Limestone
40.50 – 42.50 m	Shear Zone
42.50 – 49.50 m	Jointed dolomite
49.50 – 53.00 m	Massive dolomite
53.00 – 57.50 m	Jointed dolomite
57.50 – 62.00 m	Massive dolomite
62.00 – 64.00 m	Jointed dolomite
64.00 – 70.50 m	Massive dolomite
70.50 – 74.00 m	Jointed dolomite
74.00 – 75.00 m	Shear Zone
75.00 – 83.00 m	Jointed dolomite
83.00 – 84.50 m	Shear Zone
84.50 – 89.00 m	Jointed dolomite

89.00 – 104.00 m	Jointed to blocky dolomite
104.00 – 106.00 m	Jointed to blocky dolomite
106.00 – 120.00 m	Jointed dolomite
120.00 – 125.00 m	Thinly bedded, fractured and sheared dolomite
125.00 – 129.00 m	Fractured dolomite
129.00 – 144.00 m	Sheared and crushed dolomite
144.00 – 150.00 m	Fractured Dolomite

- RQD at founding level of foundation is Nil and core recovery varies from 60 to 75%.

p) S30 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
50.520	854.517	744.400	728.70	15.700	150.000	3

Depth	Description of Stratum
0.00 – 0.60 m	Hard silty clay with gravel
0.60 – 12.00 m	Thinly bedded Fractured Dolomitic Limestone
12.00 – 18.00 m	Crushed Dolomitic Limestone (Suspected Shear Zone)
18.00 - 22.60 m	Thinly bedded Fractured Dolomitic Limestone
22.60 - 32.00 m	Jointed and Crushed Dolomitic Limestone
32.00 – 33.00 m	Shear Zone
33.00 – 42.50 m	Jointed dolomite
42.50 – 44.00 m	Shear Zone
44.00 – 51.50 m	Jointed dolomite with core generally broken into small pieces with clay
51.50 – 61.00 m	Shear Zone
61.00 – 91.00 m	Thinly bedded and fractured dolomite with thin shear seams
91.00 – 95.00 m	Thinly bedded dolomite
95.00 – 119.00 m	Jointed dolomite with slate bands
119.00 -121.00 m	Shear Zone
121.00 -132.00 m	Hematite dolomite reddish to grey in colour
132.00 -150.00 m	Sheared, crushed and fractured grey coloured dolomite

- From 0.60- 12.00m depth, the core recovery varies from 16% to 24% and RQD - Nil.
- From 12.00- 18.00m depth, the core recovery and RQD - Nil.
- From 18.00- 21.00m depth, the core recovery varies from 21% to 30% and RQD -Nil .
- From 21.00- 30.00m depth, practically the core recovery and RQD - Nil.
- Core pieces recovered between 0.00 – 30.00m, small in length and broken along the joints.

q) S20 Borehole:

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. Borehole carried out
50.455	854.517	793.169	777.900	15.269	150.000	2

Depth	Description of Stratum
0.00 – 3.00 m	Overburden consisting of boulders
3.00 – 6.00 m	Thinly bedded Fractured Dolomitic Limestone
6.00 – 7.50 m	Crushed Dolomitic Limestone (Suspected Shear Zone)
7.50 - 18.10 m	Thinly bedded Fractured Dolomitic Limestone
18.10 - 20.00 m	Jointed dolomite
20.00 - 35.00 m	Thinly bedded Fractured Dolomite with thin shear seams
35.00 – 36.00 m	Jointed dolomite
36.00 - 81.50 m	Thinly bedded and fractured dolomite with thin shear seams
81.50 – 90.00 m	Thin bedded greyish to reddish coloured and sheared hematite dolomite
90.00 – 92.00 m	Sheared dolomite
92.00 - 97.00 m	Brownish cherty dolomite with shear zone
97.00 – 103.00 m	Gray dolomite
103.00 -109.00 m	Thinly bedded and sheared dolomite
109.00 -128.00 m	Thinly bedded and fractured dolomite
128.00 -149.00 m	Thinly bedded, fractured and sheared dolomite
149.00 -150.00 m	Jointed dolomite

- From 3.00- 6.00m depth, the core recovery varies from 65% to 91% and RQD varies between 50% to 80%.
- From 6.00- 150.00m depth, the core recovery varies from 40% to 77% (avg.) and RQD - Nil.

r) S10 Borehole

Location of foundation with borelog /Stratum details

Chainage (Km)	Top of Rail (m)	Ground Lvl. (m)	Found. Lvl. (m)	Depth of Foundation from GL (m)	Max. Depth of Borehole (m)	No. of Borhole carried out
50.400	854.517	843.016	831.000	12.016	30.000	1

Depth	Description of Stratum
0.00 – 4.50 m	Overburden consisting of boulders
4.50 – 10.50 m	Thinly bedded Fractured Dolomitic Limestone
10.50 – 12.00 m	Crushed Dolomitic Limestone (Suspected Shear Zone)
12.00 - 15.00 m	Thinly bedded Fractured Dolomitic Limestone
15.00 - 18.00 m	Crushed Dolomitic Limestone (Suspected Shear Zone)
18.00 - 25.50 m	Thinly bedded Fractured Dolomitic Limestone
25.50 - 30.00 m	Crushed Dolomitic Limestone

- From 4.50- 10.50m depth, core recovery varies from 20% to 36% and RQD varies between 0% to 30%.
- From 10.50- 30.00m depth, core recovery and RQD - NIL.
- Core pieces recovered from 4.50- 30.00m depth, small in length and broken along the joints.

2.3.4.1 Summary of Properties of rock samples from Borehole Foundation S10 -S40 & S50-S70, Tests carriedout by **NIRM & ANGRON (April'2005)** from above borehole samples:

a. S10- S40 Boreholes

Properties	Value	Properties	Value
Strength of Rock (kg/cm^2)	250 - 500	Vp (m/sec)	4658-6527
Avg. Spacing of joints (mm)	<60	Vs (m/sec)	3403-3714
RMR Value	36 – 42	Young Modulus (GPa)	61
Permeability (Lugeon Value)	10 - 3	Poisson's ratio	0.196
Sp. Gravity	2.76 - 2.84	Tensile Strength (Mpa)	11.12 – 14.46
Density (gm / cm^3)	2.63 - 2.82	Triaxial Comp. PSIGMA3 (Mpa)	10 - 20
Porosity (%)	1.37 – 2.94	Triaxial Comp. PSIGMA1 (Mpa)	177 - 417
Water absorption (%)	0.35 – 6.28	'C' (MPa)	8.9
U C S (kg / cm^2)	445 – 1581	Phi (Degree)	56°
Point L. Stren. Index (Mpa)	3.183 – 6.764	'm'	49
Slake Durability Index	99.17%	Shear Stiffness, MPa/mm	0.58- 8.84
		Normal Stiffness, MPa/mm	223 – 774

b) S50 – S80 Boreholes

Physical Properties	Value	Physical Properties	Value
Strength of Rock (kg/cm^2)	250 – 500	Vp (m/sec)	4204 – 6480
Avg. Spacing of joints (mm)	60 – 200	Vs (m/sec)	3429 – 3716
RMR Value	35 – 50	Young Modulus (Gpa)	65
Permeability (Lugeon Value)	23 – 14	Poisson's ratio	0.15 – 0.17
Sp. Gravity	2.77 – 2.89	Tensile Strength (Mpa)	9.11 – 11.98
Density (gm / cm^3)	2.61 – 2.84	Triaxial Comp. PSIGMA3 (Mpa)	8 – 25
Porosity (%)	0.7 – 7.47	Triaxial Comp. PSIGMA1 (Mpa)	83 – 297
Water absorption (%)	0.32 – 0.59	'C' (Mpa)	20.0
U C S (kg / cm^2)	539.62 – 638.91	Phi (Degree)	59.3°
		'm'	24
Shear Stiffness, MPa/mm	0.57 – 3.38	Normal Stiffness, MPa/mm	201-575

c) **Summary Laboratory Results of Joint Materials**

Joint Properties of Basic friction angle, Residual friction angle and Joint wall Comp. Strength

S.N.	Basic 'Ø°'	Residual 'Ø°'	Joint wall Comp. Strength, MPa
1	41	34	292
2	40	32	327
3	38	30	120
4	37	28	68
5	37	29	234
6	41	34	65
7	40	32	120
8	38	30	234
9	37	28	292
10	37	29	234

2.3.4.2 Summary of Physiacl & Mechanical Properties of Rock of all borehole foundations carried out by ANGRON (July'2005):

Found Location / Properties	S180	S170	S160	S150	S140	S130	S120	S110	S100	S90
Strength of Rock (kg/cm^2)	250 - 500	50 - 500	50 - 500	50 - 500	50 - 500	50 - 500	50 - 250	50 - 500	50 - 500	50 - 500
Avg. Spacing of joints (mm)	60 - 200	60 - 200	60 - 200	60 - 200	<60	60 - 200	<60	<60	<60	<60
RMR Value	34 - 50	30 - 42	31 - 47	30 - 55	37 - 40	40 - 53	35 - 40	35 - 53	35 - 53	35 - 42
Permeability (Lugeon Value)	NA	26	NA	NA	25-7	31	12 - 8	NA	2.40	17
Sp. Gravity	2.71 - 2.82	2.69 - 2.84	2.60 - 2.72	2.68 - 2.86	2.62 - 2.85	1.67 - 2.95	2.66 - 2.76	2.80	2.66 - 2.78	2.69 - 2.82
Density (gm / cm^3)	2.69 - 2.80	2.67 - 2.81	2.59 - 2.76	2.63 - 2.79	2.61 - 2.81	2.59 - 2.81	2.62 - 2.73	2.74	2.65 - 2.79	2.63 - 2.75
Porosity (%)	1.38 – 1.56	0.44 – 2.86	3.13 – 12.72	0.42 – 1.69	0.38 – 1.66	0.45 – 2.99	1.55 – 5.26	2.09	0.87 – 1.11	0.73 – 2.31
Water absorption (%)	0.39 – 0.45	0.38 - 0.67	0.84-2.63	0.25 – 0.93	0.24 – 1.24	0.46 – 3.19	0.58 – 1.74	1.00	0.71 - 0.89	0.4 - 0.97
U C S (kg / cm^2)	308.5 - 417.2	221.9 – 439.50	48 – 274.0	138.1-673.9	53.6 – 923.0	41.2 – 331.0	74.5 – 214.2	288.3	213.5 – 299.3	163.9 - 276.1
Modulus of Elasticity (Kg/cm^2)	NA	0.24 – 0.816 x 10^6	NA	0.307 x10^6	NA	0.4838 x10^6	0.1621 x10^6	NA	NA	NA
Poisson's Ratio	NA	0.10-0.347	NA	0.256	NA	0.202	0.157	NA	NA	NA

Found Locations / Properties	S80	S70	S60	S50	S40	S30	S20	S10
Strength of Rock (kg/cm^2)	250 - 500	250 - 500	50 - 500	250 - 500	250 - 500	50 - 500	250 - 500	250 - 500
Avg. Spacing of joints (mm)	<60	<60	60-200	60-200	<60	<60	<60	<60
RMR Value	40 - 42	35-53	39 – 50	35 – 50	35 – 50	35 – 42	40 – 42	37 – 42
Permeability (Lugeon Value)	NA	7 - 6	7 - 6	23 -14	8 - 3	6 - 1	5-2	70
Sp. Gravity	2.79 - 2.82	2.80 - 2.84	2.67 - 2.83	2.65 - 2.83	2.70 - 2.94	2.80 - 2.84	2.80 - 2.84	2.79 - 2.93
Density (gm / cm^3)	2.78 - 2.81	2.63 - 2.81	2.66 - 2.80	2.61 - 2.84	2.67 - 2.89	2.75 - 2.82	2.75 - 2.82	2.77 - 2.91
Porosity (%)	1.65 - 3.55	0.82 – 4.82	0.82 – 1.47	0.81 – 1.99	0.72 – 2.94	0.77 - 1.21	0.77 - 1.21	1.43 - 1.51
Water absorption (%)	0.64 - 0.82	0.29 – 0.82	0.29 – 0.82	0.28 – 0.70	0.25 – 0.69	0.33 – 0.53	0.33 – 0.53	0.38 – 0.96
U C S (kg / cm^2)	304.6 – 342.0	329.0– 666.2	123 - 521	1530 - 1640	560 - 2100	290 – 329	290 – 329	227 – 329.7
Strength Index (N / mm^2)	8.883	3.66- 7.783	4.529	6.04	6.764	2.32 -3.20	2.785	1.592
Tensile Strength (N / mm^2)	6.50 – 13.58	7.80 – 13.06	7.39 – 13.91	9.11 – 11.98	14.46 – 11.12	7.39-19.23	5.72	5.57 – 7.71
Modulus of Elasticity (Kg/cm^2)	0.24 x 10^6	0.076 x10^6	0.1875 x 10^6	0.7692 x 10^6	0.561 x 10^6	0.909 x 10^6	0.666 x 10^6	0.909 x 10^6
Poisson's Ratio	0.040	0.147	0.10-0.20	0.15-0.37	0.19	0.165-0.28	0.165 - 0.194	0.272

3.0 Evaluation & Recommondations of Safe Bearing Capacity:
The safe Bearing Ccapacity for all the foundation except (**S150 to S180**) have been calculated/estimated based on above parameters with following different considerations:RMR Value

i) RQD Value
ii) Unconfined Strength of Rock Cores tested
iii) Plate Load Tests
iv) In-situ Direct Shear Tests duly considering the excavated profiles using the charts given in the book "Foundations on Rock" by Duncan Wylie.
v) FLAC Analyses of rigid foundations on the actual slopes for assessing safe bearing capacity considering the observed settlements. Safe bearing capacity is assessed using the load settlement curves and SBC is evaluated corresponding to a very low settlement of 12 mm at these foundations on slopesSafe / Allowable Bearing Pressures from above various methods has been evaluated / determined.The Safe Bearing Capacity is a function of the strength of the rock mass, topography of the strata and limiting settlements. While RMR method and Plate Load Tests consider the limiting settlements, the other methods do not factor in the limiting settlements. Considering the limitation on the settlements to 12mm, the lower value obtained from Plate Load Tests is recommended for adoption for design purposes.

Based on above study IISc Banglore recommonded followings:

a. Open foundation for S10-S140 Piers has been proposed / contructed at thick layers of crushed / fractured dolomitic limestone / quartzite rock with an allowable earing pressure of **100 T/ m^2** .
b. Joint mapping to be done on the vertical sided and at the base of the foundation after excavations for open foundations.
c. Stratum to be strengthen by carrying out consolidated cement grouting in the rock, beneath foundation. Grouting upto a depth of about equal to the width of the foundations below the proposed founding level by drilling 1.5 x 1.5 grid has been proposed.
d. Before placing the foundation, the excavated surface has to be throughly examined for any loose pockets and to be filled with lean concrete.
e. Pile foundation of 1000 mm diameter have been provided in Pier (S150 – S180) as per the soil / rock stratum. Details are summerised below:
   - For S180 foundation, 37 No. - 1000mm dia pile capacity of 450T (friction cum end bearing), of 37.700m deep with sockecting length of 3.0m in Quartzite upto has been proposed & constructed.
   - For S170 foundation, 24 No.- 1000mm dia pile capacity of 450 T(friction cum end bearing), of 22.30m deep with sockecting length of 3.0m in Quartzite upto has been proposed & constructed.
   - For S160 foundation, 24 No. - 1000mm dia pile with capacity - 450 T(friction cum end bearing), of 21.60m deep with sockecting length of 3.0m in Quartzite upto has been proposed & constructed.
   - For S150 foundation, 24 No. - 1000mm dia pile with capacity - 450 T(friction cum end bearing), of 21.40m deep with sockecting length of 3.0m in Quartzite upto has been proposed & constructed

4.0 Selection of Geotechnical Parameters for Design of Slopes
Further, two drifts (size of 2.0 x 2.0m x 40m) were made S40 & S50 location above 15m from founding level of S40 & S50 foundations in the year'2009 and follwing studies have been carried out in drift:

Drift S40

Drift S50

4.1 Tests (Cyclic Plate Load Test & Insitu Direct Shear Test) for Foundation S40 & S50 (June'2009)
Cyclic Plate Load Test (CPLT) and Insitu Plate load test (PLT) have been carried in the drift of S40 & S50 alongwith rock samples were also tested in laboratory. The details are summerized below


Plate load test inside Drift at U-50 of Chenab Bridge(Arch foundation side) 13/05/2009

Plate Load Test in Drift

a) Summary of observation of CPLT at drift S40

SN	Parameter	CPLT-1	CPLT-2	CPLT-3
1	Total settlement of test plate under 354 t/m2 pressure	2.50mm	2.28mm	3.74mm
2	Estimated settlement of 10m2 footing under 354 t/m2 pressure	4.69mm	4.28mm	7.02mm
3	Estimated settlement of 2000m2 footing under 354 t/m2 pressure	5.56mm	5.06mm	8.31mm
4	Recommended safe bearing capacity at CPLT-2 Location	100 t/m2	100 t/m2	100 t/m2
5	Co-efficient of uniform elastic comp. Cu for test plate	850 kg/cm3	750 kg/cm3	550 kg/cm3
6	Modulus of sub-grade reaction Kz for test plate	125 kg/cm3	210 kg/cm3	85 kg/cm3

b) In situ direct shear test at drift S40

- From graph of shear stress vs normal stress, following 'C' & 'Ø' values have been evaluated:

Apparent Cohesion **(C) = 1.4 Mpa,**
Angle of friction **(Ø) = 44.42°**

- Even after completion of test, the test blocks intact and only some minor cracks found in the vicinity of boundary surfaces.

C) Summary of observation of CPLT at drift S50

SN	Parameter	CPLT-1	CPLT-2	CPLT-3
1	Total settlement of test plate under 354 t/m^2 pressure	0.27mm	1.75mm	0.91mm
2	Estimated settlement of 10m2 footing under 354 t/m^2 pressure	0.50mm	3.28mm	1.70mm
3	Estimated settlement of 2000m2 footing under 354 t/m^2 pressure	0.60mm	3.89mm	2.02mm
4	Recommended safe bearing capacity at CPLT-2 Location	100 t/m^2	100 t/m^2	100 t/m^2
5	Co-efficient of uniform elastic comp. Cu for test plate	9500 kg/cm^3	2000 kg/cm^3	2500 kg/cm^3

d) In situ direct shear test at drift S50

- From graph of shear stress vs normal stress, following 'C' & 'Ø' values have been evaluated:

Apparent Cohesion (C) = **1.44 Mpa,**
Angle of friction (Ø) = **44.61°**

- Even after completion of test, the test blocks intact and only some minor cracks found in the vicinity of boundary surfaces.

e) Physical Properties of Rock Test Results:

Location RL (M)	Unit Wt. (g/cc)	Sp. Gravity	Stake durability (%)	P Wave (m/s)	S Wave (m/s)	C (kg/cm^2)	Ø°
S 40 / 666.0	-	-	-	3860	2458	-	-
S 40 / 687.5	-	-	-	5760	3668	-	-
S40 / Drift /654.0	2.77	2.80	-	-	-	61.5	48.6
S50 / 680.0	-	-	99.30	-	-	-	-
S50 / 679.5	-	-	-	4780	3144	-	-
S50 / 684.0	-	-	-	4180	2750	-	-
S50 / 678.0 to	2.73	2.82	-	-	-	37.5	54.7
S50 / Drift /710.0	-	-	98.20	-	-	-	-

4.2 NGRI (July'2009) has carried out Geophysical Survey (MASW) inside Drift S50 for estimation of in-situ shear wave velocity (Vs) using Multi Channel Analysis of Surface Wave (MASW). MASW studies along 2 profile inside each drift of bridge site. Results shows that Vs is varrying from 400 to 1200 m/s and S velocity as 800 to 2000 m/s which seems to be unrealistic.

4.3 IITR (Dec'2009) had carried detailed laboratory tests of rock spalls excavated from S70-S80 and shear strength parameters (C & Ø) has been obtained from Direct Shear Tests (DST) as mentioned below:

- Liquid & plastic limit tests, conducted as per IS:2720 (Part V) -1995 for plasticity characteristic.
- Sample consists 91.4% gravels, 7.9% sands and 0.7% fines of non plastic.
- 4 direct shear tests, conducted in lab. Using a 300mm x 300mm shear box.
- Tests conducted at four normal stresses (25,50,100 and 200KPa) on samples at simulated density of 20 KN/m^3 .
- Results depicted in the form of shear strength vs. normal stress.
- Shear strength parameters, **Cohesion (C) -18.8Kpa**, and Angle of Shearing Resistance **(Ø) – 40.8°.**

4.4 Geotechnical Investigations (April'2010) has been carried out to reconfirm the properties for Pier foundation of S40 - S60. Two borehole has been drilled at S40 & S60 foundation loaction up to 49.50m & 66.50m respectively.The details are as follows:

a) Borelog data

Found. location	Depth (m)	Stratum
S40	0.00 – 1.00	Overburden consist of rock debris and soil
	1.00 – 49.50	Massive to highly jointed Dolomitic Limestone
S60	0.00 – 5.00	Brecciated quartzite
	5.00 – 66.50	Massive to highly jointed Dolomitic Limestone

b) Properties of Rock Sample

Found. Location	Depth (RL, m)	Dry Dens. gm / cc	Sp. Gravity	Strength (kg/cm^2)	Tangent Modulus	C (kg/cm^2)	Ø
S 40	671.5 - 675.5	2.73	-	520	14.3×10^3	0.70	59
S 40	671.5 - 675.5	2.57	2.69	329	21.9×10^4	-	-
S 60	784.916 – 787.416	2.88	-	835	21.05×10^5	79.33	53
S 60	784.916 – 787.416	2.43	-	32	99.47×10^4	-	-

***Tests were conducted in saturated condition.**

Found. Location	Dry Density (gm / cc)	Triaxial Test C (kg / cm^2)	Triaxial Test Ø°	Point Load test values (Is_{50}) (kg / cm^2)
S 40	2.75	0.00	56.92	55, 56, 27, 80

4.5 ANGRON (Dec'2013) has tested the joint filling material of Drifts S50 and concuded the test results as below **:**

Sample No.	Joint Plate	*Direct Shear (In situ MC)		*Direct Shear (In situ MC)	
PARAMETERS		**'C' MPa**	**'Ø°'**	**'C' MPa**	**'Ø°'**
1	J3	0.00	39.6	0.00	41.2
2	J3	0.00	40.0	0.00	44.1
3	J1	0.00	37.6	0.00	37.5
4	J1	0.00	36.9	0.00	38.7
5	J1	0.00	38.9	0.00	39.9

•Residual stage after repeated shearing

4.6 IIT Roorkee (Dec'2013) also tested the joint filling material of Drifts S50 and summerised as below:

Sample No.	Joint Plate	Strength Parameters at NMC		Strength Parameters after Saturation	
PARAMETERS		**'C'** MPa	**'Ø°'**	**'C'** MPa	**'Ø°'**
6	J1	0.0438	34.7	0.020	35.0
7	J3	0.0532	30.8	0.024	34.6
8	J3	0.005	39.4	0.0325	33.2
9	J3	0.0068	29.6	0.0325	29.0
10	J3	0.0507	30.3	0.0178	33.9

- *M/s CBPU carried out insitu direct shear test at pier location(S70) in Dec'2006. The determinated vaue of **'C'** & **'Ø°'** in the report are 0.57 Mpa (5.7 kg/cm^2) & 3° respectively. The reason for getting such low value is mentioned as **"Low value of (Ø) due to high saturation in rock mass due to rainy season during test and value of 'C' indicate fair to good condition of the rock mass."** in the report. The Proof Consultant has also reviewed the data and recomonded it as unrealistic and stated that "From reports, it can be seen that **'C'** & **'Ø°'** and other parameters considered in foundation design are in order to various test carried out except *** 'C'** & **'Ø°'** value as mentioned in NIRM'2005 reports. **'C'** & **'Ø°'** value of NIRM report are having high variation in values and seems to be unrealistic."*

4.9 ATES (April'2014) has also conducted Flexible Dilatometer Tests in Drift S40 & S50 to asses the Elastic Modulus, Deformation Modulus and Poisson's ratio.

4.10 **CSMRS (May'2014)** has tested the joint fill material of Drifts S40 & S50 and tabulated as below

i) Triaxial Shear Test - Consolid. Undrained with Pore Water Pressure:

Sample No.	Joint Plate	Insitu Bulk Density (g/cc)	Insitu Dry Density (g/cc)	Total Shear Parameters		Effective Shear Parameters	
PARAMETERS				**'C'** MPa	**'Ø°'**	**'C'** MPa	**'Ø°'**
1	J1/3 (S50)	1.97	1.88	0.016	28.2	0.006	36.0
2	J1/3 (S50)	1.79	1.72	0.022	26.3	0.012	32.4
3	J1/3 (S50)	1.733	1.71	0.005	39.4	0.0325	33.2
4	S40/3	1.99	1.95	0.027	23.6	0.018	31.8

ii) Direct Shear Test :

Sample No.	Joint Plate	Insitu Bulk Density (g/cc)	Insitu Dry Density (g/cc)	Shear Parameters	
PARAMETERS				**'C'** MPa	**'Ø°'**
1	J1/3 (S50)	1.79	1.72	0	38.5
2	J40/3 (S40)	1.99	1.95	0	34.7

5.1 Based on above all the study, IISc Banglore has considered following properties of Rock in consultation witth IIT Delhi & Proof Consultant (URS,UK) in design of slopes.

a) S10 – S40

Weathered Rock Mass	Values	Sub Set 1	Values
Dry Unit Weight (KN/ m^3)	25.97	Cohesion (C, MPa)	0.5
Cohesion (C, MPa)	0.8	Friction Angle (Φ) degrees	41
Friction Angle (Φ) degrees	40	**Sub Set 2**	
		Cohesion (C, MPa)	0.63
Rock Mass		Friction Angle (Φ) degrees	51
Set-1		**Set-2**	
Dry Unit Weight (KN/ m^3)	27.095	Dry Unit Weight (KN/ m^3)	26.752
Cohesion (C, MPa)	1.40	Cohesion (C, MPa)	0.531
Friction Angle (Φ) degrees	44.42	Friction Angle (Φ) degrees	49.10
Bulk Modulus (GPa)	50.55	Bulk Modulus (GPa)	6.31
Shear Modulus (GPa)	37.92	Shear Modulus (GPa)	4.73
Hoek & Brown parameter (m and s)	4.70 & 0.00127	Hoek & Brown parameter (m and s)	0.793 and 0.0005
RQD / RMR	49 / 48	RQD / RMR	10 / 37
ru	0.3		

b) S50 – S80

Weathered Rock Mass	**Values**	**Sub Set 1**	**Values**
Dry Unit Weight (KN/ m^3)	25.97	Cohesion (C, MPa)	0.8
Cohesion (C, MPa)	0.8	Friction Angle (Φ) degrees	41
Friction Angle (Φ) degrees	40	**Sub Set 2**	
		Cohesion (C, MPa)	0.72
Rock Mass		Friction Angle (Φ) degrees	51
Set-1		**Set-2 for S50 & S60**	
Dry Unit Weight (KN/ m^3)	27.095	Dry Unit Weight (KN/ m^3)	27.095
Cohesion (C, MPa)	1.41	Cohesion (C, MPa)	0.525
Friction Angle (Φ) degrees	42.61	Friction Angle (Φ) degrees	49.06
Bulk Modulus (GPa)	50.55	Bulk Modulus (GPa)	5.64
Shear Modulus (GPa)	37.92	Shear Modulus (GPa)	4.23
Hoek & Brown parameter (m and s)	4.706 & 0.00127	Hoek & Brown parameter (m and s)	0.766 and 0.0005
RQD / RMR	49 / 48	RQD / RMR	10 / 36
Set-2 for S70		**J1 and J3 (in fill material)**	
Dry Unit Weight (KN/ m^3)	25.86	**J1**	
Cohesion (C, MPa)	0.718	Cohesion (C, MPa)	0.0
Friction Angle (Φ) degrees	41.84	Friction Angle (Φ) degrees	37
Bulk Modulus (GPa)	4.09		
Shear Modulus (GPa)	3.27	**J3**	
Hoek & Brown parameter (m and s)	0.333 and 0.0009	Cohesion (C, MPa)	0.0
RQD / RMR	40 / 42	Friction Angle (Φ) degrees	41

5.0 Properties of Rock considered by M/s ITASCA in design of slope:

After going through all the documents as mentioned above, M/s ITASCA has considered following properties of rock in design of slope stability:

Implicit Value (End)	**Density** (kg/m^3)	**Bulk Modul.** (GPa)	**Shear Modul.** (GPa)	**'C'** (MPa)	**Friction Angle** (Deg.)	**Dilation Angle** (Deg.)	**Tensile Strength** (MPa)	**J**cohes. (MPa)	**J**friction **Angle** (Deg)
S10 –S40	**2,700**	**2.5**	**2.0**	**0.3**	**52**	**10**	**0.04**	**0.8**	**37**
S50 –S80	**2,700**	**2.1**	**1.7**	**0.3**	**57**	**10**	**0.01**	**0.8**	**37**

Explicit Joint	**Stiffness**		**'C'** (MPa)	**Friction Angle** (Deg.)		**Tensile Strength** (MPa)	
End	**Normal** (GPa/m)	**Shear** (GPa/m)		**Peak**	**Residual**	**J2**	**J3**
S10 –S40	**2.6**	**1.4**	**0.1**	**37**	**29**	**0.65**	**0.8**
S50 –S80	**4.4**	**2.2**	**0.1**	**37**	**29**	**0.65**	**0.8**

Dr. Loren Lorig, Dr. Branko Damjanac (Experts of M/s ITASCA USA) & Dr. Duncan Wyllie (Expert of Foundations on rock) had visited bridge site in Dec.2013 & Jan'2014 respectively. All of them appreciated the extent of geotechnical studies carried out for Chenab Bridge.

The above shows that comprehensive Geo- technical investigations were carried out at bridge site to arrive at the design parameters for foundations, cut profile & slope stability etc. Design of cut profiles of S10-S40 and S50-S70 has been completed and works are in progress at site. Design of foundations of all piers (S10-S180) is completed and the construction of foundations & piers (S90 – S180) is completed. Remaining works is in progress.

Slope Stability Analyses of ABUTMENT SLOPES FOR A Special Bridge no 44 Across River Chenab at KM 50/800 on the Katra-Loale section of the Udhampur-Srinagar-Baramulla Rail link project

Executive summary: The slope stability analyses of abutment slopes in jointed rock mass for a special bridge no 44 across river chenab at km 50/800 on the katra-loale section of the udhampur-srinagar-baramulla rail link project has been presented in this paper. The static and seismic slope stability analysis of the left and right abutment of the railway bridge proposed at about 359 m above the ground level slopes has been carried out at Indian Institute of Science Bangalore. Chenab Bridge is for crossing the Chenab river and connecting two large hillocks near Katra-Laole section in the Himalayas, India. The railway bridge is being constructed between chainage 50.4 on left abutment near Bakkal village and chainage 51.715 on right abutment near Kauri village. The railway track will be passing over the Chenab river at a height of about 359 m. The proposed railway bridge consists of total 18 piers resting on ground. Among these piers, 4 piers (S10, S20, S30, S40) are resting on left abutment and the other 14 piers (S50-S180) are resting on right abutment. The foundations at locations S10 to S70 are coming on the slopes while the other foundations are coming on a relatively flat terrain on the Kauri end approach. The rock slope is composed of highly jointed rock mass and the joint spacing and orientation are varying at different locations. Static, pseudo static and dynamic analyses of the slope are carried out numerically using program SLIDE and FLAC. The results obtained from all these analyses confirmed the global stability of the slope as the factors of safety against slope failure obtained from static and pseudo static analyses are adequate and the displacements observed from dynamic analyses are well within the permissible limits. Kinematics of the slope at different pier locations is also checked using stereographic projections and recommendations to avoid wedge failures are presented. Wedge analyses have been carried out using a large amount of joint mapping data and geotechnical parameters obtained at both these locations. Some slopes have been flattened to avoid any wedge failures and in some slopes strengthening measures using rock bolts has been designed. Slopes are stable enough to take the loads from the bridge and it would be an important civil infrastructure in India once it is completed. The entire work is under progress and the excavation of abutment slopes work is going on at both abutments.

1. Introduction

The evaluation of stability of the natural rock slopes becomes very essential for the safe design especially when the slopes are situated close to structures which are built on these slopes. The stability of a natural slope becomes more critical if the slope is situated in earthquake prone areas. Slope failures are the most common natural hazards and are mainly caused due to the additional forces due to foundations of the structures on them or rainfall induced or earthquake induced ground shaking and associated inertial forces. Earthquakes of even a very small magnitude may trigger failure in slopes in jointed rock masses which are perfectly stable otherwise. Hence the study of the behaviour of rock slopes in order to have a safe design of structures built on them. Though the strength of the rock plays an important role in the slope stability, geological structure of the rock often govern the stability of slopes in jointed rock masses.

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Geological characteristics of rock mass include location and number of joint sets, joint spacing, joint orientations, joint material and seepage pressure. The dynamic analysis of slopes in rock masses is studied by several earlier researchers using different techniques.

The railway bridge is being constructed between chainage 50.4 on left abutment near Bakkal village and chainage 51.715 on right abutment near Kauri village. The railway track will be passing over the Chenab river at a height of about 359 m. The proposed railway bridge consists of total 18 piers resting on ground. Among these piers, 4 piers (S10, S20, S30, S40) are resting on left abutment slope and the other 14 piers (S50-S180) are resting on right abutment slope. The foundations at locations S10 to S70 are coming on the slopes while the other foundations are coming on a relatively flat terrain on the Kauri end approach. A railway line is being laid in Jammu and Kashmir, India and this line is crossing the river Chenab at a height of about 359 m. A bridge is being constructed with total 18 piers at this place connecting two big hillocks and the bridge forms about 350 m deep gorge in a V shaped valley in this area. Among these piers, 4 piers (P10–P40) are resting on left abutment and the other 14 piers (P50–P180) are resting on right abutment. Slope stability analysis of the right abutment is taken up in the present study. The section of the bridge and abutments along with the foundations that could affect the stability of the slope is given in Figure 1. The photograph taken at the proposed bridge site is presented in Figure 2. Figure 3 shows the photograph of the constructed piers in the valley from P180 to P90. Presently excavations for founding the foundations (S10 to S40 on Bakkal side and S50 to S80 on Kauri side) on the abutment slopes are in progress.

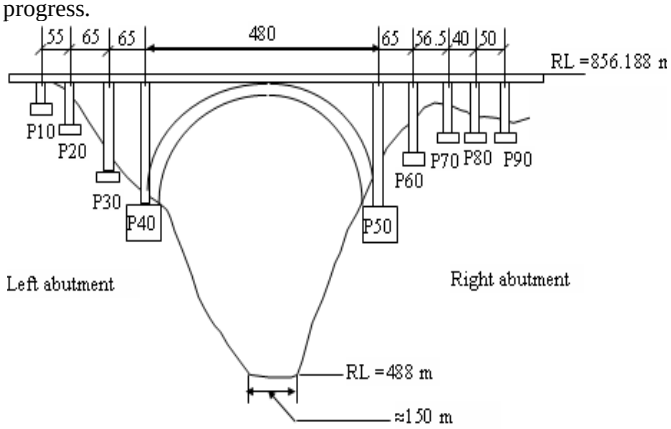

Fig. 1: Section of the Slope with the Pier Foundations

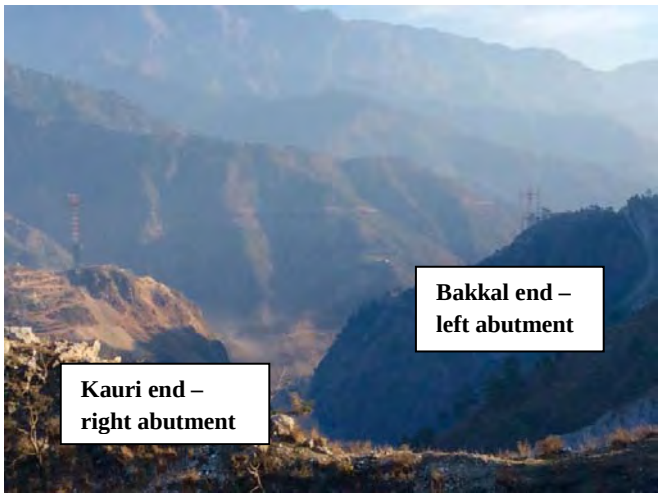

Fig. 2: Proposed Bridge Site near Bakkal and Kauri villages

Figure 3. Constructed piers at the site on the Kauri side from P90-P180

The rocks present at the bridge site are heavily jointed. The subsurface at the extent of the bridge site considered for slope stability analysis essentially consists of Dolomitic limestone with different degrees of weathering and fracturing. The main discontinuities at the site are one sub-horizontal foliation joint dipping about 20–30 degrees in North-East (NE) direction and two sub-vertical joints. Figure 4 shows the rock mass exposed at the bridge site. The figure also depicts the intensity and spacing of the prevailing joint sets at the bridge site. Figure 5 shows the joint orientations at two drift locations.

Fig. 4: Rock Mass Exposed at the Bridge Site

Fig 5. Joint orientations at drift locations

2. Rock properties

After detailed analyses of geotechnical properties and insitu test results in drifts the properties for the slopes have been selected as two sets of properties, out of which one is very conservative estimate. Analyses have also been carried out the entire slope material as a single layer and in the other case with top disintegrated soil as a two layer model. Table 1 and Table 2 show the sets of properties of rock mass chosen from the Design Basis Note for the continuum analyses of slope between S10-S40.

Table 1 . Set 1Values used by IIT,Delhi (in UDEC report of Jan. 2010)

Rock Mass	
Dry Unit Weight	27.095kN / m^3 (2762 kg/m^3)
Cohesion (c, MPa)	1.40
Friction angle (□) degrees	44.42
Hoek and Brown parameter (m and s)	4.70 and 0.00127
Bulk Modulus GPa	50.55
Shear Modulus GPa	37.92

Table 2. Set 2: (Suggested by SW vide Record of Proof Check No. D105999/GEO/08)

Rock Mass	
Dry Unit Weight	26.752 kN / m^3 (2727 kg/m^3)
Cohesion (c, MPa)	0.531
Friction angle (□) degrees	49.10
Hoek and Brown parameter (m and s)	0.793 and 0.0005
Bulk Modulus GPa	6.31
Shear Modulus GPa	4.73

Table 3 and 4 show the typical set 1 and set 2 properties adopted for slope stability for rock mass on S50-S80 slope.

Table 3. Set 1 values for hill slope for the bottom layer on S50-S80

Rock Mass	
Dry Unit Weight, kN / m^3	27.095
Cohesion (c, MPa)	1.41
Friction angle (□) degrees	42.61
Bulk Modulus GPa	50.55
Shear Modulus GPa	37.92
Hoek and Brown parameter (m and s)	4.706 and 0.00127
RQD / RMR	49 / 48

Table 4. Set 2 values for hill slope for the bottom layer for S50 and S60 (for S70 values are slightly different)

Rock Mass	
4Dry Unit Weight	27.095
Cohesion (c, MPa)	0.525
Friction angle (□) degrees	49.06
Bulk Modulus GPa	5.64
Shear Modulus GPa	4.23
Hoek and Brown parameter (m and s)	0.766 and 0.0005
RQD / RMR	10 / 36

3. **Slope Stability Analysis**

The detailed study carried out by Indian Institute of science considers all possible failure modes such as global failure, wedge failure, planar failure and toppling failure. The analyses have been carried out for abutment slopes and many cross sections at both these abutment slopes. Final profile was arrived considering all the necessary alternatives, construction methodology and accessibility to the foundation locations on the slopes. The numerical programs used for the detailed slope stability analyses are: Wedge failure analyses was carried out using SWEDGE (from RocScience); Global failure analyses using continuum approach using FLAC (from Itasca) and SLIDE (from Roc Science) programs; Planar failure analyses using DIPS (Roc Science) and Toppling analyses using Hoek and Bray Analysis. Analyses were carried out using two sets of rock mass properties which were decided based on large number of tests, borehole data and tests at drift locations. For the right abutment slopes variation in rock mass properties along the slope was also considered in the continuum analyses. Lower values of moduli were considered for quartzite in the upper part of the slope. Sensitivity analyses were also done for the possible variations in shear strength parameters. There was no ground water reported during geotechnical investigations. Piezometers installed on the slope showed no head of water indicating that the water quickly drains off through the joints. However, pore pressure ratio (r_u) value of 0.3 was considered for the wedge failure analysis as decided in the design basis note. Allowable bearing pressures on foundations determined as 1 MPa (determined by several methods and after detailed deliberations) was applied at all foundations in the continuum analyses. Initially seismic analyses were done using PHA values of 0.31g and 0.155g for MCE and DBE cases respectively. However, these values were increased later to 0.36g and 0.18g respectively considering the importance of the project. The earthquake response spectrum was derived for dynamic analyses using site specific studies carried out by IIT Roorkee by multiplying a factor of 0.36 to obtain a peak horizontal acceleration of 0.36g keeping the frequency contents and duration of the seismic event unchanged. The dynamic input is applied at the base of the slope. Design factors of safety adopted for the slopes are as follows: FS=1.5 with static loads; FS=1.2 with static loads and DBE seismic loads; and FS=1.0 with static loads and MCE seismic loads.

Using SLIDE software continuum analyses both with circular and non-circular slip surfaces were carried out using two sets of properties as decided by the entire team and as per Design basis note. Static analyses, Pseudo static analyses with MCE loading and Pseudo static analysis with DBE loading conditions were carried out with two sets of properties for all the slopes. Sensitivity analyses were also carried out with subset properties. Further, analyses with two layer model considering pore water pressures were also done. Figure 6, 7 and 8 shows the typical results from SLIDE analyses for some of the cases. Figures also show the factor of safety obtained for a circular slip analyses using Bishop's simplified approach for an equivalent continuum material.

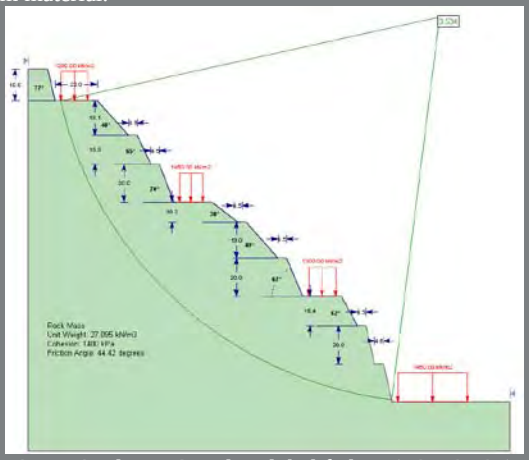

Figure 6. Static Analysis: Circular global failure (FS = 3.534) using set 1 values

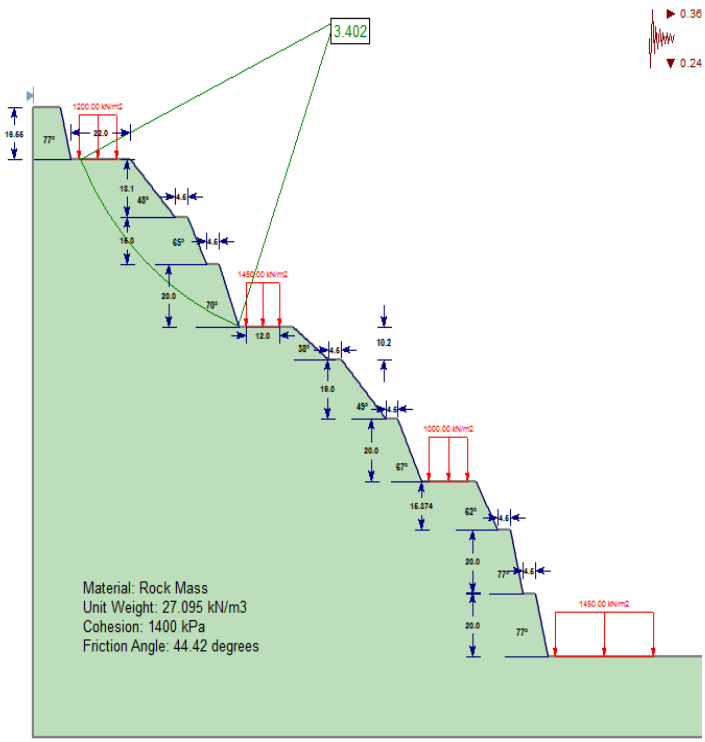

Figure 7. Pseudo-Static Analysis: Shallow slip failure (top), MCE condition (α_h = 0.36, α_v = 0.24) with set 1 values

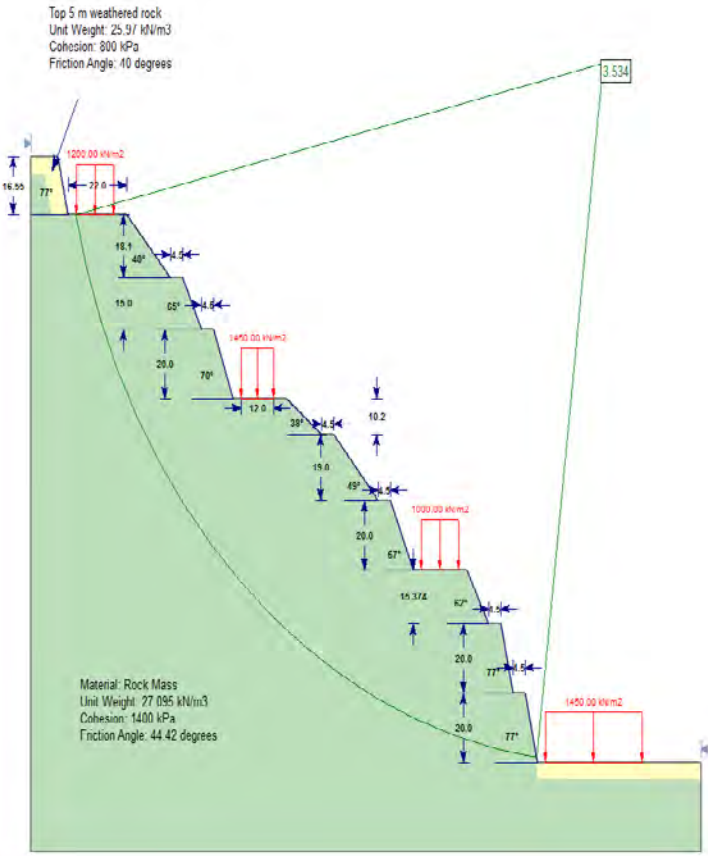

Figure 8. Two layer model: Static analysis, circular global failure Ru coefficient = 0.3, FS = 3.534

Further, Continuum analyses was carried out using FLAC. In this case also, static analyses, pseudo-static analyses with MCE and DBE conditions were carried out. Typical results from FLAC analyses have been presented in figures 9, 10 and 11.

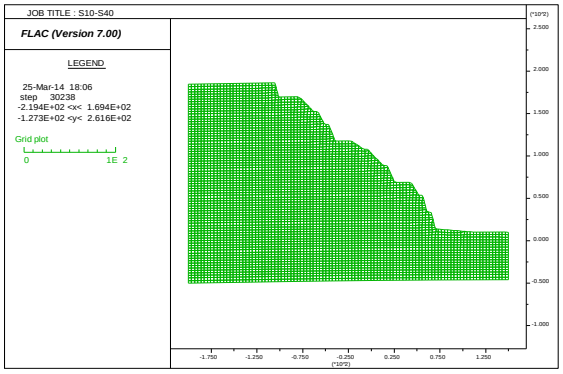

Figure 9: FLAC grid for the slope stability analysis of left abutment

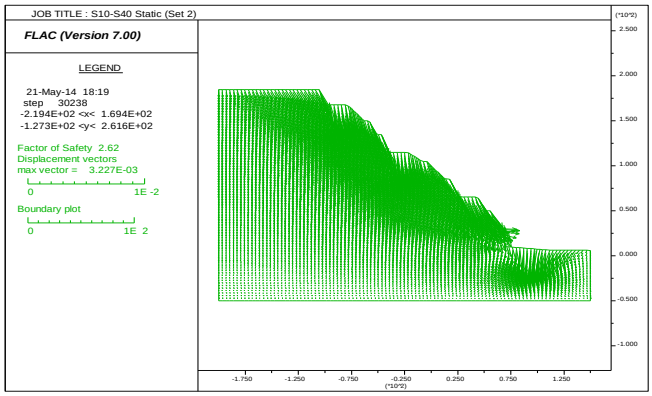

Fig 11: Displacement vectors (FOS :1.99, maximum displacement: 14.48 mm) - **DBE CASE (Seismic Force Coefficients:** a_h=0.18; a_v = 0.12)

From the displacement contours, the possible displacements at each founding level are tabulated and presented below in table 5.

Table 5. Displacements at each founding level from FLAC analyses for foundations on Bakkal side

S.No	Load Case	Displacement Type	Displacement in mm (Magnitude)			
			S10	S20	S30	S40
1	Static	Lateral	0.50	0.50	1.00	-0.75
2	Static	Vertical	-3.00	-3.00	-2.50	-3.50
3	Design Basis Earthquake (DBE)	Lateral	2.50	7.50	10.00	2.50
4	Design Basis Earthquake (DBE)	Vertical	-9.00	-8.00	-8.00	-4.00
5	Maximum Considered Earthquake (MCE)	Lateral	15.00	20.00	25.00	5.00
6	Maximum Considered Earthquake (MCE)	Vertical	-17.50	-15.00	-15.00	-5.00

In addition, a dynamic analyses with MCE and DBE conditions were also carried out. Typical acceleration time history plot for MCE of the site is given in figure 12

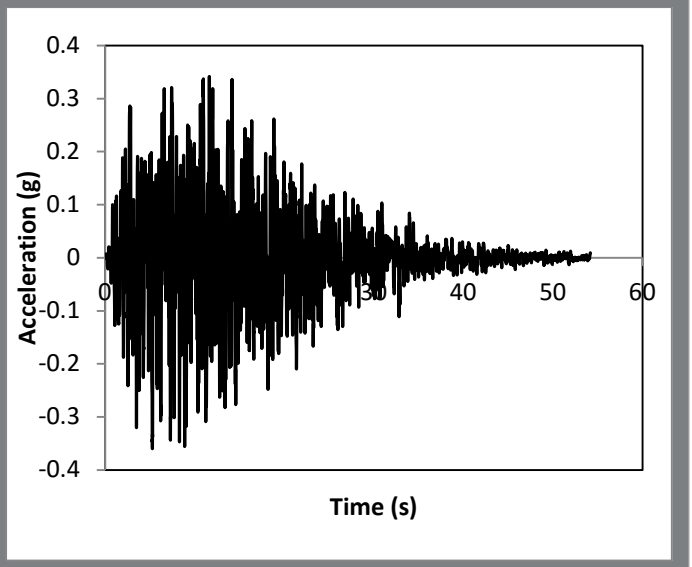

Figure 12: Acceleration-Time history of the MCE for the site

Global stability analyses with rock bolts and anchors were done to check the abutment slope up to river bed in one case and in other case 100m below the arch foundation locations (S40 and S50).

Further for local stability considerations, a complete 3-D wedge failure analysis of central and side slopes using SWEDGE were done without seismic force, and, with seismic force corresponding to MCE, with seismic force corresponding to DBE. Wedge failure analysis at different locations for different earthquake scenarios and with different friction parameters of the gouge material were analysed. Further, sensitivity analyses with three different friction angles were carried out along with flattened slopes to avoid any wedge failures and also analyses of slopes with rock anchors. Based on detailed analyses, appropriate strengthening measures were suggested using rock bolts which was executed at site. Typical rock bolting scheme adopted is shown in Figure 13. Typical rock bolting adopted for the entire slope is shown in Figure 14 at the centreline alignment. Figure 15 shows the view of slope stabilization work carried out at S10-S40 (Bakkal end) in July 2014 and Jan 2015.

Fig 13. Typical rock bolting scheme

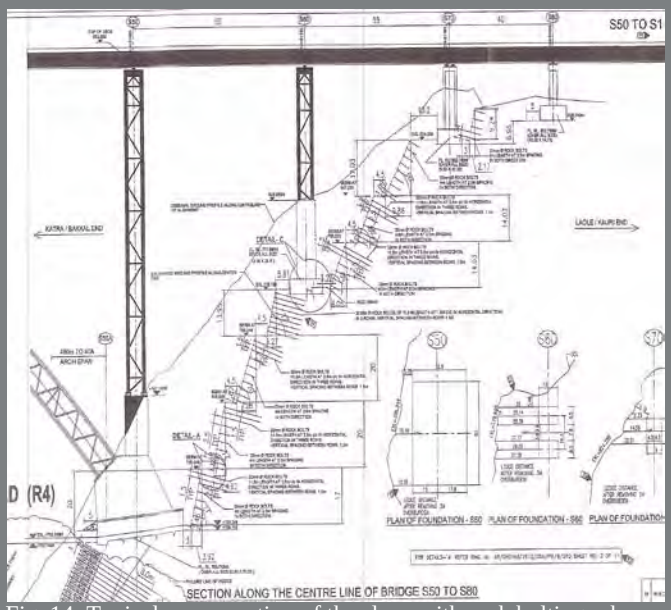

Fig. 14: Typical cross section of the slope with rock bolting scheme from s50 to S80

FIG. 15. View of slope stabilization works at s10-40 (Bakkal end) in July 2014 and Jan 2015

4. CONCLUSIONS

Results from the analyses for left abutment slope (S10-S40) showed that Factors of safety for all sets of properties with the sensitivity analyses on shear strength parameters are more than the required factors of safety for all the conditions considered. Further, no wedges are formed in central slope A. However, side slope B was recommended with slope flattening of 63 degrees to avoid any wedge failures. For side slope C, rock anchors of required capacity were recommended for the slopes.

Factors of safety obtained from the continuum analyses using FLAC are more than the required factors of safety for both static and pseudo static cases.

Results from the wedge failure analyses for the right abutment slopes (S50-S80) also shows clearly that the factor of safety for all sets of properties are more than the required factors of safety for all the conditions considered. Wedge failures are observed in the central and side slopes at locations except above and below S70 in the central slopes. Both options of flattening the slopes and provision of rock anchors were considered independently for the analyses to avoid any wedge failures. Based on the wedge failure analysis, it is recommended to provide rock bolts of required capacity at different locations for central slope D and side slopes E and F.

Under dynamic loading conditions all the displacements are less than 20mm. Thus differential displacements between each foundation will be far less than 20mm which satisfies the structural design of the arch. 32 mm dia rock bolts at 2.5m c/c spacing in both directions are proposed at all locations except below S50 and S60 to satisfy nachor force requirements from wedge failure analyses. 40mm dia pre-stressed dywidag bar anchors of 64 t capcity and 25m length at 2.5m spacing in both directions are proposed below S50 and S60 to satisfy the large anchor force requirement at these locations.

In general from detailed analyses, confirmed that the slopes are stable both in the static and pseudo static conditions. Dynamic stability analysis for the maximum credible earthquake for the area showed that the displacements are well within the permissible limits. Probability of wedge failure is assessed through kinematic analysis of the slope by drawing stereographic projections of joint planes prevailing in the bridge site and the slope. The kinematic analysis revealed the possibility of wedge failure at certain locations which necessitated the flattening of the slope and also strengthening using rock bolts. To account uncertainties in determining rock mass properties and moduli values, complete design has been provided to avoid any wedge failures and the same has been adopted at the site and some of the excavated slopes both at Bakkal and Kauri end have been shown in the figure 16.

Figure 16. Cut slopes at the Bakkal End and a view of rock bolting being carried out on Kauri End slopes

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Slope Stability Analyses for Chenab Bridge along USBRL Project

1.0 INTRODUCTION

The Udhampur-Srinagar-Baramulla Rail Link (USBRL) is the railway line that will link Kashmir Valley to the rest of India. The design and construction of the section of the railway line between Katra and Banihal (illustrated in Fig. 1) are extremely challenging engineering tasks. This section, which is 110 km long, will have 88 km (80%) through tunnels and 11 km (10%) over the bridges and viaducts.

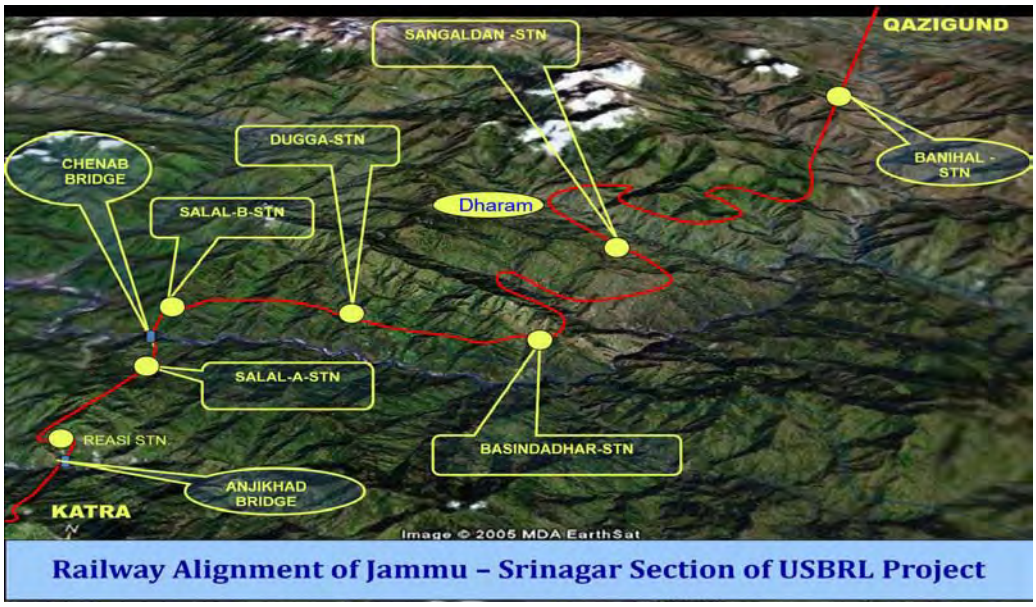

Figure 1. Railway alignment between Katra and Banihal.

The bridge over the Chenab River (location indicated in Fig. 1) will be a world-class structure. The Chenab River Bridge will be a concrete-filled truss steel arch bridge. After construction, at 359 m above the river valley, it will be the highest bridge deck and the seventh-longest spanning arch bridge (main span of 467 m) in the world.

Figure 2. View of the Chenab River Bridge site.

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Varun
Sr Engineer
ITASCA

The bridge spans the Chenab River in the valley with very steep slopes (Fig. 2) in generally good quality, but with highly fractured rock mass (Fig. 3). Overall stability of the slopes (on both left and right banks), and particularly deformation and stability of the arch abutments and pillar foundation, will have an important effect on performance of the bridge after construction. The bridge design allows for a maximum relative arch abutment displacement of up to 2 cm. Because the railway section between Katra and Banihal is located in the foothills of the Himalayas, seismic loading is expected to be an important factor affecting deformation and stability of the slopes and the bridge arch abutments and pillar foundations.

Figure 3. Beddingon the right bank of the Chenab River.

2.0 STABILITY ANALYSES

Konkan Railways Corporation Limited (KRCL) hired Itasca to analyze deformation and stability of the critical slopes and foundations for Chenab River Bridge and Anji Bridge using numerical modeling for the range of design static and dynamic loadings. Itasca is a global, engineering consulting and software firm headquartered in Minneapolis, USA with a subsidiary office in Nagpur, India. Using expertise in geomechanics, hydrogeology, and geophysics,Itasca engineers solve problems in many industries including mining, civil engineering, oil and gas, manufacturing, and power generation. Itasca also develops state-of-the-art commercial geomechanical software that is widely used throughout the world. Itasca combines its background of practical engineering and field experience with unparalleled knowledge of computer modeling and data analysis techniques to provide clients withsolutions to their most difficult practical problems.

Itasca is usingnumerical analyses to determine the maximum transient and permanent displacements of the arch abutments and the pillar foundations, as well as the safety factors of the slopes before and after construction of the bridgesunder static loads and for the design seismic loading. Based on the results of the analyses, Itasca will be able to recommend whetheradditional (compared to design) ground support measures are necessary.The proposed analyses will allow simulation of transient and residual (permanent) stress and deformation changes, as well as calculation of the distribution of the safety factor within the relevant volume of rock mass.

For this particular project, Itasca is using its three-dimensional distinct element modeling software *3DEC,* which is a numerical modeling code for advanced geotechnical analysis of soil, rock, and structural support in three dimensions.*3DEC*simulates the response of discontinuous media (such as jointed rock) subject to either static or dynamic loading. After visiting the bridge site, it became clear to Itasca personnel that the discontinuities will have a critical effect on both deformation and stability of the slopes and abutments. An important and unique capability of *3DEC* is the explicit representation of a large number of discontinuities. The discontinuous medium is modeled as an assemblage of blocks that may be rigid or deformable.Itasca is representing jointing in the rock mass realistically in terms of fracture orientation (including variability in orientation within the sets), distribution, and density. The information used to develop a synthetic model of fracturing of the rock mass is obtained from exploratory drifts mapping data, borehole logging, and joint mapping on excavated benches. Smaller discontinuities are represented stochastically based on statistical characterization of jointing; larger discontinuities are represented deterministically with position, orientation, shape, and size as mapped in the field. *3DEC* also allows very accurate representation of the pre- and post-construction surface topology.

3.0 WORK FLOW

3.1 Review of Existing Site Characterization Data

The existing data from the extensive site characterization program,including reports and raw data as mapped or measured,were reviewed and reinterpreted by Itasca. The data were analyzed to determine the geometrical characterization of jointing, including orientation, spacing and size, joint properties, as well as rock mass properties.Fig. 4 shows a plot of distribution of joint spacing for joint set J1 at Bakkal end. Fig. 5 shows a stereonet plot of the orientation of joints at Bakkal end.

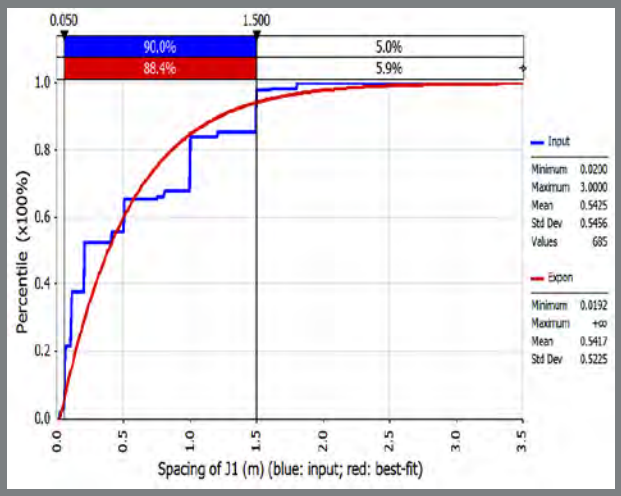
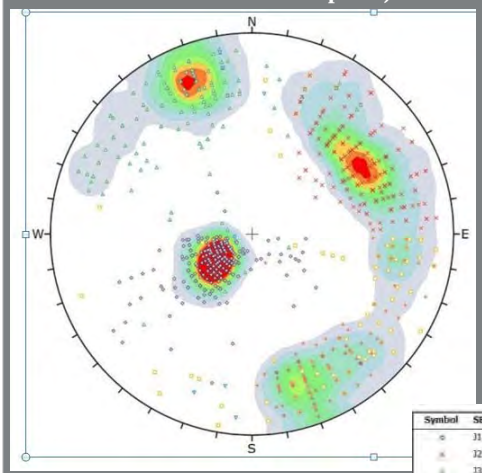


Figure 4. Cumulative distribution curve of spacing of joint set 1 (bedding plane) at Bakkal end.



Symbol	SET_ID	Quantity
	J1	355
	J2	294
	J3	217
	J4	122
	JR	7
	R	113

Color	Density Concentrations
	0.00 - 0.70
	0.70 - 1.40
	1.40 - 2.10
	2.10 - 2.80
	2.80 - 3.50
	3.50 - 4.20
	4.20 - 4.90
	4.90 - 5.60
	5.60 - 6.30
	6.30 <

Maximum Density	12.13%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Plot Mode	Pole Vectors
Vector Count	1108 (1108 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 5 Stereonet plot for data obtained from surface mapping at Bakkal end.

Development of Representative Discrete Fracture Network

Because bedding-parallel joint set J1 has large persistence and small fracture spacing, it is not practical to explicitly represent every discontinuity within this set in the model of stability of the entire slope or in the analysis of an arch abutment. Instead, the effect of bedding was taken into account using an anisotropic constitutive model called the ubiquitous joint model. Thus, the models explicitly represent the dominant discontinuities (most joints from sets J2 and J3 and some from set J1), while the effect of the majority of discontinuities from set J1, not explicitly represented in the model, are taken into account in the anisotropic constitutive model. This approach allows striking an optimum balance between the correct representation of the mechanics of deformation and failure of thinly bedded rock and creating manageable models that can be executed within a reasonable timeframe.

In order to characterize fracture sizes and persistence to be used as input in the *3DEC* models for stability analysis, the fracture trace lengths obtained based on surface mapping were used to calibrate the persistence factor of the *3DEC*fracture generation algorithm as shown in Fig. 6.

Figure 6. (a) Sketch of surface mapping obtained at Bakkal end;

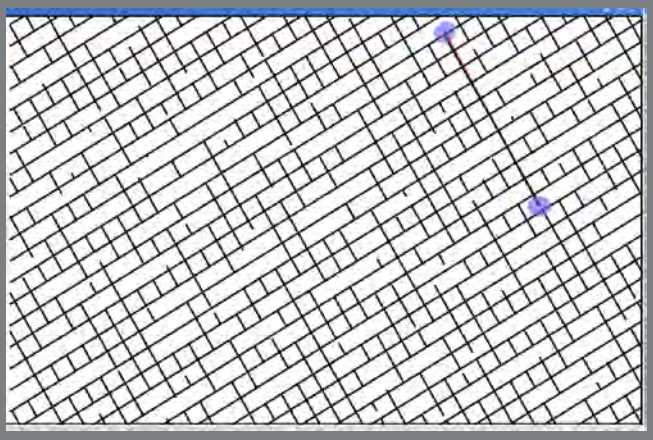


Figure 8. (b) Trace lengths of fractures generated in 3DEC on a surface with the same orientation as the excavation where the fractures were mapped.

3.3 Development of 3D Model and Stability Analysis of the Foundation Excavations at Kauri and Bakkal Endsof Chenab River Bridge

A 3D model of the abutment and extent of the rock mass expected to be affected by the excavations and arch reactions was developed using the current topography and planned excavations for both the Kauri and Bakkal ends of Chenab Bridge.

The fractures were represented using the stochastic DFN model. A fracture spacing equal to 20 times the actual spacing was used to represent explicit fractures. An anisotropic material model was used for blocks between these models to account for the small-scale fractures. The input strength parameters for the explicitly represented discontinuities in the model were derived from the direct-shear tests on the discontinuities. The mechanical properties of the anisotropic constitutive model (ubiquitous joint model) that was used for the representation of bedded rock mass were derived from the properties of the intact rock and properties of the discontinuities (obtained from the direct-shear test). The models are shown in Fig. 7 and 8 along with the planned excavation. The explicit joint sets are shown in Fig. 9.

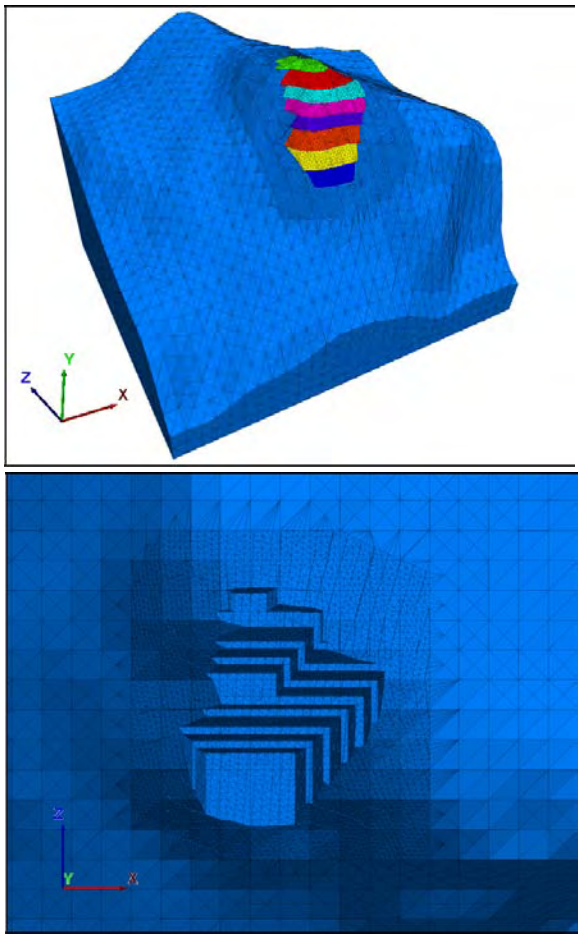
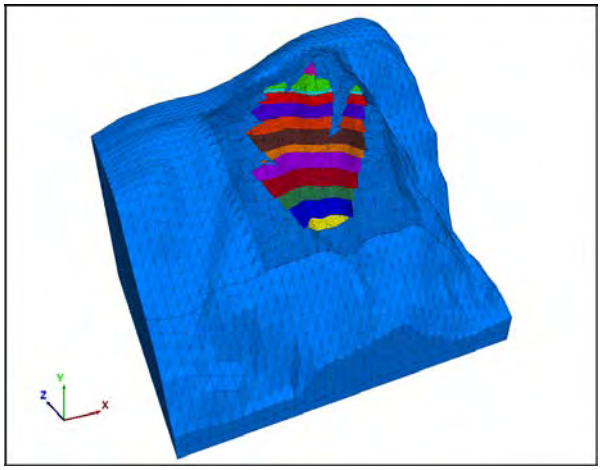


Figure 7. Numerical model of Kauri end (left) and the excavation plan (right).



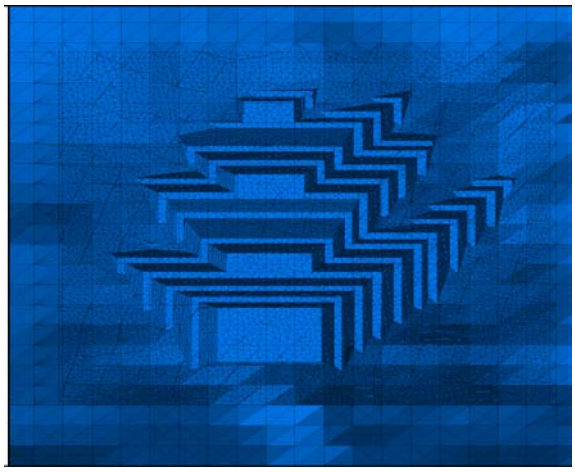


Figure 10. Numerical model of Bakkal end (left) and the excavation plan

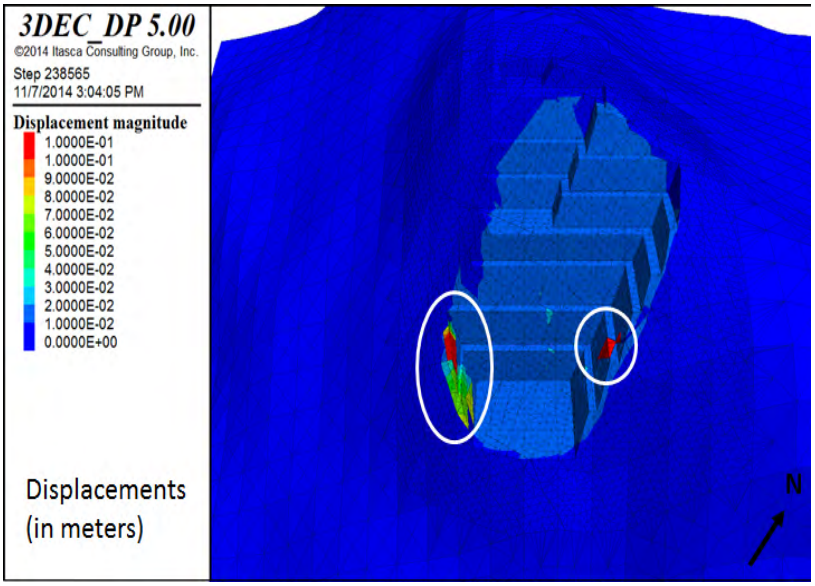


Figure 9. Excavation-induced displacements at Kauri end.

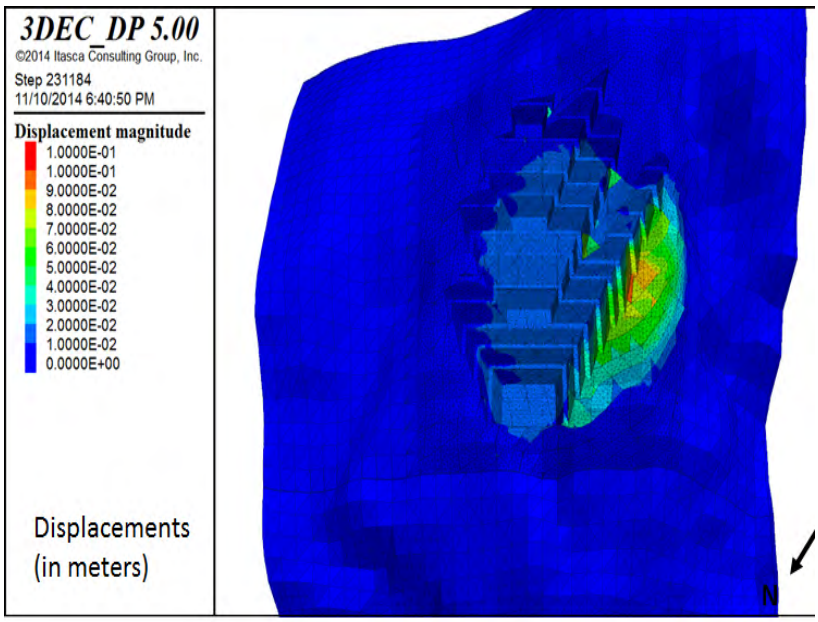


Figure 11. Excavation-induced displacements at Bakkal end.

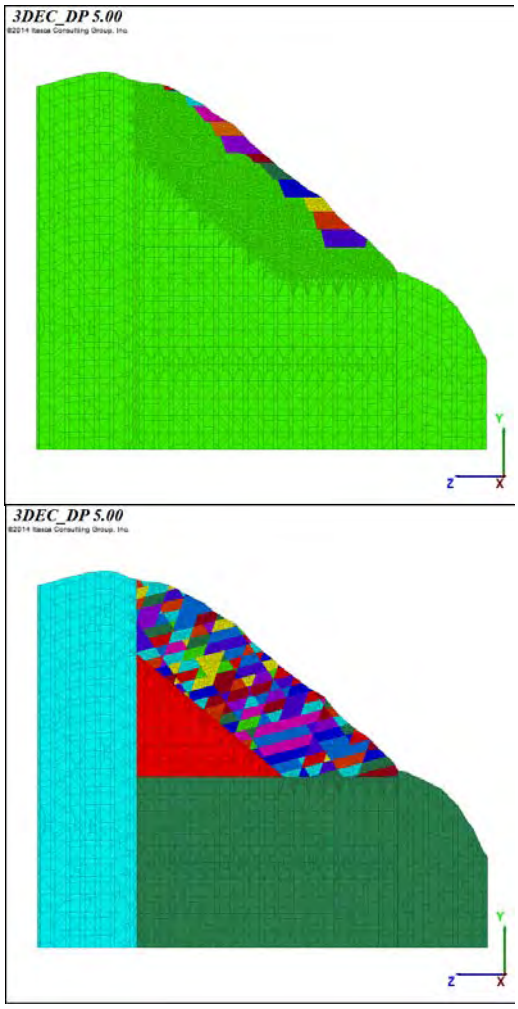


Figure 12. Excavation steps (left) and explicit joints (right) in Bakkal end model.

Multiple realizations of fracture orientations were used to address the uncertainty in the input data. The sequential bench excavationwas simulated and the model was solved to equilibrium after each excavation increment. The model doesnot consider any water pressure in joints or pore pressure in rock based on the absence of water during site investigation.

The results from numerical modeling indicatedthe movement of small wedges but no significant stability problems with the excavations. KRCL proceeded with the excavations while carefully mapping the discontinuities observed on the excavation faces. Some of this data has already been relayed back to Itasca and is being used to update the fracture networks. The updated models will also include the effect of bridge loading and will provide insight into the static stability of both ends after the bridge has been built.

Itasca is also working on a numerical model including both Kauri and Bakkal ends as well as a simplified representation of the bridge for dynamic analysis. The complete three-dimensional model will account for the effect of local site geology and surface topography on the ground motion propagation.

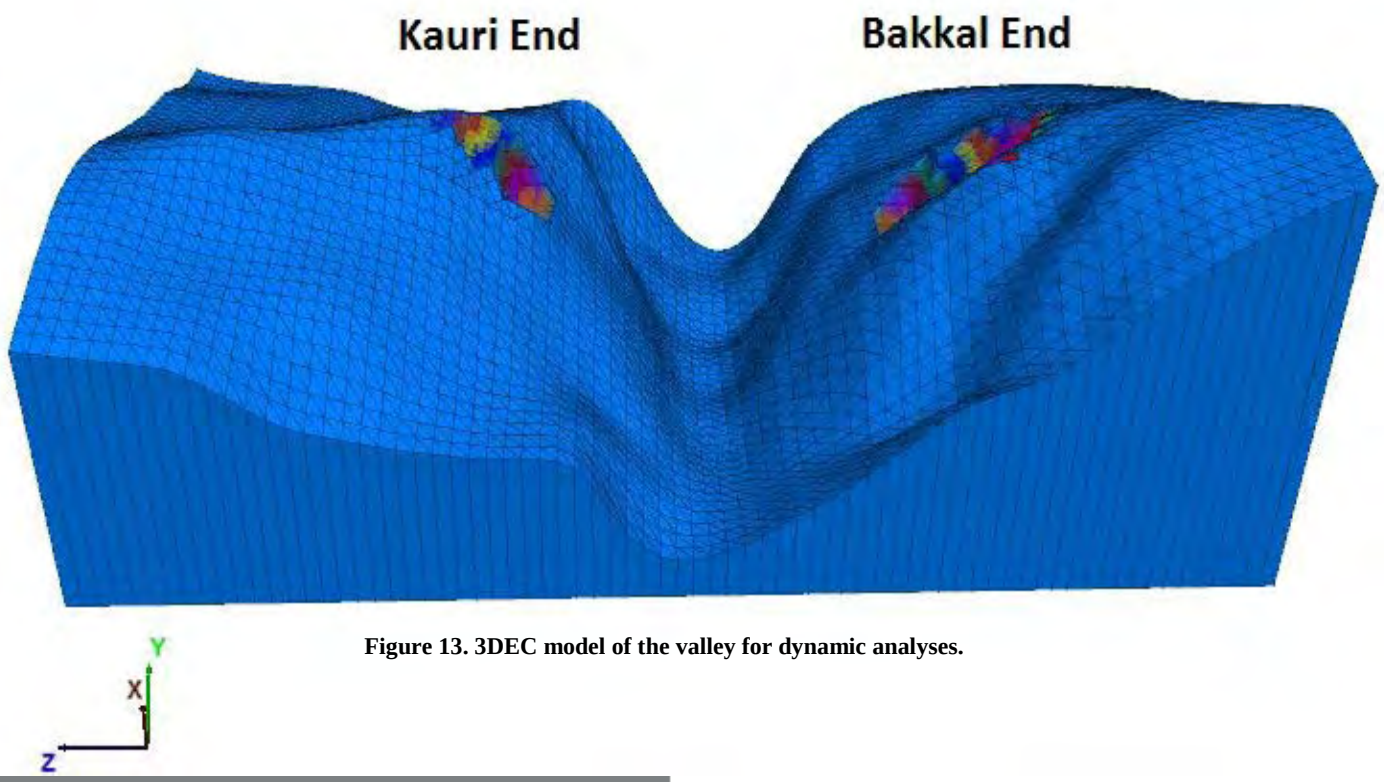


Figure 13. 3DEC model of the valley for dynamic analyses.

4.0 CONCLUSIONS

The numerical modeling of the Chenab Bridge is underway using the state-of-the-art technology available. The analysis is expected to be completed by April 2015.

SITE SPECIFIC SEISMIC HAZARD ANALYSES OF CHENAB BRIDGE LOCATION, KATRA-DHARAM SECTION OF USBRL PROJECT, J&K

INTRODUCTION

Indian Railways envisaged an ambitious project connecting Jammu to Baramulla through a railway link in the tectonically active and geologically complex Himalayan Mountains. The world's highest 359 m railway arch bridge is under construction in Reasi district of Jammu and Kashmir as a part of this mega project. This region being in zone IV/V (IS 1893-2002) has experienced many earthquakes in the past and recent times. And also faces the danger of seismic threats from the central Himalayan seismic gap. The Konkan Railway Corporation Limited (KRCL) has requested Indian Institute of Technology Delhi to carry out a comprehensive study on Site Specific Seismic Hazard Analysis of Chenab bridge location. A detailed study using Deterministic and Probabilistic seismic hazard analyses by considering site specific geological, seismotectonic and recorded earthquake events in and around the site were carried out. The goal of earthquake-resistant design is to produce a structure or facility that can withstand a certain level of shaking without excessive damage. That level of shaking is described by a design ground motion, which can be characterized by design ground motion parameters (PGA). Seismic hazard analysis involves the quantitative estimation of ground-shaking hazards at the site.

Prior to this, "Site specific design earthquake parameters for Chenab Bridge were assessed by IIT Roorkee in 2004. They have identified 12 seismic sources and concluded that the Jwalamukhi fault is the most important source for Chenab bridge hazard study as this is the thrust that can bring the most disastrous earthquake in the region. Using the Abrahamson and Litehiser (1989) attenuation relationship they suggested a MCE value of 0.31g for the region.

SCOPE OF THE STUDY

The scope of the study by IIT Delhi is to conduct seismic hazard analyses both by deterministic and probabilistic approaches in order to arrive at suitable dynamic parameters to be adopted in the design of structures in the locality. More specifically the scope includes:

i) Collection and preparation of a catalogue of seismic events in the region
ii) Checking for its completeness and suitable recurrence parameters
iii) Preparation of a Seismotectonic map considering geological and tectonic features for the region
v) Conducting Seismic Hazard analyses both by Deterministic and Probabilistic approaches.

Prof K. S. Rao, Dept. of Civil Engineering, IIT Delhi

vi)Recommendation of dynamic parameters for the design of structures in the region.

STUDY REGION

Seismic data have been collected using databases of Indian Meteorological Department, International Seismological Center, United States Geological Survey, PESMOS, Harvard CCMT and COSMOS. The area bounded within 350km radius of the study was selected, i.e. between 30.0-36.0^o N and 72.0-78.0^o E. This area was selected so that all the seismogenic sources are identified. In the study region, data of 15,946 earthquakes having magnitude range of 1.4-8.2 from 1903-2014 were collected. The earthquake data was further de-clustered for removing aftershock/foreshock and duplicate using Gardner and Knopoff (1974), Urhammer (1986) and Grunthalet al. (2004) de-clustering algorithms. This results to 5,547 earthquake events for consideration.

MAGNITUDE CONVERSION

Instrumental earthquake data reported contain body wave magnitude, surface wave magnitude, moment magnitude and local wave magnitude. In order to homogenize the different types of magnitudes, the data are converted to moment magnitude using the relation defined by Scordilis (2006).

DETERMINISTIC SEISMIC HAZARD ANALYSIS

This is done when a particular earthquake scenario is assumed. DSHA is carried out for a particular earthquake, either assumed or realistic. The tectonic features are reasonably active and well defined. The focus is on determining Maximum Credible Earthquake (MCE) motion at the site. The MCE is the largest possible earthquake along a recognized fault.

Within the 350km radius, 82 seismic sources were identified. Wells and Coppersmith (1994) regression relations are used for determining the potential magnitude for the source. Most influential sources in the study region are shown in Fig. 1. Figure 2 shows the seismogenic activities within the 350km radius.

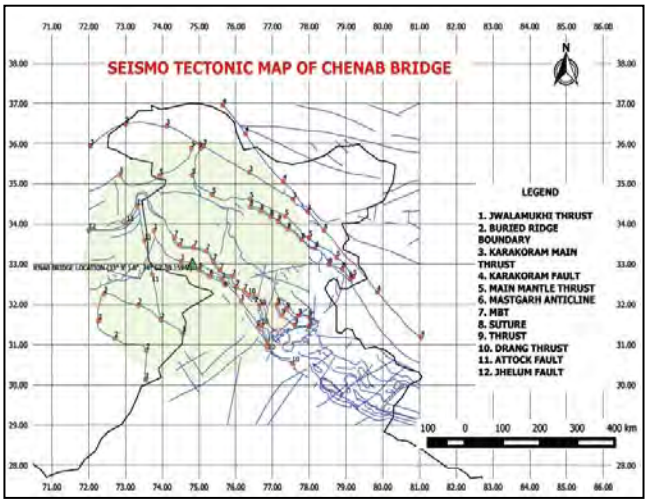


Figure 1 Major Faults/Thrusts within 350 Km Radius of Chenab Bridge Location

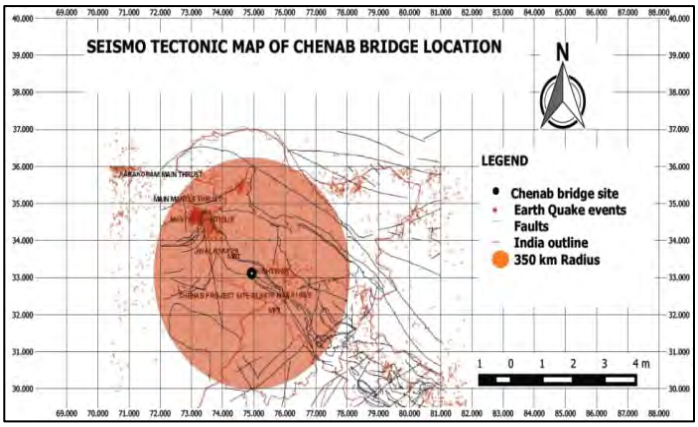


Figure 2 Seismotectonic Map of Chenab Region

Deterministic analysis requires definition of seismic sources and their distance from the site. Each fault is treated as a separate source and is analyzed to assign a maximum magnitude. Maximum considered earthquake is found to be 0.33g for the study. Different attenuation equations are available for estimation of PGA due to maximum earthquake. Each method has merits and demerits. Care should be taken while selecting the attenuation relation. The relations used in this study are those of Jain et al. (2000), Sharma et al.(2009), NDMA(2012), NGA(2012), Rao and Rathod (2013) and Raghukanthand Kavitha (2014).

PROBABILISTIC SEISMIC HAZARD ANALYSIS

Probabilistic analysis provides a framework in which uncertainties in the size, location and rate of recurrence of earthquakes and in the variation of ground motion can be identified, quantified and combined in a rational manner. Segregation of the potential sources is done by selecting all the sources producing ground motion of at least 0.30g. The selected sources are shown in Fig. 1.

COMPLETENESS AND RECURRENCE RELATIONSHIPS

Earthquakes in the ancient times have not been recorded unless they were large and destructive. It illustrates the incompleteness of a catalogue and that an appropriate process must account for incompleteness in the statistical procedure of seismic hazard analysis. Catalogue incompleteness can be defined as recorded seismicity that differs from the real seismicity (Mulargia et al. 1985). There are different techniques used to account for catalogue incompleteness. To determine the completeness periods for different magnitude classes, two different methods namely Visual Cumulative (CUVI) method (Tinti and Mulargia, 1985) and the method by Stepp (1973) were used (also known as yearly based and decade based methods respectively).

GUTENBERG -RICHTER RECURRENCE LAW

Gutenberg and Richter (1944) showed that the magnitude-recurrence relationship may be represented by a linear relationship when the log of annual rate of exceedance was plotted against magnitude. This type of recurrence model has been used because of its simplicity and also it fits the data reasonably well over a useful range of magnitude of engineering interest.

The Gutenberg-Richter law for earthquake recurrence was expressed as:

$$\log\lambda_m = a - bm \quad (1)$$

Where,

λ_m = mean annual rate of exceedance,

m = magnitude of event,

a = mean yearly number of earthquakes of magnitude ≥ 0,and

b = relative likelihood of large and small earthquakes

The recurrence parameters determined using STEPP and CUVI methods for Chenab area are:

$$a = 6.626$$
$$b = 1.204$$

i.e. $$\log\lambda_m = 6.626 - 1.204m \quad (2)$$

PROBABLE MEAN RATE OF EXCEEDANCE(λ_Y)

Probable mean rate of exceedance is calculated for each individual source using a computer code program developed in PYTHON. Using this, mean rate of exceedance is calculated by finding the total number of earthquake exceeding the threshold magnitude, its annual frequency for each source, its magnitude uncertainty and distance uncertainty, Probability of PGA exceeding a particular PGA, Mean rate of exceedance, and finally an equation to find probability of PGA or return period with remaining known two parameters

$$P = 1 - exp^{-\lambda_y T} \quad (3)$$

Where, P is the probability; $T$is the return period; λ_y is the mean rate of exceedance (PGA can be calculated from the aggregate hazard generated from the mean rate of exceedance value). The aggregate hazard is drawn out from the individual hazard values of respective faults for a range of PGA values. These curves are first obtained individually for all the 20 thrusts/faults/lineaments and then summed up to estimate the aggregate hazard at the site. Typical seismic hazard curves at Chenab bridge location for PGA at bed rock level obtained by the above method (Fig.3). The peak ground acceleration at bed rock level for 10%, 5%and 2% probability of exceedance in 50 and 100 years has been estimated for the study area and presented in Table.1.As indicated in the above table there is a 2% probability of getting PGA of 0.40g in 100 years, i.e. return period of 5000 years. 5% probability PGA of 0.36g in 100 years, i.e. return period of 2000 years, 10% probability of PGA of 0.32g in 100 years, i.e. return period of 1000 years.

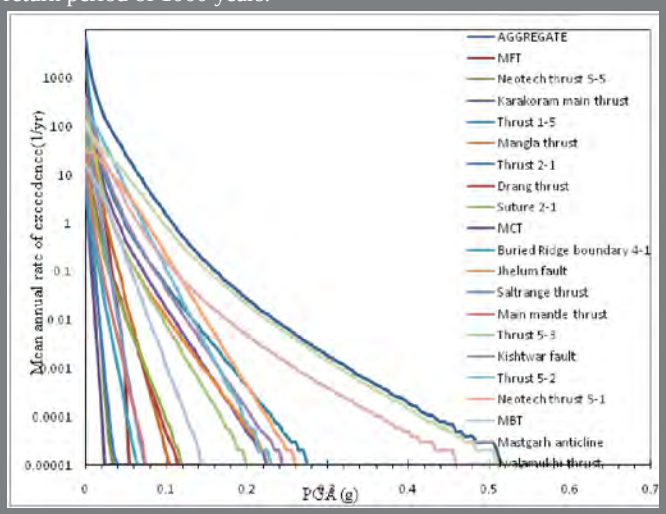


Figure 3 Aggregate Hazard Curve

Table 1 PGA Corresponding to Probability and Return Period

PGA Corresponding to	Return Period	PGA , g
2% in 50 years	2500	0.37
2% in 100 years	5000	0.40
5% in 50 years	1000	0.32
5% in 100 years	2000	0.36
10% in 50 years	500	0.30
10% in 100 years	1000	0.32

RESPONSE SPECTRUM ANALYSIS (RSA)

For design, we usually need only the maximum response. Hence, a plot of maximum response versus natural period (for a given value of damping) is constructed. Horizontal acceleration time histories for five earthquakes which fall under the 350 km radius of Chenab bridge location have been considered for the study. They are Dharmasala, Mandi, Chamba, Himachal-Punjab boarder, Jammu and Kashmir-Himachal boarder earthquakes respectively. Figure 4 shows the response spectra for the rock sites for 5% damping. The figures include also the spectra obtained from the IS1893 (Part-I) 2002 and IITR (2004) procedures respectively for comparison.

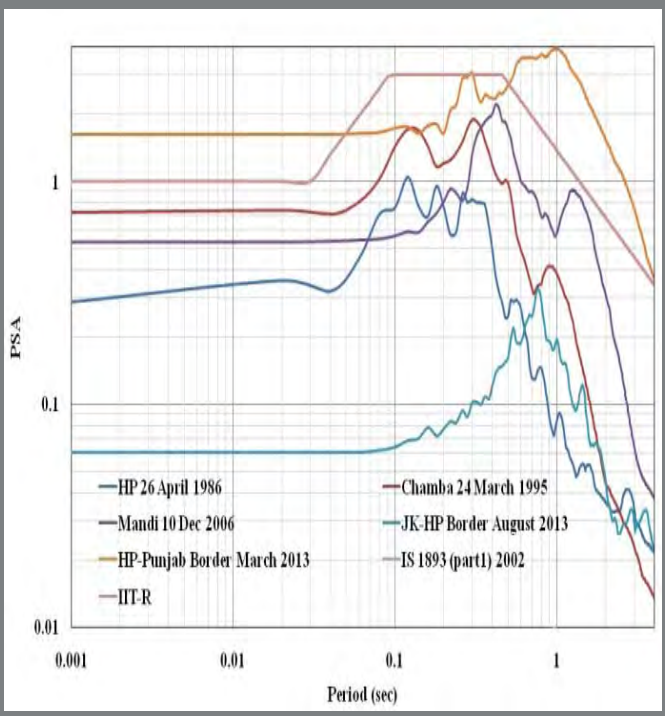


Figure 4 Response Spectra for 5% Damping

CONCLUSIONS

A comprehensive study on both deterministic (DSHA) and probabilistic hazard analyses (PSHA) was carried for the Chenab bridge location. The main conclusions are summarized below:

i) The Chenab bridge location is situated in the tectonically active Himalayan zone falling under zone IV of IS1893 (2002). Eighty two seismotectonic features such as faults/thrusts were identified which may generate considerable future earthquakes within 350 km radius of the Chenab bridge site.

ii) Out of these tectonic features, 20 of them were considered as major and their source lengths varying from 4.82 to 947.97 km. The Jwalamukhi fault which is very close to the site is significant.

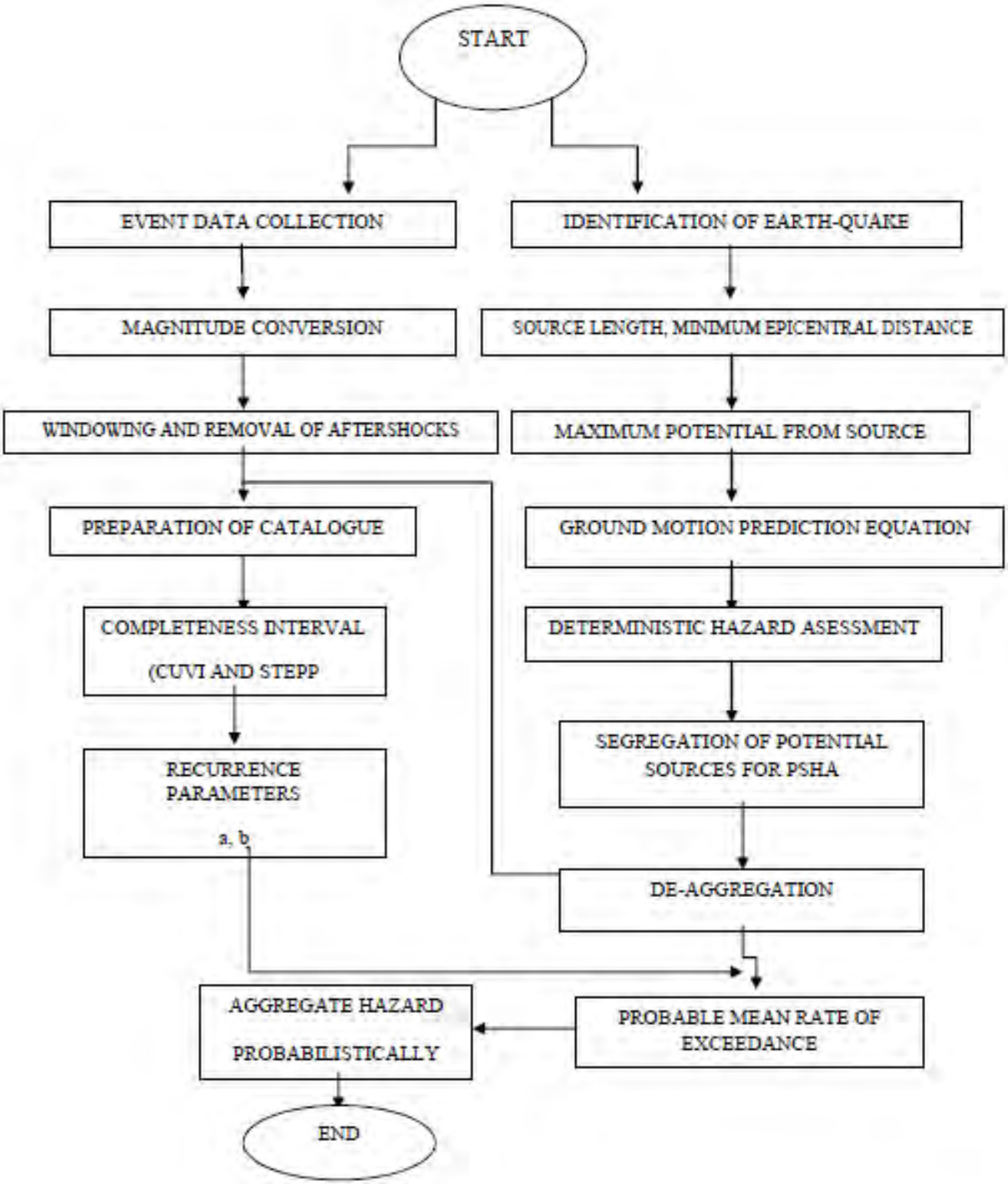
iii) Wells and Coppersmith's (1994) model was used to predict magnitudes (M_w) from the fracture lengths for all faults.

iv) PGA values were estimated for all faults using attenuation relations proposed by Jain et al.(2000), Sharma et al.(2009), NDMA(2012), NGA(2012), Rao and Rathod (2013)and Raghukanthand Kavitha (2014) models. Jwalamukhi fault is critical source, able to produce the Maximum Credible Earthquake (MCE) of 8M_w and PGA of 0.33g in the area. The DBE would be 0.165g.

v) For PSHA, the faults which are able to generate PGA values of 0.3g and above are considered. Extensive seismic data was collected from IMD, USGS, ISC, Harvard CCMT, PESMOS and COSMOS sources in the study area and the data was filtered. Total events were 15,946 and uniform magnitude, M_w conversion was done using Scordilis (2006).Windowing and removal of fore and aftershocks was carried out using Gardner and Knopoff (1974), Uhrhammer (1986) and Grunthal et al.(2004) models. The final catalogue consists of 5,547 events. Completeness was carried out by CUVI and Stepp methods and Gutenberg-Richter recurrence laws were applied. Probable mean rate of exceedance is calculated for each individual source using the Python code. Finally the aggregate hazard curve for the study area was proposed (Fig.4).

vi) In the present study, Sharma (2009) has been used for Probabilistic study since the equation gives a better result as compared to the other relationships. The peak ground acceleration at bed rock level for 10%, 5%and 2% probability of exceedance in 50 and 100 years has been estimated for the study area (Table.5). Using this, there is a 2% probability of getting PGA of 0.40g in 100 years, i.e. return period of 5000 years, 5% probability PGA of 0.36g in 100 years, i.e. return period of 2000 years, 10% probability of PGA of 0.32g in 100 years, i.e. return period of 1000 years. Earthquake sources of near and far field are also studied. Jwalamukhi thrust is the most prominent source for near field and for far field MBT and MCT can give a PGA of approximately 0.1g in the site of interest. Besides this maximum PGA for 20 seismic sources of far and near field have been determined.

vi) Horizontal acceleration time histories for five earthquakes e.g. Dharmasala, Chamoli, Chamba, Himachal Punjab boarder and J&K Himachal Punjab border earthquakes have been used to generate response spectra for 1%, 2%, 5%, 7% and 10% damping ratios for the study. These will be useful in assessing the dynamic forces for structural design.



Flow Chart for Seismic Hazard Analysis

VALIDATION SLOPE OF CHENAB BRIDGE ABUTMENTS

Introduction:-

The proposed railway line between Katra and Qazigund of Northern Railways crosses the river Chenab between the stations of Salal road 'A' and Salal road 'B' near left and right abutments respectively. A railway bridge is proposed to construct between Chainage 50.4 and ground level 846.008 m on left abutment near Bakkal village and Chainage 51.715 and ground level 848.457 m on right abutments near Kauri village. The 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, would be the Highest Bridge in the World at a height of 359m from the river bed level. The 1,315 m long bridge consists of 467m span steel arch over the river, in tandem with an 848m long viaduct upto Salal Road 'B' station. The slope is subjected to 7 loaded supports: four supports as S10, S20, S30 and S40 as on Left abutment (Bakkal End) and three supports as S50, S60 and S70 as on Right abutment (Kauri End). To enable these supports Hill is subjected to huge cutting for the mighty bridge.

The slopes defined with nomenclature as 'A', 'B' & 'C' on left abutment(Bakkal End) and 'D', 'E' & 'F' on right abutment (Kauri End). Slope 'A' & 'D' lies along the bridge alignment. Slope defined with 'B' & 'E' rest on U/S of bridge alignment and slope rest on D/S of alignment defined as 'C'& 'F' slope.

Geological and Structural Features

The railway alignment passes through the Siwaliks and Pre-Tertiary rocks overlain by unconsolidated sediments of recent to sub-recent periods. The primary lithological units are dolomitic limestone with different degree of fracturing and occasssional weathering. Cherty, bouldery, 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. Due to limited persistence and wavy and uneven roughness profiles, the rockmass is in highly interlocked and stiff state.

The mode of failure essentially of Planer, wedge, toppling, rockfall is possible in this type of rock mass. Primarily the failure controlled by the orientation and the spacing of discontinuities planes with respect to the slope face. Slope is designed with all possibilities of joint orientation as observed from surface mapping. Mapping is carried on surface as defined in Table-1.

Attitude of Discontinuitie	Left Abutment		Right Abutment	
	Strike	Dip	Strike	Dip
Rail Line Alignment			N120°-N300°	
Foliation Joint	N20°W-S20°E	10° to 40° NE	N30°W-S40°E	30° to 50° NE
Joint-1	E-W to N25°W-S52°E	Vert. or 55° to SW	N60°E-S60°W to N45°W-S45°E	80° to SE 30° to SW
Joint-2	N85°E-S85°W (rare)	85° to SE	N30°E-S30°W (rare)	70° to NW
Joint-3	N30°E-S30°W (rare)	20°-40° to SE	N30°E-S30°W (rare)	85° - either side

Table1- Joint Mapping of Left & Right Abutment

Before getting into excavation a numbers of geotechnical test carried by different agencies and slope is analysed through different slope stability analysing tools (UDEC, 3DEC, SLIDE) through different agencies. Factor safety for respective slope stands tall with all respect of slopes. However during the excavation the joint orientation changes as it proceed for further excavation. The excavated slope needs to get validation as it proceeds for further excavation. This is required as the joint changes its orientation. The persistence is also plays a major role while defining the characteristics. So a methodology is carried by KRCL and CBPU/ IISc Bangalore.The proposed slope is analysed with available geotechnical parameter as defined in table-2.

Table:-2

Sr no	Parameter	Unit	Geotechnical Property	
			Bakkal End-Left Abutment	Kauri End-Right Abutment
1.	Density (kg/m3)	(kg/m3)	2700	2700
2.	Bulk modulus	Gpa	2.5	2.7
3.	Shear modulus	Gpa	2.5	2.0
4.	Cohesion	Mpa	0.3	0.3
5.	Join Friction angle	°	37	37
6.	Residual Friction Angle	°	29	29

Seismic activity in the region around the Chenab bridge site is mainly associated with Main Boundary Thurst (MBT) and Main Central Thrust (MCT). Historical recordings reveal that the area is seismically very active and the site lies in seismic zone V as per the seismic zoning map of India (IS: 1893, Part 1: 2002). Site specific design earthquake parameters for the Chenab bridge site were estimated by IIT Roorkee based on regional geology, local geology around the site, earthquake occurrences in the region around the site, and the seismotectonic setup of the area. Accordingly, the maximum ground acceleration considered in dynamic analysis are: MCE: 0.31 g DBE: 0.16 g. During the construction the Factor of Safety may be considered with DBE/2.

Validation Procedure:

In this the case study lies with fourth berm of Bakkal End, and is validated for failure and designed accordingly. The berm 4 located between elevations RL 814.40 to RL799.40. The aligned slope is designated with nomenclature aligned A4, U/S (B41 &B42) and downstream (C41 to C45). The detailed slope is analysed through DIPS and Swedge Software of Rocscience.

The process validation of slope comes under following head

1. Mapping,
2. Kinematic Analysis,
3. Support system.

Prem Sagar Gupta
CE/Design/KRCL

Rashmi Ranjan Mallick
Dy.CE/Design/KRCL

VALIDATION SLOPE OF CHENAB BRIDGE ABUTMENTS

1. **Mapping**:

The precariously slope is excavated with controlled blasting. To avoid the rock into river catch drains are constructed at 2-3m above all foundation level. The height of the berm on either end is varies in between 15-20m. The joint J2 and J3 are not continuous as that of foliation joint (J1). Persistence varies up to 4.52±4.42m and 8.35±6.84m for J2 and J3 respectively which is lower side of berm height. The each berm excavated with 5m independent excavation depth. The neat face is mapped and provision of support system. In all case berm is excavated 3-4 sequential berm height. On excavation of each 5m (aprox) face, it is mapped. The detailed joint orientation is computed with pole concentration greater than 6%. In each case the joint persistency is looked into with respect to slope height. The mapped joints for each slope-4 are tabulated under Table-3. During the mapping the RMR for each slope calculated independently.

2. **Kinematic Analysis:**

Kinematic analysis commenced with analysis with DIPS-Rocscience software to analyze the potential for the various modes of rock slope failures (plane, wedge, toppling failures), that occur due to the presence of unfavourably oriented discontinuities. Discontinuities are geologic breaks such as joints, faults, bedding planes, foliation, and shear zones that can potentially serve as failures planes. The quantitative kinematic analysis, instead of relying on representative values, considers all discontinuities and their possible intersections to calculate failure indices. Instead of considering just the number of discontinuities or possible intersections that cause failure, normalized weights are recommended as they would remove the unwanted effects of over represented discontinuity data. ***DIPS*** software compares every dip direction/dip value with slope angle and friction angle to evaluate its potential to cause plane or wedge or toppling failure. It also calculates all potential intersection line plunge direction and amount for wedge failure potential.

Table:-3

Sr No	Slope Desig.	Slope Face (Dip/Dip Direction)	Joint Orientation (Dip/Dip Direction) in °				Possibility Failure Planner/ Wedge/Toppling
			J1	J2	J3	J4(Rare)	
1	A41	65/304	31/57.4	68/242	77.6/155	82/327	Wedge in J3-J4
2	B41	63/214	16/73	65/245	66/180	78/340	No Failure
3	*B42*	*65/304*	*21/60*	*63/230*	*82/158*	*75/340*	*Wedge in J2-J4*
4	C41	70/35	23/77	72/260	74/165	80/353	No failure
5	C42	70/305	25/53	63/231	79/155	-	No failure
6	C43	70/35	24/67	63/241	81/160	77/330	No failure
7	C44	70/305	26/59	67/234	75/157	-	No failure
8	C45	70/35	31/67	67/232	82/158	-	Wedge in J1-J3

Out of the all the slope as executed slope A41, B41, C45 are subjected with wedge failure possibilities. There are three types of possibilities of slope failure. The kinematics slope B42 and its design is studied as case study.

With the DIPS software the projection of all joints as mapped from field is plotted. The slope and joints with higher persistence is plotted on sterionet. Pole concentration with more than 6% and persistence with 6% more are considered. For B42 there three major joint set and one random joint set J4 is observed. During the analysing it must be important to notice RMR, water condition. Though the in all case the condition defines with dry, all slopes are designed with 30% water condition. Each Joint friction along with cohesion may be considered before designing. Syncline nature of geology if any may be studied both on the same face also on adjacent face to a distance of 10m. All the rock with the possibility dominance in the slope may be studied.

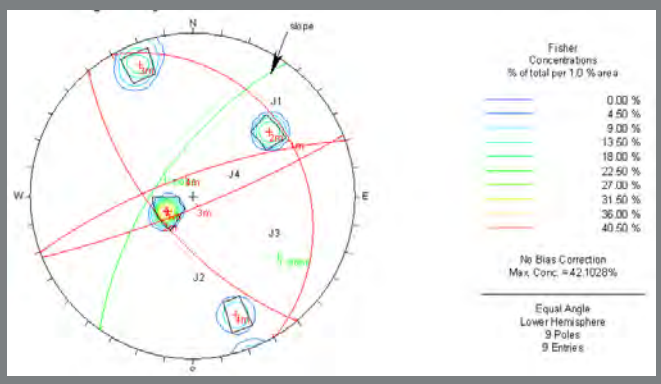


Fig-1.0: Sterionet of joint mapping at B42 of Bakkal end of Chenab Bridge.

i. **Planner Failure**: For Kinematic analysis and designing of the slope requires further joint friction angle.

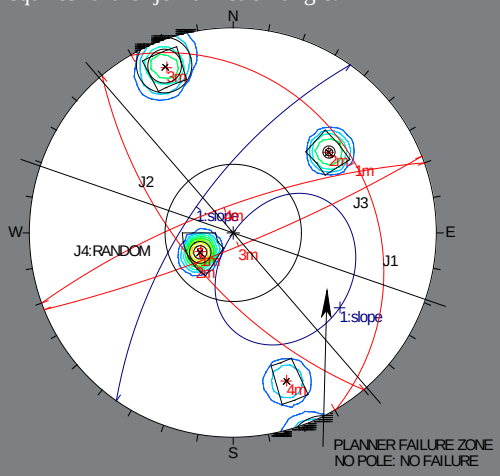


Fig2.0 Sterionet for Planner Failure

In all cases joint friction angle 37° is considered. The orientation of slope is at 65°/304°. Slope face daylight shall be looked into through DIPS. A plane failure is likely to occur when a discontinuity dips in the same direction (within 20°) as the slope face, at an angle gentler than the slope angle but greater than the friction angle along the failure plane (Hoek and Bray, 1981).

The crescent shaped zone formed by the Daylight Envelope and the pole friction circle therefore encloses the region of planar sliding. Any poles in this region represent planes which can and will slide. In the case B42 there is no sign of Planner failure. The slope is subjected limiting boundary for analysis.

ii. **Toppling Failure**: A toppling failure may result when a steeply dipping discontinuity is parallel to the slope face (within 20°) and dips into it. In Toppling failure different independent block undergoes slip or toppling. These conditions of interlayer slip is given by $(180-\psi_f-\psi_d) \geq (90-\phi_d)$ Or $\psi_d \geq (90-\psi_f) + \phi_d$ Nomenclature is well defined in Fig-3.a.

VALIDATION SLOPE OF CHENAB BRIDGE ABUTMENTS

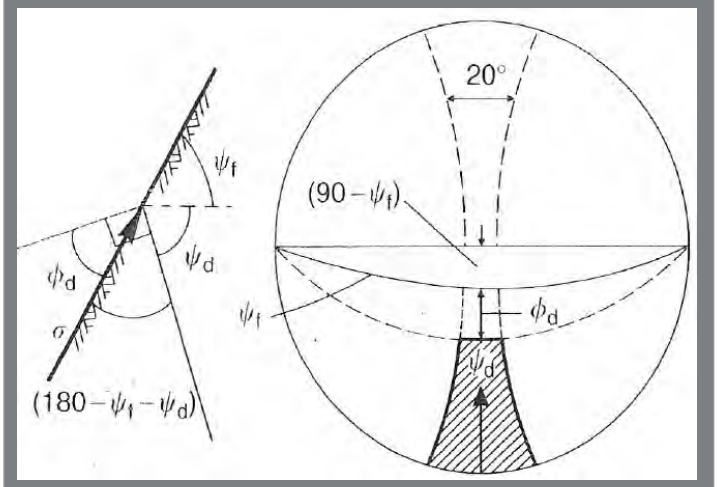


Fig-3.a Toppling Failure Theory

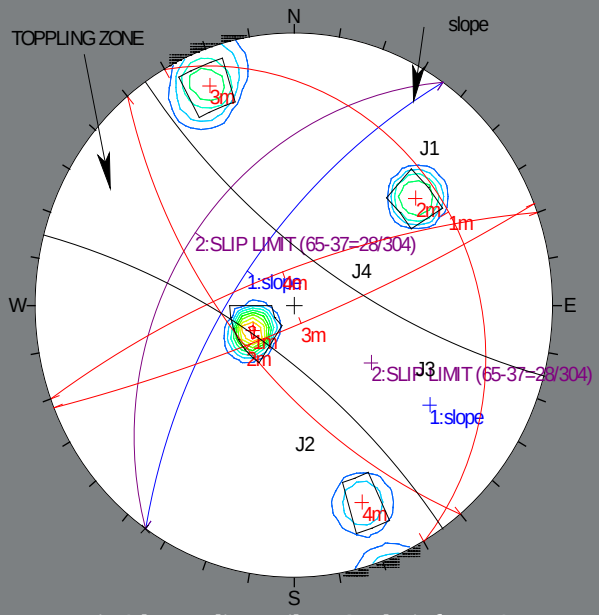


Fig-3.b Toppling Failure Analysis for B42

The poles that falls within the shaded zone causes possibility of failure in toppling. The shaded zone is defined by limiting of slope and within the 20° boundary. Slip limit is created with slope equals to slope-friction angle of respective joint (J3, for B42 slope). For B42 slope there is no possibility in toppling failure as no poles area falling within the zone.

iii. **Wedge Failure:** A wedge failure may occur when the line of intersection of two discontinuities, both of which dip out of the cut slope at an oblique angle to the cut face thus forming a wedge-shaped block.

The wedge-shaped block, plunges in the same direction as the slope face and the plunge angle is less than the slope angle but greater than the friction angle along the planes of failure (Hoek and Bray, 1981). These rock wedges are exposed by excavations that daylight the line of intersection forming the axis of sliding. The plunge and direction can be interpolated. The size of a wedge failure can range from a few cubic meters to very large slides. In slope B42 Joint intersection J2 and J4 falls within the zone (Fig-4). Hence the wedge require analysis through Swedge with J2 (63/230) & joint J4 (75/340) and with respective joint parameter (cohesion, friction)

Swedge Analysis

With the kinematics slope requires to checked the equilibrium of Wedge with all external force including earthquake. Wedge is subjected to external water force with $0.3h_u$ though dry condition exist and earthquake with DBE and MCE conditions. During DBE and MCE condition the minimum factor of safety to required is 1.2 and 1.0 respectively. The resultant Earthquake applied along the intersection and upword and downword direction to find optimum design of slopes. The wedge dimensioning can be restricted with berm height & width, persistence (J3 and J4). The wedge may fail in single or two planes or single plane (Fig-4).

When the wedge coordinates have been determined, the geometrical properties of a wedge can be calculated, including: Wedge volume, Wedge face areas, and Normal vectors for each wedge plane. All forces on the wedge can be classified as either Active or Passive.

In wedge of B42 slope is analysed and applied with rock bolting to make Factor of safety to desired level. Rock bolting capability depends on interaction grout to rock i.e.,its RMR value (=35, class iv for B42) of slope, pullout strength. Pullout test for Rock bolt is carried and respective value is considered for designing Rock bolt. While considering RMR value the strength of bolt is 4.26t/m. However with the rock pullout test the value 2.5t/m is considered for designing. Optimum angle rock bolting may be fixed. However rock bolt is provided and designed on direction perpendicular to slope face.In B42 slope 4m length 32 dia rock bolt is provided with spacing 2.5m on both direction.

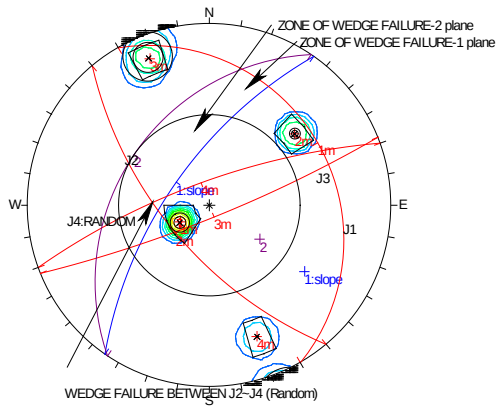


Fig.4 Wedge Failure Kinematics for B42

Conclusion and Suggestion:

Slope is designed with surface mapping. Every time we proceed to next bench on sequential cutting validation is prepared. The original orientation is checked with designed orientation. If any considerably change noticed in orientation with respect to designed orientation it requires redesign. Generally it's a rare phenomenon. In all slopes joint dips varies within 4° to 6°. Validation is done every case. The validation of each 5m depth is repeated at least for first two sequential heights before moving to next level.

With persistence the each slope shall be checked with slope above the current slope. While proceeding to further depth of overall orientation will be checked to validate the slope. If any shear seams comes under foundation, requires proper design of whole system. No loose pockets shall be allowed requires filling pockets if available. If a continuum segregate/ crushed rock appears must be checked with design and preferable long bolt is designed in consultation of geologist and designer. All surfaces shall be treated with shortcrete and weephole with preferable geotextile as per rock condition.

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

Introduction:-

The use of Shotcrete for the support of underground excavation was pioneered by the civil engineering industry. Reviews of the development of shotcrete technology have been presented by Rose (1985), Morgan (1993) and Frazěn (1992). Rabcewicz was largely responsible for the introduction of the use shotcrete for tunnel primary support in 1930's, and for the development of the New Austrian Tunneling Method for excavating in weak ground. In recent years the hydro power as well as motor & railway tunnel industry has become major user of shotcrete for underground support. The simultaneous working of multiple headings, difficulty of access and unusual loading conditions are some of the problems which are peculiar to tunneling and which require new and innovative applications of shotcrete technogy.

Rehabilitation of conventional rockbolt and mesh support can be very disruptive and expensive. Increasing number of these excavations is being shotcreted immediately after excavation. The incorporation of steel fiber reinforcement into the shotcrete is an important factor in this escalating use, since it minimizes the labour intensive process of mesh installation. Trials and observations suggest that shotcrete can provide effective support in mild rock burst conditions. While results from paststudies are still too limited to permit definite conclusions to be drawn, hence the quality of shotcrete is an important factor for primary supports in underground tunneling. The indications are encouraging enough that more serious attention will probably be paid to this application in the future. In order to assess the current state of the said tunnels, among other tests, in-place test methods were employed in order to estimate the quality of the concrete used for the tunnel primary lining. The said methods, which were aimed at the determination of concrete quality, involved visual inspection of the tunnels as well as non-destructive and destructive methods for testing the primary concrete lining.

Shotcrete Definition:

Shotcrete is a generic name for a mixture of cement, aggregate, water, with or without fibers, along with admixtures and accelerators in correct proportions which are applied pneumatically and compacted dynamically under high velocity. Maximum size of aggregate <10 mm are projected at high velocity from a spray nozzle on to a surface to form a layer of pneumatically applied concrete on that surface.

What is Shotcrete?

- Shotcrete is concrete shot from a hose and pneumatically projected at high velocity onto a surface, as a construction technique.
- Shotcrete undergoes placement and compaction at the same time due to the force with which it is projected from the nozzle. It can be impacted onto any type or shape of surface, including vertical or overhead areas.
- Shotcrete is applied in thin layers.
- Shotcrete is usually reinforced by wire-mesh or steel fibres.
- Shotcrete is being used widely around the world as underground support.

Shotcrete consist of Cement, Sand, Coarse Aggregates, Water as the basic shotcrete mix and the additives which are added to improve shotcrete properties. These are mixed in pre-determined proportions to produce a Dry-mix or a Wet-mix shotcrete.

Shotcrete- History

Shotcrete, then known as Gunite, was invented in early 1900s in America used to fill plaster model of animals. Development further for Engineering applications is as under:

- **1907**- First Device to spray dry materials invented by Carl Ethan to spray materials on to wire frames to make Animal models.
- **1910**- First Cement Gun was introduced at New York Concrete Show. The Design developed by Cement Gun Company (Now- Allentown Shotcrete Technology). Normally single/double chamber machines.
- **1920** Dry-mix process patented in Germany.
- **1930** Generic name shotcrete introduced by the American Railway Engineering Association to describe the dry-mix process.
- **1940** Initial use of coarse aggregate in dry-mix shotcrete.
- **1950** American Concrete Institute ACI Committee 506, Shotcreting, formed.
- **1952** Development of the rotary style dry-mix gun in Michigan and Illinois, USA.
- **1955** Introduction of the wet-mix shotcrete process.
- **1970** First practical use of steel fiber-reinforced shotcrete by U.S. Army Corps of Engineer
- **1975** First use of silica fume in shotcrete in Norway. **1980** First use of silica fume in shotcrete in North America in Vancouver, BC, Canada.
- **1985** First use of air entrainment in dry-mix shotcrete process in Quebec, Canada.
- **1988** First practical use of high-volume synthetic macrofibers in wet-mix shotcrete in Alberta, Canada.
- **1998** Formation of the American Shotcrete Association (ASA).
- **2000** ACI Shotcrete Nozzleman Certification Program established.

Shotcrete- History in India

In India Shotcreting has been in use since 1970s. Initially small capacity dry shotcrete machines were used. Later in Chamera-I hydroelectric project Dry Shotcreting with Aliva-280 and Teledyne Robo-arm was used. Later Nathpa Jhakri was another major Hydro Project using similar technologies. Further Wet shotcreting and Steel Fibre Reinforced Shotcrete was done at Uri HE Project in 1990s. Wet Shotcrete has now taken over from Dry shotcrete is being regularly used especially in underground works i.e. tunnels, caverns etc.



SUNIL BHASKER
Dy.CE/S&CI/UHP
USBRL Project

DEEPAK SINGH
XEN/C-I/Sangaldan
USBRL Project

RAMPAL
XEN/C-II/Sangaldan
USBRL Project

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

Factors Affecting Shotcrete Performance

Shotcrete is also a viable means and method for placing structural concrete. Properly applied shotcrete is a structurally sound and durable construction material which exhibits excellent bonding characteristics to existing concrete, rock, steel, and many other materials. It can have high strength, low absorption, good resistance to weathering, and resistance to some forms of chemical attack. Many of the physical properties of sound shotcrete are comparable or superior to those of conventional concrete or mortar having the same composition. Improperly applied shotcrete may create conditions much worse than the untreated condition. Thus the factors affecting shotcrete performance are as under:

- Shotcrete application system Dry-mix or Wet-mix
- Shotcrete equipment
- Shotcrete mix design
- Additives
- Shotcreting technique and nozzle man's skill
- Rebound and dust

Uses of Shotcrete

Shotcrete is used in lieu of conventional concrete, in most instances, for reasons of cost or convenience. Shotcrete is advantageous in situations when formwork is cost prohibitive or impractical and where forms can be reduced or eliminated, access to the work area is difficult, thin layers or variable thicknesses are required, or normal casting techniques cannot be employed. Additional savings are possible because shotcrete requires only a small, portable plant for manufacture and placement. Shotcreting operations can often be accomplished in areas of limited access to make repairs to structures. Shotcrete is suitable for a variety of new construction and repair work. Shotcrete is a very versatile method of placing structural concrete favouring its use for various applications as under:

- Tunnelling (Road, Rail, Hydraulic others)
- Mining projects
- Hydropower Installations
- Slope stabilization
- Canal lining / Shafts
- Reconstruction and Rehabilitation of Structures
- Fire and corrosion protection

Why Shotcrete Support be Used?

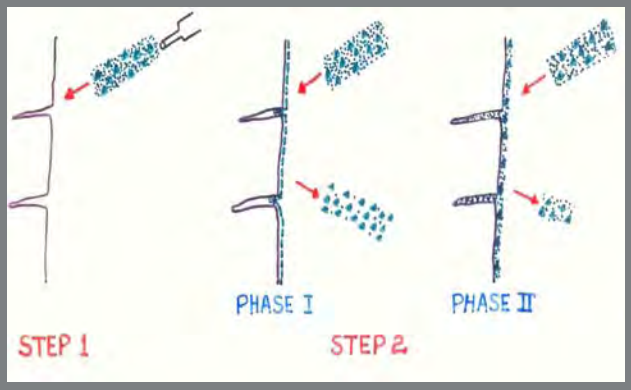
Shotcrete support system has advantages of being much faster and economical than conventional support. It provides immediate support after excavation, which controls rock mass deformation, prevents loosening of roof rock, maintains existing stability by preventing new fractures, mobilises inherent rock mass strength and requires low extra excavation.

How Does Shotcrete Stabilise Rock Mass?

Shotcrete stabilises the excavated or exposed rock mass as it maintains the existing stability. Fines are squeezed into cracks and joints which produce a wedge effect like mortar between bricks in a wall. Transfer of load in a weak zone to adjacent stable rock mass is through shear or adhesion. Shotcrete layer acts as a membrane in bending or tension thus preventing exposure of rock to changes of moisture, effect of air & temperature, washing effect of running water.

How does Shotcrete Work?

Shotcrete is sprayed on rock surface with high velocity (20 to 100 m/s). The application of shotcrete acts primarily in 2 phases. In Phase-I a thin layer of cement and fine sand particles <0.2 mm is formed which penetrates pores, cracks, joints in rock mass. This provides a foundation for further build up. In this phase nearly 100% of coarse aggregates rebound back from the surface of application. In Phase II as impact of spray continues, fines are squeezed into pores, cracks and joints. Now shotcrete starts sticking on thin layer of membrane and layer over layer builds up the required support system. Rebound at this juncture reduces substantially.



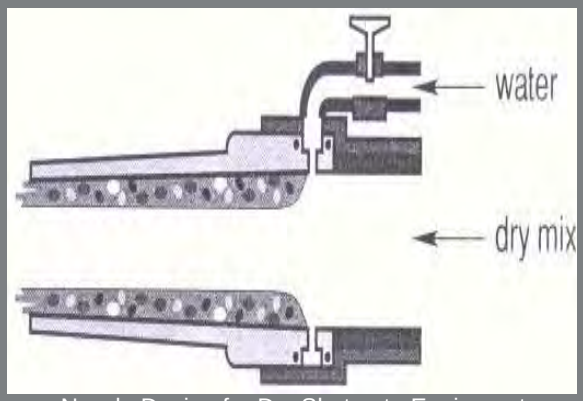
Working Principle of Shotcrete

Shotcrete Application Systems

There are two shotcrete application systems developed (1) Dry Mix Process (2) Wet Mix Process. Dry-mix process was developed first and later on wet-mix process was developed.

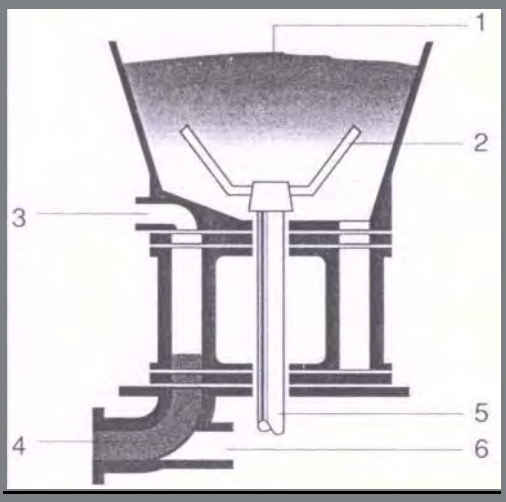
Dry-Mix Process

Cement, sand and aggregates are thoroughly mixed and are either bagged in a dry condition or conveyed directly to nozzle of hose by compressed air. Water is introduced under pressure at the nozzle. The mixture is normally fed to a pneumatically operated gun which delivers a continuous flow of material through the delivery hose to the nozzle. The interior of the nozzle is fitted with a water ring which uniformly injects water into the mixture as it is being discharged from the nozzle and propelled against the receiving surface. Actual mixing with water takes place at the wall this requires circular movements of nozzle. Dry-mix method produces high rebound and dust.



Nozzle Design for Dry Shotcrete Equipment

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

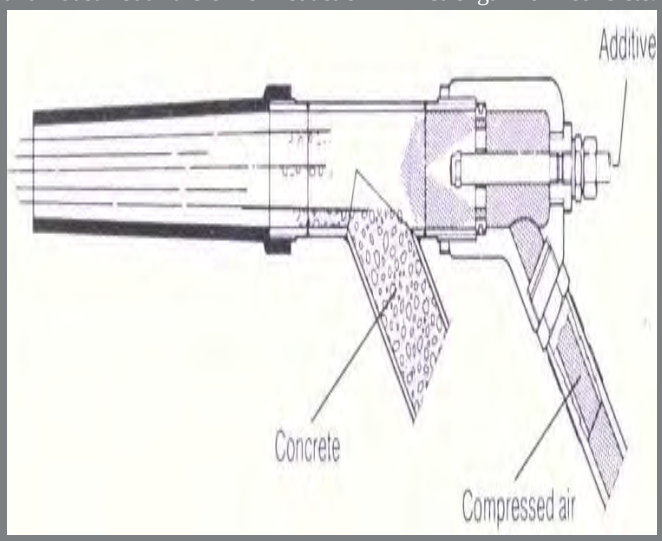


Dry Shotcrete System

1.	Material Feeding Hopper
2.	Agitator
3.	Compressed air
4.	Material exit
5.	Drive Shaft
6.	Compressed air

Wet-Mix Process

All ingredients, including water and admixtures are mixed as would be done for conventional concrete. The mixed material is fed to the delivery equipment, such as a concrete pump, which propels the mixture through the delivery hose by positive displacement or by compressed air. Additional air is added at the nozzle to increase the nozzle discharge velocity. This process causes much less rebound and dust but there is reduction in strength of concrete.



Nozzle Design for Wet Shotcrete Equipment

Comparison of Dry-Mix vis-a-vis Wet-Mix Process

The final product of either the dry or wet shotcrete process is very similar. The dry-mix system tends to be more widely used in mining, because of inaccessibility for large transit mix trucks and because it generally uses smaller and more compact equipment. This can be moved around relatively easily in an underground mine environment. The wet-mix system is ideal for high production applications in mining and civil engineering, where a deep shaft or long tunnel is being driven and where access allows the application equipment and delivery trucks to operate on a more or less continuous basis. Decisions to use the dry or wet mix shotcrete process are usually made on a site-by-site basis. A comparison based on advantages and disadvantages of both process is tabulated below:

Dry Mix	Wet Mix
EQUIPMENTS	
Lower investment	Three times costlier
Simple maintenance	Higher maintenance
More wear	Lesser wear
Higher air consumption	Lower air consumption
MIXING	
Possible at site	At mixing plant
Economical for smaller amounts	Uneconomical for smaller amounts
Performance impaired by wet sand	Wet sand acceptable
OUTPUT	
Lower output	Higher output
Can be conveyed to longer distance than wet mix	Suitable for shorter distances
DUST	
High dust	Very little dust
Poor visibility	Better visibility
REBOUND	
High rebound	Low rebound
Rebound pockets	No rebound pockets
Loss of aggregates changes mix design	Little loss of aggregates
QUALITY	
Higher strength	Lower strength
Less homogenous	More homogenous
Manually possible	Robot arm essential

Bagged Pre-mix dry shortcrete components being feed to hopper of shotcrete machine.

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

A truck mounted shotcrete robot

Mix Design

For shotcrete same mix design principle as for concrete mix design is used. Prime factors controlling strength and quality of shotcrete are water/cement ratio, air content and degree of consolidation. There are a number of considerations in mix design in which shotcrete differs from conventional concrete structure, such as (i) Aggregate gradation (ii) Cementitious content.

Theoretical design, in which sand and coarse aggregates are mixed in proportions which provide minimum void volume to be filled with water and cement, is generally satisfactory for dry mix machines For pumping wet mixes, additional amounts of finest size sand and cement are required to lubricate flow and ensure that water cannot migrate through mix. But, if proportion of fine sizes is excessive, blockage will occur, especially in long pipes. Thus considerable experience and test work are required for a successful shotcrete mix

Base Mix of shotcrete usually consists of

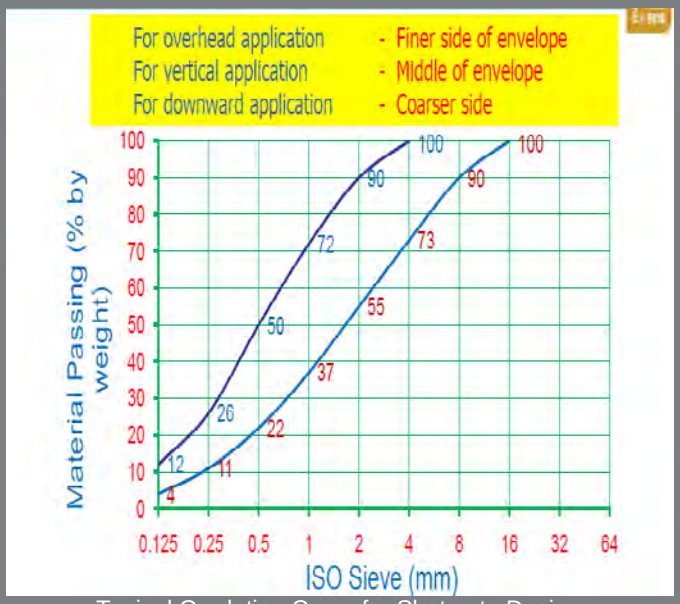
- Cement About 20 %
- Coarse aggregates - 15 to 20 %
- Sand - 60 to 65 %

Gradation analysis is essential and as a base rule, largest aggregate < 16 mm.(8-10 mm preferable). Rebound of shotcrete increases drastically with aggregates > 8 mm. Approximately. 60% to 70 % of aggregates greater than 8 mm are contained in rebound. Sufficient fine particles of cement and sand (<0.2mm) required

For dry mixes, Portland cement content should be within following limits

- Fine shotcrete 0-4mm : 450-600 kg/m³
- Shotcrete 0-8mm : 350-450 kg/m³
- Coarse shotcrete 0-15mm : 450-600 kg/m³

Water/Cement ratio of 0.40 to 0.45 is desirable for dry-mix. Accurate water control by nozzle man is required. Insufficient water causes excessive dust whereas too much water causes shotcrete to flow off surface. Water/Cement ratio for wet-mix is to be kept so as to produce slump = 50 mm to 150-175 mm. If slump is more, cohesion is lost. This also causes higher rebound of coarse aggregates and sand particles than that of cement. Therefore, cement content of in-place shotcrete is always higher than that of as-batched shotcrete (especially for dry mix in overhead application). Thus, composition of sprayed-on mix differs from initial mix. This has to be taken into account during mix design to achieve desired quality of final shotcrete.



Typical Gradation Curve for Shotcrete Design

Summary of Mix Design used at Tunnel T-1 (Revised no. T-23) on Udhampur-Katra section of USBRL Project		
Grade	M 25 (Tunnel T1 Revised no. T23)	M 20 (Tunnel T2 Revised no. 24)
W/C	0.40	0.42
Cement OPC	460 kg	500 kg
Water	184 kg	210 kg
10mm	520 kg	625 kg
Coarse Sand	620 kg	465 kg
Crusher Dust/Fine sand	630 kg	465 kg fine sand
Admixture	1.2%	1.0 %
Slump at placement	180 mm	180 mm
Accelarator	4%, 6% & 8%	3%
Silica Fume	Nil	Nil
Steel Fibre	Nil	Nil
CA:FA	30:70	40:60

* Quantities indicated for 1cum of mix.

Design of shotcrete support

The design of shotcrete support for underground excavations is a very imprecise process. Complex interaction between the failing rock mass around an underground opening, and a layer of shotcrete of varying thickness with properties which change as it hardens, defies most attempts at theoretical analysis. It is only in recent years, with the development of powerful numerical tools, that it has been possible to contemplate realistic analysis, which will explore the possible support interaction behaviour of shotcrete. Shotcrete is seldom used alone and its use in combination with rockbolts, cablebolts, lattice girders or steel sets further complicates the problem of analysing its contribution to support.

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

Current shotcrete support "design" methodology relies very heavily upon rules of thumb and precedent experience. Wickham et al (1972) related the thickness of a shotcrete tunnel lining to their Rock Structure Rating (*RSR*). Bieniawski (1989) gave recommendations on shotcrete thicknesses (in conjunction with rockbolts or steel sets) for different Rock Mass Ratings (RMR) for a 10 m span opening. Grimstad and Barton (1993) have published an updated relating different support systems, including shotcrete and fibre reinforced shotcrete, to the Tunnelling Quality Index *Q*. Vandewalle (1993) collected various rules of thumb from a variety of sources and included them in his monograph rock mass conditions.

Shotcreting Technique

A good quality shotcrete of desired strength & quality depends on many factors. Nozzle distance should be between 0.60 m to 1.2 m from surface. It should be such as to give best results i.e. highest degree of compaction, lowest rebound. Optimum nozzle distance depends on aggregate size of mix, grading curve, required surface finish, air pressure and speed of conveyed material

Spraying Angle of nozzle should be perpendicular to surface but never more than 45°. At too great an angle, shotcrete rolls or folds over, creating an uneven, wavy textured surface which can trap rebound and overspray. This is wasteful and may create porous and non-uniform shotcrete.

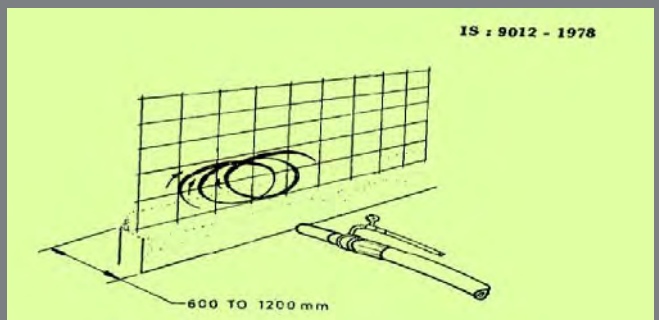


Fig. 1 Correct Use of the Nozzle

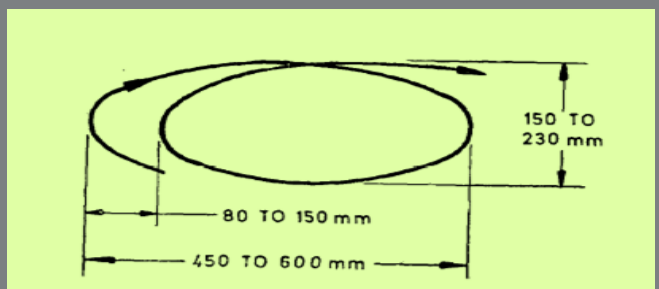


Fig. 2 Approximate Size of the Loop

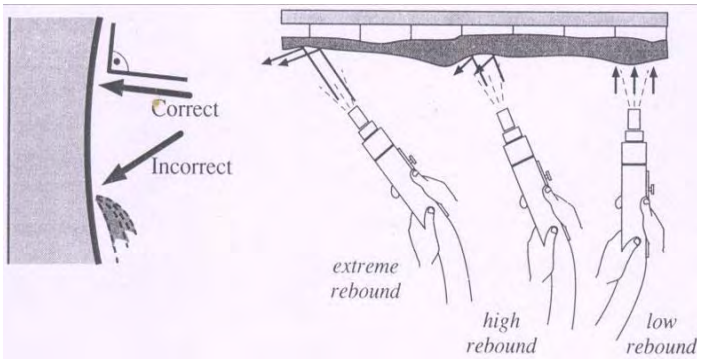
Rebound

Rebound is aggregate and cements paste which ricochets off the surface during the application of shotcrete due to collision with the hard surface, reinforcement or with the aggregate particles themselves. Rebound varies with:

- Work angle of nozzle
- Air pressure
- Cement content
- Water content
- Size & grading of aggregate
- Amount of reinforcement
- Thickness of layer

Apart from this the variation in the amount of rebound also depend type of surface and its location. As per IS Code IS:9012 the value of rebound is as below:

Surface	Percentage of Rebound (%)
Floor or slabs	5 – 15
Sloping and vertical walls	15 – 30
Overhead work	25 – 50



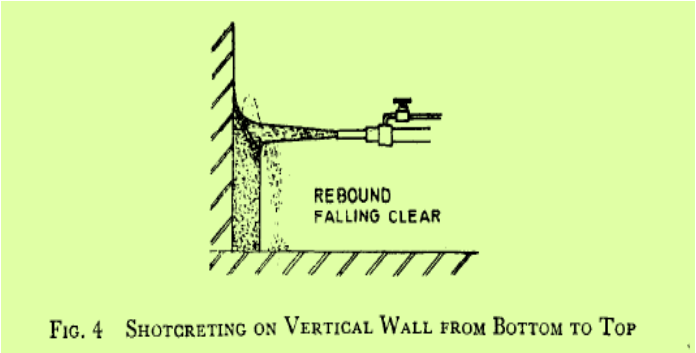


Fig. 4 Shotcreting on Vertical Wall from Bottom to Top

Effect of nozzle angle at rebound as per IS 9012

Initially the %age of rebound is large, but it becomes less after a plastic cushion has been built. For a good quality shotcrete it has to be ensured that rebound shall not be worked back into the construction. It has to be removed, if not fall clear of work. It is not to be salvaged and included in later batches due to danger of contamination, may affect cement content, state of hydration and grading of the aggregate.

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

Shotcrete Reinforcement

Plain unreinforced shotcrete is a relatively brittle material with little capacity to resist tensile stress without cracking. Therefore, shotcrete needs to be reinforced with steel reinforcement to increase the flexural strength and reduce cracks. Conventional form of reinforcement is placing of wire-mesh of 100mm to 150mm and a wire diameter of no more than 10mm, between two shotcrete layers. This has several disadvantages and the emphasis now is on Steel Fibre Reinforced Shotcrete (SFRS).

Steel Fibres

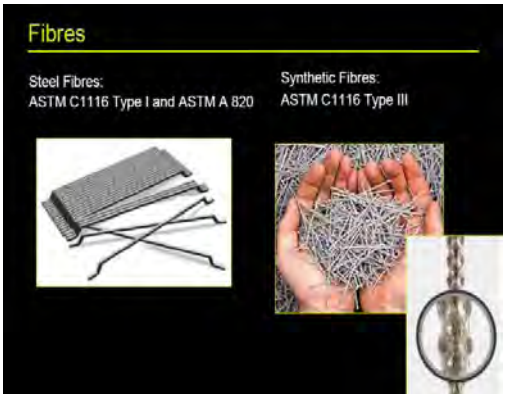
Fibers are generally used to increase the toughness of the concrete and to reduce or control the cracking. Fibers are normally supplied collated with fast-acting ater-soluble glue, or as uncollated individual fibers.

Of the many developments in the shotcrete technology in recent years steel fibers are introduced in shotcrete. Steel fiber shotcrete was introduced in 1970's and since gained worldwide acceptance as a replacement for the traditional wire mesh reinforced shotcrete. The main role that reinforcement plays in shotcrete is to impart ductility to an otherwise brittle material. The addition of steel fibers to shotcrete enhances both flexural and compressive strength of hardened shotcrete by up to 20%.

Steel fibers are straight or deformed cold drawn steel wires, straight or cut sheet fibers, fibers milled from steel blocks or melt extracted fibers which can be homogeneously mixed into concrete and mortar. Steel fibers are divided into five main groups and are defined in accordance with the basic material used for the production of the fibers.

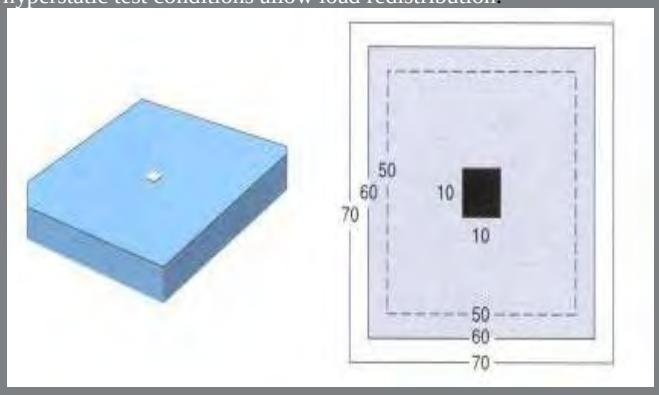
Group I Cold drawn steel wire
Group II Cut steel fibers
Group III Milled from steel blocks
Group IV Melt extracted fibers
Group V other steel fibers

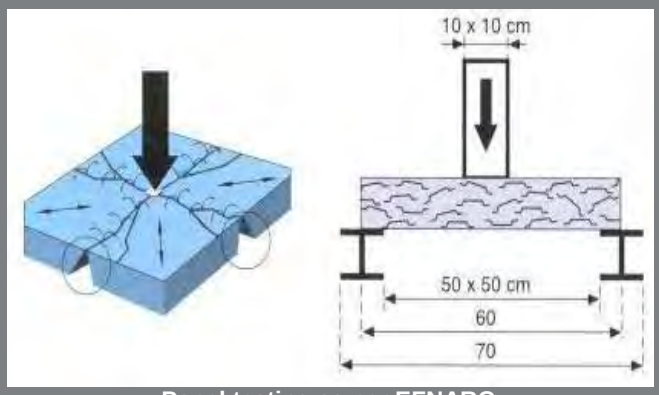
These fibers must comply with European standard EN 14889-1: minimum dosage (kg/m^3). Fibres with CE marking system 1, steel fibre for structural use (conform EN 14889-1-2006). Fibres outdrawn wire, with a tensile strength of steel wire > 1.0 MPa min. (The tensile strength of the wire must be consistent with that of the matrix, for High Performance Concrete (HPC) steel wire with a high tensile strength is required). Mixing and handling problems which hampered uniform distribution of fibres were eliminated by the manufacture of fibres with low aspect ratios (ratio of length to diameter), surface deformations, and improved shape. Best anchoring system have hooked ends for optimum anchorage. Fibre length is in the range of 30-35 mm. Maximum fibre length is least of the 2/3 of the hose diameter of the spraying machine or 2 times the maximum coarse aggregate size. Glued fibres are manufactured for improving homogeneous distribution .



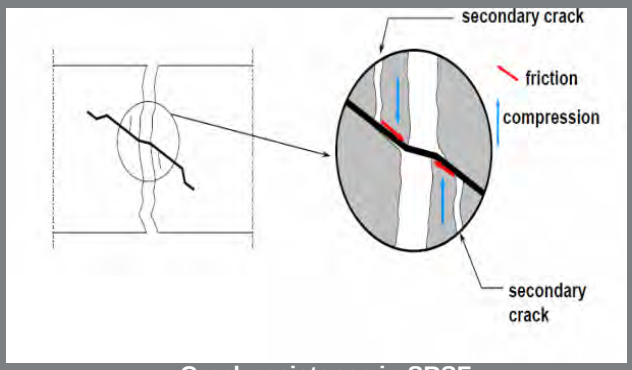
Panel test according to EN

The punching-flexion test is an ideal test to check the SFRS behaviour. This test is introduced in 1989 by the French Railway, and is in the meantime accepted by Efnarc, and is since 2006 in EN standards. A shotcrete tunnelling behaves like a slab. The hyperstatic test conditions allow load redistribution.



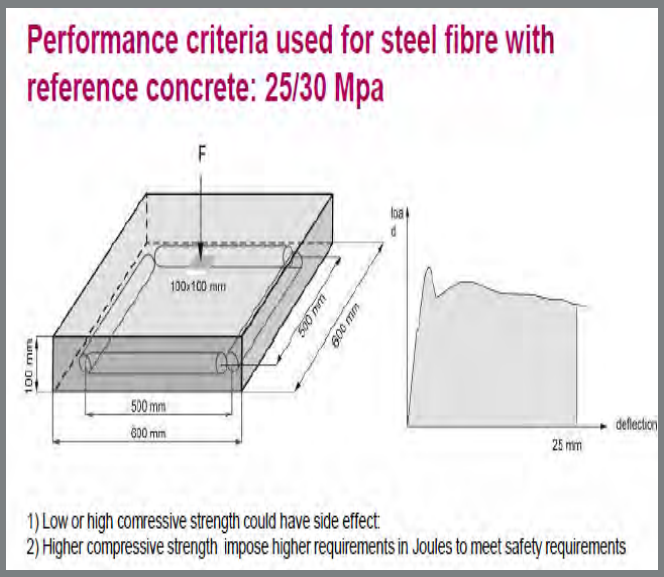


Panel testing as per EFNARC



Crack resistance in SRSF

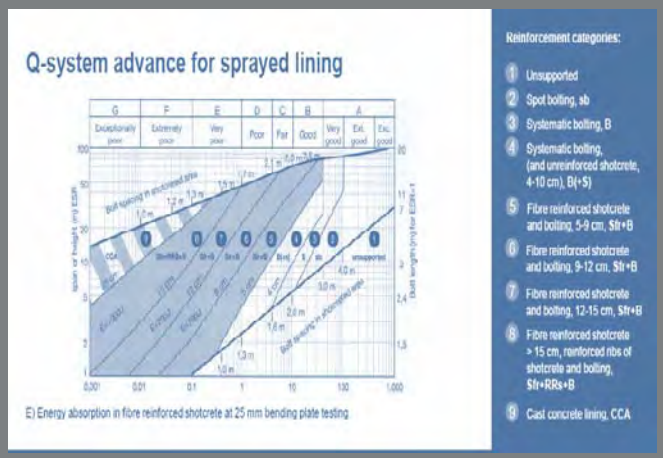
Cracking pattern



500 -J: min. requirement; rock; small diameter tunnel

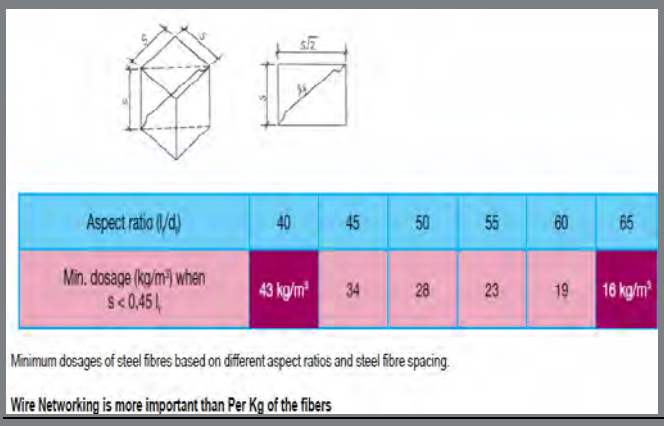
700 -J: standard; less good rock

1000.-J: high ductility for bad soil condition



Performance criteria E700/E1000 used in Q-system

L/D ratio Mc Kee theory according to EN



Aspect ratio (l/d)	40	45	50	55	60	65
Min. dosage (kg/m³) when s < 0,45 l	43 kg/m³	34	28	23	19	16 kg/m³

Minimum dosages of steel fibres based on different aspect ratios and steel fibre spacing.

Wire Networking is more important than Per Kg of the fibers

In addition to the requirement of the minimum overlap according to the Mc Kee theorie, Bekaert also recommends a minimum total length of wire fibre.

Aspect ratio: l/d (length/diameter)	Minimum kg/m³ according to min. overlap	l	D	fibres/kg	Total fibre length
40	43 kg/m³	30	0,75	9.000	11.610
65	20 kg/m³	35	0,55	14.500	10.150

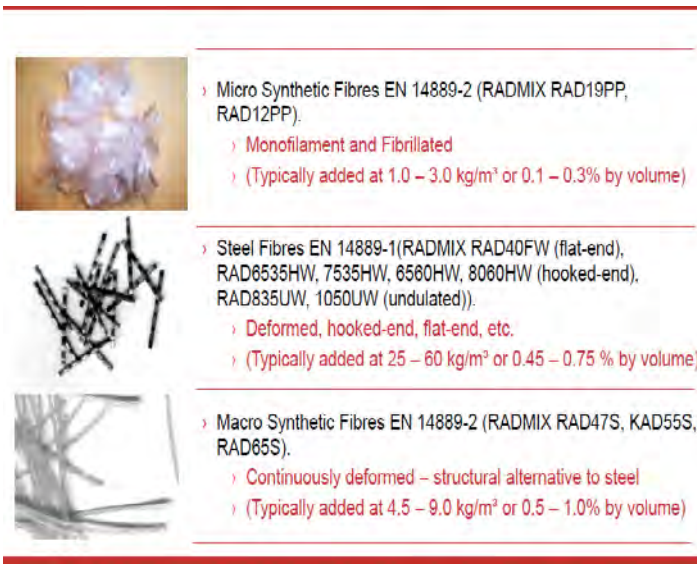
Glass-fibre reinforced shotcrete (GFRS):-

GFRS requires a special gun and delivery system. This process termed "spray-up". This type of shotcrete is specially used in making light weight panels and claddings. Guidelines for GFRS provided by prestressed concrete institute PCI (1981).

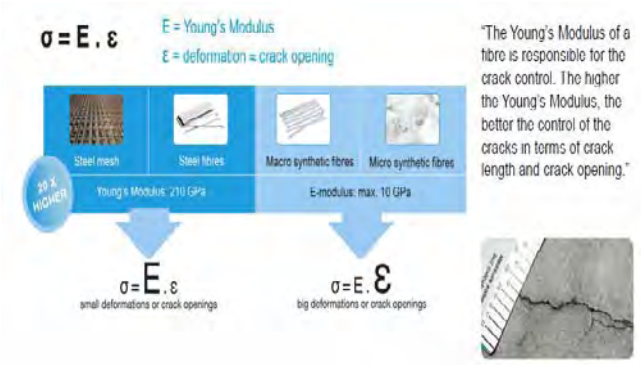
Synthetic Fibres:-

These are primarily composed of nylon, polypropylene, polyethylene, polyester and rayon. Primary benefit is reduced width of shrinkage crack. Reduces segregation of the concrete mixture. Reduces formation of shrinkage cracks while the concrete is in the plastic state. As the modulus of elasticity of concrete increases with hardening of concrete, however, most synthetic fibres at typical dosage rates will not provide sufficient restraint to inhibit cracking.

Future development includes the BP (high carbon: > 2000Mpa) fibre for fast setting sprayed concrete which develops a very high early strength cutting the cycle time but requiring a high strength steel fibre to guarantee the ductility of a high strength spray concrete.



Material	Steel Mesh / Steel fibre 	Micro / Macro Polymer Fibre Extruded polypropylene / polyethylene 
Typical length of fibres	30-60 mm	micro: 6 - 20 mm macro: 30 - 65 mm
Typical diameter of fibres	0.5 - 1.0 mm	micro: 0.015 - 0.030 mm macro: 0.5 - 10 mm
Young's Modulus	210000 MPa	3000 - 10000 MPa
Tensile strength	500 - 2000 MPa	200 - 600 MPa
Density	7850 kg/m³	910 kg/m³
Melting Point (°C)	1500°C	165°C does not reinforce
Creep behaviour in tension (Tg glastransition temperature)	+370°C	-20°C



Advantages of SFRS over Mesh Reinforced Shotcrete

- ✓ Lower shotcrete consumption.
- ✓ Shotcrete builds up on wires of mesh leaving voids behind and a poor bond.
- ✓ Fixing of mesh is difficult and time consuming.
- ✓ High rebound on wires.
- ✓ Big size aggregates hitting the wires create vibrations resulting in a poor bond.
- ✓ Thick shotcrete tends to fall under its weight on the mesh, breaking the bond. SFRS has better load carrying capacity

Bond

Good shotcrete bond is a must for proper. interaction with rock mass. Poor bond leads to creation of voids. Adhesion is better on firm, clean, dry surface. Spraying against running water leaves permanent erosion path with no bond. Adhesion to hard, dry rock is much higher while it is almost zero in soft, damp ground.

Practice of shotcreting only above springing level can succeed in hard, dry rock only. In soft, damp ground shotcrete must be built up from invert. Shrinkage cracks reduce bond, better bond in SRFS. Silica fume increases bond even on poor surfaces. Wire -mesh negatively affects the bond. Wires act as obstacles for shotcrete to reach surface. At places of overbreaks, thick shotcrete breaks the bond due to its weight and falls on wire-mesh. Shotcrete may build up on wires, creating voids behind.

Shotcreting Application Technique

The quality of the final shotcrete product is closely related to the application procedure used. These procedures includes: surface preparation, nozzling technique, lightening, ventilation, communications, and crew training. Shotecrete should not be applied directly to a dry , dusty or frozen rock surfaces. The work area is usually sprayed with an air-water jet to remove loose rock and dust from the surface to be shot. The damp rock will create a good surface on which to bond the initial layer of shotcrete paste

The nozzle man commonly starts low on the wall and moves the nozzle in small circles working his way up towards the back, or roof. Care must be taken to avoid applying fresh materials on top of rebound or over sprayed shotcrete. It is essential that the air supply is consistent and has sufficient capacity to ensure the delivery of a steady stream of high velocity shotcrete to the rock face. Shooting distances are ideally about 1 to 1.5 meters. Holding the nozzle further from the rock face will result in a lower velocity flow of materials which leads to poor compaction and higher portion of rebound.

To Reduce rebound and Spraying Mist

- Use high grade nozzle with a compact spray jet
- Use steel fibres instead of wire-mesh
- Use semi-wet spraying instead of dry spraying
- Add silica fume to increase stickiness.
- Use higher cement content and more fines
- Use smaller maximum size aggregates and a finer gradation
- Use proper moisture content
- Maintain proper spraying angle
- Manual Shotcreting requires an operator for each nozzle plus a work platform for close access to profile
- Manual operation does not always produce shotcrete of best quality, particularly over irregular surfaces
- Manual operation is arduous for shotcreters heavy protective gear, fierce rebound, fighting against high pressure
- This is a very tiring and uncomfortable job, especially for overhead shooting, and compact robotic systems are increasingly being used to permit the operator to control the nozzle remotely
- Typical robotic spray booms are trailer or truck mounted
- Wet-mix process with spray robots is preferred to dry-mix process for large amounts of spray, especially for large diameter tunnels
- Successful shotcrete requires high capacity system with high pressure delivery for good compaction, manual control of nozzle impossible
- Demand for faster tunnel construction requires shotcrete application of more than 10 m³ per hour

Personnel

A well trained operator can produce excellent quality shotcrete manually, when the work area is well-lit and well-ventilated, and when the crews are in good communication with each other using prescribed hand signals or voice activated FM radio handsets. However, this is a very tiring and uncomfortable job, especially for overhead shooting, and compact robotic systems are increasingly being used to permit the operator to control the nozzle remotely.

Hence Skilled nozzle man and pump operator are required. Nozzle man has to fight dust and rebound while dry- spraying. Nozzle man has to hold heavy hose filled with shotcrete while wet-spraying. Pump operator is responsible for providing constant flow of pre-mixed shotcrete to nozzle man.

Compact trailer-mounted robot unit for emote controlled shotcrete application.

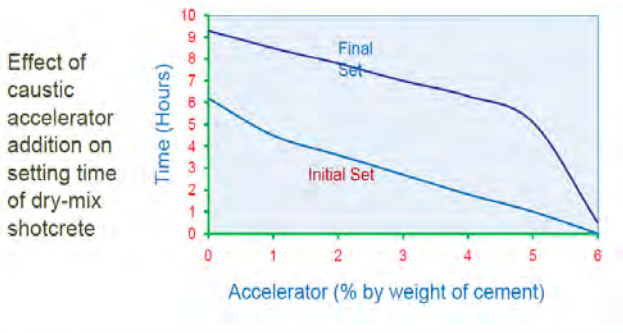
Shotcrete operator using a remotely controlled unit to apply shotcrete to a rock face in a large civil engineering excavation

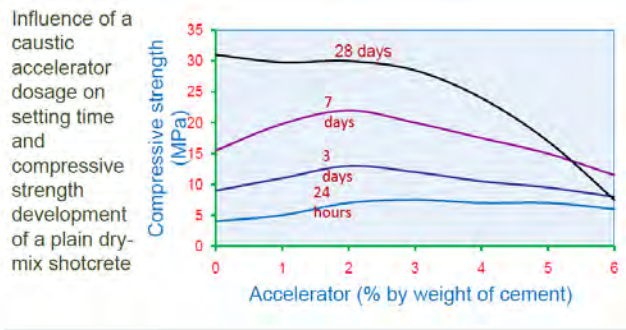
Additives

Admixtures A sprayed concrete mix may include admixtures such as plasticizers, retarders, etc., (just as conventional concrete does to improve the fresh mix properties and the hardened concrete quality), to ensure a good spraying application and to meet early strength requirements. The designation 'chloride-free' for admixtures implies that the chloride ion content does not exceed 0.1% by mass of the admixture.

Accelerators chemical admixtures are added to concrete during spraying to increase the stiffening rate, to produce a fast set and to get sufficient early strength development. A fast setting concrete may be necessary to build up the lining at the required thickness and to ensure overhead security. The dosage should be adjusted to ensure good cohesion between individual passes producing a single layer.

- Cause shotcrete to gain strength more quickly to support tunnel at an early stage
- Reduce rebound of aggregates
- Enable thicker layers to be sprayed in one pass without wasting time in waiting for previous layers to set
- Enable shotcrete application on wet surface
- Setting time of 3 mins. for initial set & 10 mins. for final set, with min. 8 hr. Compressive strength of 5 Mpa are sometimes required in tunnelling and mining.
- These and even more rapid 'flash-set' setting times can be achieved by careful selection of suitable cement and accelerometer combination
- Preconstruction evaluation of cement and accelerometer is strongly recommended to rule out incompatibility.
- Caution required in selecting accelerator dosage as accelerators result in strength reduction
- Flash setting characteristics attained at expense of drastic reduction in strength
- Many modern accelerators have much less detrimental effect on strength
- Normal dosing of accelerators is around 2% by weight of cement, but upto 7% used for extra rapid hardening
- Use of accelerator dispensing equipment results in uniformity of dispersion of accelerator and less variation in strengths
- Excellent uniformity of accelerator dispersion in dry-bagged premix supply
- Due to variation in rebounds, different accelerator dosages required for vertical and overhead applications and for hard & soft surfaces.





Benefits of Alkali Free Accelerator (AFA) Over Traditional Accelerators	
AFA – Silicates	AFA – Aluminates
• Faster setting • Higher early strength • Faster strength development • Reduced loss of final strength • Reduced permeability • Dramatic reduction in rebound • Ability to apply large thicknesses • Better durability • Reduced risk for alkali-aggregate reaction • Better working environment and improved safety (less dust) • Environment friendly	• Less sensitive to cements • Dramatic reduction in loss of final strength • Better durability • Reduced risk of alkali-aggregate reaction • Better working environment and improved safety • Reduced permeability • Environment friendly

Constituents of set accelerators

Currently, there are 3 types of set accelerators available:

Silicate based

High alkali content (Na2O) and caustic (irritant), pH above 11, require personal protection against skin and eye burns. Display fast stiffening, but can significantly decrease the final strength (upto 50%) and durability, especially when overdosed. Risk of alkali silicate reaction (ASR) and leaching of water soluble portions. Typical dosages are 6 – 14% by weight of cement. Very slow strength development and little possibility to build high total thickness overhead.

Aluminate based

Alkaline and caustic, pH above 12, require personal protection against skin and eye burns. Take part in the hydraulic reactions of cement and show very good stiffening and hardening effects, but can significantly decrease the final strength (upto 30%) and durability, especially when overdosed. Risk of alkali silicate reaction (ASR). Typical dosages are 3 – 8% by weight of cement.

Alkali –free

Alkali-free (Na2O<1%) accelerators are preferred because they are less hazardous (non caustic) and give a better working environment. Has fast setting and good strength development provided correct type of compatibility with cement. These accelerators have a positive effect even on the final strength and durability of the concrete. For permanent sprayed concrete, it is always recommended to use alkali-free accelerators. Typical dosages are 5- 10% by weight of cement

Type of set accelerator should be selected based on the type of application, specification, cement compatibility and local conditions.

Other additives

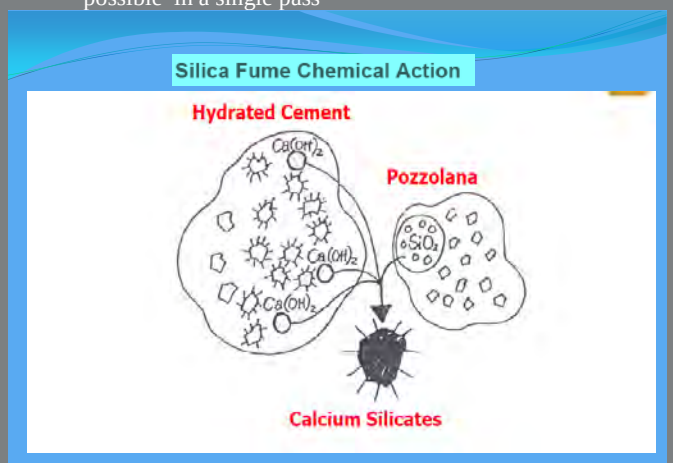
Silica Fume

It is a By-product of silicon metal production process. Fume is filtered from gases escaping furnace and contains very fine SiO_2 particles. Light grey, extremely fine powder.

- Appreciation of silica fume's finess can be obtained from values of specific surfaces:

Silica fume	20,000 m^2/kg
Fly ash	400-700 m^2/kg
Normal Portland Cement	300-400 m^2/kg

- Dosage = 8 to 12% by weight of cement.
- Creates a very dense & sticky mix by packing of ultrafine particles between particles of cement.
- Has high water demand due to high specific surface-water demand of wet-mix concrete increases from 160 to 220 l/m^3 with 8-12% silica fume.
- It is essential to use super plasticizer to keep water demand under control .
- Increases adhesion – very useful in wet conditions.
- Reduces dust and rebound.
- Increases strength.
- Usually eliminates uses of accelerators.
- Has a strong wash-out resistance against water.
- Makes overhead application of thick shotcrete layer possible in a single pass



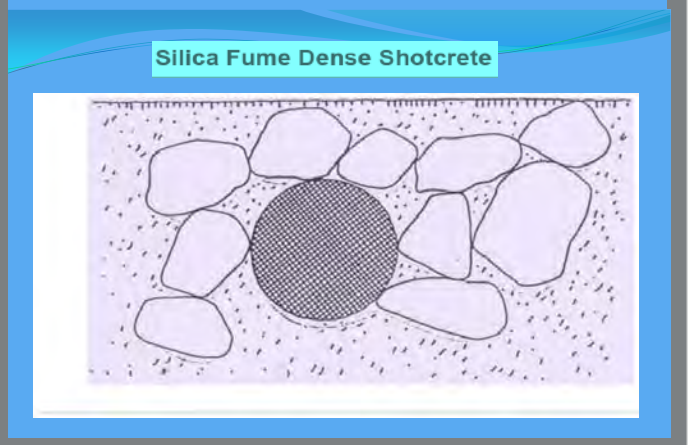


Table indicating some commercially available products of M/s SIKA

Type	Product	Use/effect	Remarks
Superplasticisers	Sika® Tard Sika® ViscoCrete®	• High water reduction • Better workability • Time controlled workability • Rapid increase in strength • Better shrinkage and creep properties • .Higher Watertightness.	• Optimum effect when added after the mix water. • Optimum dosage depends on cement type • For specific properties. preliminary tests with the cement and aggregates to be used are essential.
Retarder	Sika® Tard -930	• Adjustable workability • No cleaning of pumps and hoses necessary during the retarding phase.	
Silicafume Slurries Silicafume Powder	Sikacrete® - L SikaFume ®	• Improved fresh concrete homogeneity. • Much higher watertightness • Improved adhesion between aggregate and hardened cement • High frost and freeze/thaw resistance. • Lower rebound	• Added at the batching plant. • Optimum curing is necessary because silicafume concrete dries out very quickly on the surface.
Polymer-modified Silicafume Powder	Sikacrete® - PP1	• As for SikaFume ® plus: • Significant water reduction • For very high quality specification.	• As for SikaFume®
Pumping agents and stabilizers	SikaPump® Sika®Stabilizer	• Improvement in homogeneity and internal cohesion for unsuitable concrete mixes. • Increase in spraying output with lower energy consumption even for mixes with crushed aggregate.	Addition increase the power input of the mixer and the concrete consistency – do not adjust by adding water.

Water Reducing Agents

Plasticizers

Plasticizers are used in % dosages to achieve pumpable concretes with minimum water content and to improve quality of shotcrete. They reduce excess water contained in wet-mix to prevent shrinkage cracks

Super Plasticizers

Superplasticizers are used in sprayed concrete to minimize the amount of water in the mix, thereby improving the final quality. They are mainly used to give the required consistence for spraying and to aid pumpability. They improve workability of wet shotcrete and to increase its strength.

Superplasticizer are Organic chemical admixtures & are classified as Melamine based or Naphthalene based. Superplasticizers may be used either to increase slump without loss of strength, or to retain slump while increasing strength.

Some other additives

Polymer Latex Additives could be added to dry-mix shotcrete for adhesion improvement, permeability reduction, resistance to chloride attack. They have increased durability in freeze/thaw and have impact resistance. They can be used for steel protection & strength improvement.

Air-entraining Admixtures are used for wet-mix subjected to freeze/thaw.

SLOPE STABLISATION

Use of shotcrete for slope and rock stabilization has increased substantially in the past few years. Fibre reinforced shotcrete (FRS) allows the shotcrete to follow the contours of the slope. It takes less time and skill to reinforce shotcrete with fibres than with mesh or rebar.

Factors that the designer should keep in mind for slope-stabilization projects:

- *Slope he ight and an gle :* Steep slopes are often difficult or impossible to place mesh.
- *Ground surface condition:* Pins must be driven into the ground to hold the mesh or rebar in place.
- *Ground water:* Drainage must be provided to prevent the build up of water pressure on the back of the shotcrete facing.
- *Rock bol t or s oil nail s pacing: R* ock bolts or soil nails are required for stability reasons, the shotcrete facing must be designed to span between the anchors.
- *Earth pr essures and arching a bility:* Soil nail facings will develop positive bending moments in the sand and midspan between nails, and negative bending moments at nail-head locations.
- *Mild s teel r einforcement* may be placed in specific areas to resist large moments that exceed the capacity of steel fibre reinforced shotcrete alone.

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

SUMMARY OF MIX DESIGN CONCEPT FOR SLOPE STABLISATION BY WOODS (1992)

Steel fibers used in shotcrete should never be longer than 30 mm long, unless special large nozzles are used to spray the SFRS.

Material/Property	Comments
Cement content Silica fume Aggregate gradation	18 to 21% by mass of dry components 8 to 13% by mass of cement ACI 506R-90, Table 2.1, Gradation No. 1, or (more commonly) No. 2.
Steel fiber type Aspect ratio Steel fiber addition rate	High strength, deformed, drawn, or slit sheet steel fibre. Length : Equivalent diameter ratio of 50-70 50-80 kg/m^3 (85-135 lb/yd^2) depending on required toughness.
Admixtures	1. Water reducers used in wet-mix shotcretes at about 0.51/100kg cement (8 fl.oz/100 lb cement). 2. Superplasticizers required in wet-mix silica fume shotcrete at 1 to 1.51/100 kg cement (16 to 24 fl.oz/100 lb cement) 3. Air entraining admixtures used in wet-mix shotcrete where freez-thaw durability important (10 to 12% air content at pump, 4 to 6% air content as-shot).
Accelerator	Dosage depends on accelerator type and required setting time (2 to 5% by mass of cement typical)
Water cement ratio Slump	0.3 to 0.4 for dry-mix shotcrete. 0.4 to 0.5 for wet-mix shotcrete. 40 to 100 mm (1½ to 4 in) for wet-mix shotcrete.

SUMMARY OF MIX DESIGN CONCEPTS (MORGAN, 1990)

Material	Dry mix shotcrete		Wet-mix shotcrete	
	Kg/m^3	lb/yd^3	Kg/m^3	lb/yd^3
Cement	400	674	420	708
Silica fume	50	84	40	67
10 mm coarse aggregate	500	843	480	809
Concrete sand	1170	1972	1120	1888
Steel fibres	60	101	60	101
Water reducer	--	--	2L	52 fl.oz.
Super plasticizer	--	--	6L	156 fl.oz.
Air entraining agent	--	--	If required	If required
Water	*170	*287	180	303
Total wet mass	**2350**	**3961**	**2300**	**3876**
Total water from premoistuizer and added at nozzle (based on saturated surface dry aggregate concept)				

Shotcrete for slope stabilisation

Freeze-thaw problems: If ice lenses are allowed to form beneath a shotcrete facing, it is difficult to keep the shotcrete from cracking or heaving.

Conclusion: Fibre-reinforced shotcrete for slope stabilization and soil-nailed walls really work. The process is simple, effective, and does the job easily and quickly

TUNNELLING UNDER SQUEEZING CONDITIONS

Terzaghi (1946) provides one of the earliest definitions of squeezing rock behaviour with respect to tunnelling. "Squeezing rock slowly advances into the tunnel without perceptible volume increase. Prerequisite of squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity"

Under "squeezing ground" conditions the magnitude of deformation often far exceeds the deformability of the shotcrete lining. One solution is division of the shotcrete lining into circumferential "segments", leaving gaps between these "segmental arches". In combination with a dense rock bolt pattern was successfully used in many alpine tunnels. Missing thrust transmission between the lining "segments", resulting in a rather low radial support pressure.

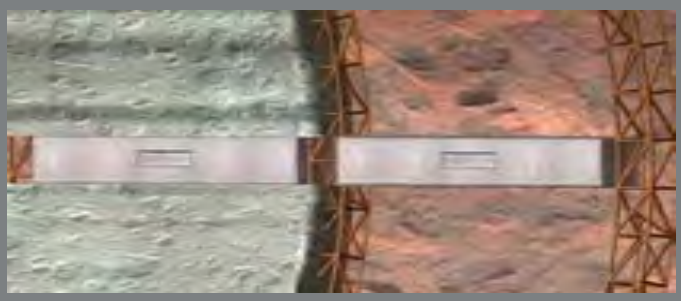
Table 1. Classifications for squeezing potential in tunnels.

Class #	Hoek (2001)		Aydan et al (1993)		Singh et al (2007)	
	Squeezing Level	Tunnel Strain	Squeezing Level	Tunnel Strain	Squeezing Level	SI
1	Few support problems	$\varepsilon_t < 1\%$	No squeezing	$\varepsilon_\theta^a / \varepsilon_\theta^e \leq 1$	No squeezing	$SI < 1.0$
2	Minor squeezing problems	$1\% < \varepsilon_t < 2.5\%$	Light squeezing	$1 \leq \varepsilon_\theta^a / \varepsilon_\theta^e \leq 2.0$	Light squeezing	$1.0 < SI \leq 2.0$
3	Severe squeezing problem	$2.5\% < \varepsilon_t < 5\%$	Fair squeezing	$2.0 \leq \varepsilon_\theta^a / \varepsilon_\theta^e \leq 3.0$	Fair squeezing	$2.0 < SI \leq 3.0$
4	Very severe squeezing problem	$5\% < \varepsilon_t < 10\%$	Heavy squeezing	$3.0 \leq \varepsilon_\theta^a / \varepsilon_\theta^e \leq 5.0$	Heavy squeezing	$3.0 < SI \leq 5.0$
5	Extreme squeezing problem	$\varepsilon_t > 10\%$	Very heavy squeezing	$\varepsilon_\theta^a / \varepsilon_\theta^e > 5.0$	Very heavy squeezing	$5.0 < SI$

Where tunnel displacments is ε_t, (ε_θ^e) is elastic strain, (ε_θ^a) tangential strain

$$SI = \frac{\text{Observed or expected strain}}{\text{Critical strain}}$$

Recently in Austria low-cost, yielding-steel elements called LSC ("Lining Stress Controller") have been introduced. In this multiple steel pipes in a concentric assembly which are installed between the shotcrete segments . Efficient in terms of reducing displacements and controlling stresses in the shotcrete support . Lightness, ease of construction and a high potential of energy absorption with a constant average collapse load at failure. Energy absorbing elements can be custom-tailored to individual requirements depending on the geological and stress conditions. One of the recent solution is the yieldable concrete elements for the application in squeezing rock conditions. They have a beam shape and are longitudinally fixed to the excavation surface at different locations.



St. Martin la Porte access tunnel: Detail of hiDCon elements within the yielding tunnel support

Application of these yieldable concrete elements in strongly swelling rock is achieved by using the new design concept called "Modular Yielding Support". In this compressible elements of high bearing capacity are placed between the structure and the rock. The tunnel support system was changed to yielding shotcrete to manage the difficult, heavily squeezing rock conditions. This support system consists of steel ribs or lattice girders, high deforming elements and shotcrete. The final lining is cast in place 80 – 100 m behind the face.

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

Saint Martin la Porte access tunnel, change from stiff to yielding support (source: Razel / Bilfinger Berger / Pizzarotti)

Relevant codes & standards

Indian standards

- IS: 456 – 2000 Plain and Reinforced Concrete - Code of Practice
- IS: 269 – 1989 Specification for 33 grade ordinary Portland cement
- IS: 1489 – Part 1: 1991 Specification for Portland pozzolana cement Part 1 Fly ash based
- IS: 1489 – Part 2: 1991 Specification for Portland-pozzolana cement Part 2 calcined clay based
- IS: 383 – 1970 Specification for coarse and fine aggregates from natural sources for concrete
- IS: 516-1959 Method of test for strength of concrete
- IS: 9012-1978 (Reaffirmed1987) Recommended practice for shotcreting
- IS: 15026-2002 Tunnelling methods in Rock Masses - Guidelines
- IS: 2645-2003 Integral Waterproofing Compounds for Cement Mortar and Concrete – Specification
- IS: 9103-1999 Concrete Admixtures – Specification
- IS: 12269-1987 Specification for 53 grade ordinary Portland
- IS: 8112-1989 Specification for 43 grade ordinary Portland cement

Standard and Codes	
	American Concrete Institute (ACI)
	American Society for testing and Materials (ASTM)
	Canadian Standards Association (CSA)
	National Ready Mix Concrete Association (NRMCA)
	Concrete Plant manufacturer's Bureau (CPMP)

Standard and Codes	
ASTM C114	Standard test Method for Chemical Analysis of Hydraulic Cement
ASTM C642-82	Standard Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete
ASTM C1116	Specification for Fibre reinforcement Concrete
ASTM C1140	Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels
ASTM C1141	Specification for Admixture for Shotcrete
ASTM C1436	Standard Specification for Material for Shotcrete
ASTM C1550	Standard Specification for Flexural Toughness of Fibre Reinforced Concrete (Using Centrally Loaded Round Panel)
ASTM C1602	Standard Specification for Mixing Water Used in Production of Hydraulic Cement Concrete
ASTM C1604	Standard Test method for Obtaining and Testing Drilled Cores of shotcrete
American Concrete Institute (ACI)	
ACI 506R-05	Guide to Shotcrete
ACI 506.2-95	Specification for materials Proportioning and Application of Shotcrete
Canadian Standards Association (CSA)	
CSA A23.1-04	Concrete Materials and Methods of Concrete Construction
CSA A2302-04	Method of Test and Standard Practices for Concrete
CSA A3001-03	Cementitious Materials for use in concrete

American Standards for Testing Materials:

- ASTM C150 - Standard Specification for Portland Cement.
- ASTM C33 - Standard Specification for Concrete Aggregates.
- ASTM A820 - Standard Specification for Steel Fibres for Fibre-Reinforced Concrete.
- ASTM C 1018-89 – Standard test method for flexural toughness and first crack strength for fibre reinforced concrete

American Association of State Highway and Transposition Officials (AASHTO):AASHTO T26 - Standard Method of Test for Quality of Water to be used in Concrete

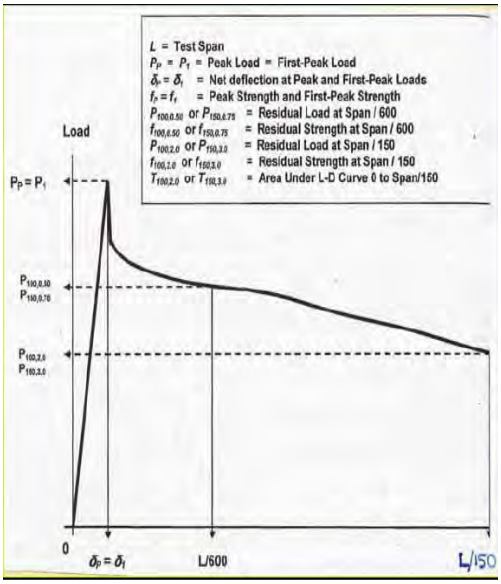
British Standards Institution:BS 5328 – Methods for specifying concrete, including ready-mixed concrete

Austrian standard:Austrian Guideline for Shotcrete, Österreichischer Betonverein (ÖBV), Oct. 1998

European Standard:European Specification for Sprayed Concrete, EFNARC 1996

- EN 480-12

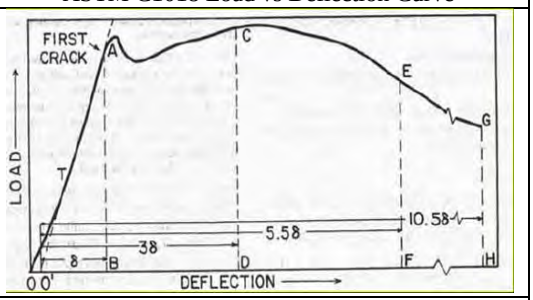
Fibre Reinforced Shotcrete Testing in North America
Steel fibre synthetic macro-fibre reinforced shotcrete in North America has been tested to the following ASTM Standards:
ASTM C1018 : Standard test method for flexural toughness and first-crack strength of fibre-reinforced concrete (using beam with third point loading) first published 1984, withdrawn 2006
ASTM C1609 : Standard test method for flexural performance of fibre-reinforced concrete (using beam with third point loading) first published 2005
ASTM C1550 : Standard test method for flexural toughness of fibre-reinforced concrete (using centrally loaded round panel)



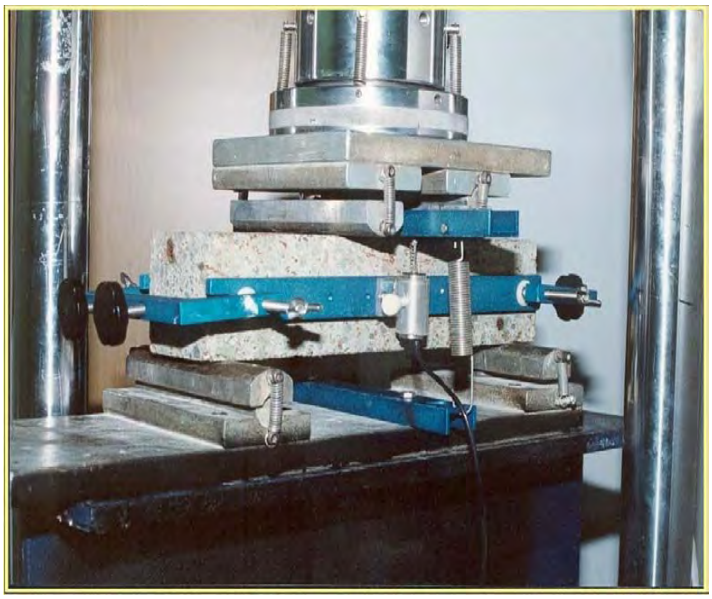
Limit for Deleterious Substance and Physical Properties of Aggregates			
Property		Maximum percentage by mass of Total Sample	
CSA Test Method	Standard Requirements	Fine Aggregate	Coarse Aggregate
A23.2-3A	Clay lumps	1	0.25
A23.2-4A	Low-density granular materials	0.5	0.5
A23.2-5A	Material finer than 80 µm	3.0	1.0
A23.2-13A	Flat and elongated particles Procedure A, ratios 4:1	-	20
A23.2-23A	Micro-Deval test	20	17
A23.2-29A	Unconfined freeze-thaw test	-	6
A23.2-16A	Impact and abrasion loss	-	50
A23.2-9A	$MgSO_4$ soundness loss	16	12

Fibre Shotcrete Tests

ASTM C1018 Load vs Deflection Curve

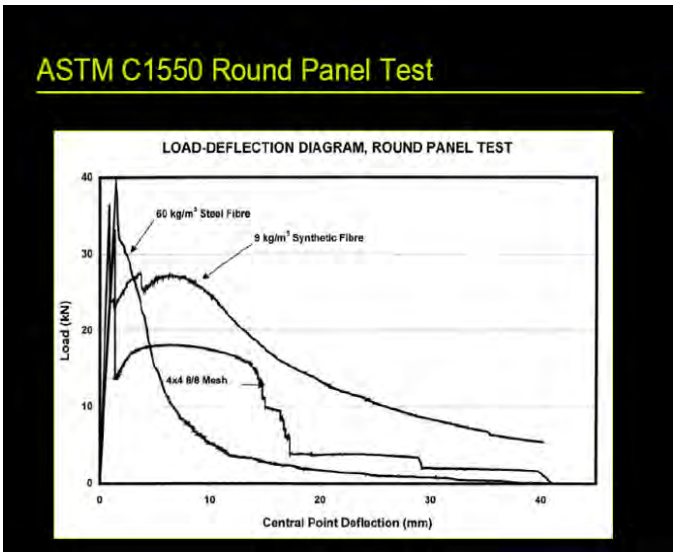


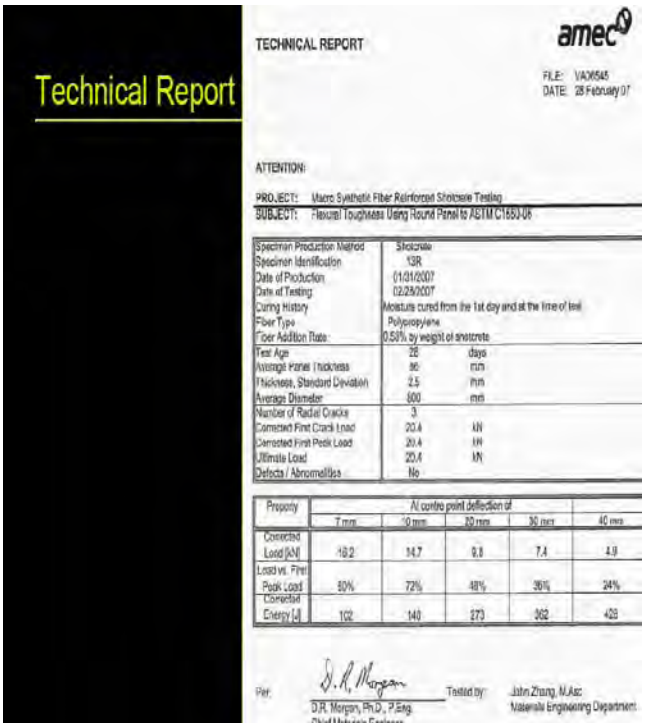
- Test Results Include :
  - First crack and ultimate flexural strengths (MPa)
  - Toughness Indices I_5, I_{10}, I_{20}, etc.
  - Residual Strength Factors $R_{5,10}$, $R_{10,20}$, etc
- Problems of reliability with test method and withdrawn in 2006



ASTM C1018 and ASTM C1609 Beam Test

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY





Technical Report

TECHNICAL REPORT

amec

FILE: VAX6545
DATE: 28 February 07

ATTENTION:

PROJECT: Micro Synthetic Fiber Reinforced Shotcrete Testing
SUBJECT: Flexural Toughness Using Round Panel to ASTM C1550-06

Specimen Production Method	Shotcrete
Specimen Identification	13R
Date of Production	01/31/2007
Date of Testing	02/28/2007
Curing History	Moisture cured from the 1st day and at the time of test
Fiber Type	Polypropylene
Fiber Addition Rate	0.50% by weight of shotcrete
Test Age	28 days
Average Panel Thickness	86 mm
Thickness, Standard Deviation	2.5 mm
Average Diameter	600 mm
Number of Radial Cracks	3
Corrected First Crack Load	20.4 kN
Corrected First Peak Load	20.4 kN
Ultimate Load	20.4 kN
Defects / Abnormalities	No

Property	At centre point deflection of				
	7 mm	10 mm	20 mm	30 mm	40 mm
Corrected Load (kN)	18.2	14.7	9.8	7.4	4.9
Load vs. First Peak Load	80%	72%	48%	36%	24%
Corrected Energy (J)	102	140	273	362	428

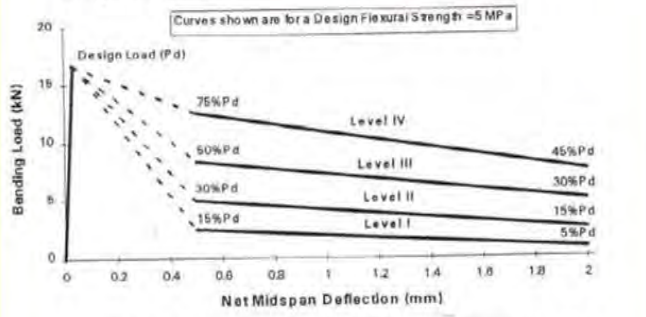
Per: D.R. Morgan, Ph.D., P.Eng.
Chief Materials Engineer

Tested by: John Zhang, M.Asc
Materials Engineering Department

Flexural Toughness Performance Levels Morgan and Chen, 1995

Toughness Performance Level	Residual Flexural Strength at Shown Deflection (by % of Design Flexural Strength)	
	1/600 Span (0.5 mm)	1/150 Span (2.0 mm)
0	No Fibre Reinforcement	
I	15%	5%
II	30%	15%
III	50%	30%
IV	75%	45%

3.4 Table A.1 is graphically demonstrated in Figure A.2 for a design flexural strength of 5 MPa.



Definition of Flexural Toughness Performance Levels for FRC

Quality control of shotcrete is more difficult than the conventional concrete since it is affected not only by the accuracy of batching but also by the skill and continued care of the crew applying shotcrete. To ensure quality of shotcrete, small unreinforced test panels, atleast 30cm X 30cm and 75cm thick shall be periodically gunned and cores or cubes extracted for compressive test and visual examination. Test cores shall also be taken from completed work as often as necessary to ensure that the control test reflect the quality of material in the structure.

Shotcrete Batching and Supply Certification and Testing		
Test Method	**Test Procedure**	**Testing Frequency**
NRMCA CPMB 100 M-07	Certification of ready mixed concrete production facilities using truck mixing	At start of project and every 2 years
NRMCA CPMB 100 M-07	Certification of ready mixed concrete truck mixers and remixer units	Before use and every 6 months
NRMCA CPMB 100 M-07	Batch records with solid materials in kg and liquids in L	Each batch of shotcrete produced

Salient features of Indian Standard on shotcrete

- Most reported values for 28 day strength are in the range of 20 to 50 N/mm^2
- 25 N/mm^2 be specified only for the most carefully executed shotcrete jobs.

SHOTCRETE IN TUNNELLING- CONCEPT, PRECAUTION & METHODOLOGY

PRECONSTRUCTION TESTING

- Test panels simulating actual job conditions shall be fabricated by the operating staff, using the equipment, materials and mix proportions proposed for the job
- It may be advisable to test two or three mixes, generally within the range of 1 part of cement to 3 to 4 parts of sand, before deciding on the final mix proportions
- Normally 20% to 40% of coarse aggregate is first tried, with subsequent mixes adjusted to reflect the results of the first trial
- Panels are fabricated by gunning on to a back form of plywood
- A separate panel shall be fabricated for each mix design, each gunning position to be encountered
- At least part of the panel shall contain the same reinforcement as the structure
- Size of panel shall be not less than 75 x 75 cm
- The thickness shall be the same as in the structure except that it shall normally be not less than 7.5 cm.
- Cubes or cores shall be taken from the panels for testing.
- Cores shall have a minimum diameter of 7.5 cm.
- Length-diameter ratio of at least 1, if possible.
- The specimens shall be tested in compression at the age of 7 or 28 days or both.
- The cut surfaces of the specimens shall be carefully examined and additional surfaces shall be exposed by sawing or breaking the panel, to check soundness and uniformity of material.

Typical Shotcrete Performance Requirements

Test Method	Property	Performance Requirement
-	Maximum water/cementing materials ratio	0.40
CSA A23.2 – 5C	Slump (at discharge into shotcrete pump)	170± 20mm
-	Temperature (at discharge into shotcrete pump)	10-30° C
CSA A23.2 – 4C	Air content – as shot	2 – 5 %
Penetrometer as calibrated against beams tested as equivalent cubes	Minimum early age compressive strength before re-entry under shotcrete	2 MPa
ASTM 1604 & CSA A23.2 – 14C	Minimum compressive strength of shotcrete cores 1 days 7 days 28 days	 10 MPa 30 MPa 40 MPa
ASTM C 642	Maximum values of boiled absorption Maximum volume of permeable voids	9 % 18%
ASTM C 1550	Minimum joules energy at 40mm deflection in round panel test	*

Plastic Shotcrete QC Testing

Test Method	Test Procedure	Testing Frequency
CSA A23.2–5C	Slump at discharge into remixer unit	Each batch of shotcrete produced
CSA A23.2–5C	Slump at discharge into shotcrete pump	At beginning of each shift
	Shotcrete and ambient temperatures at discharge into remixer unit and shotcrete pump	Each batch of shotcrete produced
CSA A23.2–7C	Air content of plastic shotcrete at discharge into remixer unit and shotcrete pump	At start of project and with any change in mixture proportions
CSA A23.2–7C Modified	Air content as-shot (without accelerator)	At start of project and with any change in mixture proportions
CSA A23.2–7C Modified	Wash-out test to determine uniformity of fibre distribution at discharge from remixer unit	At start of project and with any change in fibre type or dosage

Hardened Shotcrete QC Testing

Test Method	Test Procedure	Testing Frequency
Ref . 1	Early age compressive strength of beams as equivalent cubes	At start of project and with any change in mixture proportions or accelerator type or dosage
Penetrometer	Early age compressive strength estimate using penetrometer	One heading per shift
ASTM C1140 ASTM C1604	Compressive strength of cores extracted from shotcrete test panel at age 1 day (1 core), 7 days (1 core) and 28 days (2 cores)	One heading per week for each shotcrete robot and each shotcrete crew used
ASTM C642	Boiled absorption and volume of permeable voids	One heading per week for each shotcrete robot used
ASTM C1550	Round panel test to 40mm deflection	Two panels at each of ages 7 and 28 days at start of project for one fibre type and dosage
ASTM C1550	Round panel test to 40mm deflection	Two panels at 28 days for each week of shotcrete production or at any change in fibre type or dosage

Equipments

Numerous equipments are utilized in shotcreteing a brief over view

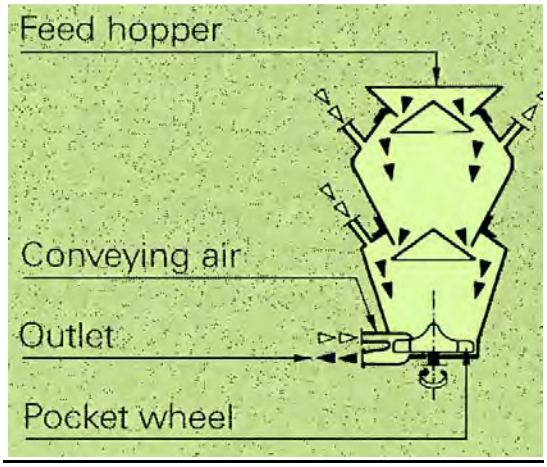
- Shotcrete guns,
- Nozzles
- Three different types of dry mix shotcrete guns
- All of which work on the suspension-conveying principle.

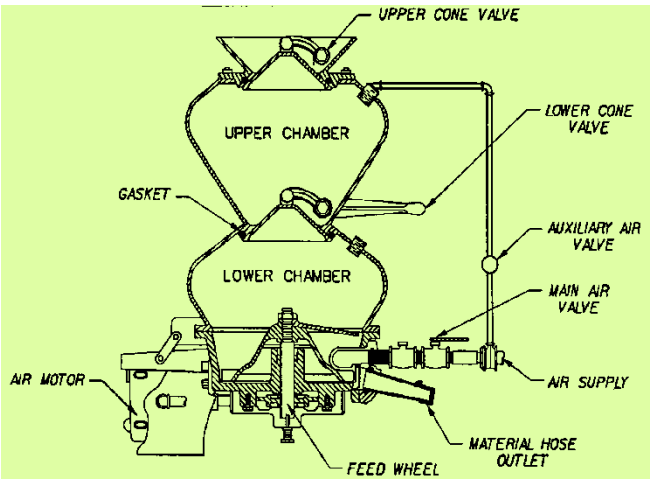
In order of their invention, they are

- Double-chamber system;
- Screw system;
- Rotating-barrel system

Operating principle of the double-chamber gun

Employs two connected chambers arranged one above the other, with the discharge outlet at the bottom of the lower one. Feed opening of the upper chamber and the connection between the two chambers can be closed off hermetically with bell-shaped valves independently of one another.



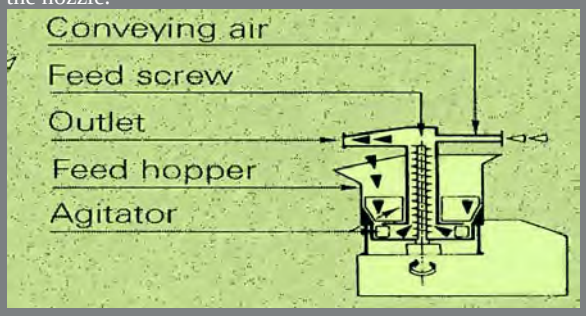


Double-chamber gun (Cement-Gun, 1914).

Dry mix being filled into the upper chamber with the bell valve between the two chambers closed. Feed opening is closed hermetically and the upper chamber is pressurized just like the lower one. The valve between the two chambers can be opened. Allowing the mixture to slide from the upper into the lower chamber. Reclosed and the pressure released in the upper chamber to permit reopening of the inlet.In the meantime, the dry mix is discharged from the lower chamber by a pneumatically driven feed wheel. The upper chamber is refilled at the same time, and the cycle is repeated. Because of the need to manipulate valves and levers alternately, the gunman needs considerable skill and muscle.

Operating principle of the screw-type gun

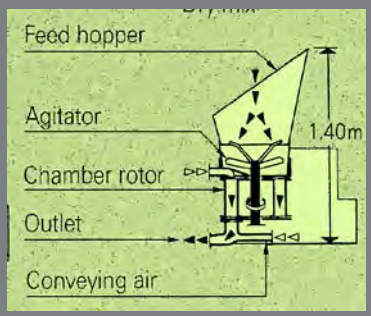
The screw-type gun was invented by Georg Senn and built by Spribag AG at Widen, Switzerland, in the early post-war years. The machine raised the dry mix from the open filling container with a screw in the form of an Archimedean screw. The screw lifted the mixture up to the blow chamber from which compressed air carried it to the nozzle.

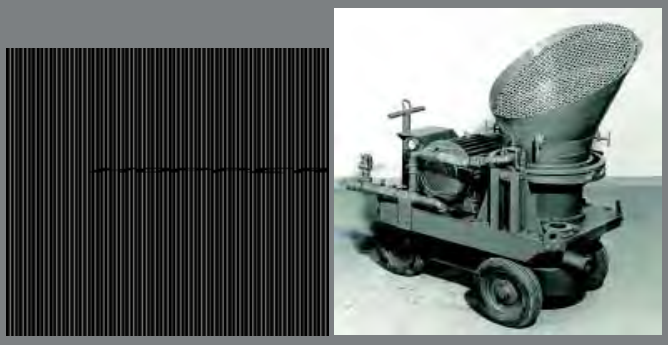


Screw-type gun (Spribag BS-12, around 1950)

Operating principle of the rotating-barrel gun

The rotating-barrel (rotor) system works on an entirely new type of principle. The dry mix passes from an open feed hopper into a rotor with vertical axis of rotation and from it into the discharge line. The rotor (or barrel) is equipped with vertical chambers of cylindrical or circularsector chambers. As the rotor revolves, each chamber in turn comes underneath a feed opening. The dry mix—which is kept moving in the hopper by an agitator—drops in. The filled rotor chambers proceed to the outlet opening on the opposite side where downward-flowing compressed air blasts them into the discharge line.





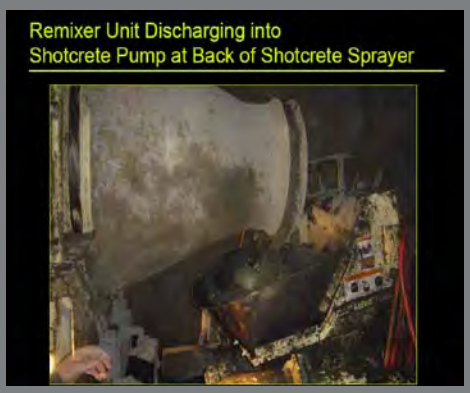
Rotating-barrel gun (Meyco GM 57, No. 1)

With the rotating-barrel system, the dry-mix process seems to have reached a progress plateau. A number of dry-mix guns are available on the international market, all of them essentially variations or combinations of the aforementioned systems.

Rotating-barrel gun (Meyco GM 090, 2002)

Rotating-barrel gun (Aliva 246.5, 2002)

More recently, special spraying devices have been developed exclusively for factory-produced mixtures. These are not machines as such but merely so-called metering or batching devices that feed the dry mix from a pressure container into the air stream of the spraying hose. The conveying rate is infinitely variable between 1.5 and 12 m^3/h. These devices exhibit relatively low wear-and-tear-related costs.

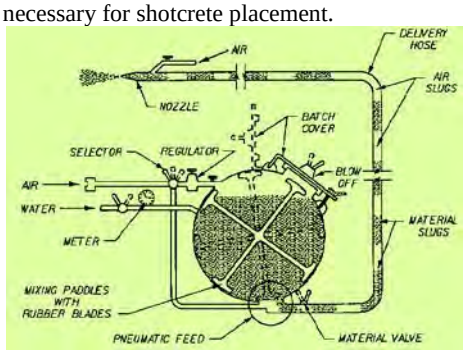


Description of guns for wet mixing. Two types of guns are available foe wet mix

- Pneumatic feed gun for wet mix,
- Positive displacement gun for wet mix

Operating principle of pneumatic feed gun for wet mix

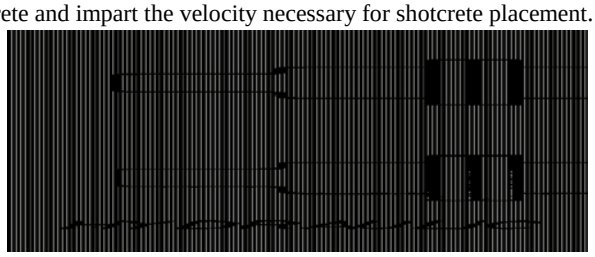
In the pneumatic-feed equipment, the premixed mortar or concrete is conveyed from the gun through the delivery hose to the nozzle by slugs of compressed air. At the nozzle additional air may be added if needed to increase the velocity and improve the gunning pattern. This equipment can handle mixtures of a consistency suitable for general shotcrete construction, using mixtures containing up to 3/4-inch aggregate. Guns with a dual mixing chamber and a two-way valve allow mixing of materials and a continuous flow operation. In the positive displacement equipment, the concrete is pumped or otherwise forced through the delivery hose without the use of compressed air. Air is injected at the nozzle to disperse the stream of concrete and impart the velocity necessary for shotcrete placement.



Cross section of pneumatic-feed shotcrete gun (Hoffmeyer 1966)

Operating principle of positive displacement gun for wet mix

In the positive displacement equipment, the concrete is pumped or otherwise forced through the delivery hose without the use of compressed air. Air is injected at the nozzle to disperse the stream of concrete and impart the velocity necessary for shotcrete placement.



Schematic of positive displacement pump (Fredricks, Saunders, and Broadfoot 1966)

Positive displacement delivery equipment requires a wetter mixture than pneumatic-feed equipment, and the velocity of the shotcrete being applied is lower. It is difficult to apply shotcrete to vertical and overhead surfaces by this method unless a suitable accelerator is used. This equipment can also satisfactorily shoot material containing 3/4-inch aggregate.

Universal Spraying Machine (ALIVA 237)

Description

The Aliva-237 is a sturdy concrete spraying machine for wet and dry spraying process. Thanks to its variable output, the Aliva-237 can be used for small jobs, such as joint filling, as well as for extensive slope consolidation work.

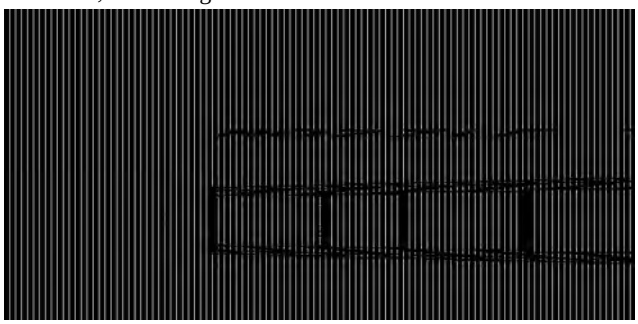
The Aliva-237 is available in the following versions:

- STANDARD - Electric drive, single speed
- TOP - Electric drive, with FC for variable rotor speed
- AIR - With air drive, variable rotor speed

Universal Spraying Machine (ALIVA 237)

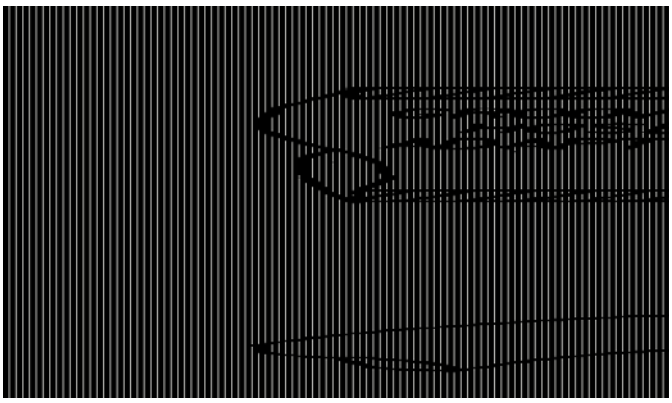
Nozzles

A **dry-mix nozzle** typically consists of a tip, water ring, control valve, and nozzle body arranged in a wide variety of nozzle tips, nozzle sizes, and configurations.



Typical dry-mix nozzle

A **wet-mix nozzle** usually consists of a rubber nozzle tip, an air injection ring, a control valve, and nozzle body.



Typical wet-mix nozzle

Some investigations have shown improved mixing action and less rebound for dry-mix shotcrete when a special pre-wetting nozzle is used and the water ring is placed in the hose 1 to 8 feet before of the nozzle. This has been particularly effective for silicafume shotcrete.

Remote-controlled nozzles

During recent years, the use of remote-controlled nozzles has become increasingly popular, particularly for underground work. These machines are truck-mounted and include a boom-mounted nozzle, a gun, and an air compressor. The remote controls allow the nozzleman to rotate the nozzle in an 18 inch diameter circle to allow proper application technique. The nozzleman can also swing the nozzle around 360 degrees and manoeuvre it closer to or farther from the surface being shot. Significant economy is realized because of higher placement rates and reduced crew size. Because of the remote location of the operator, some safety benefits can be realized from avoiding rebound of aggregates and fibres.

Mini concrete spraying system AL-503

Mini concrete spraying system AL-503 This system is used for mining and small tunnels. It is Modular concept system which can be air, electric, diesel driven. Very flexible concrete spraying. The unit consits of the telescope spraying arm AL-302 mounted on a tracked carrier

Trailer Mounted Concrete Pumps

Wet-mix shotcrete pump specifications

Manufacturer Model	Output capacity (cyh)	Material cylinders (dia x length, in.)	Outlet diameter (in.)	Maximum pressure (psi)
AIRPLACO				
Cobra 30	30	5x30	5	1000
Cobra 40	40	6x30	6	850
ALLENTOWN EQUIPMENT				
Powercreter 10	10	3x24	3	1330
Powercreter 20	20	4½x30	4	1330
Powercreter Pro	5	3x16	3	1330
BLASTCRETE EQUIPMENT CO.				
Pumpmor MX-10	12	3x24	3	2200
Pumpmor X-10	12	3x24	3	2200
MAYCO PUMP				
ST-45SD	20	4½x27	4	1467
MORGEN MFG. CO.				
Mustang 25	25	5x30	5	750
Mustang 30	30	5x30	5	750
OLINPUMP				
10-9	9	5x12	3	1500
10-22	22	5x24	3	1500
10-35	35	5x36	3	1700
10-36	36	5x36	3	2100
PUTZMEISTER AMERICA				
TS 2030	30	6x30	5	1150
TS 2040	40	6x39	5	950
HP 2015	15	4x39	4	1550
HP 2020	17	4x30	4	1300
HP 2030	22	5x30	5	1300
REED MFG.				
M5 Multispray	5	3x23	3	1345
REINERT MFG. CO.				
P-530	30	5x30	4	675
P-630	40	6x30	5	800
SCHWING AMERICA INC.				
301	35	7x20	5	1012
BPA 450-10	20	4x39	4	450
BPA 450	30	6x39	5	1100
WP 1000 X HP	35	6x39	4	1450
TRANSCRETE AMERICA INC.				
P-35 S/C	35	5½x36	4	1500
P-40	40	6x36	5	1200
WHITEMAN CONSPRAY				
SV 5D	6.8	3x22	3	1350
SV 20D	24	4x38	4	2200

Shotcrete machine used at Sangaldan

Robotic Arm Shotcrete machine used in Tunnel T-1 at Udhampur

Conclusions

Production of quality shotcrete in an underground environment requires:

- ✓ Rigorous shotcrete specifications.
- ✓ Implementation of a quality management plan.
- ✓ Pre-production qualification of all materials and shotcrete mixtures design.
- ✓ Implementation of a systematic quality control inspection and testing plan for the aggregates, mix water and plastic & hardened shotcrete.
- ✓ Use of nozzlemen and shotcrete batching, supply and application crew properly trained and qualified.

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SOME OF THE LONGEST RAILWAY TUNNELS OF WORLD

The Gotthard Base Tunnel, Swiss Alps

Status: world's **longest** Rail Tunnel (when completed in June 2016).

Features:

- The tunnel project is a part of Alps transit project located in Swiss Alps known as New Railway link through Alps (NRLA).
- Consists of two single track tunnels, *length of the east tunnel* is 57.1 Km and that of west tunnel is 57.01 Km.
- The tunnels are joined approximately every 325 m by connecting galleries.
- The tunnels starts from Erstfeld (Uri) to Bodio (Ticino).
- The work of the tunnel project began in 1996 and planned to be opened in June-2016.
- Main purpose of the project is to increase the total freight transport capacity notably between Germany and Italy.
- Diameter of each of the single-track tubes: 8.83–9.58 m.
- Total cost (as of October 2010) (US$10.1 billion).
- Alp Transit Gotthard Ltd. was planned to hand over the tunnel to Swiss Federal Railways (SBB) in operating condition in December 2016.

Y junction at multifunction station Faido of Gotthard base tunnel.

Hussain Khan
XEN/D-II/USBRL/JAT

Construction:

AlpTransit Gotthard AG is responsible for construction. It is a wholly owned subsidiary of the Swiss Federal Railways (SBB-CFF-FFS).

To cut construction time in half, four access tunnels were built so that construction could start at four (a fifth was added later) different sites simultaneously (Erstfeld, Amsteg, Sedrun, Faido and Bodio).

The TBM from Bodio arrived at MFS Faido in Sept. 2006

Four tunnel Boring machines were used, two machines operated northbound from Bodio to Faido and Sedrun and were nicknamed Sissi and Heidi respectively and other two operated southbound from Erstfeld to Sedrun and were known as Gabi I and Gabi II.

The final breakthrough in the east tube occurred on Friday, 15 October 2010.The final breakthrough in the west tube occurred on Wednesday 23 March 2011 at 12:20.

The 1st TBM from Bodio broke through on 6 September 2006, MFS Faido

On 16 December 2013, the operational test phase started on a 13 kilometer stretch in the southern section of the west tube between Faido and Bodio. Its purpose is to test the infrastructure and any ancillary systems.

SOME OF THE LONGEST RAILWAY TUNNELS OF WORLD

The Seikan Tunnel, Japan

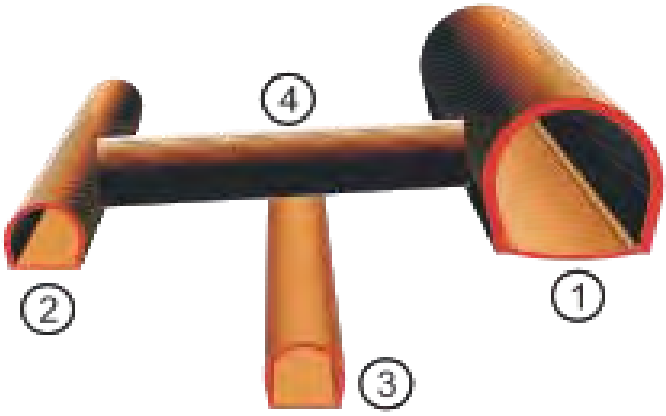
Status: world's second **longest** and deepest Rail Tunnel.

Features:

- Length of the tunnel is 53.85 Km.
- Passes through Tsugaru Strait connecting Hokkaido and Honshu Island Japan.
- Built 100 m below the sea bed and about 240 m below sea level, length include a 23.3 Km submarine portion.
- Internal dimensions: Clear height 7.85 m and width 9.7 m.
- Double track rail tunnel, mixed gauge and electrified.
- Officially opened in March 1988 for operation of express trains.
- Two stations namely TappiKaitei and Yoshioka Kaitei are located inside the tunnel and they serve as emergency escape points.

Project sponsored by Japan Railway Construction, Transport and technology agency and operated by JR Hokkaido.

Map of Seikan tunnel



1. Main tunnel. 2. Service tunnel. 3. Pilot tunnel.
4. Connecting gallery.

Geology:

The geology of the undersea portion of the tunnel consists of volcanic rock, pyroclastic rock and sedimentary rock of the Neogene period. Igneous intrusion and fault caused crushing of the rock and complicated the tunneling procedures.

Surveys and investigations:

Initial geological investigation involved drilling the seabed, sonic surveys, submarine boring and seismic and magnetic surveys. To establish greater understanding a horizontal pilot boring was undertaken along the line of service and main tunnel.

Train approaching Tappi-Kaitei Station

Construction:

Tunneling occurred simultaneously from the northern end and southern. The dry land portion were tackled with traditional mountain tunneling techniques with single main tunnel. However for the 23.3 Km undersea portion, three bores were excavated with increasing diameters respectively: an initial pilot tunnel, a service tunnel and finally the main tunnel. Beneath the Tsugaru strait, the use of TBM was abandoned after less than 2 Km owing to the variable nature of rock and difficulty in accessing the face for advanced grouting. Blasting with dynamite and mechanical picking were used to excavate.

The Channel Tunnel, (English channel strait of Dover),UK

Status: world's**Third longest** Railway tunnel (World longestundersea Rail Tunnel).

Features:

- Length of the tunnel is 50.5 Km.
- Connects Folkestone in Kent UK to Coquelles in Pas-de-Calais France.
- Two terminals are linked by three tunnels including a 9. 3 Km underground tunnel in UK, 38 Km under sea tunnel and 3.2 km underground tunnel in France.
- Tunnel consists of two 7.6 m diameter rail tunnels and a 4.8 m diameter service tunnel in between.
- Two single track rail tunnel and one service tunnel, standard gauge and electrified.
- Opened on 14 November 1994 for passenger service.
- Cost of construction: $ 14.7 bn and took six years for completion.
- Project owned by Eurotunnel and operated by Eurotunnel, Eurostar and DB Schenker Rail (UK).

Geology:

The geology of this site generally consists of northeasterly dipping Cretaceous strata. Some main characteristics are as follows:

- Continuous chalk on the cliffs on either side of the Channel containing no major faulting.
- Four geological strata, marine sediments laid down 90–100 million years ago; pervious upper and middle chalk above slightly pervious lower chalk and finally impermeable Gault Clay. A sandy stratum, glauconitic marl (tortia), is in between the chalk marl and gault clay.
- A 25–30-metre (82–98 ft) layer of chalk marl (French: craiebleue) in the lower third of the lower chalk appeared to present the best tunnelling medium.

SOME OF THE LONGEST RAILWAY TUNNELS OF WORLD

Map of the Channel Tunnel

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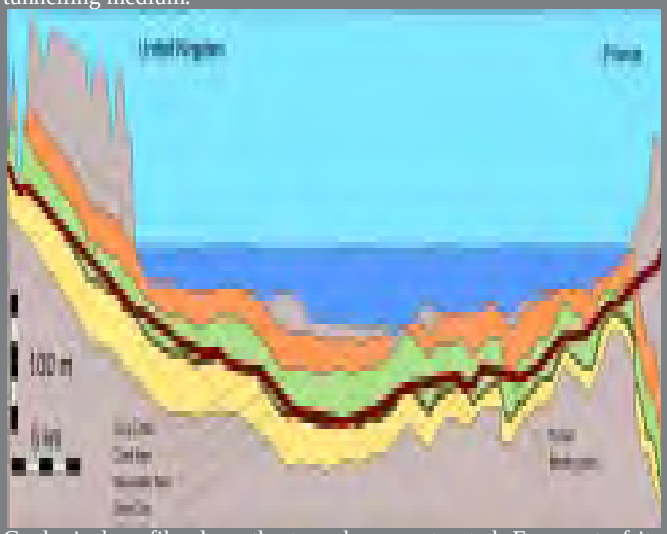


Geological profile along the tunnel as constructed. For most of its length the tunnel bores through a chalk marl stratum.

Construction:

Tunneling was a major engineering challenge, with the only precedent being the undersea Seikan Tunnel in Japan. Working from both the English side and the French side of the Channel, eleven tunnel boring machines or TBMs cut through chalk marl to construct two rail tunnels and a service tunnel. Precast segmental linings in the main TBM drives were used, but two different solutions were used. On the French side, neoprene and grout sealed bolted linings made of cast iron or high-strength reinforced concrete were used; on the English side, the main requirement was for speed so bolting of cast-iron lining segments was only carried out in areas of poor geology. On the English side, a marshalling area was 140 m below the top of Shakespeare Cliff, the New Austrian Tunnelling method (NATM) was first applied in the chalk marl here.

A Tunnel Boring Machine

On the French side, owing to the greater permeability to water, earth pressure balance TBMs with open and closed modes were used. The TBMs were of a closed nature during the initial 5 km;but then operated as open, boring through the chalk marl stratum. The French effort required five TBMs: two main marine machines, one main land machine (the short land drives of 3 km allowed one TBM to complete the first drive then reverse direction and complete the other), and two service tunnel machines. On the English side, the simpler geology allowed faster open-faced TBMs. Six machines were used, all commenced digging from Shakespeare Cliff, three marine-bound and three for the land tunnels. Towards the completion of the undersea drives, the UK TBMs were driven steeply downwards and buried clear of the tunnel. These buried TBMs were then used to provide an electrical earth. The French TBMs then completed the tunnel and were dismantled.

The Channel Tunnel exhibit at the National Railway Museum in York, England

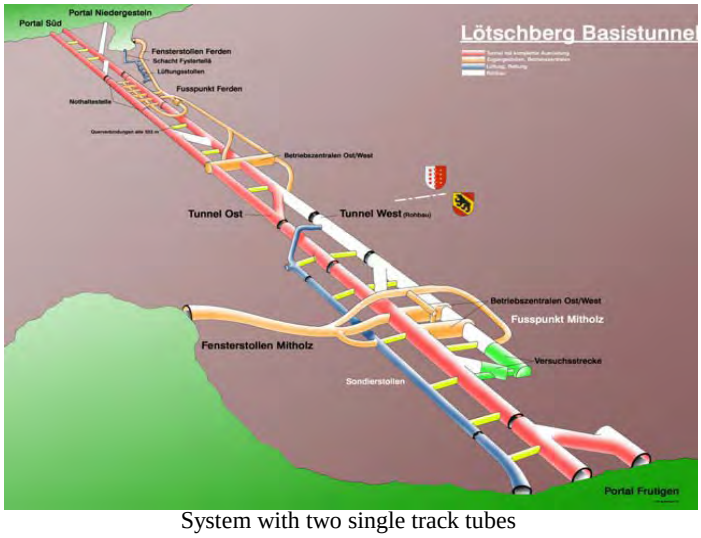
The LÖtschberg base Tunnel,Swizerland.

Status: world's**fourth longes**t Rail Tunnel.

Features:

- Length of the tunnel is 34.6 Km and is currently world's longest land rail tunnel (except for some metro tunnels)
- Built through the Alps in Switzerland below the existing LÖtschberg tunnel.
- Tunnel was designed with twin single track tubes which are interconnected by transverse tunnels at 333 m intervals.
- Tunnel consists of two 7.6 m diameter rail tunnels and a 4.8 m diameter service tunnel in between.
- Two single track rail tunnel and one service tunnel, standard gauge and electrified.

SOME OF THE LONGEST RAILWAY TUNNELS OF WORLD



System with two single track tubes

- Opened on 14 November 1994 for passenger service.
- Cost of construction: $ 14.7 bn and took six years for completion.
- Project owned by Eurotunnel and operated by Eurotunnel, Eurostar and DB Schenker Rail (UK).
- The northern portal at Tellenfeld near Frutigen is located at a height of 780 m, and the southern portal at Raron,at 660m.
- The tunnel rises by 0.3 % until it reaches its apex in the mountains between the cantons of Berne and Valaisand drops by roughly 1.1 % at its southern end.

From the south, two tunnel-boring machines (TBM's) were used through the rock: for the Steg lateraladit/base tunnel and for the first 10 km of the eastern tube from the south portal at Raron. In all the other sections, the traditional method of blasting were used.

The Guadarrama (Tünel de Guadarrama) Tunnel, Spain

Status: world's**fifth longest** Rail Tunnel (longest twin tube tunnel in Spain).

Features:

- Length of the tunnel is 28.4 Km.
- Built through the Sierra de Guadarrama mountain range.
  Tunnel is built between Madrid and Valladolid as twin tubes which are interconnected by transverse tunnels at 250 m intervals

- Tunnel consists of two 8.5 m diameter rail tunnels.
- The tunnel was built by Herrenknecht and Wirth using TBM.

Opened in December 2007 for high speed train service

The Taihang Tunnel, China

Status: world's**Sixth longest Rail Tunnel**(longest mountain Railway tunnel in china).

Features:

- Length of the tunnel is **27.8 Km.**
- Built through the Taihangmountain.
- Tunnel is built as a part of Shijazhuang-Taiyuan Railway project to cross Taihang mountains
- Tunnel is built as twin tubes which are 35 m apart.
- Tunnel was designed by PÖyry..
- Tunnel project was completed on 22th December 2007.

The **Hakkoda Tunnel,Japan.**

SOME OF THE LONGEST RAILWAY TUNNELS OF WORLD

Status: world's**7th longest**Rail Tunnel (longest land base double track railway tunnel in the world).

Features:

- Length of the tunnel is **26.45 Km.**
- Located in Northern end of Hakkoda mountain.
- Built between Tenmabayashi-mura& Aomori city in Aomori prefecture in Japan.
- Tunnel is built as a part of Tohoku Shinkansen Railway.
- Tunnel was sponsored by Japan Railway Construction, Transport and technology Agency (JRTTA).

Tunnel project was completed in Feb-2005 using NATM and began to be used in 2010.

1.0 1. INTRODUCTION

Rock being highly discontinuous material makes it extremely difficult to assess its engineering behavior. To quantify its ability to take load all varied aspect of rock mass needs to be studied. Therefore considering entire rock mass is imperative. The genesis of rock mass is more than 120 years old since Ritter, 1879 developed an empirical approach for tunnel design and till date a number of rock mass classification systems are available.

i. Terzaghi's rock mass classification based on geology of rock mass.
ii. Lauffer rock mass classification is based on stand up time of unsupported span, has been modified several time now forms part of NATM (New Austrian Tunneling Method).
iii. Rock quality Designation (RQD), developed by Deere provide quantitative estimate of rock mass based core extracted. Many empirical co-relations are available taking RQD as input parameter. Palmstrom (1982) suggested $RQD = 115 - 3.3\,J_v$, J_v is the sum of number of joints per unit length for all joint /discontinuity in a rock mass. Palmstrom's co-relations are useful where no core is available. RQD is direction dependent parameter and its value may change considerable depending on the direction chosen. Using J_v counters its directional dependence to some extent. RQD was also co-related with Terzaghi's rock load factor so as to estimate tunnel support requirement. RQD is also used as one of the input parameter for rock mass classification in both RMR and Q-system of classification that will be discussed at length in coming paragraphs.

Table 1: RQD- Values and volumetric counting

1	RQD (Rock Quality Designation)		RQD
A	Very poor	(> 27 Joints per m3)	0-25
B	Poor	(20-27 Joints per m3)	25-50
C	Fair	(13-19 Joints per m3)	50-75
D	Good	(8-12 Joints per m3)	75-90
E	Excellent	(0-7 Joints per m3)	90-100
Note: i) where RQD is reported or measured as ≤ 10 (including 0) the value 10 is used to evaluate the Q-value. ii) RQD- intervals of 5 i.e. 100, 95, 90, etc., are sufficiently accurate.			

Shailendra Kumar
Sec/CAO/USBRL

2.0 Q-SYSTEM

Q-System was proposed by Barton et al (1974) of Norwegian Geotechnical Institute.

2.1 Barton et al has defined index Q as

$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$, where

RQD : Rock quality designation

J_n : Number of joint sets

J_r : Joint roughness number

J_a : Joint alteration number

J_w : Joint water reduction factor and SRF : Stress reduction factor.

RQD/J_n, represent degree of jointing or block size. J_r/J_a, measures inter block friction and J_w/SRF , measure active stress.

2.1 Joint Set Number (J_n)

Joints within a given joint set will be nearly parallel to one another and display a characteristic joint spacing. Joints that do not occur systemically or have spacing of several meters are random joints. If poles are plotted in stereographic projection planes of a particular joint set will form a cluster. Total number of such clusters are join set number if these joints are contributing to failure. Table 2 provides value correspond to given joint set.

	Joints set number	J_n
A	Massive, no or few joints	0.5-1.0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint set	4
E	Two joint set plus random joints	6
F	Three joint set	9
G	Three joint set plus random joints	12
H	Four or more joint set, random heavily jointed "sugar cube" etc.	15
J	Crushed rock, earth like	20
Note : i) For tunnel intersections, use 3x J_n ii) For portals, use 2 x J_n		

2.1 Joint Roughness Number (J_r)

This depends on the nature of surface in contact. Joint surface could be undulating, planar, rough or smooth. Refer table no. 2. The term rough, smooth and slickenside refer to small structures in a scale of cm or mm and can be evaluated by running finger along the joint wall. The large scale roughness is measured on m scale using a 1 m long ruler. The term stepped, undulating and planner are used for large scale roughness.

Table 3: Joint Roughness Number

Joint Roughess Number		J_r
a) Rock-wall contact, and b) Rock-wall contact before 10 cm of shear movement		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough, irregular, planar	1.5
F	Smooth, planar	1
G	Slickensided, planar	0.5
Note: i) Description refers to small scale features and intermediate scale features, in that order.		
c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1
Note: ii) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening) iii) J_r =0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction.		

2.4 Joint Alteration Number (J_a)

Frictional force between joint walls considerably reduces depending on the material sandwiched between them. Table 4 provide quantitative value assigned to J_a, which depends on thickness of filled material and degree of wall contact when sheared along the joint.

Table 4: Joint Alteration Number

Joints Alteration Number		ϕ_r approx.	J_a
a) Rock-wall contact (no minerals fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling. i.e., quartz or epidote.		0.75
B	Unaltered joint walls, surface staining only.	25-35°	1
C	Slightly altered joint walls, Non-softening mineral coatings; sandy particles, clay-free disintegrated rock, etc.	25-30°	2
D	Silty or sandy clay coatings, small clay fraction (non-softening)	20-25°	3
E	Softening or low friction clay mineral coatings, i.e., Kaolinite or mica. Also chlorite, talc gypsum, graphite, etc., and small quantities of swelling clays.	8-16°	4
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4
G	Strongly over-consolidated, on-softening, clay mineral fillings (continuous, but <5mm thickness).	16-24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5mm thickness).	12-16°	8
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5mm thickness). Value of J_a depends on percent of swelling clay-size particles.	6-12°	8-12
c) No rock-wall contact when sheared (thick mineral fillings)			
K	Zones or bands of disintegrated or crushed rock. Strongly over-consolidated	16-24°	6
L	Zones or bands of clay, disintegrated or crushed rock. Medium or low over-consolidation or softening fillings.	12-16°	8
M	Zones or bands of clay, disintegrated or crushed rock. Swelling clay, J_a depends on percent of swelling clay-size particles.	6-12°	8-12
N	Thick contiuous zpnes or bands of clay. Strongly over-consolidated.	12-16°	10
O	Thic, continuous zones or bands of clay. Medium to low over-consolidation.	12-16°	13
P	Thick, continuous zones or bands with clay, swelling clay,. J_a depends on percent of swelling clay-size particles.	6-12°	13-20

2.2 Joint Water Reduction Factor (J_w)

Presence of water in the joint reduce shear strength owing to reduction in normal stress. Value of J_w can be taken from table 5.

5 Joint Water Reduction Factor		J_w
A	Dry excavations or minor inflow (humid or a few)	1.0
B	Medium inflow, occasional outwash of joint fillings (many drips/"rain")	0.66
C	Jet inflow or high pressure in competent rock with unfilled joints	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings.	0.33
E	Exceptionally high inflow or water pressure decaying with time. Causes outwash of material and perhaps cave in	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay. Causes outwash of material and perhaps cave in	0.1-0.05
Note: i) Factors C to F are crude estimates. Increase J_w if the rock is drained or grouting is carried out. ii) Special problems caused by ice formation are not considered.		

2.3 Stress Reduction Factor (SRF)

Stress reduction factor depends on stress acting in the rock mass and actual strength of rock. Acting stress may be taken as major principal stress (σ_1) and rock strength may be taken as uniaxial compressive strength (σ_c). Alternative SRF has also been co-related with maximum tangential stress (σ_θ) and σ_c. In a way ratio σ_1/σ_c and σ_θ/σ_cdescribes competency of rock around opening. Table 6 describe the value for SRF under different conditions.

Table 2: Stress Reduction Factor Values

Stress Reduction Factor		SRF
a. Weak zones intersecting the underground opening, which may cause loosening of rock mass		
A	Multiple occurrences of weak zones within a short section containing clay or chemically disintegrated, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth), For squeezing, see 6L and 6M	10
B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)	7.5
C	Single weak zones with or without clay or chemical disintegrated rock (depth≤ 50m)	5
D	Loose, open joints, heavily jointed or "sugar cube", etc, (any depth)	5
E	Single weak zones with or without clay or chemical disintegrated rock (depth > 50m)	2.5
Note: i) Reduce these values of SRF by 25-50% if the weak zones only influence but do not intersect the underground opening		

	b. Competent, mainly massive rock, stress problems	σ_c / σ_1	σ_θ / σ_c	SRF
F	Low stress, near surface, open joints.	>200	<0.01	2.5
G	Medium stress, favorable stress condition	200-10	0.01-0.3	1
H	High stress, very tight structure, usually favorable to stability, May also be unfavorable to stability dependent on the orientation of stresses compared to jointing/weakness planes*	10-5	0.3-0.4	0.5-2 2-5*
I	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
J	Spalling or rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
K	Heavy rock burst and immediate dynamic deformation in massive rock	2	>1	200-400

Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$, reduce σ_c to 0.75 σ_c. When $\sigma_1 / \sigma_3 > 10$, reduce σ_c to 0.5 σ_c. Where σc = unconfined compression strength σ_1 and σ_3 are the major and minor principal stresses. And = maximum tangential stress (estimated from elastic theory)

iii) When the depth of the crown below the surface is less than the span; suggest SRF increase from 2.5 to 5 for such cases (see F)

	c. Squeezing rock; plastic deformation in incompetent rock under the influence of high pressure.	σ_θ / σ_c	SRF
L	Mild squeezing rock pressure	1-5	5-10
M	Heavy squeezing rock pressure	>5	10-20

Note: iv) Determination of squeezing rock conditions must be made according to relevant literature (i.e. Singh et al., 1992 and Bhasin and Grimstad, 1996)

	d. Swelling rock: chemical swelling activity depending on the presence of water	SRF
N	Mild swelling rock pressure	5-10
O	Heavy swelling rock pressure	10-15

2.5 Rock Support Requirements as per Q- System

Based on the importance and safety of structure factor of safety is incorporated in estimation of support requirement in the form of excavation support ratio (ESR), its value for different structures are summarized as per table 7.

Table 5: ESR Values

	Type of excavation	ESR
A	Temporary mine openings, etc.	ca, 3-5
B	Vertically shafts*; i) circular sections ii) rectangular/square section *Dependant of purpose. May be lower than given values.	ca, 2.5 ca, 2.0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks) water supply tunnels, pilot tunnels, drifts and headings for large openings.	1.6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1.3
E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defense chambers, portals, intersections, etc.	1.0
F	Underground nuclear power stations, railways stations, sports and public facilitates, factories, etc.	0.8
G	Very important caverns and underground openings with a long lifetime, =100 years, or without access for maintenance.	0.5

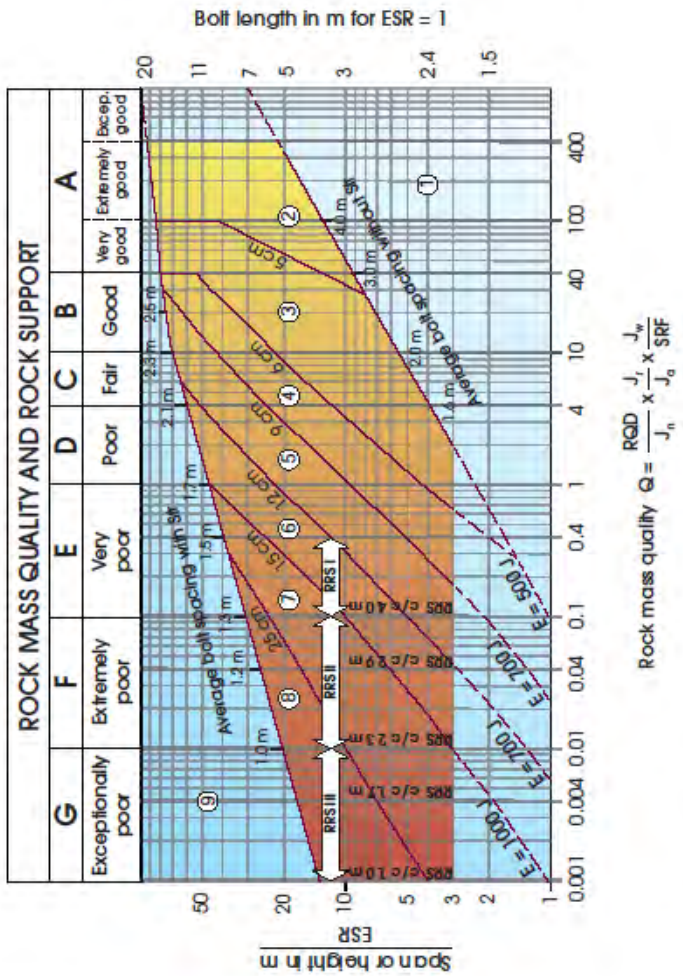


Figure 1: Rock Support Chart

Barton has defined equivalent dimension (D_e) as,

$$D_e = \frac{Span\ or\ height\ in\ m}{ESR}$$

Based on Q value and D_e by using chart shown in figure-1 primary rock support can be determined.As per figure-1 there are nine support class exist which are characterize as follows,

i. Unsupported or spot bolting
ii. Spot Bolting (SB)
iii. Fiber Reinforced Sprayed Concrete (5-6 CM), Bolting
iv. Fiber Reinforced Sprayed Concrete (6-9 CM), Bolting
v. Fiber Reinforced Sprayed Concrete (9-12 CM), Bolting
vi. Fiber Reinforced Sprayed Concrete (12-15 CM), Reinforced Ribs of Sprayed Concrete (RRS) and Bolting. RRS is similar to spread in lattice girder or steel arches with two differences, i). these structures are systematically bolted into tunnel arch and walls, ii) that can be non-circular, in order to penetrate onto eventual large over break thereby minimizing concrete
vii. Fiber Reinforced Sprayed Concrete > 15 CM),RRS and Bolting
viii. Fiber Reinforced Sprayed Concrete,RRS and Bolting
ix. Special Evaluation

Let's take an example for better illustration.Figure-2 presents a typical face mapping sheet from tunnel T-74R. There are three main discontinuities (joint set) and joint volume $J_v = 17$. So, $RQD = 115 - 3.3x17 = 58.9$, for three joint sets $J_n = 9$, Refer table-2. $\frac{RQD}{J_n} = \frac{58.9}{9} = 6.54$, Similarly referring table 3 for joint roughness number (J_r). For slightly rough joints with very small opening soft filling,

1.1 it can be concluded that rock wall contact would be ensured when sheared. $J_r = 1$, has been taken which corresponds to smooth and planer joint. Joint alteration number (J_a) has been taken as 4, refer table-4. Therefore, $\frac{J_r}{J_a} = \frac{1}{4} = 0.25$. Joint water reduction factor $J_w = 1$ as prevailing condition is dry. Unconfined compressive strength of rockσ_c =52 MPa (Estimated using Schmidt Hammer) and with overburden of 434 m vertical stress (assumed acting as maximum principal stress, σ_1) at this level would be $\sigma_1 = 25x434 = 10850\frac{KN}{m^2} = 10.85\ Mpa$. Assuming density of rock mass as$25\ KN/m^3$. Referring table-6 for$\frac{\sigma_c}{\sigma_1} = \frac{52}{10.85} = 4.8$, SRF value may be the interpolated between 0.5 and 2. This way $SRF = 2$, $\frac{J_w}{SRF} = \frac{1}{2}; Q = \frac{RQD}{J_n}.\frac{J_r}{J_a}.\frac{J_w}{SRF} Q = 6.54\ x\ 0.25\ x\ \frac{1}{2}; Q = 0.8$, For railway tunnel ESRmay be taken as 1 (refer table-7). For 10 m span, $D_e = \frac{Span}{ESR} = 10$.

Referring Baron's rock support chart in figure-1 for $Q = 0.8, \frac{Span}{ESR} = 10$, support category will be of class 5. Support may be adopted as 90-120 mm fiber reinforced shotcrete along with systematic bolting @ 2m. Barton et al has provided following information to calculate rock bolt length (L) in meter, maximum unsupported span and permanent roof support pressure (P_{roof}) in Mpa,

$$L = 2 + \frac{0.15B}{ESR}, \text{Where, } B \text{ is excavation width.}$$

$$Maximum\ unsuppoeted\ span\ (m) = 2.ESR.Q^{0.4}$$

$$P_{roof} = \frac{2.\sqrt{J_n}.Q^{\frac{1}{3}}}{3.J_r}$$

For above taken example L=3.15 m and Maximum unsupported span = 2 m.

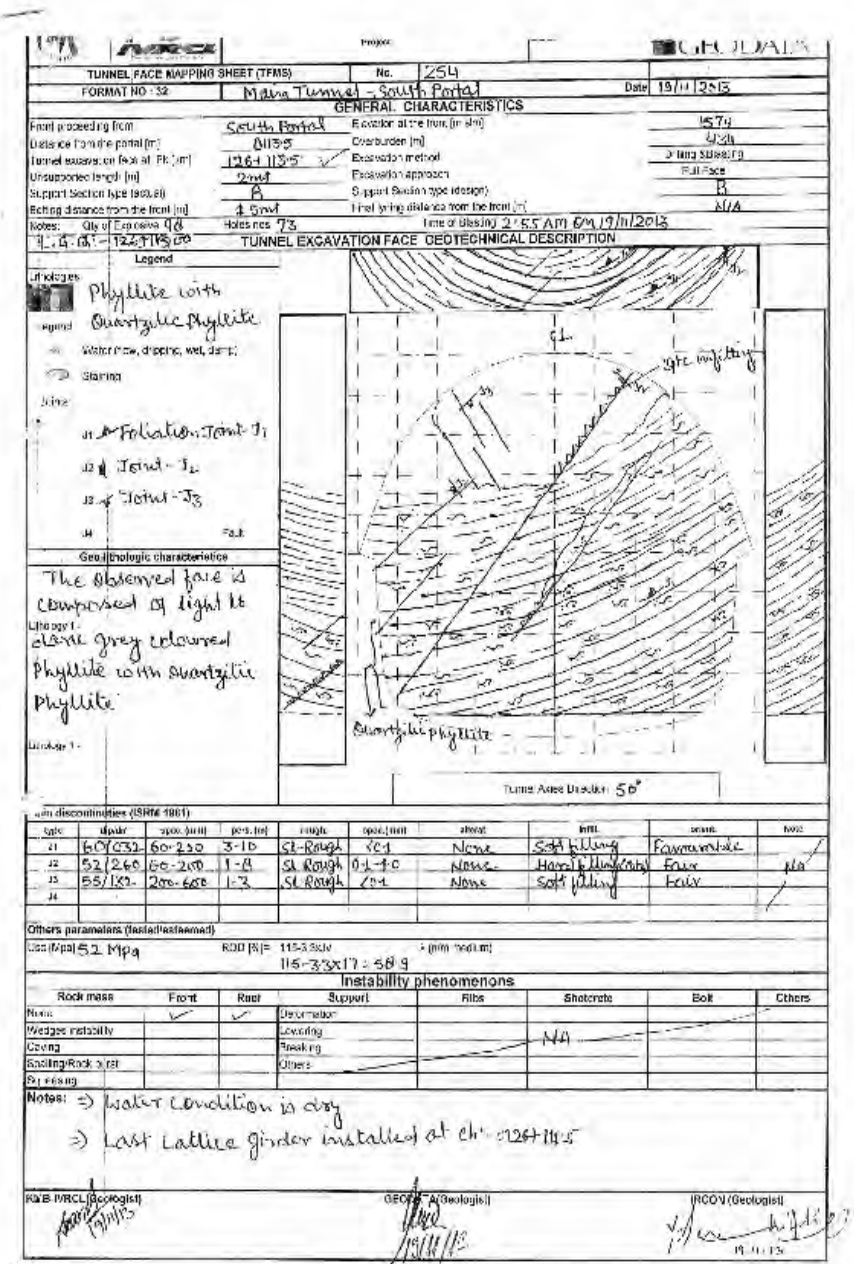

TUNNEL FACE MAPPING SHEET (TFMS)
FORMAT NO : 32
Mana Tunnel - South Portal
GENERAL CHARACTERISTICS
TUNNEL EXCAVATION FACE GEOTECHNICAL DESCRIPTION
Legend
Phyllite with Quartzitic Phyllite
Geologic characteristics
Tunnel Axis Direction: 50°
Instability phenomenons

Figure 2: A typical face mapping sheet or tunnel T-74R

3.0 ROCK MASS RATING (RMR) SYSTEM

Bieniawski in 1976 provided rock mass classification based on geomechanical classification rock mass, this is also called Rock Mass Ration (RMR) classification. Rock mass classification used in tunnel T-74R is based on RMR, therefore classification used in T-74R has been explained below which will cover RMR system.

4 ROCK CLASSIFICATION USED IN T-74R

4.1 Rock Mass Classification of Excavated section

Determination of section type depend on

a. Geological strength Index (GSI), Hoek et al, 1995
b. Intact Rock Strength
c. In situ stress
d. Rock Mass Rating, RMR, (Bieniawski, 1973 and followings)
e. Rock Mass Behaviour Class (Figure-1)

The result of geotechnical measurement and monitoring during tunnel construction should be taken into account for prediction of deformation and for determination of section type to be applied.

The behaviour of rock in a newly exposed round is time dependent, it means that rock mass quality will deteriorate with the free span if no support is installed within a reasonable time. Accordingly the maximum round length should be adopted which can be supported within the required time and becomes a factor in rock classification.

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3.2 Geological Strength Index

Rock mass along tunnel T-74R has been defined in terms of Geological Strength Index (GSI) as mentioned below,

- GSI 1 > 65 (Undisturbed rock mass consisting of cubic blocks with few fresh or slightly weathered discontinuities) - Structure Bulky/Massive.
- GSI 2 = 45-65 (Partially disturbed rock mass with multi-faceted angular blocks formed by 4 or more joint sets. The joint surfaces are slightly/moderately weathered)- Structure Blocky/ Very Blocky.
- GSI 3 = 25-45 (Tectonic disturbed rock mass with many interesting joint sets; the joint surfaces are moderately/ highly weathered) - Structure Blocky-Seamy.
- GSI 4 < 25 (Poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces. Highly weathered joint surfaces with compact or soft fillings).

4.3 Intact Rock Strength

A. Uniaxial and/or Multi-axial Compression laboratory tests

B. Point Load Test

4.4 In Situ Stresses

$$K = \frac{Horizontal\ Stress}{Vertical\ Stress}$$

Generally in high overburden K >1 but here hydrostatic case i.e K=1 has been considered.

4.5 Rock Mass Rating (RMR)

RMR value is the summation of ratings given by the Bieniawski for the following parameters,

f. Uniaxial compressive strength, (taken using Schiemitd Hammer value)

a. Rock Quality Destination (RQD), Borehole analysis: As proposed and defined by Deere, RQD is percentage of intact rock piece of length more than 100mm. It may be calculated by using,

$RQD = 115 - 3.3J_v$, where J_v is joint volume

b. Spacing of discontinuities

c. Condition of discontinuities (alteration, roughness, filling…): This depends on,

- Discontinuity Length (Persistence)
- Separation (aperture or opening of joints)
- Roughness
- Infilling (Gouge)
- Alteration (Weathering)

d. Ground water conditions (Presence of water)

e. Orientation of discontinuities (Effect of discontinuity strike and dip orientation tunnelling or slope face)

Bieniawski has given some rating to the parameter as per the details shown in Table 8. A sample of RMR calculation taken from T-74R is shown in Table 9 for better understanding. Approach for determination of section type is as per the procedure illustrated by Figure 3 and Figure 4. These figures are used sequentially in clock wise direction starting from rock block volume. Since GSI has been estimated as explained above and taking the value of intact rock strength (using Schemitz hammer), by using second graph strength of rock mass can be find out.

3.1 Determination of class of excavated section

Using the value of rock mass strength and overburden, competency of rock mass can be calculated which is defined as ratio of rock mass strength (σ_{cm}) and tangential stress (σ_θ) on the excavation contour. For a simplified case of hydrostatic stress condition and for circular tunnel, $\sigma_\theta = 2YH$, where Y is density of rock mass and H is the overburden. Finally using competency of rock and RMR value excavation behaviour and section type can be established, also refer Figure 5.

A. CLASSFICATION PARAMETER AND THEIR RATINGS.									
Parameter			Range of values						
1	Strength of intact rock material	Point Load Strength Index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive		
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 Mpa	1-5 Mpa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality (RQD)		90%-100%	75%-90%	50%-75%	25%-50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuties		> 2 m	.6-2 m	200-600 mm	60-200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuites No separation Unweathered wall rock.	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1.5 mm Cotinuious	Soft gouge > 5 mm thick or Separation > 5 mm Continuious.		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10m tunnel length (1/m)	None	< 10	10 - 25	25-125	> 125		
		(Joint waterpress)/ (Major Principal)	0	< .1	0.1 - 0.2	0.2-0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)									
Strike and dip orientation			Very favourable	Favourable	Fair	Unfavourable	Very favourable		
Rating		Tunnels & mines	0	-2	-5	-10	-12		
		foundations	0	-2	-7	-15	-25		
		Slopes	0	-5	-25	-50			
C. Rock Mass Classes Determined from total ratings									
Rating			100-81	80-61	60-41	40-21	< 21		
Class number			I	II	III	IV	V		
Description			Very good rock	Good rock	Fair rock	Poor rok	Very poor rock		
D. MEANING OF ROCK CLASSES									
Class number			I	II	III	IV	V		
Average stand-ukp time			20 yrs for 15 m span	1 yrs for 10 m span	one week for 5 m span	10 hrs for 2.5 m span	30 min. for 1 m span		
Cohesion of rock mass (kPa)			> 400	300-400	200-300	100-200	< 100		
friction angle of rock mass (deg)			> 45	35-45	25-35	15-25	< 15		
E GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS									
Discontinuity length (Persistance) Rating			< 1 m 6	1-3 m 4	3-10m 2	10-20 m 1	> 20 m 0		
Separation (aperture) Rating			None 6	<0.1 mm 5	0.1-1.0 mm 4	1.5 mm 1	> 5 mm 1		
Roughness Rating			Very Rough 6	Rough 5	Slightly Rough 3	Smooth 1	Slickensided 0		
Infilling (Gouge) Rating			None 6	Hard filling < 5mm 4	Hard filling > 5mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0		
Weathering Rating			Unweathered 6	Slightly weathered 5	Moderately weatered 3	Highly weathered 1	Decomposed 0		
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING..									
Strike perpendicular to tunnel axis					Strike parallel to tunnel axis				
Drive with dip-Dip 45-90			Drive with dip-Dip 20-45		Dip 45-90		Dip 20-45		
Very favourable			Favourable		Very favourable		Fair		
Drive against di-Dip 45-90			Drive against dip-Dip 20-45		Dip 0-20 Irrespective of strike				
Fair			Unfavourable		Fair				

Table 6: Rock Mass Calculation used in tunnel T-74R

ROCK MASS AND SULPPORT SECTION					No. 127
FORM NO: 32		LOCATION Main Tunnel Katra End (North Portal) Chainage 132+500 m			Date 17.12.2013
CLASSIFICATION OF THE ROCK MASS BEHAVIOUR					
(Bieniawsky, 1989)					
Parameter		**Value**		**Notes**	**Rates**
Uniaxial compresive strength (Mpa)		**34**		Measured	4
RQD (%)		**55.6**		Measured	13
Spacing of discontinuities (mm)		**<60-200 mm**		Observed	8
		J1	J2	J3		Average
Discontinuities conditions	Discontinuity Length, Persistence (m)	1-8,	1-6,	1-5,	Observed	2
	Opening (mm)	<0.5	<0.5	<0.5	Observed	5
	Roughness	SR	SR	SR	Observed	3
	Infilling	Clay	Clay	Clay	Observed	2
	Alteration				Judgment	5
Water		Damp			Esteem	10
Orientation					Esteem	-5
					RMR =	44
					RMR' (RMR'=RMR based on dry condition hypotesis and without orientation parameter) =	54
					GSI (RMR'-5) =	49
					Design GSI Index =	45-65
Based on RMR as for HOEKETAL, 1995 (GSI = RMR - 5) using RMR based on dry conditions hypotesis and without correction for discontinuities' orientation.						

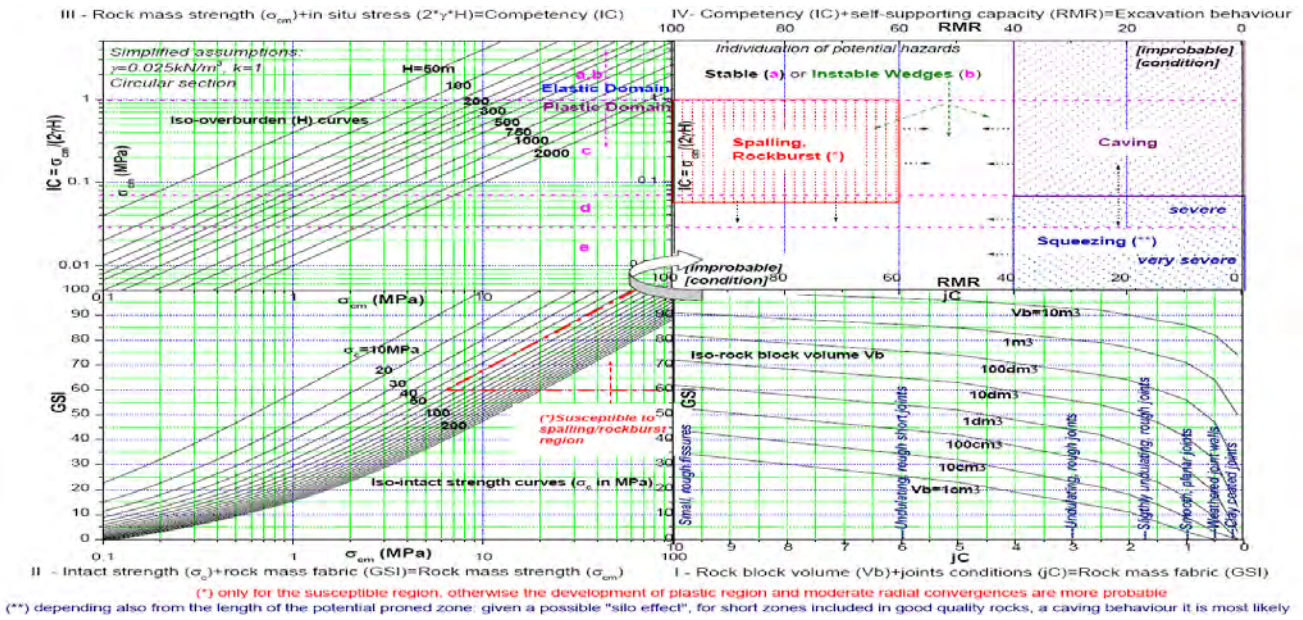


Figure 3: Approach for the evaluation of the excavation behaviour and typical potential hazards (Russo, 2008).

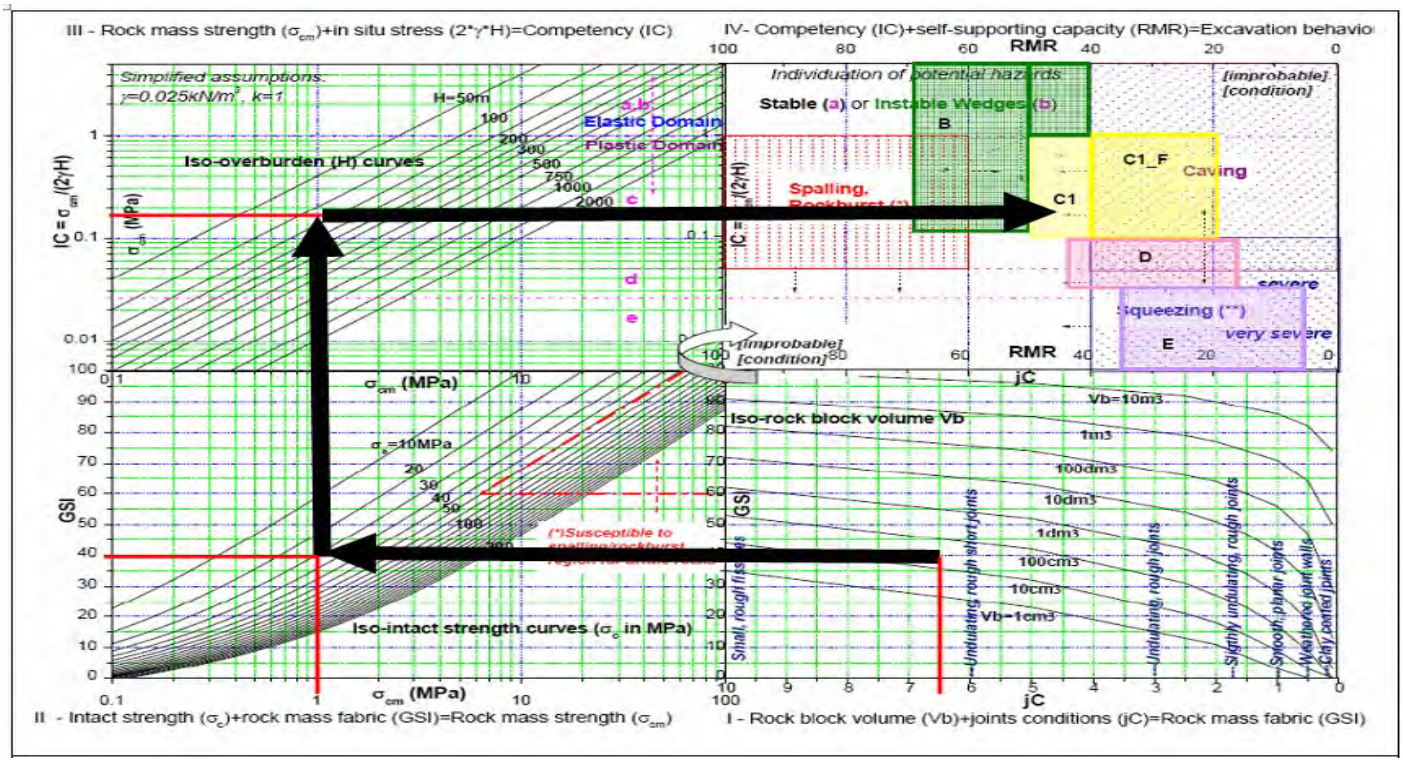


Figure 4: Example of Section Type (being applied during T-74R construction)
Reference parameters: Rock block volume [Vb] = 50 cm3; Joint Conditions [jC] = 6,5 ® Rock Mass Fabric [GSI] = 40. Intact
Strength [sc] = 1 MPa; ® Rock mass strength [cm2] = 30 MPa. In situ stress [sq] = 150 ® Competency [IC] = 0,2. Self-supporting, capacity [RMR] = 45 ® Excavation behaviour = "c". Section Type to be applied = C1.

↓ANALYSIS→		Geostructural →		Rock mass				
				Continuous ↔		Discontinuous ↔		Equivalent C.
Tensional↓				RMR				
Deformational response ↓	δ_0 (%)	Rp/Ro	Behavioural category ↓	I	II	III	IV	V
Elastic ($\sigma_\theta<\sigma_{cm}$)	negligible	-	a	STABLE				
			b		INSTABLE			CAVING
Elastic - Plastic ($\sigma_\theta\geq\sigma_{cm}$)	<0.5	1-2	c	SPALLING/ ROCKBURST	WEDGES			
	0.5-1.0	2-4	d					
	>1.0	>4	e					SQUEEZING
			(f)	→Immediate collapse of tunnel face↑				

Figure 5: Classification Scheme of Excavation Behaviour (Russo and Grasso, 2007 modified)

After classification of rock mass or section type, support requirements are assigned as per the details shown in Table 10.
For the example taken in section 1.1the GSI value calculated and shown in Table 11 is 56. For unconfined compressive strength of intact rock$\sigma_c = 52\ MPa$ and by referring to Figure 3, strength of rock mass comes out to be $\sigma_{cm} = 4\ MPa$. Competency of rock mass which is defined as $IC = \frac{\sigma_{cm}}{2.\gamma.H}$, where $\gamma = 25\ KN/m^3$. $IC = \frac{4x10^6}{2x25x10^3x434} = 0.18$, with these values of IC &σ_{cm}, it can be easily checked from Figure 3 that rock class fall near the boundary of elastic and plastic domain i.e. c and b class. Excavated section has been actually provided primary support as per classification of B class with 100 mm fiber reinforced shotcrete with rock bolts of 3 m length.

Table 7: Structural Requirement in Tunnel T-74 R

Structural Requirments of Tunnelling: T-74R (Main)

S/No.	Rock Class	C-Line (Radius in the crown) mm	Thichness of main lining (mm)	Nominal thichness of water proofing (mm)	Thichness of shotcrete (mm)	Convergence of the section (mm)	Tolerance (mm)	A-Line (Radius in the crown) mm	Excav. Type	Rock Bolt			Forepoling		Steel Ribs or Lattice Girder (25x20x20)mm bars used	Face Sealing fibre reinforced shotcrete (mm)	Round length (m)	Remarks
										Type	Length (m)	Nos	Type	Length				
1	A	3560	400	30	50	10	50	4100	Full Face	Swellex Mn12	3	4/m	-	-	-	-	3-5	
2	B	3560	400	30	100	10	50	4150	Full Face	Swellex Mn24	3	10/1 to 2m	-	-	-	-	2-3	
3	C1	3560	400	30	200	20	50	4260	Full Face	SDR	4	6	32 dia bar, 20 Nos	4	95-141mm Spacing =1 to 1.5m	-	1.25	
4	C2	3560	400	30	100	50	50	4240	Full Face	Cone Bolt+steel Mesh	1.5	12	-	-	-	50	1.5 - 2	Over Excavation of 50 mm is considered
	d>6m (Distance from face)				100					SDR	4	10						
5	D	3560	400	30	250	100	50	4390	Full Face	SDR	4	8	Fibre Glass Elements or Equivalent, if required		Steel Ribs TH44@ 1 to 1.5m	50	1.25	Over Excavation of 200 mm is considered. Lining stress controller (LSC) has to be placed between steel ribs.
													20-30 Nos	12				
5	E	3560	500	30	300	200	50	4640	Top Heading+ Benching	SDR	6	12	Fibre Glass Elements or Equivalent, if required		Steel Ribs TH44@ 0.8 to 1.2m	50-100	1	Over Excavation of 200 mm is considered. Lining stress controller (LSC) has to be placed between steel ribs. 200 mm shotcrete in temporary invert with double wire mesh.
													20-30 Nos	12				

1. In underground excavation invert has to be provided in rock class D and E
2. In undergroung excavation temporary invert at the level of top heading has to be provided in rock class E
3. If in rock class D, due to operational requirment top heading and benching is adopted than temporary invert at the level of top heading has to be provided the same way it has been provided in rock class E

Table 8: Determination of GSI for the mapping presented in Figure 2

CLASSIFICATION OF THE ROCK MASS BEHAVIOUR as per face mapping sheet presented in Figure 2 **(Bieniawsky, 1989)**						
Parameter		**Value**			**Notes**	**Rates**
Uniaxial compresive strength (Mpa)		**52**			Measured	7
RQD (%)		**58.9**			Measured	13
Spacing of discontinuities (mm)		**60-200**			Observed	8
		J1	J2	J3		Average
Discontinuities conditions	Persistence (m)	3-10	1-8,	1-3,	Observed	3
	Opening (mm)	<0.1	0.1-1	<0.1	Observed	5
	Roughness	SR	SR	SR	Observed	3
	Infilling	Clay	Clay	Clay	Observed	2
	Alteration				Judgment	5
Water		Dry			Esteem	15
Orientation					Esteem	-5
					RMR =	56
					RMR' (RMR'=RMR based on dry condition hypothesis and without orientation parameter) =	61
					GSI (RMR'-5) =	56
					Design GSI Index =	45-65
Based on RMR as for HOEKETAL, 1995 (GSI = RMR - 5) using RMR based on dry conditions hypothesis and without correction for discontinuities' orientation.						

3.3 References

Handbook on Using the Q-system, Rock mass classification and support design, NGI

NEW AUSTRALIAN TUNNELING METHOD

SYNOPSYS

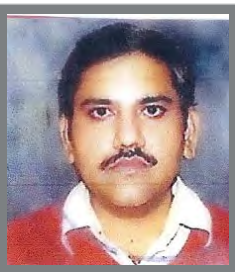
The Austrian experts who have contributed to the development of the NATM appear to agree that it is an approach or philosophy. NATM, its prime originator, Professor L von Rabcewicz says quite unequivocally, a new tunneling method-particularly adapted for unstable ground-has been developed which uses surface stabilization by a thin auxiliary shotcrete lining, suitably reinforced by rock bolting and closed as soon as possible by an invert. The method has been called The New Austrian Tunneling Method. The theme of the provision of support by shotcrete in combination with rock bolts, steel mesh, anchors and steel sets is repeated in later papers on the NATM.

The NATM approach has now been successfully used in a very wide, if not the widest, range of tunneling conditions. The versatility and adaptability so demonstrated derives from the basis soundness of the approach and from the adaptability of shotcrete as a primary and secondary support material, particularly when used in conjunction with a wide variety of supplementary support elements.

This paper explains various salient features of NATM.

INTRODUCTION

A **tunnel** is an underground or underwater passageway, dug through the surrounding soil/earth/rock and enclosed except for entrance and exit, commonly at each end. A tunnel may be for foot, vehicular road traffic, for rail traffic, or for a canal. Some tunnels are aqueducts to supply water for consumption or for hydroelectric stations or are sewers. Utility tunnels are used for routing steam, chilled water, electrical power or telecommunication cables, as well as connecting buildings for convenient passage of people and equipment. Secret tunnels are built for military purposes, or by civilians for smuggling of weapons, contraband, or people. Special tunnels, such as wildlife crossings, are built to allow wildlife to cross human-made barriers safely. Tunnels are essential part of infrastructure related to development of human race. Moreover they provide the benefit of connecting two points with the least possible distance thus bringing economy by reducing the route length and thus the cost of construction. Under some circumstances like crossing a developed city or town they may be the only option available due to lack of space in such cities. Tunnels are also preferred mode of developing infrastructure as they are practically less effected by seismic hazards, they provide uninterrupted movement of traffic as the passage is passage isi confined along the whole length, the only drawback is the uncontrollable damage, loss of life and difficulty in evacuation of passenger in the even of an catastrophe. Still with use fo modern technology, continuous monitoring, consideration of safety aspects during construction the tunnels provide the most economical and safe mode of transport. The world's longest tunnel carries water 105 mi (170 km) to New York City from the Delaware River. The lengthiest person-carrying tunnel is the Seikan Railroad Tunnel. It is a 33-mi (53-km) long, 32-ft (9.7-m) diameter railroad connection between Japan's two largest islands, Honshu and Hokkaido.



Sangrah Maurya
Dy. CE/Anji/USBRL

History of tunneling

Tunnels were hand-dug by several ancient civilizations in the Indian and Mediterranean regions. In addition to digging tools and copper rock saws, fire was sometimes used to heat a rock obstruction before dousing it with water to crack it apart. The cut-and-cover method—digging a deep trench, constructing a roof at an appropriate height within the trench, and covering the trench above the roof (a tunneling technique still employed today)—was used in Babylon 4,000 years ago.

It is probable that the first tunneling was done by prehistoric people seeking to enlarge their caves. InBabylonia, tunnels were used extensively for irrigation; and a brick-lined pedestrian passage some 3,000 feet (900 metres) long was built about 2180 to 2160 bc under the Euphrates River to connect the royal palace with the temple. Construction was accomplished by diverting the river during the dry season. The Egyptians developed techniques for cutting soft rocks with copper saws and hollow reed drills, both surrounded by an abrasive, a technique probably used first for quarrying stone blocks and later in excavating temple rooms inside rock cliffs. Abu Simbel Temple on the Nile, for instance, was built in sandstone about 1250 bc for Ramses II (in the 1960s it was cut apart and moved to higher ground for preservation before flooding from the Aswān High Dam). Even more elaborate temples were later excavated within solid rock in Ethiopia and India.

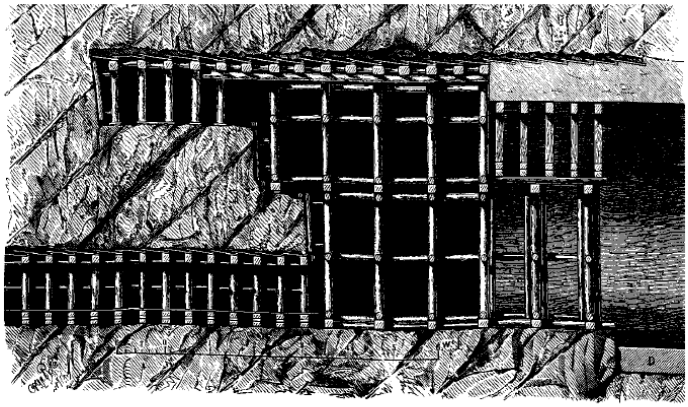
2180–2160 B.C.Babylonians dig a 3000-foot-long tunnel under the Euphrates River.

The first advance beyond hand-digging was the use of gunpowder to blast a 515-ft (160-m) long canal tunnel in France in 1681. The next two major advances came about 1850. Nitroglycerine (stabilized in the form of dynamite) replaced the less powerful black powder in tunnel blasting. Steam and compressed air were used to power drills to create holes for the explosive charges. This mechanization eventually replaced the manual process made famous by John Henry, the "steel-driving man," who swung a 10-lb (4.4-kg) sledge hammer with each hand for 12 hours a day, pounding steel chisels as deep as 14 ft (4.2 m) into solid rock.

More spectacular railroad tunnels were being started through theAlps. The first of these, the Mont Cenis Tunnel (also known as Fréjus), required 14 years (1857–71) to complete its 8.5-mile length. Its engineer, GermainSommeiller, introduced many pioneering techniques, including rail-mounted drill carriages, hydraulic ram air compressors

Excavation methods and support measures for the new tunnels being build were taken over from the practice followed for coal and ore mines. By tradition the support of the underground structures in mines was done with timber and stone packing. In the new railway and road tunnels the timber support was also used during the excavation. For the long-term stability the tunnel was fitted with thick and rigid linings.

NEW AUSTRALIAN TUNNELING METHOD

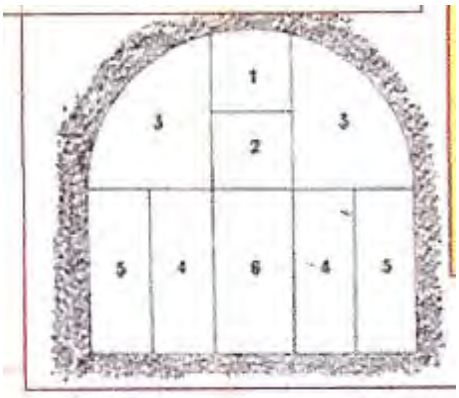


In the course of evolution many tunneling methods have evolved. Tunneling was almost done in every civilization. During the process of development of many methods have evolved, some are described in brief below.

BELGIUM METHOD:

The Belgium method was first employed in building the Chaleroy tunnel (in Belgium) in 1828. The great advantage claimed for the system by Belgian and rench engineers was the speed whereby the roof of the tunnel could be secured, a desirable advantage in poor rock.

The method fell out of favour as a result of catastrophic experiences encountered during the construction of the Gotthard Tunnel (1872-1882). The key problem was that the sequencing following Stage 3 required the arch to be underpinned. However, this proved difficult in the yielding ground conditions encountered, leading to the timbers giving way, followed by the crocking or total collapse of the masonry arch.

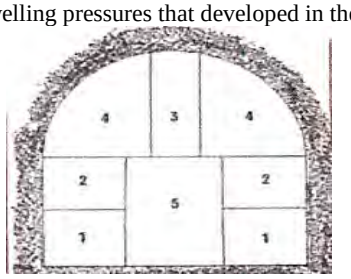


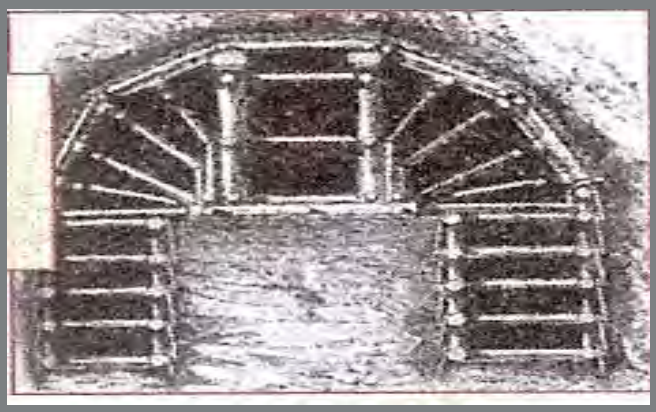
GERMAN SYSTEM:

The underlying principle of the "German System" was to leave a central bench of ground to be excavated last and to use it to support roof and wall timbering.

This allowed the arching to be built in one operation (unlike the Belgium method which had the disadvantage of building the arch and walls separately).

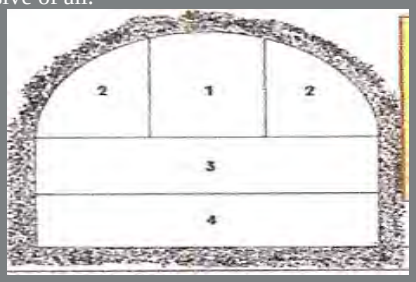
The German system came to a disastrous end when applied to the Czemitz tunnel in Austria (1866), where the timbers supporting the heading either pushed into the core, whereupon they became loose, or were crushed by swelling pressures that developed in the core.

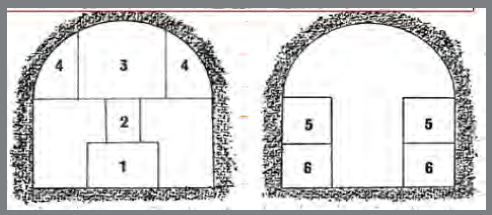




ENGLISH METHOD:

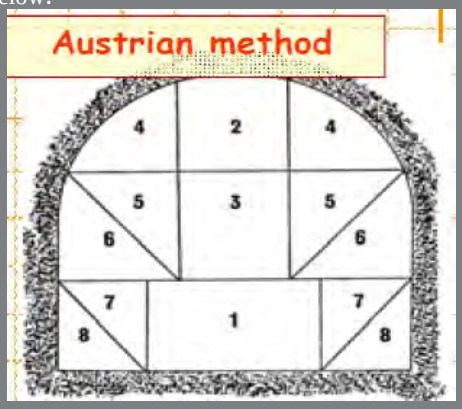
This method was used to tunnel through clay, shale and sand stones. The great advantage of the method was that the masonry was built in one piece from foundation to arch, resulting in a strong homogeneous construction. It's drawback was that tunnellers and masons had to work in turn, making the method the most expensive of all.





UPRAISE METHOD:The method involved first excavating a lower heading. Vertical upraises were then excavated from which the top heading was driven leaving a dividing floor. After the top heading was enlarged to form an arch, the dividing floor was removed and the lower heading broken out to be of full size.

AUSTRIAN METHOD:
The "Old" Austrian Tunnelling Method was first used for the Oberau tunnel in 1837, which was constructed through marls, gneiss and granite. The method resembles the English method in that the full section was excavated before the masonry was added. The key difference was that excavation was carried out in small sections.
A centre-top heading then followed (driven for the same distance). Section 3 was then removed by men working from the top heading, enabling the top structures to rest on the undisturbed timbers below.



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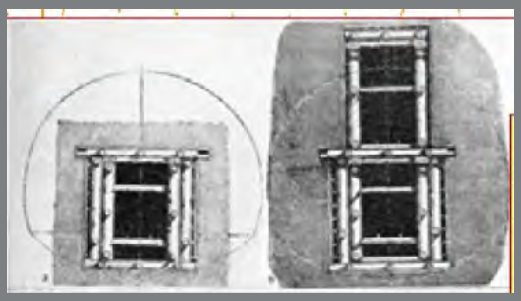
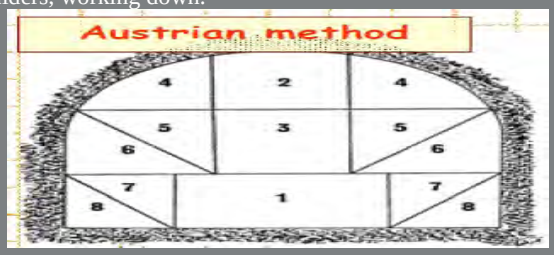
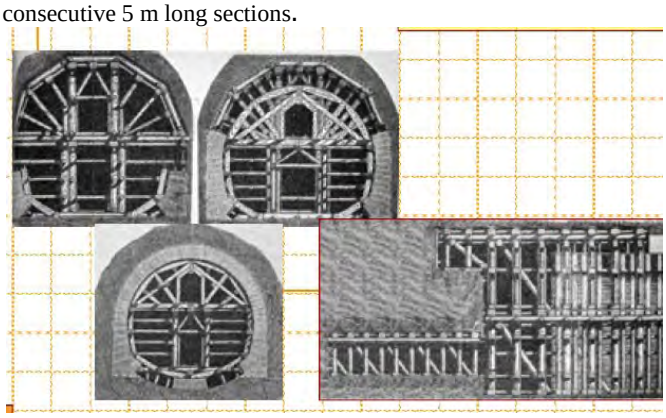


Breaking out of the tunnel to full width then began at the shoulders, working down.



Once the excavation was fully opened, the masonry lining was built up from the foundations to the crown of the arch in consecutive 5 m long sections.



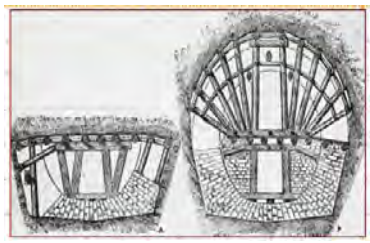
An important aspect of this method was that the driving of a large number of headings, which were immediately strutted, allowed for minimal
disturbance of the surrounding ground. The main disadvantage was that the strutting was liable to distort or give way under asymmetrical pressures.

Such were the experiences with the Christina Tunnel in Italy (1865-70), sometimes referred to as the “most dreadful underground construction ever undertaken".

The material through the tunnel was driven consisted of a peculiar type of cloy that lacked cohesion. When the clay beds were disturbed by the excavation, the clay would slide along laminations. Swelling also gave trouble at the sides and bottom, necessitating the removal of much additional material. At times, the excess swelled to 4x the tunnel area.

To deal with the soft soil, the bottom half of the tunnel was excavated first, a short section at a time, and a heavy stone invert was installed in order to get solid support for excavating and lining the top half.

ITALIAN METHOD
This system (building a lower arch first and filling it with masonry) was dubbed the Italian System. However, no records of applying this system to other tunnels exist. Today, such poor or ‘running’ ground conditions would be pre-treated using freezing or by chemical means.



Atypical practical example of the imperfections mentioned is a double-track railway tunnel in Czechoslovakia, which was driven almost a century ago through a ridge of soft, horizontally stratified sandstone. Although the rock was fairly stable, stratification and jointing caused the corners in the roof on both sides to fall out, leaving a more or less rectangular cavity instead of an arch. The tunnel was supported by an excellent dressed-stone lining, 45cm thick, but it was not backfilled. During the following decades the unsupported layers of sandstone subsided and settled on top of the arch, causing the roof of the lining to bulge downwards (Fig. 1). Had the cavities at either side behind the lining not been simultaneously filled to a

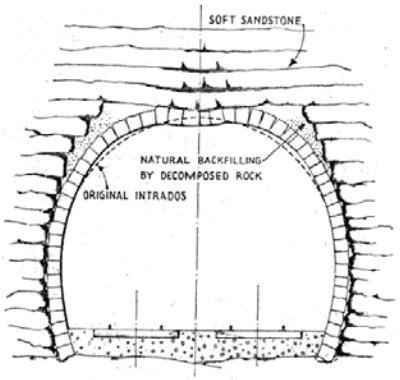


Fig. 1. Deformation of a malconstructed tunnel arch by loosening pressure

certain degree by pieces of rock falling out of the weathering corners the arch would certainly have failed.

The chronological development in tunneling that eventually led to the conceptualization of NATM includes many innovations. Like, Sir Marc Isambard Brunel in the early 19th century, introduced a circular shield for soft ground tunneling Another important contribution was made by Rizha, a German tunneling engineer, he introduced steel support instead of heavy timber. During the 1910s, revolver shotcrete machine was invented by a taxidermist Carl Akeley, shotcrete was used in mines in United States and spread to the Europe in the early 1920s. In 1948, Rabcewicz invented dual-lining supports expressing the concept of allowing the rock to deform before the application of the final lining so that the loads on lining are reduced. The dual-lining concept is followed by the term New Austrian Tunnelling Method that was proposed during a lecture by Rabcewicz in 1962 and it gained international recognition two years later.

A layer of shotcrete with a thickness of only 15cm applied to a tunnel of 10m diameter can safely carry a load of 45 tons/m^2 corresponding to a burden of 23m of rock, which is more than has ever been observed with roof falls. If a steel support structure incorporating No. 20-type wide-flanged arches at 1m centres were used under these conditions, it would fail with 65% of the load carried by the shotcrete lining, and a timber support of the conventional Austrian type would be able to carry only a very small proportion of the same load. If the temporary support deforms or fails the erroneous conclusion is usually drawn that the proposed permanent linings are not enough. In this way permanent linings that are already overdesigned become still heavier.

When we go back to the origin of NATM, Prof. L.v. Rabcewicz (November 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."***

He emphasized three key points, the first is the application of a thin-sprayed concrete lining, the second is closure of the ring as soon as possible and the third is systematic deformation measurement.The definition given above has been redefined by the Austrian National Committee on Underground Construction of the International Tunnelling Association (ITA) in 1980."The New Austrian Tunnelling 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".

With the excavation of a tunnel the primary stress field in the rock mass is changed into a more unfavorable secondary stress field. Under the "rock arch" we understand those zones around a tunnel where most of the time dependent stress rearrangement processes takes place. This includes the plastic as well as the elastic behaving zone.

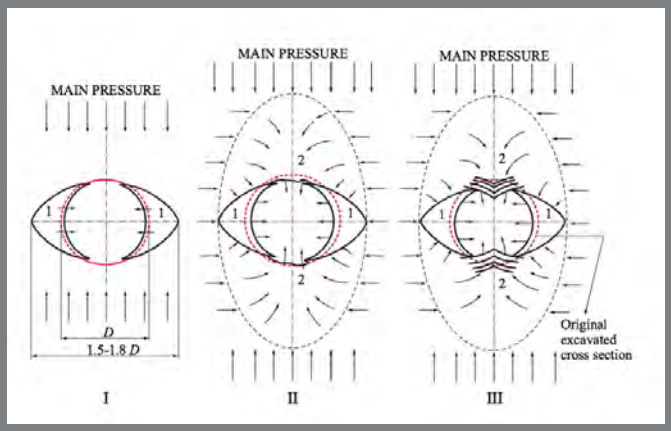
Figs. 2 and 3 show the results by UDEC analysis of the state of stress in a jointed rock mass before and after the excavation of a tunnel. It is observed that the rock mass around the excavation develop zones such that the stresses that are parallel to the excavation edgeincreases considerably in comparison to the undisturbed rock mass stress conditions, whereas the stresses normal to the excavation edge decreases. These areas later on when the stress field reaches a condition of equilibrium,constitute the main part of rock mass support and termed as rock arch. Formation of rock arch depends on no of factors like, rock mass and and its structure, its deformation properties, strength and it also depend on the strain that has occurred in the rock mass at the time of support installation. Therefore the knowledge, experience and skill of the engineer and his team excvacating the tunnel and thereintutuon and timing of support installation is crucial in efficient rock arch formation.

Shape and the development of a rock arch can be influenced by the way and timing of its excavation. as the rock mass advances from one stage of stress to another stage of stress, in the process of attaining equilibrium, each such stage needs to be treated differently to achieve optimum rock arch and the rock installed in each stage needs to be flexible. In NATM, supports are generally provided by shotcrete with or without wiremesh, rock bolt steel beams etc., these supports are inherently flexible.

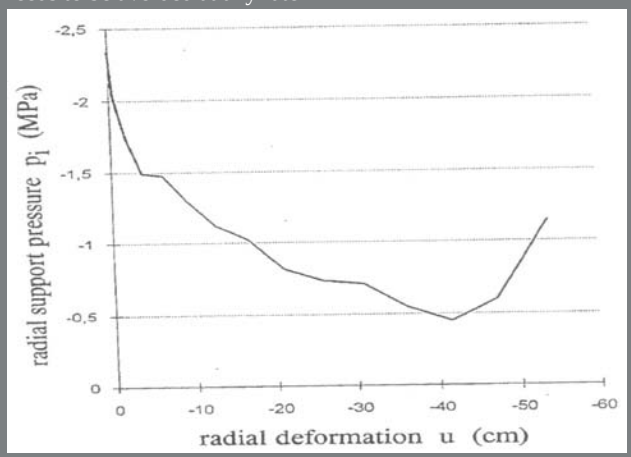
The load carrying capacity is at its best when the shape of the same is devoid of corners and as smooth as possible. In NATM it is therefore demanded that the shape of tunnel excavation shall be as round as possible without any corners to avoid concentration of stress.

When a cavity is made in undisturbed rock the original stress pattern is disturbed. In the course of time, the duration of which depends on the properties of rock, a new stress situation appears in the neighborhood of the cavity, and equilibrium is attained either with or without the assistance of lining according as to whether the shear strength of the rock is or is not exceeded. This stress rearrangement is mechanical and progressive and generally occurs in three stages provided the rock in the neighborhood of the cavity has not been disturbed by earlier tunneling. At first, wedge-shaped bodies on either side are sheared off along the Mohr surfaces and move towards the cavity, the direction of movement being vertical to the main pressure direction. The increased span thus produced causes the roof and floor to start converging. In the next stage this movement is increased; the rock buckles under continuous lateral pressure and may protrude into the cavity. Pressures arising from this action are correctly termed "squeezing pressures."

Rabcewicz in his shear failure theory around an opening stated 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.Then, Rabcewicz (1964) draws a conclusion that



Recognising progressive occurrence of pressure phenomena as described above, because, with the obsolete methods then used, the sections were usually not driven full face but divided into subsequently opened out..."Consider the ground reaction curve show in figure below the radial contact stress between the rock mass and the support (lining) and the radial deformation are plotted. In the graph shown the installation of lining was done after certain span of time, therefore certain deformation of the rock mass was allowed to occur before the installation of lining.Obiviously there in an optimum time of support installation, when the support pressure is at its lowest. If greater deformation in rock mass is allowed to occur befor the installation of lining a higher support pressure is necessary for stabilization. The loosening, disintegration and destrengthening needs to be avoided at any rate



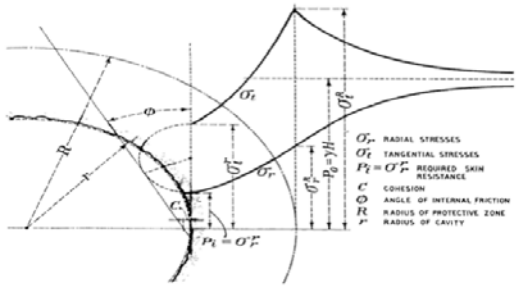
To be able to design the load bearing capacity of the lining for different types of rock or soil, the phenomena of shear failure, explained earlier, should be interpreted accordingly. The relationship between the disturbed ground around the cavity, "protective zone" and the bearing capacity of the support, "skin resistance" is required to be established (Rabcewicz 1964). Mathematical representation of these relations is described by Kastner as:

$$p_i = -c\cot\phi + p_0\left[c\cot\phi + (1-\sin\phi)\right]\frac{r}{R}^{\frac{2\sin\phi}{1-\sin\phi}} \qquad (1)$$

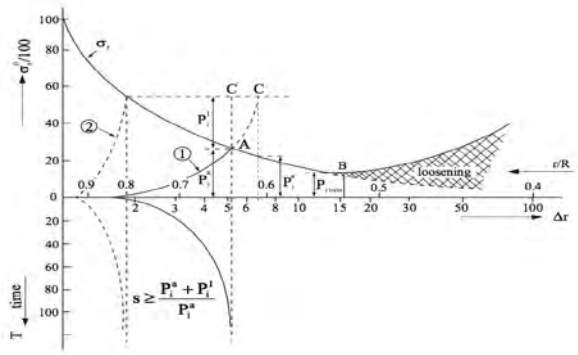
Omitting the cohesion, the Eq. (3.1) yields to

$$p_i = p_0(1-\sin\phi)\frac{r}{R}^{\frac{2\sin\phi}{1-\sin\phi}} = np_0 \qquad (2)$$

Assuming no protective zone in which r=R, then the opening reaches equilibrium without any deformation. The formulae given above are derived according to the stress distribution after a cavity has been made, as is sketched in Figure 2.



The ground response curve (Figure 3) shows the rock/support interaction and deformations in time. 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 behaviour 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.

Rabcewicz also concluded the following points in regard to the reciprocal relationship of the basic supporting system of NATM, which are shotcrete and the anchored rock arch:

i. With the same type of rock and overburden relationship between the size of the joint bodies and the excavation area is decisive for the mobility of the material

ii. With small sections (i.e.10-16 m2) and joint bodies of a few dm3, a simple shotcrete sealing with d = 3 cm = 0.017×R usually stabilises the tunnel

iii. With an underground power station of 400- 600 m2 on the other hand, a rock with joints bodies of this size behaves like a cohesionless mass, and a simple shotcrete lining of 0.017×R = 19-24 cm would never do. A systematically anchored rock arch is imperative in this case.

Support systems as proposed by Rabcewicz (1973) 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..."

The main principles of NATM derive from this goal are mentioned in the following points:-

ROUNDED TUNNEL SHAPE:

Avoid stress concentrations in corners where progressive failure mechanisms start.

MAINTAIN STRENGTH OF THE ROCK MASS:

The inherent strength of the soil or rock around the tunnel domain should be preserved and deliberately mobilised to the maximum extent possible. The surrounding rock mass is the main load bearing component and its carrying capacity must be maintained without disturbance of the rock mass. Avoid detrimental loosening by careful excavation and by mmediate application of support and strengthening means. Shotcrete and rockbolts applied close to the excavation face help to maintain the integrity of the rock mass. The support resistance of the rock mass should be preserved by using additional support elements. 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.

SUPPORT SYSTEM

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 given above.

FLEXIBLE THIN LINING:

The primary support shall be flexible in order to minimize bending moments and to facilitate the stress rearrangement process without exposing the lining to unfavorable sectional forces. Additional support requirement shall not be added by increasing lining thickness but by bolting. The lining must be thin-walled and necessary additional strengthening should be provided by mesh reinforcement, tunnel ribs and anchors rather than thickening the lining.The lining shall be in full contact with the exposed rock. Shotcretefulfils this requirement.

RING CLOSURE

The closure of the ring should be adjusted with an appropriate timing that can vary dependent on the soil or rock conditions. The ring closure time is of crucial importance and this should be done as soon as possible. "However, the principle of ring closure as quickly as possible is only applicable to tunnels in rock with low primary stresses. In tunnels with large overburdens and poor rock quality only a stress to the largest extent possible will achieve the object. Of course, this stress relief, which will continue for many months, must be controlled most accurately by measurements."

Preliminary laboratory tests and deformation measurements in the tunnel should be carried out to optimise the formation of the ground ring. However, conclusion about a rapid ring closure time in deep tunnels to minimise deformations

IN SITU MEASUREMENTS :

Observation of tunnel behaviour during construction is a integral part of NATM. With the monitoring and interpretation of deformations, strains and stresses it is possible to optimise working procedures and support requirements.

FINAL LINING

Stabilisation of the tunnel by use of a secondary lining

UNDERSTANDING

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

Primary and final support design

Support design for both shotcrete and the final lining is the main component of the NATM technical design. The flexibility and the thickness of the primary support with the additional of steel weld mesh or steel fibre reinforcement and rock bolts, forepoling and spiling especially for face stability has to be taken into account in the support design. The time dependency of the lining should be specifically subjected to design considerations as well. The timing for the closure of the ring can be optimised accordingly.

For the initial support design, Rabcewicz (1965) suggests that "A design of shotcrete should attain a high carrying capacity as quickly as possible, and it must be rigid and unyielding so that it seals off the surface closely and almost hermetically."

He points out the important point that shotcrete must gain its maximum carrying capacity in a short time. Further, the design of the support system is required to be integrated to the deformation characteristics of the ground. Then, the load bearing capacity of the media and the support system can be best understood by the rock support interaction diagram (see Figure 3). From these curves, the amount of support required to stabilise the tunnel can be obtained. Providing an adequate support at optimum time will result in a small amount of support leading to lower cost. If the support elements are installed in intimate contact with the surrounding ground, which is the case with shotcrete, rock bolts and anchors, they will deform with the ground and attract load since the stresses in the ground are redistributed.

A shotcrete layer applied immediately after opening up a new rock face acts as a tough surface by which a rock of minor strength is transformed into a stable one. The shotcrete absorbs the tangential stresses which build up to a peak close to the surface of a cavity after it is opened up. As a result of the close interaction between shotcrete and rock the neighboring portions of rock remain almost in their original undisturbed state and are thus enabled to participate effectively in the arch action. The statically effective thickness of the zone of arch action is in this way increased to a multiple of that of the shotcrete. In this way, tensile stresses due to bending are diminished and compressive stresses are easily absorbed by the surrounding rock. The zone of arch action can be increased at will by rockbolting. Disintegration always starts by the opening of a minute surface fissure; if this movement is prevented at the outset by applying a shotcrete layer the rock behind the shotcrete remains stable.

Shallow tunnels in rock of medium quality, when built by customary methods, need a fairly strong temporary support and concrete lining. When the new method of surface stabilization is adopted, only a thin layer of shotcrete, possibly locally strengthened by rockbolts, will provide both temporary support and a satisfactory permanent lining. Experience so far has shown that shotcrete, especially when combined with rockbolting, has proved unexcelled as a temporary support for all qualities of rock with standing times down to less than one hour and even for ground which normally could only be mastered by careful forepoling.

Dr. Sauer (1988) notes that the ring must be adequately supported within 1.5D of the face for a single tunnel in unstable rock conditions. However, for cohesionless and/or poor cohesion-ground, the three dimensional stress field has to be supported by an extension of the support shell ahead of the face, forepoling, or leaving an unexcavated wedge to support the face.

Kuesel (1987) points out that the dimensioning and details of the lining are barely related to stress considerations. He suggests that the first consideration should be given to the pore water. Therefore, if the lining must resist hydrostatic pressure, this ought to be governed by the lining design. In order to eliminate groundwater, either drainage or a waterproof membrane can be adopted. Kuesel's second consideration is constructability or compatibility of the lining design that is suitable for the expected ground conditions, which is mainly related to the stand-up time of the ground.

In summary, for shotcrete and secondary lining design the following should be considered:

i. Ground characteristics, such as strength and stand up time must be determined. The ground support interaction curve obtained accordingly.

ii. 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.

iii. 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.

iv. Monitoring of the stresses in/on the lining and the deformation must be provided.

v. Preliminarydesignoftheinitialliningshouldbe 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.

vi. The secondary lining is usually a precast concrete lining and they are 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 (Craig & Muir Wood 1978).

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. The importance of these investigations are emphasised by NATM's proponents. It is also believed that the main cause of failure is unexpected ground conditions. Therefore, the ground investigation must be conducted thoroughly toensure 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 fulfill analytical or computational preliminary design for eligible excavation patterns and geometry, and face advance in each round, as well as optimum support design.

Design of NATM applications in soft ground

In the case of soft ground applications, especially in soils, NATM applications are relatively recent. The main concern pointed out by Muller (1978) is that the shotcrete ring must be closed as early as possible in any soft ground application of NATM. One of the reasons for rapid ring closure is to prevent surface buildings suffering damage from settlement. Another reason is that the shorter stand-up time of soft ground is due to the bond between soil particles being weaker and cohesion is also lower than for rocks. In the near surface soft ground case, the in- situ stress will be relatively low, the ground relatively weak and unable to support redistributed loads. In a near surface tunnel excavated in soft ground, it will be generally necessary to close the invert quickly to form a load-bearing ring and to leave no section of the unexcavated tunnel surface unsupported even temporarily. It is also important that the length of the unsupported span must be left shorter compared totunnelling in rock. In addition, the stability of the working-face must be maintained. To avoid any collapse, the geometry and the size of the excavation section in one round should be optimised accordingly.

Relatively recent soft ground NATM application has brought about collapses some of which producedcatastrophic damage to surface buildings, and some of which caused environmental impact by creating large holes in urban areas. Thus, the safety regulations for underground works have limited the design consideration. After three parallel tunnels, which were being constructed as part of the Heathrow Express Rail Link in London Clay, collapsed, The Health and Safety Executive (HSE) (1996) prepared a report viz. Safety of New Austrian Tunnelling Method (NATM) Tunnels. They have proposed a number of safety measures and design criteria before, during, and after a construction of NATM tunnels. These can be summarised as follows:

• GROUND INVESTIGATION:

This investigation must be carried out 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. Also, a comparison between new and previous technology should be undertaken to assist in selection of the most appropriate technologies. Moreover, universities, research groups can contribute to the evaluation and investigation of new and/or untried methods of working.

• 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 (more details are given in HSE, 1996).

• 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, to secure the safety of those who work within the tunnel and in buildings, structures and utilities above the tunnel, stability of the face must be maintained using additional supports such as forepoles, faster excavation, draining groundwater and reducing the face size or advance per round.

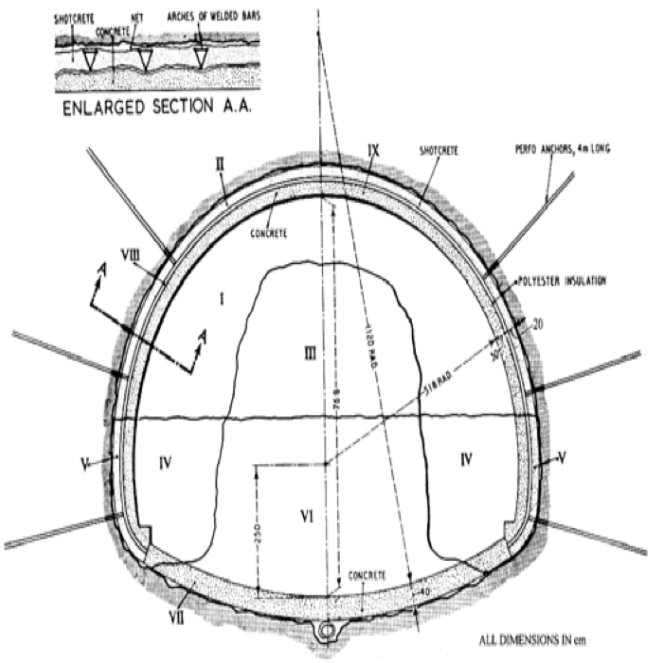
• GROUND SETTLEMENT CONTROL MEASURES:

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.

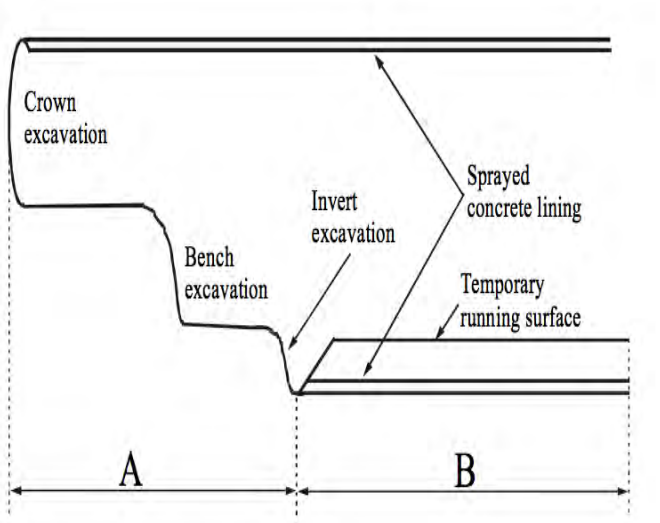
A number of different NATM tunnel sizes, geometry, and excavation patterns have been adopted in a range of geological conditions. In most cases, especially in soft ground, it is not applicable to excavate the full tunnel face. Hence, the excavation face is usually divided into small cells that will help the ground stand until completion of the lining. Generally, excavation is carried out in six or more steps depending on the size and the geometry of the tunnel. Figure 8 illustrates a typical main cross-sectional geometry for a NATM tunnel proposed by Rabcewicz (1965). The shape of the tunnel is different from conventional circular tunnels. The Roman numbers indicate the excavation order and subsequently applied support elements. The first step is the excavation of the top heading (I), leaving the central part to support tunnel face. Then, the auxiliary lining (shotcrete) II is formed and followed by removing the top central portion (III) subsequently excavation of left and right walls (IV)



The fifth step is the application of shotcrete with additional reinforcements (V) followed by excavation of a bench (VI). Finally, the invert is closed with concrete (VII) following the installation of a waterproof membrane (VIII) and concreting of the inside lining (IX).

Failure incidents for NATM in the World

Type 'A' failures, heading collapses, occurred in the area between the tunnel face and the first complete ring of the sprayed concrete lining, and the type 'B' failures occurred in the region in which sprayed concrete lining is complete (Figure 9). 'C' type of failures occurred in a different part of the tunnel which are located far away from where A and B type of collapses occurred such as collapses at portals or at breakouts from vertical construction shafts



There are a number of collapses and failures of NATM tunnels that have lead to human death and injury. These collapses brought about serious damage to public buildings and infrastructure. According to the HSE report, 39 major incidents some of which are given in Table 2, have occurred during the 30 years since NATM was first introduced.

The increase in the incidents reported is attributed to a number of factors as follows:

- There are inherent problems with NATM tunnel construction
- Hazards are not being adequately identified, managed and controlled
- There is over-confidence in the method

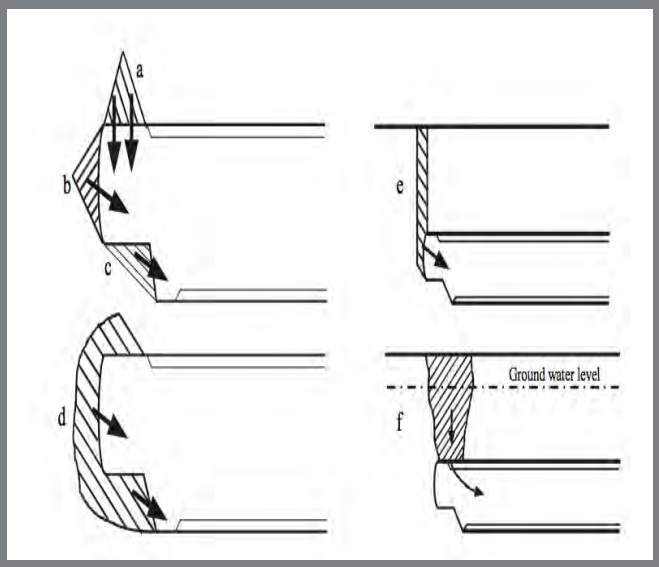
There is more open reporting of failures

- NATM is increasingly being used in more demanding environments
- NATM is being used by those unfamiliar with the technique

Figure 10 illustrates the type of collapses that have occurred in headings. These are as follows:

a) Crown failures where soil flows into the tunnel b) Local face failures where a part of the working face runs in to the tunnel c) Bench failures where a part or the entire of bench slides transversely or longitudinally into the tunnel d) Full face failures in which face, heading and bench flow into the tunnel e) Washout failures f) Pipe failures

Other types of failure that occurred are failures of the lining before and after ring closure, and both before and after ring closure, bearing failure of the arch footings, failure due to horizontal movement of the arch footings, and the failure of the side of the gallery wall which took place after closure of the lining ring. Shear failure, compressive failure, combined bending and thrust failure and punching failure of the lining came about before and after ring closure.



Causes of these collapses are reported by the HSE (1996) as follows:

- Unpredicted geological causes
- Planning and specification mistakes
- Calculation or numerical mistakes
- Construction mistakes
- Management and control mistakes

Solar Energy Power Plant at Katra - A Green Initiative

1.0 Introduction:-

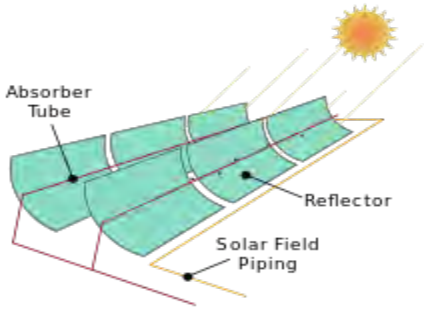
1.1 Solar Energy:-

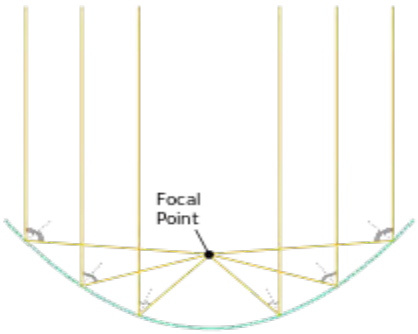
Solar Power is the conversion of sunlight into electricity, either directly using photovoltaics (PV), or indirectly using concentrated solar power (CSP). Concentrated solar power systems use lenses or mirrors and tracking systems to focus a large area of sunlight into a small beam. Photovoltaics convert light into electric current using the photovoltaic effect.

1.2 Solar Technologies:-

Solar technologies are broadly characterized as either passive solar or active solar depending on the way they capture, convert and distribute solar energy. Active solar techniques include the use of photovoltaic panels and solar thermal collectors to harness the energy. Passive solar techniques include orienting a building to the Sun, selecting materials with favourable thermal mass or light dispersing properties, and designing spaces that naturally circulate air.

1.2.1 Concentrated Solar Power:-





A diagram of a parabolic trough solar farm, and an end view of how a parabolic collector focuses sunlight onto its focal point.

Concentrating Solar Power (CSP) systems use lenses or mirrors and tracking systems to focus a large area of sunlight into a small beam. The concentrated heat is then used as a heat source for a conventional power plant. A wide range of concentrating technologies exists: the most developed are the parabolic trough, the concentrating linear fresnel reflector, the Stirling dish and the solar power tower. Various techniques are used to track the sun and focus light. In all of these systems a working fluid is heated by the concentrated sunlight, and is then used for power generation or energy storage. Thermal storage efficiently allows up to 24 hour electricity generation.

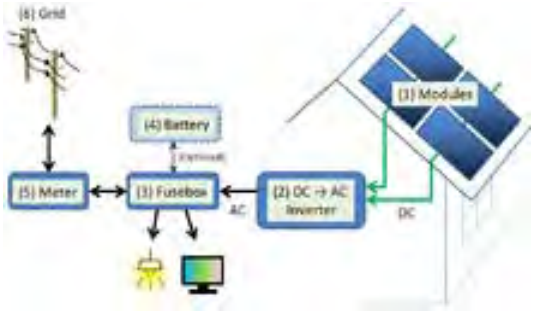
A solar power tower uses an array of tracking reflectors (heliostats) to concentrate light on a central receiver atop a tower. Power towers are more cost effective, offer higher efficiency and better energy storage capability among CSP technologies.

1.2.2 Photovoltaics:-

Photovoltic cell at Gurgoan Railway Station.

A solar cell, or photovoltaic cell (PV), is a device that converts light into electric current using the photovoltaic effect. The first solar cell was constructed by Charles Fritts in the 1880s.

1.2.3 Photovoltaic Power Systems:-



Simplified schematics of a grid-connected residential PV power system

Solar cells produce direct current (DC) power which fluctuates with the sunlight's intensity. For practical use this usually requires conversion to certain desired voltages or alternating current (AC), through the use of inverters. Multiple solar cells are connected inside modules. Modules are wired together to form arrays, then tied to an inverter, which produces power at the desired voltage, and for AC, the desired frequency/phase.

R.K. Chaudhary
CEE/USBRL/JAT

In these grid-connected PV systems, use of energy storage (batteries) is optional. In certain applications such as satellites, lighthouses, or fully green buildings, batteries or additional power generators are often added as back-ups. Such stand-alone power systems permit operations at night and at other times of limited sunlight.

1.3 Applications of Solar Technology:-

Solar energy refers primarily to the use of solar radiation for practical ends. However, all renewable energies, other than geothermal and tidal, derive their energy from the sun.

c) Electricity Production:-

View of Solar power plant at Gurgaon Railway Station.

Solar power is the conversion of sunlight into electricity, either directly using photovoltaics (PV), or indirectly using concentrated solar power (CSP). CSP systems use lenses or mirrors and tracking systems to focus a large area of sunlight into a small beam. PV converts light into electric current using the photoelectric effect.

e) Solar Thermal :-

Solar thermal technologies can be used for water heating, space heating, space cooling and process heat generation.

i) Water heating :-

Solar hot water systems use sunlight to heat water. In low geographical latitudes (below 40 degrees) from 60 to 70% of the domestic hot water use with temperatures up to 60 °C, can be provided by solar heating systems.

ii) Heating, cooling and ventilation :-

Thermal mass is any material that can be used to store heat i.e. heat from the Sun in the case of solar energy. Common thermal mass materials include stone, cement and water.

iii) Water treatment :-

Solar water disinfection system involves exposing water-filled plastic polyethylene terephthalate (PET) bottles to sunlight for several hours. Exposure times vary depending on weather and climate from a minimum of six hours to two days during fully overcast conditions.

Solar energy may be used in a water stabilisation pond to treat waste water without chemicals or electricity.

iv) Cooking:-

Solar cookers use sunlight for cooking, drying and pasteurization. They can be grouped into three broad categories: box cookers, panel cookers and reflector cookers. Working example can be seen at Shri Sai Baba Langer Kitchen, Sirdi.

a) Architecture and Urban Planning :-

Sunlight has influenced building design since the beginning of architectural history. The common features of passive solar architecture are orientation relative to the Sun, compact proportion (a low surface area to volume ratio), selective shading (overhangs) and thermal mass. When these features are tailored to the local climate and environment they can produce well-lit spaces that stay in a comfortable temperature range.

b) Agriculture and Horticulture :-

Greenhouses like these in the Westland municipality of the Netherlands grow vegetables, fruits and flowers.

Agriculture and horticulture seek to optimize the capture of solar energy in order to optimize the productivity of plants.

d) Transport and Reconnaissance :-

Some vehicles use solar panels for auxiliary power, such as for air conditioning, to keep the interior cool, thus reducing fuel consumption.

Helios UAV in solar powered flight.

A solar balloon is a black balloon that is filled with ordinary air. As sunlight shines on the balloon, the air inside is heated and expands causing an upward buoyancy force, much like an artificially heated hot air balloon. Some solar balloons are large enough for human flight, but usage is generally limited to the toy market as the surface-area to payload-weight ratio is relatively high.

2.0 National Solar Mission:-

2.1 Objectives:-

The objective of the National Solar Mission is to establish India as a global leader in solar energy, by creating the policy conditions for its diffusion across the country as quickly as possible.

2.2 Nation Target:-

The National Solar Mission is a major initiative by Govt. Of India, the main target of the mission is to create an enabling policy framework for the deployment of 20,000 MW of solar energy by 2022 and promote programme for off-grid applications reaching 1,000 MW by 2017 and 2000 MW by 2022.

2.3 Railway Solar Mission:-

(i) **Under Railway Funding:** About 10 MW solar based lighting systems at about 500 Railway stations, 4000 LC gates, 400 street lights, Rail Coach Factory Raibareil, 50 office buildings, solar water heaters (6.7 lakhs LPD) etc..

(ii) **Sanctioned works under Railway funding and their Progress:**

1 MWp solar plants at Katra Railway station is targeted for completion March 2014. Further tender for procurement of about 6 MW of solar plants at 200 railway stations, 26 nos. of roof top locations is under evaluation stage

(iii) **Solar Mission of Railways for Harnessing 1000 MW Solar Energy**

To achieve the target of harnessing solar energy of 10% of IR's Electricity Consumption by 2020, solar projects of 1000 MW are planned in PPP model without any investment by Railways. Accordingly, IR has planned to harness **1000 MW solar plants** in railway/private land and roof top spaces of railway buildings through Railway Energy management Company (REMC), a JV of MoR and RITES and Solar Energy Corporation of India (SECI), a PSU of MNRE in following phases over next 5 years:

- **Phase- I: 200 MW** ground mounted in private land with Central Financial Assistance (CFA) of Rs. 1 cr per MW from MNRE. Tender likely to be invited by Dec 2014.
- **Phase- II: 150 MW** at roof top of railway buildings with subsidy support of 30% of capital cost from MNRE.
- **Phase-III: 500 MW** ground mounted in railway/private land with VGF support upto Rs. 2.5 cr per MW under national Clean Energy Fund (NCEF) under MoF to be processed by MNRE.
- **Phase IV: 150 MW** ground mounted in railway/private land with VGF support upto Rs. 2.5 cr per MW or CFA of Rs. 1 cr of MW as approved by MNRE.

Request has been made to MNRE to extend VGF (Viability Gap Funding)/subsidy support for harnessing solar energy by Railways for 1000 MW. Initial planning is to harness initially solar energy in those states where the cost of power is higher.

2.4 Importance and relevance of solar energy for India:-

a) **Cost:** Solar is currently high on absolute costs compared to other sources of power such as coal. The objective of the Solar Mission is to create conditions, through rapid scale-up of capacity and technological innovation to drive down costs towards grid parity. The Mission recognizes that there are a number of off-grid solar applications particularly for meeting rural energy needs, which are already cost-effective and provides for their rapid expansion.

b) **Scalability:** India is endowed with vast solar energy potential. About 5,000 trillion kWh per year energy is incident over India's land area with most parts receiving 4 to 7 kWh per sq. m per day. Hence both technology routes for conversion of solar radiation into heat and electricity, namely, solar thermal and solar photovoltaics, can effectively be harnessed providing huge scalability for solar in India. Solar also provides the ability to generate power on a distributed basis and enables rapid capacity addition with short lead times.

c) **Environmental impact:** Solar energy is environmentally friendly as it has zero emissions while generating electricity or heat.

d) **Security of source:** From an energy security perspective, solar is the most secure of all sources, since it is abundantly available. Theoretically, a small fraction of the total incident solar energy (if captured effectively) can meet the entire country's power requirements. The solar imperative is both urgent and feasible to enable the country to meet long-term energy needs.

3.0 Solar Power Plant At Katra:-

3.1 During the inauguration of Udhampur-Katra Section on 4th July 2014, Hon'ble Prime Minister Sh. Narendra Modi has mentioned that Katra Station can be converted into Solar Railway Station to make it environmental friendly and be a part of the National Solar Power Mission. Chairman Railway Board had committed to take up the Solar Power Plant at Katra immediately to make it part of environmental friendly movement. Accordingly, a decision was taken for installation of 1 MW Solar Power Plant at Katra based on the feasibility study done by M/s Gamsons Solar Limted, Tata Power Solar Limited and Solar Energy Corporation of India Limited

3.2 Briefing of Events: - On 4th July 2014, Hon'ble Prime Minister of India made announcement for installation of Solar Power Plant at Katra, while inaugurating Udhampur-katra Section and dedicating it to the nation.

i) On 18th July 2014, Chairman Railway Board called CAO/USBRL and CEE/USBRL and directed to go-ahead with the installation of 1 MW Solar Power Plant at Katra based on the feasibility report from various organisation such as M/s Gamsons Solar Limted, Tata Power Solar Limited and Solar Energy Corporation of India Limited.

ii) The administrative approval was granted by CAO/USBRL for funding the project of Solar Power Plant from USBRL Project (National Project) and includes the same in the revised estimate of USBRL Project after studying PPP Model vs Capital Investment Model. The target was communicated by AM/Plng. to Railway Board as 31st March 2015.

iii) The proposal was concurred by FA&CAO/USBRL on 5/08/14 whereas NIT was issued on 01/08/14 before the vetting of the proposal in view of the urgency and the project was being monitored directly by Chairman Railway Board.

iv) The tender was initially to be opened on 5/09/14, which was postponed to 16/09/14 due to issue of amendments on various technical/qualifying issues.

v) Briefing Note and Comparative Statement were vetted on 18/09/14.

vi) TC Recommendations were put-up on 4/10/14 and the same were accepted on 7/10/10 by Competent Authority.

Letter of intent was issued on 7/10/14 and detailed LOA was issued on 9/10/14 to M/s Rajasthan Electronics & Instrumentation

3.3 Brief details of 1 MW Solar Power Plant:-

1	Name of Work	:	Design, Manufacture, Supply, Installation, Testing & Commissioning of Solar Power plant of IMWp capacity with all the electrical and associated equipments including civil works at Shri Mata Vaishno Devi Katra Railway Station building and other structures with AMC for five years.
2	Estimated Value	:	Rs. 9.05 Cr
3	Date of Opening of Tender	:	16.09.2014
4	Date of Award	:	07.10.2014
5	Completion Period	:	4 months
6	Final Awarded Value	:	Rs 8.52 Cr
7	Tender Awarded to	:	M/s Rajasthan Electronics and Instruments Limited, Jaipur. (Govt. of India PSU)

Technical Details of Plant:-

1	System Power Rating	:	1 MWp.
2	Type of Solar PV module	:	Crystalline silicon.
3	Module capacit	:	250 Wp
4	Length of module	:	1.67 m
5	Width of module	:	0.9 m
6	Weight of module	;	22 kg
7	Type of module mounting structure	:	Aluminium mounting structure for Platform shelter and galvanised MS structure for Roof Top of Buildings.
8	Weight of Aluminium structure	:	15 kg/ kW
9	Type of inverters	:	String inverters of 50 kWp capacity each
10	Support structure, design and foundation wind withstanding.	:	Wind pressure upto 30 m height (kg/m2)-195, wind speed 200 KM per hour.
11	Total Connected load of Katra Railway station	:	3.13 MW
12	Average Annual Solar Global Radiation (as per SEC-NREL)	:	5.09 kWh/m2/day
13	Annual Electric Power generation expected I	:	14,45,000 UNITS
14	Expected Annual Saving	:	Rs. 1 crore (approx.) per annum.

3.4 Technical Specifications :-

3.4.1 Deviations to RDSO Specifications and other defect liabilities:-

In order to ensure the quality and protect the interest of railways, minor deviations with respect to RDSO Specifications were inserted in technical specifications to make it better and appropriate for site requirement and also the Clause of defect liability and warranty clauses were added as per the letter written to letter written to ED/EM/Railway Board with copy to ED/EM/RDSO regarding deviations adopted to RDSO Specification no RDSO/PE/SPEC/PS/0092-2008 (Rev '0') Amendment-5 for Grid Connected Solar Power Plant to be provided at Katra. Beifely deviation to RDSO specifications are:-

SN	RDSO Specification Clause	Modified clause by USBRL/Northern Railways	Remarks
5.	***Clause 2 .0-*** Scope covers grid connect solar photovoltaic (SPV)system capacities in range of 10 KWp to 500 KWp.	This specification has been usd as standard for 1MWp solar power plant as total 1MWP capacity will be installed at 5 different locations at Katra Railway Station indicated below:- i) Roof top of PF-1-430KWp ii) Roof top of PF-2/3-230KWp iii) Roof top of FOB-1-75KWp iv) Roof Top of FOB-2 – 75KWp v) Roof Top of Station Building- 100KWp.	Since individual plants are not exceeding 500KWp capacity.
6.	***Clause 6.9.1-*** Imported SPV module or cell will not be accepted, unless MNRE's policy/rules permit the same.	As per MNRE's policy, only indigenously manufactured PV modules should be used in Solar PV system power plants. However, other imported components can be used, subject to adequate disclosure and compliance to specified quantity norms and standards. For imported components also warranty shall be given by PV modules indigenous manufacturer.	To ensure the warranty for entire life of PV modules.
1.	***Clause 6 .9.8-*** Bird spike shall be provided so as to avoid bird sitting on the solar modules at the highest point of array/module structure.	Deleted.	Bird spike cannot be given as it will cast a shadow on the PV array resulting in loss of generation and might lead to hot spots in PV module.
2.	***Clause 6 .9.9-*** SPV modules shall be highly reliable, light weight and shall have a service life of more than 25 years. SPV modules shall have a limited power loss of not more than 10 % of nominal output at the end of 10 years and not more than 20% of nominal output at the end of 25 years.	The monthly guaranteed generation of power shall not be less than the values as guaranteed by the bidder along with their offer. The bidder is required to submit the details of guaranteed generation power per month for entire 25 years taking into account the radiation level and meteorological conditions as per NASA satellite for location of Katra. Minimum cumulative generation shall not be less than 14,45,000 units per annum by 1MWp plant at the time of commissioning. 90% of generation guaranteed at the end of 10th year and 80% generation guaranteed at the end of 25th year in line with RDSO specification Clause 6.9.9 subject to correction factor as per actual average global solar radiation measured by the calibrated pyranometer for 3 months at site.	To ensure the guaranteed output of the plant over 25 years.
3.	***Clause 6.10.3-*** The array structure shall be made of hot dipped galvanised MS angles (or alternate MS section) of suitable size.	Replaced with technical specifications for Aluminium mounting structures with relevant drawings.	Since solar panels mostly will be installed on platform sheds and FOBs. Thus aluminium being lighter and rust free is suitable.
4.	***Clause 6.16.1-*** the concept plan/ design of each sub-system shall be submitted to RDSO for approval. The wiring diagram and operation and maintenance information details shall be given as detailed in IEC 62446.	The concept plan/ design of each sub-system shall be submitted to Northern Railway for approval. The wiring diagram and operation and maintenance information details shall be given as detailed in IEC 62446.	Due to urgency of this work, as RDSO may take long time to coordinate with vender, visit site of work and convey approval.

7.	***Clause 6 .17.1-*** RDSO shall conduct prototype testing of grid connect solar generating system of individual capacity separately.	It is clarified that the firms who have already got approved their prototype tests, may submit the test report of the same and there is no need to repeat the prototype test. The firms, who have not got their prototype tested, may not be required to carry out these tests from RDSO. However, such firms are required to submit the type test reports of all acceptance and routine tests specified in RDSO specifications shall be carried out at the firm's premises by the inspecting agency or representative of Railway.	To save time involved in prototype testing by RDSO considering the urgency of work.
8.	***Clause 1 0.0-*** Guaranty/warranty (deleted)	Performance guaranty tests and warranty clauses added.	To protect interest of Railways.

3.5 Execution Details.

3.5.1 Project at Glance

Location	Capacity	Serviceable Area Available (sq.m)
Platform No. 1 Shelter (South Facing)	400 KW	3420
Platform No. 1 (North Facing)	150 KW	3420
Platform No. 2 and 3 (South Facing)	240 KW	2060
Platform No. 2 and 3 (North Facing)	60 KW	2060
Katra Railway Station building top and other service buildings	150 KW	1200
Total	1000 KW (1 MW)	

3.5.2 Completion schedule: - March 2015

3.5.3 Type **and Quality of SPV Module**

A. SPV MODULES

Agency	:	REIL
Type	:	250W 60
Peak Power Output	:	250 Watts
Maximum Voltage	:	30/34 VDC
Dimension in mm	:	1658X997X42/ 1973X997X42
Type of Cell used	:	mono/poly crystalline

B. SPV MODULES

Type and Quality

a) The total Solar PV Array Capacity shall be 1MWp. Crystalline Silicon type solar module manufactured by REIL shall be used. Modules are being supplied by REIL with a warranty that:

b) Manufacturing process is in compliance with the standards approved by MNRE & shall have IEC 61215 certification.

c) The amount of power of any supplied module will not vary more than 3% - 5% from the specified power rating of the modules.

d) By pass diodes shall be mounted on each module terminal.

e) Channels for frame shall be of anodized aluminium and shall be electrolytically compatible with the structural material used for mounting the module.

3.5.4 Execution of Methodology.

In brief following methodology / activities shall be covered in execution of the project:

a) In house co-ordination activity of various departments such as Design Production, Material Management and Quality Assurance Departments.

b) Process of identification and finalization of vendors for BOS items such as mounting structures, Inverter, Transformer, wires & cables from approved vendors and finalization of vendors.

c) Submission of data sheets of various items to Railway for approval.

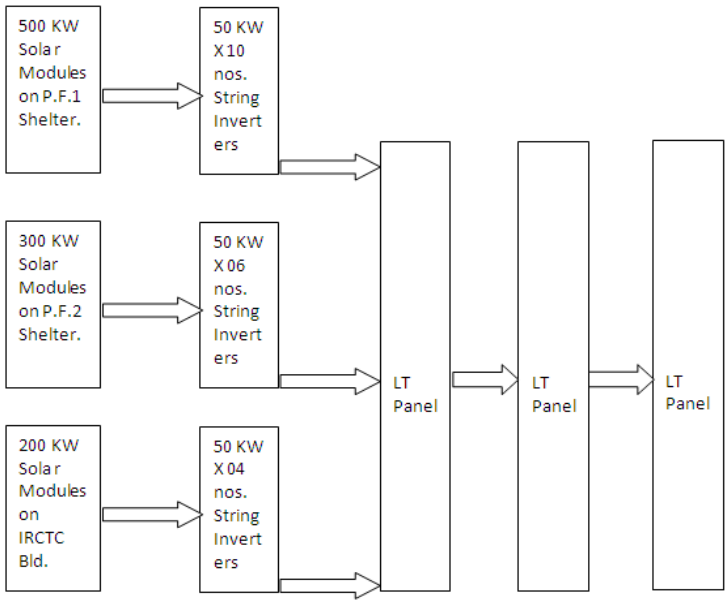
d) Site visit by REIL Engineer for planning and ascertaining on site requirements,

e) Submission of pre-dispatch inspection schedule and Inspection of material by Railway consultant at manufacturers works.

f) Supply of material at site.

g) Installation and commissioning of Systems by REIL Engineers in consultation with Railway representative and handing over of the same.

3.5.5 1 MWp SPV System Block Diagram:-



4.0 Benefits accrued from the Solar Power Plant at Katra:-

a) Generation of clean and green energy in a Power deficit area of J&K state.

b) Annually generation of 14, 45,000 units of electricity.

c) Annual Reduction of 10000 tones Carbon dioxide.

d) Annual saving of Rs.1 Cr (approx.) on energy bill.

e) Environmental awareness at strategically location and important Railway Station for Pilgrim Tourism.

5.0 Conclusion:-

1. The work of Solar Power plant at Katra was awarded in a record time in a Government set up. There is a need to spread this initiative at a larger scale all Indian Railways to harness the benefits of solar energy and contribute to make our nation energy surplus pool.
2. There is a need to improve/ revise RDSO Specification of Grid connected solar pants.
3. There is a need to provide proper training and awareness courses on solar technologies to Railway officers.

Rehabilitation of Tunnel No. T-10 (28/280 – 29/250) on Katra – Reasi section

1.0 Introduction:-

Construction work of Tunnel No. 10, on Katra – Reasi section of USBRL project, was started in June'03. The total length of this tunnel is 970m (28/280 – 29/250). The alignment of tunnel lies on a curvature of 4 degree. The index plan of this tunnel is depicted in **Fig. 1**. There is a maximum overburden of 190m. This tunnel has been constructed under the holy town of Katra. This tunnel passes through highly unfavorable geological formations, due to which tunnel work was extremely difficult. Strata from 28/578 to 28/840 involved loose soil containing pebbles compounded with seepage of water due to which tunneling was extremely difficult in such a strata. Though, the tunneling work progressed without problem from Katra end i.e., from chainage 28/280 upto 28/578, thereafter the work of tunneling could be continued with great difficulties. During initial construction, squeezing of tunnel and heaving of tunnel invert was observed at many locations. Tunnel collapses and cavity formations were also encountered. Though, the initial tunnel excavation work was completed in 2009, the problems of squeezing/heaving and cavity formation necessitated intensive rehabilitation work in this tunnel.

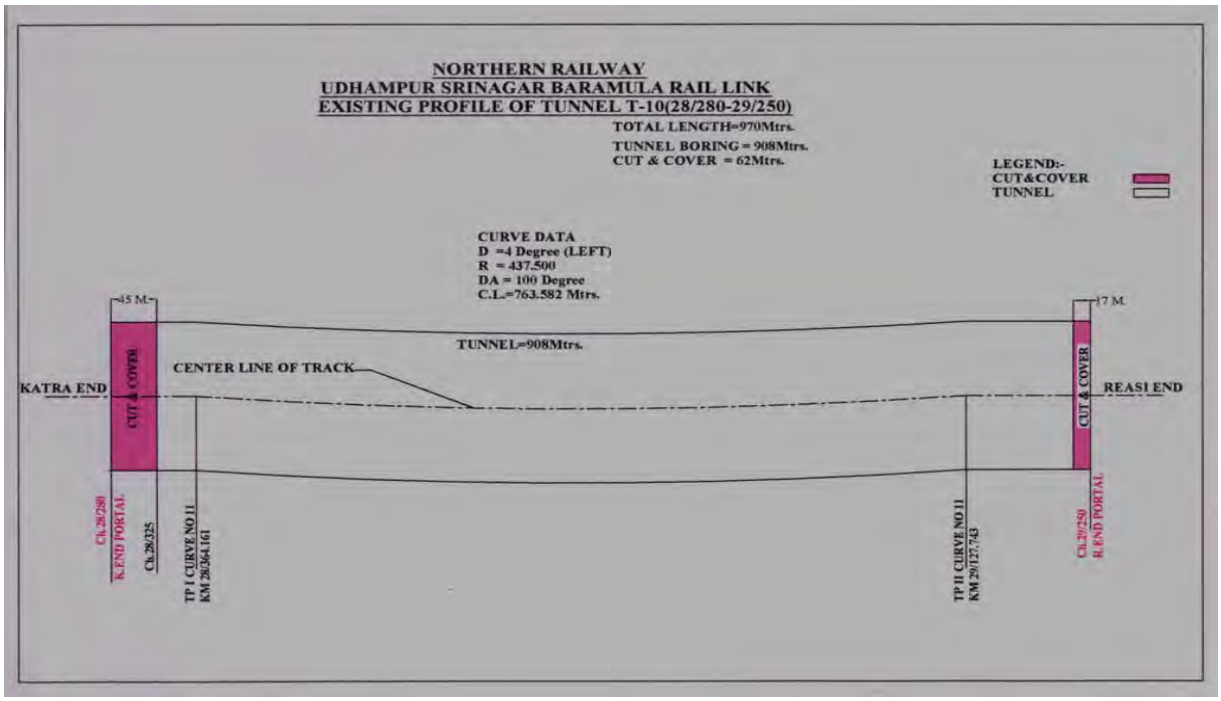


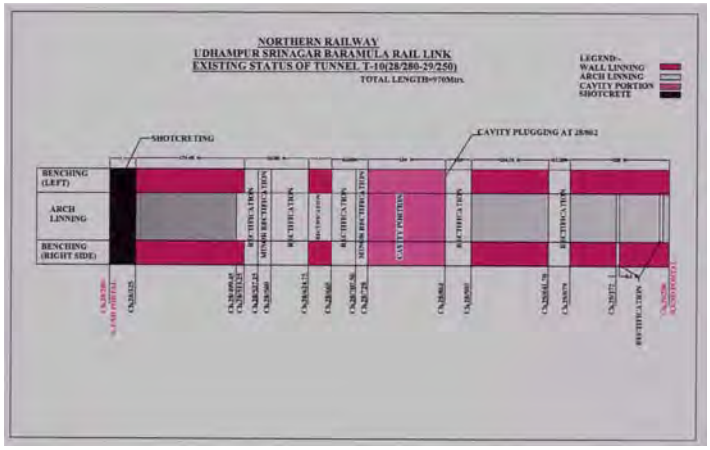
Fig. 1 Plan of Tunnel T-10

2.0 Details of portions of tunnel T-10 requiring rehabilitation:-

Due to poor geological strata, leading to very less stand up time, squeezing/heaving of tunnel & cavity formations occurred in following stretches:

i)	28/537 to 28/513	24m - squeezing/heaving of tunnel.
ii)	28/624 to 28/560	64m - squeezing/heaving of tunnel.
iii)	28/707 to 28/665	42m - squeezing/heaving of tunnel.
iv)	28/862 to 28/724	138m - Cavity formation.
v)	28/907 to 28/862	45m - squeezing/heaving of tunnel.
vi)	29/080 to 29/043	37m - squeezing/heaving of tunnel.

The status of tunnel before start of rehabilitation work is depicted in **Fig. 2**.



Sandeep Gupta
CE/South/USBRL

3.0 Description of geology encountered:

The tunnel probably passed through sheared dolomite/crushed dolomite i.e part of Reasi Thrust. Dolomite pieces are mixed with clay (Yellow in colour). Topography of the area also indicates that the collapsed tunnel portion is below Southern slope of Tara Kot Hill which is made up of jointed & sheared dolomite. Reasi end of the tunnel has been excavated through dolomite scree mixed with reddish clay and charged with water. The examination of the area indicates that tunnel is located along the Reasi thrust & due to curve in tunnel, the present collapsed portion may be passing through sheared and crushed dolomite charged with water.

4.0 Causes of tunnel failure leading to squeezing/heaving and cavity formation/collapse :-

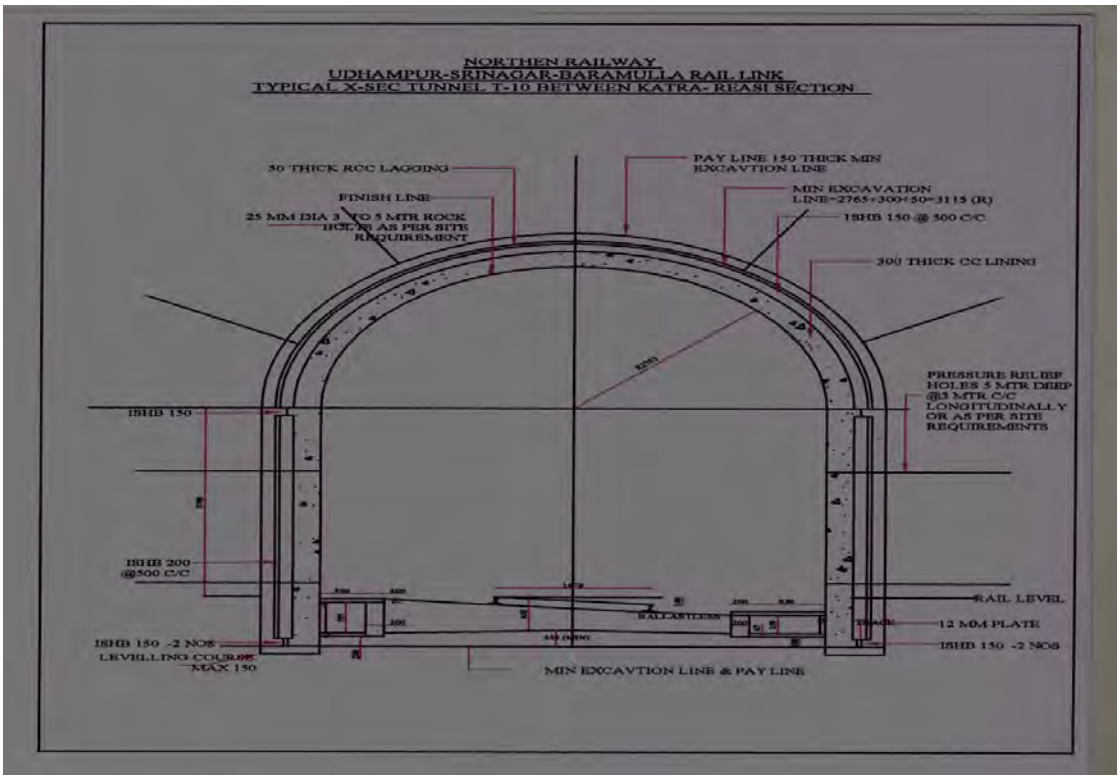


Fig. 3 D-shaped section.

As already brought out in para 2.0 above, due to extremely poor geology coupled with D-shape of the tunnel, problem of tunnel failure leading to squeezing/heaving and cavity formation/collapse was encountered in tunnel T-10. Typical x-section of D-shaped portion of T-10 is depicted in **Fig. 3**.

5.1 Merits/demerits of D-shaped tunnels :

MERITS	DEMERITS
i. The section is economical	**i. The SPL beam is a weaker part to transmit the compressive forces coming from crown.**
ii. Bending/fabrication of arch rib is easy having a constant curvature.	**ii. Less efficiency to encounter the lateral forces, exerted by the vertical sides of tunnel which may result the squeezing of vertical portion of tunnel. Ultimately failure of the structure.**
iii. The vertical steel (PSS) are easy to erect.	**iii. Chances of upheaval of floor.**
iv. Erection of lining shutters and lining is easy.	

5.0 Rectification work/methodology:-.

5.1 The work of rectification involved following :-

i) 28/537 to 28/513 2 4m-tunnel rectification by re- profiling.
ii) 28/624 to 28/560 64m- tunnel rectification by re-profiling.
iii) 28/707 to 28/665 42m-tunnel rectification by re-profiling.
iv) 28/862 to 28/724 138m-Tackling cavity formation using pipe roofing methodology.
v) 28/907 to 28/862 45m-tunnel rectification by re- profiling.
vi) 29/080 to 29/043 37m- tunnel rectification by re-profiling.

As already explained in the previous paras, the squeezing/heaving of tunnel in the above locations occurred on account of extremely poor geology and adoption of D-shaped tunnel profile, which is less efficient to encounter lateral forces. Further, in a D-shaped tunnel, SPL beam is the weaker part for transmitting the compressive forces coming from the crown. The SPL beam (ISHB-150) buckled at many places in the above locations. The remedial measure for this is to eliminate the provision of SPL beam during re-profiling. To tackle upheaval of bottom invert, it was proposed to provide bottom invert beams at every rib during re-profiling.

Since, the original D-shape was not able to withstand the load and the squeezing effect, it was decided to adopt a much stable profile i.e., elliptical section with permanent support system comprising of ISHB-150 @ 500mm c/c. The earlier support system was at a spacing of 750mm c/c. The proposed elliptical shape section with elephant foot is shown in **Fig. 4**.

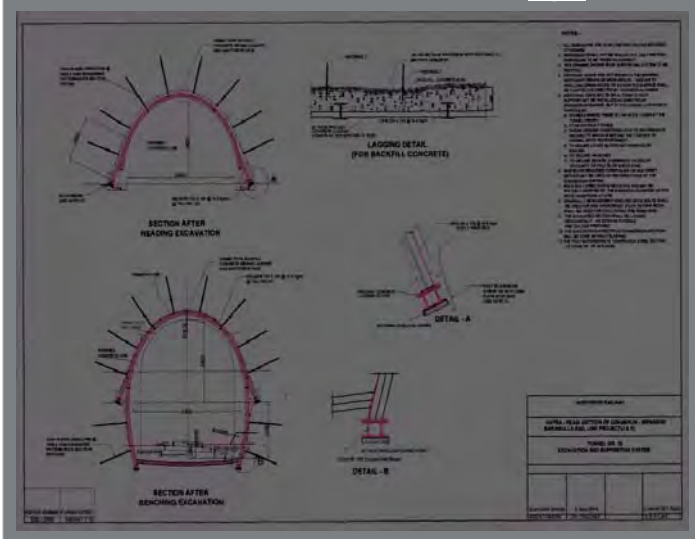


Fig. 4 Elliptical profile adopted.

5.2 Comparative merits / demerits of elliptical shaped tunnels :

MERITS	DEMERITS
i.The SPL beam is eliminated by providing elephanta foot.	i. The section is costly.
ii. The vertical part is also in curvature which provides arch action and more stability against lateral pressure of earth.	ii. Close supervision and high skilled staff is required to fabricate the PSS having the varying curvature of the shape.
iii. Upheaval pressure is also encountered due to curved bottom invert beams.	iii. The erection of PSS, lining shutters and lining etc, needs more close supervision.

5.2 Excavation work was started to rectify the collapsed D-shaped tunnel by reprofiling with elliptical shape in the above locations. Due to very poor strata, the stand up time of the excavated profile was very less. To increase the stand up time, method of pipe roofing was adopted using 114mm dia seamless pipes at a spacing of 300mm c/c. Micro-piling rig machine **(Exhibit 1)** was used for forepoling the seamless pipes for creating an umbrella. The working space required for micro-piling machine about 10m length with 750mm extra clearance at sides in a D-shaped tunnel. For creating this extra working space for micro-piling machine, excavation was done in cycles of 50cm length. Sealing shotcrete of 50mm thickness was applied after every cycle of excavation. Lattice girders were provided at a spacing of 50cm alongwith wiremesh and shotcrete of 150/200mm thick.

Exhibit 1: Micro-piling Rig used at T-10

5.3 ***Pipe Roofing Methodolog y :***

In order to increase the stand up time of excavation profile, pipe roofing methodology was adopted to provide umbrella roofing below which the work of re-profiling/excavation for tackling cavity was executed.

Pipe roofing involves the installation of a set of parallel seamless steel pipes around the contour of the tunnel. These seamless steel pipes are drilled into the crown at an inclination of 3 deg. to 15 deg. (with ref. to horizontal) in a way as to form an umbrella **(Exhibits 2 & 3)**. These seamless pipes are drilled using micro-piling rig. With this method, it was possible to cover advance lengths of 12-15m of which 10-12m are for actual excavation, maintaining an overlap of 2-3m. **Fig. 5** depicts pipe roofing methodology and **Fig. 6** the actual profile of pipe roofing at T-10.

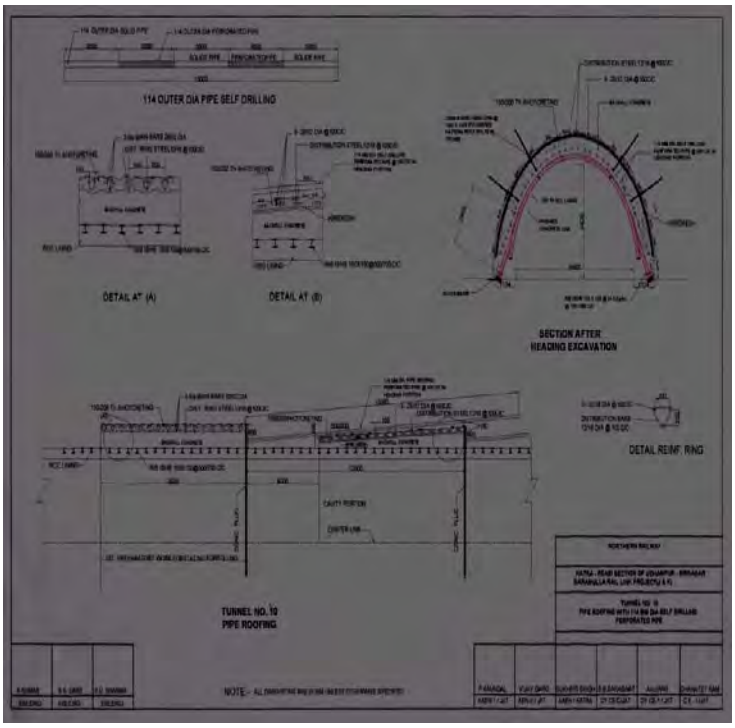


Fig. 5 Pipe roofing scheme.

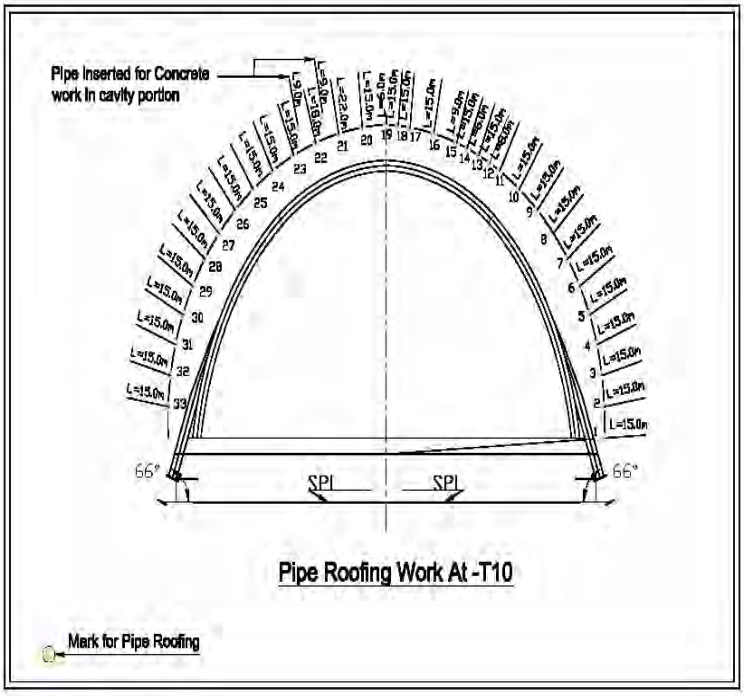


Fig. 6 Actual profile of pipe roofing.

Exhibit 2 : Pipe Roofing at T-10 in progress.

Exhibit 3 : Completed round of pipe roofing at T-10 before excavation.

5.4 Re-profiling work :

In locations where re-profiling was required, first of all efforts were made by doing re-profiling, tackling one rib at a time. Re-profiling in heading was done first followed by benching. Rib-spacing while re-profiling was reduced from 750cm c/c to 500cm c/c. After fixing ribs, RCC laggings (450 X 200 X 50) were provided and backfilling done with M-20 concrete with the help of concrete pump. Contact grouting was done to consolidate the surrounding area beyond excavation line with pressure pump applying pressures upto 5-10 kg/Sq. cm. Backfilling thickness was also increased from 150 to 350mm. The x-section is changed from D-shape to elliptical shape with haunch arrangement (elephant foot) at SPL level to increase stability, thereby, eliminating SPL beam.

In locations where stand up time of excavated earth was very less on account of poor geology, re-profiling as above was carried out after providing protection in the form of pipe umbrella using pipe roofing methodology.

5.5 Rectification of Cavity zone between Ch. 28/724 to Ch. 28/862 :-

In order to rectify and making through the tunnel in the cavity zone following action were taken:-

a. Back filling of the voids in the collapsed material in the tunnel section and upper zone over the crown with colgrout (sand cement grout)

b. Install pipe roofing with colgrout/neat cement grout before taking up heading and benching.

The length of collapsed tunnel was divided in segments of about 12m length for effectiveness of the treatment. The detailed methodology and sequence of operation is as under:

1. SEQUENCE AND METHODOLOGY.

Concrete wall of about 600 mm thick was constructed at the face of the cavity zone plugging the entire face. Then 100 mm MS casing pipes were provided in the masonry wall at the various locations as shown in the drawing below for drilling and colgrouting. 50mm PVC pipes were provided in the masonry wall as shown in the drawing as grout relief holes.

2. DRILLING.

100mm horizontal holes were drilled through the preplaced casing pipes for the depth of about 12m and 75mm perforated MS pipe was provided in the casing .Similarly all grout hole locations as shown in the drawing were dilled and perforated pipes were installed in them. Finally, water was injected through the perforated pipes at nominal pressure of about 2kg/c2 in sequence commencing from the bottom row and moving up for about 10 minutes at each location so as to ensure that area around perforated pipes are clear and no holes are blocked.

3. COLGROUTING

After ensuring that all holes are clear off and there is no blockage, colgrouting of the drilled holes were commenced from the bottom row, moving to upward rows after ensuring completing colgrouting of all holes in that particular row.

In all the holes, initially 3 batches of neat cement grout of 1:1 water: cement ratio was pumped for lubrication and followed with the final formulation of 1:3 cement : sand , consistency, colgorut. The sand used should be of size 4mm and below, free from all dust particles and foreign materials.

Delivery pipe of the colgrout pump was connected to one of the holes at the bottom row and the colgrout of the given consistency was pumped into it. Pumping continued till refusal or grout starts overflowing from the relief holes provided or from the adjacent grout hole. In the exceptional circumstances, if the colgrout intake was more than about 40% of the theoretical volume of the influence area at that location, further pumping in that hole was suspend, pumping was done in next hole as per the pattern suggested above. Re grouting was done in the suspended hole after 4 hours till the overflowing of grout material from relied holes. Similarly all holes drilled on the face were grouted.

4. PIPE ROOFING AND GROUTING/COLGROUTING.

After completion of the colgrouting of the body of the tunnel, pipe roofing was taken up.

Pipe roofing with 114 mm dia perforated pipes was installed over the crown of the tunnel at a inclination of 5° to 10° upward from the face of the wall. Pipe roofing was spaced at 300/500 mm C/C and was installed upto the depth of about 12 to 13 m.

Initially alternate pipes were installed to required depth which was further grouted with neat cement grout of 1:1 consistency. After installation of the first series of pipe roofing the second series (the holes in between the pipes already installed) of pipe roofing was installed upto the required depth & grouted with neat cement grout (Exhibit 4).

5. Excavation.

On complete installation of required pipe roofing and grouting in heading at required locations, the excavation was done by taking a patch of 3-4m at a time depending upon the ground conditions. The work of tunnel reprofiling was carried out in these patches of 3-4m by repeating the procedure as detailed in para 5.5 above.

Exhibit 4 : Concreting in the cavity portion in progress

6.0 Machinery Used:-

a) Drilling Grout Holes : Atlas Copco Crawler mounted hydraulic drill ROC 203
b) Drilling pipe roofing: Atlas Copco Crawler mounted hydraulic drill ROC 203.
c) Colgrouting : Atlas Copo Mai Pump capable of pumping colgrout at 5 to 6 Kg/c2.
d) Excavator: - Tata Hitachi 110 with Rock breaking attachment.
e) Dumpers:- Tata 9 ton capacity
f) Transit Mixers: - 5 cum capacity.

7.0 Present status of work :-

Out of a total length of 446m which required rectification work, the work in 221m length including cavity rectification work has been completed in the present tender. The balance work of rectification in 225m length and concrete lining work is being planned through a separate tender.

SWELLEX ROCKBOLTS

Friction grip rockbolts provide an efficient, faster and safe rock support solution which help in shortening of cycle time and immediate support of rock mass. There are a number of friction grip rockbolt systems in market now a day which can be used depending upon cost economics and technical requirement. In order to have an understanding of this system for tunnel engineers, the system brand named as Swellex Rock bolts developed and manufactured by Atlas Copco is explained below. Readers are advised to verify the technical details and explore other products in market depending upon the requirement before taking final decision.

Swellex rockbolts are made up of collapsed tubes which expand through the use of water pressure after inserting into the already drilled holes. They are extremely simple to install.

Developed and marketed by Atlas Copco Construction and Mining Ltd. in Sweden, the 'Swellex' system is illustrated in Figure 1. The dowel consists of diameter tube which is folded during manufacture that can be inserted into a already drilled 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.

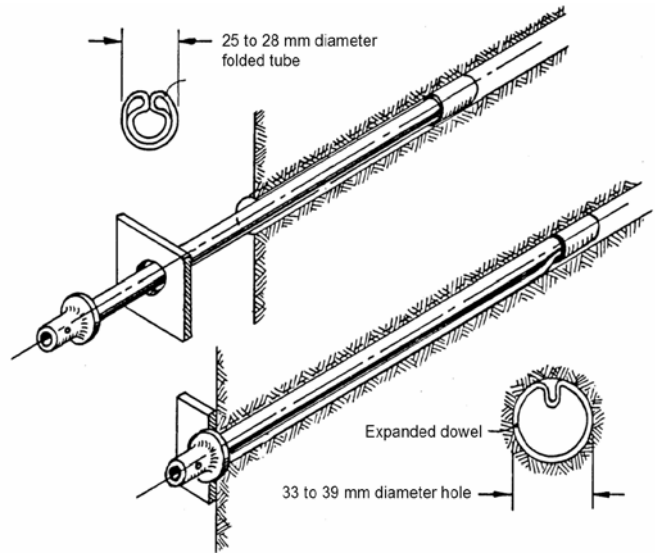


Figure1: Details of the Swellex frictional anchor (Atlas Copco Construction and Mining Ltd.)

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

Vinod Kumar
Dy CE/N/USBRL

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 tends to be less than that of alternative reinforcement systems, when installation time is taken into account. This system is ideal for use with automated rockbolters.

Swellex 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 also provides different versions of corrosion protected Swellex bolts such as: -

(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.

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.

Some of the technical details of Swellex Rockbolts from Atlas Copco are as under: -

SWELLEX ROCKBOLTS

	Pm 12	Mn 12	Pm 16	Mn 16	Pm 24	Pm 24C	Pm 24H	Mn 24	Sp 12	Sp 16	Sp 24
Minimum breaking load, expanded profile (KN)	110	110	160	150	240	240	200	220	100	160	240
Minimum yielding load, expanded profile (KN)	90	90	140	100	200	200	190	180	90	140	200
Minimum elongation "A"(%)	10	20	10	20	10	10	10	20	6	6	6
Profile diameter (mm)	28	28	36.5	36	37	37	37	37	28	37	37
Material thickness(mm)	2	2	2	2	3	3	3	3	2	2	3
Original Tube Diameter (mm)	41	41	54	54	54	54	54	54	41	54	54
Upper Bushing Diameter (mm)	28	28	38	38	38	38	38	38	28	38	38
Bushing Head (mm)	30/37	30/37	41/50	51/50	41/48	41/53		41/50	30/32	41/50	41-58

SWELLEX ROCKBOLTS

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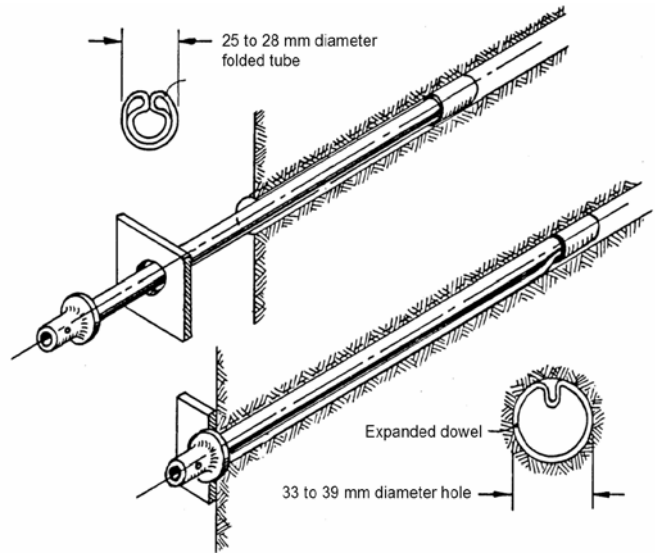

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Atlas Copco also provides different versions of corrosion protected Swellex bolts such as: -

(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.

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.

Some of the technical details of Swellex Rockbolts from Atlas Copco are as under: -

SWELLEX ROCKBOLTS

	Pm 12	Mn 12	Pm 16	Mn 16	Pm 24	Pm 24C	Pm 24H	Mn 24	Sp 12	Sp 16	Sp 24
Minimum breaking load, expanded profile (KN)	110	110	160	150	240	240	200	220	100	160	240
Minimum yielding load, expanded profile (KN)	90	90	140	100	200	200	190	180	90	140	200
Minimum elongation "A"(%)	10	20	10	20	10	10	10	20	6	6	6
Profile diameter (mm)	28	28	36.5	36	37	37	37	37	28	37	37
Material thickness(mm)	2	2	2	2	3	3	3	3	2	2	3
Original Tube Diameter (mm)	41	41	54	54	54	54	54	54	41	54	54
Upper Bushing Diameter (mm)	28	28	38	38	38	38	38	38	28	38	38
Bushing Head (mm)	30/37	30/37	41/50	51/50	41/48	41/53		41/50	30/32	41/50	41-58

Training on Tunneling Technology at NIRM/Bangluru and NGI/Norway

Introduction

Railway Board nominated a group of 20 Railway Officers for training on Tunneling Technology. This training was coordinated by NIRM (National Institute of Rock Mechanics), Bangluru from 15.5.14 to 30.5.14 at NIRM/Bangluru and 2.6.14 to 11.6.14 at NGI/Norway.

This paper has been prepared covering the training portion at NGI/Oslo, Norway. At Norwegian Geotechnical Institute (NGI), Oslo Sh. Rajinder Kumar Bhasin. was the training coordinator

NORWEGIAN GEOTECHNICAL INSTITUTE

Norwegian Geotechnical Institute (NGI) is a leading international centre for research and consulting within the geosciences. NGI offers expertise on the behavior of soil, rock and snow and their interaction with the natural and built environment.

NGI is a private foundation with office and laboratory in Oslo Norway, branch office in Trondheim and daughter companies in Houston, Texas, USA, and in Perth, Western Australia. NGI was awarded Centre of Excellence status in 2002.

The core competence is within geotechnics, engineering geology, hydrogeology and environmental geotechnology, with expertise within geomaterial properties and behavior,

numerical modelling and analysis, and instrumentation and monitoring. NGI's strength lies in the expertise of its personnel working in collaboration with clients and cooperating partners, to find practical solutions for the industry and society. Geotechnical Engineering laboratory of NGI is the best in the world.

SUNIL BHASKER
Dy.CE/S&CI/UHP
USBRL Project

NGI works within the following sectors:
(i) Offshore energy;
(ii) Building, Construction and Transportation;
(iii) Natural Hazards;
(iv) Environmental Engineering.

NGI has the honour to be the custodian of the Terzaghi and Peck Libraries. All academic material from Karl von Terzaghi and his colleague Ralph B. Peck has been given to NGI to be available for interested researchers.

NGI was the host of the International Centre for Geohazards (ICG) from 2003-2012, one of Norway's first Centres of Excellence (CoE). NGI's partners were NORSAR, the Norwegian Geological Survey (NGU), the University of Oslo (UiO) and the Norwegian University of Science and Technology (NTNU).

The training module for Indian Railway Officers at NGI was divided into two parts: -
(i) Classroom training;
(ii) Field Visits

CLASSROOM TRAINING

The various topics covered are as under: -

(b) **Earthquake Effects on transportation network: -** This chapter was covered by Amir M.Kaynia, Discipline Leader Earthquake Engineering. The earthquake risks to infrastructure (Roads, Bridges and viaducts, Tunnels in soil & rock), typical damages, analysis tools with examples were discussed.

(c) **The effect of Earthquakes in underground structures & optimization of rock support in tunnels with case histories:-** The speaker for this topic was Mr. Rajinder Kumar Bhasin, Senior Geologist, Building, Construction and Transportation, Rock Engineering from Norwegian Geotechnical Institute. In this topic, the advantages of applying the numerical techniques, with examples of optimizing the rock support for a subsea road tunnel in west of Norway, Froya Hitra Tunnel, numerical modelling of world's largest rock cavern (Olympic Rock Cavern), Stad ship Tunnel etc. were discussed.

(a) **Principles of Norwegian Method of Tunneling (NMT): -** Mr. Eystein Grymstad was speaker for this topic. Principles of Norwegian Method of Tunneling (NMT), preinvestingations, Rock Mass classification in tunnels, difference between, Norwegian Method of Tunneling (NMT) & New Austrian Method of Tunneling (NAMT), excavation sequences, Rock Support System, Q system etc. were covered in this session.

Sprayed concrete robot built in Norway

Four boom drilling jumbo

(e) **Numerical tools for design of rock support in tunnels (Cetin Dam Project): -** The Çetin Dam is a rock-fill dam with an asphalt-concrete core, currently being constructed on the Botan River in Siirt Province, Turkey. The dam will be located directly downstream of the Botan and Büyük River confluence and have a height of 145m (476ft). The primary purpose of the dam is hydroelectric power generation. The numerical simulations done for this project were discussed.

(f)**Introduction to the Q-system of rock mass classification: -** Arnstein Aarset, Technical Leader Engineering geology and rock slide, Building, Construction and Transportation, Rock Engineering at Norwegian Geotechnical Institute was the speaker for this topic. The Q-system of rock mass classification alongwith its development was covered in detail. Its use in Strindheim highway tunnel, Espa road tunnel & New underground railway station, Holmestrand, Norway was also discussed.

Reinforced ribs of sprayed concrete

(d) **Geophysics: -** Dr. Andreas Aspmo Pfaffhuber, Head of Section, Natural Hazards, Geosurveys was the speaker for this subject. Ground geophysics with resistivity tomography, Geotechnical AEM (Airborne Electromagnetic Surveys), case studies, opportunities and limitations were discussed.

FIELD VISITS

Various field visits were arranged by NGI to project sites in Norway in near vicinity of Oslo. Details of these visits are as under:

(a) NORWAY'S UNDERGROUND OLYMPIC STADIUM

The 62.0 m span Ice Hockey Cavern has been constructed beneath a hillside in Norway for the 1994 Winter Olympics. This Ice Hockey Cavern is located in Gjovik city, some 25 km south of Lillehammer which was the centre of the 1994 Winter Olympic Games in Norway. The Ice Hockey Cavern has a finished span of 62m, a length of 91m and a height of 24m with spectator capacity of 5300. The cavern has only 25-50m overburden. An existing underground swimming pool, telecommunications facility and civil defense facility are located adjacent to it. The work was executed by Veidekke-Selmer Group, a joint venture of Norway's two main tunneling firms. About 170 tons of explosives was used to blast and remove approximately 1,40,000 cubic meter of rock or 1.2 Kg explosive per cubic meter of rock.

THE 61 M WIDE CAVERN WILL HOUSE AN ICE HOCKEY RINK WITH A SEATING CAPACITY OF 5,300 THAT WILL BE USED FOR THE 1994 WINTER OLYMPIC GAMES IN NORWAY.

The average span width for a man-made cavern is 15-25m, so building a cavern of over 60m posed some exciting challenges. The cavern was constructed by using Norwegian method of Tunneling (NMT) in contrast to the New Austrian Method (NATM), which is most suited to soft ground that can be machined or hand excavated. NMT is designed for hard ground where jointing, over break and loosening are dominant. Rock support is in the form of rock bolts and fibre reinforced shotcrete rather than the rigid steel sets or lattice girders, which are inappropriate in Norway's harder rocks.

The detailed series of investigations involving geological mapping and core drilling, laboratory testing, geophysical measurements, stress measurements and numerical analysis were carried out by Norwegian Geotechnical Institute (NGI) in Oslo, the foundation for Scientific and Industrial Research (SINTIF) in Trondhein, Norway and consulting engineers Noteby, also located in Oslo. The Rock Cavern was designed by Fortifikasjon, Oslo, Norway.

The Q-system, a rock mass classification method developed by NGI, was used extensively to obtain and derive the geotechnical parameters needed to predict the behavior of the rock mass. The typical Q-range was 1-30, for a mean Q-value of 9.4. Data from boreholes showed slightly higher rock quality than was actually found in the cavern during construction. This was probably due to blasting, which creates artificial joints and opens up healed joints. The rock in the cavern is red and gray jointed gneiss of Precambrian age.

The jointing in the cavern is irregular and rough walled with quite large variations in dip and strike. These jointing properties, alongwith rather high horizontal stresses of about 3.5-4 Mpa 45 m below the surface and perpendicular to the long axis of the hall, makes the cavern very stable provided that the appropriate reinforcement is done.

Excavation and drilling began in March 1991 and was completed by December 1991. Blasting commenced with the driving of two separate access tunnels of 20 and 45 sq. m cross section from opposite sides of the long axis of the hall. Several deformation and stress measuring instruments were installed. Alongwith six multiple-position borehole extensometers installed from the surface prior to construction, these instruments formed a network of devices to record future rock deformation and settlements during construction.

After the top heading, which also served as the exploratory tunnel, was completed, the contractor blasted two 14m wide sloping lateral tunnels on opposite sides of the heading. The 36m span top heading was completed. This induced only 1-4 mm deformation, mainly due to high horizontal stress. The temporary support consisted of 27 mm diameter, 4m long expansion shell bolts with a capacity of 10 tons. Permanently bolting consisted of 25mm diameter, 6 m long rebar bolts of 22 tons capacity, arranged in a 2.5m by 2.5m pattern and 12 m twin strand cables, each 12.5mm in diameter and with 16.7 tons of yield strength, arranged in a 5.0m by 5.0m pattern. Both bolts and cable strands were untensioned and fully grouted.

Fig 1: - Entrance Gate

The contractor drove the main access tunnel separately from the bottom level of the caver, then blasted two access tunnels sloping up to the abutment tunnels on each side of the hall. The remaing part of the work consisted od blasting the rock pillars that stood between the sloping lateral tunnels and the abutments. Permanent rock support works at the roof were completed before finally blasting the bench. Wall support consisted of 3 m and 10 m long untensioned grouted bolts with varying thickness of steel-fiber-reinforced shotcrete.

Fig 2: - Inside View

Fig 3: - Rock Type

(b) **VISIT TO HOLMESTRAND STATION SITE**

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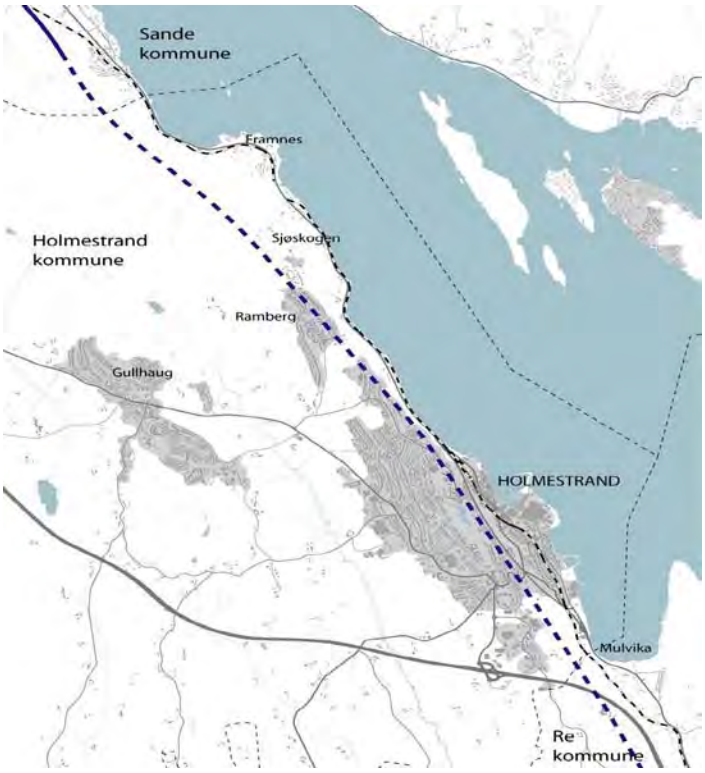
The Holmestrand station is being built as underground station as doubling of Holm - Nykirke Project
It is one of the 3 major projects on the Vestfold line of Norway.

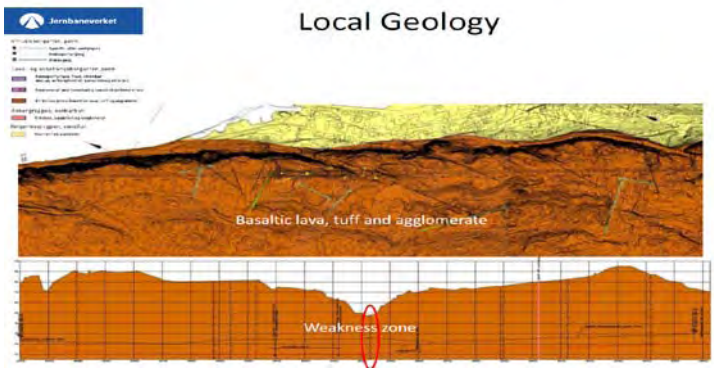
Holmestrand - Nykirke Project

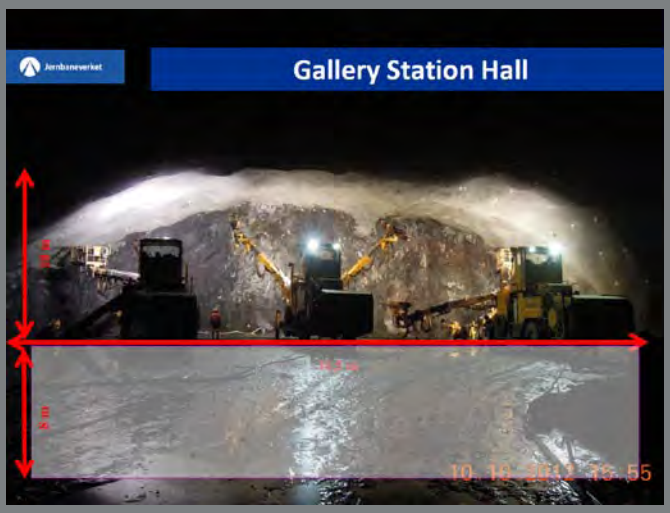


- Total length 14,2 km
- 12,3 km tunnel
- 11 escape tunnels
- 2 entrances to the new underground Holmestrand station
- New public transport terminal
- $2 \cdot 10^6$ solid m^3 rock excavated (drill & blast)
- Built for speeds up to 250 km/h
- 5 tunnel contracts
- Cost 6.2bn NOK (approx. $1bn) (management framework 5,3bn NOK)
- Planned opening date 2016

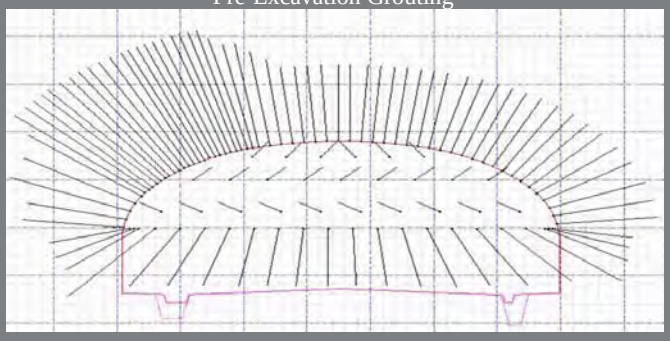
UHN-04 Station Contract is being executed by Skanska.
Main quantities of work in new underground Holmestrand station are as under:

- **Excavation of main tunnel and station cavern** **555,050 solid m^3**
- **Excavation of access tunnels etc.** **33,390 solid m^3**
- **Rock bolts** **40,485 pieces**
- **Shotcrete** **28,000 m^3**
- **Rock Grouting** **22,965 tons**
- **Formwork** **40,816 m^2**
- **Rebar** **1,856 tons**
- **Concrete** **12,570 m^3**



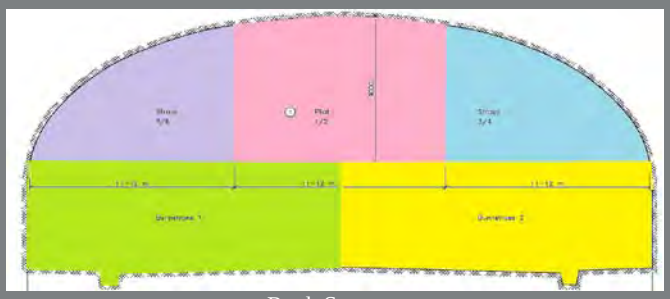


Pre-Excavation Grouting

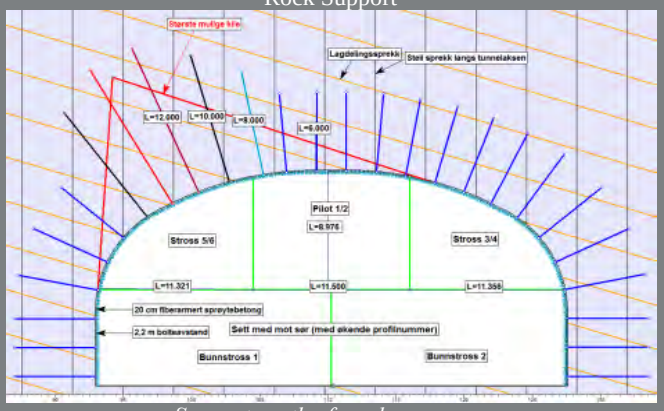


Systematical pre-grouting in the station hall with a grouting cover every 10.meters
Leakage requirement= 5 l/min/100 meter
Cement based grouts – Mostly standard Portland cement
High pressure – up to 80 bar

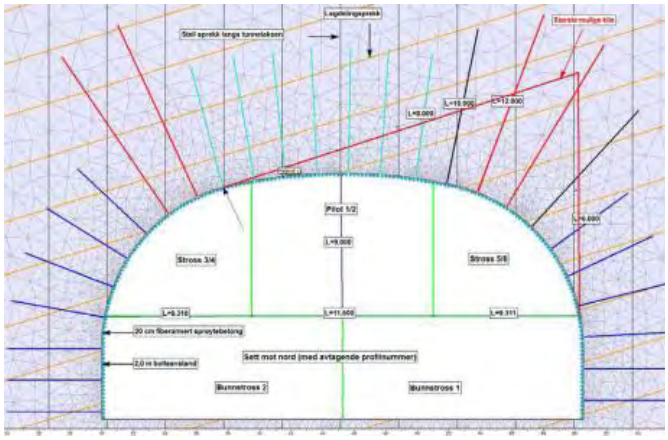
Blasting Sequences Station Hall



Rock Support

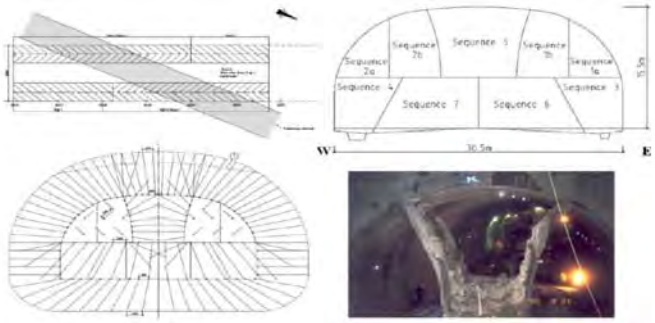


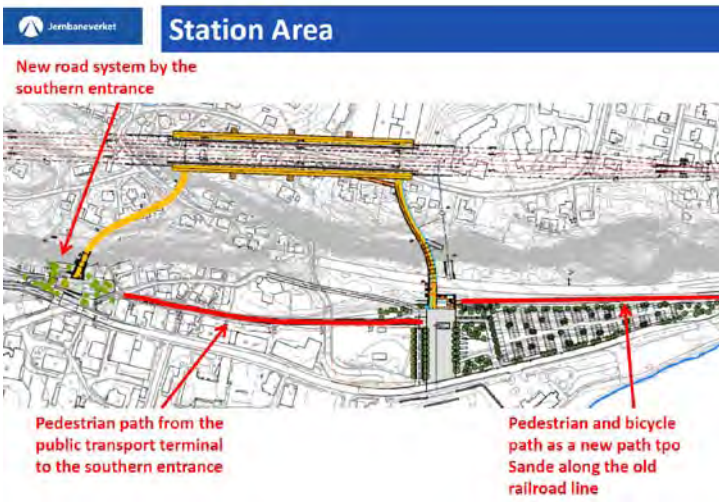
Support north of weakness zone:
Rock bolts 6-12 m CT-bolt & dywidag-stag c/c=2,2 m
Shotcrete w/fibre – minimum thickness 20 cm

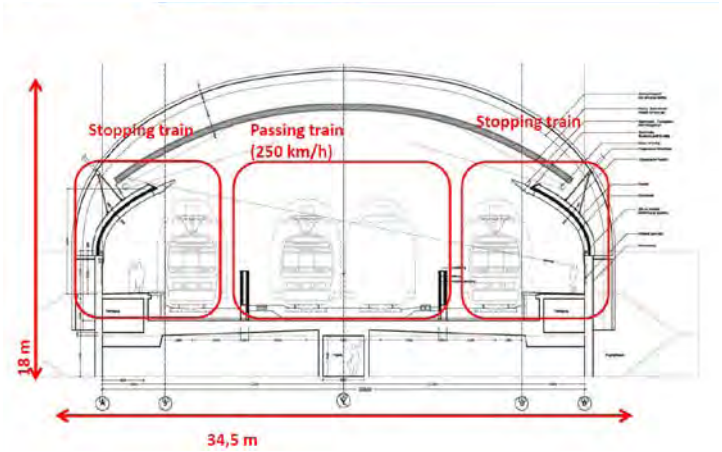


Support south of weakness zone:
Rock bolts 6-12 m CT-bolt & dywidag-stag c/c=2 m
Shotcrete w/fibre – minimum thickness 20 cm

Weakness Zone – Planned Excavation







Cross Section of the State of the art Holmestrand Railway Station
Area = 487 m^2

DEEPAK SINGH (IRSE)

I, Deepak Singh was born in Meerut district of Uttar Pradesh. After completing my schooling from Meerut, I joined Indian Institute of Technology, Roorkee (IIT-Roorkee) for pursuing B.Tech in civil Engineering. After passing out from IIT in 2005, I joined Bharat Heavy Electricals Ltd (BHEL) and worked there for 3 years before clearing UPSC Engineering examination 2007 and thereafter joining Indian Railways.

After successfully completing training in railways, I joined as Asstt Divisional Engineer, Jind in Delhi Division of Northern Railways where I served for almost 2 years until my posting in prestigious USBRL project at Sangaldan in 2012.

Here in Sangaldan, I have been entrusted with the task of supervising tunneling works, bridge works & Station yard civil works being executed by KRCL through AFCONS. Besides, land acquisition proceedings also forms a part of my job profile. Being a part of such a big and prestigious civil project and that too in such a remote and sensitive place in itself is a truly learning and enriching experience and has tremendously added to my knowledge and work experience and is definitely going to benefit me in the years to come.

Hobbies: Sketching

Technical field of interest: Fluid Mechanics

SLOPE STABILITY OF SANGALDAN AREA PORTALS – T42P1 A CASE STUDY

1.0 General

The tunnel alignment from RD 92.360 to RD 101.300 comes under is Geological slope debris material/ slope wash material. Alignment comes with numbers of tunnels and bridges. Tunnels T41, T42, T43, T44- T45, T46, T47 are coming through this lithology. The portals in between these tunnels are coming under Murree formation and demands good stabilised portals decorated with slope stabilised measures. The 950m long Sangaldan Station is located between km 92.360 and km 93.310 is part of the USBRL project. It involves 17m to 55m height of cutting and it involves slope stability for hills, the portal T42P1 & T41P2 with a limited encroachment. As a case study the portal T42P1 is considered for discussion.

2.0 **Geological Parameters:**

Sangaldan area mainly comes under Murree formation. Murree formation is defined with alternate band of clay stone, siltstone and sand stone. However in many case the portal is situated in material seems to be slope wash material with defined geological parameter. The sedimentary rocks of Murree Formation and metamorphics of Ramban Formation are tectonically separated by a thrust i.e. Murree Thrust. The Murree Thrust passes through tunnel T47. Considering the nature of material the slopes are designed to give maximum safety. Due to folding/shearing and presence of Murree thrust rocks units are closely fractured, shattered and broken into small pieces (especially sand stone). With the passage of time these rocks have undergone the process of weathering, erosion and deposited on underlying rocks. In the course of time slided rock material partly thrusted became within soft and weathered clay stone/siltstone. Due to this rock mass has become soft and at places appear like overburden.

3.0 Design Parameter Geotechnical investigation

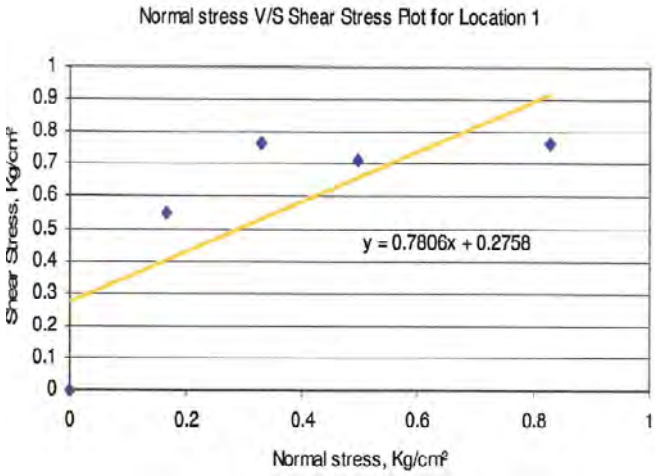
For the construction of portal with the safety consideration all the portals are designed with the geology and geotechnical parameters.

3.1 ***Geotechnical P arameter*:** Several geotechnical tests were conducted by KRCL through CSMRS, New Delhi. Shear strength parameters of Rock Mass is achieved through numbers in situ shear test. The averaged values of cohesion (c) and angles of friction (ϕ) at location were found to be 0.31 kg/cm^2 and 38.2^o respectively. Similarly the cast in situ footing load test were conducted. The cast in situ footing size was kept 150x150x35cm. The bearing capacity and the settlement of the foundation were determined with the help of load settlement curve for actual footing. The ultimate load carrying capacity of formation is 13.33 t/m^2. A numbers of bore log were carried to achieve the rock types and its property.

Fig. 6: Test setup

Fig. 7: Location 1 - chainage 92/612.427 92/624.568

Fig 1.0 Geotechnical Investigation at Sangaldan Yard



Prem Sagar Gupta
CE/Design/KRCL

Rashmi Ranjan Mallick
Dy.CE/Design/KRCL

No.	RL	Chainage	c (kg/cm^2)	φ (degree)
1	1240	92612.427 to 92624.568	0.28	38.0
2	1251	92744.473 to 92756.534	0.31	41.8
3	1270	92853.212 to 92865.248	0.38	31.8
All Tests (13 Nos.)			0.31	38.2

Fig 2.0 Normal stress vs shear stress plot at Sangaldan Yard

3.2 ***Water ta ble***: Seven numbers piezometers are installed between RD92.360 to RD93.294. The slope is analysed with the defined applied water head.

3.3 ***Earthquake Parameters***: Historical recordings reveal that the area is seismically very active and the site lies in seismic zone V as per the seismic zoning map of India (IS: 1893, Part 1).

The minimum factor of safety during normal and earthquake condition shall be considered as 1.5 and 1.2 respectively for optimum designs. However as BIS7894, the minimum Factor safety governs 1.0 for Earthquake condition.

4.0 Design of portals:

4.1 Geometry of T42P1: The portals T42P1 is situated at RD 93.313 and with defined/restricted boundary at. The station is subjected with two loop line infused 70m into the hillock and emerging a cavern inside and forming portal T42P1. To accumulate within the defined boundary the slope is suggested as under (Fig 1.0). The slope was day lighted. It was treated with cladding with defined designed parameter. The slope is designed again to give maximum safety considering the entire imposed load.

4.2 Analysis: With geological data interpolation it was quite understood the possibilities of failure type. There is no defined rock joint pattern and it is a continuum type approach.

a. The possibility failure is Flow/slip Circular type. The slope is analysed with Bishop's and Janbu's failure criteria.

b. In analysing the potential of slip, one has to consider the location and determination of the factor of safety for a given slip surface. In practice the factor of safety is determined for assumed slip surface locations. The technique used to analyze these failures is a limit equilibrium technique called the 'method of slices'. The method of slices involves the discretization of the slip surface into elements, each of which has normal and shear forces applied to it. Each element has three unknowns associated with it: the normal (N) and shear (S) forces, and the location of the line of action of the normal force relative to the element itself (n). There is 3n number of unknowns. The reduction of the number of unknowns is usually done by sub-dividing the mass under consideration into 'slices', and analysing each slice on the basis of limiting equilibrium, i.e. each N and S is linked through the strength criterion of the slip surface. Examination of a typical slice with the various forces applied to it. The analysis of the factor of safety 'F' of the entire mass then depend on whether the slip surface is generally non-circular, or specifically truly circular. To account all these, a rigorous analysis SLIDE software of Rocscience used.

c. To account the stability the longitudinal section portal with greater height of cutting is selected at the centre of railway alignment (fig 3.0). It has been analysed with numbers different condition of cohesion and friction angle (ϕ) variables of cohesion and friction angle. Due to loop line some additional area is encroached and buttress is created to give maximum safety to slope. Buttress is created with compacted backfill ahead of slope. To retain this buttress and to give station yard an embossing outlook, a varying retaining wall is provided with an enhanced factor of safety.

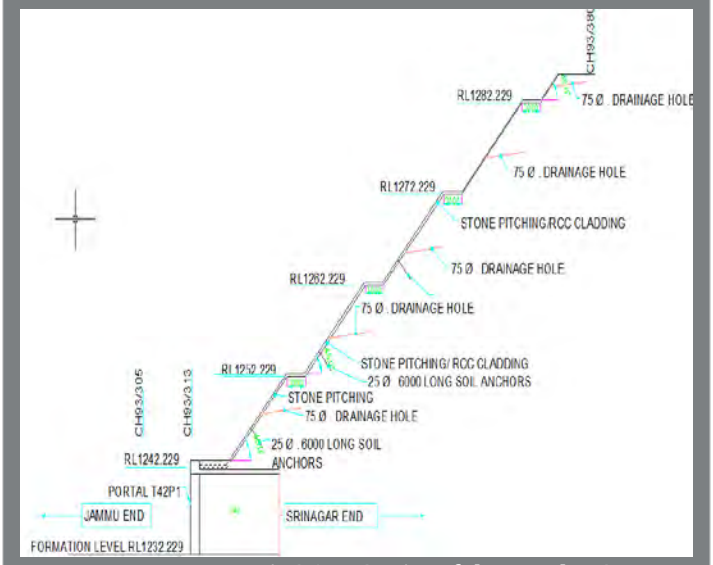


Fig 3.0: L Section of the portal T42P1.

4.3 SLIDE Analysis:

In this T42P1 slope, it is necessary to determine which subset of the infinitely possible failure surfaces is "critical". Critical failure surfaces are those that have low FS (generally <= 1.2 for Earthquake and <= 1.5 for normal condition as per DBN), and are large in size or in a sensitive location. In the case slope analysed the FOS comes at critical slip are 1.55 and 1.27 for general and Earthquake condition.

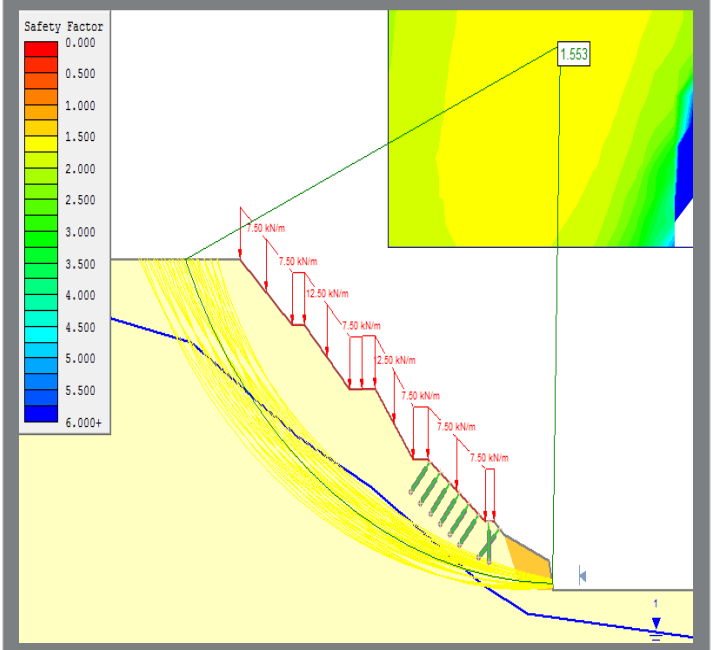


Fig. 4(a): Analysis of Slope during Normal Condition

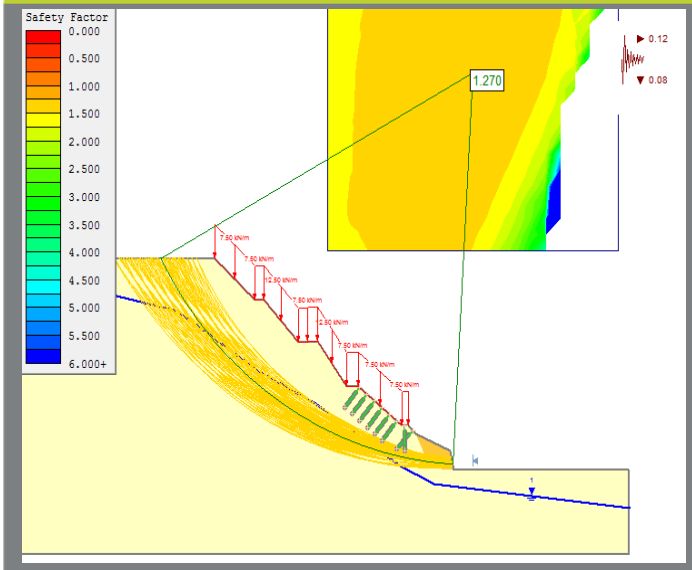


Fig.4 (b): Analysis of Slope during Earthquake Cond.

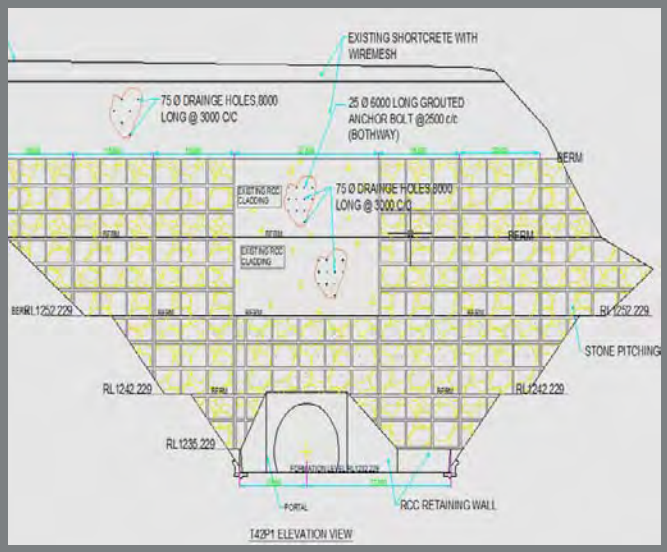


Fig. 5(a): T42P1 Elevation View

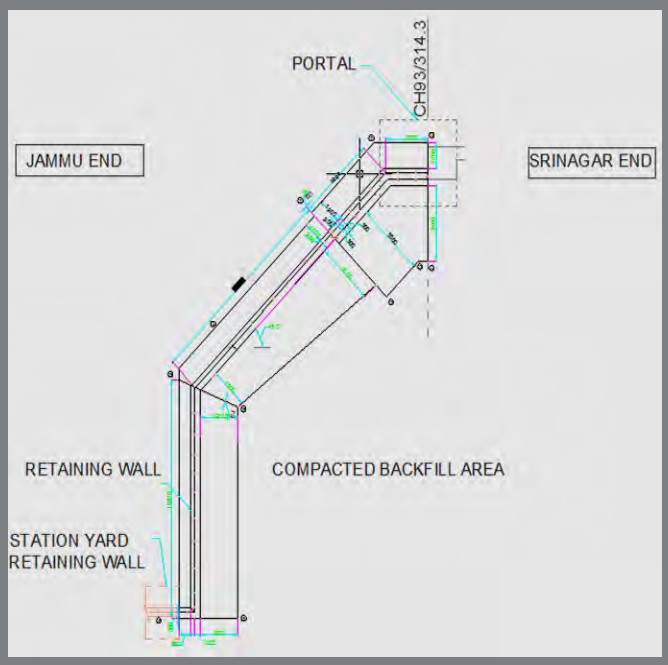


Fig. 5(b): T42P1 Right Flair

5.0 Additional Safety Features

- Stone pitching is provided in a grid of 5mx 5m. Cemented Stone pitching restricts surface erosion. These stone pitching is provided above the aggregate filters to facilitate no trap of surface water above the slope. An half cut additional drains provided in each bay to release water trapped between surface and stone pitching. The slopes face is provided additional long drains to improve slope stability.
- In addition to stone pitching RCC retaining wall is provided to limits toe failure. The additional factor of safety is observed. Varying height RCC retaining wall is provided.
- Though the slope is stable without Rock bolting additional soil nailing are provided to encounter any small slides.

Slopes shall be will be monitored on regular basis. Drains are provided at top and at each berm levels.

Concreting In Cold Water (Analysis and Solutions)

SYNOPSIS: -

Profuse use of concrete in construction industry is well known to even a layman. It was nearly four to five thousand years back when different civilizations in different parts of the world started using crude concrete in construction of their buildings but real technical approach of its use came into being only in nineteenth century when John Smeaton in 1793 produced hydraulic lime and Joseph Aspdin in 1824 invented Portland cement. Since then development in concrete has been manifolds. Today hardly any structure can be imagined without use of concrete in one way or another and civil constructions in USBRL, our prestigious National Project are no exceptions. The execution of this construction project has not only to pass through tough terrains of the valley but also has to confront extreme cold weather for about quarter period of a year. The concreting during these three months i.e. December, January and February when mercury dips even below zero (see Fig.3) becomes a challenging task for the engineers. This article aims at analysing deep rooted reasons for it and devising pragmatic strategies to get a sound concrete

Fig.1 Joseph Aspdin invented cement

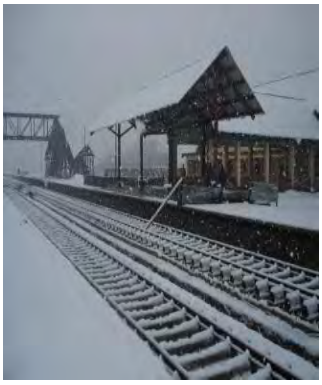


Fig.2 Photograph of a snow-clad Station

Introduction:-

Concrete, in the broadest sense, is any product or mass made by using cementing medium. Cement-Concrete is one of the types of concrete which uses cement-water paste as cementing medium. This paste on hardening binds coarse and fine aggregate together to give a desired shaped conglomerated mass with a capacity to bear designed loads. The quality and strength of concrete produced besides directly correlated to various internal factors also depends upon ambient temperature. This article will restrict its discussion on concreting during cold weather i.e. during low and very low temperatures. The study is proposed to be carried out under following four headings:

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a) Chemistry of setting and hardening
b) Effect of Cold Weather
c) Strategies and Methods for concreting in cold weather
d) Suggestions & Conclusions

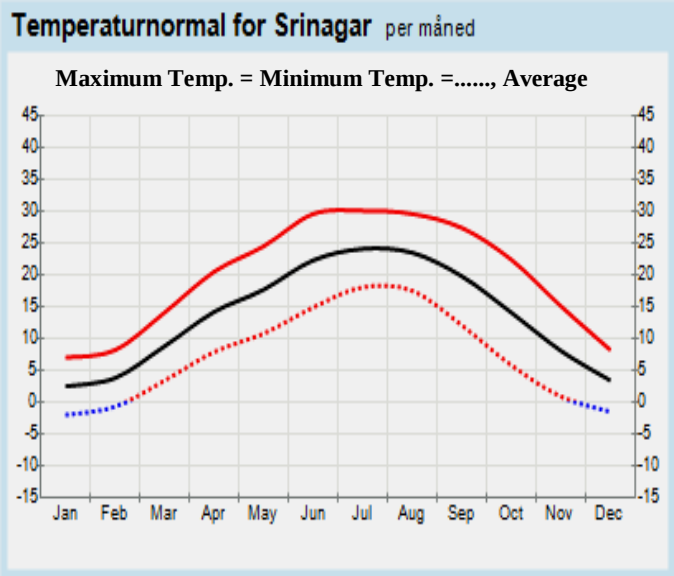

Fig.3 Monthly Temperature record of Srinagar

A) CHEMISTRY OF SETTING AND HARDENING:-

CEMENT COMPOUNDS IN DRY STATE:

In dry state cement consists of four main compounds; commonly called **Bogue's compounds**: **C_3S (**Tricalcium silicate), **C_2S (**Dicalcium silicate), **C_3A (**Tricalcium aluminate**)** and **C_4AF (**Tetracalcium aluminoferrite**)** where C= Cao, S= SiO_2, A=Al_2O_3, F=Fe_2O_3, H=H_2O

PROCESS OF HYDRATION:

Reaction of cement compounds with water is called hydration which is carried over by two primary mechanisms:

i) **Through solution** – Cement compounds dissolve to produce a super-saturated solution from which different hydrated products get precipitated.

ii) **Solid-state hydration** – Water attacks directly at the surface of the cement compounds and proceeds to interior of them.

PRODUCTS OF HYDRATION:

i) C-S-H Gel: Although **C_3S** is more reactive than **C_2S** but both reacts with water to give a product called tobermorite gel or simply C-S-H gel. This gel which is made of poorly formed and infinitely small thin fibrous crystals contains about 50-60% of Calcium Silicate Hydrate(C-S-H) and 20-25% of $Ca(OH)_2$. The C-S-H i.e. Calcium-Silicate-Hydrate ($CaO.SiO_2.H_2O$) is desirable from strength point of view whereas $Ca(OH)_2$ produces deterioration and is thus undesirable. As gel consists of crystals so it is porous having about 28% pores.

ii) Ettringite or monosulphate: The other two compounds **C_3A** & **C_4AF** contributes little to the strength and that too at the early stage of hydration. Both of these produce **ettringite** (Calcium aluminate trisulphate hydrate) during the first hour of hydration. Ettringite which has short prismatic needle shaped crystals becomes unstable with gradual depletion of sulphates and is converted into monosulphates, the final product of hydration.

Formula	Common Name	*General % age	Reaction Rate	Heat Liberation	C-S-H		Strength
					Quantity	Quality	
C_3S	Alite	50-75	Moderate	High	Less	Poor	High
C_2S	Belite	05-20	Slow	low	more	good	Low initially, high later
C_3A	Celite	0-15	Fast	Very high	-	-	Low
C_4AF	Felite	0-15	moderate	moderate	-	-	Low

***The percentage of these compounds in cement depends upon proportion of raw material used in manufacturing.**

Strength: Generally **C_3S** contributes most to the strength during the first four weeks and **C_2S** influences later gain in strength. At the end of one year two compounds contribute equally. **C_3A** & **C_4AF** contributes little. This is shown in adjoining Fig.4.

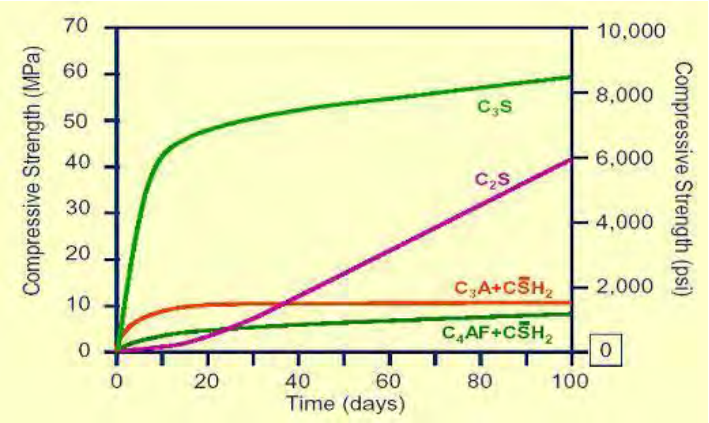

Fig.4: Gain of compressive strength by compounds of cement

Degree of Hydration: Hydration process is fast at start and then gradual at such slow speed that 100% hydration may not be completed in a year time. For instance, after 28 days in contact with water, grains of cement have been found to have hydrated to a depth of only 4 µm and 8 µm after a year. Reaction of Celite(C_3A) with water is very fast, so it has high degree of hydration. Degrees of hydration for compounds of cement are as shown in graph in Fig.5.

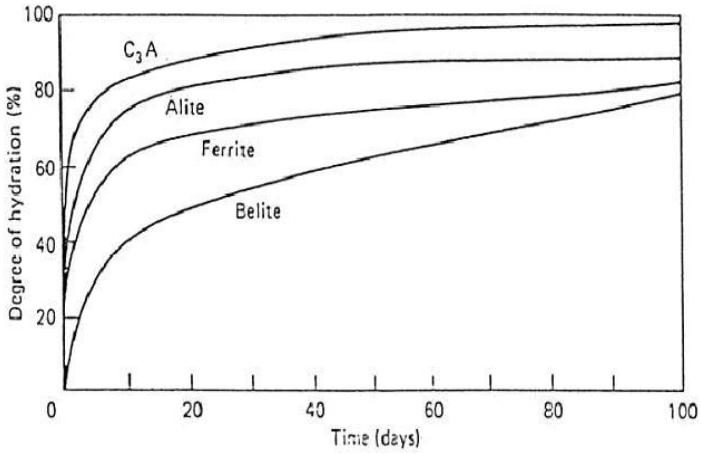

Fig.5: Degree of Hydration of compounds of cement with time

PHYSICAL BEHAVIOUR OF HYDRATION:- The formation of the C-S-H crystals provides "seeds" upon which more calcium silicate hydrate can form. These crystals grow thicker making it more difficult for water molecules to reach the un-hydrated tricalcium silicate (C_3S crystals are more reactive than C_2S but are not reachable to water). The speed of the reaction is now controlled by the rate at which water molecules diffuse through the C-S-H coating. This coating thickens over time causing the production of calcium silicate hydrate to become slower and slower. Diagram in Fig.6 will clear this:

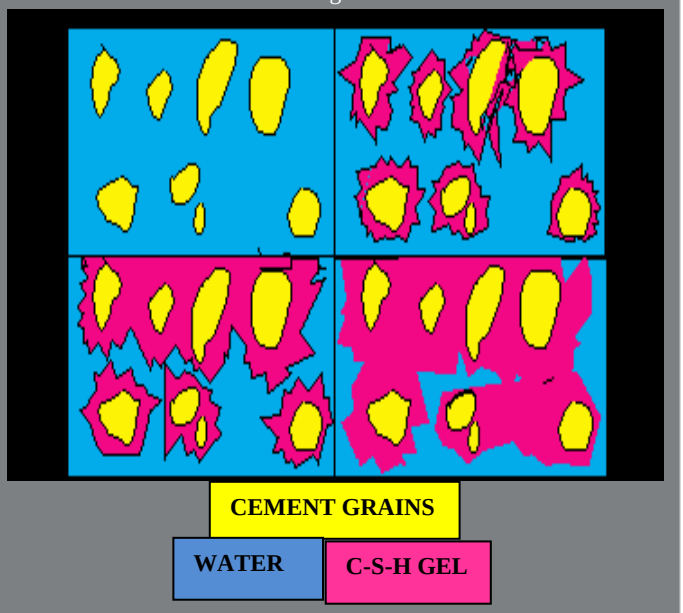

Fig.6 Different stages of hydration

Diagram (a) Hydration has not yet occurred and the pores (empty spaces between grains) are filled with water.
Diagram (b) represents the beginning of hydration
Diagram (c) the hydration continues. Although empty spaces still exist, they are filled with water and calcium hydroxide.
Diagram (d) shows nearly hardened cement paste. The majority of space is filled with calcium silicate hydrate(C-S-H). That which is not filled with the hardened hydrate is primarily calcium hydroxide solution. The hydration though slow will continue as long as water is present as there are still un-hydrated compounds in the cement paste.

Thus in nut shell we can say that hydration is fast at start and goes on slowly for year or more owing to nearly impervious C-S-H layer. Thus at any time hardening paste consists of hydrates of various compounds, unhydrated cement particles and water as shown in Fig.7. For example after 28 days there are 85-90% hydrates and 10-15% unhydrated cement particles. The gel pores in hydrated gel contain about 15% water for further hydration. This gel water is complementary to bound water (23%) which is used for chemical reaction of cement compounds. Any water in excess of (15+23)=38% causes undesirable capillary cavities.

Air Voids	Air Voids
Water	Capillary cavities
Cement Grains	Hydrated Products
Aggregate	Unreacted cement
	Aggregate
Fig.7: At time t=0	At time't' after set

CONCEPT OF SETTING AND HARDENING:-

Setting is the stiffening of the cement paste and refers to a change from a fluid to a rigid stage. The paste acquires some strength during setting whereas **Hardening** refers to the gain of strength of already set cement paste.

In practice, the terms initial set and final set are used to describe arbitrarily chosen stages of setting.

Setting is caused by a selective hydration of cement compounds C_3A and C_3S whereas C_2S stiffens in a more gradual manner to lend final hardening (Again refer Fig.4).

B) EFFECT OF COLD WEATHER

As per IS: 7861 (Part II)-1981 Any operation of concreting done at about 5°C atmospheric temperature or below should be termed as Cold weather concreting.

However **ACI Committee 306** defines it as a period when for more than 3 successive days the average daily air temperature drops below 5°C (40°F) and stays below 10°C (50°F) for more than one-half of any 24 hour period.

Cold weather produces four types of effects on concrete:

i) **Delay in setting and hardening**: At low temperatures concretes takes a longer time to gain strength during initial setting and hardening. Delayed setting makes concrete vulnerable to frost whereas delayed hardening results in delays in removal of form work.

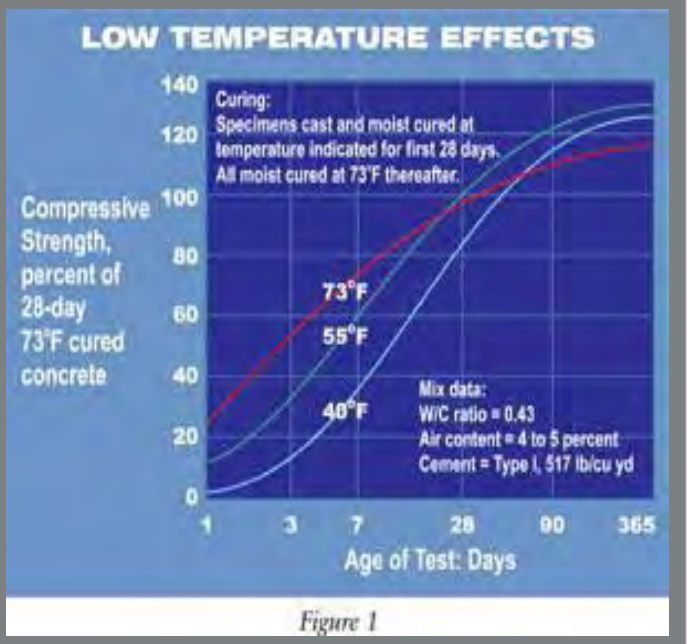

Fig. 8: Gain of compressive strength by cement with time at 23°C(73°F), 13°C(55°F) & 5°C (40°F)

ii) **Freezing of concrete at early stage:** Free water in plastic concrete can freeze when temperature goes below 0°C. The freezing where at one hand prevents hydration, it also results in expansion in volume of water by 9% and that of concrete by about 2%. If the concrete at that time has not attained sufficient strength the disruptive forces can weaken the bonds between aggregate and cement paste resulting in lowering of compressive strength by 30% to 50% as shown in this Fig.8

iii) **Freezing and Thawing:** Due to variation in climate even hardened concrete gets subjected to freezing and thawing cycles. With increase in cycles the dynamic modulus of elasticity is decreased and percentage loss in compressive strength is increased.

iv) **Stresses due to temperature differentials**: At the time of removal of form insulations in cold weather large temperature differentials i.e. very low temperature at outer surface and high temperature within the concrete member especially in the thick ones are created. It may promote cracking and can produce harmful effect on durability of concrete.

Visible Defects observed in different case studies:

Let us study and analyse above adverse effects on some concrete members constructed without taking any special precaution in cold weather:

Columns deterioration in a cold country: Freezing may cause distress in the concrete. Distress to critically saturated concrete from freezing and thawing will start with the first freeze-thaw cycle and will continue throughout successive winter seasons resulting in repeated loss of concrete surface.(Fig.9)

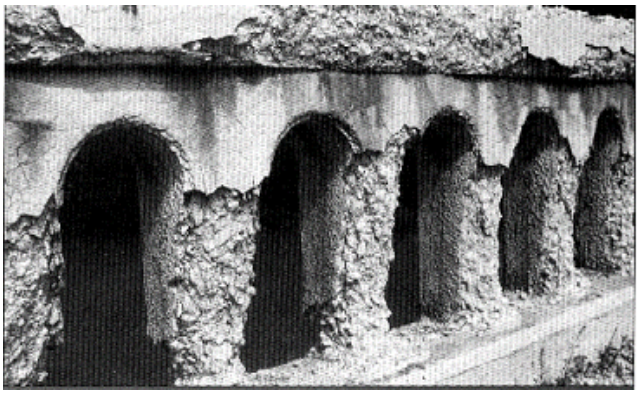

Fig.9: Loss of concrete from columns

Worn-out platform coping casted in cold weather: (Fig.10) Concrete is porous; it can absorb water. When absorbed water freezes, it expands, and if no method is used to accommodate the expansion, it can lead to cracking, or flakes of concrete may break loose and separate from the surface. This is especially true if absorbed moisture freezes before the concrete has aged enough to gain significant strength.

Fig. 10: Worn-out P.F. coping at Panzgam Station

The appearance of concrete damaged by premature freezing will vary with environmental conditions, but it often appears as widespread spalling or de-lamination of the surface layer, since cracking is often parallel to the surface.

Scaling is defined as a general loss of surface mortar or mortar surrounding the coarse aggregate particles on a concrete surface. This problem, shown in Fig.11, is typically caused by the expansion of water due to freezing and thawing cycle.

Fig. 11 Scaling of floor surface

Cracks in concrete roads caused by cold weather: These are not directly related to cold weather concreting defects. Soil under the pavement heaves due to frost in cold weather which results in uneven support of the pavement. The heaving itself is caused by the formation of "ice lenses" in the soil below the pavement. Ice lenses result in expansion of soil causing longitudinal cracking and differential vertical movement of the slabs. (See Fig.12)

Fig.12: Cracks by soil heaving in cold weather

D-Cracking of concrete pavements: It is caused by the freeze-thaw deterioration of the aggregate within concrete. D-cracks are closely spaced crack formations parallel to transverse and longitudinal joints that originate at the bottom of the slab and progresses upward until it reaches the wearing surface as shown in Fig.13

Fig.13: D-Cracking in pavements

Popping off of wall-tiles from outer face of a building at Srinagar: Mortar to be used in cold temperatures require longer curing times but if it is allowed to freeze during the initial curing process it breaks the chemical reaction vital for the permanent bond. Comparative less adhesion between resulting weak mortar and back of tiles can give rise to popping off of tiles. Fig.14 shows popping out tiles from the wall of a building.

Fig.14: Side view of a building at Srinagar

C) STRATEGIES AND METHODS

Special care must be taken when placing concrete in cold weather

BASIC APPROACH

If freshly placed concrete cools below 0ºC the water in the mix will freeze and expand. This could damage the concrete so much so that it becomes useless and has to be removed. This is because the disruptive forces produced due to increased volume of ice tend to weaken the bonds between aggregate and cement paste resulting in lowering of compressive strength by more than 50%. However, if the concrete is able first to reach strength of about 2 N/mm^2, it is likely to resist this disruptive expansion. This stage is called '**Frost Safety Level**' and time taken by concrete to reach this level is called '**minimum prehardening period**'.

This period decreases with increase of concrete grade as well as with increase in curing temperature. The prehardening period for different grades of concrete and for different curing temperatures is shown in Fig.15. However hydration process and thus strength development will progress though at a slow rate due to low temperatures.

The aim therefore, during cold weather must be to determine the prehardening period and keep the concrete warm (above 5ºC) for this minimum period and then ensure that the strength is permitted to develop.

Grade of concrete	5°C	10°C	15°C	20°C
M-20	71	46	32	24
M-25	65	42	30	22
M-30	59	38	27	20
M-40	50	33	23	17

(Priod in HRS. at different Curing temperatures) starting from point of first stiffening

Table 2: MINIMUM PREHARDENING PERIOD

The severity of the weather determines the precautions that need to be taken. For the purposes of concreting, BIS has divided the cold weather into following three categories:

I) Ambient temperature below 5ºC but no frost When the temperature is low but does not drop below freezing there is no danger that the concrete will be permanently damaged but it will take longer to develop strength.

Precautions for concreting:

i) Formwork should not be removed too soon otherwise there is a risk that corners and arrises could be knocked off and that concrete in beams and suspended slabs may be too weak to carry its own weight and collapse.

ii) Increase the cement content of the mix to speed up the gain in strength.

II) Slight frost at night: Any freshly placed concrete must be protected straightaway from freezing.

Precautions for concreting:

i) Prior to placing ensure that sub-bases, formwork, reinforcement and any transporting or placing equipment are free from ice and snow.

ii) Use insulated formwork or timber formwork or frost blankets for protection from frost during night.

iii) For beams, columns and walls cover the exposed surfaces with insulating material or erect temporary covers and provide heating with space heaters.

III) Severe frost day and night

Heated concrete (10ºC at time of delivery) is required.

Precautions for concreting:

i) Heat the water and if necessary aggregate also to make sure that concrete is delivered at not less than 10°C. It should be placed quickly and insulated immediately.

ii) If heated concrete can only be delivered at temperature less than 10°C but not less than 5°C then after placing and insulating, the concrete should be provided continuous heating.

iii) Don't forget to provide precautions as mentioned in I & II also.

STRATEGIES

Basically **two ways** are adopted to get good concrete in cold weather**:**

1. MINIMISING THE EFFECTS OF LOW TEMPERATURES

The methods in this strategy are aimed at increasing the rate at which concrete gains strength in early stage after placement. The methods are:

i) Increasing Cement Quantity means using a higher-strength concrete. It can significantly increase the rate of strength gain.

ii) Using High-early strength cements (these contain **higher percentage of C_3S** and less percentage of C_2S) results in heat being generated more rapidly within the concrete thereby increasing its temperature. Refer Table 1 which shows high rate of heat liberation by C_3S than C_2S. Due to this reason HES cements give high heat of hydration as well as double the initial strength than ordinary cements after one day. It is also clear from Table 3.

iii) Using less water in the mix will **lower Slump** thus increasing the rate of strength gain. The higher the water content, the more susceptible the concrete is to early freezing. Slump less than 100 mm is desirable and 150 mm may work, but a high water to cement ratio is not acceptable.

iv) Reducing time between mixing and placing will minimise the drop in temperature of the concrete. There will be some temperature loss after mixing while the truck mixer is travelling to the construction site and waiting to discharge its load. The concrete should be placed in the forms before its temperature drops below that recommended value at which the concrete should be maintained for the duration of the protection period. So this time should be reduced to minimum.

v) Addition of accelerating admixtures in the concrete reduces the setting time and accelerates the rate of strength gain by increasing the rate at which the cement hydrates. But use of admixtures containing chlorides should either be avoided, or their chloride content limited. See harmful effects of chlorides in Fig.15. A combination of (CaCl2) and (NaCl) gives better results than anyone alone.

vi) Using Hot Water in the mix will raise the temperature of the concrete, which in turn accelerates the rate at which the cement hydrates. Mixing hot water is the easiest and most practical. The mass of aggregates and cement in concrete is much greater than the mass of water; however, water can store about five times as much heat as can cement and aggregate of the same weight. For cement and aggregates, the average specific heat (heat units required to raise the temperature by 1°C per kg of material) can be assumed as 0.925 kJ compared to 4.187 kJ for water. The temperature of mixing water should not exceed 70°C and mixing should ensure consistency in the temperature of the concrete delivered to site. When temperature of either water or aggregate exceeds 40°C they should be mixed in the mixer first to reduce the temperature and then cement added to avoid any flash set.

vii) Using Hot Aggregate will raise the temperature of concrete in cold weather. If coarse aggregate is free of frost or ice lumps then only sand needs to be heated up to 40°C (mixing water at 60°C) otherwise coarse aggregate has also to be heated but not exceeding 65°C. In no case aggregate temperature exceeds 100°C.

viii) Air Entraining Agents provides some protection against the effects of sudden freezing by incorporating an air-entraining admixture into the concrete. These admixtures create minute air pockets or spaces into which pore water can move as it expands during freezing. It increases the durability of concrete under freezing conditions by 3 to 7 times.

Compound/ property	Type I (OPC)	Type III (High Early Strength
C_3S	50	60
C_2S	25	15
C_3A	12	10
C_4AF	8	8
Compressive strength at 1 day	7 MPa	14MPa
Heat of hydration (7 days)	330 Joule/g	500 joule/g

Table 3: Comparison of ordinary & H E S cements

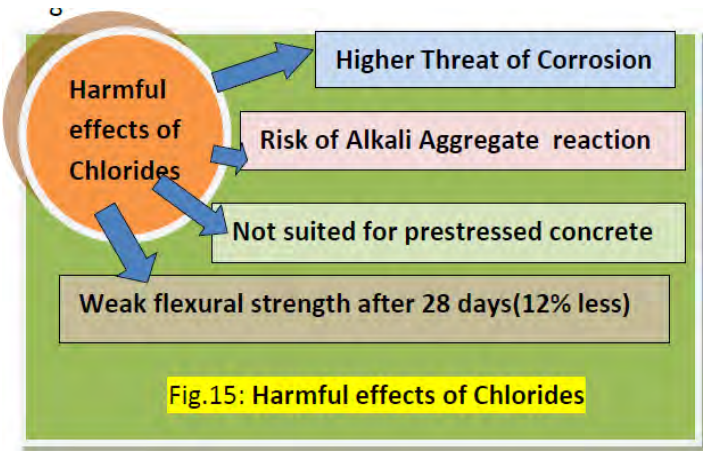

Fig.15: Harmful effects of Chlorides

2. ON-SITE PRECAUTIONS

The second type of strategy involves taking precautions against damage to the concrete by a sudden and unexpected frost or whenever the air temperature drops below 5°C.

The precautions include:

i) **Protecting concrete from cold ambience** like cold/frozen ground winds and frosts etc. keeps the heat inside, maintains temperature of concrete above 5°C and allow hydration and strength development to take place.

Two types of situations are confronted:

a) CONCRETING ON GROUND

In winter, the site around the structure may be frozen. The ground must be thawed before placing concrete. Enclosures, insulating blankets and heaters can be used to thaw subgrades. When the subgrade is frozen to a depth of approximately 80 mm the surface region can be thawed by steaming, spreading a layer of hot sand, gravel, or other granular or covering the subgrade with insulation for a few days. Heaters can thaw frozen ground at a rate of 0.3 m per 24 hours to a depth up to 3 m. Once cast, footings should be backfilled as soon as possible with unfrozen fill.

b) CONCRETING ABOVE GROUND

Portable heaters can be used to heat the undersides of floor and roof slabs. Enclosures must be constructed to retain the heat under floor and roof slabs. Before placing concrete, the heaters under a formed deck should be turned on to preheat the forms and melt any snow or ice remaining on top. When slab finishing is completed, insulating blankets or other insulation must be placed on top of the slab to ensure that proper curing temperatures are maintained. In some situations, the provision of a heated enclosure may be required to completely encase the concrete element.

Following are **methods used to protect concrete**:

ENCLOSURES

Enclosures are light frames covered with tarpaulins to retain heat. Heated enclosures are very effective but are most expensive. Enclosures can be of wood, canvas, tarpaulins, polyethylene film as shown in Fig.16 or of prefabricated rigid-plastic. When enclosures are being constructed below a deck, the framework can be extended above the deck for about 2m to serve as a windbreak.

Fig.16: Tarpaulin heated enclosure at 4th floor

INSULATING MATERIALS

Heat and moisture can be retained in the concrete by covering it with commercial insulating blankets. (See Fig. 17) The effectiveness of insulation can be determined by placing a thermometer under it and in contact with the concrete. If the temperature falls below the minimum required, additional insulating material, or material with a higher thermal value 'R', should be applied. Corners and edges of concrete are most vulnerable to freezing so temperatures at these locations should be checked often.

Fig.17: Stack of insulating blankets

Concrete pavements can be protected by spreading 300 mm or more of dry straw or hay on the surface for insulation as shown in Fig. 18.Tarpaulins, polyethylene film, or waterproof paper should be used as a protective cover over the straw or hay to make the insulation more effective and prevent it from blowing away. For maximum efficiency, insulating materials should be kept dry and in close contact with concrete or formwork.

Fig.18: Straw used on pavement

HEATERS

Three types of heaters are used:

Indirect-fired heaters are vented to remove the products of combustion. Fig. 19 shows working of an indirect fired heater in which a vent pipe is seen taking the combustion gases out.

Direct-fired units can be used to heat the enclosed space beneath concrete placed for a floor or a roof deck. CO_2 gas will combine with calcium hydroxide on the surface of fresh concrete to form a weak layer of calcium carbonate that interferes with cement hydration. So these should be avoided.

Fig.19: Indirect-fired heaters

Hydronic systems transfer heat by circulating a glycol/water solution in a closed system of pipes or hoses as shown in Fig.20 these systems transfer heat more efficiently than forced air systems without the negative effects of exhaust gases and drying of the concrete from air movement.

Fig.20: Hydronic system showing hoses

Electricity and Steam can be another source of heat for winter concreting.

COOLING AFTER PROTECTION: Rapid cooling of concrete at the end of the heating period should be avoided. Sudden cooling of the concrete surface while the interior is still warm may cause thermal cracking, especially in massive sections such as bridge piers, abutments, dams, and large structural members.

So cooling should be gradual and source of heat and cover protection be slowly removed. The maximum allowable temperature drop during the first 24 hours after the end of the protection is given in Table 4. The temperature drops apply to surface temperatures. Notice that the cooling rates for surfaces of mass concrete (thick sections) are lower than for thinner members.

Maximum Allowable Temperature Drop (during first 24 Hrs after end of protection period

Section Size Minimum dimension in mm				
Size	<300	300-900	900-1800	>1800
Temp.	28°C	22°C	17°C	11°C

Table 4: Allowable Rate of temperature drop (ACI306R-88)

INSULATED FORMWORK:-

Insulated formwork will protect concrete from freezing. Timber formwork is a reasonable thermal insulator and will probably suffice for moderately cold conditions. (See Fig.21) Additional insulation will be required for more-severe conditions. Metal formwork offers little or no protection and should be insulated.

Insulated Concrete Forms (ICF) as shown in Fig.22 have the ability for carrying out concreting of walls at temperatures colder than that of conventional concrete forms due to their high insulating properties.

Fig. 21: insulated column form made of high-density plywood inside, rigid polystyrene in the middle and rough plywood outside.

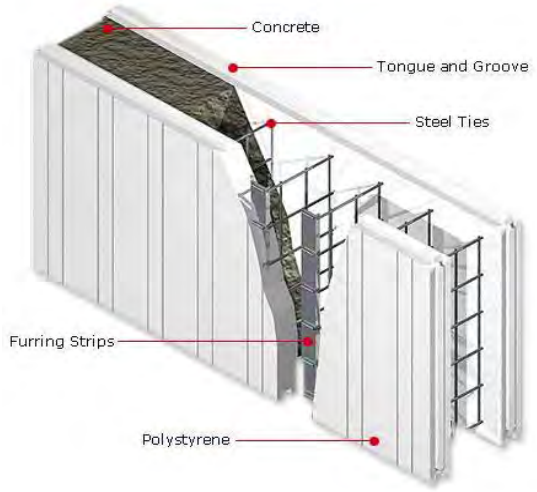

Fig.22: Insulated Concrete Forms are stay-in-place concrete forms

ii) **Delayed Stripping of Formwork**

It is recommended that the formwork be left in position for as long as practical to protect the concrete from frost. Forms serve to distribute heat more evenly and help prevent drying and local overheating in heated enclosures.

Before shores and forms are removed, fully stressed structural concrete should be tested to determine if in-place strengths are adequate, rather than waiting for an arbitrary time period. In-place strengths can be monitored using field-cured cylinders, probe penetration tests, pullout testing or maturity testing.

However IS Code 7861(Part II), in absence of any other data, gives general guidelines for stripping of formwork as shown in Table 5 below:

TABLE 5: MINIMUM TIME LIMIT FOR STRIPPING FORMWORK (days)

Cement	Weather/Air temp.	Beams sides, walls, columns	Slabs (props left under)	Beam soffits (props left under)	Removal of props to slabs	Removal of props to beams
OPC Concrete	Cold weather(3°C)	5	7	14	14	28
	Normal Weather(16°C)	1	3.5	7	7	14
RHC Concrete	Cold weather(3°C)	3	4	8	8	16
	Normal Weather(16°C)	1	2	4	4	8

iii) **Curing:-**

During prolonged periods of freezing conditions moist or water curing is rarely appropriate. Common solutions include use of an insulation blanket or covering, particularly where concrete has been placed in insulated forms. When formwork is removed, the member should be further cured by covering it with plastic sheeting, or tarpaulins, properly lapped at joints and secured to ensure wind-tightness. Note that newly released concrete from insulated formwork or heated enclosures should never be saturated with cold water. Rather, care should be taken not to suddenly expose warm concrete surfaces to cold conditions. The temperature of surfaces should always be allowed to fall slowly to avoid thermal cracking due to a temperature differential between the surface and interior of the element.

D) SUGGESTIONS & CONCLUSIONS

PLANNING AND EXECUTION

When faced with cold-weather concreting situations, the first step is to decide whether it is profitable to operate during this period of time or whether it makes more sense to wait until warmer weather. Then go in for following steps:

i) Planning: The owner, contractor and concrete supplier should meet to discuss how and which specific methods should be used for cold-weather concreting. Planning should take place well before any freezing temperatures are expected.

ii) Pre-preparations: Collect information about temperature forecasts and probable winter season. Talk to the ready-mix producer about concrete temperature needed for project.

Insulations, material for enclosures, thermometers, formworks, admixtures and heaters as required should be ready at hand.

iii) Decide placement temperature and protection period: This protection period is the time for which placement temperature should be controlled. It depends on factors such as cement type (Type III reduces time) and amount of cement (higher cement content reduces time), accelerators (reduce time) and service.

iv) Temperature insights: When concrete is placed, the concrete's surface temperature is taken. Edges and corners of the concrete must be taken care of as these are more prone to freezing. Monitoring of the concrete temperature, outdoor temperature, time of readings, air temperature and weather conditions should be done.

These readings are typically taken by thermometers under some sort of temporary cover. Thermometers are placed in those parts of concrete where maximum stress will appear at removal of forms. A bimetallic thermometer is shown in Fig.23.

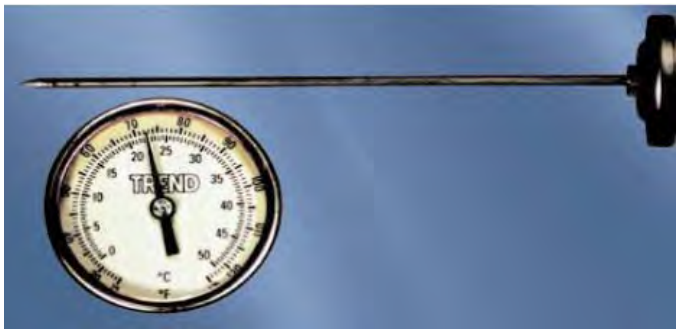

Fig.23: Bimetallic thermo-meter with a metal sensor for checking fresh concrete temp.

v) Testing: Apply the protections during and after concreting. Quality control is carried out. The test results should be used to fix the time for removal of insulations. Control specimens are cast and tested.

SOME PRACTICAL TIPS FOR COLD WEATHER CONCRETING

1. Schedule the proper equipment and manpower to be in place well ahead of time.
2. Remove all snow and ice from all concrete forms and the subbase before placing the concrete.
3. Don't place concrete on a frozen subgrade; thaw the subgrade with steam or protect it with insulation.
4. Do not begin final finishing operations while bleed water is present. Rising water from concrete is trapped below the surface separating it from the concrete and causing de-lamination. Cold weather accentuates this effect.
5. Don't over work cooled slabs that exhibit delayed setting characteristics. Over-working the surface during finishing will reduce the air content of the surface concrete, leaving it weaker and more vulnerable to scaling due to freezing conditions.
6. Provide triple insulation thickness at the corners and edges of walls and slabs.
7. Avoid using unvented heaters; carbon dioxide from the heaters can cause soft, dusting floors.
8. Keep heaters attended at all times; they are a fire hazard when burning and are of no value when out of fuel.
9. Keep ice from forming. Once ice has formed, hydration stops and strength development is seriously impaired. Fresh concrete frozen during the first 24 hours can lose 50% of its potential 28 day strength.
10. Do not seal freshly placed concrete. Sealer if applied too soon may 'lock in' enough moisture inside concrete which creates greater than 80% relative humidity inviting alkalis silica reaction and sulphate attack to take place.
11. Use non-chloride, non-corrosive accelerators for pre-stressed concrete or when corrosion of steel reinforcement or metal in contact with concrete is a concern.
12. Don't use Calcium chloride as admixture in excess of maximum dosage of 2% by weight of cement.
13. Remove the heat protection in a gradual manner that ensures that temperature of the concrete will not drop faster.
14. Place and maintain concrete at the recommended temperature. The mixing temperature should not exceed 8°C above recommend values.
15. Prefer heating water. If aggregate has to be heated due to severe conditions it should not be overheated. The temperature should not exceed 100°C at any time and average temperature should remain less than 65°C for coarse aggregates.
16. Do not add hot water to cement directly to avoid any flash set. Mix hot water with aggregate first and then cement should be added.
17. Take into account the temperature loss after mixing while the truck mixer is travelling to the construction site and waiting to discharge its load.
18. Use dry straw or hay or husk for covering the pavements as it will have high thermal value.
19. Never start curing concrete with cold water after it is released from insulated formwork or heated enclosure. The warm concrete on coming in sudden contact of cold water may crack due to 'thermal shock'.
20. Leave the formwork in position if these are not immediately required elsewhere, as this will accelerate the hardening process and shorten the time for striking load-bearing formwork.

CONCLUSIONS

The teams of industrious engineers of USBRL Project are working day and night to complete this prestigious project on targeted dates except for the three months of December, January and February in which severe cold weather retards their galloping pace of construction almost to a grinding halt. The teams with their working contractors have developed a pre-mindset of pulling shutters down to all activities in this period and if an additional fifteen-day transition period for closing and starting is assumed, a large chunk of four months goes waste every year. The past records show that temperature in region fluctuates around zero and rarely goes below freezing point and that too for very limited period. So expensive preparations and protective methods are not needed to successfully work in these months. Only dedicated approach and strict adherence to precautions for cold weather working will suffice. The only need of the hour is meticulous planning and advance preparations. It should be kept in mind that engineers in other cold countries are successfully working below minus 20°C. Conditions are far less challenging here and as such precious time of those four cold months should also be utilised.

Slope Failure in Rocks and its Kinematics

1.0 Introduction:

Slope failure is a common geological phenomenon occurring whenever when an imbalance takes place between shear strength and shear stress in a soil or rock mass resulting in a often wide spread disaster of men and material . These slope failures are major natural hazards that occur in many areas throughout the world. Rock mass is a discontnium medium. It is characterised by faults, joints and bedding plane. These features are represented by strike and dip (or dip and dip direction).

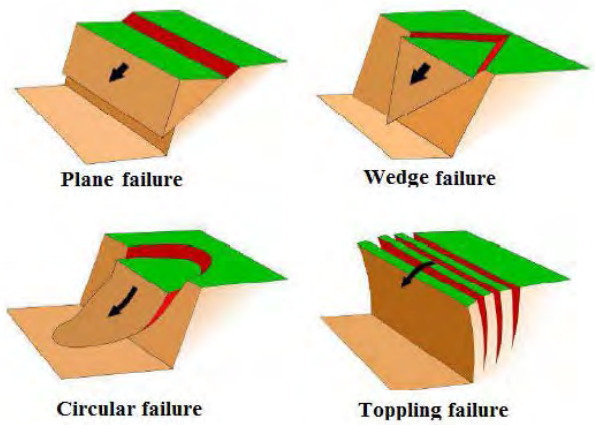

Fig 1.0 Type of Failure

Slopes expose two or more free surfaces because of geometry. Depending upon the depth and orientation of excavation face with respect to joint set and joint intensity different mode of rock mass failure will take place. The mode of failure essentially of Planer, wedge, toppling, rock fall and rotational type (circular/non-circular) (Fig 1). The first four are more predominant in rock slopes and are primarily controlled by the orientation and the spacing of discontinuities planes with respect to the slope face.

The pattern of the discontinuities may be comprised of a single discontinuity, or a pair of discontinuities that intersect each other, or a combination of multiple discontinuities that are linked together to form a failure mode. Persistence in thickly bedded rock mass plays major role while deciding the failure type. Circular and non circular failure occurs in soil; mine dump, heavily jointed or fractured rock mass and very weak rock. This type of failure is very much common in Sangaldan yard and portal area. The types of slope failure are primarily controlled by material properties, water content and foundation strength.

2.0 Plane Failure (Planner Failure)

A rock slope undergoes this mode of failure when combinations of discontinuities in the rock mass form blocks or wedges within the rock which are free to move. The pattern of the discontinuities may be comprised of a single discontinuity or a pair of discontinuities that intersect each other, or a combination of multiple discontinuities that are linked together to form a failure mode.

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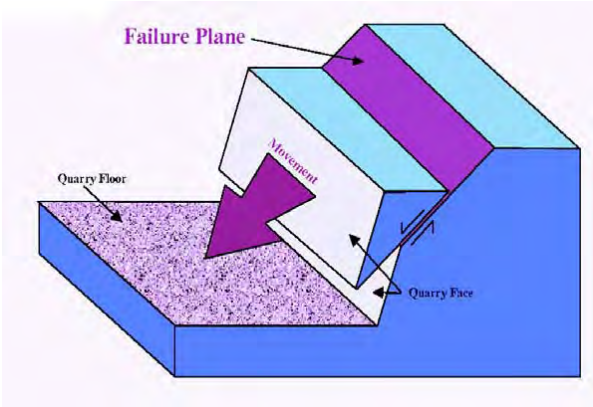

Fig 2.0 (a)Planner Failure

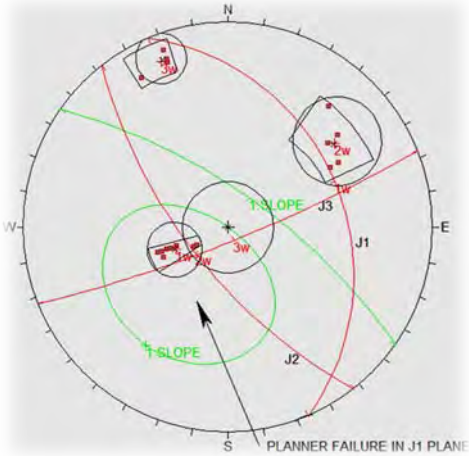

Fig 2.0 (b) Stereographic Projection

A planar failure of rock slope occurs when a mass of rock in a slope slides down along a relatively planar failure surface. The failure surfaces are usually structural discontinuities such as bedding planes, faults, joints or the interface between bedrock and an overlying layer of weathered rock. Block sliding along a single plane. In case of a plane failure, at least one joint set strike approximately □15° to slope strike and the joint angle is less than the slope angle. Fig 2.0 Stereography defines poles of joint 'J1' lies within the boundary of friction angle for Joint 'J1', daylight of slope face and within the 15° to slope strike limit.

3.0 Wedge Failure:

Wedge failure of rock slope results when rock mass slides along two intersecting discontinuities, both of which dip out of the cut slope at an oblique angle to the cut face, thus forming a wedge-shaped block (Fig. 3(a) & 3(b)). Wedge failure can occur in rock mass with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip towards the plane of the slope. This mode of failure requires that the dip angle of at least one joint intersect is greater than the friction angle of the joint surfaces and that the line of joint intersection intersects the plane of the slope. (i) Joint J3 and J4 makes intersection defines wedge possibility with a direction S80W. (ii)Another possibility of wedge is between J2~J3 acting in the direction S30W. Persistence with standard deviation more than 6% of joints defines wedge possibility. On considering persistence, type (ii) is most possible type. These wedge failure can be designed with rock bolting to counter sliding mass with all external force (i.e., water force, Earth quake force etc.) or flattening the slope or slope height as desired with persistence. There is possibility of single plane failure if the intersecting joint lies outside central cone and slope angle equals with friction angle.

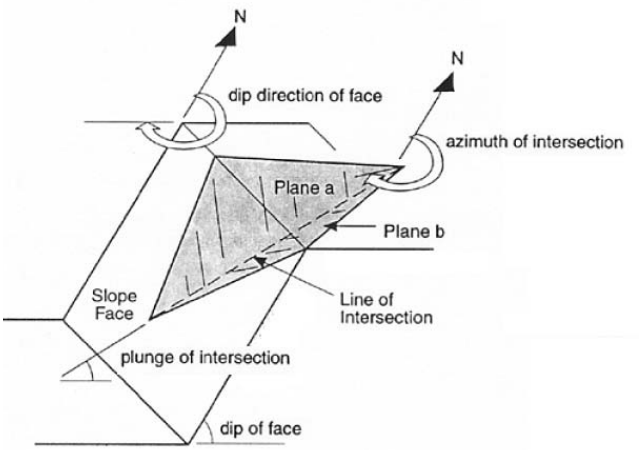

Fig 3.0 (a)Wedge Failure

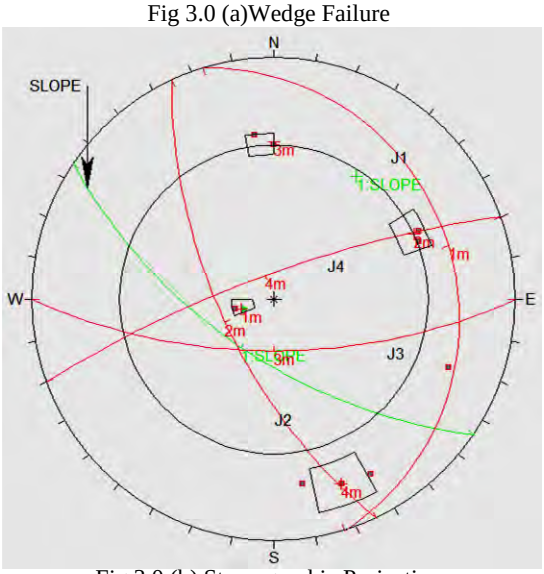

Fig 3.0 (b) Stereographic Projection

4.0 Toppling Failure:

Toppling failures occur when columns of rock, formed by steeply dipping discontinuities in the rock rotates about an essentially fixed point at or near the base of the slope followed by slippage between the layers (Fig 4(a) & 4(b)). The centre of gravity of the column or slab must fall outside the dimension of its base in toppling failure. Jointed rock mass closely spaced and steeply dipping discontinuity sets that dip away from the slope surface are necessary prerequisites for toppling failure. The removal of overburden and the confining rock, as is the case in mining excavations, can result in a partial relief of the constraining stresses within the rock structure, resulting in a toppling failure. This type of slope failure may be further categorized depend on the mode such as flexural toppling, block toppling, and block flexural toppling. In case of a toppling failure, at least one joint set strike approximately 10° to slope strike and the joint angle is less than the slope angle. However the strike angle difference approximately about 15° shall be checked.

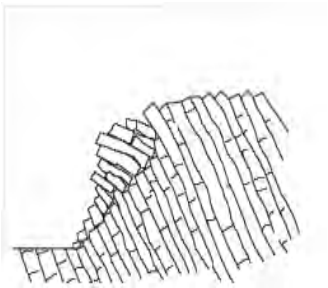

Fig 4.0(a) Block Toppling Failure

Fig 4.0(b) Flexural Toppling Failure

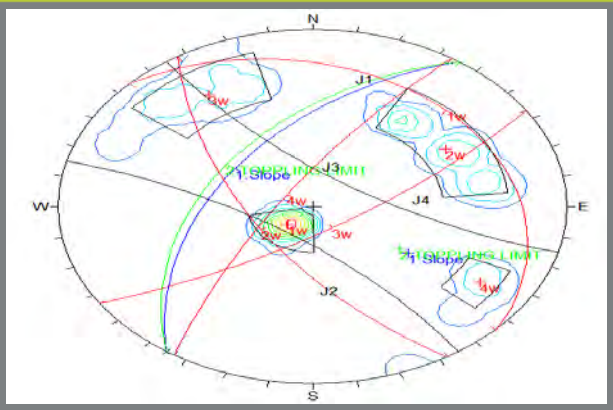

Fig 4 (c) Stereographic Projection Block Toppling Failure

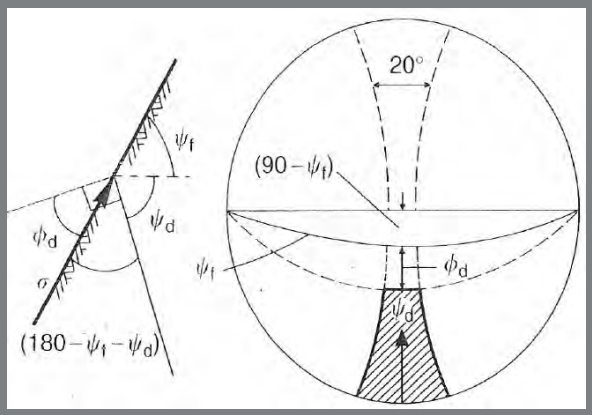

Fig 4.0 (d) Kinematics of Lower hemisphere Stereographic Projection

During toppling failure different independent block undergoes slip or toppling. These conditions of interlayer slip is given by

$(180-\psi_f-\psi_d) \geq (90-\phi_d)$ by $\psi_d \geq (90-\psi_f) + \phi_d$

4.1 Block Toppling

As illustrated in Fig 4(a) block toppling occurs when individual columns in a strong rock are formed by a set of discontinuities dipping steeply into the face. A second set of widely spaced orthogonal joints defines the column height. The short columns forming the toe of the slope are pushed forward by the loads from the longer overturning columns behind. This sliding of the toe allows further toppling to develop higher up the slope. The base of the failure generally consists of a stepped surface rising from one cross joint to the next. Typical geological conditions, in which this type of failure may occur, are bedded sandstone and columnar basalt in which orthogonal jointing is well developed.

4.2 Flexural Toppling

The process of flexural toppling is illustrated in Fig 4(b) that shows continuous columns of rock separated by well developed, steeply dipping discontinuities, breaking in flexure as they bend forward. Typical geological conditions in which this type of failure may occur include thinly bedded shale and slate in which orthogonal jointing is not well developed. Generally, the basal plane of a flexural topple is not as well defined as a block topple. Sliding, excavation and erosion of the toe of the slope allows the toppling process to start and it retrogresses back into the rock mass with the formation of deep tension cracks that become narrower with depth. The lower portion of the slope is covered with disordered fallen blocks. Therefore it is sometimes difficult to recognize a toppling failure from the bottom of the slope.

GEOLOGY OF CHENAB BRIDGE ALINGMENT ON KATRA - QAZIGUND RAIL LINE SECTION USBRL PROJECT REASI DISTRICT (JAMMU & KASHMIR)

General

The Construction of broad gauge Railway line in Himalayas is a challenging job because of its complex geological conditions and other tectonic activities witnessed in the past, because of which rocks have undergone intense deformation with the development of major and minor folds thrust zones with water bearing horizons. The Himalayas can be divided in to four major zones. (i) Siwalik foot hills or outer Himalayas or Sub Himalayas (10 - 50 km wide), (ii) Lesser/Middle Himalayas (60 - 80 km), (iii) Great/Centeral Himalayas (80 km) and (iv) Trans Himalayas (40 km). The altitude of Himalayas from Siwalik foot hills to Centeral Himalayas in J & K varies from 1000m to 5000m.

Chenab Bridge with a length of 1315m located between Km 50.400 (Bakkal End) with RL 854.500 m and Km51.715 (Kauri End) with RL 855.517 m is one of the highest Railway bridge in the world with a height of 359 m is located in the outer Himalayas. The area along the slopes from the Chenab river bed in the upslope along the bank slopes exposes both thickly and thinly bedded grayish, whitish cherty dolomite with some thick phyllitic quartzite bands towards top belonging to Sirban limestone Group. The area further upslope towards the left side of the rail line section at both the banks exposes thick bands of hard quartzite and cherty dolomite. The various rock units along banks are favorably disposed dipping almost inside the hill or making an acute angle with the direction of the alignment. These rock units are further overlain by thin cover of slope debris with thickness varying from 0.5m to 3m as observed in the slope cuts particularly along the left bank slope. The rock mass represented by bedded sequence has variable properties like hardness, compressive strength, prone to wreathing and chemical action etc.

The rock mass in general is hard and competent but due to certain factors like presence of some weathered seams along joint plans, shear surfaces and thinly bedded nature of the strata have interfered with the overall strength of the rock mass which is a common phenomena in the rock mass along the slope and needs slope stability measures like removing of the disturbed blocks (Photo 1) weathered zones and providing rock bolts, shotcrete, grouting and anchors if required to strengthen the rock mass. Except some localized area no serious adverse feature exactly along the central line of the alignment has been recorded in the area. The ground water conditions in general are dry along both the banks.

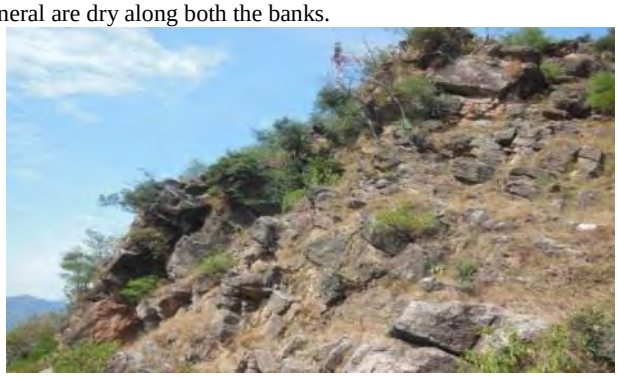

Photo 1: showing loose blocks with open joints along the slope.

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The general trend of the rock mass varies from N20˚W – S20˚E TO N65˚W-S65˚E with dip varying from 15˚ to 45˚ northeasterly. The rock mass have under gone intense folding due to which swing in the strike direction of the rock mass and variation in the angle of dip has been recorded. The general slope angle along both the banks varies from 40° to 85° even vertical at places. The rock units are traversed by at least three joint sets (Photo 2).

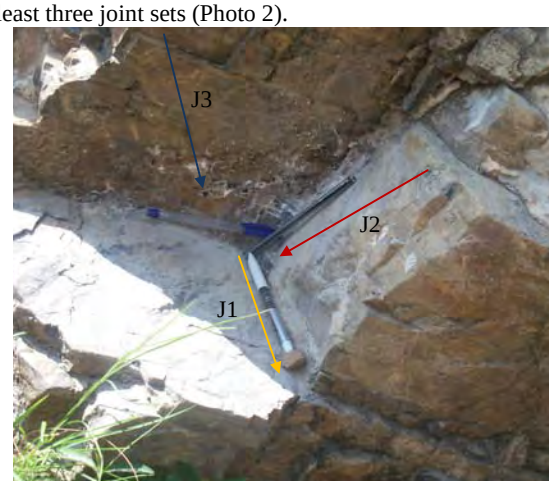

Photo 2: showing all three joint sets J1,J2 andJ3

Joint: J1 – It is bedding joint dipping generally in the north east direction or slightly in the upstream direction which in general is a favorable disposition. Due to folded nature of strata there is considerable variation in strike as well as angle of dip which varies from strike N 20° W- S 20° E to N 65° W – S 65° E with dip varying from 15° to 45° northeasterly. This joint set is generally tight in nature but occasionally open near surface. Emplacement of calcite veins both along and across the bedding plane have also been recorded. These joint planes in general have slightly smooth surface with some joint planes having rough undulating and wavy surface also. The spacing of the these joint planes varies from 10 cm to 20 cm in thinly bedded sequence and about 1 m to 2 m in thickly bedded sequence. The joint with clay fillings or clay like material are vulnerable, if oriented towards the river valley, but it is not the case with this slope. Calcite coatings in open joint have also been recorded besides clay filling by the surface runoff particularly in the rainy season in open joints upto some depth. It is persistent and shows visible continuity from 2 m to 15 m and above.

Joint: J2 – The joint J2 cutting across bedding plane of the rock mass by dipping almost opposite to the dip direction of the bedding plane. The visible persistence of this joint is low to high varying from 2 m to 15 m. The general strike of the joint is N 20° W – S 20° E to N 65° W-S 65° E and dip varying from 50° to 70° south-westerly. This joint is both open and tight in nature with carbonate coating along the joint planes. This joint in general is consistent in orientation but swing in the orientation has also observed at places which are attributed to the folded nature of the rock mass in the area. The water condition is almost dry. During the rain the rainwater entering the joints have little significance as water flows down instead of entering deep into the rock mass.

Joint: J3- It is another prominent joint set recorded in the area and is generally widely spaced with medium to high persistence which varies from 2 m to 20 m even more. Exact persistence of this joint cannot be measured accurately as these are covered with scree or slope debris at places. Besides some of the places are not accessible physically for measurement. The strike of these joint planes varies from N10°E-S10°W to N60°E-S60°W with amount of dip varying from 60^0 to 80^0 southeasterly. Profuse calcite coating and yellowish staining along the joint planes, besides clay filling and deposition of calcareous material have also been recorded at places along the joint planes. Occasional gouge material along the joint planes has also been recorded at

places. The formation and deposition of the clayey material and calcareous material is not exclusively due to the shearing of the rock mass, sometimes it is also due to chemical disintegration of the rock mass along the joint planes and calcite veins under wet conditions particularly during the raining season as evidenced by perfused calcareous coating recorded along some open joint planes. Presently the joint surface in general is dry but damp conditions may occur at depth.

Kouri End

The bridge alignment is almost parallel or slightly oblique to the strike direction of the rock mass. The general slope angle of this side varies from 40° to 85° even vertical at places. The rock units are traversed by at least three joint sets, (Photo 3) the crosscut relationship of these joint sets have resulted into formation of small wedges in the thinly bedded sequence and occasionally large scale wedges in the thickly bedded sequence. Besides number of bedding parallel and cross bedding shear zones/ shear seems have also been recorded in the area (Photo 3) which is very common in Sirban limestone. The rock mass up to the top of the ridge in general is hard and competent, but presence of discontinuities like joint planes, shear zones and degree of weathering interfere with the overall strength of the rock mass. Due to tectonic activities in the Himalayan region the geological features like folding and faulting are common, at Kouri end localized folding and faulting is observed at places due to which change in the strike and dip angel have been recorded having limited persistence (Photo 4 & 5).

Photo 3: Showing bedding shear zone and three sets of joints at portal of exploratory drift along right bank.

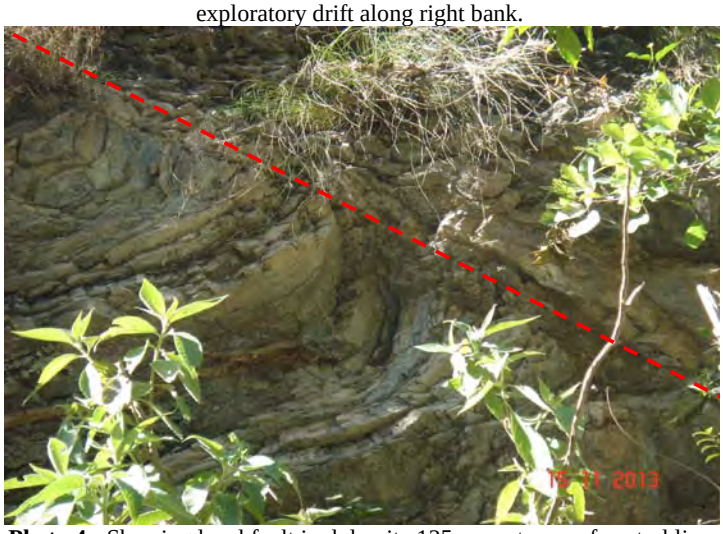

Photo 4: Showing local fault in dolomite 125 m upstream of central line along right bank

The contact between Sirban limestone and Subathu Formation is marked by angular unconformity represented by recemented clasts of quartzite, which is quiet hard and competent (Photo 6).

Photo 5: Showing anticline of recumbent fold about120 m U/S of central line along right bank.

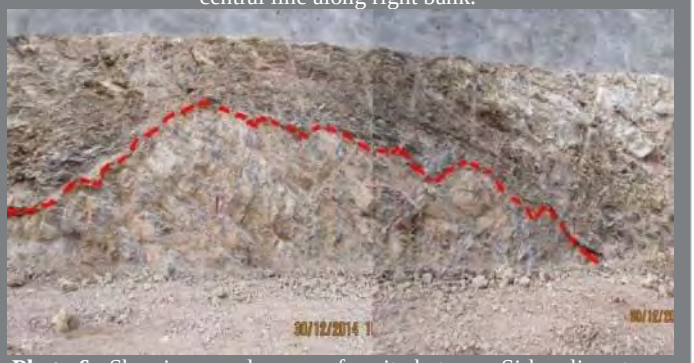

Photo 6: Showing angular unconformity between Sirban limestone and Subathu Formation.

The area behind the ridge top towards the tunnel of T 6 P1 over which most of the piers have been constructed generally have gentle and also flat slope (Photo 7). The area is quiet stable and there is no any apprehension of the slope stability Problem. Some of the piers are located on quartzite, phyllatic quartzite (Photo 8) and bauxite belonging to upper member of the Sirban limestone sequence. Some of the piers are located on Subathu Formation represented by khaki shale, limestone and quartzite.

Photo 7: Showing piers of the viaduct constructed on gentle slope behind the river bank.

Photo 8: Showing phyllatic quartzatic with thin shale bands at kouri end.

BAKKAL END

The area along the left bank slope from the Chenab river bed level both thinly and thickly bedded sequence (Photo 9 &10) as in case of right bank are exposed, which is overlain by thinly bedded reddish dolomite bands (Photo 11) followed upslope by grayish whitish thickly bedded cherty dolomite with prominent compositional bends having variable composition strength and thickness belonging to Sirban group. The general trend of the rock mass varies from (J-1) N 20° W - S 20° E to N45^o W to S 45° E with dip varying from 15° to 45° north easterly and with two joint sets (Photo 11). Due to folding in the rock mass change in the strike and angel of dip have been recorded at the places. No folding or faulting on regional scale has been recorded in the rock mass except some local folds and faults with very limited persistence.

Photo 9: Showing thickly bedded sequence of cherty dolomite.

Except some localized areas no serious adverse feature exactly along the centre line of the bridge has been recorded in the area till date. Although some bedding shear planes with thickness of 5cm to 25 cm have been recorded with persistence of around 5m. (Photo 12)

Photo 10: Showing thinly bedded sequence of cherty dolomite.

Photo 11: Showing reddish dolomite with three sets of Joint J1, J2 and J3.

Photo 12: Showing fractured Cherty dolomite with bedding parallel shear zones.

In order to strengthen the rock mass at foundations of the piers and along the bank slopes, as a long term measures various remedial were adopted, which include removal of loose and disturbed blocks along the slopes, cutting and benching, rock bolting, shotcreteing. Besides catchment area drainage to divert the surface runoff from the surrounding area entering into the slope along the center line. These remedial measures have shown encouraging results. (Photo 13 &14)

Photo 13: Showing cut slope and slope stability measures at Kouri end.

Photo 14: Showing different shortcreted burm levels at Bakkal end.

Seismic Design of Tunnels

Introduction:-

1.1 Purpose:-

While the general public is often sceptical about the performance of underground structures, tunnel designers know that underground structures are among the safest shelters during earthquakes, based primarily on damage data reported in the past.

This Study, geared to advance the state of the art in earthquake engineering of transportation tunnels, has the following goals:

- To maintain a consistent seismic design philosophy and consistent criteria both for underground and other civil engineering facilities.
- To develop simple yet rational simple yet methods of analysis for evaluating earthquakes effects on undergrounding structures. The methodology should be consistent for structures with different section geometries.

1.1 Scope of this Study:-

- Summary of observed of earthquakes effects on underground structures.
- A comparison of seismic design philosophies for underground structures and other civil engineering facilities. Based on this comparison, seismic design criteria were developed for underground tunnels.
- A quantitative description of ground behaviour travelling seismic waves. Various modes of ground deformations and their engineering implications for tunnel design are discussed.
- A review of current seismic design methodology for both circular mined tunnels and cut-and-cover rectangular tunnels.
- The development of a refined (yet simple) method for evaluating the earthquake ovaling effect on circular linings. To ease the design process, a series of design charts was developed.

The development of a simplified frame analysis model for evaluating the earthquake racking effect on cut-and-cover rectangle tunnels. These analyses were used to generate design data that could be readily incorporated into the recommended simplified frame analysis model.

1.2 Background:-

Importance of Seismic Design:-One of the significant aspects of the 1989 Loma Prieta earthquake in the San Francisco area was its severe impact on the aboveground transportation system and the earthquake event resulted in.

- Collapse of the I-880 viaduct claiming more than 40 lives.
- Direct damage costs to the transportation facilities alone totalled nearly $2 billon (Werner and Taylor, 1990).

The San Francisco Bay Area Transit (Bart) subway was found to be one of the safest places during the event, and it became the only direct public transportation link between Oakland and San Francisco after the earthquake.

Extract from
Seismic Design of Tunnels
A simple state of the Art Design approach

Parson Brinckerhoff
Monograph 7

Seismic Design before the '90s

Interestingly, some tunnels and shafts built without special earthquake provisions have survived relatively strong earthquake in the past – for example, the Mexico City subway during the 1985 Mexico City earthquake. On the other hand, some underground structures have been damaged severely in other events.

The lack of a rational methodology for engineers and the nonexistence of applicable codes has led to widely varied measures taken by different engineers. For example:

- Seismic effects are ignored and failure to check the resistance of the structures to earthquakes, even in highly seismic areas.
- Seismic design for underground structures are done using the same methodology developed for aboveground structures, without recognizing that underground structures are constrained by the surrounding medium. This may lead to the construction of unsafe structures or structures that are too conservatively designed.

1.4 General Effects of Earthquakes:-

Earthquake effects on underground tunnel structures can be grouped into two categories.

Ground Shaking:-

This refers to the vibration of the ground produced by seismic waves propagating through the earth's crust. The intensity of the shaking attenuates with distance from the fault rupture. Ground shaking motions are composed of two different types of seismic waves, each with two subtypes.

- Body waves travel within the earth's material. They may be either longitudinal P waves or transverse shear S waves and they can travel in any direction in the ground.
- Surface waves travel along the earth's surface. They may be either Rayleigh waves or Love waves.

The ground is deformed by the travelling waves; any tunnel structure in the ground will also be deformed. However, tunnel structures also must be designed to carry other sustained loads and satisfy other functional requirements. Tunnel structural design must consider the structural members' capacity in terms of strength as well as ductility and flexibility of the overall configuration.

Ground Failure:-

This includes various types of ground instability such as faulting, landslides, liquefaction, and tectonic uplift and subsidence. These be potentially catastrophic to tunnel structures, although the damages are usually localized and design of a tunnel structure against ground instability problems may often lead to escalated cost.

It may not be economically or technically feasible, however, to build a tunnel to resist potential faulting displacements. The best solution to the avoidance of faults may not always be possible and may be to avoid fault, tunnels crossing faults may be inevitable in some cases. The design approach to this situation is to accept the displacement, localize the damage, and provide means to facilitate repairs

1.5 Performance Record in Earthquakes:-

Dowding and Rozan (1978):-

71 cases of tunnel response to earthquake motions have been reported and characteristics are as follows:-

- These tunnels served as railway and water links with diameters ranging from 10 feet to 20 feet.
- Most were constructed in rock with variable rock mass quality.
- Construction methods and lining types varied widely. Permanent ground supports ranged from no lining to timber, masonry brick, and concrete linings. Dowding and Rozen concluded, primarily for rock tunnels.

- Tunnels are much safer than aboveground structures for a given intensity of shaking.
- Tunnels deep in rock are safer than shallow tunnels.
- No damage was found in both lined and unlined tunnels at surface accelerations up to 0.19g.
- Minor damage consisting of cracking of brick or concrete or falling of loose stones was observed in a few cases for surface accelerations above 0.25g and below 0.4g.
- No collapse was observed due to ground shaking effect alone up to a surface.
- Severe but localized damage including total collapse may be expected when a tunnel is subject to an abrupt displacement of an intersecting fault.

Owen and School (1981):-

These authors documented additional case histories to Dowding and Rozens', for a total of 127 case histories. These added case histories, in addition to rock tunnels, included:

- Damage reports on cut-and-cover tunnels and culverts located in soil.
- Data on underground mines, including shafts.

  Discussion of some of the damaged cut-and-cover structure is of particular interest. These structures have the common features of shallow soil covers and loose ground conditions:
- Cut-and-cover railroad tunnel with brick lining was destroyed during the 1906 San Francisco earthquakes. Where brick lining with no moment resistance was used, the tunnel structure collapsed.
- Five cases of cut-and-cover conduits and culverts with reinforced concrete linings were damaged during the 1971 San Fernando earthquake. The damages experienced by the linings included:
  - ➢ Failure of longitudinal construction joints.
  - ➢ Development of longitudinal cracks and concrete spalling.
  - ➢ Formation of plastic hinges at the top and bottom of walls.
- Damage to cut-and-cover structures appeared to be an important factor contributing to the severity of damage to underground structures. Damage initially inflicted by earth movements. Such as faulting and landslides, may be greatly increased by continued reversal of stresses on already damaged sections.

Wang (1985):-

In describing the performance of underground facilities during the magnitude 7.8 Tang-Shan earthquake of 1976, the author reported the following:

- An inclined tunnel passing through 13 feet of soil into limestone was found to have cracks up to 2 cm wide on the side wall. The plan concrete floor heaved up to 5 to 30 cm.
- Damage to underground facilities decreased exponentially with depth to 500 m. Schmidt and Richardson (1989) attributed this phenomenon to two factors:
  - ➢ The increasing competence of the soil/rock with depth
  - ➢ The attenuation of ground shaking intensity with depth

Sharma and Judd (1991):-

The authors extended Owen and Scholl's work and collected qualitative data for 192 reported observations from 85 worldwide earthquake events. They correlated the vulnerability of underground facilities with six factors: overburden cover, rock type, peak ground acceleration, earthquake magnitude, epicentral distance, and type of support.

- The reported damage decreases with increasing overburden depth.
- More damage for underground facilities constructed in soil than in competent rock.
  - ➢ For PGA values less than 0.15g, only 20 out of 80 cases reported damage.
  - ➢ For PGA values greater than 0.15g, there were 65 cases of reported damage out of a total of 94 cases.
- More than half of the damage reports were for events that exceeded magnitude M=7.
- Damage increases with 25 to 50 km from the epicentre.
- There were only 33 cases of with reinforced concrete linings. Of the 33 cases, 7 were undamaged, 12 were slightly damaged, 3 were moderately damaged, and 11 were heavily damaged.

Sharma and Judd attributed this phenomenon to the poor ground conditions that originally required the openings to be lined. Richardson and Blejwas (1992) offered two other possible explanations:

- ➢ Damage in the form of cracking or spalling is easier to identify in lined openings than in unlined cases.
- ➢ Lined openings are more likely to be classified as damaged because of their high cost and importance.

2.0 Seismic Design Philosophy for Tunnel Structures:

2.1 Seismic Design vs. Conventional Design:-

- Seismic loads cannot be calculated accurately. Any specified seismic effect has a risk (probability of exceedance) associated with it.
- Seismic motions are transient and reversing. The frequency or rate of these cyclic actions is generally very high, ranging from less than one Hz to greater than ten Hz.
- Seismic loads are superimposed on other permanent or frequently occurring loads.

2.2 Surface Structure vs. Underground Structures:-

Surface Structures:-

Surface structures are not only directly subjected to the excitations of the ground, but also experience amplification of the shaking motions depending on their own vibratory characteristics. If the predominant vibratory frequency of the structures is similar to the natural frequency of the ground motions, the structures are excited by resonant effects.

Underground Structures:-

In contrast, underground structures are constrained by the surrounding medium. It is unlikely that they could move to any significant extent independently of the medium or be subjected to vibration amplification. The underground structures can be considered to display significantly greater degrees of redundancy thanks to the support from the ground.

2.3 Proposed Seismic Design Philosophy for Tunnel Structures:-

Current seismic design philosophy for many civil engineering facilities has advanced to a state that dual design criteria are required. The higher design level is aimed at life safety while the lower level is intended for continued operation.

In these projects the two design events are termed as:

- The operating Design Earthquake (ODE), defined as the earthquake event that can reasonably be expected to occur during life design life of the facility (e.g., at least once). The ODE design goal is that the overall system shall continue operating during and after an ODE and experience little to no damage.

The Maximum Design Earthquake (MDE), defined as an event that has a small probability of exceedance during the facility life. The MDE design goal is that public safety shall be maintained during and after an MDE.

Loading Criteria:-

Maximum Design Earthquake (MDE).Given the performance goals of the MDE (i.e., public safety), the recommended seismic loading combinations using the load factor design method are as follows:

$U=D+L+E1+E2+EQ$ (Eq. 2-1)

Where , $E1$= effects due to vertical loads of earth and water

$E2$= effects due to horizontal loads of earth and water

EQ= effects due to design earthquake (MDE)

Comments on Loading Combinations for MDE

- The structure should first be designed with adequate strength capacity under static loading conditions.
- The structure should then be checked in terms of ductility as well as strength when earthquake effects, EQ, are considered. For tunnel structures, the earthquake effect is governed by the displacement/deformations imposed on the tunnels by the ground.
- In checking the strength capacity, the effects of earthquake loading should be expressed in terms of internal moments and forces, which can be calculated according to the lining deformations imposed by the surrounding ground. If the "strength" criteria repressed by Equation 2-1 or 2-2 can be satisfied based on elastic structural analysis, no further provisions under the MDE are required. Generally the strength can easily be met when the earthquake loading intensity is low (i.e., in low seismic risk areas) and/or the ground is very stiff.

If the flexural strength of the tunnel lining, using elastic analysis and Equation 2-1 or 2-2, is found to be exceeded, one of the following two design procedures should be followed:

1. Provide sufficient ductility at the critical locations of the lining to accommodate the deformations imposed by the ground in addition to those caused by other loading effects. In general the more ductility is provided; the more reduction in earthquake forces can be made in evaluating the required strength. An inelastic "shear" deformation may result in strength degradation; it should always be prevented by providing sufficient shear strengths in structure members, particularly in the cut-and –cover rectangular frame.
2. Re-analyze the structure response by assuming the formation of plastic-hinge analysis, a redistribution of moments and internal forces will result.

If new plastic hinges are developed based on the results, the analysis is re-run by incorporating the new hinges until all potential plastic hinges are properly accounted for.

- For cut-and-cover tunnel structures, the evaluation of capacity using Equation 2-1 should consider the uncertainties associated with the loads E1 and E2. And their worst combination.
- In many cases, the absence of live load, L, may present a more critical condition than when a full live load is considered. Therefore, a live load equal to zero should also be used in checking the structural strength capacity using equations 2-1 and 2-2.

For the ODE, the seismic design loading combination depends on the performance requirements of the structural members. Generally speaking, if the members are to experience little to no damage during the lower level event (ODE), the inelastic deformations in the structure members should be kept low. The following loading criteria, based on load factor design, are recommended:

$$U = 1.05D + 1.3L + b_1 E1 + E2 + 1.3EQ$$

(Eq. 2-3)

b_1= 1.05 if extreme loads are assumed for E1 and E2 with little uncertainty.

- Structure should first be designed with adequate strength capacity under static loading conditions.
- For cut-and-cover tunnel structures, the evaluation of capacity using Equation 2-3 should consider the uncertainties associated with the loads E1 and E2, and their worst combination.
- Redistribution of movements (e.g., ACI 318) for cut-and-cover concrete frames is recommended to achieve a more efficient design.
- If the "strength" criteria expressed by Equation 2-3 or 2-4 can be satisfied based on elastic structural analysis, no further provisions under the ODE are required.
- If the flexural strength of the tunnel lining, using elastic analysis and is found to be exceeded, the structure should be checked for its ductility to ensure that the resulting inelastic deformations, if any, are small. If necessary, the structure should be redesigned to ensure the intended performance goals during the ODE.

3.0 Running Line Tunnel Design:

3.1 Overview:-

The response of tunnels to seismic shaking motions may be demonstrated in terms of three principal types of deformations (Owen and Scholl, 1981):

- Axial
- Curvature
- Ovaling tunnels

3.2 Types of Deformations:-

Axial and Curvature Deformations:-

Axial and curvature deformations develop in a horizontal or nearly horizontal linear tunnel when seismic waves propagate either parallel or obliquely to the tunnel. The tunnel lining design considerations for these types of deformations are basically in the longitudinal direction along the tunnel axis.

Figure 3 shows the idealized representations of axial and curvature deformations. The general behaviour of the linear tunnel is similar to that of an elastic beam subject to deformations or strains imposed by the surrounding ground.

Ovaling or Racking Deformations:-

The ovaling or racking deformations of a tunnel structure may develop when waves propagate in a direction perpendicular or nearly perpendicular to the tunnel axis, resulting in a distortion of the cross-sectional shape of the tunnel lining. Design considerations for this type of deformation are in the transverse direction.

Figure 4 shows the ovaling distortion and racking deformation associated with circular tunnels and rectangular tunnels, respectively. The general behaviour of the lining may be simulated as a buried structure subject to ground deformations under a two-dimensional, plane-strain condition.

Ovaling and racking deformations may be caused by vertically, horizontally or obliquely propagating seismic waves of any type.

3.3 Free-Field Axial and Curvature Deformations:-

Background:-

The intensity of earthquake ground motion is described by several important parameters, including peak acceleration, peak velocity, peak displacement, response spectra, duration and others. For aboveground structures, the most widely used measure is the peak ground acceleration and the design response spectra, as the inertial forces of the structures caused by ground shaking provide a good representation of earthquake loads.

Peak ground acceleration is not necessarily a good parameter, however, for earthquake design of underground structures such as tunnels,

because tunnel structures are more sensitive to the distortions of the surrounding ground than to the inertial effects. Such ground distortions- referred to in this report as free-field deformations/strains –are the ground deformations/strains caused by the travelling seismic waves without the structures belong present.

A Practical Approach to Describing Ground Behavior:-

For practical purposes, a simplified approach was proposed by Newmark (1968) and has been considered by others. This approach is based on theory of wave propagation in homogeneous, isotropic, elastic media. The ground strains are calculated by assuming a harmonic wave of any type propagating at an angle with respect to the axis of a planned structure.

Figure 5 (Kuesel, 1969) represents free-field ground deformations along a tunnel axis due to a sinusoidal shear wave with a wavelength, L, a displacement amplitude, D, and an angle of incidence, and therefore the maximum values of strain, is often made, because the angle of incidence, and therefore the maximum values of strain, is often made, because the angle of incidence for the predominant earthquake waves cannot be determined reliably.

Simplified Equations for Axial Strains and Curvature:-

The free-field axial strains and curvature due to shear waves and Rayleigh waves (surface waves) can be expressed as a function of angle of incidence, as shown in Table 1.

Equations caused by compressional P-waves are also available, but it is generally considered that they would not control the design.

Application of the strain equations presented in Table 1 requires knowledge of:-

- The effective wave propagation velocity
- The peak ground particle velocity
- The peak ground particle acceleration

The peak velocity and acceleration can be established through empirical methods field measurements, or site-specific seismic exposure studies. The effective wave propagation velocity in rock can be determined with reasonable confidence from in-situ and laboratory tests.

It has been suggested that for horizontally or obliquely propagating waves the propagation velocities in soil overburden are affected significantly by the velocities in the underlying rock. That is to say, the actual velocity values in the soils may be much higher than those calculated based on the soil properties alone (Hadjian and Hadley, 1981). This phenomenon is attributable to the problem of deformation compatibility. The motion of a soil particle due to a horizontally propagating wave above the rock cannot differ greatly from the motion of the rock, unless the soil slides on top of the rock (a very unlikely occurrence) or the soil liquefies. For a very deep (thick) soil stratum, however, the top of the soil stratum is less coupled to the rock and is more free to follow a motion that is determined by its own physical properties.

Wave Type		Longitudinal Strain (Axial)	Curvature
Shear Wave	General Form	$e = \frac{V_s}{C_s}\sin q \cos q$	$\frac{1}{r} = \frac{A_s}{C_s^2}\cos^3 q$
	Maximum Value	$e_{max} = \frac{V_s}{2C_s}$ for $q = 45$	$\frac{1}{r}_{max} = \frac{A_s}{C_s^2}$ for $q = 0$
Rayleigh Wave	General Form	$e = \frac{V_s}{C_s}\cos^2 q$	$\frac{1}{r} = \frac{A_R}{C_R^2}\cos^2 q$
	Maximum Value	$e_{max} = \frac{V_R}{C_R}$ for $q = 0$	$\frac{1}{r}_{max} = \frac{A_R}{C_R^2}$ for $q = 0$

Table 1. Free-Field Ground Strains

q = Angle of Incidence with respect to Tunnel Axis

r = radius of Curvature

V_S, V_R = Peak Particle Velocity for Shear Wave and Rayleigh, respectively

C_S, C_R = Effective Propagation Velocity for Shear Wave and Rayleigh Wave,

A_S, A_R = Peak Particle Acceleration for Shear Wave and Rayleigh Wave, respectively

3.4 Design Conforming to Free-Field Axial and Curvature Deformations:-

Background and Assumptions:-

When these equations are used, it is assumed that the structures experience the same strains as the ground in the free-field. The presence of the structures and the disturbance due to the excavation are ignored. This simplified approach usually provides an upper-bound estimate of the strains that may be induced in the structures by the travelling waves.

Design Example 1: The Los Angeles Metro:-

In this project, it was determined that a shear wave propagating at 45 degree (angle of incidence) to the tunnel axis would create the most critical axial strain within the tunnel structure.

- Design Earthquake Parameters: Peak Ground Acceleration, A_s =0.6 (maximum Design Earthquake, MDE)
- Peak Ground Velocity, V_s =3.2 ft/sec
- Soil surrounding Tunnel: Fernando Formation
- Effective Shear Wave Velocity: C_s =1360 ft/sec
- Tunnel structure: Cast-in-place circular segmented reinforced lining, with Radius R=10 feet

$$e_{max} = \pm\frac{V_s}{2C_s} \pm \frac{A_s R}{C_s^2}\cos^3 q$$

$$= \pm\frac{3.2}{2\text{x}1360} \pm \frac{0.6\text{x}32.2\text{x}10}{(1360)^2}\cos^3 45$$

Axial — Curvature

$$= \pm 0.00118 \pm 0.000037$$

$$= \pm 0.00122$$

e_{allow} **= 0.002**, the lining is considered adequate in compression under the Maximum Design Earthquake (MDE).

The calculated maximum axial strain (=0.00122) is cyclic in nature. When tension is in question, a plan concrete lining would obviously crack. The assumed lining is reinforced, however, and the opening of these cracks is transient due to the cyclic nature of seismic waves. Even in the unreinforced concrete lining cases, the lining generally is considered adequate as long as:

- The crack openings are small and uniformly distributed.
- The resulting tension cracks do not adversely affect the intended performance goals of the lining.

Applicability of the Free-Field Approach:-

The example presented above demonstrates the simplicity of the free-field deformation approach. Because it is an upper-bound assessment of the adequacy of his design. This approach offers a method for verification of a design rather than a design itself.

Note, however, that this method is:

- Pertinent to a tunnel structure that is flexible relative to its surrounding medium, such as all tunnels in rock and most tunnels in stiff soils. In this case it is reasonable to assume that the tunnel deforms according to its surrounding medium.

- Not desirable for situations involving stiff structures buried in soft soil, because under this condition, the calculated ground deformations may be too great (due to the soft nature of the soil) for the stiff structures to realistically accommodate. Once the calculated ground strain exceeds the allowable strain of the lining material, there is very little an engineer can do to improve his design.

  For instance , if the effective shear wave velocity of the previous example is reduced to 350 ft/sec to reflect a much softer soil deposit, the tunnel lining will then be subjected to a combined maximum axial strain of 0.0052 in compression (see Design Example 2 in the next section): If the free-field deformation approach were used in this case, it appears that the only solution to this problem would be to provide needless flexible joints, forming a chainlike-like tunnel structure to accommodate the ground deformation.

3.5 Tunnel-Ground Interaction:-

When it is stiff in its longitudinal direction relative to its surrounding soils, the tunnel structure resists, rather than conforms to, the deformations imposed by the ground. In general, the tunnel-ground system is simulated as an elastic beam on an elastic foundation, with the theory of wave propagating in an infinite, homogeneous, isotropic medium. When subjected to the axial and curvature deformations caused by the travelling waves in the ground, the tunnel will experience the following sectional forces.

- Axial forces, Q, on the cross=section due to the axial deformation
- Bending moments, M, and shear forces, V, on the cross-section due to the curvature deformation.

Simplified Interaction Equations:-

Maximum Axial Force: Q_{max}. Caused by a shear wave with 45 degree angle of incidence can be obtained:

$$Q_{max} = \frac{\dfrac{K_a L}{2\pi}}{1 + 2\left(\dfrac{K_a}{E_c A_c}\right)\left(\dfrac{L}{2\pi}\right)^2} D$$

(Eq. 3-1)

Where, L= wavelength of an ideal sinusoidal shear wave.

The calculated maximum axial force, $Q_{max,}$ shall not exceed an upper limit defined by the ultimate soil drag resistance in the longitudinal direction. This upper limit is expressed as:

Maximum Bending Moment, M_{max}. The bending moment resulting from curvature deformations is maximized when a shear wave is travelling parallel to the tunnel axis. The mathematical expression of the maximum bending moment is:

(Eq. 3-3)

Maximum Shear Force, V_{max}. The maximum shear force corresponding to the maximum bending moment is derived as:

- The tunnel-ground interaction effect is explicitly accounted for in these formulations. The ground stiffness and the tunnel stiffness are represented by spring coefficients (K_a or K_t) and sectional modulus (E_cA_c or E_cI_c), respectively.
- The application of these equations is necessary only when tunnel structures are built in soft ground.

  Of L, D, and K_t (or K_a) can be reasonably estimated.

  A reasonable estimate of the wave length can be obtained by

  $$L = T\,C_S$$

  T is the predominant natural period of the shear wave travelling in the soil deposit in which the tunnel is built, and C_s is the shear wave propagation velocity within the soil deposit.

  Often, T can also be represented by the natural period of the site. Dobry, Oweis and Urzua (1976) presented some procedures for estimating the natural period of a linear or equivalent linear model of a soil site.
- D, should be derived based on site-specific subsurface conditions by earthquake engineers. The displacement amplitude represents the spatial variations of ground motions along a horizontal alignment. The displacement spectrum chart prepared by Housner for the SF BART project was expressed by $D=4.9 \times 10^{-6} L^{1.4}$, where the units of D and L are in feet. This spectrum is intended for tunnel tubes in soft San Francisco Bay muds and was derived for a magnitude 8.2 earthquake on the San Andreas Fault. The equation shows clearly that:
  - The displacement amplitude increases with the wavelength.
  - For any reasonably given wavelength, the corresponding ground displacement amplitude is relatively small. Using the given wavelength and the corresponding displacement amplitude, the calculated free- field ground strains would be significantly smaller than those calculated using the simplified equations shown in Table 1:
  - The spring coefficients should be representative of the dynamic modulus of the ground under seismic loads.

The derivations should consider the fact that loading felt by the surrounding soil (medium) is alternately positive and negative due to the assumed sinusoidal seismic wave.

For preliminary design, it appears that the expressions suggested by St. John and Zahrah (1987) should serve the purpose:

$$K_t = K_a = \frac{16\pi G_m (1 - \nu_m)}{(3 - 4\nu_m)} \frac{d}{L}$$

(Eq. 3.6)

Where, **G_m = Shear**
n_m = Poisson's
d = diameter
L = wavelength

- A review of Equations 3-1, 3-3 and 3-4 reveals that increasing the stiffness of the structure , although it may increase the strength may attract more forces as a result.

A linear Tunnel in Soft Ground

Geotechnical Parameters:

- Effective shear wave velocity, C_S =350 ft/sec.
- Soil unit weight, g_t = 110 pcf =0.110 kcf.
- Soil Poisson's ratio, n_m =0.5 (saturated soft clay)
- Soil deposit thickness over rigid bedrock, H =100 ft.

Structural Parameters:

- Lining thickness, t =1 ft.
- Lining diameter, d =20 ft.
- Lining moment of inertia, I_c = 0.5 x 3148= 1574 ft^4 (one half of the full section moment of inertia to account for concrete cracking and nonlinearity during the MDE).
- Lining cross section area, A_c =62.8 ft^2
- Concrete Young's modulus, E_c =3600 ksi =518400 ksf.
- Concrete yield strength, f_c =4000 psi.
- Allowable concrete compression strain under combined axial and bending compression, e_{allow} = 0.003 (during the MDE)

Earthquake Parameters (for the MDE)

- Peak ground particle acceleration in soil, A_s =0.6 g.
- Peak ground particle in soil, V_s =3.2 ft/sec.

First, try the simplified equation as used in Design Example 1. The combined maximum axial strain and curvature strain is calculated as:

$$e_{max} = \pm\frac{V_s}{2C_s} \pm \frac{A_s R}{C_s^2}\cos^3\theta = \pm\frac{3.2}{2\times 350} \pm \frac{0.6\times 32.2\times 10}{(350)^2}\cos^3 45^\circ$$

$$= \pm 0.0046 \pm 0.0006 = \pm 0.0052$$

(i.e., $e_{max} > e_{allow}$ =0.003)

Now use the tunnel-ground interaction procedure.

1. Estimate the predominant natural period of the soil deposit

$$T = \frac{4H}{C_s} = \frac{4\times 100'}{350} = 1.14 \text{ sec.}$$

2. Estimate the idealized wavelength

$$L = T\times C_s = 4H$$

$$= 400\ ft$$

3. Estimate the shear modulus of soil:

$$G_m = \rho C_s^2 = \frac{0.110 kcf}{32.2}\times 350^2 = 418.5 ksf$$

4. Derive the equivalent spring coefficients of the soil

$$K_a = K_t = \frac{16\pi G_m (1-\nu_m)}{(3-4\nu_m)}\frac{d}{L}$$

$$= \frac{16\pi\times 418.5\ (1-0.5)}{(3-4\times 0.5)}\times\frac{20}{400}$$

$$= 526\ kips/ft$$

5. Derive the ground displacement amplitude, D: Wavelength, site-specific subsurface.

For free- field axial strain:

$$\frac{V_s}{2C_s} = \frac{2\pi D}{L} \text{ fi } D = D_a = 0.291 ft$$

For free-field bending curvature:

$$\frac{A_s}{C_s^2}\cos^3 45^\circ = \frac{4\pi^2 D}{L^2} \text{ fi } D = D_b = 0.226\ ft$$

6. Calculate the maximum axial force (Equation 3-1) and the corresponding axial strain of the tunnel lining:
7. Calculate the maximum bending moment (Equation 3-3) and the corresponding bending strain of the tunnel lining:

$$M_{max} = \frac{K_t\left(\frac{L}{2\pi}\right)^2}{1+\frac{K_t}{E_c I_c}\left(\frac{L}{2\pi}\right)^4} D_b$$

$$e_{bending} = \frac{M_{max} R}{E_c I_c}$$

$$= 0.00051$$

8. Compare the combined axial and bending compression strains to the allowable:

$$e_{max} = e_{axial} + e_{bending}$$

$$= 0.00077 < e_{allow} = 0.003$$

9. Calculate the maximum shear force (Equation 3-4) due to the bending curvature:

$$= 652\ kips$$

10. Calculate the allowable shear strength of concrete during the MDE:

$$\phi V_c = 0.85\times 2\sqrt{f_c}\, A_{shear}$$

Where, f = shear strength reduction factor (0.85)
Note: Use of f= 0.85 for earthquake design may be very conservative.

11. Compare the induced maximum shear force with the allowable shear resistance:

$$V_{max} = 625\ kips > \phi V_c = 486\ kips$$

This problem may not be of major concern in actual design because:

- The nominal reinforcements generally required for other purposes may provide additional shear resistance during earthquakes.
- The ground displacement amplitudes, D, used in this example are very conservative. Generally the spatial variations of ground displacements along a horizontal axis are much smaller than those used in this example, provided that there is no abrupt change in subsurface profiles.

3.6 Special Considerations:-

- Unstable ground, including ground that is susceptible to landslide and/or liquefaction
- Faulting, including tectonic uplift and subsidence
- Abrupt changes in structural stiffness or ground conditions

Unstable Ground:

It is generally not feasible to design a tunnel lining of sufficient strength to resist large permanent ground deformations resulting from an unstable ground.

- Ground stabilization (compaction, draining, reinforcement, grouting, and earth retaining systems)
- Removal and replacement of problems soils.
- Reroute or deeper burial.

Faulting:

With regard to fault displacements, the best solution is to avoid any potential crossing of active faults. If this not possible, the general design a tunnel structure to accept and accommodate these fault displacements. For example, in the North Outfall Replacement Sewer (NORS, City of LOS Angeles) project, the amount of fault displacement associated with an M=605 design earthquake on the Newport-Inglewood fault was estimated to be about 8 inches at the crossing. To accommodate this displacement, a design scheme using an oversized excavation and a compressible backpacking material was provided. The backpacking material was designed to withstand the static loads, yet to crushable under faulting movements to protect the pipe.

It is believed that the only transportation tunnel in the U.S. designed and constructed to take into consideration potential active fault displacements is the Berkeley Hills Tunnel, part of the San Francisco BART system. This horse-shoe-shaped tunnel was driven through the historically active creeping Hayward Fault with a one-foot oversized excavation. The purpose of the over-excavation was to provide adequate clearance for rail in this section could be realigned and train services could be resumed quickly afterward.

The tunnel was lined with concrete encased ductile steel ribs on two-foot centers. The concrete encased steel rib lining is particularly suitable for this design because it provides sufficient ductility to accommodate the lining distortions with little strength degradation.

These conditions include, but are not limited to, the following:

- When a regular tunnel section is connected to a station end wall or a rigid, massive structure such as a ventilation building.
- At the junctions of tunnels.
- When a tunnel traverses two distinct geological media with sharp contrast in stiffness
- When tunnels are locally restrained from movements by any means.
- A movable joint, such as the one used at the connection between the Trans-Bay tube and the ventilation building (Warshaw, 1968)
- A rigid connection with adequate strength and ductility.

Structures are subjected to potential differential movements due to the difference in stiffness of two adjoining structures or geological media requiring a dynamic analysis talking into account the soil-structure interaction effect. The calculated different movements provide necessary data for further evaluations to determine whether special seismic joints are needed.

A linear tunnel entering a large station may experience a transverse differential deflection between the junction and the far field due to the large shear rigidity provided by the end wall of the station structure. If a conventional design a rigid connection at the interface is proposed, additional bending and shearing stresses will develop near the interface. These stress concentrations can be evaluated by assuming a semi-infinite beam supported on an elastic foundation, with a fixed end at the connection. According to Yeh (1974) and Hetenyi (1976), the induced moment, M(x), and shear, V(x), due to the differential transverse deflection, d, can be estimated as:

$$M(x) = \frac{K_t}{2l^2} d e^{-lx} (\sin lx - \cos lx)$$

$$V(x) = \frac{K_t}{l} d e^{-lx} \cos lx \qquad \text{(Eq. 3-7)}$$

$$l = \left(\frac{K_t}{4E_c I_c}\right)^{\frac{1}{4}} \qquad \text{(Eq. 3-8)}$$

Where, x = distance from the connection

I_C =moment of inertia of the tunnel cross section

E_C =Young's modulus of the tunnel lining

K_t =transverse spring coefficient of ground

(In force per unit deformation per unit length of tunnel)

The maximum bending moment and shear force occur at x=0 (i.e., at the connection). If it is concluded that an adequate design cannot be achieved by using the rigid connection scheme, then special seismic (movable) joints should be considered.

4.0 OVALING EFFECT ON CIRCULAR TUNNELS

4.1 Ovaling Effect:-

Ovaling of a circular tunnel lining is primarily caused by seismic waves propagating in planes perpendicular to the tunnel axis (see Figure 2). Usually, it is the vertically propagating shear waves that produce the most critical ovaling distortion of the lining. The results are cycles of additional stress concentrations with alternating compressive and tensile stresses in the tunnel lining. These dynamic stresses are superimposed on the existing static state of stress in the lining.

- Compressive dynamic stresses added to the compressive static may exceed the compressive capacity of the lining locally.

Tensile dynamic stresses subtracted from the compressive static stresses reduce the lining's moment capacity, and sometimes the resulting stresses may be tensile.

4.2 Free-Field Deformations:

Vertically propagating shear waves causes a circular tunnel to oval and a rectangular underground structure to rack (sideways motion), as shown in Figure 3. Numerical methods are often required to arrive at a reasonable estimate of the free-field codes with variable degree of sophistication are available (e.g., SHAKE, 1972; FLUSH, 1975; and LINOS, 1991).

The most widely used approach is to simplify the site geology into a horizontally layered system and to derive a solution using one-dimensional wave propagation theory.

Simplified Equation for Shear Deformations:

For a deep tunnel located in relatively homogeneous soil or rock, the simplified procedure by Newmark (presented in Table 1) may also provide a reasonable estimate. Here, the maximum free-field shear strain, g_{max}, can be expressed as:

$$g_{max} = \frac{V_s}{C_s}$$

(Eq. 4-1)

Where, **V_s =peak particle velocity**

C_s =effective shear wave propagation velocity

The values of C_s can be estimated from in-situ and laboratory tests. An equation relating the effective propagation velocity of shear waves to effective shear modulus, G_m, is expressed as:

$$C_s = \sqrt{\frac{G_m}{r}}$$

(Eq. 4-2)

Where, **r =mass density of the ground**

The propagation velocity and the shear modulus to be used should be compatible with the level of shear strains that may develop in the ground under design earthquake loading. This is particularly critical for soil sites due to the highly non-linear behaviour of soils. The following data are available:

- Seed and Idriss (1970) provide an often used set of laboratory data for soils giving the effective shear wave velocity and effective shear modulus as a function of shear strain.
- Grant and Brown (1981) further supplemented the data sets with results from a series of field geophysical measurements and laboratory testing conducted for six soil sites.

4.3 Lining Conforming to Free-Field Shear Deformations:

The lining's transverse sectional stiffness is completely ignored. This assumption is probably reasonable for most circular tunnels in rock and in stiff soils, because the lining stiffness against distortion is low compared with that of the surrounding medium.

If the non-perforated ground in the free-field is used to derive the shear distortion surrounding the tunnel lining, the lining is to be designed to conform to the maximum diameter change, ΔD, shown in figure 8. The diameter strain of the lining for this case can be derived as:

$$\frac{\Delta D}{D} = \pm \frac{g_{max}}{2}$$

(Eq. 4-3)

Where, **D = the diameter of the tunnel**

G_{max} = the maximum free-field shear strain

On the other hand, if the ground deformation is derived by assuming the presence of a cavity due to tunnel excavation, then the lining is to be designed according to the diameter strain expressed as:

$$\frac{\Delta D}{D} = \pm 2g_{max}(1 - v_m)$$

(Eq. 4-4)

Where, **n_m = the Poisson's Ratio of the medium**

Equations 4-3 and 4-4 both assume the absence of the lining. In other words, tunnel-ground interaction is ignored.

The perforated ground deformation would yield a much greater distortion than the non-perforated, free-field ground deformation by intuition:

- Equation 4-4, the perforated ground deformation, should serve well for a lining that has little stiffness (against distortion) in comparison to that of the medium.
- Equation 4-3, on the other hand, should provide a reasonable distortion criterion for a lining with distortion stiffness equal to the surrounding medium.

It is logical to speculate further that a lining with a greater distortion stiffness than the surrounding medium should experience a lining distortion even less than that calculated by Equation. This latest case may occur when a tunnel is built in soft to very soft soils.

4.4 Importance of Lining Stiffness:

To quantify the relative stiffness between a circular lining and the medium two ratios designated as the compressibility ratio C, and the flexibility ratio, F are defined by the following equations:

$$C = \frac{E_m (1 - v_1^2) R}{E_1 t (1 + v_m)(1 - 2v_m)}$$

Compressibility Ratio,

(Eq. 4-5)

$$F = \frac{E_m (1 - v_1^2) R^3}{6 E_1 I (1 + v_m)}$$

Flexibility Ratio,

(Eq. 4-6)

Where, n_l = Poisson's Ratio of the tunnel lining.

Of these two ratios, flexibility ratio is the more important because it is related to the ability of the lining to resist distortion imposed by the ground.

Flexibility ratio equal to 1.0 implies that the lining may just have enough stiffness to replace that of the soil being excavated. Ideally, the lining should distort in accordance with the free-field, non-performed ground deformation.

Summary and Conclusions:

The ovaling effects on the lining, however, may in some cases be overestimated, depending on the relative stiffness between the ground and the lining. The main reason for this drawback is the uncertainty of the tunnel-ground interaction.

This drawback, however, may be immaterial for most applications in the real world. For most circular tunnels encountered in practice, the flexibility ratio, F, is likely to be large enough (F>20) so that the tunnel-ground interaction effect can be ignored (Peck, 1972). In these cases, the distortions to be experienced by the lining can be reasonably assumed to be equal to those of the performed ground. Problems arise when a very stiff structure is surrounded by a very soft soil. A typical example would be to construct a very stiff immersed tube in a soft lake or river bed.

What are Stereonet. Programming in MATLAB for Developing Stereonet and Polar Plot. Using MATLAB to Plot Stereonet and Polar Plot of Chenab Bridge Site

There are many methods to represent spherical features in two dimensional plane such as orthographic, cylindrical projection etc. These things has been extensively dealt in cartography where earth's surface have been plot- ted in two dimension sheet. Stereographic projections are frequently used by engineers as it helps geometrical construction of the problem. Stereographic projection is also called equal angle projection so as to distinguish it from another type of projection called equal area projection, which is also being used by geologist and engineers. As its name suggest and in comparison to equal area projection the grid formed in stereographic projection are not of same area. Equal angle projection preserve equal angle in projection.

Sterionets are used to represent geological data graphically as shown in Figure 2. Now with the use of computer programing such graphical representation can be directly plotted without using aid like stereonet.

Stereographic projection is projection of any point on the surface of sphere to the horizontal plan containing the center of sphere when viewed from the top of sphere as shown by Figure 1. Such projection can be mathematically shown as expressed below if any point on sphere Q(X, Y, Z) then its stereo- graphic projection on a plan Z = 0 when viewed from the point P (0, 0, R), where R is the radius of sphere is T(x, y, z) defined as

$$x = \frac{X}{1-\frac{Z}{R}} \qquad (1)$$

$$y = \frac{Y}{1-\frac{Z}{R}} \qquad (2)$$

$$Z=0 \qquad (3)$$

The stereographic projection of any point laying on the upper hemisphere will fall outside the plan bounded by a circle of corresponding radius that of sphere and any point on lower hemisphere will be inside on that circle. Since in rock engineering it is a plan and its various other interface which are being represented with will be falling in both upper and lower hemisphere, so, only one hemisphere projection is contemplated, generally lower hemisphere, rea- son being that plane could be represented within the circle that will require lesser space for graphical representation (Upper hemisphere points can also be plotted using the same logic of lower hemisphere projection by changing the co-ordinate reference so that with respect to this co-ordinate it lies in lower hemisphere). Before going into details about stereographic projection strike, dip and dip direction has been explained which are very frequently used to represent a plane in rock mechanics.

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Strike: An inclined plane is expressed by the orientation of a line which is formed as a result of its intersection with horizontal plan. In other words when two plane intersects, it forms a line which lies in both the plan. The orientation of that line if expressed in terms of geographical reference such as N-S or E-W is the strike of a plane where the other intersecting plane is taken as horizontal plane as standard reference.

Dip: This is an angle by which a plane is inclined with with the horizontal plane.

Dip Direction: Dip direction is the orientation or bearing of a line in the horizontal plane and perpendicular to the strike.

Take a point Q on the sphere (representing earth) of radius R, refer Figure 2. Stereographic projection of point Q is the co-ordinate of point T in horizontal plane. Point Q also lie on the inclined plane inclined by angle δ from the horizontal and measured in vertical plane. Refer Figure 3. Clearly, $Z = X.\tan(\delta)$. $x = \frac{X}{1-\frac{Z}{R}}$

$$x = \frac{X}{1-\frac{Z}{R}} = \frac{X}{1-\frac{X.\tan(\delta)}{R}} = \frac{X.R}{R-X.\tan(\delta)}$$

$X = \frac{x.R}{R+x.\tan(\delta)}$, Similaly,

$$y = \frac{Y}{1-\frac{Z}{R}} = \frac{Y}{1-\frac{X.\tan(\delta)}{R}} = \frac{Y.R}{R-X.\tan(\delta)} = \frac{Y.R}{R-\tan(\delta).\frac{x.R}{R+x.\tan(\delta)}} = \frac{Y.(R+x.\tan(\delta))}{R},$$

$$Y = \frac{Y.R}{R+x.\tan(\delta)},$$

$$X^2 + Y^2 + Z^2 = R^2,$$

$$X^2 + Y^2 + X^2.\tan^2(\delta) = R^2,$$

$$[1+\tan^2(\delta)].X^2 + Y^2 = R^2,$$

$$[1+\tan^2(\delta)].x^2 + y^2 = [R + x.\tan(\delta)]^2,$$

$$x^2 + y^2 - 2.R.x.\tan(\delta) = R^2,$$

$$[x - R.\tan(\delta)]^2 + y^2 = R^2[1+\tan^2(\delta)],$$

$$[x - R.\tan(\delta)]^2 + y^2 = [R.\sec(\delta)]^2,$$

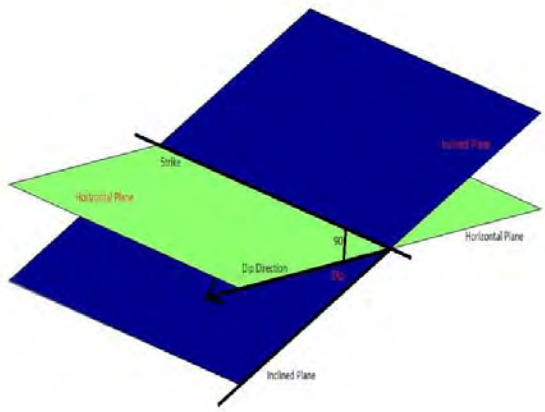


Figure 1: Intersection of two planes

The above equation represents a circle. Therefore the locus of a point laying on the surface of a sphere and also on the inclined plane with angle δ, represents a circle with center $(R.tan\delta)$ and radius equals to $R.sec\delta$. Figure 3 shows the stereographic projection of the points formed by the intersection of inclined plane and sphere in lower hemisphere. The inclined plane discussed here are the great circles, having the same centre as that of the sphere. Therefore plotting of stereographic projection all such great circles in lower hemisphere will form longitude type circles within sphere.

If stereographic projections of great circles in lower hemisphere are plotted with different dip angle, it will form longitude type plots within the sphere. Since a great circles in a given hemisphere subtends an angle of 180o at the center and if this angle is divided in some equal number of parts then connecting the stereographic projection of all such points of equal angle of different great circles forms a latitude type plots within the sphere as shown in figure 4. Such plot is called stereonets or Wullf net and used for manual plotting of planes. Stereonet is generally plotted at 2o spacing so that the maximum error will be less that a degree, refer Figure 4, which shows stereonet plotted @ 10o . A normal 2° spaced stereonet can be plotted by using a MATLAB function **stereonet(2,2). This paper discuss the formulation used for programming of stereographic projection and also share the programe for common use.**

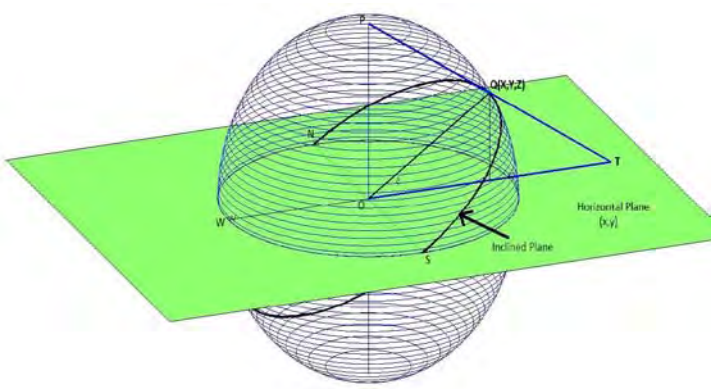


Figure 3: Stereographic projection of a point

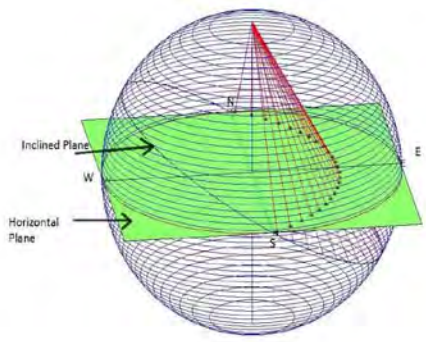


Figure 4: Horizontal Plane

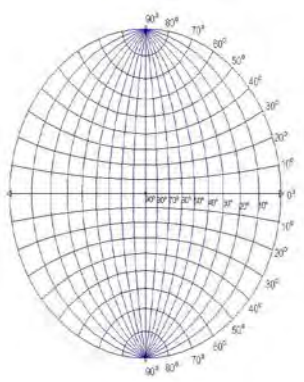


Figure 5: Stereonet @ 10°

1. ***Plotting of a plane and its pole***: Refer Figure 5, which shows an inclined plane with its local axes defined as(X^1, Y^1, Z^1). Axes reference defined as (X, Y, Z) , when rotated about its Z axis in such a way that its X and Y axis gets rotated by an angle β in the horizontal plane. Thus the new orientation of axes will be(X_1, Y^1, Z). If (X_1, Y^1, Z) is rotated about Y^1 axis in such a way that Z and X1 axis gets rotated by an angle α. Thus the new orientation of axes will be(X^1, Y^1, Z^1). The inclined plane can be defined by (X^1, Y^1, Z^1) as its local co-ordinate. More specifically this inclined plane laying in X^1, Y^1 plane and Z^1 is a normal to that plane. Therefore OY^1 is the strike and OX_1 is dip direction and α is the dip angle. Stereographic projection is defined with respect to (x, y) as defined by equation 1, 2 and 3. Any point defined in local co-ordinate system (X_1, Y^1, Z^1) can be converted to global co-ordinate system (X, Y, Z) by using transformation T defined as,

$$T = \begin{bmatrix} cos(X, X^1) & \cos(X, Y^1) & \cos(X, Z^1) \\ \cos(Y, X^1) & \cos(Y, Y^1) & \cos(Y, Z^1) \\ \cos(Z, X^1) & \cos(Z, Y^1) & \cos(Z, Z^1) \end{bmatrix}$$

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix} = T.\begin{bmatrix} X^1 \\ Y^1 \\ Z^1 \end{bmatrix},$$

In three dimension, it is not easy to determine the cosines as defined by T matrix. Required transformation between (X, Y, Z) and (X^1, Y^1, Z^1) can be obtained sequentially, first by transforming any point defined in (X^1, Y^1, Z^1) to (X_1, Y^1, Z) and then (X_1, Y^1, Z) to(X, Y, Z), as shown below,

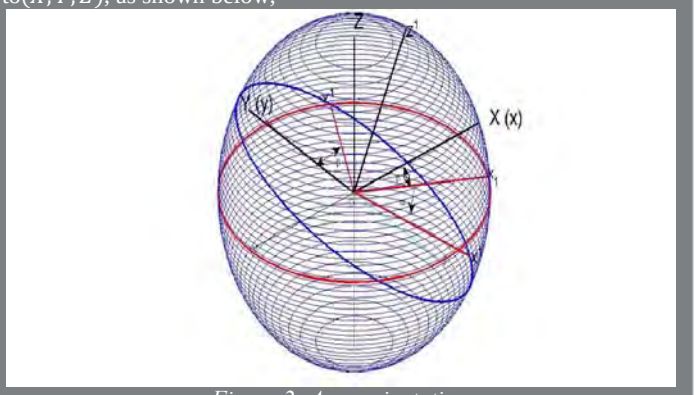


Figure 2: Axes orientation

$$\begin{pmatrix} X_1 \\ Y^1 \\ Z \end{pmatrix} = \begin{bmatrix} cos(X_1, X^1) & \cos(X_1, Y^1) & \cos(X_1, Z^1) \\ \cos(Y^1, X^1) & \cos(Y^1, Y^1) & \cos(Y^1, Z^1) \\ \cos(Z, X^1) & \cos(Z, Y^1) & \cos(Z, Z^1) \end{bmatrix}.\begin{pmatrix} X^1 \\ Y^1 \\ Z^1 \end{pmatrix}$$

$$\begin{pmatrix} X_1 \\ Y^1 \\ Z \end{pmatrix} = \begin{bmatrix} cos(\alpha) & \cos(90^o) & \cos(90^o - \alpha) \\ \cos(90^o) & \cos(0^o) & \cos(90^o) \\ \cos(90^o + \alpha) & \cos(90^o) & \cos(\alpha) \end{bmatrix}.\begin{pmatrix} X^1 \\ Y^1 \\ Z^1 \end{pmatrix}$$

$$\begin{pmatrix} X \\ Y \\ Z \end{pmatrix} = \begin{bmatrix} cos(X, X_1) & \cos(X, Y^1) & \cos(X, Z) \\ \cos(Y, X_1) & \cos(Y, Y^1) & \cos(Y, Z) \\ \cos(Z, X_1) & \cos(Z, Y^1) & \cos(Z, Z) \end{bmatrix}.\begin{pmatrix} X_1 \\ Y^1 \\ Z \end{pmatrix}$$

$$\begin{pmatrix} X \\ Y \\ Z \end{pmatrix} = \begin{bmatrix} cos(\beta) & \cos(90^o - \beta) & \cos(90^o) \\ \cos(90^o + \beta) & \cos(\beta) & \cos(90^o) \\ \cos(90^o) & \cos(90^o) & \cos(0^o) \end{bmatrix}.\begin{pmatrix} X_1 \\ Y^1 \\ Z \end{pmatrix}$$

$$\begin{pmatrix} X \\ Y \\ Z \end{pmatrix} = \begin{bmatrix} cos(\beta) & \sin(\beta) & 0 \\ -\sin(\beta) & \cos(\beta) & 0 \\ 0 & 0 & 1 \end{bmatrix}.\begin{bmatrix} cos(\alpha) & 0 & \sin(\alpha) \\ 0 & 1 & 0 \\ -\sin(\alpha) & 0 & \cos(\alpha) \end{bmatrix}.\begin{pmatrix} X^1 \\ Y^1 \\ Z^1 \end{pmatrix},$$

$$\begin{pmatrix} X \\ Y \\ Z \end{pmatrix} = \begin{bmatrix} cos(\beta).\cos(\alpha) & \sin(\beta) & \cos(\beta).\sin(\alpha) \\ -\sin(\beta).\cos(\alpha) & \cos(\beta) & -\sin(\beta).\sin(\alpha) \\ -\sin(\alpha) & 0 & \cos(\alpha) \end{bmatrix} . \begin{pmatrix} X^1 \\ Y^1 \\ Z^1 \end{pmatrix} \quad (4)$$

It is pertinent to mention that for an inclined plane as defined above and in their local co-ordinate system (X^1, Y^1, Z^1) $Z^1 = 0$. Thus for a given plane by using equation 4 any point on that plane, defined with respect to its local axes can be transformed to a fixed co-ordinate system and subsequently stereographic projection of that point can be obtain by using equations 1, 2 and 3.

The pole of a plane is the stereographic projection of that point of the sphere which the normal of the plane passing through its center cuts at the surface of the sphere. Such point on the sphere can be determined by taking the cross product of the two vectors laying in the plane. For example, let's take two points on the plane, defined in its local co-ordinate axes (X^1, Y^1, Z^1) as $A(R, 0, 0)$ and $B(0, R, 0)$. These points can be transformed to global co- ordinate by using equation 4. Say, these transformations are $A(x_a, y_a, z_a)$ and $B(x_b, y_b, z_b)$. Since all plane contain center of sphere, so $\overrightarrow{OA} = x_a\vec{i} + y_a\vec{j} + z_a\vec{k}$ and $\overrightarrow{OB} = x_b\vec{i} + y_b\vec{j} + z_b\vec{k}$. Then $\overrightarrow{OA} \times \overrightarrow{OB}$ is a vector normal to the plane under consideration. Normalizing this vector to the magnitude of the radius of sphere will give the co-ordinate on the surface of sphere where the normal of the plane cuts the sphere.

A **matlab function spp(strike,dip,pole)** plots the stereographic projection of a plane and its pole. Poles are often plotted to figure out predominant orientation of plane which take lesser space than plotting planes. Before feeding the data in the function it should be ensured that the dip direction is measured 90° clockwise from strike line, dip is measured positive in lower hemisphere and measured from the dip direction in vertical plane similarly it is negative for upper hemisphere, pole=1, to plot pole and pole=0, to suppress the plot of pole. Matlab function [d,p]=pole(strike,dip) gives the co-ordinate of the pole, d on the surface of the sphere and p is its stereographic projection i.e. pole as discussed above.

2. **Stereographic projection of a line**: Take a line which is passing through the center of sphere cuts the surface of sphere at point A and B in upper and lower hemisphere respectively. If only one hemisphere is considered then OB will be that line. Stereographic projection of line OB represented by the projection of point B as the point O already lies in the horizontal plane.

   A matlab function [d]=LHLine(bearing,plunge), where d is the co-ordinate of the point on the surface of sphere cut by the line. Bearing is the orientation of line in horizontal plane and plunge is the dip of line from horizontal plane along the vertical. Plunge is positive in lower hemisphere and negative in upper hemisphere. Bearing and plunge are similar to dip direction and dip.

3. Stereographic projection of intersection of two planes: Intersection of two plane which are great circles will be a line passing through the center of sphere. Let the two planes are defined by their normal vectors as $\overrightarrow{R_1} = a\vec{i} + b\vec{j} + c\vec{k}$ and $\overrightarrow{R_2} = m\vec{i} + n\vec{j} + p\vec{k}$ where $a^2 + b^2 + c^2 = 1$ and $m^2 + n^2 + p^2 = 1$, so that $\overrightarrow{R_1}$ and $\overrightarrow{R_2}$ become unit vectors. If $\overrightarrow{R_3} = x\vec{i} + y\vec{j} + z\vec{k}$ represent the intersection line and (x, y, z) are a point on the surface of the sphere, then $x^2 + y^2 + z^2 = R^2$, R is the radius of the sphere. $\overrightarrow{R_3}.\overrightarrow{R_1} = 0$ and $\overrightarrow{R_3}.\overrightarrow{R_2} = 0$

$$\boldsymbol{ax + by + cz = 0} \quad 5$$

$$\boldsymbol{mx + ny + pz = 0} \quad 6$$

On eliminating x from equation 5 and 6,

$$\frac{y}{z} = \frac{ap - mc}{bm - an} = k_1 \quad 7$$

On eliminating y from equation 5 and 6,

$$\frac{x}{z} = \frac{bp - cn}{an - bm} = k_2 \quad 8$$

Since, $x^2 + y^2 + z^2 = R^2$,
$z^2.(1 + k_1^2 + k_2^2) = R^2$,
$z = -\frac{R}{\sqrt{(1+k_1^2+k_2^2)}}$,

Taking only negative value for lower hemisphere projection. Now using equation 7 and 8, $y = k_1.z$ and $x = k_2.z$. Projection of point (x, y, z) as per equation 1, 2 and 3 is the stereographic projection of intersection of two planes. Intersection of two planes form a line, its bearing will be defined by the slope of the line connecting points (0, 0) and (x, y) with y axis (assumes as North, as bearing usually measured from north) in horizontal plane. The plunge of this line is the angle by which it dips down from the horizontal plane along its vertical, may be expressed as, $\sin^-[\frac{|z|}{\sqrt{(x^2+y^2+z^2)}}] = \sin^-[\frac{|z|}{R}]$

A matlab function [Ps,bearing, plunge]=sisp(strike1,dip1,strike2,dip2), find the properties of line of intersection of two planes. Strike and dip of the two planes are the inputs for this function. Ps is the stereographic projection of the line of intersection. Bearing of line of intersection is measured from north in clockwise direction. Plunge is measured positive in lower hemisphere.

Another matlab function **[Y]=anglebetline(P_1, P_2),** gives the angle (Y) in degree between the two line OP_1 and OP_2 passing through the center of sphere. $P_1 = [x_1, y_1, z_1]$ and $P_1 = [x_2, y_2, z_2]$ are the points on the sphere cut by the lines.

Plane not passing through the center of sphere: Any plane cutting the great sphere will form a circle, containing those points on the sphere. If the plane contains the center of sphere then that circle will be great circles with radius R and center of sphere as its center. If the plane does not contain the center of sphere then that circle will have radius lesser than R and different center. As explained by the figure 6, the locus of line making a constant angle with another line is a cone. The intercept of such cone on the surface of sphere will be a circle. Let us take a line (On) lying on (X, Z) plane with slope φ. The locus of lines making an angle of α with the line on is a cone as depicted in Figure 6.

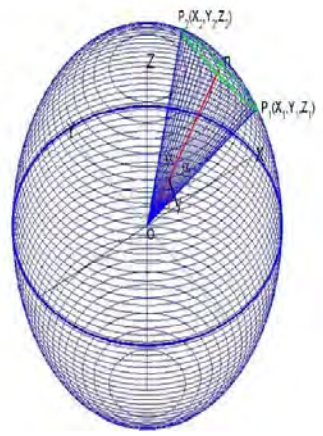


Figure 6: A circle, plane of which not containing center of sphere

For point P_1 and P_2 lying in (X, Z) plane (i.e the same plane which contains line On and the plane in which angle φ has been measured), $Y_1 = Y_2 = 0$, Following expression will also hold good, for any point (X, Y, Z) lying on the circle formed by the cone,

$$\frac{(Z_2-Z_1)}{(X_2-X_1)} = \frac{(Z-Z_1)}{(X-X_1)}$$

$$Z = Z_1 - X_1.\frac{(Z_2-Z_1}{X_2-X_1} + \frac{(Z_2-Z_1)}{(X_2-X_1)}.X,$$

$$Z = Z_o + K.X,$$

$$Z = Z_o + K.X \qquad 9$$

Where, $K = \frac{(Z_2-Z_1)}{(X_2-X_1)}$ and $Z_0 = Z_1 - K.X_1$

$$(X_1, Z_1) = [R.\cos(\phi \pm \alpha), R.\sin(\phi \pm \alpha)]$$

Stereographic projection of this half circle, equation 1, 2 and 3 can be used as expressed below,

$$x = \frac{X}{1-\frac{Z}{R}} = \frac{X.R}{R-Z} = \frac{X.R}{R-Z_o-K.X}$$

$$(R+K.x)X = x(R-Z_o)$$

$$X = \frac{x.(R-Z_o)}{(R+K.x)} \qquad 10$$

Similarly, $y = \frac{Y}{1-\frac{Z}{R}} = \frac{Y.R}{R-Z} = \frac{Y.R}{R-Z_o-K.X}$, placing the value of X from equation 10,

$$y = \frac{Y.R}{R-Z_o-K.X} = \frac{Y.R}{R-Z_o-K.\frac{x(R-Z_o)}{(R+K.x)}} = \frac{Y.(R+K.x)}{(R-Z_o)}$$

$$Y = \frac{y.(R-Z_o)}{(R+K.x)} \qquad 11$$

Putting the value of X from equation 10 into equation 9

$$Z = Z_o + K.X = Z_o + K.\frac{x.(R-Z_o)}{R+K.x} = \frac{R.(Z_o+K.x)}{(R-K.x)}$$

$$Z = \frac{R.(Z_o+K.x)}{(R+K.x)} \qquad 12$$

Since, any point (X, Y, Z) also lies on sphere, so $X^2+Y^2+Z^2 = R^2$. Putting the values from equation 10, 11 and 12

$$(R-Z_o)^2.(x^2+y^2) + R^2.(Z_o+K.x)^2 = R^2.(R+K.x)^2$$

$$(R-Z_o)^2.(x^2+y^2) + R^2.[(Z_o+K.x)^2 - (R+K.x)^2 = 0$$

$$(R-Z_o)^2.(x^2+y^2) + R^2.[(Z_o-R)(Z_o+R+2K.x)] = 0$$

$$(R-Z_o).(x^2+y^2) - R^2.(Z_o+R+2K.x) = 0$$

$$x^2+y^2 - \frac{R^2}{(R-Z_o)}.(Z_o+R+2K.x) = 0$$

$$\left(x - K.\frac{R^2}{R-Z_o}\right)^2 + y^2 = \frac{R^2.(R+Z_o)}{(R-Z_o)} \qquad 13$$

This represents a circle. Therefore stereographic projection of half circle is also a circle. The above expression has been derived for zero strike. For a definite value of strike, axis transformation method as explained earlier may be adopted or graphically it is achieved by rotating the stereonet.

A matlab function **halfcircle(strike,dip,alpha)** plots stereographic projection of such a plane having given strike, dip and angle alpha (angle between normal to the plane and conical surface), alpha =90o plots great circles.

Examples

1. Find out the stereographic projection of a plane having strike N 30° E and dip 20° to E 30^0 S. Also show the pole of the plane.

Given Strike = N 30° E, Dip = 20^0

Run function, spp(30,20,1) on matlab console after setting the correct path where files are stored.

>> **spp(30,20.1)**

to plot it on stereonet, run following commands,

>> **spp(30,20,1);** hold on; stereonet(2,2).

See Figure 7 and Figure 8

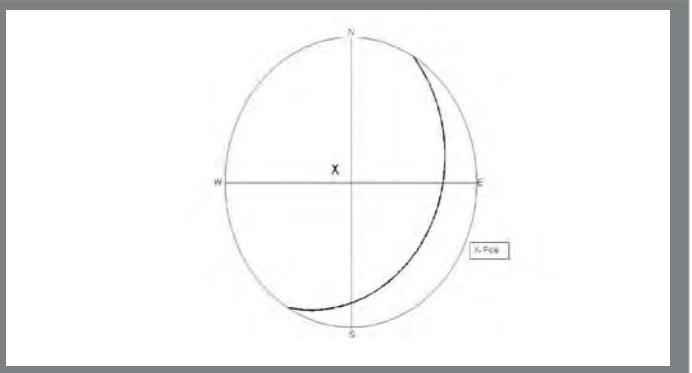


Figure 7: Stereographic projection of a plane, Strike $N30^oS$, dip $= 20^o$ to $E30^oS$

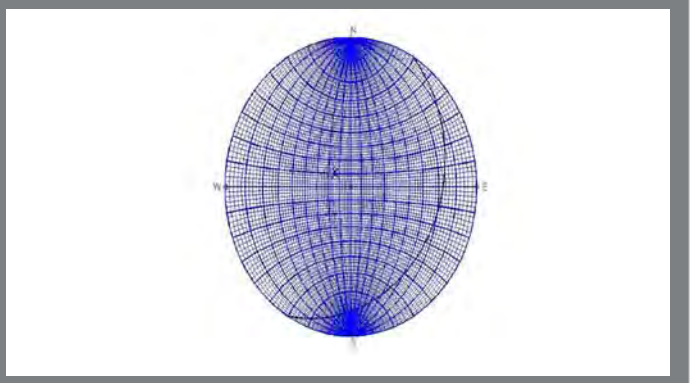


Figure 8: Stereographic projection of a plane; Strike $N30^oS$, dip $=20^o$ to $E30^oS$

2. Find the stereographic projection of a line plunging 20° below horizontal from N 30° W ?

bearing of line (measured from north)=360° − 30° =330°

>> [p1] = LH Line(330 , 20);

3. Find the angle between the line 1 and line 2, where line 1 plunges at 20° below horizontal from N 30° W and line 2 plunges at 35° below horizontal from N 40° E. Also find the strike and dip of the plane containing the two lines?

Bearing of line-1=360° − 30°=330°

Bearing of line-2=40°

Run the following commands at matlab prompt

>> [p1] = LH Line(330, 20); [p2] = LH Line(40, 35); Y = anglebetline(p1, p2) Y=Angle between lines=67.6487^o

Normal to the plane containing line 1 and 2 can be obtained by taking the cross product of p1 and p2. For lower hemisphere projection take either cross(p1,p2) or cross(p2,p1) which gives negative value in Z coordinate. More conveniently matlab function [Pn]=normalv(p1,p2) can be used to find out the normal to the plane containing line two lines p1 and p2. It should be noted that both the lines contains the center of sphere and so the vector representing normal to the plane containing these lines also passes through the center.

>> [Pn] = normalv(p1, p2)

$$[P_n] = [-0.2839\vec{i} \quad -0.5062\vec{j} \quad -0.8144\vec{k}]$$

Where $\vec{i}, \vec{j}, \vec{k}$ are unit vectors in X, Y, Z directions. Vector P_z perpendicular to horizontal plane in lower hemisphere will be $P_z = |-\vec{k}|$, so $P_z = [0 \quad 0 \quad -1]$,
$P_n = [-0.2839 \quad -0.5062 \quad -0.8144]$
Dip of the plane containing the two line is the angle between the horizontal plane and the plane under consideration which can be find out by finding the angle between their normal.
$>> dip = anglebetline(P_z, P_n)$

$$dip = 35.4748^o$$

Cross product of P_z and P_n will give strike vector (V_s), i.e the intersection of two planes which is a line and the angle between this line and Y axis is the strike of the plane .
$\gg V_s = cross(P_z, P_n)$, on normalizing V_s, $V_s = \frac{V_s}{|V_s|}$,
$V_s = [-0.8722\vec{i} \quad 0.4892\vec{j} \quad 0\vec{k}]$
Slope of V_s(measured anticlockwise from X axis) = $slopeq([0 \quad 0], [V_s(1) \quad V_s(2)]) = 150.7143^o$
Slope of V_s(measured anticlockwise from Y axis) = $150.7143^o - 90^o = 60.7143^o$
Slope of V_s(measured clockwise from Y axis) = $360^o - 60.7143^o = 299.3^o$
$Strike = 299.3^o$ i.e. $N\ 60.7^o W$
A matlab function
[**strike**, **dip**]
= **planeconlines**(**bearing1**, **plunge1**, **bearing2**, **plunge2**)
can be used which has in built functions to perform above explained tasks. Given line-1 have bearing1, plunge1 and line-2 have bearing2, plunge2. Output arguments of the function are strike and dip of the plane comprising these lines.

$\gg [strike, dip] = planeconlines(330,20,40,35)$

$\gg strike = 299.2867^o$

$\gg dip = 35.4748^o$

***Strereograp□ic projection of t□e plane and lines are dipicted in* Fig**

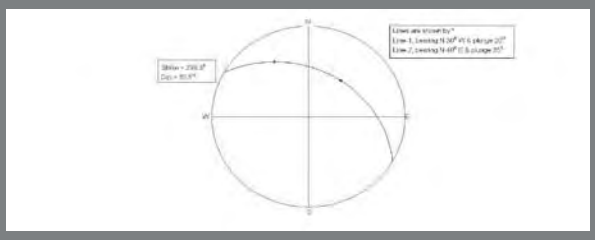


Figure 11: Stereographic projection of a plane formed by two different lines

4. Show the stereographic projection of plane-1 having strike E 30$^{\mathbf{o}}$ S and dip 20$^{\mathbf{o}}$ and plane-2 having strike W 20$^{\mathbf{o}}$ N and dip 30$^{\mathbf{o}}$. Find the bearing and plunge of the line formed by the intersection of these planes?

For plane-1, strike = E 30° S = 120° and dip = 20°

For plane-2, strike = W 20° N = 290° and dip = 30°

dip direction = strike +90° and the planes are dipping from their dip direc- tions by their respective dip angle. With these data using following function,

$\gg spp(120^o, 20^o, 0, 'k'); hold\ on; spp(290,30,0,'r')$
$\gg [P_s, bearing, plunge] = sisp(120^o, 20^o, 290^o, 30^o)$

Bearing and plunge of line which is formed by the intersection of two planes are as follows,
$bearing = 293.86^o\ and\ plunge = 2.2^o$. Now,
$\gg hold\ on; LHLine(bearing, plunge);$
Will plot the line of intersection of two plans as shown in Figure 10.

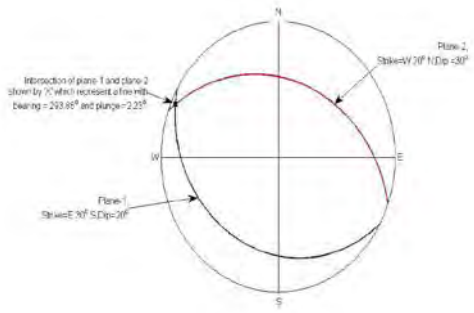


Figure 9: Stereographic Projection of Intersection of Two Planes

5. Show the stereographic projection of plane-1 having strike N 30$^{\mathbf{o}}$ E and dip 20$^{\mathbf{o}}$ and plane-2 having strike N 20$^{\mathbf{o}}$ W and dip 30$^{\mathbf{o}}$. Find the bearing and plunge of the line formed by the intersection of these planes?

For plane-1, strike = N 30° E = 30° and dip = 20° For plane-2, strike = N 20° W = 340° and dip = 30° Run the following commands on matlab prompt

$\gg spp(30^o, 20^o, 0, 'k'); hold\ on; spp(340,30,0,'r')$
$\gg [P_s, bearing, plunge] = sisp(30^o, 20^o, 340^o, 30^o)$

Bearing and plunge of line which is formed by the intersection of two planes are as follows,
$bearing = 120.92^o\ and\ plunge = 19.99^o$. Now,
$\gg hold\ on; LHLine(bearing, plunge);$
Will plot the line of intersection of two planes as shown in Figure 11.

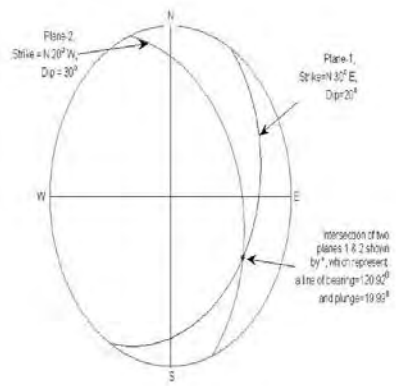


Figure 10: Stereographic Projection of Intersection of Two Planes

6. Plot the stereographic projection of a plane with strike $N\,30^o\,W$ and dip 20° and the angle $\alpha = 60^O$ which its normal passing through the center of sphere makes with the periphery of sphere cut by the plane ?

$$strike = N\,30^o\,W = 330^o$$
$$dip = 20^o$$
$$\alpha = 20^o$$

Run the following functions at Matlab prompt,
$\gg halfcircle(330,20,60)$,
Figure 12 depicts the three dimensional view and Figure 13 shows the stereographic projection of a plane formed by rotation its normal to a constant angle.

7. Find out the strike and dip of a plane, given that the normal to this plane through the center of sphere is defined by a line which plunges by 50° to N 45° W ?

A Matlab function $[strike, dip] = planentline(bearing, plunge)$ can be used to find out dip and strike.
$\gg [strike, dip] = planentline(315,50)$,
$strike = 45^o and\ dip = 40^o$, function $spp(strike, dip, mode)$ can be used to plot the plane as shown in Figure 14.
$\gg hold\ on; spp(45,40,1)$,

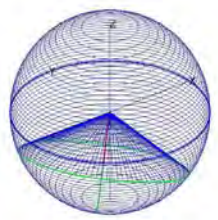


Figure 13: A plane which is not a grate circle and is formed by rotating its normal at a constant angle

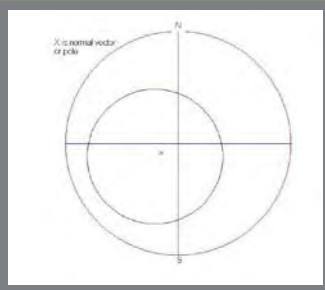


Figure 12: Stereographic projection of a plane which is not a great circle

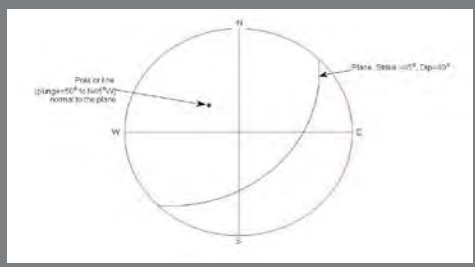


Figure 14: Stereographic projection of a plane

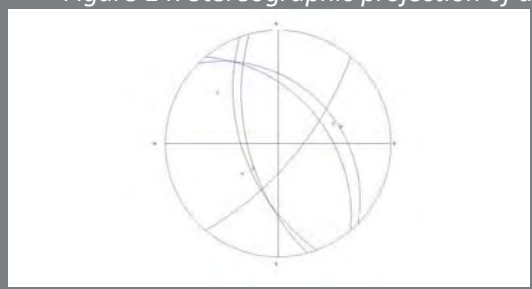


Figure 15: Example showing stereographic projection of planes and its Poles which are shown by small circles

8. Following table containing geological data from Chenab Bridge slope B2/2-1. Plot its stereographic projection and show their poles?

Joint Set	Strike	Dip	Dip Direction
J1	$N40^oW$- $S40^oE$	45°	$N\ 50^o$
J1	$N45^oW$- $S45^oE$	35°	$N\ 45^o$
J2	$N20^oW$- $S20^oE$	55°	$N\ 250^o$
J2	$N15^oE$- $S15^oW$	60°	$N\ 255^o$
J3	$N40^oE$- $S40^oW$	70°	$N\ 130^o$

Based on the above date, input parameters to the MATLAB function $spp(strike, dip, pole)$ as explained in paragraph 1 would as follows,

Joint Set	Dip Direction	Dip	Strike $(Dip\ Direction - 90^o)$
J1	$N\ 50^o$	45°	-40°
J1	$N\ 45^o$	35°	-45°
J2	$N\ 250^o$	55°	160°
J2	$N\ 255^o$	60°	165°
J3	$N\ 130^o$	70°	40°

$\gg spp(-40,45,1); hold\ on,$
$\gg spp(-45,35,1); hold\ on,$
$\gg spp(160,55,1); hold\ on,$
$\gg spp(165,60,1); hold\ on,$
$\gg spp(40,70,1); hold\ on,$
Refer Figure 15 for stereographic projection of these planes and their poles are plotted.

Thanks
Your comments, suggestions and the request for the source code of the MTLAB functions may be addressed to ambastha.see@gmail.com or xencusbrl@gmail.com.

Stations Along USBRL Project- Reasi

***Introduction*:-**

Reasi is a town and a notified area committee and tehsil in the Indian state of Jammu and Kashmir. Situated at the bank of River Chenab, it is the headquarters of the Reasi district. In the eighth century, Reasi was a part of the Bhimgarh state established by Bhim Dev. The name Reasi is derived from the town's old name "Rasyal".

Reasi Skyline in summer

***Geography*:-**

Reasi is located at 33.08°N 74.83°E.[1] It has an average elevation of 466 metres (1,529 feet).

***Reasi area*:-**

Reasi is a District, located 64 km from Jammu. The population of 10,000 is predominantly Hindu. Majority of the population ekes out its livelihood from small business ventures, govt jobs and agriculture. Of the 12293 hectares of agricultural land in the area, 1011 hectares is irrigated. Important crops are Maize, Wheat, Paddy and Bajra. Vegetables are also grown. Climatically, most parts of the area falls in the sub tropical zone and the rest in the temperate zone. Summers are generally warm and winters are cold with snowfall on the higher reaches.

M.P.Singh
Dy.CE/Reasi/USBRL

***Brief history*:-**

The erstwhile Bhimgarh State now called Reasi was established by Bhim Dev in the eighth century. Brief account of the successive rulers is known from 1652, when Hari Dev was the king of Jammu. In 1810, during the rule of Diwan Singh, Jammu was under turmoil. Palace intrigues and mutinies shook the administration. It was at this time that Maharaja Ranjit Singh sent Gulab Singh to take control. Gulab Singh came down heavily on the rebels and established the rule of law. After defeating the rebels in the Reasi area he handed over the administration to his trusted commander, General Zorawar Singh, when he became the King of Jammu in 1822. In 2005, the first municipal election was held and Mr.Kuldeep Mengi elected as a first chairman of municipal corporation REASI.

During the devastating floods of September 2014, Saddal Village in the Reasi district was completely and totally wiped off the face of the earth by a landslide including all roads leading to the village, with none surviving.

***Demographics*:-**

As of 2011 India census,[2] Reasi had a population of 36,355. Males constitute 54% of the population and females 46%. Reasi has an average literacy rate of 75%, higher than the national average of 59.5%: male literacy is 78%, and female literacy is 70%. In Reasi, 13% of the population is under 6 years of age. Reasi has 177 villages with total area of 74932 square km and total population of 71501 individuals [1]. Main spoken languages are Dogri, Urdu, Gojri and Kashmiri.

***Historical places*:-**

Mata Vaishno devi , Bhumika Temple, Deva Mai, Nau Pindian, Baba Dhansar, Siar Baba, Bhimgarh Fort, Kalika Temple, Sula Park, Sihar Baba & Shivkhori are few of the attractions of Reasi besides its picturesque locale and surroundings. **Vaishno Devi,** also known as **Mata Rani** and **Vaishnavi**, is a manifestation of the Hindu Mother Goddess or Durga. The words "Maa" and "Mata" are commonly used in India for "mother", and thus are often used in connection with Vaishno Devi. **Vaishno Devi Mandir** (Hindi: वैष्णोदेवी मन्दिर) is a Hindu temple dedicated to the Hindu Goddess, located at the Trikuta Mountains within the Indian state of Jammu and Kashmir.

***Vashno Devi*:-**

Vaishno devi is one of 108 shakti peetha, the story of Vaishno Devi is found in Chanddi up-purana. When Vishnu Bhagwan cuts Sati Maa's body into 108 peethas then 52 body parts fell onto earth atmosphere and rest of the 56 fell on other planets. Out of which the "Blessing Hand" of Goddess Sati had fallen on "Planet Venus" or Shukra. Goddess Lakshmi is considered to be the ruling planet of Shukra hence also known as Goddess of Venus. But in the beginning of Treta Yug, there was a devil called "Mur", he got the boon from Brahma that he cannot be killed by any human being, god, trinity, or any one born on Earth. Eventually in a gods-demon war, Demon Mur ran behind his greatest enemy Vishnu, who went to the place where was the shakti peetha of Goddess Sati on the Planet venus. Goddess Lakshmi insisted goddess Parvati, the next incarnation of Adi Parashakti after Sati, for help, thereby goddess Sharda and goddess Parvati in the form Kalika, went to the region. Goddess Lakshmi incarnated in spiritual form, because physically, she had already been taken birth as Goddess Vedavati who later on would become Sita in next incarnation. So Mother Parvati herself gave her physical appearance, with a soul of "Lakshmi" and Goddess of knowledge, Saraswati. The girl born was named "Kumari". Saraswati, Kali and Lakshmi dropped three pindies (Sacred stones) as their true forms, where they will be present till the end of Kaliyuga. To make the devil not known about the birth of the miracle goddess, Lord Shiva shifted the mountain to Earth. When the devil entered the place who was running after Lord Vishnu saw a little girl. She killed him and thereby, Vishnu got another name called "Murari".

Since her soul was of Lakshmi so she wanted to have Vishnu as her consort. She incarnated in the house of Lord Ratnakar, this time Narada named her "Trikuta" also by the names kumari etc.The girl right from her childhood displayed a hunger for knowledge which was like a vortex and which no amount of teaching and learning could sufficiently satiate. Subsequently, Vaishnavi started looking into her inner self for knowledge, and soon learned the art of meditation and realized that meditation and penance only could bring her close to her greater objective. Vaishnavi thus relinquished all household comforts and went deep into the forest for Tapasaya (meditation).

When she heard that Lord Rama moved to forest for exile as such Trikuta also went to forests. Meanwhile, Lord Rama, during his fourteen years of exile happened to visit Vaishnavi who recognized him immediately as no ordinary being but the incarnation of Lord Vishnu, and immediately asked him to merge her into himself so that she could become one with the supreme creator.

However Lord Rama, knowing that it was not the appropriate time, dissuaded her by saying that he would visit her again after the end of his exile, and at that time if she succeeded in recognizing him, he would fulfill her wish. True to his words, Rama visited her again after being victorious in the battle, but this time he did so in the disguise of an old man. Unfortunately, Vaishnavi was unable to recognize him this time and was distraught. Upon this, Lord Rama consoled her that the appropriate time for her being one with the creator had not come, and that time would come eventually in 'Kaliyug' when He (Rama) would be in his incarnation of 'Kalki'. Rama also directed her to meditate, and set up an Ashram at the base of Trikuta hills, to elevate her level of spirituality so as to bless mankind and rid the poor and destitute of their sufferings. Only then would 'Vishnu' merge her into himself. Vaishnavi, immediately set off for the northern part and after immense hardships, reached the foot of the Trikuta Hills. After reaching there she set up her ashram there and began to meditate.

As predicted by Lord Rama, her glory spread far and wide, and people began to flock to her Ashram to seek her blessings. As time passed, a Tantrik named Gorakh Nath who had a vision of the episode between Lord Rama and Vaishnavi in the retrospective time frame, became curious to find out whether Vaishnavi has been able to attain a high level of spirituality or not. He, therefore, sent his most able disciple 'Bhairon Nath' to find out the truth. Bhairon Nath on locating the ashram started observing Vaishnavi secretly, and realised that though a 'Sadhvi' she always carried bow and arrows with her, and was always surrounded by langoors (apes) and a ferocious looking lion. Bhairon Nath was enamored by Vaishnavi's extraordinary beauty, and losing all good sense he began to pester Vaishnavi to marry him. Meanwhile a staunch devotee of Vaishnavi, Mata Sridhar organised a Bhandara (community meal) in which the whole village and Guru Gorakh Nath along with all his followers including Bhairon were invited. During the course of Bhandara Bhairon Nath attempted to grab Vaishnavi but she tried her best to daunt him. On failing to do so, Vaishnavi decided to flee away into the mountains to continue her Tapasaya undisturbed. Bhairon Nath however chased her to her destination.

The goddess after halting at (present day) Banganga, Charan Paduka, and Adhkwari, finally reached the holy cave Shrine.when mata vaishnavi was fleeing she came in presence with a muni and seeked his help . He directed her towards a cave in Adhkawari . It is said that she stayed in the cave for 9 complete months. After that when she came out hanuman was there to guard her. She wanted to wash herself but there was no water in the vicinity. So she took an arrow and striked it towards the land when water came out from that place .It is the origin of Ban-Ganga. The name of the river came from this story. After this she moved on furthur to now situated Shrine and took rest and started to meditate. Bhairon after knowing her presence went there where he was stopped by hanuman. It was a fierce battle the goddess was compelled to kill him. Bhairon Nath met his ultimate fate when the goddess, just outside the mouth of the cave, beheaded him. The severed head of Bhairon fell with force on a distant hilltop. Bhairon Nath upon his death realised the futility of his mission and prayed to the deity to forgive him. The almighty Mata (Mother Goddess) had mercy on Bhairon and gave him a boon that every devotee of the goddess would have to have the Darshans of Bhairon after having the Darshans of the Goddess and only then would the yatra of a devotee be complete. Meanwhile, Vaishnavi decided to shed off her human form and assuming the face of a rock she immersed herself into meditation forever. Thus Vaishnavi, in the form of a five-and-a-half-foot-tall rock with three heads or the Pindies on the top is the ultimate destination of a devotee. These Pindies constitute the Sanctum Sanctorum of the holy cave known as the shrine of Shri Mata Vaishno Devi Ji, which is revered by one and all.

Stations Along USBRL Project- Reasi

Vaishnodevi pilgrims (yatris) visit the temple every year, and it is the second most visited religious shrine in India, after Tirumala Venkateswara Temple. The Sri Mata Vaishno Devi Shrine Board maintains the shrine. A rail link from Udhampur to Katra is recently completed to facilitate pilgrimage. The nearest airport is Jammu Airport which has a very high flight frequency, and is served by all leading domestic airlines. The temple contains three idols of Maha Saraswati, Maha Lakshmi, and Maha Kali, which are all images of Vaishno Devi.

Access:-

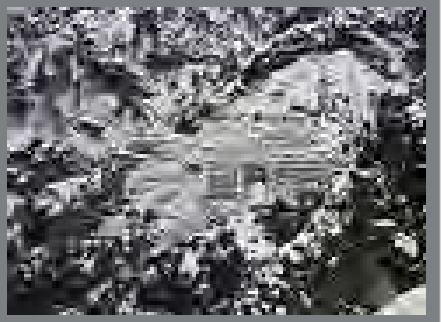
Maa Vaishno Devi temple can be reached from Katra. Katra is a small but bustling town around 45 km. from Jammu. From Katra, after getting the 'Yatra Parchi' (Journey Slip) at Banganga point for darshan, devotees can proceed to the Bhavan.

The way to Bhavan is steep and requires a long walk uphill from Katra. Alternatively ponies and palanquins are also available. Helicopter service can also be taken for a large part of the trip. The Trust offers comfortable stay for pilgrims. Katra is at an altitude of about 2500 feet (about 762 meters); Banganga is at an altitude of about 2800 feet (863 meters); Bhavan is at the altitude of about 6200 feet (about 1900 meters); and, the distance from Katra to Bhavan is about 13 km.

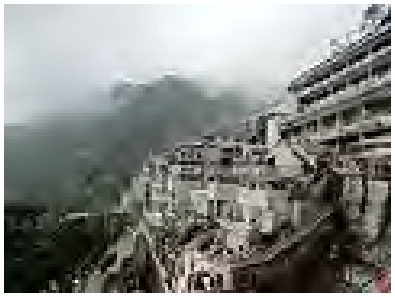
Indian Railways will start rail services up to Katra from a date prior to 9 July 2014 when Union Rail Budget of 2014-15 is to be presented in the Parliament (Tentative).[4] They are in communication with the Jammu and Kashmir Government to integrate permit slips for Vaishno Devi Darshan with the train ticket. The passenger can get permit slips while booking the train ticket.

Picture Gallery

Vaishno Devi Bhavan



Vaishno Devi Bhavan in Winters (Darbar)

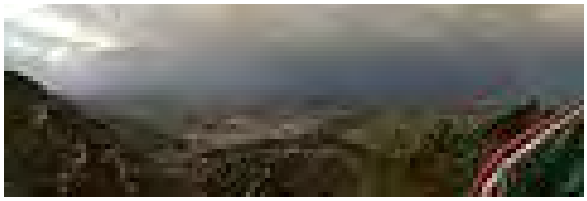


Closer View of Vaishno Devi Bhawan

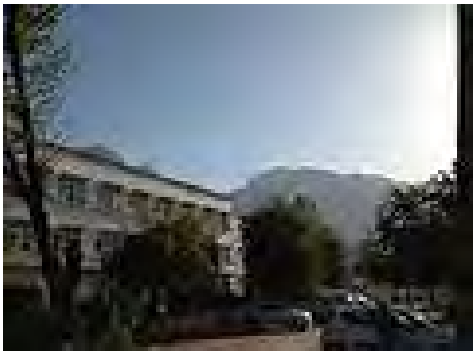
Dwaar where the Yatra begins (Baann Ganga)

Baann Ganga River

Bhairav Mandir



Panoramic view of Katra en route to temple



Niharika Guest House, Katra

***Siar Baba Waterfall*:-**

Sihar Baba Waterfall in Summer

Baba Dhansar

The holy place of **Baba Dhansar** is located at Karua Jheel (Pond) near village Karua, 17 km from Reasi towards Katra in Reasi district of Jammu & Kashmir State, India. The approach involves a walk of 200 metres from the road. It is a mythological that belief that when Lord Shiva went to the Amarnath cave to tell Parvati the story of his immortality, he left his serpent king, Sheshnag at Anantnag. Shesh Nag came in the human form as Vasudev. One of the sons of Vasudev was Dhansar who was a saintly person.

As the local belief goes, in the ancient times there was a demon who lived near Karua Jheel(lake) and committed atrocities on the people of village Karua. The villagers sought help of Baba Dhansar to get rid of the Demon. It is believed that Baba Dhansar prayed to Lord Shiva for help. Lord Shiva arrived and helped in killing the Demon. The temple of Baba Dhansar and a cave of Lord Shiva near Karua Jheel has become a place of worship. Karua Jheel is considered sacred where bathing is not permitted. However, the devotees may take a bath downstream. People believe that their wishes are fulfilled if they take bath in the stream and pray with complete faith. A large number of devotees visit the place every year on the day of Mahashivratri when an annual fete (mela) is organized.

Photo gallery

Nag temple at Baba Dhansar

Sacrificial stones at baba Dhansar

Bhimgarh Fort

Stations Along USBRL Project- Reasi

Bhimgarh Fort, generally known as the Reasi Fort, near Reasi, a town approximately 64 km north-west of Jammu. The Fort is located on a hillock approx 150 metres high. Initially the Fort was constructed of clay and later on one of the heirs of Maharaj Rishipal Rana, the founder of Reasi, reconstructed it using stone. It was used by the royal family members for taking shelter during emergencies.

The renovation of the Fort was started by Gulab Singh of Jammu and Kashmir in 1817 and continued till 1841. A new entry gate and a stone wall one metres wide and 50 metres long was built all around, thereby making it less vulnerable to attacks.

The main entry gate is made of Baluka stones with Rajasthani carving. The front wall with loopholes is approximately 50 metres long and one metre wide. This has a statue of the Goddess Mahakali and of God Hanuman.

The Fort has a temple, a pond, a number of rooms of different sizes, armoury and treasury. After the death of Maharaja Gulab Singh, his heir Maharaja Ranbir Singh and Maharaja Pratap Singh used Bhimgarh Fort as a treasury and armoury. It was during Maharaja Hari Singh's rule that an English Minister ordered that the armoury be destroyed and shifted the treasury to Jammu.

The Fort was handed over to the Jammu and Kashmir State Archaeology Department in 1989 on the orders of state government. In 1990, the fort was renovated by the Vaishno Devi Sthapna Board. The surrounding areas of the fort were given a facelift with the construction of gardens and pathways. The fort was then opened to the public. Though, the fort is devastated by the earthquakes and lack of maintenance, it still stands out as an important landmark in the town.

Maha Kali Mandir *:-*

Maha Kali Mandir is the most prominent temple in Reasi town situated in district Reasi of J&K state. It is located on a hilltop close to the Reasi Bus Stand in the heart of the main city. It is said that 300 years ago Kalika Mata came in the dream of Pandit Jagat Ram Sharma and indicated her presence in the form of a Pindi (stone) lying under ground on this hill. On excavation of the area the Pindi was found. Thereafter a small temple was constructed on the hill and Pandit Jagat Ram Sharma became the caretaker.

Over the years the temple has gained prominence due to a strong belief of the people in Kalika Mata. The local people have contributed to the development of this temple. The devotees visit the temple in large numbers during the days of Navratri.

Shivkhori *:-*

Shivkhori is a famous cave shrine of Hindus devoted to lord Shiva, situated in the Reasi district of Jammu and Kashmir state in India.

Location ***:-***

In Reasi district, there are many shrines such as Mata Vaishno Devi, Merhada Mata, Baba Dhansar, Siad Baba. Shiv Khori is one of them located in Ransoo a village in the Pouni block in Reasi district, which attracts lakhs of devotees annually. Shiv Khori is situated in between the hillocks about 140 km north of Jammu, 120 km from Udhampur and 80 km from Katra. Buses and light vehicles go up to Ransoo, the base camp of pilgrimage. People have to traverse about 3 km on foot on a track recently constructed by the Shiv Khori Shrine Board, Ransoo duly headed by the District Development Commissioner, Udhampur as chairman and Sub Divisional Magistrate, Reasi as Member Secretary.

Description ***:-***

Khori means cave (Guffa) and Shiv Khori thus denotes Shiva's cave. This natural cave is about 200 metres long, one metre wide and two to three metres high and contains a self made lingam, which according to the people is unending. The first entrance of the cave is so wide that 300 devotees can be accommodated at a time. Its cavern is spacious to accommodate large number of people. The inner chamber of the cave is smaller.

The lingum

The passage from outer to the inner chamber is low and small; at one spot it divides itself into two parts. One of these is believed to have led to Kashmir where Swami Amarnath cave is located. It is now closed as some sadhus who dared to go ahead never returned. To reach the sanctum sanctorum, one has to stoop low, crawl or adjust his body sideward. Inside a naturally created image of Lord Shiva, about 4 metres high, is visible. The cave abounds with a number of other natural objects having resemblance with Goddess Parvati, Ganesha and Nandigan. The cave roof is etched with snake formations, the water trickles through these on Shiva Lingam. Pigeons are also seen here like Swami Amar Nath cave which presents good omens for pilgrimages.

Stations Along USBRL Project- Reasi

***Legend*:-**

A number of legends have propounded about the discovery of this holy cave. One of the most important legends among them is that a demon named Bhasmasur after a long meditation of Lord Shiva obtained blessing to end the life of any one with that blessing. After obtaining it, the said devil tried to end the Lord Shiva-On seeing the evil design of the demon, the Lord Shiva run to save himself from the power of the demon and entered in this cave which is presently known as Shiv Khori. After this, Lord Vishnu in the guise of Mohini came forward and asked the demon to dance with her according to her tune. As and when the demon started dancing as per the actions of Mohini, the said demon took his hand at his head and with his own power, he was himself destroyed. As per the legend, 330 million deities exist in this cave in shape of pindis and natural milky water is falling on them from the top of the cave. In this cave there is also caves which directly go to amarnathji according to a saint who lived there named as bababa ramesgirigi

As per the other legend regarding discovery of this cave is that the historic Shiv Khori cave is believed to be discovered by a Muslim shepherd. He was in fact in search of his missing goat and went by chance inside the cave to find the same. However he was very much startled to see a number of saints inside the cave, who were impressed by Lord Shiva's divine power and he too started pooja there. Later on the shepherd disclosed this to a number of other people in spite of his promise made with the saints not to disclose about them or this cave. It is said that the shepherd after narrating it to other people had died. According to the legends it is believed that a number of famous saints have been closely associated with this cave, who had spent decades inside this cave for spiritual attainment and meditation.

***Other details*:-**

About 40 to 50-year ago, only a few people knew about the Shiv Khori shrine but it has gained much popularity during decades. In earlier times the number of yatries was just in thousands but after the constitution of Shiv Khori Shrine Board during December 2003, the number of devotees has superseded previous records as the number of devotees in year 2005 crossed 300,000. This year,it is expected to exceed 500,000 tourists. Some 30 percent of devotees reach the shrine from within the state and 70 percent from different states of the country.

A 3-day Shiv Khori mela takes places annually on Maha Shivratri and thousands of pilgrims from different parts of the state and outside visit this cave shrine to seek blessings of Lord Shiva. Maha Shivratri festival is usually held in the month of February or during first week of March every year. Keeping in view the increasing rush of pilgrims to the holy cave shrine, the Shiv Khori Shrine Board has taken up a number of steps to develop this spot in a bid to provide more and more facilities to the devotees, like construction of Shrine Guest House at a cost of Rs. 1.9 million at village Ransoo, the base camp of yatra, Reception Centre and Pony shed at an estimated cost of Rs. 8 million, tile work of entire 3-km long track is nearing completion, plantation of ornamental and medicinal plants on track and development of parks etc. Other arrangements like electrification of the cave with modern techniques, provision of oxygen and electric generators, exhaust fans, construction of shelter sheds for yatris with toilet facilities near the cave site, 15 shelter sheds en route Ransoo to cave shrine, railing from the base camp to cave, additional facility of 15,000/EfnrKing water reservoir, proper sanitation, provision of 25 KV capacity electric transformer, clock room, starting of permanent bus services from Katra, Udhampur and Jammu, Police post and Dispensary and a STD PCO are under active consideration of the Shiv Khori Shrine Development Board.

To meet the ever growing rush of devotees in having smooth darshans of the Lord Shiva, an exit tunnel has been constructed by the shri Shivkhori Shrine Board this year in February.

Recently, the management and development of the Shiv Khori has been taken over by Sri Mata Vaishno Devi Shrine Board who is looking after VaishnoDevi pilgrimage.

***Transport*:-**

Reasi is 64 km from Jammu and can be reached by Road, Rail or Air. Nearest Airport is 80 km and railway station 26 km. In 2010 rail link is expected to reach Reasi.

***Geology and mining*:-**

Reasi has rich ores of bauxite, iron and precious stones.

***Current times*:-**

Being far away from the Jammu - Udhampur highway and somewhat inaccessible due to the hilly area, economic progress in Reasi has been rather slow. However, the economic activity picked up since the 1980s with the construction of the Salal Hydroelectric Project. The militancy in the 1990s came as a setback to prosperity but induction of Army in the area has given a sense of security to the people. But what may change the face of Reasi in the not so distant a future, is the Jammu - Srinagar Railway Line which will pass through Reasi and is likely to bring development and prosperity to this area, after providing district status development of Reasi would be tremendous. Almost all major banks have their presence in reasi to meet financial need of newly created district.

***World's First in Reasi*:-**

Reasi is looking forward to have the World's tallest bridge by December 2009. Chenab Bridge (359 meters) being built by Konkan Railways will overtake The Millau Viaduct (323 meters) in southern France.

Amarnath Yatra

Introduction:-

The Amarnath is considered as one of the most sacred of Hindu Temples, dedicated to Lord Shiva. It is not listed in "twelve jyotirlingams*" of India but even then large number of devotees undertakes yatra* every year in tough terrains and harsh weather of Kashmir. It speaks manifolds about their immense devotion and staunch faith in pilgrimage of Amarnath.

Facts about Holy Cave and Yatra:-

Holy Cave of Amarnath:-

Amarnath Holy Cave is situated in a narrow gorge at the farther end of the Lidder Valley at about 145 km from Srinagar, the capital city of Jammu & Kashmir. The holy cave is perched at altitude of 3888 m high above MSL where as other famous shrines of northern region like Mata Vaishno Devi is at 1585 m and Kedarnath at 3657 m.

THE HOLY CAVE

The cave is 200 ft long, 55 feet wide and 50 feet deep and enshrines a unique ice Shivalingam* (an Ice Stalagmite) created naturally by water dripping through the limestone roof of the cave.

This ice Shivalingam is believed to wax and wane with the Moon's cycle. There are also two smaller ice-lingams* formed in the cave which represent Lord Shiva's consort, Goddess Maa* Parvati and his son, Lord Ganesha**.**

THE ICE SHIVALINGAM

Bhupinder Salwan
XEN/SINA

Timing of Yatra:-

As per Hindu belief Yatra should take place between Jyestha* Purnima* and Shraavan* Purnima*. This day of Shraavan Purnima i.e. Full moon day of Shraavan month is considered as most auspicious day for visiting the shrine and pay homage to Lord Shiva. However yatra generally starts in first week of July and extends up to middle of September.

It seems that in ancient times the Yatra might not be confined to these two months because Pt. Jawahar Lal Nehru wrote in his autobiography that he went in April to have the darshan* whereas noted author Yashpal Jain wrote that he went in the month of September.

History of Amarnath Yatra:-

i) **Start of First Yatra**: Exact period is not known when Amarnath Yatra took its start. According to **Puran,** a spiritual and ancient treatise of Hinduism, **Bhrigu** Rishi* had once visited Amarnath Cave. **Kalhana,** an ancient Sanskrit poet has written in his book 'Rajtarangini' in 11th century that Amarnath Yatra was in use for one thousand years before Christ. But yatra remained closed time to time for several hundred and thousand years due to disturbance caused by outside invaders in Kashmir in medieval age and various natural calamities. Since then Holy cave and yatra route might have remained obscured for many years.

ii) **Re-discovery of Yatra** : According to traditional story once a Muslim shepherd namely Buta Mullick was given a sack of coal by a Hindu Anchorite. After opening the sack he found that it was full of gold. Overjoyed as he was, he rushed to look for that very sadhu* to express gratitude to him. He followed him to far distance place where that sadhu disappeared. He found himself reached in a cave in which there was a very beautiful "Ice-Lingam". Buta Mullick came and described the whole incident to Hindu sages. He also showed the route to them. This route had been forgotten by Hindu devotees due to many bad occurrences for several hundred years. Since then Yatra again started.

In 1858 A.D. Dogra King Gulab Singh was so much pleased with this rediscovery that he issued a decree that representative of Mullick family would always be present at the holy cave shrine along with a mahant* and pandit of Ganeshpora* during the yatra every year and that one third of the offerings received at Holy cave would go to Mullick's family. Buta's family was also granted a large estate near Pahalgam and exempted from paying land revenue to the state. Presently Subhang Mullick of Buta Mallik's family acts as main priest of the Amarnaath Shrine. He does evening Aarti* of Lord Shiva everyday during yatra period.

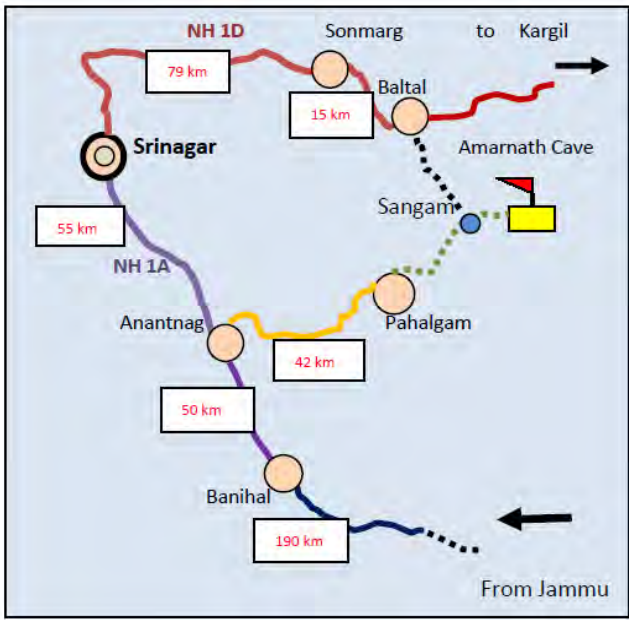
iii) **Tradition of Chhadi Mubarak*:** A Hindu Priest of Hoshangabad in M.P. used to go to Amarnath yatra with "Chhadi Mubarak" in 16th century. But it was very tiring and time taking. It took several months of one trip. Fifth Sikh Guru Arjundev gave a piece of land at Amritsar and provided facilities to the priest to start his yatra from there. From that period yatra was continued from Amritsar.

In 19th century, Dogra King Gulab singh donated land for hermitage known as Dashnami Akhada at Srinagar for starting yatra of "Chhadi Mubarak". Since then it is being continued till date.

Now a days "Chhadi Mubarak" yatra starts from Dashnami Akhara of Srinagar under leadership of Mahant Krishnanand Saraswati. Several hundred sages and devotees accompany the procession.

This yatra is 143 km long and procession takes halt at Pampur, Bijbehara, Anantnag and proceeds through Matan, Aishmukam and Pahalgam. The procession takes rest at Pahalgam for two days and then proceeds for 'The Holy cave'. It reaches the 'Holy cave' on festival of "Raksha-Bandhan"(full moon night of Sraavan month of Vikrami calendar* which falls in July-August). After ritual worship at 'Holy cave' the "Chhadi Mubarak" returns

Physical Tour of Amarnath Yatra:-



Let us start our journey from Jammu:

There are **three ways** to reach the holy cave.

i) The traditional route is through Pahalgam. It is about 42 km east of Ananatnag, a small town situated on Jammu-Srinagar NH-1A. Thus total distance between Jammu to Pahalgam comes to about 282 km and one can reach Pahalgam from Jammu by taxi or bus in about 7-8 hours.

ii) The other route is through Baltal. It is about 79 km from Srinagar and is at about 3 hours run.

iii) By Helicopter from Baltal or Pahalgam

1. Pahalgam-Amarnath Holi Cave route:

This is traditional route although the original pilgrimage subscribes that the yatra should be carried from Srinagar. The trek from Pahalgam to Amarnath cave is on an ancient peregrine route. The 45-km distance is covered in 3-4 days, with night halts at Chandanwari, Sheshnag (Wawjan) and Panchtarni.

The step by step description of various halts is as follows:

Pahalgam: It is small town situated on the banks of Lidder River. It is a famous hill station surrounded by high mountains. It generally becomes the base station for the yatra on this route.

Chandanwari: It is the first halt at a distance of 16 km from Pahalgam. The distance can be covered by foot as well by road transport. The journey runs along the Lidder river. Pilgrims can camp here on the first night out. A major attraction here is a bridge covered, year round, with ice even though the surroundings are free from it.

Pissu Top: From Chandanwari, there begins a steep ascent to Pissu Top and one has to travel on foot or ride over a horse or taken in a chair by four people up to Pissu Top and further. About 600 m height has to be traversed in trekking 3000 m of distance from Chandanwari to Pissu Top. This halt is also called Pishu Ghati. It is said that in competition to be first to reach for darshan of Lord Shiv there was a war

PISSU TOP

Sheshnag: At distance of 9 km from Pissu Top comes Sheshnag, a mountain which derives its name from its seven peaks, resembling the heads of a mythical snake.

Seshnag symbolizes the cosmic ocean in which Lord Vishnu, the preserver of this universe, moves, reclining on a seven-headed mythical snake. This stretch has more or less smooth gradient. The deep blue waters of Sheshnag lake and glaciers beyond it cast a spiritual spell on the visitors.

SHESHNAG

Panchtarni: From Sheshnag one has to climb a steep height across Mahagunas Pass at 4276 m (14000 ft) for 4.6 Km and then descend to the meadow lands of Panchtarni at a height of 3657 mtrs (12000 ft).Mahagunas Pass is the highest point of the route. Mahagunas Top is lifeless, except for the pilgrims and the Indian army. After descending from Mahagunas comes the meadow-lands of Mahagunas is full of rivulets; waterfalls and springs. The greens of the previous journey turn here into brown and barren mountains. **Panchtarni** is the last camp enroute to the holy cave. There are cold and harsh winds at Panchtarni. Some Yatris can also be affected by deficiency of oxygen. Some may get the feeling of nausea.

MAHAGUNA PASS

PANCHTARNI

At Panchtarni, at the feet of Bhairav Mount, Five Rivers flow which apparently are said to be originated from Lord Shiva's Hair.

Holy Amarnath Cave: On the way to the Holy Cave, which is 6 km from Panchtarni, one comes across the Sangam of Amravati and Panchtarni rivers. Sangam is at 3 km from Panchtarni and this is the point where the other route coming from Baltal meets. Some pilgrims take bath at Amravati near the Holy Cave before going for worship. Near the cave is found white soil known as Bhasam. It is the most beloved soil of Lord Shiva. The pilgrims apply this Holy soil to their body and then go for Shivlingam Darshan.

Entrance to the cave is regulated, and darshan a hasty affair because there are many others waiting outside in queues to pay homage before the revered Shivalingam. Even then not more than 10000 yatris, and maximum, as an exception 20,000 yatris only can be allowed to go for darshan per day. The devotees sing bhajans*, chant incantations, and priests perform aarti and puja*, invoking the blessings of Shiva, the divine, the pure, the absolute.

2. Baltal-Amarnath Holi Cave Route:

This is second route which starts from Baltal and goes through small halts at **Domail, Barari Marg and Sangam.** This is a 14 km. steep trek to be covered on foot. It is possible to hire ponies. Dandies*/palkies* are also available for handicapped and old aged pilgrims. Baltal-Holy Cave route is more popular because of its shorter distance. However, the path, as can also be seen in above photograph, is katcha*, pebbled and narrow as compared to the Chandanwari-Holy Cave route. There are some steep rises & falls on this way also but main advantage is that pilgrims can return back to base camp Baltal through this route only in one day. Baltal is also known as Neelgrath.

BALTAL-HOLY CAVE PATH

3. By Helicopter from Baltal or Pahalgam:

The third way is to reach Panchtarni from Baltal or Pehalgam by Helicopter Service and then reach further to Amarnath Cave on foot or by hiring ponies or palkies. Pawan Hans Limited and Global Vectra Helicorp Ltd have been engaged for providing Helicopter Services on the Baltal-Panchtarni-Baltal Sector and Himalayan Heli Services Pvt Ltd for Helicopter Services on the Pahalgam-Panchtarni-Pahalgam Sector.

In above photograph helicopters are seen stabled on helipad of Baltal whereas tented area for temporary halts are seen in background and bus/taxi stand seen on left side of it.

BALTAL BASE CAMP

Mythological Background

The shrine is dedicated to The Lord Shiva. As per old Hindu scriptures and vedic literature the God (Parmeshwar) has three deities who carry on the world. This is known as Holy Trinity. Brahma- the creator, Vishnu - the perpetuator of life and Shiva (Mahesh) -the purifier and perpetuator of good anddestroyer of evil. Out of three, Lord Shiva is considered as living God. He has three places of residence. One is Kailash Parvat another is Lohit Giri under which the river Brahamputra flows and third is Muzwan Parvat. This Holy Cave is lying in this Muzwan Parwat.

There are basically three legends about the Holy Amarnath Cave yatra which are described here in order of their popularity and importance:

Legend of Amarkatha:

Centuries ago Maa Parvati, the divine consort of Lord Shiv requested Him to tell her about creation of universe and secret of death and rebirth. Maa Parvatiasked her Lord why her body was destroyed every time and she had to die again and again while He remained Immortal. Bhole Shankar (Lord Shiv) replied that its secret is Amar Katha.

On consistent demand from Maa Parvati He made up his mind to tell the immortal secret. He looked for lonely place where no living being could listen to it. He saw this cave with His divine eyes and moved towards it with Maa Parvati. In preparation to that when He reached a very beautiful and lonely place on the bank of river Lidder (also called Nilganga) where He asked Nandi (Bull of Lord Shiva on which He rides)) to remain there and do not allow any creature to go ahead. So that place was called 'Bailgam' and later on it converted into Pahalgam. After a few miles ahead He washed all his Vibhuti* and Chandan* of His body so that place is called 'Chandanwari'.

LORD SHIVA

A few miles away Shivji separated snake from His neck and kept him to remain on the bank of a glacial lake called Sheshnag lake. Again a few miles ahead He told His son 'Ganesha' to be seated on a mountain top. This place later was known as Mahagunush Top. When He reached at a flat ground He opend his dreadlocks (Jata) and shook vigorously. A few drops of water fell down on earth and became five streams. That's why this place is called "Panchtarani". Here He also left the Five Elements behind (Earth, Water, Air, Fire and Sky) which make living being.

Then Bhole Shankar entered the Holy Amarnath Cave along with Maa Parvati. Lord Shiva takes His Samadhi* on the Deer Skin and concentrate. To ensure that no living being is able to hear the Immortal Tale, He created Rudra* named Kalagni and ordered him to spread fire to eliminate every living thing in and around the Holy Cave. After this He started narrating the secret of immortality to Maa Parvati. But as a matter of chance a pair of pigeon's egg in a hole remained protected as it was non-living at that time. The eggs burst and squabs in form of nascent born pigeons came out of eggs when Lord Shiva started to deliver "Amarkatha". In the mid of katha Maa Parvati went into sound sleep but squabs heard Amarkatha in full and became immortal. This pair still remains in the hole of cave.

IMMORTAL PIEGONS

The pair of pigeons was often seen by pilgrims when they trek the arduous route to pay their obeisance before the Ice-Lingam.

Legend of Kashyap Rishi:

The valley of Kashmir was under water. It was a big lake. **Kashyap** Rishi drained the water through number of rivers and rivulets. In those days **Bhrigu** Rishi came that way on a visit to The Himalayas. He was the first to have seen this Holy Cave and have darshan of Shivlingam. When people heard of the Lingam, Amarnath for them became Shiva's abode and a centre of pilgrimage.

Legend of Chhadi Mubarak:

In primitive age Kashmir was a big lake which was ruled by King Nagraj. He invited some human being after taking permission of his Guru **Kashyap** Rishi. Some demons also came along with human beings and began to live. Later on demons began to vex men as well as King Nagraj. So Nagraj went to his Guru and told his problem. Rishi prayed to Lord Shiva Who gave him a "silver stick" and directed them to take it to that holy cave where HE will appear and bless HIS devotees. "Silver Stick" i.e., 'Chhadi' is symbol of safety and authority. Since then yatra is done by a Hindu priest with this "Chhadi Mubarak". A large number of hermits, anchorites and devotees accompany this priest to holy cave on full moon day of Sraavan (in August) every year and perform puja, archana* and religious rituals.

CHHADI MUBARAK

Useful Travelling Information:

Management:

The Holy Cave Shrine of Shri Amarnathji is managed by **S**hri **A**marnathji **S**hrine **B**oard (SASB), which was constituted by an Act of the Jammu & Kashmir State Legislature in 2000. The Governor of Jammu and Kashmir is its ex-officio Chairman

Registration for Yatra:

Registration of all the pilgrims is necessary and can be done from any of the J&K Tourism Offices located in different parts of the country. The provision of on-line facility has been discontinued because of the gross misuse of this facility in the past year. Registration of the Yatris is done on a first-come-first-serve basis. Stipulated time is mentioned in the registration during which the yatris have to commence the pilgrimage. As per the Standard Operating Procedure (SOP) framed on the directions of Supreme Court of India, only 15000 registered pilgrims are allowed beyond Pehlgam and Baltal in a day. This avoids mayhem at Darshan place in Amarnath Cave.

Registration counting also becomes helpful to regulate and control number of pilgrims beyond the capacity for which arrangements for food, shelter, medical facilities etc. are made.

Method for Registration:

i) Download the form online or get it from branch of designated bank.

ii) Complete the requirements as asked for in the form; one main requirement is 'Compulsory Health Certificate' issued within a specific period prescribed by the Board from the authorized Doctors or Medical Institutes. List of authorized Doctors and Medical Institutes is available online.

iii) Submit duly filled-in Application Form, Compulsory Health Certificate and a prescribed payment to cover the handling charges to Shrine Board.

iv) Yatra Permits are issued to eligible pilgrims and process is completed.

Amarnath Yatra

No one below the age of 13 years or above the age of 75 years and no pregnant lady with more than six weeks pregnancy are registered for the Yatra.

Modes for travel:

The State Transport Corporation provides regular bus service to bring pilgrims to the base camps of Baltal or Pahalgam. Private transport and tour operators also provide private vehicles and taxis to bring pilgrims from Jammu or Srinagar to these base camps. Barring Pahalgam –Chandanwari stretch the rest of the Yatra has to be done on foot. Ponies and dandies are available but they have to be hired at a cost decided by the government.

Helicopter facility from Pahalgam and Baltal to Panchtarni and back is available. Present Helicopter fare for one side per passenger from Baltal-Panchtarni and Pahalgam-Panchtarni sector is around Rs.2000/-and Rs.4,200/- respectively. The helicopter service is operated by private operators and regulated by Government Agencies. From Panchtarni, it is only a 6 km trek to the Holy Cave. Pilgrims availing Helicopter Service are not required to seek advance Registration as their Helicopter Tickets are treated as Yatra permits for undertaking the pilgrimage. However, a valid 'Compulsory Health Certificate' is still required with ticket.

Boarding and Lodging:

The arrangements for food and rest on both the routes are made by the government of Jammu & Kashmir and private agencies as well. There are many non-profit non government agencies that provide many facilities including food and lodging free of charge to the pilgrims.

FREE LANGAR

J&K Tourism Development Corporation (JKTDC) makes arrangements to provide tented accommodation and shelter with bedding at nominal charges. There are other private charitable and non-profit organizations that also provide different kinds of facilities for overnight rest at camps at Panchtarni. Many charitable organizations provide food free of cost at the base camp and Panchtarni. Temporary toilets and facilities for bathing are also made available for the yatris.

Health & Security:

Although average low temperature in July and August is 16° to 18°C at Amarnath but weather here is unpredictable. It can change to a chilly rainy day in minutes with temperatures dipping sharply to subzero. So it is advisable to carry rain coats, woollens, umbrellas, walking sticks, caps, monkey caps, gloves, warm socks, torch with extra batteries, personal medical kits and some eatables etc. all wrapped in a waterproof bag or a rucksack. In view of the hazardous nature of the yatra, it is advised to get medical insurance, preferably before departure from the home stations. However, arrangements for insurance are available at the two base-camps during the yatra period. Moreover the Shrine Board provides every Registered Yatri with an insurance cover of Rs.One lakh against accidental death within the state during the period of one week ahead of the commencement of the Yatra to one week after the conclusion of the Yatra.

Doctors, Para medical staff, equipment and medical supplies are available at medical posts on both the routes. Various Government agencies provide number of medical facilities on all routes at Chandanwadi, Sheshnaag, Sangam, Panchtarni and near the Cave Shrine. These facilities are free for all pilgrims. Trained doctors are available round the clock to mitigate any medical emergency.

A very large contingent of Central Reserve Police and State Police Force is put on duty to ensure safety and security to the pilgrims for the period of the yatra,. Security Forces posts their men on the route as well as in the vicinity of the shrine.

MEDICAL CAMP AT BALTAL

Communication:

Requirement of uninterrupted connectivity of human beings with their dear ones has become order of the day. No prepaid SIM of any other state works here in J&K. So it is advisable to arrange for post paid SIM before leaving home.

Moreover all registered pilgrims can get Temporary Pre-paid SIM cards, which shall be operative for a period of seven days. This Pre-paid SIM card facility is extremely valuable for those Yatris who do not possess a Post Paid mobile facility. Pre-paid SIM cards to the registered yatris are given at Special Counters which are set up at base camps. The needy pilgrim should provide proof of Identity, proof of permanent address and valid registration papers at the special counters for SIM card. Helpline numbers are also available online.

Some Safety Advices:

i) Keep your Yatra permit and an Identity Proof always with you. Also keep all information about you in a small pocket book such as names and mobile numbers and addresses of members of your group with you.

ii) Ladies should wear shalwar-kameez* or pants or a track suit rather than a sari.

iii) Carry water bottle with you to avoid dehydration.

iv) Try to trek in a group keeping other members in sight.

v) Do not use slippers or other unsuitable footwear as you might slip on slippery rise and falls along the route.

vi) Do not attempt to take short cuts as they may prove dangerous.

vii) Make sure that the pony-walla*, the labourer, the dandy-walla or palki-walla is properly registered and carries a token.

viii) Take a good sturdy walking stick while trekking.

ix) Do not relax or wait at points where there are warning notices.

Conclusion

It is a deep aspiration of every Hindu to have Amarnath Yatra in his life. If, by grace of Almighty, we get a chance for that we should respect the place where supreme deity of this pilgrimage Lord Shiva resides and should strive hard to keep it pollution free. We should not use polythenes and plastic packaging and should throw waste in proper dustbins. We should refrain from defecating in the open and promote use of lavatories and urinals that are fitted in camps and on the pilgrimage route. Our small contributions can help in big way to make this revered shrine a clean, pious and beautiful place to visit.

History of Saffron – the royal spice

Ancient Greek legends tell of brazen sailors embarking on long and perilous voyages to the remote land of Cilicia, where they travelled to procure what they believed was the world's most valuable saffron. The best-known Hellenic saffron legend is that of Crocus and Smilax. The handsome youth Crocus sets out in pursuit of the nymph Smilax in the woods near Athens; in a brief dallying interlude of idyllic love, Smilax is flattered by his advances, but all too soon tires of his attentions. He continues his pursuit but she resists. She bewitches Crocus and legend has it that he is transformed into a saffron crocus. Its radiant orange stigmas were held as a relict glow of an undying and unrequited passion, ever since.

Saffron, the most costly spice (next is vanilla), finds mention of its use in ancient texts from Greece to India to China to Persia to South Asia. It has been referred to have been used by priests to please their Gods; for ages kings and queens used it for their mystic powers and medicinal value. The history is rich.

The origin of the English word *saffron* is somewhat uncertain. It is understood to stem from the Latin word *safranum* via the 12th-century Old French term *safran*. *Safranum* may derive via the Persian intercessor or *za'ferân*. Some accounts mention *za'farān* is the arabicised form of the Persian word *zarparān*, *zar* + *par* + *-ān*—"having yellow leaves". An even older form is the Akkadian *azupiranu*, "saffron".The Latin form *safranum* is also the source the Catalan *safrà*, Italian *zafferano*, but Portuguese *acafrao* and Spanish *azafrán* come from the Hispanic Arabic *al-zaferán*, which comes itself from the Arabic *Z'.fran*. The Latin term *crocum* is certainly a Semitic word. It is adapted from theAramaic form *kurkema* via the Arabic term *kurkum* and the Greek intermediates *krokos* or *karkum*, which once again signify "yellowish".

Human cultivation and use of saffron spans more than 3,500 years and spans cultures, continents, and civilizations. Saffron, a spice derived from the dried stigmas of the saffron crocus (*Crocus sativus*), has through history remained among the world's most costly substances. With its bitter taste, hay-like fragrance, and slight metallic notes, the apocarotenoid-rich saffron has been used as a seasoning, fragrance, dye, and medicine. Saffron is a native to Southwest Asia, according to certain accounts it was first cultivated in Greece.

The wild precursor of domesticated saffron crocus was likely *Crocus cartwrightianus*, which originated in Crete or Central Asia. The saffron crocus propagates by vegetative multiplication via manual "divide-and-set" of a starter clone or by interspecific hybridisation. It is possible that humans may have bred *C. cartwrightianus* specimens by screening for specimens with abnormally long stigmas. The resulting saffron crocus was documented in a 7th-century BC Assyrian botanical reference compiled under Ashurbanipal, and it has since been traded and used over the course of four millennia and has been used as treatment for scores of disorders. The *C. sativus* clone was slowly propagated throughout much of Eurasia, later reaching parts of North Africa, North America, and Oceania. Global production on a by-mass basis is now dominated by Iran, which accounts for some nine-tenths of the annual harvest. For the ancient Mediterraneans, saffron gathered around the Cilician coastal town of Soli was of top value, particularly for use in perfumes and ointments. Herodotus and Pliny the Elder, however, rated rival Assyrian and Babylonian saffron from the Fertile Crescent as best—to treat gastrointestinal or renal upsets. It is said that Cleopatra used a quarter-.

-cup of saffron in her warm baths, as she prized its colouring and cosmetic properties. Egyptian healers used saffron as a treatment for all varieties of gastrointestinal ailments. In Greco-Roman times saffron was widely traded across the Mediterranean by the Phoenicians. The ancient Greeks and Romans prized saffron as a perfume or deodoriser and scattered it about their public spaces: royal halls, courts, and amphitheatres alike. When Nero entered Rome they spread saffron along the streets; wealthy Romans partook of daily saffron baths.

According to Chinese herbalist Wan Zhen, "the habitat of saffron is in Kashmir, where people grow it principally to offer it to the Buddha." He also reflected on how it was used in his time, that the flower withers after a few days, and then the saffron is obtained. It is valued for its uniform yellow colour. It can be used to aromatise wine. Saffron was detailed in a 7th-century BC Assyrian botanical reference compiled under Ashurbanipal. Documentation of saffron's use over the span of 4,000 years in the treatment of about ninety illnesses has been uncovered. Saffron-based pigments have indeed been found in depictions of prehistoric places in northwest Iran. The Sumerians later used wild-growing saffron in their remedies and magical potions. Saffron was an article of long-distance trade before the Minoan palace culture's 2nd millennium BC peak. Ancient Persians cultivated Persian saffron in 10th century BC. At such sites, saffron threads were woven into textiles, ritually offered to divinities, and used in dyes, perfumes, medicines, and body washes. Saffron threads would thus be scattered across beds and mixed into hot teas as a curative for bouts of melancholy. Non-Persians also feared the Persians' usage of saffron as a drugging agent. During his Asian campaigns, Alexander the Great used Persian saffron in his infusions, rice, and baths as a curative for battle wounds. Alexander's troops imitated the practice from the Persians and brought saffron-bathing to Greece.

Conflicting theories explain saffron's arrival in South Asia. Kashmiri and Chinese accounts date its arrival anywhere between 2500–900 years ago. Historians studying ancient Persian records date the arrival to sometime prior to 500 BC, attributing it to a Persian transplantation of saffron corms to stock new gardens and parks. Phoenicians then marketed Kashmiri saffron as a dye and a treatment for melancholy. Its use in foods and dyes subsequently spread throughout South Asia. Buddhist monks wear saffron-coloured robes; however, the robes are not dyed with costly saffron but turmeric, a less expensive dye, or jackfruit. Monks' robes are dyed the same colour to show equality with each other, and turmeric or ochre were the cheapest, most readily available dyes. Some historians believe that saffron came to China with Mongol invaders from Persia. Yet saffron is mentioned in ancient Chinese medical texts, including the forty-volume pharmacopoeia for medical treatments for various disorders. Nevertheless, around the 3rd century AD, the Chinese were referring to saffron as having a Kashmiri origin.

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History of Saffron – the royal spice

Ancient Greek legends told of sea voyages where adventurers sought what they believed were the world's most valuable threads. Another legend tells of Crocus and Smilax, whereby Crocus is bewitched and transformed into the first saffron crocus. Ancient perfumers in Egypt, physicians in Gaza, townspeople in Rhodes, and the Greek courtesans used saffron in their scented waters, perfumes and potpourris, mascaras and ointments, divine offerings, and medical treatments. In late Hellenistic Egypt, Cleopatra used saffron in her baths so that lovemaking would be more pleasurable. Egyptian healers used saffron as a treatment for all varieties of gastrointestinal ailments. Roman colonists took it with them when they settled in southern Gaul, where it was extensively cultivated until Rome's fall. Competing theories state that saffron only returned to France either in 8th-century AD or in the 14th century AD.

European saffron cultivation plummeted after the Roman Empire went into eclipse. The 14th-century Black Death caused demand for saffron-based medicaments to peak, and Europe imported large quantities of threads via Venetian and Genoan ships from southern and Mediterranean lands such as Rhodes. Saffron cultivation was introduced into England in around 1350 AD. The crop seems to have been initially grown in monastic gardens for medicinal use, only being planted in the less kind conditions of open fields many decades later. Soil and climatic conditions meant that by the sixteenth century, saffron cultivation had centred on Eastern England. However, an influx of more exotic spices—chocolate, coffee, tea, and vanilla—from newly contacted Eastern and overseas countries caused European cultivation and usage of saffron to decline. Europeans introduced saffron to the Americas. American saffron cultivation survives into modern times, mainly in Lancaster County, Pennsylvania.

Traditional Kashmiri legend states that saffron first arrived in the 11th or 12th century AD, when two Sufi ascetics, wandered into Kashmir. They having fallen sick, beseeched a cure for illness from a local tribal chieftain. When the chieftain obliged, the two holy men reputedly gave them a saffron crocus bulb as payment and thanks. To this day, grateful prayers are offered to the two saints during the saffron harvesting season in late autumn. The saints, indeed, have a golden-domed shrine and tomb dedicated to them in the saffron-trading village of Pampore, India. However, some scholars dispute this. According to them saffron has been cultivated in Kashmir for more than two millennia.

The domesticated saffron crocus, *Crocus s ativus*, is an autumn-flowering perennial plant unknown in the wild. It is a sterile plant and the purple flowers of *C. s ativus* do not produce viable seeds; reproduction hinges on human assistance: clusters of corms, underground, bulb-like, starch-storing organs, must be dug up, divided, and replanted. A corm survives for one season, producing via this vegetative division up to ten "cormlets" that can grow into new plants in the next season. The compact corms are small, brown globules that can measure as large as 5 cm (2.0 in) in diameter, have a flat base, and are shrouded in a dense mat of parallel fibres; this coat is referred to as the "corm tunic". Corms also bear vertical fibres, thin and net-like, that grow up to 5 cm above the plant's neck. The plant grows to a height of 20–30 cm (8–12 in), and sprouts 5–11 white and non-photosynthetic leaves known as cataphylls. These membrane-like structures cover and protect the crocus's 5 to 11 true leaves as they bud and develop. The latter are thin, straight, and blade-like green foliage leaves, which are 1–3 mm in diameter, either expand after the flowers have opened ("hysteranthous") or do so simultaneously with their blooming ("synanthous"). *C. s ativus* cataphylls are suspected by some to manifest prior to blooming when the plant is irrigated relatively early in the growing season. Its floral axes, or flower-bearing structures, bear bracteoles, or specialised leaves that sprout from the flower stems; the latter are known as pedicels. In spring, the plant sends up its true leaves, each up to 40 cm (16 in) in length. In autumn, purple buds appear. Only in October, after most other flowering plants have released their seeds, do its brilliantly hued flowers develop; they range from a light pastel shade of lilac to a darker and more striated mauve. The flowers possess a sweet, honey-like fragrance. Upon flowering, plants average less than 30 cm (12 in) in height. A three-pronged style emerges from each flower. Each prong terminates with a vivid crimson stigma 25–30 mm (0.98–1.18 in) in length.[18]

To get 1 lb (450 gm) of dry saffron requires the harvest of 50,000–75,000 flowers; a kilogram requires 110,000–170,000 flowers. Forty hours of labour are needed to pick 150,000 flowers. Stigmas are dried quickly upon extraction and (preferably) sealed in airtight containers. Saffron prices at wholesale and retail rates range from US$1,100–11,000/kg, in international market depending upon its quality. A pound contains between 70,000 and 200,000 threads. Vivid crimson colouring, slight moistness, elasticity, and lack of broken-off thread debris are all traits of fresh saffron.

Crushed saffron threads are soaked in hot—but not boiling—water for several minutes prior to use in cuisine. This helps release the beneficial components. Saffron's aroma is often described by connoisseurs as reminiscent of metallic honey with grassy or hay-like notes, while its taste has also been noted as hay-like and sweet. Saffron also contributes a luminous yellow-orange colouring to foods. Saffron is widely used in Indian, Persian, European, Arab, and Turkish cuisines. Confectioneries and liquors also often include saffron. Common saffron substitutes include safflower (*Carthamus t inctorius*, which is often sold as "Portuguese saffron" or "açafrão"), annatto, and turmeric (*Curcuma l onga*). Saffron has also been used as a fabric dye, particularly in China and India, and in perfumery.[83] It is used for religious purposes in India, and is widely used in cooking in many cuisines, ranging from the Milanese *risotto* of Italy to the *bouillabaisse* of France to the *biryani* with various meat accompaniments in South Asia.

Saffron also has a long history of use in traditional medicine. According to biomedical research there is some evidence to suggest that saffron may help alleviate the symptoms of major depressive disorder. Preclinical studies indicate that saffron could be a promising candidate for cancer chemoprevention studies. Early studies suggest that it may protect the eye from the direct effects of bright light, and from retinal stress in additional to slowing down macular degeneration and retinitis pigmentosa. Most saffron-related research refers to the stigmas, but this is often not made explicit in research papers.

Flooding of Srinagar
(Damages Caused and Rehabilitation)

1.0 Introduction:

These were the early days of September, 2014 and life in Srinagar was wading through damped ambience created due to untimely rains. Nobody had ever imagined in his far off dreams that this so called 'Paradise on Earth' the epitome of beauty was going to be gobbled up by the brutal water-dragon in next days to come.

The floods were wreaking havoc in State of Jammu-Kashmir causing huge damages. The fate of Srinagar turned out to be no different. This exquisite city was also ferociously mauled and bruised by the wrath of water-Gods on a fateful day. In this article we will concentrate our discussions on what happened particularly to Srinagar and its surroundings and efforts being made to come out from this catastrophe.

2.0 How Srinagar looks before floods:

Srinagar, the summer capital of Jammu & Kashmir, also called 'the Venice of the East' is located on both the sides of the Jhelum River, a tributary of the Indus. The river passes through the city and meanders through the valley all along.

A Panoramic View of Jhelum before floods

Nine Bridges joins both the sides of city across the Jhelum. The city is also known for its House-boats, Art-craft and beautiful Mughal Gardens.

2.1 **Demography and Facilities**:

The capital city of Srinagar is located 1585 metres above sea level. The Srinagar district with a population of around 11.93 Lakh souls (2011-census), is spread over an area of 294 Sq.Kms. It comprises two tehsils or towns viz Srinagar North and Srinagar South, besides 136 revenue villages.

Six mega and several small hospitals cater to health-careof its residents. Two reception centres provide travelling facilities for tourists as well for local people.

The city has four major universities and institutions besides several school and colleges imparting professional as well as academic education to students at all levels.

2.2 Business and People:

The markets of Srinagar before floods happened to be vibrant with people and tourists. The shops in Lalchowk, Residency Road and Polo-view could be seen studded with mobiles, electronics goods and handicraft works. Near the Lalchowk people used to throng market of Maisooma for arts & craft, spare parts and general goods.

On moving further towards west comes Budshah chowk, a bazaar of Kashmiri carpets, Pashmina shawls and embroidered & woollen clothes. If we cross Central Secretariat and High Courts at Batmaloo and land ourselves in Karan Nagar "the Commercial hub of Srinagar" we would have the feeling of strolling in some Delhi's business centre. Come on the other side near Dal Lake and people can be seen busy purchasing dry fruits at Batwara. Have a look from Dal-Gate towards the lake and your eyes will catch a mesmerising glimpse of lane of beautiful Hotels. Pashmina shawls, intricately hand-woven carpets, delicately carved wood-work, paper-mache work and glittering copperware are some popular items of the area. The Kashmir Government Art Emporium near zero bridge is one of the best places to shop for beautiful Kashmiri handicrafts. And last but not least the floating vegetable market on the Dal Lake can be seen selling fresh vegetables and flowers for the region.

The people of the city are mostly involved in Hotel, Catering, Transport, Tourism, Handicraft and Dry fruit business. They, though not desirous of joining rat-race of accumulating money, used to work sufficiently hard in order to live prosperously.

3.0 How it actually happened:

3.1 Flood History:

Floods in the state are generally caused by Jhelum River crossing the danger mark and thereby inundating the Valley in the process. The in-depth information about flood history has been given in a book written by Sir Walter Roper Lawrence 'The Valley of Kashmir' in 1895 and in another written by Saligram Bhatt in 2004 'Kashmir Ecology and Environment: New Concerns and Strategies'. According to them there have been almost more than 30 major floods in the archived history of Kashmir valley out of which the more devastating are as under:-

In AD 879 the channel of the Jhelum River was blocked by slipping of the Khadanyar mountains below Baramula resulting in submergence of large part of the valley.

In 1841 a major flood caused much damage to life and property. But first flood of devastating proportions hit the state later in 1893, when 52 hours of continuous rainfall lead to unprecedented calamity.

In 1903, ten years after, the valley recorded the most devastating flood, which up to now was considered as the greatest flood ever known. It started on 23 July 1903 and converted the whole Srinagar city into a lake.

After moderate floods in 1929, 1948 & then in 1950, a major flood again hit the valley in September 1957 which left nearly 15000 houses damaged and 100 people lost their lives. The flood was caused by the Jhelum's overflow.

In 1959 there was a major flood in the valley but in 1992 again a heavy flood occurred which caused over 200 casualties and resulted into over 60,000 people marooned in several north-western border districts of Kashmir.

And then is this flood in 2014 which qualified to be categorised as the **extreme of the extreme** floods in the archived history of Kashmir.

3.2 How the calamity started:

The Jammu and Kashmir state and adjoining areas had received heavy rainfall from 2 September 2014 onwards. Some areas received more than 350 mm rain in just four days when the annual average rainfall for Kashmir is around 1,000 mm. Srinagar itself was pounded by nearly four times more rain than normal for this time. This triggered flooding and landslides. On 3rd of September Chief Engineer, flood control Kashmir declared a flood alert in the Southern districts of Kashmir. Educational institutions in the South were closed. Many of the districts particularly of South-West Kashmir were badly hit. Next map will show these areas.

Bhupinder Salwan
XEN/SINA

SSE/SINA

Flooding of Srinagar
(Damages Caused and Rehabilitation)

Flood-Hit Region of Jammu-Kashmir

On 4th of September, flood alert was declared all across Jammu and Kashmir.

3.3 The day of disaster:

On 5th September, the Jhelum River was reported to be flowing at 6.83 m in Srinagar which was 1.34 m above the danger mark and at 10 m at Sangam at Anantnag district above the danger mark. The discharge rate in the river was recorded as 70000 m^3/second against the normal discharge of 25000 m^3/second.

Srinagar-Anantnag highway was declared unsafe for traffic on September 5 but still there was no indication of any major floods in Srinagar city. The raised water level in major Jhelum tributaries, persistently standing for more than a week, had caused leakage at permeable places of bunds of it. This further aided in breaching process at fragile locations and in the intervening night of 6th and 7th September the Jhelum breached its embankments at 84 places from Pampore down to Chathabal all along the Srinagar city inundating it completely. According to the satellite data, 100 sq km area in Srinagar was inundated although Bandipora (148 sq kms) and Pulwama(102 sq kms) were worst affected. But area of Srinagar was diabolically ravaged as it mostly comprised of high density residential areas.

People in the city had been caught unawares. People ran for their lives to the upper stories of their houses in the night. Some people in RajBagh area told that it took 18 minutes for the water to fill the first story of their house. Even in the Northern Railway complex of Tulsi Bagh the employees which happened to be present there mentioned that water was gushing in such a speed that water level galloped to 12 feet in mere 37 minutes.

Thousands of people including tourists were stranded and property worth billions came instantly under the water. Phone and mobile networks went completely down creating more panic. On the morning of 7th September communication network was completely disrupted. Electric supply had already been disconnected from main power stations for avoiding any electrocution. This further accentuated the problems of people struggling to save themselves while their well-wishers outside were at loss and nonplussed with no information of well being of their dear ones. There was a total chaos.

People moving through flood water to safer places

Water filled the Srinagar city and inundated areas of Shivpora, Indranagar, Rajbagh, Jawahar Nagar, Lal Chowk, Munawarabad, Bemina and many more areas. The Rajbagh was submerged in more than 20 feet deep water whereas Tulsi Bagh area also got nearly 12 feet of The worst affected areas are shown in next diagram.

A Construction Officers' Rest House near Srinagar Railway Station had been started in early months of 2014 by USBRL, Northern Railway. The floods showed no mercy to it also and encroached on its wooden floors in form of nearly 5 feet thick sheet of water for a period of about ten days.

A view of entrance Gate of ORH Srinagar *(Image captured from its balcony as author of article was himself trapped here during floods)*

Similarly Railway Complex at Tulsi Bagh which was inundated up to 12 ft. depth of water took about 20 days for complete receding of water.

4.0 Assessing the Damage caused by Flood:

The then J&K State Chief Secretary termed this flood as a disaster of international ramifications. State had never witnessed such a disaster before. It completely derailed the life in its affected areas and caused irreparable loss to every segment. Let us discuss them one by one:

4.1 Damage to Life and living stock:

About 12.5 Lakh families were affected by flood across the State. 284 lives have been lost due to floods in the State, 203 in Jammu region and 81 in Kashmir region. District wise deaths due to floods in Kashmir region are shown in Table below:

Flooding of Srinagar
(Damages Caused and Rehabilitation)

S. No.	District	No. Of Deaths
1	Srinagar	45
2	Kulgam	9
3	Anantnag	8
4	Budgam	6
5	Pulwama	4
6	Bandipora	3
7	Ganderbal	2
8	Shopian	2
9	Kupwara	2

45 human casualties alone in district Srinagar itself narrate the horrible state of that dreadful time. Worst affected was Rajbagh and Jawahar Nagar area where deaths were mainly due to collapsing of vulnerable and weak houses. 13 bodies were found in debris of a collapsed house due to floods at Jawahar Nagar. Unconfirmed sources also reported that nearly 300 cows in a military farm being maintained in Bemina area were drowned and died in flood water. More than 1000 poultry farms have been completely washed away. Bodies of many dead remained submerged in water and debris for days together.

4.2 Damage to Houses and Infrastructure:

A total of 5642 villages were affected by the flood across the State; 2489 in Kashmir and 3153 in Jammu division. Out of these nearly 800 villages remained submerged for over two weeks. Katcha houses there, as was expected, were natural victims of flood-fury and as per official figures 21162 of them turned into rubbles. The long standing water around the puuca structures also adversely affected their safety and stability. The old aged houses and those whose masonry was in mud mortar were severely hit and succumbed early. A total of 5142 pucca houses were stated to have been completely destroyed in Srinagar only.

The spate of water also damaged more than 550 bridges and culverts in the State. Besides, nearly 6000 km of road network was also eroded by the flood water.

4.3 Damage to Business, Shops and Agriculture:

As per official statement of State Govt. the most conservative estimate of loss of Business is at least 30,000 crore rupees. Business of export items like carpets, saffron, apples, willow supply and walnuts was destroyed to hilt. Sacks of soiled almonds, cashew nuts, saffron and dates have to be thrown into the Jhelum river. The retailers of grocery and daily needs were worst affected as they had nothing left on their racks.

A garment shop owners sorting their left over goods.

As per Kashmir Electronic and Mobile Distributors' Association floods damaged around 70 per cent of 500 shops of electronic items and mobile phones in Srinagar.

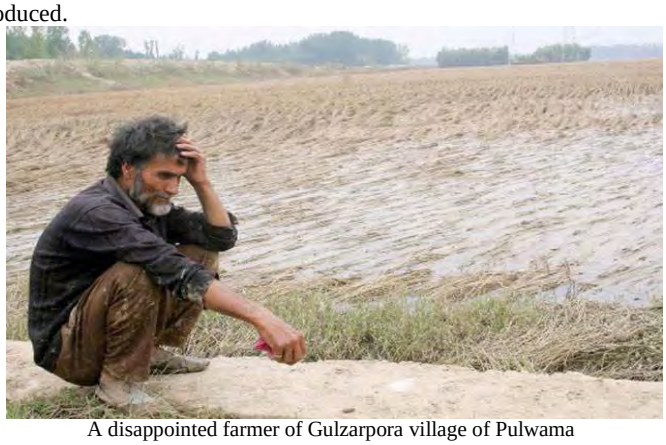
Similarly the business establishments dealing in books and other allied items became the worst sufferers as the floodwater damaged their all stocks and turned them into useless pulp. Majority of booksellers, publishers and stationers were located in and around Lal Chowk where water rose to nearly 10 feet high. Due to lack of awareness 90 percent of booksellers and stationers were not insured. Owners of these shops tried to cover some loss by selling less damaged books on throw-away prices. They lamented that no publisher, distributor or any institution came to their rescue in any way.

Tourism industry also suffered a huge set back. A large number of hotels near the Dal Lake have also suffered irreparable damages. Tourist reception centres in the city as well as near Srinagar Railway Station were badly devastated in the floods. Most of the houseboats in the Dal Lake were severely damaged by the floodwater, making them dangerous for people to live in. Twelve houseboats in the lake and river were badly damaged. The tourism industry was hit also in another way as besides facing a sharp dip in inflow tendency of tourists many cancelled their scheduled bookings.

The Cars and vehicle industry has another story to tell. Many of the automobile workshops and showrooms in Srinagar were also submerged under water and have incurred huge losses. Workshop of Toyota's showroom near Bemina bye-pass got submerged in the water but the main showroom where new vehicles were parked remained safe as it was situated at first floor level. The other showrooms like that of Hyundai, Mahindra and of General Motors near Hyderpora bye-pass could not be so lucky and their many vehicles went under water. However many cars were saved by sheer click of mind of some executive authority of these garages. As the water started rising, their managers got succeeded in parking some new cars in a row in side-lane of rising approach of flood Channel Bridge nearby and thus kept them out of reach of water. Approximately fifty thousand vehicles were damaged. Thousands have been repaired but a few thousands more are still in queue awaiting repairs in workshops.

Cars queuing in Srinagar for repair

As far as Agriculture is concerned Srinagar district mostly comprised of residential area and as such it does not suffer much on this account. However Pulwama, Budgam, Kupwara and Anantnag were worst hit. A large quantity of silt was also deposited in agriculture fields in many areas large. 250 families of the Island Villages in Dal Lake, Srinagar namely Tind Mohalla, Ball Mohalla and Akhooni Mohalla were also affected where vegetables including Nadroo, a principal crop of these villages, also got damaged. According to a survey carried out by Jammu and Kashmir Agriculture department, Pulwama district suffered maximum in terms of damage to agricultural produce with losses estimated to be Rs 1104 crore. The amount includes Rs 778 crore losses to Saffron crop as flood water inundated Pampore town of Pulwama, where most of the saffron is produced.



A disappointed farmer of Gulzarpora village of Pulwama

Medical care facilities also suffered a loss of over Rs. 150 crore. Shri Maharaja Hari Singh Hospital Hospital (SMHS) remained under water for almost 15 days. Medical infrastructure on ground floors of Lalla Ded Hospital, G. B. Pant Hospital and SKIMS (Sher-i- Kashmir Institute of Medical Sciences) were badly ruined.

'LALLA DED' Hospital in Srinagar seen surrounded by flood water

Specialized medical equipments like MRI systems, X-ray machines, ECG machines and ventilators etc. of these hospitals were completely destroyed by flood waters. The departments of radiology, ophthalmology, entire OPD and blood banks of SKIMS were gone completely.

The floods destroyed thousands of school buildings also. According to official figures, out of 11526 primary and middle school buildings, 1986 collapsed. The partially damaged buildings became unsafe for schooling. Some school buildings which remained in good condition have to be converted into shelters for flood affected people.

About two million books in libraries and private collections were damaged. All the books in the first floor of Central Library of the Government College of Women suffered irreparable damages.

Damaged books in a college Library

4.5 Damage to Heritage and Government assets:

Many priceless manuscripts, antiques and artefacts were destroyed in floods. Almost 90% per cent of the artefacts in Sri Pratap Singh (SPS) Museum were irretrievably damaged. Many of the 6th century Gilgit manuscripts, ancient Sharada manuscripts in Sanskrit and Kashmiri language and oldest surviving Buddhist manuscripts at the Jammu and Kashmir Academy of Culture and Arts were also damaged. The Academy lost all its publications dating since 1958. Apart from the manuscripts, other artefacts like Kashmiri shawls, exquisite sculptures and paper mache figures have been destroyed.

The records and files of many government offices were washed away in deluge. The ground floor of the High Court near Jahangir Chowk area came under water damaging all vital records and case files. The official residence of the Chief Justice at Sonawar also remained under water for over two weeks.

4.6 Damage to Railway infrastructure and property:

Construction Officers' Rest House at Srinagar Station was also inundated with flood water. The wooden planked flooring in ground floor of rooms was damaged with the initial surge of water. Water kept standing for about next ten days and in the course damaged the wooden doors and cabinets etc. and tarnished the wall finishing and its looks. The generator and compressor machines of centrally air conditioned system of the building also remained submerged and went completely out of order.

Damaged Furniture and Records in Tulsi Bagh Rly.Complex

In the Railway and IRCON offices at Tulsi Bagh Holiday Home complex water came gushing in. The staff tried hard and and saved some of the precious records. But most of its office furniture, record and electrical gadgets on ground floor badly damaged.

5.0 Causes of Floods:

Before moving to rehabilitation works let us have a brief glance on factors which gave birth to this catastrophe. The analysis of reasons behind these floods is based on various reports, researches and views of experts and can be summarised as follows:

5.1 Heavy Downpour: Normally September is not a rainy season in Kashmir. But this time the combined effect of western disturbances from the Caspian Sea and Monsoon currents paved way for heavy rains. Many areas received more than 350 mm rain in just four days from 2nd to 6th of September against annual average rainfall of 1,000 mm. Heavy and incessant rains acted as catalyst to cause the glaciers melt faster. Rains and snow-melts combined together to make the rivers flow way above normal levels. This resulted in overflow of water and breaches in banks.

5.2 Unplanned Urbanization: It was natural for water to overflow its banks in the event of heavy rainfall into flood plains. This was the basic right of way for the overflowing water. Flood plains for some years were being indiscriminately developed into housing colonies. This not only resulted in blockage of natural drainage causing floods but also caused extensive damage to these residential areas when inundated.

5.3 Neglect of de-silting of major rivers and flood channels: Illegal felling of trees and forest fires are continuously contributing to soil erosion in catchments of Jhelum, Chenab and Tawi. Mountain streams wash down this loose soil and deposits in the rivers. Due to this river Jhelum and spill channel had been heavily silted up over the past fifty years. This has resulted in carrying capacity reduced from 45,000 cusecs in 1975 to 32,000 cusecs in 2012. (One cusec is the flow of one cubic feet of water in one second at any given point). Reduced capacity contributed in more silting as formation of minimum draft of water required for mechanized transport of silt ceased. Ultimately more reduced capacity restricted fast passage of water at time of floods.

5.4 Failure of official machinery: As per the past practice the bank at Kandizal was used to be breached to divert part of flood waters and gates at Nalla Amir Khan, Chattabal weir and Doodh Ganga diversion opened to drain off excess water. These exercises were necessary to keep the water level of river within safety limits but probably neither of these essential and emergent actions was taken by the responsible departments in time. Besides, the weak bunds of the river were also not strengthened well in advance. This caused breaches at vulnerable places of bund and resulting floods in Srinagar.

Moreover the worst floods hit in 1841, 1903 and 1957. It means the average periodicity of flood cycle was approximately 55 years and next major flood should have been expected to be looming large but no flood fighting preparedness was carried out by the government. Even the proposals for various necessary works like dredging and increasing water carrying capacities of channels remained swinging like pendulum between State Flood Authorities and Central Water Commission.

6.0 Immediate Rehabilitation & Relief works

6.1 Saving Human Lives:

The immediate need of the hour at time of strike of this calamity was to ensure safety of human lives and evacuate the population trapped in submerged areas. Although the Jammu and Kashmir government had mounted a massive relief operation, it fell too short of the challenge. People in many areas of the capital city and South Kashmir areas complained of inadequate or delayed responses from authorities. The Prime Minister also visited the inundated areas of upper Srinagar to take stock of the devastation and announced ex-gratia reliefs to the affected people.

'Mission Sahayata' was launched by the Army in which as per official statements of defence spokesman 82 aircraft and helicopters, 10 battalions of Border Security Force, 329 columns of Indian Army, 300 boats and 13 engineer teams were deployed in rescue operations in Kashmir. The helicopter-based rescue mission did not meet much success due to bad weather and slanting roof tops and was limited to save trapped army personnel. Finally more water boats were provided for rescue teams to reach the people and intensified the relief operation although submerged debris, boundary walls, cars and other materials made it difficult for boats to enter the area for relief purposes. The National Disaster Response Force (NDRF) with their eight teams also worked hard in tandem with them in these life saving endeavours.

Soldiers rescuing marooned people

The two army jawans, Naik Khem Chandra & Mir Owaisi Hussain were also washed away in the floods during rescue operations in Pulwama in south Kashmir on September 7 and were later declared dead. Army has to face resistance from stone pelters and miscreants also in Srinagar and in one incident army was resorted to open fire in air to disperse them in Lal Chowk area.

But it was the assistance, efforts and exemplary bravery of local youth without which the rescue efforts of the Army and other agencies would not have been possible. They guided the NDRF and Army boats to reach the places requiring instant help. Even Lt. General of Army in his statement admitted this fact and appreciated them. Barring a few instances of accusations on Army of selective rescue operations and stone pelting by the bystanders who created hindrances in relief works everyone worked in resonance by helping each other. Young and dynamic local boys also strained every nerve to evacuate trapped flood victims. They made makeshift rafts of empty drums and wooden boards and worked day and night to save precious lives of people.

Makeshift boats in the Bemina area of Srinagar

The total of about 200,000 people was rescued, including 87,000 from Srinagar city by collective effort of all.

Construction Officers' Rest House at Srinagar which has ten lavishly built up suites with a grand Conference Hall and a Dining Hall was also feared to be inundated in rising flood waters. So on 6th of September the authors with their team and housekeeping staff made all out efforts and succeeded in lifting the precious items like furniture, LCDs, electronic gadgets and tapestry from ground floor to the first floor.

Author being rescued from Construction ORH Srinagar

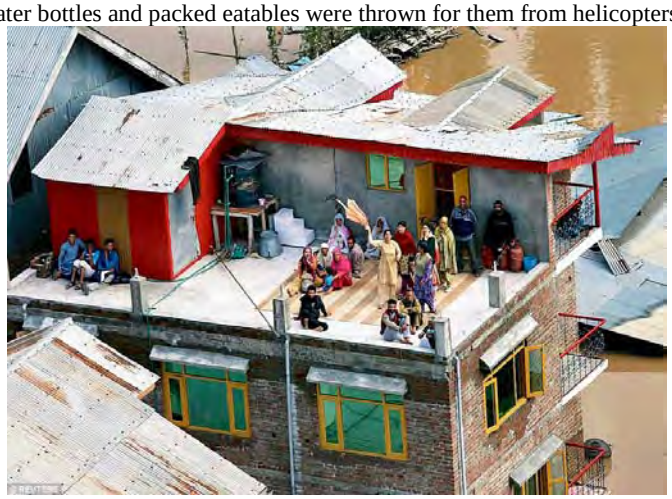
However during succeeding night the whole team was trapped with about 8 feet water surrounding the building and have to be rescued by clinging and crawling to a rope stretched between first floor balcony and rails of the track.

Some Railway staff and their families of Open Line were also trapped in the first storeys of quarters in Railway Colony Srinagar. They were rescued by their colleagues and officers by using makeshift boats.

Railway Officers along with IRCON staff who was residing in guest houses and leased accommodations in Rajbagh area were also trapped in water up to two storeys. They all are successfully saved after four days of floods using boats.

6.2 Providing food, shelters and assistance:

Floods had trapped many people on their roof tops while water had surrounded their houses up to nearly first floor. Those which were still to be rescued had been lying there without food and water. Food Packets, water bottles and packed eatables were thrown for them from helicopters



People trapped on roof tops in Srinagar

There have been incidents when the helicopters were targeted by people with stones when flying at low altitude. One of the helicopters was hit by several stones causing some minor damage on the body near the rotary wings. But confronting all odds more than 4,00,000 litres of water, 1,31,500 food packets and over 800 tonnes cooked food was airdropped and distributed in the flood-affected areas.

Many relief shelters were provided by the Government as well as by NGOs for the people who had to abandon their submerged homes. Many of such people, except for the clothes they were wearing, could not salvage a single thing. In Srinagar region, camps were established at BB Cantt, Avantipur, Old Airfield, Sumbal, Chattargam and Jijamata Mandi.

Thirty three percent of relief shelters were housed in schools but health, hygiene and wastage disposal mechanism remained a major issue in majority of them. Blankets were distributed in them but that proved insufficient. On demand of State relief and rehabilitation Commissioner Sh.Vinod Kaul 25,000 tents and 40,000 blankets were dispatched to the state. More relief materials including water bottles were also provided by IRCTC and food packets were airlifted from Hyderabad, Ahmedabad, Baroda and Amritsar for them. 25 tonnes of Indian gram (channa) were transported to the flood affected areas by NAFED.

Prime Minister of India announced an assistance of Rs.745 crore to the State Government in addition to Rs.1100 crore already earmarked for the disaster. Chief Ministers of Maharashtra, Telangana, Madhya Pradesh, Tamilnadu, Gujrat and Andhra Pradesh announced an financial assistance and appealed to the people to come forward and help those in need. Corporate Houses like Mahindra and Mahindra Group and Samsung Electronics also donated Rs.two crore and Rs.three crore respectively towards the Prime Minister's National Relief Fund.

6.3 Evacuating standing water & preventing diseases:

The Raj Bagh, Jawahar Nagar and adjoining areas form a saucer like surface which had got filled with ten to twenty feet deep water as a result of breaches in Jhelum. Some water receded but area had been under stagnant flood water for number of days. Municipal authorities were draining out the water from waterlogged areas by using 20 fire engines but quantity of water here was too large to handle. So the authorities used heavy-duty suction pumps sent from Vishakhapatnam to evacuate water from these low-lying residential areas. Twelve sewage pumps brought from Delhi were also deployed for the purpose.

Diseases threatened as Kashmir flood waters turned fetid. So precautionary measures were also taken to prevent the spread of water borne diseases and health advisories were issued. Children in the age group of 6 months to 15 years were vaccinated.

Floating carcasses had become a big problem with most houses still waterlogged

Dead Cattle of a milk-farm in Srinagar

Over 1500 carcasses were removed from Srinagar only and scientifically disposed off. Dead animals were buried after disinfecting them with bleaching powder to avoid ground contamination in future. Besides, hundreds of tons of garbage were removed from the city and dumped away from residential areas.

Masks were distributed among aid workers and local residents to avoid the pathogens and intolerable foul odour but that did not prove of much help in any way.

Twenty five water filtration plants with the capacity to filter 400,000 litres a day and 13 tonnes of water purification tablets were sent by Central Government. Aerial fumigation was done to prevent waterborne diseases like cholera.

People had started coming up with respiratory infections, skin irritation, chest infection, diarrhoea and gastric problems. Infants and children were given immediate attention as are more vulnerable to these diseases. So makeshift medical centres were installed but shortage of life-saving drugs, intravenous fluids, sanitary products and medical staff started cropping up. The Centre then sent all medical help, including sending doctors and paramedic staff to the State. Boats laden with drug supply, doctors and health workers were mobilised to reach faraway affected places and people. Voluntary Health Association of India (VHAI), a non-profit and registered society striving to build up a strong health movement in the country, also provided emergency support and established Medical Health Camps at Island Village of Dal Lake, Srinagar and distributed Relief Material there.

10 tonnes medicines and other health care materials including mobile oxygen generation plant reached Srinagar from Delhi. Thirty generator sets of 3 to 5 KVA capacity were also sent to Srinagar to augment continuous power supply in relief camps and field hospitals. Armed Forces Medical Services also sent its 80 medical teams for medical camps.

6.4 Restoring Basic Services:

Entire communication network of the State had been crashed during the floods. The local television and radio stations in Valley had remained off air and were disconnected with the rest of the country. BSNL launched an operation on a war-footing with Indian Army and Indian Air Force (IAF) to restore mobile services through satellite network and the telecom network. BSNL air lifted new equipments from Jammu to Srinagar to partially restore 90 towers of 3G telecom service and managed mobile services to some extent.

Government deployed technicians and restored power supply in the Valley and for the task.

The air connectivity to Srinagar and Jammu was nearly normal from start. The train service between Baramulla-Srinagar which had been suspended was again restarted on 15, September, 2014.after carrying out day-night work on repair of a washed-out bridge embankment extended train service up to Banihal was made functional within next eight more days. The National Highway NH-1A had been cut off at various places near Ramsoo, Ramban and Banihal area due to heavy landslides. With painstaking efforts of five task forces of Border Roads Organisation it was again opened on September 8 whereas Highway of Srinagar-Leh was re-opened on September 9.

Water supply was also put back on track. The service in worst affected areas like RajBagh where even inner water lines had been damaged was also temporarily resumed.

7.0 Long Term Rehabilitation Measures

It takes long time and systematic approach to stand tall out of such calamities. Reconstruction, rehabilitation & business revival remained a gigantic task ahead; both on physical as well as on financial front. The State Cabinet on 10 October, 2014 approved a proposal to seek package from the centre amounting to Rs 44000 crore.

The main provisions demanded in this proposal are as under:

a) Issuing directions to all banks and financial institutions for re-scheduling of loans/granting of moratorium.
b) Waiving of loans up to Rs 3 Lakh for individuals.
c) Interest subvention of five percent on the loans.
d) Payment of ex-gratia relief against the loss of private structures to the tune of Rs. 9.0 Lakh for fully damaged pucca house, Rs. 6.0 Lakh for fully damaged kucha house, Rs. 4.0 Lakh for partially damaged kucha/pucca house including boundary walls and Rs. 1.0 Lakh for other structures.
e) Loan up to Rs. 20.0 Lakh at 4 percent interest for those whose houses are fully damaged and providing hired accommodation for about a year till their houses are rebuilt.
f) Compensating the affected people for the loss of land due to flash floods and landslides at 50 percent of the land's value in that area.
g) Loan facility (without any collateral security) up to Rs. 5.0 Lakh to be provided at 2 percent interest rate annually for small shopkeepers not registered under VAT and loan up to Rs 20 lakh at 4 percent and loan up to Rs. 50 lakh rupees at 6 percent annual interest for registered dealers with the sales tax authority

The review of various proposed steps to be taken and already started being taken in different sectors taken in meetings by different authorities are discussed as follows:

7.1 Business and Industry Sectors:

The process of rehabilitation critically depends on sources of financial support. These cover State government's proposal of aid, funding through Corporate Social Responsibility and assistance in schemes by banks. Kashmir Economic Alliance has urged the government of India to release relief and rehabilitation package. They have also appealed to the Union Petroleum and Natural Gas Ministry to revoke its order reducing kerosene quota to Valley by 18.48 lakh litres per month. Corporate Houses also came forward to extend its support to the flood-affected people of Jammu and Kashmir.

Shopkeepers worked overtime with labour to clear up the rubble and dredge out remaining water from their shops to restore their functioning and most of them succeeded in completing before the onset of chil-e-kalan (40 days period of deadly cold from 23rd December to 31st January) in Kashmir. Businessmen also faced another road-block from Commercial Tax Department who wanted the traders to submit certification from Revenue department to the effect that waters have gushed into their establishments for granting exemption to them.

Many shopkeepers went ahead with massive sales on soiled and dampened goods for clearing stocks. Shopping malls like retail chain of V-Mart announced the reopening of their stores.

But most debilitating impact on business is that purchasing power of a common man has dipped low.

7.2 Housing Sector:

The State Government had sanctioned Rs 75000 for fully damaged houses and Rs 3800 for partially damaged structures as interim relief. In a meeting of officers convened to review the rehabilitation process by District Development Commissioner, Srinagar, on 14 February, 2015 it was revealed that an amount of Rs. 85.74 crore has already been distributed among the house owners as relief. Flood affected people living in rented accommodations were also being provided Rs. 5000 per month while those living in rural areas were being provided Rs. 2000 per month as rent expending an amount of Rs. 12.58 crores.

The reconstruction of damaged houses although is in full swing now but an inevitable delay took place due to following reasons:

i) Some victims got re-assessment of losses while others went through resolution of disputes between the families.
ii) Non-availability of non-local labourers from states as Bihar and Uttar Pradesh and the fact that local labour was very expensive.
iii) Permission for construction of building from state local body institutions like Srinagar Municipal Corporation (SMC), Srinagar Development Authority (SDA) and the Lakes and Waterways Development Authority LAWDA.

While field staff continued to promptly re-verify, re-assess and redress the grievance cases and the government had announced a policy on October 8, 2014 that the respective competent authority had been authorized to grant permissions within a period of three days the scarcity of non local labour will certainly result in reconstruction and rehabilitation work to take longer time.

For those who have to continue to remain on road or live in tents or in relief camps government has announced distribution of free ration for next 6 months. Allegations of nepotism, corruption and erratic selection however surfaced during these relief distributions.

7.3 Insurance Claims:

The J&K High Court had directed the four PSU insurance companies to pay 95% of the claim amount of those who had taken insurance cover for Rs 25 lakh or less and 50% for those claims by persons who had taken cover for more than Rs 25 lakh. Attorney general had pleaded with the Supreme Court on behalf of the insurance companies that this HC order may lead to abuse by unscrupulous elements and companies must be permitted to at least conduct a preliminary survey of the damage before settling claims. But the apex court did not change the HC orders. Accordingly insurance companies settled claims.

7.4 Offices and Records:

Many records of various departments in the Civil Secretariat had been damaged in floods. So the government issued guidelines for reconstruction and rebuilding of the records. The departmental committees have been constituted for categorising the records into very old, old and current categories.

7.5 Infrastructure and Agriculture:

The hospitals have resumed the out-patients department. Electricity, drinking water and oxygen supply has also been restored. All useable diagnostic equipment was placed on the first floor or above after isolating the inundated ground floor for repairs. The damaged equipments like CT scan and other radio diagnosis facilities have been processed to be replaced. Till time private centres have been tied up with for X-rays and other medical tests.

Public works department has temporarily repaired 288 damaged bridges and 125 km damaged road.

A plan amounting to Rs 453.51 crore has been formulated to reclaim and rejuvenate the damaged land and for carrying extensive soil conservative measures.

The above discussion reveals slow progress of rehabilitation works. The one of the main reasons is delay in release of relief package. A severe winter and impediment of model code of conduct for elections have further delayed it. Central Government might hand over the relief package to the next state government taking charge.

8.0 Could this Catastrophe be averted:

There are no methods to stop rains which were an originating cause of floods. But even then floods and resulting devastation in Srinagar could have been prevented if following were taken care of:

Reading the warning signs in time even when non-stop rains, nearly 400 mm in last four days.

- Bringing level of Jhelum River within safety limits by opening gates/breaching bunds in time.
- Chasing the flood control programmes like increasing carrying capacity of Jhelum and flood channels, dredging and strengthening of Banks.
- Careful planning of housing colonies and stopping encroachments on flood basins and wet lands.
- Coordination between State Flood department and Central Water Commission on proposals.

Well equipped preparedness of Disaster Management

Conclusions:

Ignorance is not always bliss. Flood-hit Srinagar has perhaps paid the price of ignorance that local authorities had shown. The memories should not be so short lived as to lose focus on the need for urgent strategies and farsighted action plans to minimize the impact of such probable disasters in future. The governments should go for meticulous planning and then start taking persistent and timely actions on both short term and long term fronts. The people of State should also lend support and helping hand in their implementation. Only then such disasters can be warded off in future.

Mushrooms: Vegetable of Future

Indian agriculture is known for providing employment, livelihood, food, nutritional and ecological securities to its people. Today, owing to changed life style and usage pattern in terms of food habits and growing demand, cultivated mushrooms and green house generated vegetables have found their importance and therefore grown on farms throughout the world. In India, mushroom is a unique non-traditional cash crop grown indoors, both as a seasonal crop and round-the-year under the controlled environmental conditions.

World Production

World production of mushroom is growing and now exceeds three million tonnes worth a market value of US $ 10 billion and *Agaricus bisporus* accounts for most of this production. China is the largest exporter of preserved mushrooms with a market share of 41.82%. Netherlands (25.11%) and Spain (7.37 %) are the other major countries. India ranks sixth with a market share of 4.44 %.

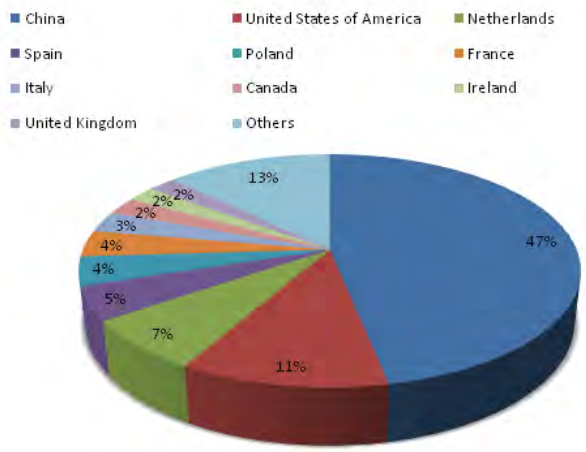


Fig 1: World production of fresh Mushrooms

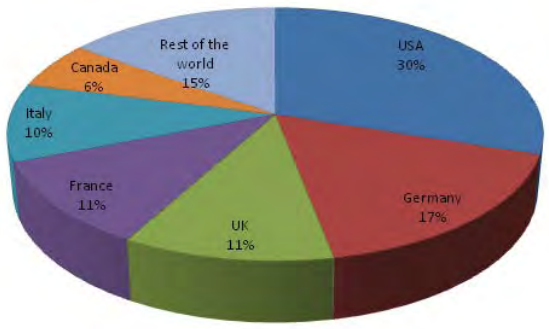


Fig 2: World Mushroom consumption

Mohini Prabha Singh

D/o M.B.Azad AXEN/HQ

Though China (47 %) is the major producer of Mushrooms, nevertheless maximum portion is consumed by USA (30 %) followed by Germany (17 %).

Indian status

Currently India stands 54 in the world ranking of producers. Mushroom production in India has been estimated at 48000 tonnes per annum. Punjab alone produces 20-25 % of total produce followed by Himachal Pradesh and Haryana. Currently three varieties of mushrooms are being cultivated in India. These are the white button mushroom (*Agaricus bi sporus*), the paddy straw mushroom (*Volvariella vo vacea*) and the oyster mushroom (*Pleurotus sajor caju*) of these *A.bisporus* is the most widely and economically cultivated variety throughout the world.

The natural advantages for mushroom cultivation in India are the availability of cheap labor as this is a labor intensive process, presence of seasonal variations enabling us to cultivate different mushrooms under natural conditions in the form of crop rotation in different seasons and regions and lastly the abundance & availability of variety of agrowastes at low prices for mushroom cultivation.

Three mushrooms are commonly cultivated throughout India namely button (*Agaricus bi sporus*), Oyster (*Pleurotus* spp.) and paddy straw mushroom (*Volvariella vo lvacea*). Their ability to grow on agricultural wastes in less area makes them an attractive proposition for income generation. The added advantage is in improvement in dietary leading to solution to malnutrition, pollution abatement and diversification of agriculture. Mushroom cultivation produces about 32 tons of dry protein per year in per acre of land while by fish farming only 3 quintals of proteins can be produced. The agriculture wastes which are burnt and cause environmental pollution, if can be used for mushroom cultivation will not only check pollution but will also play an important role in carbon sequestration and proper utilization of waste products.

Mushroom cultivation also provides employment generation. There is ample scope to earn more from mushroom cultivation using some innovation like attractive packaging for longer shelf-life, processing units for canned items, value addition and new products such as mushroom nuggets (*burries*), biscuits, *papads*, pickles, soup powder, etc.

Food security

Mushrooms are of excellent food value as they provide a full protein food containing all the twenty one amino acids besides containing useful amount of fats, vitamins and minerals. Mushroom protein being easily digestible (70-90%) is considered superior to vegetable proteins. Two essential amino acids lysine and tryptophan are enormously present in mushrooms which are not found in cereals. Being low in caloric value (300 – 390 Kcal/100 g dry wt), low fat and high protein, they are considered as 'delight of diabetic patients'. Folic acid and Vitamin B-12 which are normally absent in vegetarian foods are present in mushrooms (3 g fresh mushroom can supply 1 micro g vitamin B12, recommended for daily uptake).

Medicinal properties

Mushrooms have antitumour, anti-cancer and many other therapeutic properties. Being rich in folic acid they counteract pernicious anemia. The polysaccharides found in mushrooms have proven antitumour activity. Antimalarial, antifungal and antiviral principles are attributed to many mushrooms. Ergosterol is present that is converted to vitamin D by human body. High K:Na ratio is desirable for patients with hypertension. Besides this the high fiber content helps easier digestion.

Value addition

Value added products from mushroom are a promising enterprise. Mushroom being highly perishable forces the producer to preserve and process it. Preservation is essential to make it available all through the year to retain maximum nutrients, texture and flavor

and to increase its per capita consumption in developing countries. Presently the mushroom products available are bakery products (biscuits, bread, and cakes), pickles, chutneys, nuggets, papads & fast food items like burgers, cutlets and pizza etc.

Laboratories

Three institutes are currently involved in Mushroom studies namely Central Food Technological Research Institute (CSIR), RRL Jammu (CSIR) and National Research Center for Mushroom (ICAR).

Government support

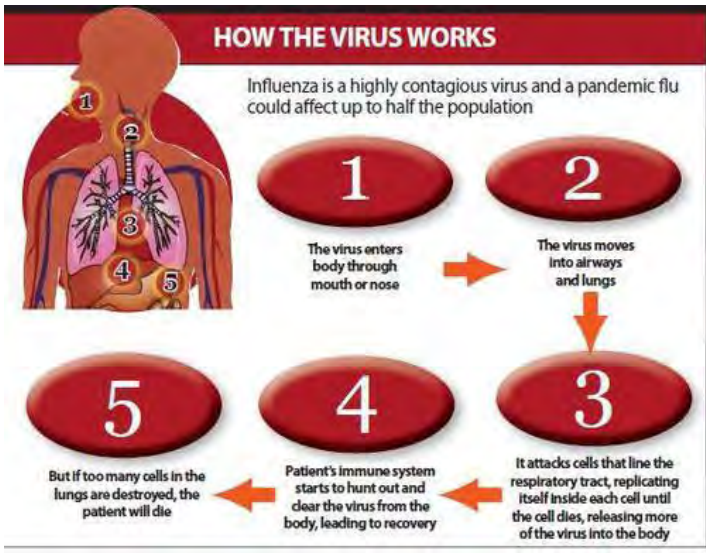
There is a central scheme on mushroom farming by Ministry of rural development which can be availed through CAPART and other is through Min. of Food Processing. The main focus of this scheme to train, information dissemination, technical and financial assistance for preparation of cultures/spawn cultivation, harvesting, storage, processing, packaging, marketing linkages with farmers to employment opportunities and generating income with special emphasis on women. Other than this different types of financial assistance and soft loans are also available from National Co-operative Development Corporation (NCDC), National Horticulture Board (N.H.B.), A.P.E.D.A., State Govt. Agencies responsible for development of Agriculture and Agro-based ventures.

Major constraints

Mushrooms can solve many problems of under nutrition and malnutrition. Despite this, mushroom cultivation and its utilization is not catching up fast due to lack of awareness to incorporate in mid-day meals, in regular diet of school children, in Govt. canteens and hospital wards to alleviate the above discussed problem. There is also lack of good quality mushroom spawn laboratory, cold storage facility and small scale processing. Nevertheless, under prevailing circumstances such sources of protein could be exploited to save the country from hunger and malnutrition. Edible mushrooms can therefore be used as a weapon against starvation and in a way contributes to food security by being easily available, affordable and usable.

Swine Flu Essentials

Swine flu is a highly contagious respiratory disease in pigs caused by one of several swine influenza. Transmission of swine influenza viruses to humans is uncommon. However, the swine influenza virus can be transmitted to humans via contact with infected pigs or environments contaminated with swine influenza viruses.



Signs and symptoms

Manifestations of H1N1 influenza are similar to those of seasonal influenza. Patients present with symptoms of acute respiratory illness, including at least 2 of the following:

- Fever
- Cough
- Sore throat
- Body aches
- Headache
- Chills and fatigue
- Diarrhoea and vomiting (possible)

In children, signs of severe disease include apnea, tachypnea(fast breathing), dyspnea(difficulty in breathing), cyanosis (bluish discoloration of skin and mucous membrane) , dehydration, altered mental status, and extreme irritability.

Diagnosis

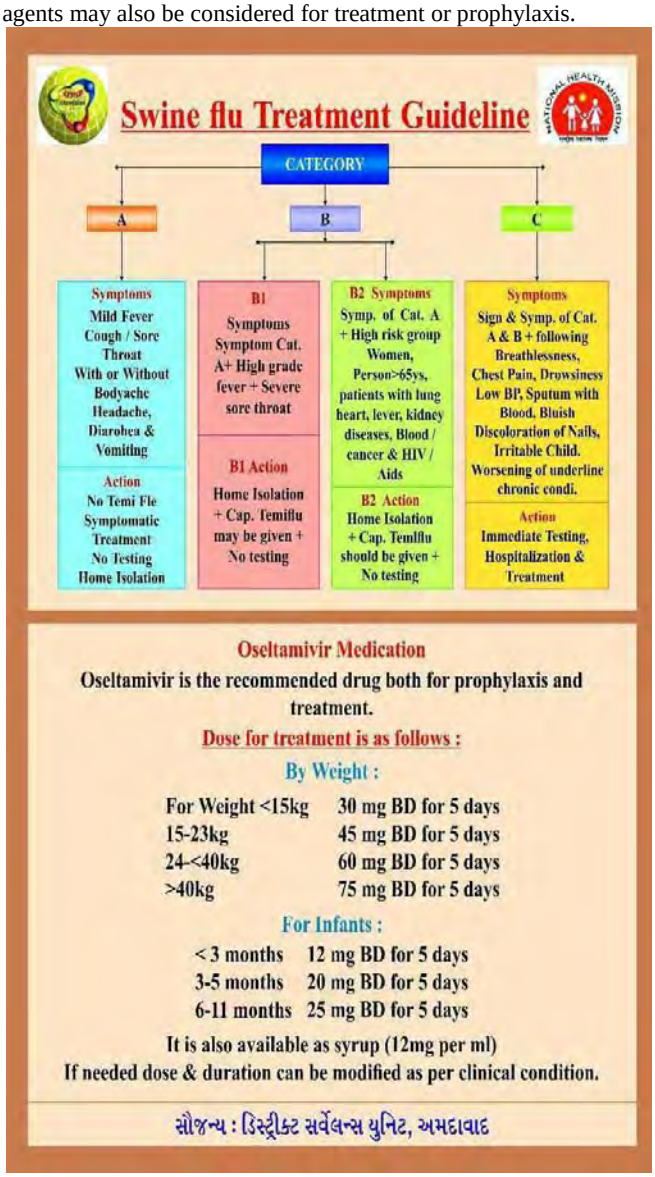
The criteria for suspected H1N1 influenza are as follows:

- Onset of acute febrile respiratory illness within 7 days of close contact with a person who has a confirmed case of H1N1 influenza A virus infection, or
- Onset of acute febrile respiratory illness within 7 days of travel to a community where one or more H1N1 influenza A cases have been confirmed, or
- Acute febrile respiratory illness in a person who resides in a community where at least one H1N1 influenza case has been confirmed.

Dr. Steven George
DyCMO/KRCL/Reasi

Management

Treatment is largely supportive and consists of bedrest, increased fluid consumption, cough suppressants, and antipyretics and analgesics (eg, acetaminophen, nonsteroidal anti-inflammatory drugs) for fever and myalgias. Severe cases may require intravenous hydration and other supportive measures. Antiviral agents may also be considered for treatment or prophylaxis.



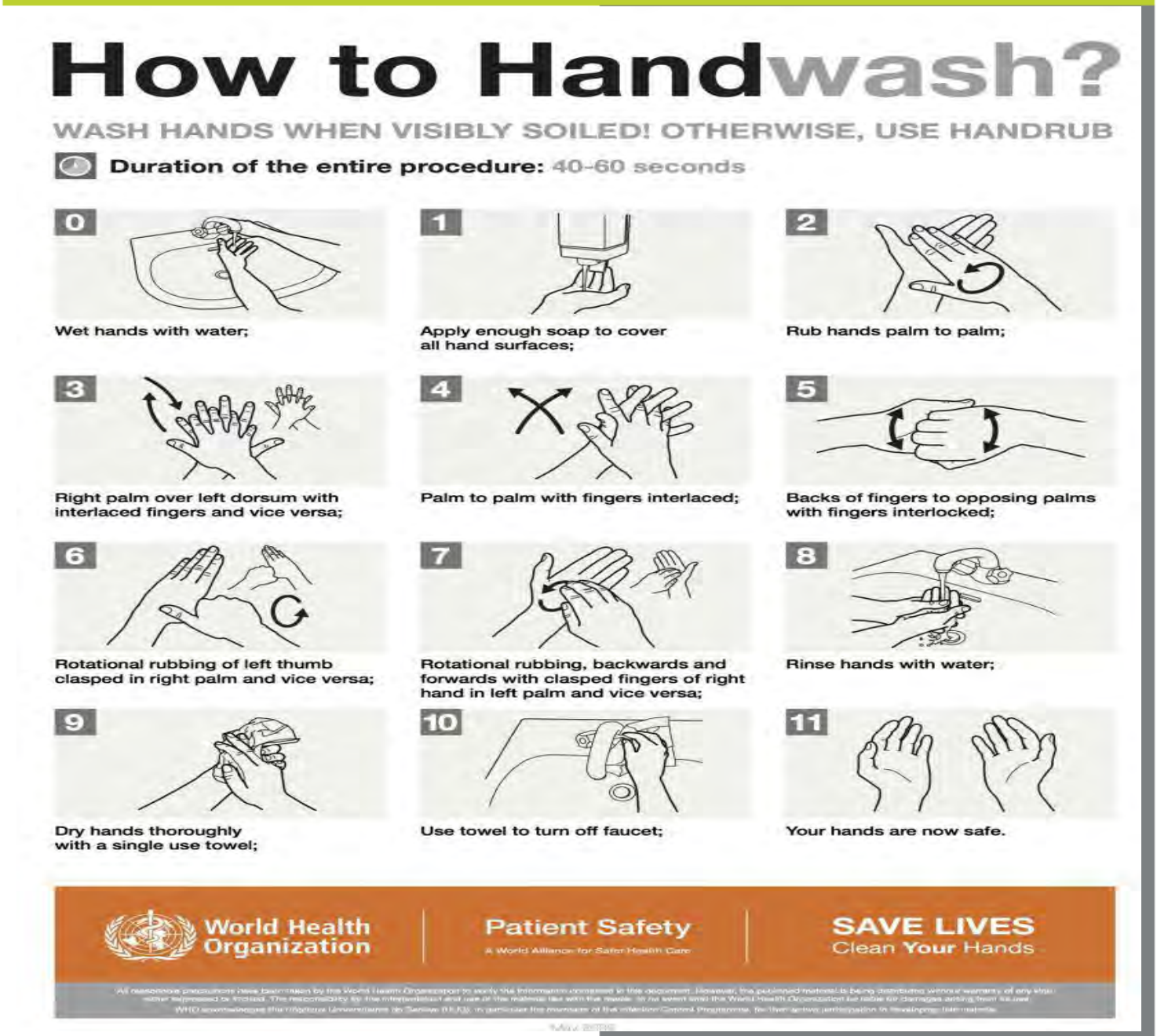
History

The ability to trace outbreaks of swine flu in humans dates back to investigation of the 1918 Spanish influenza pandemic, which infected one third of the world's population (an estimated 500 million people) and caused approximately 50 million deaths. In 1918, the cause of human influenza and its links to avian and swine influenza was not understood. The answers did not begin to emerge until the 1930s, when related influenza viruses (now known as H1N1 viruses) were isolated from pigs and then humans.

Here are few tips for you to keep away from the pandemic.

Wash your hands frequently

Use the antibacterial soaps to cleanse your hands. Wash them often, for at least 15 seconds and rinse with running water. Keep hands off your face when outside home to prevent transmission of virus.



2. Get enough sleep

Try to get 8 hours of good sleep every night to keep your immune system in top flu-fighting shape.

3. Drink sufficient water

Drink 8 to10 glasses of water each day to flush toxins from your system and maintain good moisture and mucous production in your sinuses.

4. Boost your immune system

Keeping your body strong, nourished, and ready to fight infection is important in flu prevention. So stick with whole grains, colourful vegetables, and vitamin-rich fruits.

5. Keep informed

The government is taking necessary steps to prevent the pandemic and periodically release guidelines to keep the pandemic away. Please make sure to keep up to date on the information and act in a calm manner.

Salient Features of the Proiect

Item	Udhampur -Katra	Katra- Qazigund	Qazigund -Baramulla	Total
Route length	25	128*	118	271
(km)				
Ruling gradient	1 in 100 ©	1 in80©	1 in 100 ©	
Max Curvature	5 o	2.75 o	2.75 o	
Bridges	38	62	811	911
Max. height of Bridge	85 m	359 m	22 m	
Length of Bridges(m)	1488	7310	4210	13008
Longest span	154m Steel Girder over river Jhajjar	467 m Steel Arch over river Chenab	45 m	
Tunnels Length	10.9	105	0	115.9
(km)				
Tunnels (No)	9	29	0	38
% Tunnels Length	43.6	79.84	0	
Longest tunnel	3.15 km.	11.27 km	-	
Max Depth of Cutting	20 m	40 m	12 m	

The Jammu Udhampur Srinagar Baramulla Rail Link Project was envisioned with a view to provide a reliable and alternate transportation system in the state of Jammu and Kashmir and to connect the state and the Kashmir valley with rest of the Indian Railway Network. With the above vision, Government of India planned a 326 km. long Railway Line. The Project was declared as a "National Project " in year 2008.

Some of the special features of the project are as under:-

- The Jammu-Udhampur-Katra-Quazigund-Baramulla Railway line is the biggest project in the construction of a mountain railway since independence. From Jammu to Baramulla, length of the new rail line is 326 km. and it passes through the young Himalayas, one of the most geologically complicated and challenging terrains in the world. The Geology, tectonic thrusts and faults, drainage and ground water of the region have great bearing on the construction of this project.

Sites are remotely located, inaccessible and therefore difficult from logistic and topographic consideration.

- Providing access to the work sites involves construction of large network of Access Roads, the most challenging job for completion of this project. In particulars the stretch between river Chenab and Banihal is passing through a virgin territory and require construct of about 200 km of access road.
- The alignment crosses deep gorges of Chenab River near Salal Hydro Power Dam, which necessitates construction of long span bridges. The Chenab Bridge, 359 m above river bed, will be the highest bridge in the world, and longest span for BG Rail line with arch span of 467 m.
- The project also involved construction of Pir-Panjal tunnel, the longest transportation tunnel of Indian Railways across PirPanjal range connecting Jammu & Kashmir provinces of J&K State. The tunnel is located between the Banihalrailway stations in South and Qazigund in NorthTotal length of the tunnel is 11.2 km with overburden of 1100 m. ***This tunnel had been completed and Section from Banihal to Quazigund opened to public by Hon'ble Prime Minister on 26.6.2013.***
- The stretch betweenKatra to Qazigund representing 128 km length is the most difficult part of this project. Almost 80% of length of this stretch is in tunnel and 10% on bridges and rest on embankment.
- Some of the special features of this stretch are:-
  - Alignment in this stretch passes through the world's one of the most difficult terrain, both in terms of logistics and geological strata.
  - Terrain characterizes sedimentary/metamorphic rocks which are yet to be stabilized.
  - *Various type of geological formation are met with in this stretch having altogether different characteristic / properties.*
  - *Alignment running across major tectonic features such as Reasi Thrust, MurreeThrust,Panjal Thrust & Local faults*
  - *The structural discontinuities occurring in the form of faults, thrusts, shears and joints are likely to pose problems in the construction activities along the rail alignment*
  - *Adverse climatic condition due to heavy snowfall in winter resulting in sub-zero temperature and reduced working period.*
  - *Many of station on this project are located on tunnel/ bridges.*
  - *World most advance and modern technology is being used for construction.*
  - ***When completed this will be a marvel of engineering with unparalleled benchmark.***

Geology along the Project Alignment