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TAI JOURNAL

Volume 9, No. 1 *January 2020*

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CONTENTS

Page No.

FROM THE PRESIDENT DESK 3

EDITORIAL 4

Articles

- Ventilation System in Large Hydro Projects in India: A Case Study of Successful Implementation of Effective Ventilation System at Tehri Pump Storage Project – *Rakesh Kumar Khali* 5
- Fibre Reinforced Sprayed Concrete for Ground Support – *E.S. Bernard and A.H. Thomas* 13
- Design & Construction of Pedestrian Access Tunnels below an Existing Operational Metro Tunnels & Station – *Senthilnath. G. T* 34
- I-System: Index of Ground-Structure Definition, Applications, and Utilisation in Design/Practice – *H. Bineshian* 43
- Micropile Foundation for Bridge Over River Shyok in Ladakh Region – First Bridge of its kind in The Countary - A Case Study– *Anil Kumar and Manish Kapil* 66
- Crossing of Tunnels between the Piles of Operational Metro Station – *A. Raj & A. Kumar* 74
- Tunneling Challenges in the Business Capital of India – *S. Vishwakarma, V.K. Thakur, U.S. Virat and K. Joshi* 79
- Tunnel Fire Protection of the Latest Generation 86

TAI News 89

Forthcoming Events 96

TAI New Members Joined 97

Latest Publications 99

ABOUT JOURNAL

TAI journal is half yearly journal of Tunnelling Association of India which publishes authoritative articles encompassing the development of innovative uses of underground space and the results of high quality research into improved, more cost-effective techniques for the planning, geo-investigation, design construction, operation and maintenance of underground and earth-sheltered structures.

The journal provides an effective vehicle for the improved national exchange of information on developments in underground technology – and the experience gained from its use – and is strongly committed to publishing papers on the interdisciplinary aspects of creating, planning, and regulating underground space.

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AIMS & SCOPE

TAI Journal is a half yearly journal of Tunnelling Association of India whose aim is to encourage planning of the use of subsurface and to promote advances in the preparatory investigations for tunnels and in the design, construction and maintenance of tunnels.

The journal publishes authoritative articles encompassing the development of innovative uses of underground space and the results of high quality research into improved, more cost-effective techniques for the planning, geo-investigation, design, construction, operation and maintenance of underground and earth-sheltered structures.

The journal provides an effective vehicle for the exchange of information on developments in underground technology and the experience gained from its use. Besides, the journal is also strongly committed towards publishing papers on the inter-disciplinary aspects of creating, planning, and regulating underground space.

The Journal shall also provide information on the related technical events in India and abroad such as conferences/ training programmes/ exhibition etc. Information related to International Tunnelling and Underground Space Association (ITA) shall also be highlighted.

The original unpublished manuscripts that enhance the knowledge and expertise in the area of design, construction of underground structures, innovative use of underground space are encouraged. The articles/technical papers are peer reviewed by editorial Board consisting of renowned experts before publication.

The journal has both print and on-line version. There is no publication charges on the author.

V.K. Kanjlia
Secretary General
Tunnelling Association of India

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&
President
Tunnelling Association of India (TAI)

FROM THE PRESIDENT DESK

Dear Colleagues,

Greetings from TAI, New Delhi.

On behalf of TAI fraternity, I wish to express our solidarity with our TAI Members and tunnel and underground fraternity during the outbreak of the coronavirus disease (COVID-19). We wish safe and healthy life to all.

Further, Tunnelling and underground construction in urban areas is increasing throughout the world for various reasons. This requires handling of complex challenges for safe construction and to minimize impacts on existing buildings, structures, utilities and the public. Tunnelling in urban areas frequently requires construction with shallow cover, in difficult ground conditions of soft soils and mixed ground, with high groundwater infiltration, frequently in close proximity to major sensitive structures and historic buildings. The potential impact of tunnelling on structures and on surface traffic, adjoining businesses and everyday life of the residents is high.

According to UN reports, the world population in April 2020 is 7.8 billion people with anticipated growth in population by 2050 to 10 billion people. There are 22 cities with population of 10 to 20 million people and 34 cities with population between 5 and 10 million people. The global trend of urbanization and industrialization has caused migration from rural areas to cities. Mega cities are growing in population and the number of mega cities is increasing. For example, Tokyo is the most populated city in the world with 32 million and Seoul, Delhi and Mexico City have populations of over 20 million each. As a result of the human growth and the concentration of populations in centralized urban areas, the need for infrastructure and transportation facilities is increasing. According to the International Tunnelling Association, the tunnel and underground market is estimated at $7.5 trillion. It is further predicted that the tunnel and underground construction market is expected to outpace other heavy civil/infrastructure markets.

Recent advancement in tunnelling technologies, improvement in safety measures, and efficiency in design and construction, implementation of better risk management and the ability to build larger diameter tunnels in more complex geology enable the construction of many newer underground facilities that were previously not possible or having high risks. Detailed geotechnical investigations, good understanding of the ground behaviour during tunnelling, and a robust design are essential for successful tunnelling in urban areas. In addition, a comprehensive instrumentation and monitoring system with predetermined threshold limits and potential remedial measures is crucial, along with pre-qualification of all involved parties.

I hope articles in this journal shall be very useful. Through this journal our attempt is to provide useful information to our readers. This would be possible if you can contribute your experience in the form of technical paper.

We are looking forward to your contributions of articles/case studies related to execution of tunnel and underground works in different strata in different parts of this world

Dr. Mangu Singh

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EDITORIAL

In recent years, lots of soft ground tunnels, including metro tunnels and road tunnels, have been constructed in many cities. Nearly all tunnels have cross passages between two tubes. Cross passage construction between twin tunnels is one of the most critical aspects of an underground metro tunnelling project. Tunnelling professionals often face a world of unpredictability. They do not need more unpleasant surprises that can arise during cross-passage construction. This includes soil, rock, water and gas conditions that are not as expected or accurately investigated. Designer and contractors often prescribe significant temporary steel support systems within the mainline tunnels during cross passage construction to help support the estimated loads and minimize deformation during break-in and break-out. Such an approach is expensive and time-intensive. A significant improvement could be realized by incorporating the required support into the liner system.

Further, Cross passage construction often requires expensive and time consuming dewatering and ground improvement to stabilize the soil. These processes require significant effort and are often a significant source of risk and cause of delay. Conventional dewatering via surface wells has been replaced in some instances by vacuum dewatering from the tunnel level and by ground freezing. Ground improvement is accomplished through jet grouting and fore poling. One common way to add predictability into cross passage construction is by modifying the ground conditions, to give tunnellers a better idea of what to expect. Often, this is achieved by drilling down from the surface and treating the ground at the location of the cross-passage or, in some other way that makes the ground easier to work. There are many kind of methods to build in cross passages in soft ground such as, cut-and-cover, mining methods, and combined method, in which the most successful and popular one, mining method with ground improved by artificial ground freezing and its risk prevention and safety protection measures, are introduced.

To focus on the above important issue, Tunnelling Association of India is planning to organize a workshop on Best Practices for safe cross passage – Investigation, Design and Construction during this year at New Delhi to create awareness about the cross passage construction between twin tunnels which is one of the most critical aspects of an underground metro tunneling project. The objective of this workshop was to update the knowledge of tunnelling professionals, i.e., Engineers, Geologists and tunnel technician to understand various issues & challenges being faced during investigations, design, construction and how they can be tackled. During the workshop, the éminent national and international ressource speakers shared their experiences of best practices in cross passages construction on different projects.

Tunnelling Association of India is grateful to authors of the various papers for their contributions included in this issue.

Through this journal our attempt is to provide useful information to our readers on tunnel and underground space development which would help them in better understanding and update their knowledge on the State of the art technology in this field. We are sure these papers will be of interest to the readers.

We also request the readers to contribute useful articles/case studies on the topic related to execution of tunnel and underground works in different strata, for this journal

G.P. Patel
Secretary General
Tunnelling Association of India &
Secretary
Central Board of Irrigation and Power

VENTILATION SYSTEM IN LARGE HYDRO PROJECTS IN INDIA: A CASE STUDY OF SUCCESSFUL IMPLEMENTATION OF EFFECTIVE VENTILATION SYSTEM AT TEHRI PUMP STORAGE PROJECT

RAKESH KUMAR KHALI
Group Project Manager, Hindustan Construction Co. Limited

ABSTRACT

Ventilation System in any large underground projects is the lifeline for the man and machinery involved in the project. The success of construction in such projects is greatly influenced by the effectiveness of ventilation systems. This paper focuses primarily on helping organizations involved in planning and execution of such projects where appropriate ventilation system for ensuring congenial atmosphere during the entire life span of the project and for carrying out periodic audit of the ventilation systems, so that remedial measures, if so required, could be undertaken well in advance for incorporating necessary modification in the ventilation system.

For this purpose, the paper provides the practical approach adopted at the Tehri Pump Storage Project (4x250 MW) for planning, and for assessment for appropriateness of the ventilation system in the light of the norms, for standard of ventilation accepted worldwide.

The author has been actively associated in planning, construction and Project management of various large hydroelectric projects, Rail & road tunnels, large size of caverns, Metros etc. in India and abroad. The paper gives an overview with regard to selection of size, type of flexible ducts, limitations & scope of axial flow fans in series, parallel, and booster operations. Steps have also been suggested for proper installations, repair and maintenance of flexible ducts for ensuring sustained effectiveness of ventilation system and for timely completion of the Tehri Pump Storage Project and its safety & economy.

1. INTRODUCTION

Construction of large tunnels and associated large cavern excavations require an effective system of ventilation along with other construction activities, with the sole objective of timely completion of construction, with due regards to safety and overall economy. The constructions of underground structures are risky and expensive. One of the main aspects concerning an appropriate design of an underground construction is the site ventilation. In other words, ventilation systems is required during the construction of any tunnel or underground excavation whether the same is constructed by blasting, boring or placing prefabricated tubes in a trench. Blasting, boring and several other activities while working on tunnels and underground caverns are followed by production of dust mist, fumes, and toxic flammable gases. These contaminants must and can only be removed by ventilation, which provides a suitable, safe working environment for the construction workers. Among other requisites ventilation system for the construction sites must be flexible to grow and move with the construction progress. It also involves rather complex flow patterns including leakage flows, booster stations, filtering, conjunctions and disjunction etc.

At Tehri PSP, most modern construction equipments have been selected to enable faster excavation with the help of a la mod, large sized equipments like dump trucks, loaders, concrete mixers etc., often with rather sufficient regard for the need to accommodate the desired system of ventilation. The objective therefore is to arrive at the appropriate size of ducts and K.W. rating of the fans specific to the layout of the proposed system of ventilation. A suitable system of ventilation commensurate with the method of construction and equipment needs to be designed, installed and maintained. The systems of ventilation should detail out, interalia the following information

(i) The type and optional diameter of ducts

(ii) The layout of the duct line

(iii) The number and position of fan.

(iv) The dynamics of performance of fans.

(v) The power cut.

(vi) The quality and strength parameter of the ducts.

Presently three major construction ventilation schemes are typically applied; use thereof depending of the site requirements:

(i) Forced ventilation – employing flexible ducts to introduce fresh air directly to the work sites.

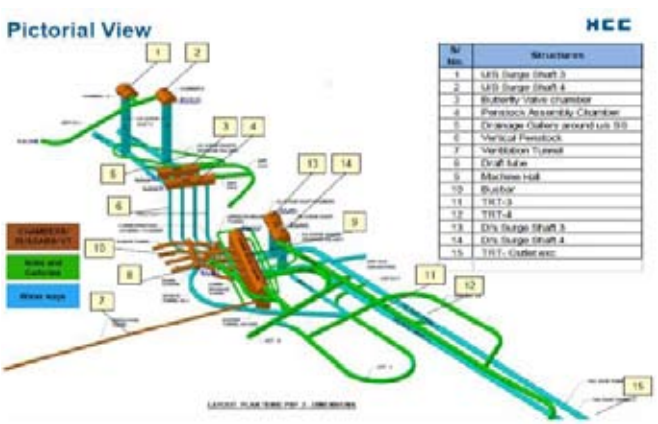
(ii) Exhaust ventilation – sucking consumed and loaded air out of the tunnel, employing reinforced flexible ducts or spiral steelducts.

(iii) Circulation ventilation schemes – mainly used in double bread projects, where fresh air is introduced via one bore and consumed air is extracted via the second.

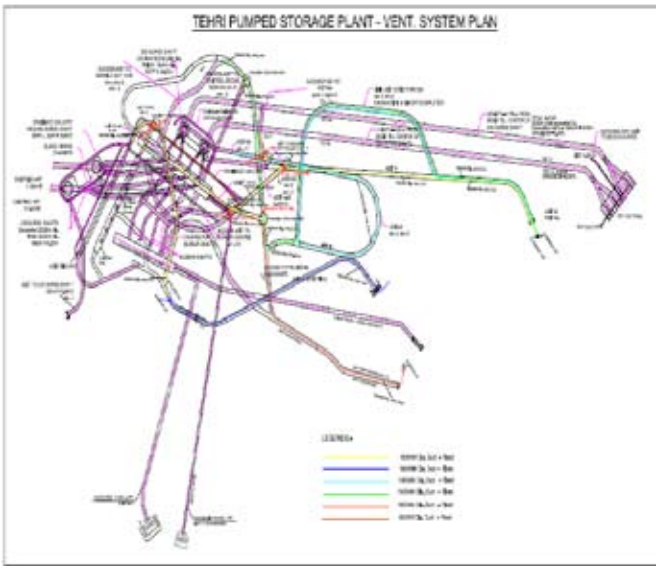
The author presents here the case study of ventilation system used for complex structures of Tehri Pump Storage Project (4X250MW), with the design detail calculations required to select fans and flexible ducting at different structurs of the project. Successful operation of ventilation system was achieved by meticulous planning, proper design for each face and by centre staging the utmost safety of the humans working inside the tunnel.

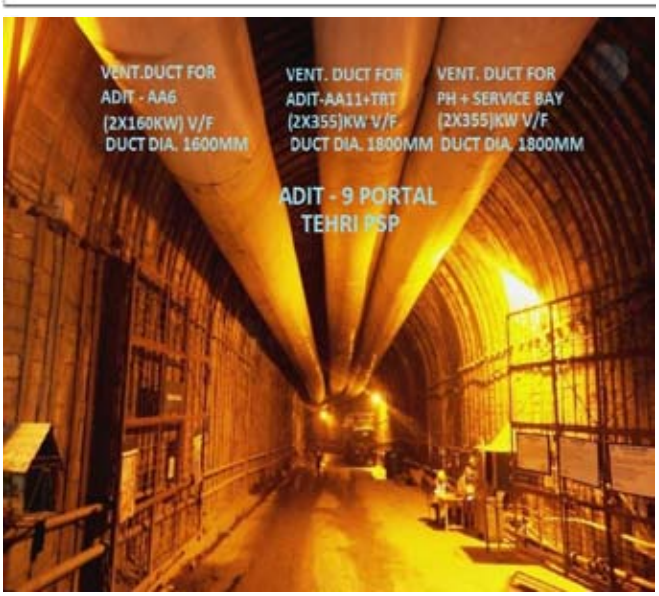
2. VENTILATION SYSTEMS FOR TEHRI PUMP STORAGE PROJECT

Layout plan of Tehri PSP (3D plan) is appended below to explain the requirement of ventilation systems.



Ventilation system used for all adits, beginning with the excavation of Adit-3 & Adit-1 is the forcing system, with the fan station located outside the portal and, forcing fresh air through flexible ducting of suitable diameter.





For Adit 1 & 3, the system was not suitable as the contaminated air discharge could affect the existing operations. As such it was necessary to exhaust the contaminated air through appropriate ducting, necessitating installation of the fans inside the tunnel at appropriate locations. These fans were planned to exhaust air from the working faces through semi rigid ducting, and forcing the air through flexible ducting to outside the portal, as shown below in Annex 5A-1, 2 ‘5A-5)

The summery of ventilation system is given below:

Sl. No.	Locations	Excavation	Fan station
1.	Adit-3 Portal	Excavation of Power House Cavern, Cavern for Busbar shaft and busbar	2 X Z V N 1 - 18-355/4
2.	Adit-9 Portal	Excavation of Drainage gallery around Power House, Access adit to service bay, Access adit to pressure shaft bottom, Adit AA7	3XZUN-18-355/4
3.	Adit-9 Portal	Adit via access route to top of downstream surgeshaft	2XZUNI-14-75/4
4.	Adit-9 Portal	Adit to bottom downstream surge shaft intermediate adit to TRT, TRT3 & TRT4	AXZVNI-18-355/4
5.	Adit -1	Adit-1 Butterfly chamber (BVC) Penstok assembly chamber(PAC), Adit to HRT 3 & 4	2 X Z U V 1 - 18/355/4
6.	Adit-AA1	Adit to upstream surgeshaft	1XZUNI-14-75/4
7.	Portal & ventilation tunnel	Ventilation Tunnel to PSP	2 Z g E L 7 - 37/2

3. SAMPLE DESIGN CALCULATION FOR VENTILATION TUNNEL FACE

(i) Fresh air requirements for working crew

1.	Maximum number of persons working at the face at one given time	20 Nos.
2.	Requirement of fresh air in cum/sec/ person as per specs	0.05
3.	Total requirement of fresh air for 20 persons in cum/sec.	1

Fresh air requirement for peak demand for Diesel equipments

Sl. No.	Equip-ment	BHP	Qty.at peak	Total BHP	Total demand BHP @ load factor 0.80 & Diversity factor 1:4	Fresh air@ 0.1cum/ sec / BHP
1.	Wheel Loader	215	1	215	86	8.60
2.	Dumper	120	3	360	86.40	8.64
						17.24

Total fresh air requirement at the face for the working crew and equipments at peak time 18.24 cum/sec

(a)	Return air velocity for 18.24 cum/sec of fresh air for (18.89-0.5=18.49) sqm of Tunnel	1.08 m/ sec
	Since the air velocity of 1.59 m/sec is harmful for workforce, working inside the tunnel, the fresh air demand of 26.88 cannot be met. Hence the scheme is to be designed on the basis of maximum allowable air velocity i.e 0.6 m/sec	
	Fresh air requirement on the basis of permissible velocity as per specs	
(b)	Cross-sectional area of excavated tunnel	18.89 sqm
(c)	Return air velocity as desired as per specs	0.4 m/ sec
(d)	Requirement of fresh air to achieve this velocity	7.556 cum/sec
	Fresh air requirement on the basis of velocity of (06m/sec)	
	Cross-section of excavated tunnel (44-2)	18.89 sqm
	Maximum return-air velocity permissible	0.4m/ sec
	Requirement of fresh air to achieve the velocity	7.556 cum/sec
	Hence for velocity of 0.6m/sec, the fresh air requirement will be 7.56 cum/sec	
	Losses due to leakage etc.	15%
	Total air to be pumped at portal end	8.7 cum/sec 9 cum/ sec ..A

(ii) Pressure drop calculations

Proposed dia of vent duct	800 mm
Type of air duct	flexible
Air volume to be pumped	9.0cum/sec
Total air volume to be pumped at portal	
considering 15% leakages/losses	10.4 cu/sec
Length of air duct	500 m
Pressure drop per meter of duct of 800 mm dia	
for 10.4 cum/sec	0.115 mmwg
Total pressure drop for length 500m of duct	57.5 mmwg
Droop due to leakage etc	15%
Hence total pressure drop	66.13
Total pressure regd at portal	4800……B

Hence proposed fan with the capacity (KW) = AXB/(CX1000)

Where c is efficiency of motor

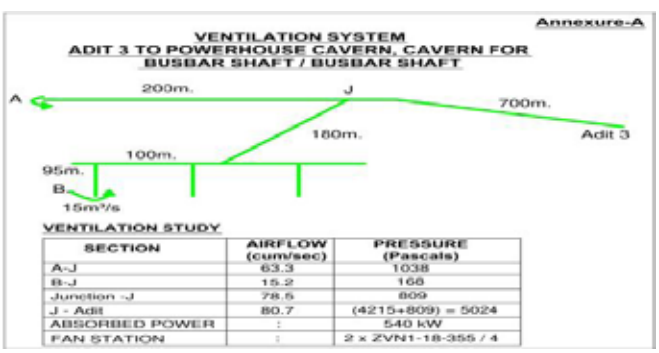
Delivery 9cum/sec@pressure 66.125 mmwg

Proposed fan capacity = 2×37.5kw considering C is 75%

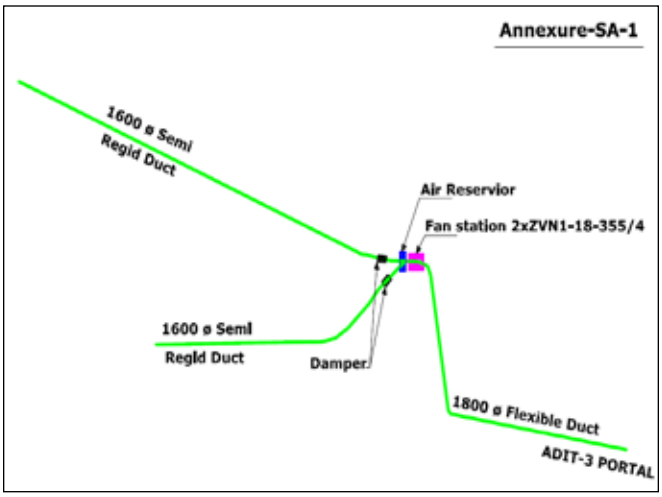
4. VENTILATION DESIGN FOR DIFFERENT STRUCTURES OF TEHRI PSP

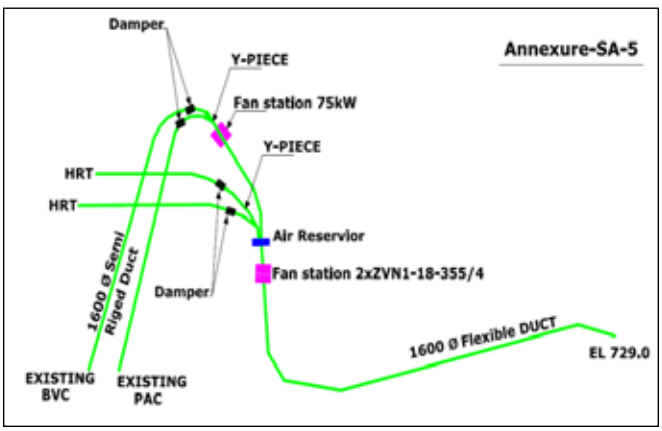
4.1 Excavation of Powerhouse Cavern, Cavern for Bus Bar Shaft, and Bus Bar Shafts.

The ventilation system would consist of a 200m long duct line of semi-regid ducting 18000mm dia across the Power House Cavern to the junction; and semi regid ducting of 380m,which should be installed in one Bus Bar Shaft and Cavern for Bus Bar Shaft and Cavern for Bus Bar. The station comprising two fans ZVN1-18-355/4 should be installed near the junction. This would exhaust the contaminated air from the working faces and force it out though the 1800mm dia flexible ducting to outside portal Adit 3. The ventilation study is detailed in Annexure 'A'



Accessories, e.g., Y – Pieces will need to be installed as shown in Annexures – (SA-1) & (SA-5) to regulate the air at the individual working faces.





The flexible ducting from portal of Adit 3 to the Powerhouse is 1800 mm dia, whereas from the junction Adit 7A to Cavern for Bus Bar Shafts, and Bus Bar Shaft is 1600 mm dia.

Initially the heading excavation of 7m height will be undertaken in the Powerhouse Cavern.

The Cross section of the Heading will be = 22X7 = 154m^2

Considering a return Air Velocity (RAV of 0.4m/sec), the air flow for the Heading face will be = 154 x0.4=61.6m^3/s (say 62m^3/sec)

For the Bus Bar Cavern and Bus Bar Shafts, the Cross section will be 37.72sq. mtrs & considering RAV of 0.4m/s, the Air Flow at the working face will be 15m^3/sec

The Fan Station at he portal will have the following configurations:

Air Flow at Fan	- 80.7m
Total pressure	- 5025 Pascal
Estimated absorbed power	- 540 kW
Fan Station	- 2 x ZVN 1-18-355/4
Installed Power	- 710 kW

4.2 Excavation of Drainage Gallery around Powerhouse, Access Adit to Service Bay, Access Adit to Pressure Shaft Bottom, Adit AA7

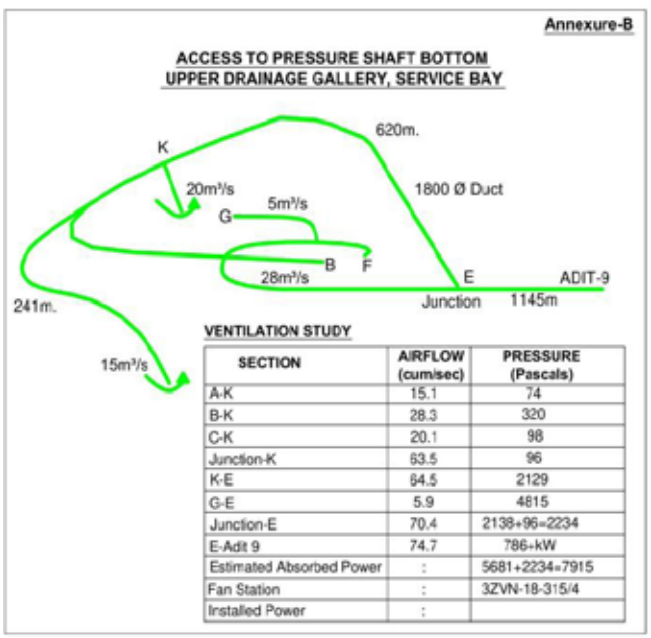
The flexible Ducting of 1800mm dia is used throughout the excavation, except for the drainage gallery where a duct of 600mm dia is considered.

The Drainage Gallery has a Cross Section Area of 11.7 sq. mtrs, considering on RAV of 0.4 m/s. The Air Flow at the face will be 11.7 x 0.4 = 4.68 m^3/s (say 5m^3/sec)

The Adit to the pressure shaft bottom has a section of 67.7 sq.mrs. x 0.4 = 27.08 m^3/sec 9say 28 m^3/sec)

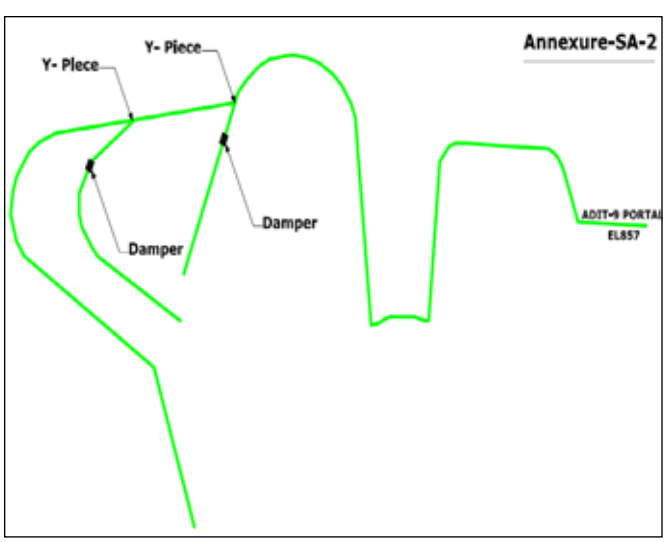
The Adit AA7 has a Cross-Section of 37.7sqmts, and at an RAV of 0.4 the air flow at the face will be 37.7 x 0.4 = 15.08 m^3/sec (say 15m^3/sec)

The detailed design is given in Annexure – B below.



VENTILATION STUDY

SECTION	AIRFLOW (cum/sec)	PRESSURE (Pascals)
A-K	15.1	74
B-K	28.3	320
C-K	20.1	98
Junction-K	63.5	96
K-E	64.5	2129
G-E	5.9	4815
Junction-E	70.4	2138+96=2234
E-Adit 9	74.7	786+kW
Estimated Absorbed Power	:	5681+2234=7915
Fan Station	:	3ZVN-18-315/4
Installed Power	:	



The schematic arrangement is shown is Annexure –SA-2 above.

The Fan Station at the portal will have the following configurations:

Air flow at Fan	- 74.7m^3/s
Total pressure	- 7915 Pascals.
Estimated Absorved Power	- 786 kW
Fan Station	- 3 x ZVN-1-18-355/4
Installed Power	- 1065 KW

4.3 Adit via Access Route to the top of the Downstream Surge Shaft

The Flexible Ducting from the Adit 9 Portal up to Chainage 700m is 1800mm dia and thereafter 1600 mm dia.

The heading excavation will be done for the full cross-section up measuring to 7m height.

The Cavern Heading Volume is 13090 m^3

Volume of Tunnel = 940 x 60.78 = 57133 m^3

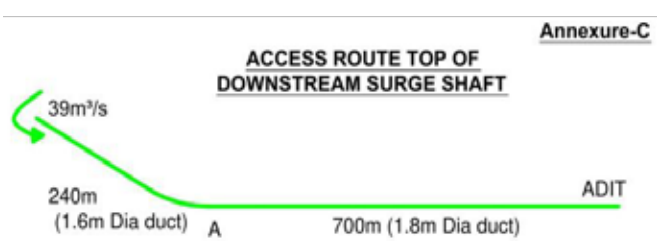
Total volume = 70223 m^3

A 2 changes per hour the air flow at the face = 72223 x 2/3600 = 39m^3/sec

The Fan Station will have the following parameters.

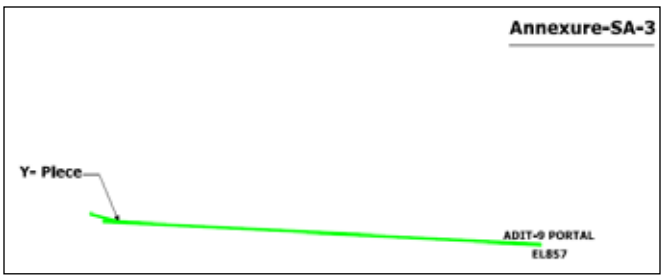
Air flow at fan	- 40.6 m^3/sec
Total pressure	- 2734 Pascal
Estimated absorbed Power	- 148 kW
Fan Station	- 2 x AVN 1-4-75/4
Installed Power	- 150 kW

The detailed design is given in the Annexure – C below.



VENTILATION STUDY

SECTION	AIRFLOW (cum/sec)	PRESSURE (Pascals)
A-B	39.3	74
A-Adit	40.6	320
Estimated Absorbed Power	:	98
Fan Station	:	96
Installed Power	:	2129



The schematic arrangement is shown in Annexure –SA-3 above.

4.4 Adit to Bottom Downstream Surge Shaft, Intermediate Adit to TRT, TRT3, & TRT4

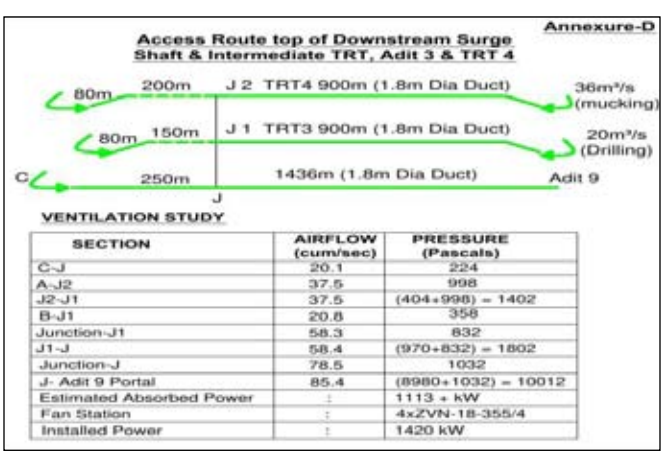
The Flexible Ducting from Adit to the junction of Adits, to Downstream Surge Shaft & Adit to intermediate TRT, will have flexible ducting of 1800 mm dia. The Ducting from the junction to Downstream Surge Shaft Cavern will be 1600mm dia. The flexible Ducting in the TRT3 & TRT4 will be 1600 mm dia.

It is assumed that the TRT3 & TRT4 will be excavated from the Access Adit for the access to TRT up to the outfall, and then towards the downstream Surge Shaft, one face is left for mucking and the other fordrilling.

The air flow at the downstream surge shaft face has been considered at 20 m^3/sec (slightly higher RAV is calculated)

An RAV of 0.4 m/sec has been calculated during mucking operations for the full TRT Cross-section. The calculated air flow is 90 x 0.4 = 36 m^3/sec.

For drilling operations the air flow at the face has been calculated as 20 m^3/sec. the detailed design is given in Annexure – D below.



VENTILATION STUDY

SECTION	AIRFLOW (cum/sec)	PRESSURE (Pascals)
C-J	20.1	224
A-J2	37.5	998
J2-J1	37.5	(404+998) = 1402
B-J1	20.8	358
Junction-J1	58.3	832
J1-J	58.4	(970+832) = 1802
Junction-J	78.5	1032
J- Adit 9 Portal	85.4	(8980+1032) = 10012
Estimated Absorbed Power	:	1113 + kW
Fan Station	:	4xZVN-18-355/4
Installed Power	:	1420 kW

The Fan Station has the following parameters:

Air Flow at Fan	85.4 m^3/sec
Total pressure	10012 Pascal
Estimated absorbed Power	1113 kW
Fan Station	4 x ZVN 1-18-355/4

4.5 Adit 1 to Butterfly Valve Chamber (BVC), and Penstock Assembly Chamber (PAC), Adit to HRT 3 & 4

The schematic arrangement has been shown in Annexure –SA-5 above and the system design is given below:

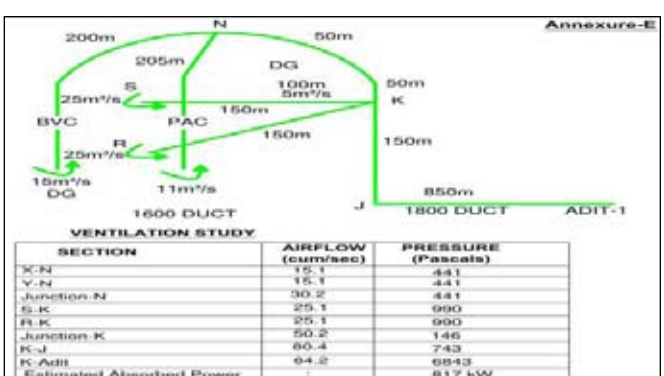
The fan station comprising 3 nos. ZVN1-18-355/4 in series are located at the junction of the two duct lines as shown in the layout drawing. It will be necessary to install an air tank fabricated with G.I sheets with two inlet duct adapters of 1800 mm dia and an outlet duct adapters of 1800 mm dia as shown in Annexure 'C' above.

It may be necessary to install an intermediate Fan of 91800 mm dia x 75 kW) in line as shown in Annexure, if required to improve the exhaust ventilation for the working faces of the air tank.

The Air flow based on the RAV of 0.4 m/sec are for the various faces is given below:

Ball Valve Chamber	- 15 m^3/sec
Penstock Assembly Chamber	- 11 m^3/sec
Adit to HRTs 3&4	- 25 m^3 /sec each.

The ventilation studies have been detailed in Annexure – E below.



VENTILATION STUDY

SECTION	AIRFLOW (cum/sec)	PRESSURE (Pascals)
X-N	15.1	441
Y-N	15.1	441
Junction-N	30.2	441
S-K	25.1	990
R-K	25.1	990
Junction-K	50.2	146
K-J	80.4	743
K-Adit	84.2	6843
Estimated Absorbed Power	:	817 kW
Fan Station	:	3xZVN1-18-355/4
Installed Power	:	1065 kW

The Fan Station parameters are as under:

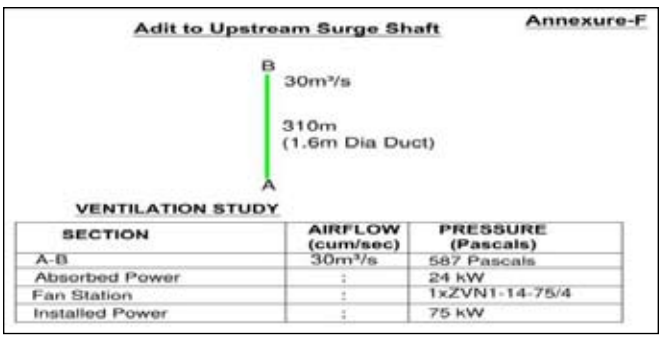
Air Flow at Fan	- 84.2 m^3/sec
Total pressure	- 6843 Pascals
Estimated Absorbed Power	- 817 kW
Fan Station	- 3 x ZVN1-18-355/4
Installed Power	- 1065 kW

4.6 Adit AA1 to Upstream Surge Shaft

The Ducting from Adit AA! To the end of the Surge Upper Chamber is 310m long. The Flexible Ducting is 1600mm aid throughout. The excavation from the Adit will continue to the Upstream Surge Shaft where initially the heading of the full width will be done.

The Cross Section of the heading area is 11 x 6.5 = 71.5 sq.mtrs. with an ARV of 0.4m/sec the air flow at the face will be 71.5 x 0.4 = 28.6 (say 30 m^3/sec)

The ventilation design is appended in Annexure – F below.



The Fan Station has the following parameters:

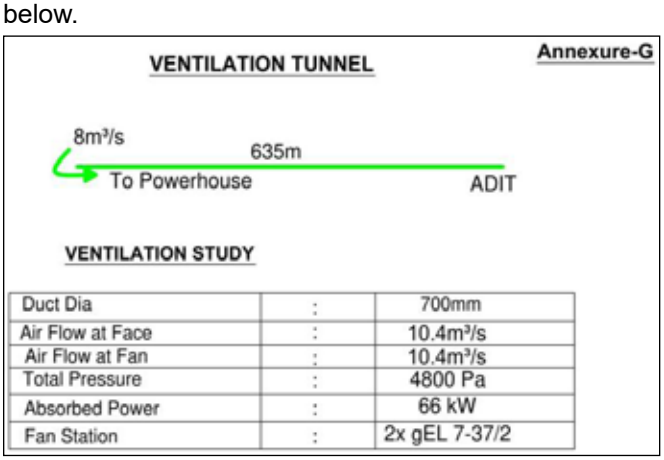
Air Flow at Fan Station	- 30.3 m^3/sec
Total pressure	- 587 Pa
Estimated Absorbed Power	- 27 kW
Fan Station	- 1 x ZVN 1-14-75/4
Installed Power	- 75 kW

4.7 Ventilation Tunnel to PSP

The Flexible Duct is 700 mm dia (Which is the maximum size that can be accommodated in the tunnel of 4.6m D-shaped)

The Cross Section of the Ventilation Tunnel is 18.89 sqmts. and considering a RAV of 0.4/sec. the air flow at the face = 18.89 x 0.4 = 7.56 (say 9 m^3/sec)

The ventilation design is appended in Annexure – G below.



The Fan Station has the following parameters.

Air Flow at the fan	10.4 m3 /sec
Total Pressure	4800 Pascal
Estimated Absorbed Power	66 kW
Fan Station	2 x gEL-7-37.5/2
Installed Power	75 kW

5. REPAIR, CARE & MAINTENANCE OF FLEXIBLE DUCTS

In case, flexible ducts are damaged, as it is. Sometimes-unavoidable due to deficient operations, the higher leakage of air further compounds the problem, leading to higher power-cost and consequent reduction in the air-flow at the face. It is, therefore, essential to repair the flexible ducts as soon as these are damaged due to myriad reasons in course of construction, mostly by the mobile equipment. Such damages can be rectified with the help of either Cold or Hot Repairing Kits. “A stitch in time saves nine” is an apt idiom to ensure sustained and satisfactory standard of ventilation. Proper arrangements were made at Tehri PSP for repairing the ducts with cold and hot repairing kits.

5.1 Cold Repairing Kit

Damaged part of the duct should be properly cleaned, and the slit / cut should be loosely stitched with nylon thread. Apply a thin layer of Vinyl adhesive on and around the damaged part, allow it to dry for 10 seconds and place the freshly cut and sized extra piece of PVC cloth strip over the damaged part, and paste its top with the transparent adhesive tape duly supplied with Cold Repair Kit.

5.2 Hot Repairing Kit

Hot repairing does not require any adhesive. Bigger cuts can be repaired either by using a Hot Air Gun or electrically operated hand-held Welding Iron. For this purpose, the duct should be dismantled and brought outside the tunnel, as it requires a flat firm surface to work upon. After due cleaning, the Welding Iron is switched-on and allowed to be heated up-to a temperature just sufficient to soften the PVC coating. The tip of the Welding Iron is sand-witched between the damaged part and freshly cut / sized strip of PVC coated fabric and the Welding Iron Is gradually moved, hand-held, while being pressed under with the help of a rubber-roller. Care should be taken to ensure that the hot tip of the Welding Iron is not too hot to burn the fabric.

6. AUDIT OF VENTILATION SYSTEM

As the faces advance (especially in case of long tunnels), it is not uncommon to notice gradual decline in the standard of ventilation, mainly due to improper installation of ducts, leakage through damaged parts of ducts, ill designed ventilation system etc. It is therefore, necessary to carry out regular audit of the ventilation system of the tunnels, by undertaking actual measurements of the volume flow. A team of repair and maintenance deputed under the leadership of Project Electrical Engineer in the project to conduct regular audit and take appropriate action in removing the bottle necks after physical verification of the duct-line on regular basis.

7. CARE & MAINTENANCE OF FLEXIBLE TUBINGS

In order to ensure effective system of ventilation, especially while constructing complex underground structures like Tehri PSP, efforts were made and adequate precautions taken for smooth flow of air through the Flexible Ducts, right from the beginning, i.e., from the fan installed at the portal up to the face. It is therefore absolutely essential to take following precautions while installing flexible ducts.

(i) Straightness of duct-line is essential for better results. Sudden deviation should be avoided.

(ii) The joint rings should be properly "strapped-on" at the joints and the tension bolts should be properly tensioned. Coupling should be strapped on the duct in such a manner that all the tension bolts are on the one side of the duct-line to facilitate subsequent examination / tightening whenever required.

(iii) The ducts, after extension, should be stretched fully to avoid any formation of kinks.

(iv) Damaged ducts should be repaired with the help of Cold / Hot Repair Kits or should be replaced immediately.

(v) Wires and nails should never be used for repairing.

(vi) Undue sag of the duct-line should be avoided by securing the duct-lines with the help of "vertical hangers" at every 3 m to 4 m interval.

(vii) Vertical hangers should be used only for supporting overhead Head-Wire-Rope Suspension Assembly and not to support ducts directly from it's suspension hooks.

(viii) The haul-roads should be kept clean of any fallen debris so as to avoid reduction in the clearance below the duct-line, and to avoid damage to the ducts.

(ix) In case of vertical / inclines shafts, flexible–ducts should be securely fastened to the wire-rope-suspension assemblies.

(x) A minimum distance of 8 m should be kept between the fan and portal to avoid re-circulation of air.

(xi) A dequate protection should be provided at the suction end of the fans installed at the portal to avoid any entry of rainwater / foreign material into the duct-line.

(xii) Special team of persons should be trained and dedicated for installation, repair and maintenance of the duct-line. Periodic inspection of the duct-line should be carried out by a competent person, especially authorized for this job and necessary remedial action should be undertaken to keep the duct-line in proper working condition

8. CONCLUSION

Selection of ventilation systems in a project like Tehri Pump Storage Project is really difficult to connect with various structures while executing work simultaneously at all fronts. Lot of brainstorming was done with the representatives of leading ventilation companies and manufacturers of fans. And after a detailed planning; proper ventilation system was designed and adopted successfully at this project. The Tehri PSP is in advance stage of construction and most of the excavation work is in completion stage. Since the planning was perfect, successful completion of excavation of major structures like Powerhouse, Control Block, Bus bar tunnels, Butterfly valve chamber, Upstream and Downstream chambers and Tail Race Tunnels was meticulously accomplished. This was possible due to selection of effective ventilation system-the lifeline of the project. Every care was taken to keep our people safe by providing all PPE's, proper lighting and good ventilation system synergized with an essential periodic health check of workers.

Tunnel workers persistently suffer from the fear of claustrophobia inside the tunnel and are constrained from working with their full potential. Assurance of a foolproof ventilation gives them the confidence and fearlessness to work with their full efficiency of labour. In the Tehri Pump Storage Project, installation of the ventilation system has achieved substantial targets and the work has progressed with sustained felicity and speed.

FIBRE REINFORCED SPRAYED CONCRETE FOR GROUND SUPPORT

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1. INTRODUCTION

This document provides guidance on the use of Fibre Reinforced Sprayed Concrete (FRSC) for ground support and related uses in an underground environment. It provides essential information for users of fibre as reinforcement in sprayed concrete with an emphasis on when and how fibres can be useful. It has been written for designers and owners of underground infrastructure as a summary of the essential points to consider when appraising the use of FSRC. It is intentionally brief and discusses only the essential aspects of fibre usage in sprayed concrete, but numerous references are provided for interested parties to pursue, if more detail is required. The application of FRSC for underground support covers a broad scope since many different types of ground condition exist (soft, hard, plastic, squeezing, blocky, etc.) and FRSC can be used for both short and long term stabilisation. In many types of competent ground FRSC linings are used for the purpose of ground support, protecting the ground from deterioration by weathering processes, or limiting moisture gradients or evaporation at the surface (Fig. 1). It may also be included for the purpose of channeling water ingress, smoothing the surface of a tunnel for air-flow and waterflow requirements, or encapsulating structural members for fire protection. Fibre reinforced sprayed concrete linings have been found to be a highly adaptable and useful means of ground control, but it is not the only method of ground control available. Other processes, such as mesh reinforced sprayed concrete, cast-in-place linings and pre-cast segmental concrete linings, may and often do present superior solutions for certain applications.

There are several reasons why fibres are used in sprayed concrete. The primary reason is to provide post-cracking reinforcement to span tensile cracks which in turn helps to control crack widths and imparts toughness to what otherwise would be a brittle material. Unreinforced concrete is brittle because it has a low tensile strength compared to its compressive strength and offers little resistance to crack propagation. Providing toughness is one of the main reasons for inclusion of fibres in many ground support applications, but additional reasons exist. Microsynthetic fibres are used to limit explosive spalling in fires and can also provide advantages in terms of limiting fallouts and sloughing of very young sprayed concrete.

There are generally three types of fibre used in sprayed concrete: steel fibres, macro-synthetic fibres, and micro-synthetic fibres. There is a comprehensive resume of the mechanical properties of the fibres used for concrete reinforcement in ACI 544.5R-10, Report on the Physical Properties and Durability of Fiber-Reinforced Concrete. The general performance requirements of structural fibres are discussed elsewhere (e.g. ITAtech, 2016) so only brief details will be presented here. In general, it must be considered that the bond and friction characteristics between the concrete and fibres, as well as fibre mechanical properties, will influence the performance of the hardened FRSC. Both steel and macro-synthetic fibres are relatively large discrete fibres between 18 and 65 mm in length and around 0.3–2.0 mm in equivalent diameter. These types of fibre are used to impart structurally useful, post-cracking tensile strength to cracked sprayed concrete. In contrast, micro-synthetic fibres are much smaller, typically of 6–20 mm length and around 0.018–0.30 mm equivalent diameter and are usually used to impart spalling resistance during fires but are otherwise considered structurally ineffective. For cold-drawn wire steel fibres, ITAtech (2016) recommends a minimum tensile yield strength of steel wire of greater than 800 N/ mm^2 when the concrete class is less than or equal to C40/50 (f_{ck} = 40 N/mm^2) and a yield strength of greater than 1500 N/ mm^2 for stronger concretes. The mean tensile yield strength for macro-synthetic fibres should be greater than 500 N/mm^2.

The maximum length of steel fibre that can be used in sprayed concrete is equal to about 2/3 of the nozzle diameter; the maximum length of macro-synthetic fibre that can be used in sprayed concrete is approximately equal to the nozzle diameter (ACI 506, 2016). The majority of FRSC used in tunnels is produced and sprayed using the wetmix process on account of the relatively low productivity and high levels of rebound and dust normally seen with the dry-mix process. However, wet-mix sprayed concrete production can represent

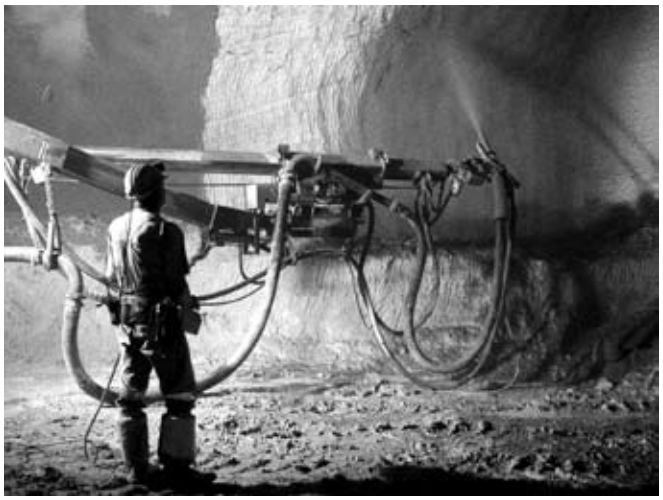

Fig. 1 : Remotely-controlled spraying in an underground environment for ground support.

a logistical challenge in remote location tunnels or in deep mines; in such cases, the flexibility possible through transporting and storing prebagged dry-mix sprayed concrete material underground prior to its usage represents an advantage. Most macro-synthetic fibres are composed of polypropylene (Bernard et al, 2014), but micro-synthetic fibres can be composed of either polypropylene, nylon, PVA, glass, or a number of other polymers. Only polypropylene micro-synthetic fibres are effective for control of explosive spalling in fires. International standards such as ISO 13270, EN 14889 (Parts 1 & 2), ASTM A820, and ASTM D7508 set out the classification of fibres, according to size, composition, and other parameters.

2. BEHAVIOUR OF FIBRE REINFORCED SPRAYED CONCRETE LININGS

The behaviour of fibre reinforced sprayed concrete linings depends on the age of the lining, geometric aspects of design, ductility of the fibre-concrete composite, and interaction with the ground. Fibres are critical to ductility, which indirectly affects interaction between the lining and the ground by facilitating a re-distribution of load away from cracked regions toward more structurally competent regions. The main means by which FRSC supports hard ground is by providing restraint to the surface of the ground thereby preventing progressive unravelling. In soft ground, it usually acts as a supporting arch. The following provides a basic overview of the behaviour of FRSC linings at early and mature ages in both hard and soft ground conditions.

2.1 Early-age behaviour of FRSC linings

The ability of a FRSC lining to stabilise ground at early ages (say, less than 24 h) will depend on numerous factors, including the following:

(1) *Bond*. The effectiveness of FRSC at early ages depends on bond development to the ground and strength development within the newly sprayed lining. In hard rock environments, the early-age bond between the sprayed concrete and ground will often depend on whether the ground has been adequately prepared through washing and scaling prior to the sprayed concrete being applied (Clements et al, 2004; Bryne et al., 2013). Fibres have no effect on bond strength either at early or late ages. Any loose material on the surface of the excavation that could reduce the bond between fresh sprayed concrete and the ground must be removed before spraying. Similarly, water ingress through the rock surface during spraying, or suction due to excessive dryness, can compromise bond and lead to fall-outs of slabs of rock and sprayed concrete (Kuchta, 2002; Shotcreting in Australia, Recommended Practice, 2010; ACI 506, 2016). Some types of rock, such as schist or shale, are friable or layered and can be very difficult to get sprayed concrete to bond to since failure within rock layers can lead to fall-outs. Similarly, certain types of soft ground, such as sandy layers, can be very difficult to get sprayed concrete to bond to since failure within ground layers can lead to fall-outs. A geotechnical assessment of the ground is generally required before it is possible to prescribe the most suitable approach to ground support and spraying.

(2) *Strength*. The ability of a young lining to stabilize the ground depends critically on the development of compressive and shear strength over the first few hours (Chang and Stille, 1993; Morgan et al., 1999a,b; Golser, 1999; Bernard, 2008). Success in stabilizing the ground will therefore depend on whether the rate of increase of strength in the concrete mix is adequate, and whether each particular batch of concrete has been properly prepared so that the potential performance is achieved under operating conditions. Errors in batching or spraying can lead to inadequate performance, primarily low strength or residual strength, so Quality Assurance processes are normally prescribed to ensure such errors are detected and corrected. These procedures must be adhered to meticulously so that reliable performance is achieved. Fibres contribute sprayed concrete (Bernard and Hanke, 2002), so errors in the batching of fibres may not become apparent until later in the life of a FRSC lining.

(3) *Thickness*. FRSC linings in soft ground tend to be more than 200 mm thick so the sprayed concrete lining must be built up in several passes. Each layer must gain enough strength to support its own weight as well as any ground loading before the next layer is applied. To minimise the risk to operatives posed

by fall-outs of fresh sprayed concrete, an exclusion zone must be imposed around the spraying area until it is safe to work under it (Crossrail, 2014). There are various ways to check that the correct minimum thickness have been sprayed, varying from thickness pins and tell-tales on steel arches to laser surveying methods.

(4) *Workmanship*. Effective ground control with sprayed concrete also depends on the quality of spraying (Göransson et al, 2014). Good bond development to the rock depends on many factors, including the ability of the nozzle operator to apply the sprayed concrete properly to ensure that the composition and thickness are uniform and meet minimum performance requirements (Larive, 2009). In addition, excessive set accelerator must not be deposited on the rock before the sprayed concrete as this will seriously compromise bond. Proper encapsulation of bars, if present, is a challenge that requires attention to quality spraying technique and suitable properties in the mix (Beaupre and Jolin, 2001; Basso et al, 2016), but the quality of spraying can also affect the properties of fibre reinforced sprayed concrete in the absence of bars (Jolin et al., 2015a,b). Competent workmanship is essential to effective ground support and safety with sprayed concrete with guidance available from many sources (Lehto et al, 2012, Jolin et al., 2015a,b; ACI CP-60-09, EN 14487-2).

(5) In-place Performance. Low temperatures can reduce the rate of hydration of concrete at early ages, and thus the rate of strength gain (Dufour and Beaupre, 1999), making it necessary to either heat the concrete prior to application or wait for an extended period of time before it is safe to re-enter under freshly sprayed ground (Dufour, 2000; Atkinson and Martin, 2007). Similarly, some types of cement and Supplementary Cementitious Materials (SCM) gain strength quite slowly and require longer waiting times than is normal for pure cement mixtures. Test methods such as penetrometers, bolts (EN 14488-2) and ASTM C116 beamlets (Morgan et al., 1999 a,b, Bernard and Geltinger, 2007) are commonly used to assess the early-age strength of a mix and determine how long it takes to reach the minimum accepted strength before re-entry under newly sprayed ground is permitted.

Generally a compressive strength of 1.0 N/mm^2 is required to permit the safe re-entry of personnel into a newly sprayed area (ÖVB, 2013; Bernard, 2008). This can take 1–12 h or more to achieve and is affected by numerous factors including mix design, accelerator use and ambient temperatures and therefore must be measured as mentioned above. However, machines with protective canopies are often permitted to re-enter and begin, for instance, bolt installation, once compressive strength has reached 0.6 N/ mm^2.

(6) *Ground Conditions*. Ground stresses due to large-scale instability or squeezing conditions (Tyler and Clements, 2004), the inherent weakness of the ground (as noted above) or excessive wetness due to water ingress, can make it very difficult for a freshly sprayed lining to stay in place as it hydrates. Special measures, such as prior installation of mesh to act as an anchor for the young FRSC, may be required to prevent the concrete falling out before it has gained sufficient strength, whilst dedicated drainage measures would be required to reduce the risk of washing out fresh sprayed concrete or damaging its final quality.

(7) *Load Transfer*. The primary mechanism for load transfer between the ground and the lining – and in fact the only mechanism in soft ground – is the bond and ground pressure between the sprayed concrete and the ground. Bolts are generally drilled through the sprayed concrete layer in a pattern predetermined by the geotechnical engineer based on assessment of the ground. Bolt cover plates of sufficient size must be placed over the top of the sprayed concrete and held in place with fasteners. Load transfer between the bolts and the sprayed concrete lining is a critical element of the ground support design in rock tunnels. Shear transfer between a delaminated lining and the bolts may be inadequate, if small or no plates are used, possibly leading to fallout of the lining. Plates placed on the rock face under the sprayed concrete are usually ineffective in transferring load between the lining and bolt. In cases where a plate must be placed under a lining (usually for reasons of durability) a 'spider', 'bow tie', or 'elephant's ear' arrangement of reinforcing bars should be included around the bolt head to provide shear transfer in the event of significant lining movement (ACI 506; Pells, 2002).

2.2 Mature FRSC lining behaviour

As the sprayed concrete matrix hydrates, several changes in behaviour occur. The compressive strength increases rapidly. The shear resistance increases dramatically, thereby reducing the likelihood of shear failure. The bond to the rock also increases, but not at a dramatic rate. Most importantly, the fibres become more effective due to increased friction between each fibre and the concrete matrix during pullout. This last phenomenon has the effect of increasing post-crack performance in tension. The effect of these changes on the behaviour of a FRSC lining is manifested as follows:

(1) *Compressive Capacity*. The compressive strength of the sprayed concrete increases as the lining ages but is largely unaffected by the presence of fibres.

The load resistance of the lining in compression increases substantially over the first few days after spraying, which is useful if convergence is significant or the stand-up time is short because this support can promote compressive arching action in the ground. However, due to the unevenness of loading in most hardrock tunnels, and the irregular geometry of most linings, arching action across several bolt spans is only an effective means of structural resistance for thick linings (typically 150+ mm) of favourable geometry for which the span/thickness ratio is less than about 30. In soft ground the lining often relies on rapid compressive strength development to maintain support to the ground since the loads may increase rapidly as the tunnel advances (Duddeck and Städing, 1990; Röck et al, 2012). In some types of ground condition, there may also be longer term changes in loading due, for example, to creep of the ground or the recovery of pore fluid pressures. Creep of the sprayed concrete may help to even out concentrations of load and thereby reduce peak bending moments over time.

(2) *Shear Capacity*. As FRSC matures, the shear resistance increases dramatically. However, most cracked FRSC exhibits deflectionsoftening behaviour in shear, thus the post-crack shear resistance of FRSC linings is almost always lower than the shear resistance of an uncracked lining (Mirsayah and Banthia, 2002). Load re-distribution is also poor around shearing zones due to limited alternative load paths, thus fibres cannot be considered an effective means of augmenting the shear resistance of a lining. Fortunately, the shear resistance of concrete is substantial, especially when a compressive membrane stress is present, so shear failures are rare in mature FRSC linings.

(3) *Toughness*. The progressive development of the post-crack tensile performance of fibres is manifested most directly as an increase in the post-crack bending resistance of the lining (Banthia and Trottier, 1994; Bjøntegaard et al 2014; Bernard, 2015a). The post cracking load bearing capacity provided by the fibres assists in the re-distribution of load around areas of ground instability, while at the same time restraining movement of the ground, thereby allowing the ground to more effectively support itself. The fibres also help to prevent fall-outs of fragments of a sprayed concrete lining should cracking become extensive. Bolts constitute the primary means of global ground support around underground hard rock excavations and thin-walled FRSC linings for hard rock are only intended to provide support between bolts. The majority of FRSC linings in hard rock must therefore be considered as being limited to local support of the ground. Only when linings are sprayed as thick arches of suitable geometry, as is typically done in soft ground, can sprayed concrete be considered to act as global ground support around an excavation (BTS, 2004; Grasberger and Meschke, 2003).

(4) *Ground Interaction*. The bond between the ground and the sprayed concrete and its progressively stiffer support offers an excellent form of support to both soft ground and hard rock. Unlike some other ground control methods described below, FRSC promotes a beneficial ground-structure interaction which can be designed to maximise the load bearing function of the ground. This interaction is most effective when sustained adhesion exists between the lining, including bolts, and rock (Banton et al, 2004; Schmidt et al, 2015). Unfortunately, adhesion has been found to progressively deteriorate in some types of ground (such as sandstone), even if best practice is adopted during ground preparation and spraying, thereby leading to possible delamination of linings with the passing of time (Grønhaug, 1996). This may be caused by shrinkage of the sprayed concrete, or hydrological changes in the rock behind the lining, or progressive crack formation in the rock around the excavation leading to fragmentation of the ground (Hoek et al, 1998; Stacey, 2001). This applies equally to fibre and traditional mesh-reinforced sprayed concrete linings. If delamination of a lining occurs, tensile reinforcement at cracks and load transfer between the lining and bolts (Pells, 2002) become particularly important to structural stability.

2.3 Wet compared to dry process sprayed concrete

Either process can produce sprayed concrete suitable for normal construction requirements. Differences in capital and maintenance cost of equipment, operational features, suitability of available aggregate, and placement characteristics may make one or the other more attractive for a particular application. The typical advantages cited for the selection of the dry-mix process are: it is easier to handle the delivery hose due to its lighter weight, and the simplified logistic requirements possible when working with pre-bagged material that can be transported and stored in advance of the spraying work. For the wet-mix process, the advantages most commonly reported are: the relatively lower dust and rebound losses, the higher production rates, and more consistent Quality Control. Adequately controlling the set accelerator dosage rate can be a challenge for both methods of application.

Consequently, the wet-mix process is often favoured for large volume application of FRSC since the lower rebound losses (of material and fibres) and higher productivity are significant economic incentives. From a performance

point of view, very little information is available where a direct comparison between the wet- and the dry- process can be analysed; mixture design, fibre type, and dosage are generally different. Limited experimental data has shown that although fibre rebound is higher with the dry-mix process, the excellent matrix and bond created with the fibre may lead to better toughness results at similar initial fibre dosages in the dry process (Jolin et al., 2015a,b).

3. THE CHOICE OF GROUND CONTROL METHODS

Whilst FRSC has become widely used, it is far from being the only means of stabilising ground available to the tunnelling community. The alternatives to FRSC for ground support include bolts on their own, ribs and lagging, steel sets, bare mesh and bolts, plain sprayed concrete, mesh-reinforced sprayed concrete, cast-in-place concrete, and precast segments. Each has either been used in the past or continues to present advantages in certain applications. This section briefly reviews these alternatives to set the context for evaluating whether or not to use FRSC.

3.1 Rock bolts on their own

Bolts are the primary means of stabilising hard rock around excavations, thus stabilisation of an excavation can frequently be achieved through the use of bolts alone. Numerous excavations in highly massive ground have been successfully stabilised using only bolts. However, the surfaces between bolts remain potentially unstable and therefore may present a hazard to users in both the short and long term. Concrete sprayed over the ground prior to installation of bolts increases safety as it provides protection against local fall-outs in hard rock. Inclusion of fibres in this sprayed concrete helps to ensure that fall-outs are less likely.

3.2 Ribs and lagging

This is an old form of support that nevertheless remains in use in some applications. The main shortcoming to this method is that the support is not in continuous contact with the ground, in the way that sprayed concrete is, so it allows the ground to converge or collapse onto the support before the support begins to act fully. This leads to greater ground movement and higher loads. Despite this limitation, it remains in use for applications such as controlled collapse of ground in some types of mine.

3.3 Steel sets and lattice girders

Steel sets or lattice girders are sometimes incorporated into sprayed concrete linings to stiffen or "strengthen" the lining (Collotta et al, 2010). The amount of steel present in the sets or lattice girders contributes little to overall stiffness or strength in the short term, but may contribute to long term stiffness in mature linings due to the effects of compressive creep in the concrete. Their presence creates a safety hazard as workers have to work under the partially supported face to install these arches. Lattice girders are rarely incorporated in design calculations as a structural member since they are spaced at one metre or more apart and they contribute little in comparison to the rest of the concrete lining (Thomas, 2009); their main purpose in soft ground tunnels is to ensure the correct shape of the tunnel. There are better ways of "strengthening" a sprayed concrete lining or handling adverse ground behaviour, including thickening the lining or using spiling to improve crown stability (Nye, 2012). Spiling can be installed with arches to guide the drilling, but spiles can also be installed in the absence of arches. Both types of arch element introduce a durability risk because it is very difficult to encapsulate them fully in sprayed concrete (see Section 3.6). Modern laser survey techniques have removed the need to install lattice girders to ensure that the correct shape and thickness are sprayed – as demonstrated on recent major projects in the UK such as Crossrail and the Hindhead road tunnel (Pickett and Thomas, 2012; Thomas and Dimmock, 2017).

3.4 Bolts and mesh

The use of bare steel mesh with bolting was widespread in the underground mining industry until recently (Lang and Stubley, 2004). The primary reason why the industry has moved away from bare mesh is that this form of passive control merely catches fall-outs of unstable rock and does little to prevent progressive unravelling and instability. Bare mesh braced across the surface of ground does not lock the ground surface together and does not prevent it unravelling in the way sprayed concrete does, so the effectiveness of this form of 'ground control' is questionable. Bare mesh also suffers rapid corrosion (Robinson and Tyler, 1999; Hadjigeorgeou et al, 2008; Dorion and Hadjigeorgiou, 2014), and is a 'soft' form of support compared to sprayed concrete linings. While this may be useful for high levels of ground deformation, both these factors can cause problems in the long term **(Fig. 2)**. Unprotected bolts and exposed mesh may require costly rehabilitation within a short period after installation due to the corrosive nature of most ground water (Hassell et al, 2004).

3.5 Unreinforced sprayed concrete

Unreinforced sprayed concrete has historically had its uses, such as for thick-shell linings in soft ground but has been replaced by FRSC in the majority of hard-rock applications. This is because fall-outs can occur following cracking or excessive compressive loading, or in response to localized delamination. Fibres improve the resilience of arched thick-walled linings and resistance

Fig. 2 : Effect of corrosion on unprotected steel mesh and rebar leading in underground environments.

to fall-outs by providing a degree of tensile capacity and load redistribution should cracking occur. A simple review of the critical capacities required for a given application is a good way to determine if plain sprayed concrete is adequate (Grøv and Blindheim, 1997).

3.6 Mesh reinforced sprayed concrete

This remains in use in many projects, especially in soft ground or weak rock when linings are sprayed against a smooth substrate, but it suffers from a number of problems. One of the primary shortcomings with mesh is the poor productivity and slow rate of installation compared to FRSC. The manual handling requirements inherent to mesh also compromise safety during installation. In the past, worker injury during installation of mesh was one of the most significant causes of death and injury in underground mining (Surface Rock Support for Underground Mines, 1999; Lang and Stubley, 2004; Potvin and Nedin, 2004). These issues have largely led to the elimination of mesh in underground mining operations in many countries. Despite this, mesh and lattice girders continue to be used in temporary linings in many tunnels in soft ground, primarily because a shell of sprayed concrete reinforced with two layers of mesh has a higher bending capacity than FRSC.

In hard rock excavations, rates of sprayed concrete consumption and bolting frequency are both higher with mesh than with FRSC. Mesh is expensive to use over blasted ground because it usually spans irregular hollows in the surface which must either be filled before installation of mesh, or more commonly, must be filled with sprayed concrete by spraying through the mesh until the bars are encapsulated. This leads to high rates of concrete consumption compared to when fibre reinforcement is used (Windsor and Thompson, 1999). For the same reasons, the mesh is not very effective in structural terms either. In contrast the same support can be achieved by spraying a thin layer of FRSC over the irregular contours of the rock surface. While the cost of normal steel mesh may appear to be lower than the cost of fibres required to achieve similar levels of in-place performance, the cost of the additional concrete required to fill voids and compensate for losses due to vibration of bars, and the high rate of rebound off the mesh itself, combine to make mesh uneconomic compared to fibres in the majority of hard rock applications.

The photographs in Fig. 3 illustrate another of the potential problems associated with mesh and bars, which is shadowing and ineffective encapsulation of the reinforcement. Concrete sprayed against mesh or bars usually sticks to the first bars encountered (Basso et al, 2016), especially when a set accelerator is used, leading to frequent incomplete filling of voids behind the bars and possibly rendering the bars structurally ineffective (Fischer and Hoffman, 2015). Not only will these bars not provide support to the ground, but they will corrode very rapidly as the back-face of each bar is likely to remain free of concrete and is therefore exposed to the atmosphere or groundwater (Jolin et al, 2004). Since set accelerators are essential when spraying overhead, the problem of inadequate encapsulation and risk of subsequent corrosion explain the increasing preference for FRSC over mesh in long-term applications (Pickett and Thomas, 2012; Thomas and Dimmock, 2017). Not surprisingly, experience with inadequate encapsulation has shown that the protection against corrosion offered by the surrounding sprayed concrete to the reinforcement falls drastically when the encapsulation quality deteriorates (ACI 506.6T-17).

Despite similar disadvantages, the use of steel bars to reinforce sprayed concrete in certain applications offers the major advantage of providing far higher levels of post-crack tensile and flexural capacity than is possible using

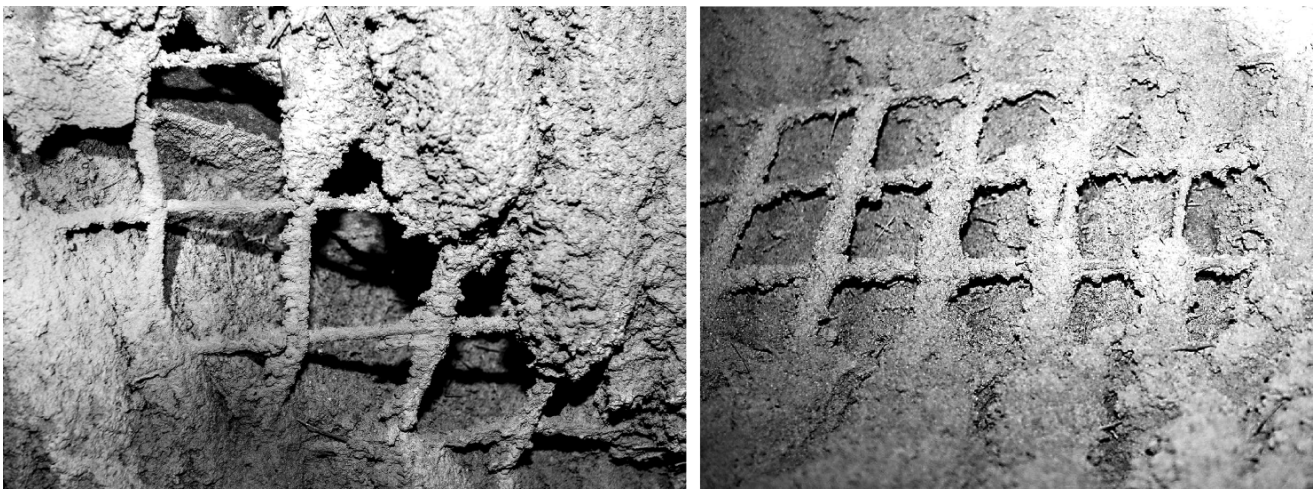

Fig. 3 : Incomplete encapsulation of steel mesh and rebar leading to ineffective support and long-term corrosion.

fibres (Thomas, 2009; Thomas, 2014; Sigl 2015). This is a high cost solution but the advantages are significant in largediameter or non-circular sprayed tunnel linings with uneven ground loading. Steel bars are occasionally used in rock tunnels to form socalled "reinforced ribs of sprayed concrete" (RRS in the Q-system chart), as additional support in extremely poor ground conditions (NGI, 2013).

3.7. Cast-in-place concrete

This remains the preferred method of final lining support in many tunnels in soft ground and often in rock, largely due to concerns over water-tightness (Thomas, 2009) but sometimes also over the long-term durability of set-accelerated sprayed concrete (Beaupre and Jolin, 2002). Functional requirements for surface finish (e.g. a smooth and reflective surface for the sidewalls in heavily trafficked road tunnels) also often dictate the use of cast-in-place linings as well as the lower cost compared to FRSC for long tunnels with a constant cross-section. In terms of waterproofing, casting concrete against a sheet membrane is much easier than spraying concrete onto a sheet membrane and spray applied waterproofing membranes are only used in areas where the water pressure is low. The relative costs of sprayed and cast final linings can vary substantially between projects. Permanent FRSC linings have been used for many decades in the hydropower sector and for transport tunnels in rock in some countries, notably in Scandinavia and Australia. FRSC has also been used for permanent linings in soft ground for more than 20 years with major projects such as Crossrail in the UK having successfully used FRSC for both the initial and the permanent lining (Thomas, 2009; Pickett and Thomas, 2012; Gakis et al, 2015; Batty et al, 2016). Recent evidence has indicated very good durability for sprayed concrete provided the dosage rate of set accelerator is moderate (Zhang et al, 2016). Cast-in-place concrete has a proven record of durability but suffers the disadvantage that it is slow to install, requires relatively expensive formwork, and requires some other form of shortterm support (usually sprayed concrete) to stabilise the ground before the secondary lining can be installed.

3.8. Pre-cast concrete segments

Tunnel segments are normally installed immediately behind a Tunnel Boring Machine (TBM) and are therefore restricted to tunnels in which the geometry is regular for relatively long distances (for example, running tunnels for metro lines). TBMs generally offer substantial advantages in terms of superior excavation rates and, in the case of Closed Face TBMs, active control of groundwater and earth pressure (ITAtech, 2016). However, segments of varying geometry are difficult to use and thus changes in excavation geometry are often accommodated through the use of sprayed concrete (for example, in stations and cross-overs).

3.9 Waterproofing

As for normal cast concrete, completely waterproof concrete (without any additional waterproofing membrane) can be difficult, if not impossible, to achieve with FRSC alone, especially when high ground water pressure is present (more than 5 bar) or the ground is saturated and very permeable. If high hydrostatic pressures are expected, only a full circular structure can provide sufficient resistance to water loads. If thin-walled linings or a partial arch are used, drainage is normally included (Aldrian and Geifing, 2009). For moderate ground water pressures, FRSC can be used effectively with spray applied polymer-based waterproofing membranes to provide a dry tunnel lining (Thomas, 2009; ITAtech, 2013; Holter, 2015). FRSC can be used with sheet membranes which are covered by a geotextile during spraying. However, there are significant practical difficulties with spraying onto sheet membranes.

3.10 Advantages of FRSC

Engineers are faced with numerous choices as to the method of ground support to use when presented with the challenges of a particular project. Under these circumstances, and considering the alternatives outlined above, it is pertinent to ask: what are the main advantages of fibre reinforced sprayed concrete? The following points define the advantages of FRSC that have been revealed through extensive experience and cost comparisons in projects the world over.

The most significant advantage of Fibre Reinforced Sprayed concrete is that it is the most versatile and robust method of stabilisation available and can be used for a wide range of conditions with the same equipment and personnel. This versatility leads to cost-effective solutions, especially in highly variable ground and for smaller projects.

FRSC is especially suited to drill and blast excavation techniques in hard ground and therefore accounts for much of the ground stabilisation carried out internationally when this type of excavation process is used. FRSC is also unrivalled in its ability to accommodate complex geometrical excavations and junctions such as those typically required for underground metro stations.

Fibre Reinforced Sprayed concrete is very useful for controlling highly unstable ground because of its ability to lock the surface of the ground together quickly and prevent unravelling. However, it is sometimes useful to include local steel mesh with the FRSC, especially around points of water ingress, to support the concrete until it hardens. Running water ingressing from the ground should not be sealed using sprayed concrete but should be allowed to drain via secondary measures. The area can then be sprayed over with concrete and the inflow stopped later by injection treatment of the ground. The process of applying FRSC offers a substantial improvement in safety compared to earlier methods such as use of mesh (Potvin and Nedin, 2004; Bernard, 2013), particularly for primary stabilisation immediately behind the face when applied using remotely controlled spraying equipment. Lost time injuries in mines and tunnels have been shown to fall markedly compared to meshing once FRSC was adopted as the support method (Lang and Stubley, 2004).

Fibres can offer substantial reductions in concrete usage compared to mesh when used over irregular surfaces in rock excavations, but this depends to some extent on whether a smooth surface is required for the final inner surface. The reduction in concrete usage can more than offset the higher cost of fibres compared to mesh per square metre of surface stabilised. FRSC and bolts have been used as part of ground control in long TBM driven tunnels for various projects because this support system can be optimised to suit the ground as it is encountered and can thereby prove quite competitive (Spörri and Bracher, 2006; Thomas, 2009).

Lastly, fibres offer greatly improved corrosion resistance compared to steel mesh or bars (Hoff, 1987; Thomas, 2009; ITAtech, 2016), and macro-synthetic fibres offer a very high level of resistance to degradation at cracks (Halvorsen et al, 1976; Mangat and Gurusamy, 1987; Nordström, 2009; Kaufmann, 2014; Bernard, 2015b). This has opened the path for using FRSC as permanent ground support (Thomas and Dimmock, 2017), as shown on major projects like Crossrail in the UK.

3.11 Disadvantages of FRSC

The most commonly cited disadvantage of Fibre Reinforced Sprayed concrete is the relatively high material cost of fibres compared to alternative forms of reinforcement such as steel mesh. However, when the total cost of ground stabilisation using FRSC is compared to alternatives such as mesh, then FRSC is almost always more competitive.

Fibre Reinforced Sprayed Concrete linings do not offer as high a flexural capacity as bar-reinforced concrete. This is because the total amount of reinforcement that can be incorporated using bars is much greater than is possible using fibres. However, bars present a problem of proper encapsulation with concrete, especially when spraying overhead using a set accelerator, which raises durability concerns, and bars are slow and expensive to install. If bars are included, the dosage rate of set accelerator must be reduced until the surface of the concrete is flush with the level of the bars, and then normal spraying can be resumed; preconstruction mock-up trials are often required for complex reinforcement layouts to demonstrate the ability of the nozzleman, equipment and mix design to satisfy quality requirements.

Properly trained personnel are essential to all sprayed concrete applications. A number of certification/education programs have been developed over the years to help in achieving high quality sprayed concrete structures. The ACI and Asquapro Certification programs (Gagnon et al., 2004; ACI CP-60-09; Larive, 2009) are hand-spraying based programmes that place an emphasis on the quality of the reinforcement encapsulation as the main evaluation factor. Another popular programme offered by EFNARC (1996) concentrates on the safe and correct operation of robotic applicators and wet mix sprayed concrete. However, both programmes (and others available so far) are oriented toward sprayed concrete in general and have nothing included specifically for fibre reinforced sprayed concrete. In cases where fibre related quality issues are present with regards to the workmanship, it may be necessary for trials to be instituted to examine these issues through specifically adapted construction mock-ups.

The surface finish of fibre reinforced sprayed concrete is generally rough and usually needs additional attention if appearance is important. The "as-shot" finish that typifies most sprayed concrete is seldom considered satisfactory for structures in public areas, so some form of aesthetic improvement is usually required (Thomas, 2009; Hicks, 2010). If steel fibres are used, a smoothing layer must be added where people may be exposed to injury by rubbing against the protruding fibres on the surface. This can increase the total cost of FRSC linings.

4. DESIGN ISSUES RELATED TO USE OF FIBRES

The following sections discuss the main design issues related to FRSC. Tunnel design involves numerous simplifications and the art of the tunnel designer is in choosing what is important and what is not important. Some of the factors which must be considered include: is the tunnel deep or shallow? If shallow, with sensitive structures above, control of deformation may demand more support or a more restricted excavation sequence than would normally be required to maintain stability. Another important consideration is the relationship between the strength of the ground and the stresses that must be redistributed around the opening. If the ground is relatively strong, then controlling local instability is the role of the support (due to wedges, for example). If the ground is relatively weak, comprising either soft ground at moderate depth, or rock at great depth, to maintain stability the support will have to resist more widespread radial pressure. The overall design of a tunnel is a broad subject and readers can find more information in publications such as: Hoek and Brown (1980), Hoek et al (1998), BTS (2004) and Thomas (2019).

4.1 Fibres in sprayed concrete linings

The compressive strength of sprayed concrete can be taken to be unaffected by the presence of fibres, except in cases where a very high fibre dosage interferes with compaction. However, fibres are not used to effect a change in the compressive strength of sprayed concrete - they are primarily used to reduce crack widths and support tensile stresses across cracks. The maximum capacity of fibre reinforcement is limited by the inherent tensile capacity of individual fibres, bond between fibres and concrete, the dosage rate of fibres used, and time-dependent processes such as creep rupture which can lead to a reduction in long term capacity compared to short-term capacity (Bernard, 2014).

Given constraints on the maximum dosage rates possible in sprayed concrete, the typical strength of a sprayed concrete matrix, and the effects of creep, the maximum sustained direct tensile stress that can be supported by FRSC is only about 1–2 N/mm^2 for any type of fibre. This is considerably lower than the short-term tensile capacity of FRSC and is also lower than the flexural-tensile strength of the concrete matrix. Because of this, and the uncertainty over the effects of long-term tensile creep, it is generally recommended that a deflection hardening FRSC lining is required to support a high sustained tensile stress (e.g. as in an internally pressurized water tunnel) (ITAtech, 2016). This almost always requires the inclusion of steel bar reinforcement.

FRSC almost always exhibits deflection-softening behaviour in shear following cracking, thus the shear resistance of a lining should be assessed on the basis of the shear strength of the concrete matrix alone. The apparent shear strength of the concrete will vary depending on whether restraint against dilation is applied to the failure surface or a compressive in-plane stress is present (Nielsen, 1999; MC2010, 2012).

4.2 Early-age load resistance

Structural (macro) fibres, whether made of steel or polymers, rely on the compressive strength and hardness of the concrete matrix to generate resistance to pull-out, thus they provide little post-crack performance in young sprayed concrete (of less than ~10 N/mm^2 compressive strength, or ~0–12 h age, Bernard and Hanke, 2002). Young fibre reinforced sprayed concrete linings can therefore be considered to be unreinforced. Experience has shown that most early-age failures of FRSC linings in hard ground involve a punching shear mode of failure leading to fall-outs of sprayed concrete with or without rock attached (Potvin and Nedin, 2004; Bernard, 2008). In soft ground, failures can also occur in bending or more rarely compression (Macht et al, 2000). If a higher short-term load resistance is required in very young sprayed concrete (for example, in highly unstable ground), it is recommended to use steel mesh (with lattice girders) instead as this can help to support the sprayed concrete until it gains sufficient strength as well as providing a higher bending capacity.

4.3 Mature age loading

Several types of loading may be imposed upon FRSC linings in the medium and long term. These include but are not limited to the following.

4.3.1 *Gravity loads*

For many hard rock applications in which FRSC linings are used to control ground between rock bolts, gravity loads are often the only load considered. This essentially consists of loose rocks or wedges, usually at the crown and upper parts of the lining. If this is the only load considered, a FRSC lining of 50–100 mm thickness will usually suffice for typical bolt spacings of 1–2 m (e.g. see Q-system, Barton and Grimstad, 2004; NGI, 2013).

4.3.2 *Static ground pressure*

For soft or hard rock applications, fractured or soft ground may be considered to exert a pressure as a distributed load on the outer surface of the lining. Wedges subject to squeezing may also impose a pressure greater than can be expected under the action of gravity alone. The forces induced in a lining by such ground pressures are likely to be substantially higher than for gravity loads alone, since they are governed by the in situ ground pressures and any yielding in the ground.

The loads are resisted mainly in compression by an arch or ring of FRSC. Hence, the lining thickness is usually required to be greater than 100 mm. Uneven geometry or loading will induce bending moments which may require consideration together with the expected axial compressive loads using an N-M (axial force/ bending moment) interaction diagram (Thomas, 2009; Thomas, 2014; Gakis et al, 2015). If bending stresses are high compared to axial compressive stresses, fibres may not be sufficient to support the resulting tensile stresses and steel bars are likely to be required with or without fibres.

4.3.3 *Squeezing ground*

When the in situ ground stresses are much higher than the strength of the ground, squeezing conditions may occur. This can result in very high local bending moments and axial loads in a lining, which usually require deep rock bolts, bar reinforcement, yielding arches, or some other type of heavy reinforcement to be used inside the sprayed concrete (Grimstad et al, 2002; Chryssanthakis, 2016). Fibres will improve the ductility and safety of the system but on their own are very unlikely to provide sufficient capacity to resist such loading conditions. Squeezing ground is an exceptional case which will need a special design (Thomas, 2009).

4.3.4 Dynamic loads

Several types of dynamic loading can be considered in the context of FRSC linings. Dynamic loading associated with blasting occurs in almost every tunnel excavated by drill and blast techniques (Ansell and Holmgren, 2001). With this type of excavation, a young sprayed concrete lining is likely to be exposed to shock near the face of the heading on every excavation round. Research and field experience have shown that young sprayed concrete is highly resistant to this form of loading, but resistance to shock waves travelling from within the rock diminishes as the concrete hardens (Ansell, 2010; Ansell and Ahmed, 2012). Dynamic loading associated with local micro-seismic events (rock bursts) generally involve a shock wave travelling through the rock resulting in rock or mature sprayed concrete being ejected at the surface of an excavation (Ortlepp and Stacey, 1994). From the limited studies so far, it appears that fibre reinforced sprayed concrete provides limited resistance to this type of dynamic loading, with both steel and macro-synthetic fibres being similarly ineffective. If local seismicity is anticipated, steel mesh or strapping placed over the surface of the sprayed concrete has been shown to offer improved resistance (Potvin et al, 2010). Rehabilitation is normally needed after each seismic event. Long wavelength dynamic loading associated with earthquakes may cause delamination between the lining and ground but experience has shown that FRSC linings generally perform well in earthquakes (Hashash et al, 2001; Hashash et al, 2005). Other types of dynamic load occasionally encountered include surge pressures in water tunnels and fluctuating air pressure caused by the passage of trains in rail tunnels (Bryne et al, 2014).

4.3.5 *Service equipment*

Equipment such as ventilation and fire-fighting water lines required in a completed tunnel are usually anchored within the body of the lining using bolts. For this reason, point loading of a lining may need to be considered. If substantial loads are involved, anchorage may require that the bolting extends into the surrounding ground. Fatigue resistance may need to be considered, if cyclic loads are associated with service equipment.

4.3.6 *Abrasion, cavitation, and impact*

Although not normally considered loads, abrasion and cavitation are forms of stress that can lead to damage to a FRSC surface under certain circumstances. Ore passes, water tunnels, the walls of traffic tunnels, and drains are examples of FRSC surfaces that could be subject to substantial abrasion or impact. Experience has shown that high strength is useful for improving abrasion resistance, and inclusion of certain types of aggregate can also improve performance (Van Heerden, 2004). However, fibres generally have little effect on the abrasion resistance of a sprayed concrete surface unless used at extremely high dosage rates. Fibres can improve the resistance of concrete to impacts. Cavitation resistance is primarily related to the strength of the concrete matrix and hardness of the aggregate and is only slightly affected by the presence of fibres (Horszczaruk, 2005; ACI 210, 1993).

4.3.7 *Fire resistance*

Resistance to fire can be assessed in many different ways, but for sprayed concrete linings is most commonly assessed in terms of resistance to explosive spalling (that is, the violent ejection of pieces of concrete from the surface during a fire due to vapour pressures generated inside the concrete). Explosive spalling occurs when sprayed concrete undergoes a rapid temperature rise, such as may arise in response to a hydrocarbon-fuelled fire (Dehn and König, 2003; Chatenoud et al, 2008). The propensity for sprayed concrete to experience

explosive spalling is affected by the initial moisture content, its strength and density, and the gradient and magnitude of the temperature rise. It is also affected by the age of the concrete, type of aggregate used, and the level of compaction (Xing et al, 2011). Resistance to spalling can be improved by using relatively low strength sprayed concrete and is especially improved by the presence of mono-filament micro-synthetic fibres made of polypropylene (Khoury and Willoughby, 2008; Khoury, 2008). Extensive research has been published in relation to this phenomenon, and a minimum dosage rate of micro-synthetic fibre is now typically prescribed for FRSC linings when fire resistance is required (Silfwerbrand, 2011; Maraveas and Vrakas, 2014). Other types of fibre, including steel and macro-synthetic fibres, have a minor effect on resistance to spalling. Post-crack performance during and after a fire depends on many factors and remains a subject of continued research. The fire resistance of an FRSC lining should be considered in light of the overall fire design, with consideration of the risk and magnitude of potential fires, and the level of residual structural performance required after a fire.

4.4 Assessment of FRSC performance

The performance of FRSC can be assessed in many ways, but most commonly performance is categorised in terms of either mechanical properties or chemical resistance to the environment. The mechanical strength of uncracked FRSC can be measured in compression, bending, or shear, and elasticity is sometimes also assessed. The typical compressive strength range of sprayed concrete ranges from zero immediately after spraying to as much as 50–60 N/mm^2 at a mature age (depending on the mix design used). Experience has shown that once young sprayed concrete has hardened beyond 0.4 N/mm^2 it is generally resistant to fall-out under its own weight. A compressive strength of 1.0 N/mm^2 is widely taken to represent sprayed concrete that is safe to re-enter under, provided bolts are also installed and stressed (Potvin and Nedin, 2004). In hard rock applications, in-place compressive strength is seldom measured once it has exceeded about 2 N/mm^2. In soft ground conditions, strength and stiffness are monitored more carefully over an extended period of time due to concerns about stability, especially under high or uneven ground pressures (Hellmich et al, 2003; Thomas, 2009).

Post-cracking performance (also known as 'toughness') is commonly assessed in terms of energy absorption or residual flexural strength. Numerous test methods are available, but the more common ones are based on either beams (EN14488-3, EN14561, ASTM C1609, JCI SF4) or panels (ASTM C1550; EN14488-5; Norwegian Concrete Association, 2011, see Fig. 4). Post-crack performance will depend on many factors, including the type of fibre used, the dosage rate, the strength and age of the concrete matrix, and the degree of compaction achieved (Morgan et al., 1999a,b, ITAtech, 2016). Performance levels vary greatly between brands thus choice of a fibre must be based on assessed performance rather than prescriptive directives such as 'minimum X kg/m^3 steel fibre' or 'minimum Y kg/m^3 macro-synthetic fibre' (see also ISO 13270, EN 14889 (Parts 1 & 2), ACI 544.5R-10). For this reason, preconstruction trials are highly recommended prior to acceptance of a given mix and fibre for major projects.

Fig. 4 : Spraying ASTM C1550 round panels for Quality Control purposes.

The choice of which test method to use for assessment of the postcrack performance of FRSC depends on many factors (see ITAtech, 2016 for a more detailed discussion). If using an empirical method of design such as the Q-system, performance should be assessed using either the EN14488-5 square panel test, or the ASTM C1550 round panel. A relationship between energy absorption levels generated in these two test methods was developed by Bernard (2002) and incorporated into the Qsystem by Papworth (2002). Numerous examples exist of projects involving FRSC linings that were designed by the Q-system (Decker et al., 2010; Nitschke and Winterberg, 2016). The assessment of energy absorption should be consistent with other design criteria. For example, assessing energy absorption on tests at large central deflections implies larger open cracks which may conflict with other durability or serviceability requirements.

If using a deterministic design method, especially one based on an N-M interaction diagram, it is common to base performance assessment on beam tests such as ASTM C1609 or EN14651. Design approaches considering the combined axial loads and bending moments can incorporate the tensile strength of the fibres when calculating the bending moment resistance in a manner that is otherwise similar to N-M diagrams generated for conventionally reinforced beam-columns.

However, the fibres in FRSC provide only a modest enhancement of bending resistance (Thomas, 2014). It should also be noted that notched and un-notched beams will produce different estimates of flexural performance for the same FRSC mix, with notched beams generally resulting in estimates of flexural-tensile capacity that are about 20 percent higher than is apparent using un-notched beams (Wille and Parra-Montesinos, 2012; Finazzi et al, 2014).

Beams generally result in estimates of post-crack performance that exhibit a within-batch Coefficient of Variation (COV) that is substantially higher than is commonly achieved for panels. The EN14488-3 beam test method was previously used for sprayed concrete cut from sprayed panels but has been found to exhibit extremely high withinbatch variation in performance (with a population COV of 40–50 per cent, Hanke et al, 2001), making this test method all but useless for Quality Control testing of FRSC. Its primary use remains for flexural testing of FRSC extracted from in situ panels when the lining thickness is too small for other types of beams to be produced. Alternatively, when the post-crack performance of in situ FRSC is required (especially when only limited volumes of material are available), the Double Punch test can be applied to cores extracted from the works and used to obtain a rough estimate residual tensile strength (Chen, 1970; Molins et al, 2006; Galobardes and Figueiredo, 2015).

4.5 Long term performance & durability

The long-term performance of FRSC can be assessed in terms of either the durability of the concrete matrix, deterioration of the fibres, retention of post-cracking performance with aging, or mechanical changes in behaviour attributable to either creep or shrinkage (Wittman, 1982; Bažant, 2000).

The durability of a sprayed concrete matrix can be comparable to or better than cast concretes due to the high cement contents and lower water/binder ratios used. This is especially true when pozzolans such as fly ash and silica fume are included (Thomas, 2009; Zhang et al, 2016). However, durability of the matrix may be compromised by over-dosing of set accelerator during spraying, thus it is important to limit the maximum dosage rate of set accelerator (typically to less than 6% by weight of cement, Beaupre and Jolin, 2002).

The main form of deterioration experienced by fibres within sprayed concrete is corrosion, but this primarily occurs with steel fibres. In the absence of cracks, steel fibres will generally remain free of corrosion provided the concrete has not carbonated. Even if they corrode, the expansion of the corrosion products is insufficient to cause break-up of the concrete matrix (Hoff, 1987). Hence steel fibre reinforced concrete is generally regarded as having good durability in an uncracked state. At cracks, the rate of steel fibre corrosion will depend on exposure conditions, crack width, and the possible presence of flowing water through the crack (Halversen et al, 1976; Nordström, 2001, 2009; Kaufmann, 2014, Marcos-Meson, et al, 2018). Maximum acceptable crack widths for steel FRSC in an underground environment are generally taken to lie between 0.15 and 0.20 mm (AFTES, 2013; MC2010, 2012). Macro-synthetic fibres do not corrode and therefore are not subject to concerns about maximum crack widths. Tensile creep may cause crack widths to increase with the passage of time both for steel and macro-synthetic FRSC (Kusterle, 2016). Some studies have shown relatively minor differences in creep characteristics at early ages for different reinforcement types under a load of less than 50% of peak capacity (Larive et al, 2016). However, this depends heavily on the fibre type and load intensity. Due to their higher stiffness, metallic fibres will usually exhibit a smaller initial crack opening than that exhibited by synthetic fibres at a similar relative load level.

If the strength and hardness of a concrete matrix substantially exceed the original design strength, some types of fibre may change their post-cracking behaviour from the preferred pull-out mode to one of fibre rupture. This affects steel fibres more commonly than macrosynthetic fibres and can lead to a reduction in post-crack performance for ULS conditions but does not affect performance in relation to serviceability (Bjøntegaard et al, 2014; Bernard, 2015a). This can be avoided by ensuring that the fibres chosen are suitable for the actual strength of the FRSC during the mix design stage and that the long-term strength of the concrete is not unnecessarily over designed.

The creep of a sprayed concrete matrix is high compared to normal cast concrete, both in the short and long term, due to the high cementitious content and low coarse aggregate content of the sprayed concrete matrix (Goodier et al., 2008; Thomas, 2009; Collota et al, 2010). This can lead to relaxation of stresses in a lining with the passage of time (Chen and Small, 2008; Thomas, 2009), which may reduce the likelihood of flexural cracking but may also lead to an increase in deformations in the long term. Creep of cracked FRSC is a complex issue affected by many parameters including the type of fibre used, relative magnitude of stress imposed, and the age of the sprayed concrete (MacKay and Trottier, 2004; Larive et al, 2016; Garcia-Taengua et al, 2016). The application of normal concrete design codes, as is typically done in soft ground FRSC linings, ensures that the stresses in the concrete are fairly low compared to its strength (i.e. less than 50%) which reduces creep. If long-term crack widths are required to be limited under a sustained tensile stress, it is recommended to include steel bars (Gilbert and Bernard, 2015).

Unrestrained drying shrinkage is generally quite high for cast sprayed concrete, averaging about 700–1000 microstrain at 56 days age, and has been measured to be even higher for set accelerated sprayed concrete (Thomas, 2009; Lagerblad et al, 2010). Limited longterm testing has indicated unrestrained drying shrinkage of about 1200–1500 microstrain at 300 days (Goodier et al., 2008). However, restraint in the form of a stiff substrate will reduce this level of shrinkage compared to unrestrained beamlets of the type commonly used in tests. The prediction of in-place restrained drying shrinkage and its consequences is very complex (RILEM, 2011). Drying shrinkage cracks are seldom noticed in temporary FRSC linings due to the effects of ground convergence, but shrinkage cracking can be a problem in long-term permanent linings (Grasberger and Meschke, 2003; Ansell, 2010) especially if applied a long time after excavation. Anecdotal evidence indicates that the drying shrinkage of dry-mix FRSC is similar to that experienced by wet-mix FRSC of a similar quality.

5. DESIGN METHODS

The benefits of including fibres in sprayed concrete can be incorporated into normal design methods for both hard rock and soft ground. This section will focus on how to achieve this but will only consider the aspects of design where FRSC differs from normal unreinforced or bar reinforced concrete. As discussed earlier, FRSC behaves in the same way as plain concrete in terms of compressive and shear loading. In the context of a highly redundant system comprising the ground, other rock support such as bolts, and a lining, FRSC is a ductile material that normally displays substantial residual flexural capacity to facilitate redistribution of ground loads. The ability of the system as a whole to redistribute load means that deformation softening FRSC is sufficient for most situations. Deformation hardening is not required. In this last respect, FRSC resembles mesh reinforced concrete.

For further information on the design of tunnels and ground support, more detailed guidance can be found in publications including Hoek and Brown (1980), Hoek et al (1998), BTS Tunnel Lining Design Guide (2004) and Thomas (2009). Bertuzzi and Pells (2002) also described the specific design case of single-pass sprayed concrete linings on sandstone.

5.1 Design of FRSC linings in hard rock

The majority of support for hard rock tunnels is designed using empirical methods or relatively simple analytical methods (Thomas, 2009, 2014). Empirical methods such as the Q-system (Barton et al., 1974; Barton and Grimstad, 2004, NGI, 2013) are widely used and well documented and take the form of design charts for rock support based on the span of the tunnel and a rock mass classification. In this method, previous successful ground control strategies are encapsulated in the form of a chart delineating the spacing between bolts, minimum sprayed concrete lining thickness, and minimum toughness levels required for various degrees of potential instability. Empirical methods such as the Q-system account indirectly for the redistributive capacity of FRSC by including toughness parameters in the estimation of support requirements. The degree of support required is often estimated in the design stage and then the actual support is adjusted using an observational approach as the tunnel advances, based on the actual ground conditions and the performance of the rock support. The Q-system originally included recommendations for FRSC toughness based on the EFNARC (later EN14488-5) square panel test method (Roux et al, 1989), but Bernard (2002) and Papworth (2002) extended this to include performance specification based on the ASTM C1550 round panel test method.

A range of analytical tools are available to supplement designs based on empirical methods. Broadly speaking, these can be split into two sub-groups: tools which consider support of blocks (e.g. UNWEDGE or the method proposed by Barrett and McCreath, 1995) and tools which consider the rock as a continuum (e.g. the Convergence Confinement Method). The latter consider the lining as a linear elastic material so the post-crack ductility provided by fibres in FRSC is not exploited directly. Early age properties (i.e. less than 24 h old) can be estimated from published relationships (e.g. Chang and Stille, 1993, Thomas, 2009, Bernard, 2008). Bernard (2011) described an approach to estimating the capacity of early-age and mature sprayed concrete linings subject to punching failure.

The method proposed by Barrett and McCreath (1995) considers lining failure in hard ground conditions by various modes but the authors concluded from the results of large scale tests that failure occurred first in either adhesion or direct shear, followed by bending or punching shear. While some parameters were reported in the original paper, designers may prefer to use their own data or established relationships (e.g. Eurocode 2; Bernard, 2008) to estimate parameters such as shear strength or flexural strength at the ages required in the design.

Numerical models are used for more complex excavations in rock. However, the lining is almost always modelled using linear-elastic beam or plate elements so the ductile capacity of the fibres is not exploited since non-linear post-crack performance adjustments are usually not included. This means that the re-distributive capacity associated with fibre reinforcement is not simulated directly in the model to account for possible interaction and re-distribution of load between the lining and ground. The results generated using these methods of analysis

(and also some analytical tools) may be plotted on an N-M (axial force-bending moment) interaction diagram and compared to available capacity (Thomas, 2014) as shown in Fig. 5.

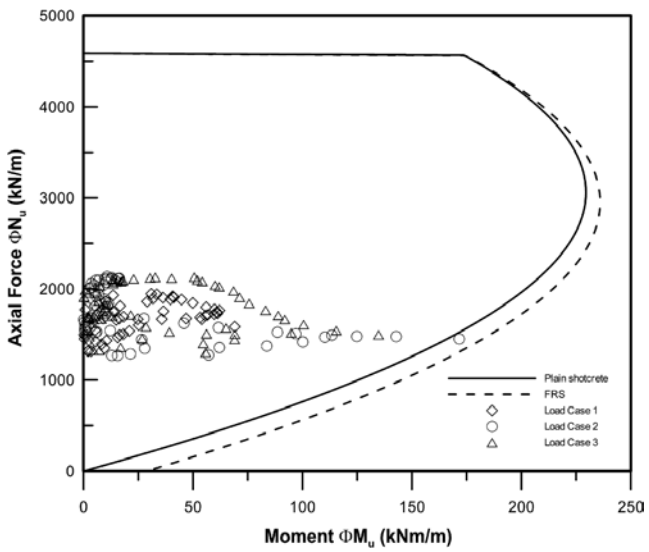

Fig. 5 : Example of an N-M interaction diagram for a tunnel in sandstone, showing the capacity increase associated with inclusion of fibres in the lining.

It should be noted that the apparent capacity of a lining as described in an N-M diagram depends on details and assumptions incorporated into the section model used to estimate capacity. These section analyses are often based on MC2010 (2012) or reinforced concrete fundamentals, and most commonly reveal only the peak moment-axial load resistance of a section and do not indicate the degree of post-crack ductility available for a selected design. If ductility is important, it may be necessary to conduct several analyses of lining capacity at selected values of maximum crack width to establish a more comprehensive model of lining behaviour (Nitschke and Bernard, 2017). The introduction of an axial compressive stress can substantially increase the 'system ductility' for a FRSC lining (Gilbert and Bernard, 2018). If a substantial axial compressive stress is anticipated then a lining exhibiting relatively low ductility in pure bending may be sufficient for most design circumstances because the beneficial effect of the axial compressive stress acts to transform the post-crack behaviour so that it exhibits substantial strain-hardening under combined compression and bending.

5.2. Design of FRSC linings in soft ground

Sprayed concrete linings are often used in soft ground. Many soft ground tunnels are built in sensitive urban environments and a high level of rigour is required in design to preclude adverse outcomes such as excessive surface settlement. The application of the technology has improved greatly by learning from the past (ICE 1996) and through recent technological advances. For the reasons described earlier, there has been a trend towards using FRSC instead of the traditional mesh reinforced sprayed concrete. The general design process of a sprayed concrete lined tunnel is the same, irrespective of whether the reinforcement is in the form of bars, mesh or fibres. This process is described in detail elsewhere (ICE 1996, BTS 2004; Thomas, 2009). Due to the limited bending capacity of FRSC, there may be cases (such as junctions) where the FRSC may need to be supplemented with additional bar or mesh reinforcement. Soft ground can be considered to behave as a continuum that is amenable to design using various analytical tools and numerical models.

Analytical tools are often used – especially in the early stages of design - to estimate lining thicknesses and reinforcement. These assume that the lining is a linear elastic material so no account is taken of the ductility provided by FRSC. The same is true of mesh reinforced linings. The results may be plotted on an N-M (axial force-bending moment) diagram and this can be modified to account for the additional tensile capacity provided by FRSC (e.g. using, for example, MC2010 – Thomas, 2014). Various guidance documents (eg. TC162-TDF, 2003; TR63, 2007; CNR DT 204, 2007) exist for including the benefits of fibres in concrete design, of which MC2010 is possibly the most comprehensive.

A more detailed discussion of design codes for FRC can be found elsewhere (e.g. ITAtech, 2016). The principles outlined in MC2010 represent basic models of FRC behaviour which may be supplemented as further information about post-crack behaviour becomes available on a project. For example, MC2010 provides no guidance on time-dependent changes in the post-crack performance of young sprayed concrete, or long–term time-dependent aspects of performance (such as crack width increase with time) that affect both steel and macro-synthetic FRSC (Kusterle, 2016; Garcia-Taengua et al, 2016, Plizzari and Serna, 2018). Designers are required to use their own judgement and calculations when accounting for factors not explicitly addressed in MC2010.

Numerical models have the ability to explicitly simulate many of the complicating features of tunnels, thereby overcoming the limitations of analytical tools. Normally the lining is modelled using linear-elastic beam or plate elements so, in this context, no difference exists between the modelling of normal concrete and FRSC. A general discussion of the issues related to modelling of sprayed concrete linings within tunnels can be found in Thomas (2009). If the designer would like to model a section of the lining in more detail, considering deformation softening explicitly, this can be done by modelling the lining using zones and applying a strain softening constitutive model to account for the tensile behaviour. Alternatively, plastic

hinges can be added between the linear elastic beam or plate elements to permit additional plastic movement. However, when the results of such analyses are reviewed, the rotations at the hinges must be checked to verify that the strains do not exceed the limits of the FRSC. As before, the results from the models can be compared with the FRSC capacity on an N-M (axial forcebending moment) diagram (Fig. 5). At present, the effects of shrinkage are seldom considered, yet shrinkage is believed to fundamentally affect lining behavior. More development work is needed in this area.

6. CONCLUSIONS

FRSC is a well-established material for ground support with many advantages. FRSC can be used very effectively in both hard rock and soft ground. By virtue of its bond to the ground, FRSC provides intimate ground support and promotes beneficial ground-structure interaction. Structural fibres (steel or macro-synthetic) add toughness and ductility to sprayed concrete. FRSC is not suitable for every situation and this document has described some of its limitations. FRSC is difficult to apply in very wet conditions so additional measures are needed to control water ingress. The limited bending capacity of FRSC may require changes to the design, shape and construction sequence to keep bending moments within the limits of the material. While FRSC is not synonymous with permanent sprayed concrete, it offers many advantages in this respect and FRSC is a very durable material.

7. STANDARDS

ASTM International, Standard A820/A820M, “Standard Specification for Steel Fibers for Fiber-Reinforced Concrete”, ASTM, West Conshohocken.

ASTM International, Standard C1116/C1116M, “Standard Specification for Fiber-Reinforced Concrete”, ASTM, West Conshohocken.

ASTM International, Standard C1550, “Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)”, ASTM, West Conshohocken.

ASTM International, Standard C1609/C1609M, “Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)”, ASTM, West Conshohocken.

ASTM International, Standard D7508/D7508M, “Standard Specification for Polyolefin Chopped Strands for Use in Concrete”, ASTM, West Conshohocken.

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Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Appendix A. Supplementary material

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DESIGN & CONSTRUCTION OF PEDESTRIAN ACCESS TUNNELS BELOW AN EXISTING OPERATIONAL METRO TUNNELS & STATION

SENTHILNATH G.T.
M.Eng, M.Sc, CPEng, MIEAust, NER

ABSTRACT

A new underground station is added into Outram Park Station as a part of Thomson Line MRT/metro in Singapore and this is built next to two operating metro lines. Once the underground station is completed, it is required to connect this infrastructure with the platforms of existing metro lines for the passengers to interchange between the metro lines. This paper presents design and construction issues of two such pedestrian linkway access tunnels built for this purpose. Two tunnels with cross-sectional area ranging from 50 m2 to 85 m2 are excavated below an existing operational rail tunnel (with vertical clearance of less than 2.7m) and below an operational metro station. The tunnel excavation below the operational metro tunnel is carried out using pipe roof method and the tunnel excavation below the existing station base slab is carried out by underpinning the structure as the excavation progressed. This paper gives an insight into the design considerations, construction methodology and summarizes the learnings based on instrumentation and monitoring results.

***Keywords** : Pedestrian Linkway, Pipe jacking, excavation methods, underpinning*

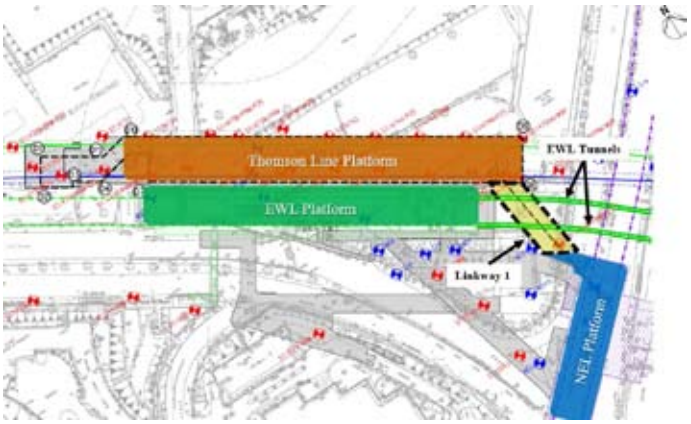
1. INTRODUCTION

With the construction of new underground public transport systems, there is a need to construct Linkway tunnels to integrate them with the existing transport network and to link them with accessible entrance and exit locations. Often, the preferred construction method for such Linkway tunnels are cut and cover constructions with temporary excavation retaining walls and strutting. However, in a densely populated area, this would require multi-staged traffic diversions and advance utility diversions. In some cases, the Linkway tunnels can also be below existing buildings making it impossible to carry out cut and cover construction method.

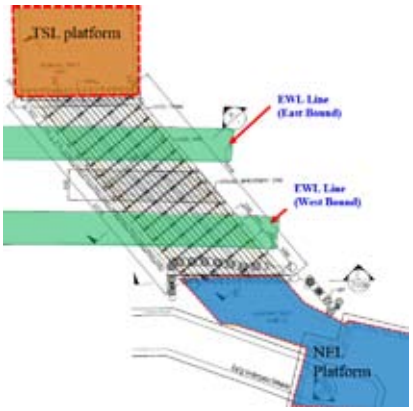
The pedestrian Linkway tunnels discussed in this paper are part of infrastructure connecting the new Outram Park Station in Thomson Line (TSL) to existing Outram Park stations in East-West Line (EWL) and North-East Line (NEL) metro routes. Linkway 1 is constructed underneath the existing live tunnels to provide access from NEL to TSL station structure. Linkway 2 is constructed underneath the existing station with linking access directly to the platform of EWL by escalators and staircase. Figure 1 and Figure 2 show schematic locations of Linkway 1 and Linkway 2 respectively.

Linkway 1 tunnel has to accommodate pedestrian Linkway of dimension 13.1 m wide and 6.5m deep and this tunnel is constructed below operational EWL tunnels at around 45 degree angle with a minimum vertical clearance of 2.65m. The existing tunnels have an internal clear diameter of 5.3m, built with precast concrete segmental rings of 1.0m width and 225mm thickness. The rings are arranged in 5+1 key configuration with a C40 design concrete grade. The EWL tunnels (EB = East Bound, WB = West Bound) are almost at same level as the tunnels are close to the station. The rail levels are ~91.6mRL, which is approximately 1.548m below their respective tunnel axis.

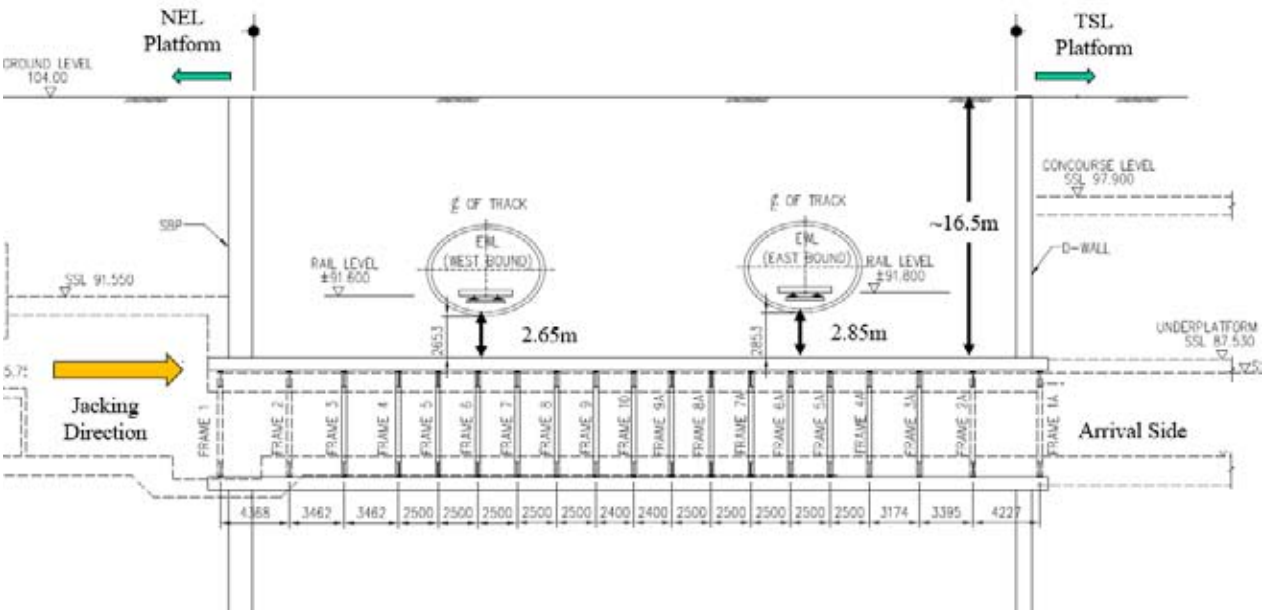
Linkway 2 excavation is carried out below the existing EWL station to create an underground link between TSL platforms and EWL platforms. Due to high volume of passengers anticipated at this interchange, the passenger Linkway will be of 9.3m wide and 6.2m deep. The existing station is seated on a raft foundation which is mostly highly weathered rock of siltstone and mudstone (known locally as Jurong Formation with weathering grade ranging from SIV to SVI). Linkway 2 excavation will begin from TSL platform and progress towards the centre of EWL platform. Along the centre of EWL station slab, two side drift (east branch and left branch) tunnels are excavated to accommodate the staircase and escalator parallel to the platform. The central portion of the base slab with opening in the slab, is a thickened strip that extends the full length of the EWL station. This strip is 1.5m thick and includes two hidden reinforced concrete beams located within its depth. These beams in the raft were designed in 1990's to act as ground bearing strip footings to the internal station columns sitting exactly above this EWL central station slab.



a) Overview of Linkway 1 location (plan)



b) Plan of Linkway 1 location



c) Longitudinal section of Linkway 1 and EWL tunnels undercrossing

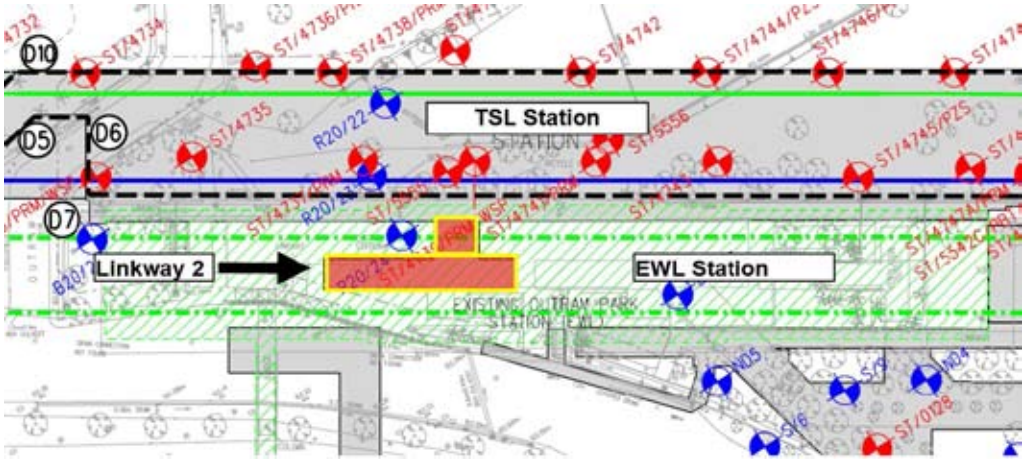
Fig. 1 : Schematic location of Linkway 1

2. GEOLOGY AND GROUND CONDITION

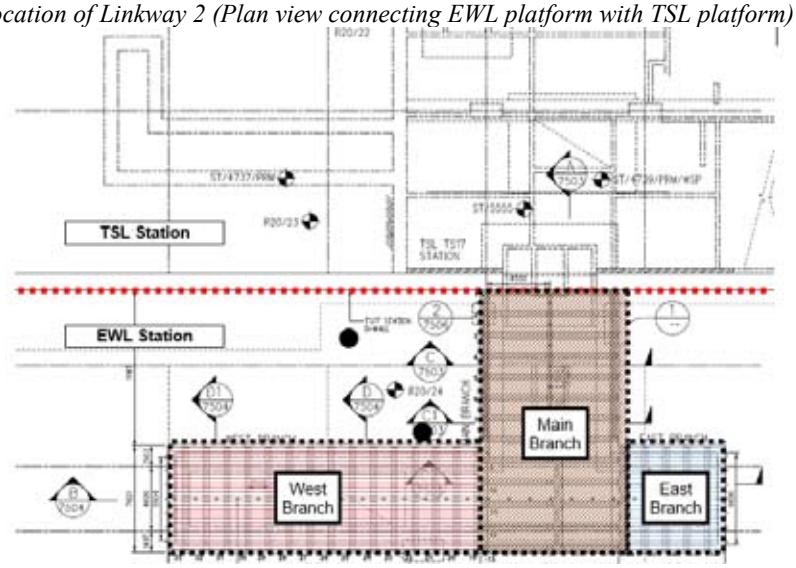
Both the proposed Linkway tunnels will be excavated in Jurong formation. Nearest boreholes indicate that the tunnel excavation will encounter weathering grade SV and SIV. Few boreholes suggest slightly sandy / gravelly silt at the crown. During the soil investigation it was noted that weathered layers of sandstone in combination with water pressure could lead to instability of excavation. Within the first 3-4m from ground, there is a thin layer of fill material underlain by fluvial deposit and occasional pockets of organic clay. Below this is a completely weathered zone of Jurong formation. The properties improve generally with depth. The Jurong formation is of late Triassic to early Jurassic age and comprises interbedded sandstones, siltstones and mudstones that in location, dip steeply. The rock mass is dominated by the fractures and depending upon the condition of the fractures varied flow of water ingress are experienced. Under wet condition, the rock mass is found to deteriorate rapidly. Rock strength was variable ranging generally from 0.5 MPa to 5 MPa. For design purposes, following parameters were used.

Table 1 : Design Parameters

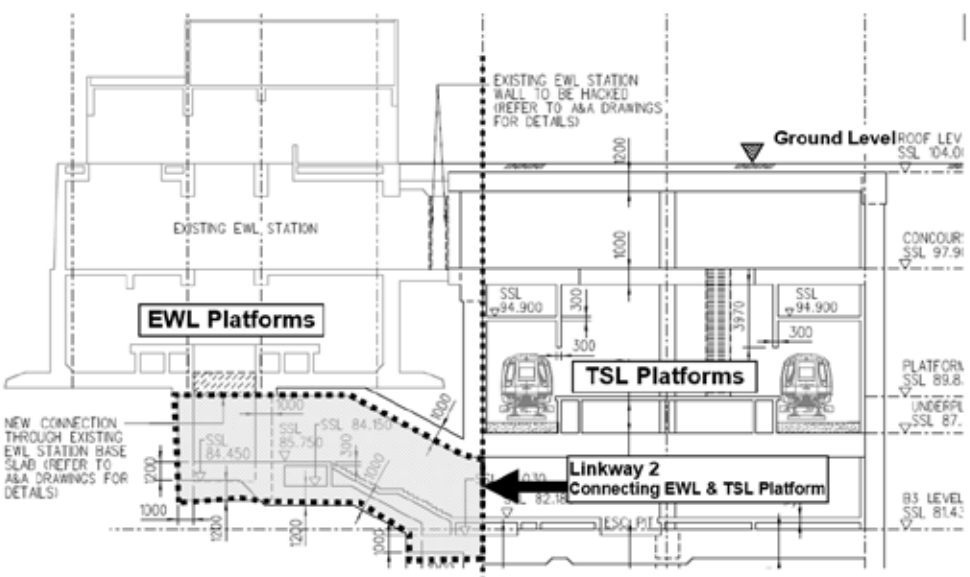
	Jurong Formation			
	SVI	**SV**	**SIV**	**SIII**
Unit weight [kN/m^3]	20	21	24	25
Effective angle of friction [degree]	31	33	33	36
Effective cohesion [kPa]	0	10	15	30
Undrained shear strength cu [kPa]	100	250	NA	NA
Young's Modulus E' [MPa]	33	208	250	400



a) Location of Linkway 2 (Plan view connecting EWL platform with TSL platform)



b) Plan of Linkway 2: Main Branch and side branches (plan view)



c) *Longitudinal section of Linkway 2 (along main branch)*

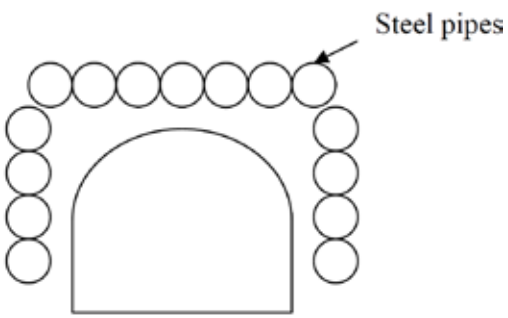
Fig. 2 : Schematic location of Linkway 2

Field permeability test indicated permeability in the range of 1x10-7 m/sec for the weathered Jurong Formation.

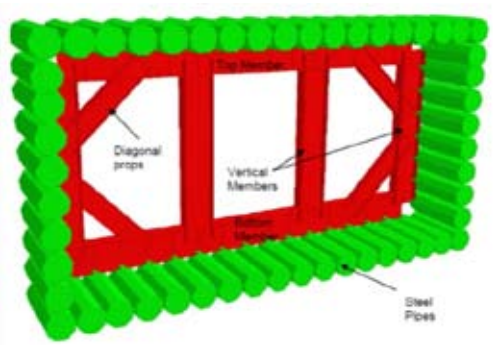
3. DESIGN CONCEPT

3.1 Linkway 1

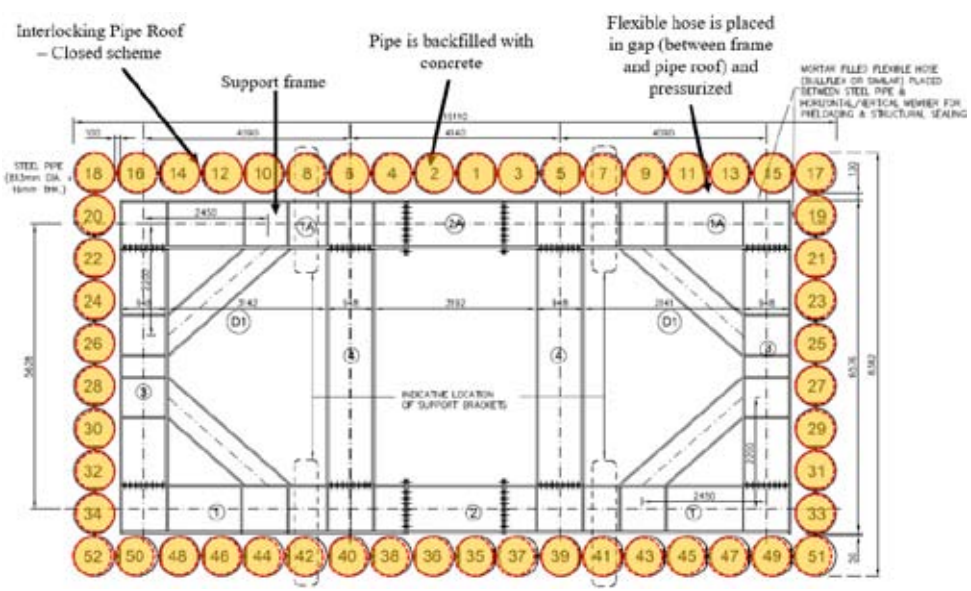
Tunnel displacement causes deformation of the ground between the tunnel and the subsurface infrastructure and hence results in subsidence at the underground infrastructure. In this case, the clearance between the Linkway tunnel and the existing EWL tunnels is less than 3m hence there is a high risk of EWL tunnel damage if there is any face collapse during Linkway 1 excavation. To address this concern, based on experience in Singapore (Badurdeen and Senthilnath 2017), it was proposed to install pipe-jacked steel pipes to form a reinforcement layer between the EWL tunnel and the Linkway 1 excavation tunnel. This formed a high stiffness boundary and provided an advance face support to enhance the stability of the tunnel prior to tunnel excavation. The steel pipes of 0.813m diameter was proposed to be jacked over the projected tunnel crown to form a layer of reinforced layer which prevents the flow of soil into the tunnel at the periphery and at the face when the Linkway 1 excavation is in progress. Subsequently, steel rib/ steel frame supports are installed inside the Linkway tunnel at 2.5m spacing as the tunnel excavation progressed. Typical roof reinforcement with pipes is shown in figure 3(a). Lim et al. (2000) observed that around half of the settlement during the tunnel excavation in Jurong formation could be attributed to elastic compression during water drawdown. Hence in our design scheme, we proposed a water tight system with interlocking pipes (provided with T-clutch joints) – shown in Figure 2 (b) and (c) and ground water recharge wells were planned along the excavation length to recharge the ground water drawdown due to excavation as a contingency measure.



a) Typical pipe roof reinforcement: Gate type arrangement (Tan et al 2003)



b) Pipe roof arrangement adopted in Linkway 1



c) Details of adopted pipe roof arrangement and steel frame supports

Fig. 3 : Proposed water-tight piperoof system

3.1.1 *Installation of Pipe-Roof and tunnel excavation*

Once all the interlocking pipes (52 nos) are jacked, first frame is installed within the shaft – (acting as a portal frame) and the excavation of tunnel was started within the box structure created by the interlocking pipes. The subsequent support frames are installed as the excavation progressed along the Linkway tunnel. The pipes at the roof level (pipe no. 1 to 18) act as a continuous beam supported at the frame locations. In intermediate stage of post-excavation (and just before the installation of frame), the roof level beams experience the maximum bending moment and deflection. The last installed frame experiences the maximum loading as the excavation proceeds. With the next frame installation, there was slight unloading of the previous frame. This was anticipated in the design and hence a staged excavation model was used to simulate the complete excavation cycle. Figure 4 presents typical bending moment distribution in the top pipe-roof in a given stage.

The usual approach of Rabchiwich / NATM is to utilize the strength of the soil. But to utilize the strength, we need to mobilize some movement in the ground. However, in our case, the allowable limits are very strict hence the member sizes are relatively heavy and also happen to take more loads as stiffer elements tend to attract forces.

3.1.2 *Impact on existing tunnels during tunnel excavation*

To assess the ground deformation due to Linkway 1 excavation, a 3D model numerical analysis was carried out. Since the analysis was on a 3D model, no ground relaxation was applied prior to any excavation step or activation of ground support elements. The model includes simulation of each excavation step and subsequent installation of steel frames. To arrive at reasonably conservative estimate of construction impact on existing tunnel, the ground support of the excavation face in Linkway 1 (such as glass fibre face dowels, sprayed concrete, etc) was not considered in the analysis model. The analysis was carried out with an assumption that the Linkway 1 excavation will be carried out from both sides of the tunnel however during the final stages, as the unexcavated soil between the either face of excavation is reduced, the construction sequence is simulated (and was proposed in design drawings) for excavation from one side. Figure 5 presents the numerical model and visualization of frame location / unexcavated soil during the final rounds of excavation.

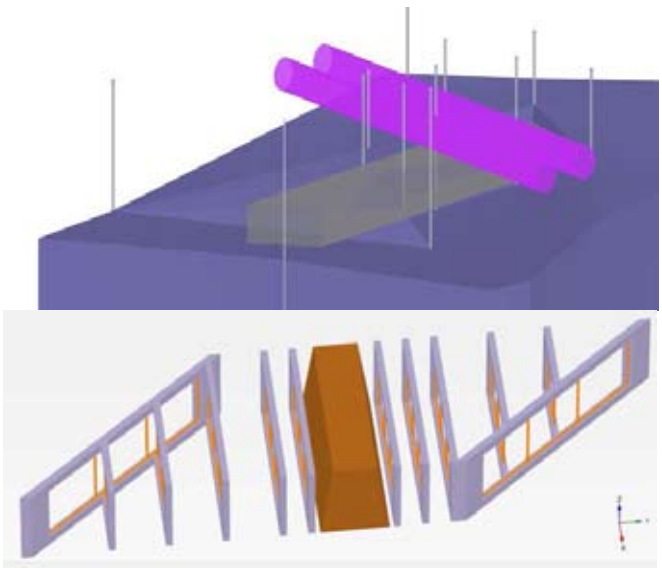


Fig. 5 : Numerical model and excavation sequence visualization

In addition to assessing the ground displacement due to Linkway 1 excavation, it was important to understand the behaviour of the tunnels and change in diameter of tunnel (if any) due to Linkway 1 excavation. Following structural limit states were studied in this project to ensure that the existing tunnel linings are within structural limits.

- *Ovalisation* : To check distortion involving increase in vertical diameter and decrease in horizontal diameter. This causes increase in bending moments in the concrete segments and causes rotation at the segment joints.
- *Squat* : To check distortion in EWL tunnel segments involving increase in horizontal diameter increase and decrease in vertical diameter. The effects are like ovalisation. However, in this case, the tunnels

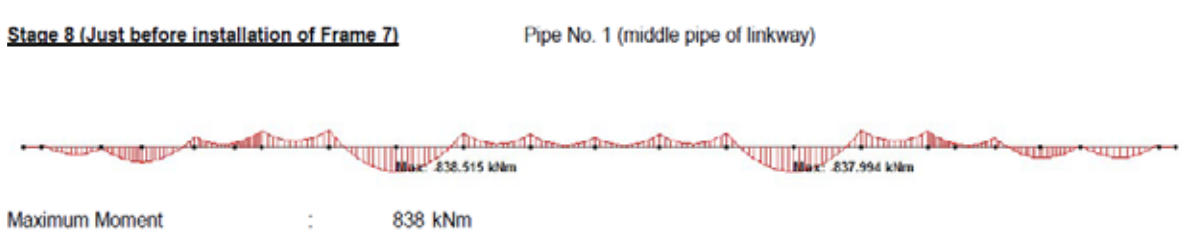
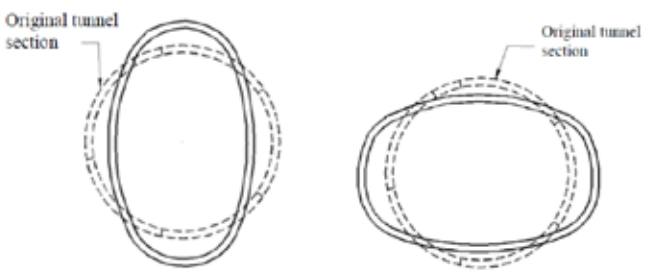


Fig. 4 : Typical bending moment distribution in the roof pipe

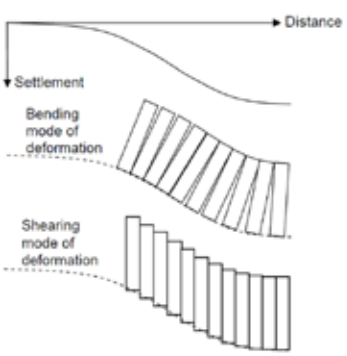
had already squatted when taking up the ground loads. Therefore, the remaining tolerable movement was the difference between the existing squat and the tolerable squat.

- *Stepping & Opening of Circular Joint* : In the longitudinal profile, the tunnel tends to have two types of deformation modes a) Bending mode of deformation and (b) shear mode of deformation. Based on the type of behaviour, it is possible to have either opening of joints or stepping between the segments.

Figure 6 illustrates the different structural limit states described above for damage / impact assessment of existing tunnels.



a) Ovalisation and Squat



b) Stepping and opening of circular joint

Fig. 6 : Structural Limit states considered for existing tunnels

3.1.3 *Jacking force*

Minimum jacking force is calculated based on minimum required force to overcome the skin friction along the pipe contact surface area and maximum jacking force is calculated based on pressure transmission along the steel pipe and joint configuration between the pipes using ATV nomograms (Röhner et al 2010). Figure 7 illustrates the components considered for estimation of jacking force.

3.1.4 *Factors influencing face stability during pipe jacking*

Face stability calculations are required to determine the likelihood of the excavated face moving or collapsing into the void created during excavation. Face stability depends upon the type and variability of the ground being excavated, the ambient stress and ground water conditions, the rate of advancement and the construction methods adopted. The methods for calculating stability of tunnel faces are generally well-established and are adjusted for pipejacking / microtunnelling in terms of rate of advance, size of face and face support conditions. Face pressure of 1.2 bar to 2.2 bar is recommended based on the soil condition at face and the depth of the pipe.

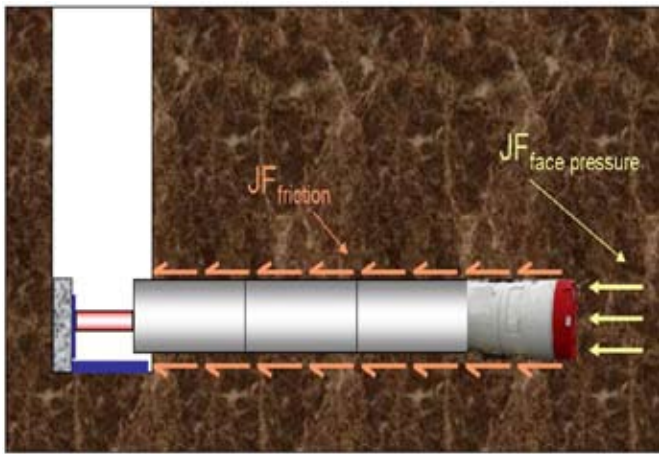


Fig. 7 : Jacking force estimation

3.1.5 *Machinery & Equipment*

Because of the size of the support frames, the clearance between the vertical members had to be carefully checked for excavation, turning and reversing of the excavators along the Linkways. During the excavation, the diagonal member in the support frames were sequenced to allow the movement of vehicles. Figure 8 shows the tight site constraints faced during the excavation.

Fig. 8 : Site constraint for movement of excavators

To ensure proper contact between the frames and the pipe-roof, grout bags above the frames were included as design element. In addition, jacking up of the frame using hydraulic jacks and temporary additional vertical members were carried out. The frames are jacked up and shim plates are installed in the gap created by jacking and the bolts are tightened and sealed. The temporary additional vertical members are reused for next frame jacking.

3.1.6 *Monitoring and instrumentation of Linkway 1*

To clearly understand the actual ground response during Linkway 1 excavation, a series of real time prism targets were installed within the operational tunnels and were monitored by automated robotic total stations installed within the tunnel. Each monitoring array consists of five prism targets (as indicated in Figure 9) to understand the movement of tunnel as well as to understand deformation in the tunnel shape (if any).

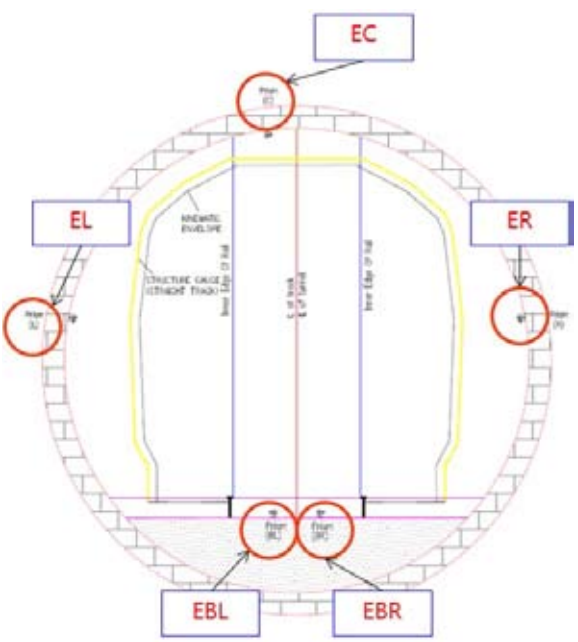
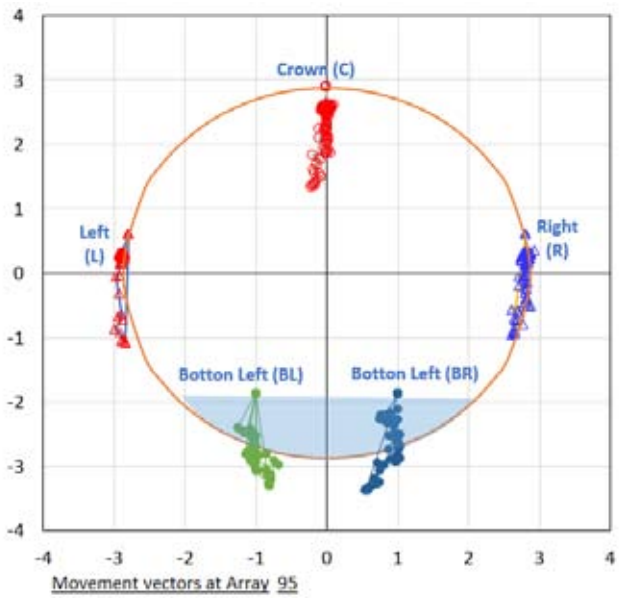
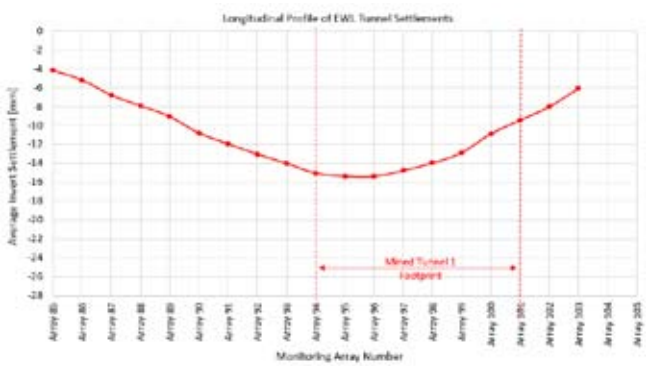


Fig. 9 : Prism monitoring locations

An automated site specific data visualization tool as proposed by Senthilnath G T (2017) was developed for this project to represent the change in shape and longitudinal profile of the tunnel as the Linkway1 tunnel excavation proceeds. The data visualization tool indicated uniform settlement of all the points (in a given cross section) which indicates there was no ovalization or squatting of the tunnel. Hence longitudinal profile settlement (i.e stepping and opening of circumferential joints) was critical during the Linkway 1 excavation. Figure 10 (a) and (b) presents typical prism readings in cross-section and longitudinal section of the tunnel alignment.



(a) Tunnel displacement in cross section



(b) Tunnel displacement in longitudinal section

Fig. 10 : Tunnel movement visualization

3.2 Linkway 2

As shown in Figure 11, the Linkway 2 excavation starts from the TSL platform box (by creating an opening in the D-wall) and the excavation proceeds below the existing EWL station base slab. Excavation was planned with steel support frames installed to underpin the base slab as the excavation progressed. However, as the excavation height is more than 10m, the excavation face is divided into top heading and bench excavation. The load transfer of underpinning members during bench excavation is ensured using a truss arrangement as shown in Figure 12 (the truss arrangement is not shown in the 3d render for clarity) The steel frames along with shotcrete sprayed on side walls and invert resist the lateral earth pressure and uplift pressure from the invert in Linkway 2 in the temporary stage. To check the composite behaviour of steel sets with shotcrete lining, the forces from the analysis are cross checked against the capacity curve derived based on Carranza-Torres and Diederichs (2009).

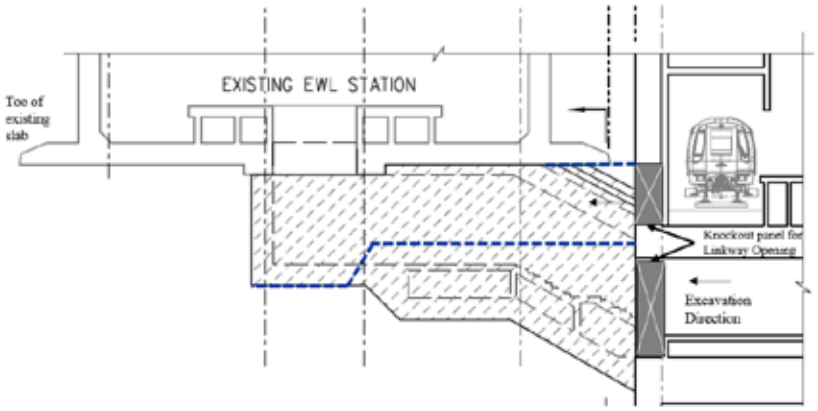


Fig. 11 : Excavation of main branch of Linkway 2

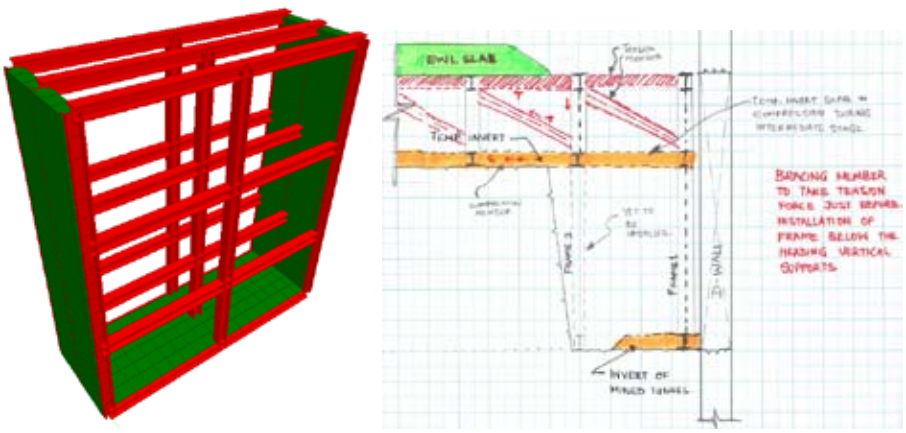


Fig. 12 : 3D render of underpinning system and load transfer system in truss member

For simplicity, contribution of wire mesh in the bending capacity of the shotcrete lining is not considered. The forces obtained from composite – unit width model is split between Steel frame and shotcrete based on their EI and EA ratio for the design capacity checks.

3.2.1 *Impact on station slab*

The Linkway 2 excavation is carried out below EWL structure which was built in 1980s. The structure was designed using the former British code of practice CP110 which was later superseded by BS 8110. Despite this, current design checks in the EWL slab due to Linkway excavation was carried out as per the latest BS 8110. To understand the impact of Linkway construction on the slab, it was necessary to understand the in-situ stresses in the structural members of the EWL station. For this purpose, the client had used Autodesk Robot to create 3D Finite Element (FE) models of the existing EWL station. This model is used to understand the existing condition at various reference sections along the Linkway tunnel.

The additional stresses due to Linkway 2 excavation is estimated based on free span and column loadings on the slab. In addition, to limit the centre stress in the slab, an intermediate vertical support is proposed for the main branch of excavation in temporary case. A 2D PLAXIS analysis is carried out to estimate the maximum ground displacement and slab movement due to mining activity. Without any face support, for 1.5m round length and 7m wide excavation, ahead-of-face ground relaxation is estimated to be in the range of 50 to 55% (based on Panet and Sakurai formulation). However, because of face bolting, ground relaxation of 30% is considered in the FE analysis. Maximum EWL slab displacement is estimated to be less than 5mm.

3.2.2 *Face Support and Grouting*

Design of tunnel face reinforcement is made to limit the ahead of face ground relaxation. The tunnel face reinforcement ensures that the unsupported span of EWL slab is maintained to a minimum length of less than 2.5m (1.5m round length + 1.0m allowance for slope/ ground relaxation zone). Stability of face is based on limit equilibrium of a failure mechanism that consists of a wedge and a prism. Mohr-Coulomb failure condition is assumed for the ground. Inclination of the slip plane depend on the shear strength and on the stratigraphy of the ground and has been determined iteratively. The slip plane which results in maximum support requirements is considered for the face support requirements. During

the ongoing excavation (without anchorage plates), the supporting effect of the face bolts relies solely on the strength of the bond between grout and soil. If anchorage plates are present (during an excavation standstill) and the bond length inside the wedge is insufficient, the bolt force will be transmitted to the ground through the plates. Minimum number of face support bolts are calculated based on limit equilibrium check and design based on nomograms by Anagnostou et al (2007).

3.2.3 *Monitoring and instrumentation of Linkway 2*

To monitor the ground performance during Linkway 2 excavation, a series of settlement markers were installed on the EWL base slab and in addition, strain gauges were installed on the alternate support frames to measure the axial loads coming on the steel sets. Except for the T-Junction (between the main branch and the side branches) axial loads in all the support frames were well within the prediction. This could be attributed to conservative approach used for loading consideration on the EWL slab to avoid utilizing the EWL slabs to their structural limits.

4. DISCUSSIONS AND CONCLUSION

Experience gained during the construction of the Linkway confirmed that if dry conditions are maintained, the Jurong formation (Sandstone, siltstone and mudstone with varying degrees of weathering) is a good material for tunnel excavation with advance roof support. From the observation of the ground performance during tunnelling, it became very clear that Jurong formation with high SPT blow counts is fairly incompressible and as anticipated, it experienced limited plastic settlements during the load redistribution in the ground. The actual ground movements due to pipe roof installation slightly exceeded the design prediction however the measured settlement during the Linkway excavation agree well with the prediction. The instrument readings suggested that influence of Linkway 1 excavation had a temporal variance as the excavation progressed. This suggests that a detailed analysis of response at each stage needs to be studied for a project of similar nature. The final ground response is not necessarily the most critical movement during the Linkway 1 excavation.

Currently temporary works for both the Linkway tunnels are completed and permanent works RC casting is in progress. For the successful completion of construction activities within the railway protection zone, very detailed sequence planning, periodical interface meeting with other designers and full compliance with requirements of the Code of Practice for Railway Protection works and/or railway operator was of utmost importance. This has ensured minimum disruption to the operation of the railway system.

Acknowledgement

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I-SYSTEM: INDEX OF GROUND-STRUCTURE DEFINITION, APPLICATIONS, AND UTILISATION IN DESIGN/PRACTICE

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ABSTRACT

An optimised geotechnical/geomechanical design approach includes empirical, analytical, seismic, and observational stages. Empirical and observational parts of the design are very important in initiation of the approach and in finalisation of the judgements containing the derivation of the ground behaviour, identification of ground hazards, and determination of support systems. Nowadays engineering classifications are main part of the empirical/observational stages of the design for human built structures in ground; though, they have limitations in application. I-System is a new classification that is developed to cross the limitations that are involved with other empirical classifications. It is smartly applicable for any types of civil/mining/oil/gas structures in ground comprehensively including but not limited to abutments of bridges/dams, caverns, deep/shallow foundations, galleries, metro stations, shafts, slopes, trenches, tunnels (any type or method), wells, etc. It considers easily derivable geomechanical, geotechnical, geostructural, geophysical, geohydrological, geometrical, and dynamic properties and configuration of ground in relation to the structure together with the method of excavation/ construction. It is based on over 22 years' research and verification in design and construction of surface, semi-surface, and subsurface/underground structures in any ground medium irrespective of being rock and/or soil.

***Key Words** : classification, design, GCD, ground, I-System, index, rock, rock mass, soil, SRH, structure, support.*

1. INTRODUCTION

Design approach for structures in ground includes 4 important stages (Bineshian et al., 2019) as shown in Figure 1. The design methodology should pass empirical, analytical, seismic, and observational procedures to get the optimised design badge of good for construction, while empirical and observational parts are playing very crucial role and determinative factors for this purpose. Figure 2 shows the design procedure for structures in ground and required data for the same.

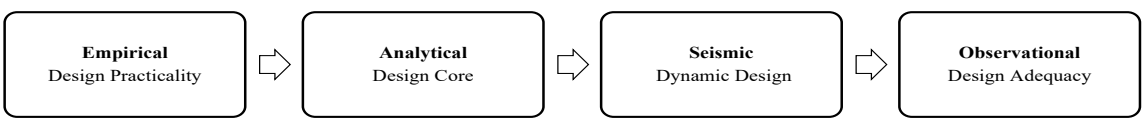

Figure 1. Design approach for structures in ground.

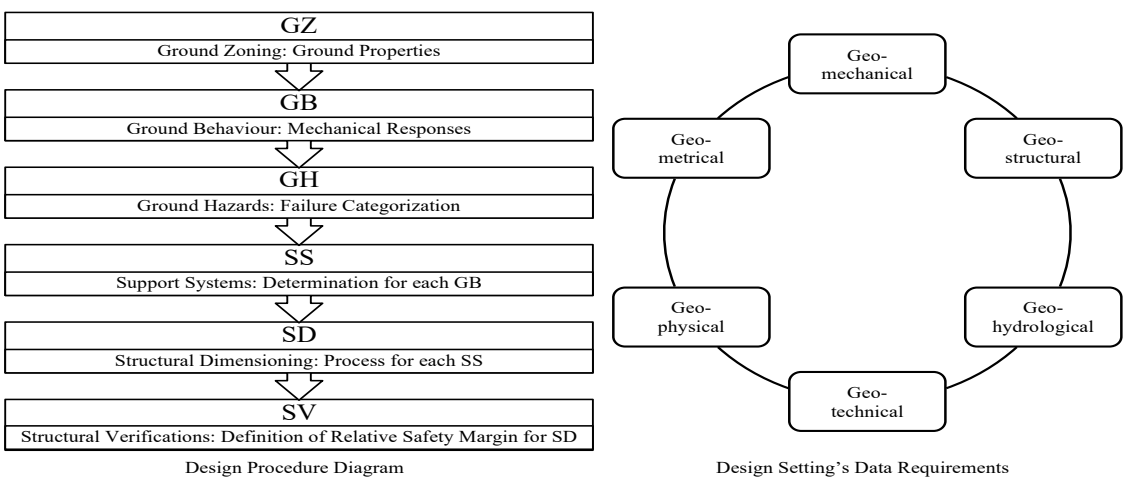

Figure 2. Design procedure for structures in ground

Engineering classifications are the main part of the empirical and observational design elements in a healthy design approach for structures in ground (Bineshian, 2012, Bineshian and Ghazvinian, 2012a and 2012b). Comprehensiveness and practicality of the engineering classifications are essential in application in NATM (New Austrian Tunnelling Method), NMT (Norwegian Method of Tunnelling), SEM (Sequential Excavation Method), SCL (Sprayed Concrete Lining Method), etc. nonetheless existing engineering classifications come with limitations in use for both rock and soil. Limitations, inaccuracy, and imprecision involved with existing classifications make engineers uncertain in determination and dimensioning of structures specially when they encounter ground complications (Bineshian, 2014, Bineshian, 2017, Bineshian et al., 2019).

RMR and Q are popular existing classifications developed by Bieniawski (1973) and Barton et al (1974) respectively (Bieniawski, 1976, Barton et al., 1974). They are only applicable for rock medium. RMR is proposed for surface and underground works but its water pressure consideration is doubtful, quantification of joint orientation is uncertain, and the effect of water on rock mass is inattentive (Bineshian et al., 2013). Q is proposed for tunnels merely, which comes with several limits in input parameters including discontinuity's aperture, orientation, persistency, and size and rock strength.

Additionally, there is a shortcoming in most existing classification systems when observed rock mass characteristics are used to estimate the conditions for planning without including input of the excavation method of structure. An excavation damage factor or similar should be applied, but none of the empirical or other tools in rock engineering makes use of this (Palmstrom and Broch, 2006).

I-System or in a short form (I) as a classification system is developed and verified based on 22 years' research in 4 continents in design and construction of mountain and metro tunnelling, dam works, road works, slope stabilizations, bridges, and mining to address and resolve the aforesaid issues with existing classifications. Table 1 demonstrates a summary for application of RMR and Q compared to the I-System.

Table 1. Application summary for popular existing engineering classifications compared to the I-System.

Applications	Media		Structure (Civil, Mining, Oil and Gas)	
	Rock	Soil	Surface	Underground
RMR (Bieniawski, 1973)	✓	✗	–	✓
Q (Barton et al., 1974)	✓	✗	✗	✓
I-System (Bineshian, 2019)	✓	✓	✓	✓

✓ Applicable
✗ Partially Applicable
– Not Applicable

The I-System provides precise prediction of ground behaviour together with recommendations on required Support System/s (SS), Excavation Technique/s (ET), Instrumentation Technique/s (IT), Prevention Technique/s (PT), and Forecast Technique/s (FT) followed by Design Remark/s (DR).

Application of the I-System is verified against different types of ground and complicated challenging condition and scrutinised by several cases in over 42 projects and therefore outputs for each case is optimised in comparison to analytical, numerical, and observational methods to compensate the demerits of other classifications and strengthen its comprehensiveness.

In this paper definition of the I-System in details together with its applications and utilisation is provided.

2 DEFINITION AND MAIN CONCEPT

Providing a solution to engineers in their challenges with complicated ground conditions is the key perception and approach in development of this new all-in-one classification for ground in accord with real condition and delivering practical recommendation/s. Also, it had been in mind to provide a trusted utility for empirical part of design. In development of this classification system, drawbacks and limitations of other classifications including RMR and Q are properly addressed and consequently fixed (Bineshian, 2019).

The new classification is conceptually different from any existing classifications due to its applicability for different conditions of ground and structures including but not limited to abutments of bridges/dams, caverns, deep/shallow foundations, galleries, metro stations, shafts, slopes, trenches, tunnels (any), wells, etc. and its comprehensiveness in providing precise prediction of ground behaviour based on several geomechanical hazards (failure mechanisms) encountered in civil, mining, and oil and gas projects.

It is scrutinised and verified in different challenging

ground conditions by yielding suitable estimation of ground quality, intelligent primary and final SS (Support System/s) determination, and recommendations on right ET (Excavation Technique/s) for encountered condition, proper IT (Instrumentation Technique/s) for monitoring, and appropriate PT (Prevention Technique/s) against possible failures or collapses. It also suggests verified FT (Forecast Technique/s) to predict the ground condition during excavation and provides practical DR (Design Remark/s) that is helpful in understanding of ground behaviour, failure mechanism/s, load configuration, and dimensioning of structure.

It is the first ever classification, which is applicable for both rock and soil that considers problematical and structural configurations, opening's scale effect, earthquake effect, and excavation technique's impact. Moreover, it is also first ever classification that carefully provides prediction for special ground behaviour including but not limited to Squeezing/Swelling/Heaving (SSH) as a kind of Time Dependent (TD), Visco-elasto Plastic (VP), fully plastic, gravity driven (GD), and Burst Prone (BP) grounds.

The I-System is designed to have five indices - which defines the mechanical behaviour of ground in relation to the structure irrespective of type of the media being rock or soil based on easily derivable most important geomechanical, geotechnical, geostructural, geohydrological, geophysical, and geometrical parameters/properties and two indices of influencing parameters that have impact on the structure including Dynamic Forces (DF#) and Excavation Technique (ET#) (Eq. 1). This comprehensive

$$(\mathrm{I}) = (\mathrm{A_i} + \mathrm{C_i} + \mathrm{H_i} + \mathrm{P_i} + \mathrm{S_i}) \times \mathrm{DF_i} \times \mathrm{ET_i} \quad (1)$$

$$\mathrm{A_i} = (\mathrm{a_{dn}} + \mathrm{a_{ds}} + \mathrm{a_{di}}) \times \mathrm{a_{da}} \times \mathrm{a_{dd}} \times \mathrm{a_{dp}} \times \mathrm{a_{dr}} \quad (2)$$

$$\mathrm{C_i} = \mathrm{c_{pc}} \times \mathrm{c_{sc}} \quad (3)$$

$$\mathrm{H_i} = \mathrm{h_{gc}} \times \mathrm{h_{gs}} \quad (4)$$

$$\mathrm{P_i} = \left[\mathrm{p_{cc}} + \mathrm{p_{dc}} + (\mathrm{p_{ps}} \times \mathrm{p_{pm}})\right] \times \mathrm{p_{bw}} \;\&\; \mathrm{p_{bw}} = f(\mathrm{V_p}, \mathrm{V_s}) \quad (5)$$

$$\mathrm{S_i} = \mathrm{s_{cs}} \times \mathrm{s_{se}} \quad (6)$$

$$\mathrm{DF_i} = f(\mathrm{PGA_{SD}}, \mathrm{ERZ}, \mathrm{MSK}) \;\&\; \mathrm{PGA_{SD}} = f(\mathrm{PGA}, \mathrm{G_S}, \rho, \mathrm{d}) \quad (7)$$

$$\mathrm{ET_i} = f(\mathrm{ET}, \mathrm{PPV}) \quad (8)$$

A_i is the Armature Index as ground's skeleton armature, which is designed here to model almost most of important geomechanical aspects of rock mass media through the discontinuity properties of the ground. A_i has 20 score out of 100. Table 2 defines parameters of A_i.

Table 2. Armature Index (A_i): $\mathbf{a_{dn}}$, $\mathbf{a_{ds}}$, and $\mathbf{a_{di}}$, $\mathbf{a_{da}}$, $\mathbf{a_{dd}}$, $\mathbf{a_{df}}$, and $\mathbf{a_{dp}}$.

dn	$\mathbf{a_{dn}}$	ds	$\mathbf{a_{ds}}$	di	$\mathbf{a_{di}}$ [IF ($\mathbf{a_{dn}} \geq 2.50$ & $\mathbf{a_{ds}} \geq 4.00$) THEN↓ ELSE 0]
0 - 9	10.00	0	10.00	N/A || Jointless	0.00
10 - 14	7.50	1	9.00	0 - 10	-1.00
15 - 19	5.00	2	7.00	11 - 30	-1.50
20 - 24	2.50	3	4.00	31 - 60	-2.00
≥ 25	0.00	≥ 4	0.00	61 - 90	-2.50
N/A || Jointless	0.00	N/A || Jointless	0.00		

da	$\mathbf{a_{da}}$	dd	$\mathbf{a_{dd}}$	df	$\mathbf{a_{df}}$	dp	$\mathbf{a_{dp}}$
N/A || Jointless	1.00	N/A || Jointless	1.00	N/A || Jointless	1.00	N/A || Jointless	1.00
Tight	1.00	Unweathered/Unaltered	1.00	High Friction - Rough/Uneven	1.00	< 0.90 × D	1.00
Semi-Tight	0.95	Semi-Integrated	0.95	Moderate Friction - Nonsmooth	0.95	≥ 0.90 × D	0.90
Open	0.90	Weathered/Altered	0.90	Low Friction - Smooth/Even	0.90		

$\mathbf{a}_{da}$	Factor related to da influencing A_i,
$\mathbf{a}_{dd}$	Factor related to dd influencing A_i,
$\mathbf{a}_{df}$	Factor related to df influencing A_i,
$\mathbf{a}_{di}$	Score related to di as an effect on A_i,
$\mathbf{a}_{dn}$	Score related to dn of A_i,
$\mathbf{a}_{dp}$	Factor related to dp influencing A_i,
$\mathbf{a}_{ds}$	Score related to ds of A_i,
da	Discontinuity Aperture based on the opening of the discontinuities,
dd	Discontinuity Disintegration based on weathering or alteration of surface of the discontinuities,
df	Discontinuity Friction based on the friction condition of the discontinuities,
di	Discontinuity Inclination based on the dip angle of the most unfavourable discontinuity,
dn	Discontinuity Number/s based on number/s of individual discontinuities per m of scanline; either horizontal, vertical, or average,
dp	Discontinuity Persistency, and
ds	Discontinuity Set/s reflecting the number/s of set/s of the discontinuities.

It is to be noted that when **a**dn and **a**ds are zero then the score for **a**di also to be assigned as zero. It is logic, because it happens when the number of discontinuities is ≥ 25 and number of discontinuity sets is ≥ 4, which in this case inclination for the discontinuities are not easily derivable.

C_i is the Configuration Index as ground's problematical and structural configuration that covers important problematical geostructural features of rock or soil. C_i has 20 score out of 100.

Table 3 defines parameters of C_i.

Table 3. Configuration Index (C_i): $\mathbf{c}_{pc}$ and $\mathbf{c}_{sc}$.

pc	$\mathbf{c}_{pc}$
Homogeneous || Isotropic || Jointless || Granular	1.00
Fractured - Slightly	0.95
Faulted - Brittle Single	0.90
Folded - Anticline/Syncline	0.85
Folded - Dome/Basin	0.80
Fractured - Moderately	0.75
Faulted - Graben/Horst	0.70
Folded - Complex/Plunging	0.65
Fractured - Highly	0.60
Faulted - Brittle/Ductile Multiple	0.55
Differed - Unconformities	0.50
BP - High Stress Zone; High Overburden	0.45
Tectonised - Complex of Geostructures	0.40
Sheared - High Shear Stresses - e.g. Mylonite	0.35
TD - Flaky/Micaceous/Cleated - Coals, Mudstone, Phyllite, Schist, Shale, Slate, Young Sandstones	0.30
VP - Incremental-Sudden Large Shear Movement, Cyclic Mobility-Flow Liquefaction, Limited-Continuous Debris Discharge - Flowing/Overrunning	0.25
sc	**$\mathbf{c}_{sc}$**
Massive || Grainless Continuum	20.00
Layered (> 100 cm)	17.00
Layered (100 - 10 cm)	15.00
Clastic Breccia/Conglomerate	13.00
Layered (< 10 cm)	11.00
Foliated/Laminar/Platy	9.00
Coarse Grained Skeleton	7.00
Cohesive Matrix Skeleton	4.00
Single Grained Skeleton - Dense Texture	2.00
Single Grained Skeleton - Loose Texture	0.00

$\mathbf{c}_{pc}$	Impacting factor related to pc indicating ground's tectonic state on C_i,
$\mathbf{c}_{sc}$	Score related to sc as an effect of ground's texture, fabric, and structure on C_i,
pc	Problematical Configuration, and
sc	Structural Configuration.

H_i is the Hydro Index as ground hydro effect on mechanical behaviour and hydro related properties, which considers GCD (Bineshian, 2017, Choudhary et al., 2019) or Wetness condition and softness due to presence of water in scale of Mohs. H_i has 20 score out of 100.

Table 4 defines parameters of H_i.

Table 4. Hydro Index (H_i): $\mathbf{h_{gc}}$ and $\mathbf{h_{gs}}$.

gc (GCD) or [Wetness]	$\mathbf{h_{gc}}$	gs Mohs	$\mathbf{h_{gs}}$
(≤ 0.99) || [Dry]	20.00	≥ 7	1.00
(1 - 1.99) || [Humid]	19.00	6	0.60
(2 - 2.99) || [Damp]	18.00	5	0.50
(3 - 4.99) || [Moist]	16.00	4	0.40
(5 - 6.99) || [Leak]	15.00	3	0.30
(7 - 9.99) || [Wet]	13.00	2	0.20
(10 - 14) || [Drip]	11.00	1	0.10
(15 - 24) || [Shower]	9.00	Moulded by Light Finger Pressure	0.05
(25 - 49) || [Flow]	6.00	Exuded between Fingers	0.00
(50 -99) || [Gush]	3.00		
(≥ 100) || [Burst]	0.00		

gc Ground Conductivity,
GCD The Ground Conductivity Designation as a criterion to score the hydraulic conductivity of ground (see following for the definition of the same),
gs Ground Softness as effect of hydrosoftness on medium/infilling material based on Mohs Scale,
$\mathbf{h_{gc}}$ Score related to gc using GCD or Wetness diagram as criterion for hydropressure effect on surrounding ground as main part of H_i,
$\mathbf{h_{gs}}$ Impact factor related to gs on H_i,
Wetness A diagram defined here to classify the ground's water content, which is classifying the ground water condition (observational identification) in 11 ranges (see Figure 3).

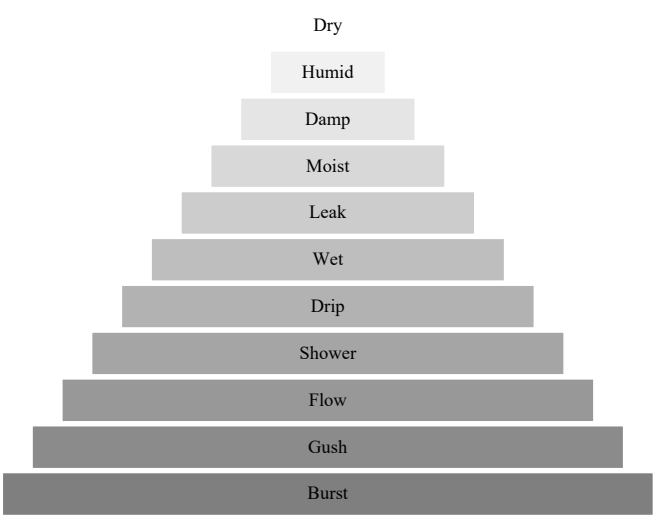

Figure 3. Wetness diagram.

GCD (Ground Conductivity Designation) is an empirical equation based on a simple single stage water injection test for examination of the ground's hydraulic conductivity. It is essentially based on water-intake/ water-loss measurement that is pumped thorough a drilled hole (NX preferred) at any direction/angle before

and after grouting/injection. GCD is developed by author in 2013 - based on Lugeon concept and over 18 years practical verification - as a quick examination technique in practice. It is further examined in 2015 in USBRL project and the results were published by the author for the first time in 2017 (Bineshian, 2017) and later on in 2019 (Bineshian et al. 2019, Bineshian, 2019, Choudhary et al. 2019).

Eq. 9 represents dimensionless empirical equation for GCD. Figure 4 presents schematics for the GCD test setup.

Table 5 shows the classification for ground hydraulic conductivity and ground solidification quality.

$GCD = Q_w / (P_m + L_i)$ (9)

In which,

GCD Ground Conductivity Designation (dimensionless),

Q_w Water flow rate (lit/min),

P_m Maximum water pressure during injection (MPa), and

L_i Length of water injected portion (packed length) of hole or perforated SDA (Self-Drilling Anchor) in m.

It is recommended to repeat the test for 3 times and make an average of values to improve the precision of GCD estimation. It is also preferred to conduct the test on a necked hole except otherwise the hole is not sustained. In case that the hole is not sustained the fully perforated SDA or casing to be used. If during the water injection, pressure is not obtained, or it is lesser than 0.20 MPa, then the ground hydraulic conductivity would be considered as VH, which means that the quality of grouting or injection executed at the location of the hole is classified as VP. In this case the section must be further grouted/injected by proper consolidation material/s and configuration to improve the ground solidification quality.

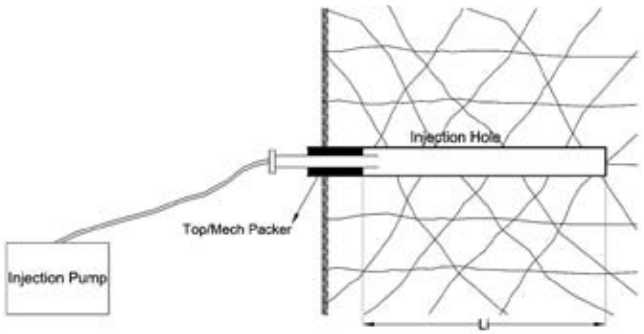

Figure 4. GCD setup (Bineshian, 2017).

Conduction of GCD test is an easy and quick practice while provides an appropriate economic estimate for prediction of hydraulic conductivity of ground to examine the ground solidification for further consolidation, water ingress reduction, or sealing.

Table 5. Ground Conductivity Designation.

GCD	Ground Hydraulic Conductivity	Ground Solidification Quality
> 100	VH - Very High	VP - Very Poor
100 - 50	H - High	P - Poor
50 - 15	M^+ - Medium	F - Fair
15 - 5	M^- - Moderate	G - Good
5 - 1	L - Low	VG - Very Good
< 1	VL - Very Low	E - Excellent

If in any case the GCD measurement is not used then observational ground water condition to be considered as a criterion for scoring the hgc using the Wetness diagram (Figure 3) in conjunction with Table 4. Wetness diagram is based on observational identification. It classifies the ground Wetness into 11 ranges.

P_i is the Properties Index as ground shear properties by way of a definition as a function of texture, fabric,

Table 6 defines parameters of P_i.

Table 6. Property Index (P_i): p_{cc}, p_{dc}, p_{ps}, p_{pm}, and p_{bw}.

cc	p_{cc}	dc	p_{dc}
Indurated	8.00	Never Indented by Thumbnail	6.00
Large Size Particles	6.50	Indented Hardly by Thumbnail	5.00
Picked Difficult	5.00	Indented by Thumbnail	4.00
Picked Easily	3.50	Indented by Thumb	3.00
Shovelled Difficult	2.00	Moulded by Strong Finger Pressure	2.00
Shovelled Easily	0.50	Moulded by Light Finger Pressure	1.00
Foot Imprint Easily	0.00	Exuded between Fingers when Squeezed in Hand	0.00

shape, and size of soil materials together with body wave velocity. P_i is designed to be an important part, which is going to model the important geotechnical characteristics of soil media as part of the I-System's comprehensiveness in applicability. P_i has 20 score out of 100.

ps	p_{ps}	pm	p_{pm}	bw (Vp) or [Vs] m/sec	p_{bw}
N/A (e.g. Grainless)	3.00	N/A (e.g. Grainless)	2.00	(≥ 6000) || [≥ 3300]	1.00
Boulder	3.00	Angular	2.00	(5999 - 5000) || [3299 - 2900]	0.90
Cobble	2.50	Sub-angular	1.50	(4999 - 4500) || [2899 - 2600]	0.80
Pebble	2.00	Flat	0.75	(4499 - 4000) || [2599 - 2200]	0.70
Gravel	1.50	Rounded	0.00	(3999 - 3500) || [2199 - 2000]	0.65
Sand	1.00			(3499 - 3000) || [1999 - 1500]	0.60
Silt	0.50			(2999 - 2500) || [1499 - 1000]	0.55
Clay	0.00			(2499 - 2000) || [999 - 750]	0.50
				(1999 - 1000) || [749 - 300]	0.45
				(≤ 999) || [≤ 299]	0.40

bw Body Wave Velocity,
cc Cohesiveness Consistency as shear properties of soil (Cohesion),
dc Denseness Consistency as shear properties of soil (Non-cohesiveness; Friction),
$\mathbf{p}_{bw}$ Factor related to bw including Vp or Vs as Geophysical property/ies of ground, which corrects P_i; it is derived either from available references considering the type of materials of surrounding ground or to be measured using Tunnel Seismic Prediction (TSP),
$\mathbf{p}_{cc}$ Score related to cc of P_i,
$\mathbf{p}_{dc}$ Score related to dc of P_i,
pm Particles' Morphology as a function of shape of soil's grains/granules,
$\mathbf{p}_{pm}$ Influencing parameter related to pm on P_i,
$\mathbf{p}_{ps}$ Influencing parameter related to ps on P_i,
ps Particles' Size as a function of size of soil's grains/granules,
Vp Primary Wave Velocity in m/sec, and
Vs Shear/Secondary Wave Velocity in m/sec.

S_i is the Strength Index as ground strength behaviour under confining condition. It is an important index in classification of ground-structure in the I-System and therefore important influencing parameters of ground and structure both together are considered to define the index irrespective of type of the media. In definition of S_i, unconfined compressive strength of the ground, scale effect and/or shape factor of the structure, and stress ratio between vertical and horizontal virgin stresses at the location/depth of the placement of the structure is carefully considered. S_i has 20 score out of 100.

Table 7 defines parameters of S_i.

Table 7. Strength Index (S_i): $\mathbf{s_{cs}}$ and $\mathbf{s_{se}}$.

cs - UCS	$\mathbf{s_{cs}}$
≥ 200 MPa	20.00
199 - 150 MPa	19.00
149 - 100 MPa	18.00
99 - 75 MPa	16.00
74 - 50 MPa	14.00
49 - 30 MPa	12.00
29 - 20 MPa	10.00
19 - 10 MPa	9.00
9 - 5 MPa	8.00
4.90 - 2 MPa	7.00
1.90 - 1 MPa	6.00
999 - 400 KPa	5.00
399 - 200 KPa	4.00
199 - 100 KPa	3.00
99 - 50 KPa	2.00
49 - 30 KPa	1.00
≤ 29 KPa	0.00

se			$\mathbf{s_{se}}$
UndS - D/H		$\sigma_v \geq \sigma_h$	$\sigma_v < \sigma_h$
≥ 2.50		0.80	1.00
= 1.90 - 1.30		0.85	0.95
= 1.20 - 0.80		0.90	0.90
= 0.70 - 0.50		0.95	0.85
≤ 0.40		1.00	0.80
SurS - B/H			$\mathbf{s_{se}}$
≥ 2.50			1.00
= 1.90 - 1.30			0.95
= 1.20 - 0.80			0.90
= 0.70 - 0.50			0.85
≤ 0.40			0.80

B/H Surface Structure's (SurS) shape/scale factor as the ratio of the width of the berm of slope or trench to the height of slope or trench,
cs Compressive Strength considering the Uniaxial Compressive Strength of the ground,
D/H Underground Structure's (UndS) shape/scale factor as the ratio of the width or horizontal span of the underground opening to the height of the opening,
$\mathbf{s_{cs}}$ Score related to cs of S_i,
se Scale Effect,
$\mathbf{s_{se}}$ Scale effect factor related to se on S_i,
UCS Unconfined Compressive Strength,
σ_h Horizontal Stresses at the location or depth of the placement of the structure, and
σ_v Vertical Stresses at the location or depth of the placement of the structure.

DF_i is the Dynamic Forces Impact on the ground-structure behaviour that representing the earthquake influence as a function of Scaled Design Peak Ground Acceleration (PGALM) or Earthquake Risk Zone (ERZ) or Medvedev-Sponheuer-Karnik Scale for earthquake (MSK). In case of utilisation of PGA_{LM} = □(PGA, GL, ρ, d) by designer (PGASD = MSF × PGAD, MSF = 6.9 × exp [-M/4] - 0.058 as Magnitude Scaling Factor ≤ 1.8, M is the Earthquake Magnitude, PGAD is the Design Peak Ground Acceleration, GL is the shear modulus, ρ is the unit mass of ground, and d is the depth of placement of the structure) it is recommended it to be calculated then Table 8 to be used for the associated value within the 7 ranges of PGALM for picking up the correct DF_i. However, if ERZ or MSK is used, the earthquake zoning based on either of these criteria should be picked from references available for the project area and then Table 8 to be utilised for the associated DF_i. It should be noted that ERZ is categorised in 7 classes of damage risk zones as shown in Table 8; EH (Extremely High; MSK XI-XII), VH (Very High; MSK IX-X), H (High; MSK VII-VIII), M (Moderate; MSK V-VI), L (Low; MSK IV), VL (Very Low; MSK III), and EL (Extremely Low; MSK I-II). DF_i has a range of 1.00 to 0.75. Table 8 defines parameters of DF_i.

Table 8. Dynamic Forces Impact (DF_i).

(PGA_{SD}) or [ERZ] or {MSK}	DFi
(< 0.05g) ‖ [EL] ‖ {I-II}	1.00
(0.06g - 0.10g) ‖ [VL] ‖ {III}	0.99
(0.11g - 0.15g) ‖ [L] ‖ {IV}	0.97
(0.16g - 0.25g) ‖ [M] ‖ {V-VI}	0.94
(0.26g - 0.35g) ‖ [H] ‖ {VII-VIII}	0.90
(0.36g - 0.50g) ‖ [VH] ‖ {IX-X}	0.85
(> 0.50g) ‖ [EH] ‖ {XI-XII}	0.75

ET_i is the Excavation Technique Impact on the ground-structure behaviour representing vibration impacts on structure during the excavation, which is designed to be a function of Excavation Technique (ET) or Peak Particle Velocity (PPV).

In case that PPV is selected to be used as criterion for scoring the ET_i, then it is recommended it to be measured using blasting vibration measurers; otherwise, type of ET is the criterion to pick the proper score for ET_i. ET_i has a range of 1.00 to 0.50. Table 9 defines parameters of ET_i.

Table 9. Excavation Technique Impact (ET_i).

(ET) or [PPV mm/sec]	**ETi**
(ManDigg)	1.00
(ME/NonExBreak) || [< 2]	0.99
(ResiBlast) || [2 - 9]	0.98
(CommBlast) || [10 - 24]	0.97
(IndBlast) || [25 - 59]	0.96
(InfraBlast) || [60 - 119]	0.95
(CtldBlast) || [120 - 449]	0.90
(MineBlast) || [450 - 499]	0.80
(ProdBlast) || [500 - 599]	0.65
(UnCtldBlast) || [≥ 600]	0.50

Man Digg — Manual Digging (Small scale excavation without use of explosives or Non Ex Break)

ME — Mechanised Excavation (Medium-large scale excavation without use of explosives or Non Ex Break)

Non Ex Break — Non-Explosive Breaking (Ground fragmentation using expansive materials)

Resi Blast — Residential Blasting (Engineered blasting near residential area)

Comm Blast — Commercial Blasting (Engineered blasting near commercial area)

IndBlast — Industrial Blasting (Engineered blasting near industrial area)

InfraBlast — Infrastructures Blasting (Engineered blasting for demolishing the infrastructures)

CtldBlast — Controlled Blasting (An ordinary engineered blasting for civil works)

MineBlast — Mining Blasting (Controlled blasting with underground/surface mining standards)

ProdBlast — Production Blasting (Controlled blasting for rock production in large scale)

UnCtldBlast — Un-Controlled Blasting (Non-engineered blasting)

Derivation of parameters from ground and their use in the classification is easy and confusion-free. This make the classification accurate in selection of the input data resulting credible output. The I-System's value ranges between 100 – 0 and classifies the ground-structure interaction to 10 classes as (I)-01 to (I)-10 from best to worst class.

The indices of A_i, C_i, H_i, P_i, and S_i having 20 per cent share out of a total score of 100. DF_i and ET_i are factors ranging between 1–0.75 and 1–0.50 respectively, which impact the summation of indices (see Figure 5 as the I-System's scoring diagram).

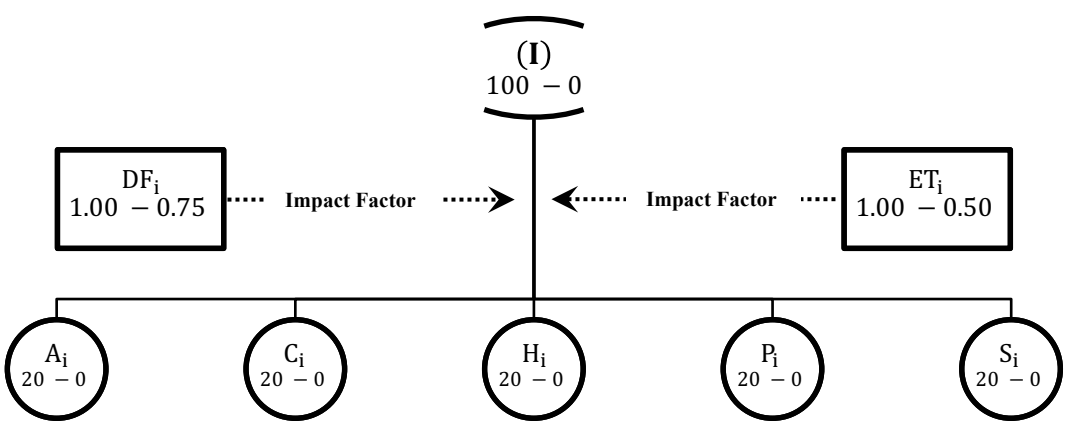

Figure 5. I-System scoring diagram.

Table 10 provide the I-System classes for underground structures containing the range for each class and the recommended:

- Support System/s (SS),
- Excavation Technique/s (ET),
- Instrumentation/monitoring Technique/s (IT),
- Prevention Technique/s (PT),
- Forecast Technique/s (FT), and
- Design Remark/s (DR) to be used for structural dimensioning and verification (SD and SV in Figure 2).

Nomenclatures are provided in Section 10.

Table 10. I-System Classes for Underground Structures: SS, ET, and IT.

I		Recommended Measures		
%	**Class**	**SS**	**ET**	**IT**
100-91	**(I)-01**	Scng	FF-ME/DnB, PL4200^{-}	Nil
90 - 81	**(I)-02**	Scng, IndiB25	FF-ME/DnB, PL4000^{-}	Nil
80 - 71	**(I)-03**	Scng, SpotB25	FF-ME/DnB, PL3800^{-}	Nil
70 - 61	**(I)-04**	Scng, SpotB25, PatchPS50	FF-ME/DnB, PL3600^{-}	3DMS@400m
60 - 51	**(I)-05**	Scng, SpotB32/SysHB25.LS, PS50, PSFS50, RDH54.L	FF-ME/DnB, PL3300^{-}	3DMS@200m
50 - 41	**(I)-06**	Scng, SysB32.L.S/SysHB32.L.S, FRS100, FRFS50, RDH54.L	HnB/(FF if ≤ 45 m^2)-ME/DnB, PL3000^{-}	3DMS@100m, StrainM@300m
40 - 31	**(I)-07**	Scng, CPS32.L.S/FP32.250.L.X1, SysB32.L.S/SysHB32.L.S, LG25.20.150.1000-, FRS200, FRFS150, RDH54.L	HnB/(FF if ≤ 35 m^2)-ME/NonExBreak/DnB, PL2000^{-}	3DMS@75m, StrainM@250m, PressC/LoadC@300m
30 - 21	**(I)-08**	FP32.200.L.X1/FP76.250.L.X1/PR100.300.L.X1, SysLB32.L.S, LG32.25.180.1000-/RigidR150UC23.1000-, FRS225/FRC225, FaceButt.L, FRFS200, RDH54.L+CF	PSE-ME/NonExBreak, PL1000^{-}	3DMS@50m, StrainM@200m, PressC/LoadC@250m, SingleRodE@400m
20 - 11	**(I)-09**	PR100.250.L.X1/FP76.200.L.X1/FP32.200.L.X2, FaceB25.L.S/FaceP300-, FaceButt.L, PreG/I, RigidR150UC23.750-+RingC, SysN32.L.S, FRS225/FRC225, FRFS200, RDH54.L+CF	PSD-ME, PL750^{-}	3DMS@25m, StrainM@150m, PressC/LoadC@200m, MultiRodE@400m, StrainG@500m
10 - 0	**(I)-10**	PR100.200.L.X1/FP76.200.L.X2, PreG/I, PostG/I, FaceB32.L.S/FaceP300-, FaceButt.L, RigidR200UC46.500-+RingC, SysN32.L.S, FRS250/FRC250, FRFS225, (RDH54.L, WDH54.L)+CF	PSD-ME, PL500^{-}	3DMS@15m, StrainM@100m, PressC/LoadC@150m, MultiRodE@300m, StrainG@400m, DIC@25m

Table 10. Continued; I-System Classes for Underground Structures: PT, FT, and DR.

I		Recommended Measures		
%	Class	PT	FT	DR
100-91	(I)-01	Avoid: UnCtldBlast	TSP/PH100.BH.L	Active load configuration, SPL and/or SFL not required
90 - 81	(I)-02	Avoid: UnCtldBlast	TSP/PH100.BH.L	Active load configuration, SPL and/or SFL not required
80 - 71	(I)-03	Avoid: UnCtldBlast	TSP/PH100.BH.L	Active load configuration, SPL and/or SFL not required
70 - 61	(I)-04	Avoid: ProdBlast/UnCtldBlast	TSP/PH100.BH.L	Active load configuration, SFL not required
60 - 51	(I)-05	Avoid: ProdBlast/UnCtldBlast	TSP/PH100.BH.L/PH54.EC.L	Load configuration to be maintained as active, SFL not required
50 - 41	(I)-06	Avoid: ProdBlast/UnCtldBlast	TSP/PH100.BH.L/PH54.EC.L	Load configuration to be maintained as active
40 - 31	(I)-07	Apply CPS, Avoid: MineBlast/ProdBlast/UnCtldBlast	TSP/PH100.BH.L/PH54.EC.L	Critical load bearing capacity
30 - 21	(I)-08	Apply FP/PR, Maintain Buttress, Avoid: FF & DnB	TSP/PH54.EC.L	Passive load configuration, sensitive to scale, unsupported span, & stand-up time
20 - 11	(I)-09	Apply PreG/I & PR/FP, Maintain Buttress, Avoid: FF, NonExBreak/DnB, & Ductile SS	TSP/PH54.EC.L	Passive load configuration, sensitive to scale, unsupported span, & stand-up time
10 - 0	(I)-10	Apply PreG/I & PR, Maintain Buttress, Avoid: FF, NonExBreak/DnB, & Ductile SS	TSP/PH54.EC.L	Passive load configuration, sensitive to scale, unsupported span, & stand-up time

In a similar way Table 11 provide the I-System classes for surface structures containing the range for each class and the recommended:

- Support System/s (SS),
- Excavation Technique/s (ET),
- Instrumentation/monitoring Technique/s (IT),
- Prevention Technique/s (PT),
- Forecast Technique/s (FT), and
- Design Remark/s (DR) to be used for structural dimensioning and verification (SD and SV in Figure 2).

Nomenclatures are provided in Section 10.

Table 11. I-System Classes for Surface Structures: SS, ET, and IT.

I		Recommended Measures		
%	Class	SS	ET	IT
100-91	(I)-01	Scng	(PreS, DD12000$^-$), (ProdBlast, PD6000$^-$)	Nil
90 - 81	(I)-02	Scng, IndiB25	(PreS, DD12000$^-$), (ProdBlast, PD4000$^-$)	Nil
80 - 71	(I)-03	Scng, SpotB25	(PreS, DD9000$^-$), (ProdBlast, PD4000$^-$)	Nil
70 - 61	(I)-04	Scng, SpotB25/SpotA25, PatchHEAM/PatchWeldM, DH54.L	(PreS, DD9000$^-$), (ProdBlast, PD3000$^-$)	3DMS@200m
60 - 51	(I)-05	Scng, SpotB32/SpotA32, HEAM/WeldM, DH54.L	(PreS, DD6000$^-$), (ProdBlast, PD3000$^-$)	3DMS@150m
50 - 41	(I)-06	Scng, SysA25.L.S, FRS150, DH54.L	(PreS, DD6000$^-$), (ProdBlast, PD2000$^-$)	3DMS@75m, IncM@500m
40 - 31	(I)-07	Scng, SysA32.L.S, FRS250, PostG/I, DH54.L	ME/NonExBreak	3DMS@25m, IncM@400m
30 - 21	(I)-08	RWall-SolP/FRS300/FRC300, SysN32.L.S, WH54.L+CF	PSE-ME	3DMS@10m, IncM@300m
20 - 11	(I)-09	DWall-TanP/FRS350/FRC350, SysN32.L.S, WH54.L+CF	PSE/OC-ME	3DMS@10m, IncM@200m, DIC
10 - 0	(I)-10	DWall-SecP/FRS400/FRC400, SysN32.L.S, WH54.L+CF	PSE/OC-ME	3DMS@10m, IncM@150m, DIC

Table 11. Continued; I-System Classes for Surface Structures: PT, FT, and DR.

I		Recommended Measures		
%	Class	PT	FT	DR
100-91	(I)-01	Avoid: UnCtldBlast	VPH54.L	Permanent stable condition, SPL and/or SFL not required
90 - 81	(I)-02	Avoid: UnCtldBlast	VPH54.L	Check against plain failure criteria, SPL and/or SFL not required
80 - 71	(I)-03	Avoid: UnCtldBlast	VPH54.L	Check against plain/wedge failure criteria, SPL and/or SFL not required
70 - 61	(I)-04	Avoid: ProdBlast/UnCtldBlast	VPH54.L	Check against plain/wedge failure & rock fall criteria, SPL and/or SFL not required
60 - 51	(I)-05	Protect Crest with FRS to Prevent Increment in Pore Water Pressure, Avoid: ProdBlast/UnCtldBlast, & Bulk Removal of Toe	ERT/VPH54.L	Check against plain/wedge/toppling failure & rock fall criteria, SFL not required
50 - 41	(I)-06	Cover Slope Crest with WPM & FRS at a Width Equal to Height to Help Prevention of Tension Crack Generation, Avoid: ProdBlast/UnCtldBlast, Surcharge at Crest, & Toe Lightening	ERT/VPH54.L	Check against plain/wedge/toppling failure & rock fall criteria
40 - 31	(I)-07	Cover Slope Crest with WPM & FRS at a Width Equal to Height to Help Prevention of Tension Crack Generation, Avoid: ProdBlast/UnCtldBlast, Sharp/Tall Slope, Short Berm, Surcharge at Crest, & Toe Lightening	ERT/SRT/VPH54.L	Check against plain/wedge/toppling failure & rock fall criteria
30 - 21	(I)-08	Cover Slope Crest with WPM & FRS at a Width Equal to Height to Help Prevention of Tension Crack Generation, Avoid: NonExBreak/DnB, Sharp/Tall Slope, Short Berm, & Surcharge at Crest	MASW/SRT/ERT/VPH54.L	Check against circular failure criteria
20 - 11	(I)-09	Avoid: NonExBreak/DnB, Unretained Wall/s, & Surcharge at Crest	MASW/SRT/VPH54.L	Check against circular failure criteria
10 - 0	(I)-10	Avoid: NonExBreak/DnB, Unretained Wall/s, & Surcharge at Crest	MASW/SRT/VPH54.L	Check against circular failure criteria

Furthermore, 3 special classes of (I)-BP, (I)-TD, and (I)-VP are also provided in the system for particular types of ground behaviour/hazards (related to GB and GH in Figure 2).

Tables 12 and 13 provide special classes of the I-System for underground and surface structures respectively containing the recommended:

- Support System/s (SS),
- Excavation Technique/s (ET),
- Instrumentation/monitoring Technique/s (IT),
- Prevention Technique/s (PT),
- Forecast Technique/s (FT), and
- Design Remark/s (DR) to be used for structural dimensioning and verification (SD and SV in Figure 2).

Nomenclatures are provided in Section 10.

Table 12. I-System Special Classes for Underground Structures.

I	Recommended Measures					
Class	**SS**	**ET**	**IT**	**PT**	**FT**	**DR**
(I)-BP	Scng, SysDB25.L.S/ConeB25.L.S/YieldB25.L.S, FRS150, HEAM/CableL+WeldM, FRFS50	HnB-ME/DnB, PL2700^{-}	3DMS@25m, StrainM@100m, PressC/LoadC@300m, MultiRodE@600m	Avoid: ProdBlast/UnCtld Blast, Rigid SS, & Naked Faces	TSP/PH100.BH.L	Bursting initiation time and depth of plastic zone around periphery to be measured
(I)-TD	Mild-Severe SSH: YieldR1000+RingC, SRH100.L.S.X2, YieldFRS200/YieldFRC200, LSC, SysDB25.L.S || Minor SSH: RigidR200UC46.1000-+RingC, FRS200/FRC200+SRH100.L.S.X1+SysLB32.L.S	HnB-ME, PL1000^{-}	3DMS@10m, StrainM@100m, PressC/LoadC@150m, MultiRodE@300m, StrainG@400m, DIC@25m	Apply SRH, Apply SysLB for Minor SSH, Avoid: FF, DnB, Rigid SS, & SysLB for Mild-Severe SSH	TSP/PH100.BH.L	Nonuniform deformation, dawdled load relaxation, scale sensitive
(I)-VP	BulkH300+, FaceP300-, PR100.150.L.X1, PreI/JetG/PreF, PostG/I, RigidR200UC46.500-+RingC, FRS300/FRC300, FRFS275, (RDH54.L, WDH54.L, ADH54.L)+CF	PSD-ME, PL500^{-}	3DMS@10m, StrainM@100m, PressC/LoadC@150m, MultiRodE@400m, StrainG@400m, DIC@25m	Apply PreG/I & PR, Maintain Buttress, Strictly Avoid: FF, NonExBreak/DnB, Ductile SS, & Build-up of Hydrostatic Pressure/Thrust at Face	TSP/PH54.EC.L	Passive load configuration, sensitive to scale, unsupported span, & stand-up time

Table 13. I-System Special Classes for Surface Structures.

I	Recommended Measures					
Class	**SS**	**ET**	**IT**	**PT**	**FT**	**DR**
(I)-VP	JetG/PreG/I/PreF, DWall-SecP/TanP, WH54.L+CF	PSE/OC-ME	3DMS@10m, DIC	Apply PreG/I/Freezing, Strictly Avoid: NonExBreak/DnB, Unretained Wall/s, & Surcharge at Crest	MASW/VPH54.L	Liquefaction prone, vibration sensitive, high passive lateral load configuration in design of retaining structure, long term consideration in time dependent behaviour

3 APPLICATIONS

The I-System is applicable for estimation of quality of both rock and soil in relation to the structure at any scale and type. It can be considered as empirical and observational parts of the design approach (see Figure 1). The ISystem is applicable in design procedure and/or in practice (see Figure 2) for

- categorizing the ground properties in relation to the Ground Zoning (GZ),
- discovering the Ground Behaviour (GB),
- identifying the associated failure mechanism/s with the discovered GB as the Ground Hazard/s (GH),
- determining the required Support System/s (SS), and
- assisting in Structural Dimensioning and Verification (SD and SV).

It is also applicable (see Tables 10 -13) to

- find the appropriate technique/s for excavation further to the determination of the required support system/s (ET),
- select the suitable choice for instrumentation/monitoring during the construction (IT),
- implement the right technique for prevention of hazard/s (PT), and
- designate the required technique for forecasting/prediction (FT).

The I-System is applicable to serve the above-stated purposes for surface, semi-surface, subsurface, and underground structures in the field of civil, mining (quarry/coal/open-pits /other), oil and gas exploration/drilling works. Range of application of the I-System includes but it is not limited to the use in design and/or practice for:

- abutments of bridges/dams,
- caverns,
- deep/shallow foundations,
- exploration/grouting galleries,
- open-cut/underground metro stations,
- shafts,
- slopes,
- trenches,
- tunnels (any type or method), and
- wells, etc.

4 UTILISATION

Utilisation of the I-System is an approach based on the following steps:

- Stage 1. Derivation of the Input: Derive the parameters from a site visit or available data in the references.
- Stage 2. Calculation of the Indices: Calculate A_i, C_i, H_i, P_i, S_i, DF_i, and ET_i using the derived data in Stage 1 and Eq. 2–9 and Tables 2–9.
- Stage 3. Calculation of the I-System: Calculate the I-System using Eq.1 and calculated indices in Stage 2.
- Stage 4. Determination of the (I)-Class: Determine the (I)-Class using the calculated I-System value in Stage 3 and Tables 10–13.
- Stage 5. Utilisation of Output: Utilise the recommendations for SS, ET, IT, PT, FT, and DR provided in Tables 10 – 13 in design and/or practice.

5 CASE HISTORIES

For clarification on the method of application of the I-System and as evidence for the comprehensiveness of applicability of the same, three cases of most recent applications of the system are provided here.

The applied cases provided here includes the use of the I-System for rock mountain tunnelling by NATM in Section 5.1, soil tunnelling by NATM in Section 5.2, and surface excavations in Section 5.3. The client of the projects considered here is Northern

Figure 6 summarises the utilisation approach explained above in a simple diagram.

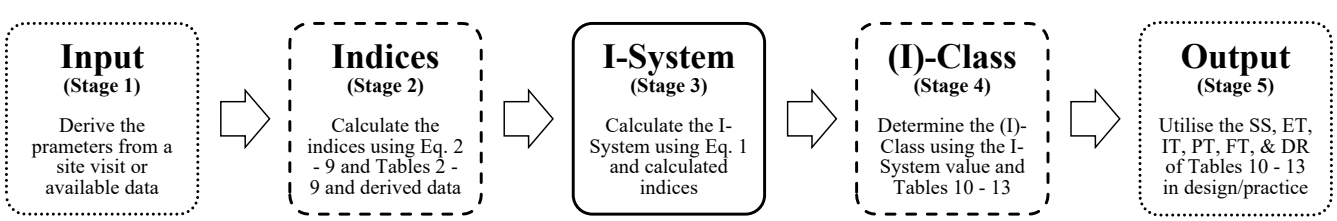

Figure 6. Utilisation diagram of the I-System.

Railway and engineer in charge is Konkan Railway Corporation Limited (KRCL).

5.1 UNDERGROUND ROCK WORKS

Tunnel T5 is one of the most challenging NATM tunnels in USBRL Project in North India. It is a 6 km twin tube tunnel in young Himalayas terrain in dolomite rock formation, which pass through several hazardous zones.

On 2019/05/11 an incident was reported at T5's main tunnel at chainage CH46598. The I-System were utilised to assess the condition of the face and to provide a solution to cross the zone.

Based on Section 4 following steps were taken:

- Input data were simply derived from the site visit observations and from the collected available data of the client (all derived input data are presented in Table 14).
- Indices were calculated using Tables 2 – 9 to score the input parameters. Eq. 2 – 9 were used for the calculation of indices.
- I-System value was calculated using calculated indices (Table 14) and Eq. 1. Figure 7 provides calculated value for the I-System.
- (I)-Class was determined using Tables 10 -13. Figure 7 provides determined (I)-Class for the same.
- Output of the I-System was derived from Tables 10 – 13, which was given as recommendation/solution to cross the challenging zone. Figure 7 provides the output for the case.

Table 14. Input Data: Utilisation of the I-System for T5 CH46598 in rock.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi	
Ai	**2.77**
dn Discontinuity Number/s - per m	≥ 25
ds Discontinuity Set/s	3
di Discontinuity Inclination - °	31 - 60
da Discontinuity Aperture	Open
dd Discontinuity Disintegration	Semi-Integrated
df Discontinuity Friction	Low Friction - Smooth/Even
dp Discontinuity Persistency	≥ 0.90 × D
Ci	**5.25**
pc Problematical Configuration	Sheared - High Shear Stresses - e.g. Mylonite
sc Structural Configuration	Layered (100 - 10 cm)
Hi	**6.50**
gc Ground Conductivity	(7 - 9.99) || [Wet]
gs Ground Softness - Mohs	5
Pi	**6.60**
cc Cohesiveness Consistency	Picked Easily
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	Sand
pm Particle Morphology	Sub-angular
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(3499 - 3000) || [1999 - 1500]
Si	**8.10**
cs UCS	19 - 10 MPa
se Scale Effect	D/H = 1.20 - 0.80 & σv ≥ σh
DFi	**0.85**
(PGASD) || [ERZ] || {MSK}	(0.36g - 0.50g) || [VH] || {IX-X}
ETi	**0.99**
(ET) || [PPV mm/sec]	(ME/NonExBreak) || [< 2]

I-System = 25% ↓ (I)-Class = (I)-08 ↓ RECOMMENDED MEASURE/S
SS - Support System FP32.200.L.X1/FP76.250.L.X1/PR100.300.L.X1, SysLB32.L.S, LG32.25.180.1000-/RigidR150UC23.1000-, FRS225/FRC225, FaceButt.L, FRFS200, RDH54.L+CF **ET - Excavation Technique/s** PSE-ME/NonExBreak, PL1000- **IT - Instrumentation Technique/s** 3DMS@50m, StrainM@200m, PressC/LoadC@250m, SingleRodE@400m **PT - Prevention Technique/s** Apply FP/PR, Maintain Buttress, Avoid: FF & DnB **FT - Forecast Technique/s** TSP/PH54.EC.L **Design Remark/s** Passive load configuration, sensitive to scale, unsupported span, & stand-up time

Figure 7. Output Data: Utilisation of the I-System for T5 CH46598 in rock.

Following explanation further clarifies Figure 7 using Section 10;

SS – Support System to be applied:

- **PR100.300.L.X1** (One row of Piperoofing with 100 mm dia, 300 mm spacing, and specified Length) or FP76.250.L.X1 (One row of Forepoling with 76 mm dia, 250 mm spacing, and specified Length) or FP32.200.L.X1 (One rows of Forepoling with 32 mm dia, 200 mm spacing, and specified Length),
- **SysLB32.L.S** (Systematic Long Bolting with 32 mm dia and specified Length and Spacing),
- **LG32.25.180.1000-** (Lattice Girder with 32 mm dia rebar at intrados and two 25 mm dia at extrados with 180 mm spacing between the intrados and extrados and spacing between the LGs below 1000 mm) or RigidR150UC23.1000- (Rigid Rib made with Universal Column as per Australian Standard of 150UC23 and spacing of below 1000 mm),
- **FRS225** (Fibre Reinforced Shotcrete with 225 mm thickness) or FRC225 (Fibre Reinforced Concrete with 225 mm thickness),
- **FaceButt.L** (Face Buttress with specified Length),
- **FRFS200** (Fibre Reinforced Face Sealing with 200 mm thickness), and
- **RDH54.L+CF** (Radial Drainage Holes with 54 mm dia + Collar Filtration).

ET – Excavation Technique/s to be implemented:

- **PSE-ME/NonExBreak,1000-** (Mechanised Excavation using Partial Sequential Excavation or Non-Explosives Breaking with Pull Length below 1000 mm).

IT – Instrumentation Technique/s to be used:

- **3DMS@50m** (3D Monitoring Station at every 50 m),
- **StrainM@200m** (Strain Meter at every 200 m),
- **PressC/LoadC@250m** (Pressure Cell or Load Cell at every 250 m), and
- **SingleRodE@400m** (Single-Rod Extensometer at every 400 m).

PT – Prevention Technique/s to be considered:

- Apply **PR/FP** (Piperoofing or Forepoling),
- Maintain Buttress, and
- Avoid FF (Full Face Excavation), DnB (Drill and Blast).

FT – Forecast Technique/s to be utilised:

- TSP/PH54.EC.L (Tunnel Seismic Prediction or Probe Hole and Exploratory Coring with 54 mm dia and specified Length)

DR – Design Remark/s to be taken into consideration:

- Passive load configuration, and
- Sensitive to scale, unsupported span, and stand-up time.

Above-mentioned recommendations from the I-System were advised for the problematic zone at T5 and tunnel was crossed the section successfully without any safety or construction issues. Nomenclatures used in Figure 7 are provided in Section 10.

5.2 UNDERGROUND SOIL WORKS

Tunnel T2 is another challenging NATM tunnel in USBRL Project in North India with a length of almost 5.6 km twin tube in young Himalayas terrain, which for some stretches of the length it passed through soil formation. Work were stopped due to a gravity driven failure on 2018/09/23 till 2018/11/03 at CH37488.

The I-System were utilised and condition were assessed. Same approach as discussed in the Section 5.1 were followed to calculate the I-System; in this case for soil NATM tunnelling. Table 15 and Figure 8 provides the input and output of the I-System for the case respectively. To avoid repetition of the approach, explanation on the utilisation stages are skipped here.

Nomenclatures used in Figure 8 are provided in Section 10.

Table 15. Input Data: Utilisation of the I-System for T2 CH37488 in soil.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi	
Ai	0.00
dn Discontinuity Number/s - per m	N/A || Jointless
ds Discontinuity Set/s	N/A || Jointless
di Discontinuity Inclination - °	N/A || Jointless
da Discontinuity Aperture	N/A || Jointless
dd Discontinuity Disintegration	N/A || Jointless
df Discontinuity Friction	N/A || Jointless
dp Discontinuity Persistency	N/A || Jointless
Ci	4.00
pc Problematical Configuration	Homogeneous || Isotropic || Jointless || Granular
sc Structural Configuration	Cohesive Matrix Skeleton
Hi	2.40
gc Ground Conductivity	(25 - 49) || [Flow]
gs Ground Softness - Mohs	4
Pi	8.13
cc Cohesiveness Consistency	Picked Easily
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	Gravel
pm Particle Morphology	Angular
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(3999 - 3500) || [2199 - 2000]
Si	7.20
cs UCS	9 - 5 MPa
se Scale Effect	D/H = 1.20 - 0.80 & $\sigma v \geq \sigma h$
DFi	0.85
(PGASD) || [ERZ] || {MSK}	(0.36g - 0.50g) || [VH] || {IX-X}
ETi	0.90
(ET) || [PPV mm/sec]	(CtldBlast) || [120 - 449]

I-System = 17%
↓
(I)-Class = (I)-09
↓
RECOMMENDED MEASURE/S

SS - Support System
PR100.250.L.X1/FP76.200.L.X1/FP32.200.L.X2, FaceB25.L.S/FaceP300-, FaceButt.L, PreG/I, RigidR150UC23.750-+RingC, SysN32.L.S, FRS225/FRC225, FRFS200, RDH54.L+CF
ET - Excavation Technique/s
PSD-ME, PL750-
IT - Instrumentation Technique/s
3DMS@25m, StrainM@150m, PressC/LoadC@200m, MultiRodE@400m, StrainG@500m
PT - Prevention Technique/s
Apply PreG/I & PR/FP, Maintain Buttress, Avoid: FF, NonExBreak/DnB, & Ductile SS
FT - Forecast Technique/s
TSP/PH54.EC.L
Design Remark/s
Passive load configuration, sensitive to scale, unsupported span, & stand-up time

Figure 8. Output Data: Utilisation of the I-System for T2 CH37488 in soil.

Proposed remedy for T2 in the above-said problematic zone is as follows (see Figure 8):

SS – Support System to be applied:

- **PR100.250.L.X1** (One row of Piperoofing with 100 mm dia, 250 mm spacing, and specified Length)

or **FP76.200.L.X1** (One row of Forepoling with 76 mm dia, 200 mm spacing, and specified Length) or **FP32.200.L.X2** (Two rows of Forepoling with 32 mm dia, 200 mm spacing, and specified Length),

- **FaceB25.L.S** (Face Bolting with 25 mm dia and specified Length and Spacing) or **FaceP300-** (Face Plug with thickness below 300 mm),
- **FaceButt.L** (Face Buttress with specified Length),
- **PreG/I** (Pre-Excavation Grouting/Injection),
- **RigidR150UC23.750**$^{-}$ (Rigid Rib made with Universal Column as per Australian Standard of 150UC23 and spacing of below 750 mm) + **RingC** (Ring Closure),
- **SysN32.L.S** (Systematic Soil Nailing with 32 mm dia and specified Length and Spacing),
- **FRS225** (Fibre Reinforced Shotcrete with 225 mm thickness) or **FRC225** (Fibre Reinforced Concrete with 225 mm thickness),
- **FRFS200** (Fibre Reinforced Face Sealing with 200 mm thickness), and]
- **RDH54.L+CF** (Radial Drainage Holes with 54 mm dia + Collar Filtration).

ET – Excavation Technique/s to be implemented:

- **PSD-ME, PL750**$^{-}$ (Mechanised Excavation using Partial Sequential Digging with Pull Length below 750 mm)

IT – Instrumentation Technique/s to be used:]

- **3DMS@25m** (3D Monitoring Station at every 25 m),
- **StrainM@150m** (Strain Meter at every 150 m),
- **PressC/LoadC@200m** (Pressure Cell or Load Cell at every 200 m),
- **MultiRodE@400m** (Multi-Rod Extensometer at every 400 m), and
- **StrainG@500m** (Strain Gauge at every 500 m).

PT – Prevention Technique/s to be considered:

- Apply **PreG/I** (Pre-Excavation Grouting/Injection) and **PR/FP** (Piperoofing or Forepoling),
- Maintain Buttress, and
- Avoid **FF** (Full Face Excavation), **NonExBreak/ DnB** (Non-Explosives Breaking or Drill and Blast), and **Ductile SS** (Support System).

FT – Forecast Technique/s to be utilised:

- **TSP/PH54.EC.L** (Tunnel Seismic Prediction or Probe Hole and Exploratory Coring with 54 mm dia and specified Length).

DR – Design Remark/s to be taken into consideration:

- Passive load configuration, and
- Sensitive to scale, unsupported span, and stand-up time.

Works at T2 were restored by application of the above-mentioned recommendations using the I-System; no further instabilities occurred due to application of the abovesaid remedy during the course of crossing the challenging zone.

5.3 SURFACE ROCK AND SOIL WORKS

Portal works of Tunnel T13 (the longest tunnel in USBRL Project in North India with 9 km length in young Himalayas terrain) is considered here as a real example of application of the I-System for surface structures in rock and soil. Entire slope of the T13 portal is placed in a formation with the mix of sandstone fragments in soil matrix.

The I-System is utilised for design purpose for the slope of the outlet portal of the T13 tunnel. Same approach as explained in Section 4 is applied for the case.

Proposed remedy for the slope of the T13's portal is as follows (see Table 16 and Figure 9):

SS – Support System to be applied:

- **Scng (**Scaling),
- **SysA25.L.S** (Systematic Anchoring with 25 mm dia and specified Length and Spacing),
- FRS150 (Fibre Reinforced Shotcrete with 150 mm thickness), and
- **DH54.L** (Drainage Holes with 54 mm dia and specified Length).

ET – Excavation Technique/s to be implemented:

- **(PreS, DD6000**$^{-}$**)** Presplitting with Drilling Depth of below 6000 mm, and
- **(ProdBlast, PD2000**$^{-}$**)** Production Blasting with Pull Depth below 2000 mm.

IT – Instrumentation Technique/s to be used:

- **3DMS@75m** (3D Monitoring Station at every 75 m), and
- **IncM@500m** (Inclinometer at every 500 m).

PT – Prevention Technique/s to be considered:

- Cover Slope Crest with **WPM** (Waterproofing Membrane) and **FRS** (Fibre Reinforced Shotcrete) at a with equal to height to help prevention of tension crack generation, and
- Avoid **ProdBlast/UnCtldBlast** (Production Blasting or Un-controlled Blasting), application of Surcharge at Crest, and Toe Lightening.

FT – Forecast Technique/s to be utilised:

- **ERT/VPH54.L** (Electric Resistivity Tomography or Vertical Probe Holes with 54 mm dia and specified Length).

DR – Design Remark/s to be taken into consideration:

- Check against plain/wedge/toppling failure and rock fall criteria.

Above-mentioned recommendations for the slope of the T13's portal using the I-System is under application at the time of write up of the paper. Table 16 and Figure 9 represents the input and output details of the I-System calculation procedure respectively.

6. PROJECT REFERENCES

The I-System has been developed in several cases for a course of 22 years. It is scrutinised, and examined directly/indirectly in several projects in different countries as listed below and its applicability with accepted engineering accuracy and precision is verified; following list represents the same.

Table 16. Input Data: Utilisation of the I-System for T13 Portal in Rock and soil.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi	
Ai	5.48
dn Discontinuity Number/s - per m	15 - 19
ds Discontinuity Set/s	3
di Discontinuity Inclination - °	11 - 30
da Discontinuity Aperture	Semi-Tight
dd Discontinuity Disintegration	Weathered/Altered
df Discontinuity Friction	Moderate Friction - Nonsmooth
dp Discontinuity Persistency	≥ 0.90 × D
Ci	9.00
pc Problematical Configuration	Fractured - Highly
sc Structural Configuration	Layered (100 - 10 cm)
Hi	12.00
gc Ground Conductivity	(≤ 0.99) || [Dry]
gs Ground Softness - Mohs	6
Pi	16.00
cc Cohesiveness Consistency	Indurated
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	N/A (e.g. Grainless)
pm Particle Morphology	N/A (e.g. Grainless)
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(4999 - 4500) || [2899 - 2600]
Si	12.60
cs UCS	74 - 50 MPa
se Scale Effect	B/H = 1.20 - 0.80
DFi	0.85
(PGASD) || [ERZ] || {MSK}	(0.36g - 0.50g) || [VH] || {IX-X}
ETi	0.99
(ET) || [PPV mm/sec]	(ME/NonExBreak) || [< 2]

I-System = 46%
↓
(I)-Class = (I)-06
↓
RECOMMENDED MEASURE/S

SS - Support System
Scng, SysA25.L.S, FRS150, DH54.L
ET - Excavation Technique/s
(PreS, DD6000-), (ProdBlast, PD2000-)
IT - Instrumentation Technique/s
3DMS@75m, IncM@500m
PT - Prevention Technique/s
Cover Slope Crest with WPM & FRS at a Width Equal to Height to Help Prevention of Tension Crack Generation, Avoid: ProdBlast/UnCtldBlast, Surcharge at Crest, & Toe Lightening
FT - Forecast Technique/s
ERT/VPH54.L
Design Remark/s
Check against plain/wedge/toppling failure & rock fall criteria

Figure 9. Output Data: Utilisation of the I-System for T13 Portal in Rock and soil.

- Railway projects - tunnels, slopes, and bridge abutments: USBRL project comprising of Tunnels T1 – T13 with over 45 km length, Tunnel T40 Wider Section, T12P2 Portal, Kauri Road, Chenab Bridge,
- Metro projects - tunnels and stations: Ahmedabad, Bangaloru, Chennai, Delhi, Lucknow, Mumbai, Pune, Tehran,
- Road projects - tunnels, trenches, and slopes: Diftah-Shis, Eqlid, Jammu Ring, Kaikoura, Mumbai Coastal, Penjween, Resalat Hwy, Tohid, WestConnex,
- Urban projects - canal and trenches: Arabian Canal in Persian Gulf,
- Subordinate tunnels - small size and micro tunnels: Shahmirzad, Shiraz, Torrington,
- Hydropower/dam projects - slope stabilizations, dam foundation, bridge foundations, dam abutments, caverns, shafts, and tunnels: Aassi, Balaroud, Broadlands, Galehroud, Kahir, Karuma, Kheirabad, Seidoun, Seimareh, Snowy Mountains, Sonateh,
- Mining projects - stopes, adits, drifts, wells, shafts, and pits: Kalgoorlie Super Pit, Kolomdar, Pashkalat, Tazareh.

Some of the above-mentioned projects are still in the phase of construction.

7. CONCLUSIONS

The I-System is developed to compensate demerits of existing engineering classifications including their limitations, drawbacks, impreciseness, and inaccuracy. It is applicable for all types of geomaterials with acceptable precision and accuracy provided by simplicity and certainty in its approach for derivation of input parameters and clarity and trust in output data.

It is developed in challenging projects in varieties of ground fields and verified for perfectness. There is no limitation/s in its application for any type of surface, subsurface, and/or underground structures. It comes with a simple equation containing important parameters, which can be derived from doubtless input tables, references, or test results. It is based on certain essential indices, which defines mechanical behaviour of surrounding ground of structure considering impact of dynamic forces as well as excavation technique impact.

It practically takes into consideration most important mechanical aspects of ground for an appropriate optimised design. It has the capacity to be a credible comprehensive classification system to be utilised in practice and design intelligently for all ground related structures.

8. ACKNOWLEDGEMENT

While this research was funded by the author entirely, during 22 years of development course of the I-System, author has worked in/with several firms in projects directly/ indirectly. I thank all of them for sharing their pearls of wisdom with me during the course of this research. Their name is briefly included here but not limited to: SAB, Ziggurat, Mahab, GM, Perlite, University of Western Australia, Geodata, LnT, Amberg, KRCL, and NR.

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10. NOMENCLATURE

(I)	Index of Ground – Structure or I-System
3DM	3D Monitoring - Using Bi-Reflex Target Markers
3DMS	3D Monitoring Station
ADH	Axial Drainage Hole/s - NX Size Drainage Hole/s (with/without casing) with the Orientation Parallel to the Axis of Underground Opening and Perpendicular to Face with L ≤ 3/2D and Spacing Determined based on Ground Water Condition
B	Width of Berm of a Slope/Trench
BH	Blind Hole - Triangular Patterned Probing Parallel to Axis of Underground Space Using Blind Hole/s with L = 2D and 100+ mm Dia
BP	Burst Prone - Ground Condition with Rock Burst or Coal Burst Behaviour
BulkH	Bulk Head - Shotcrete/Concrete Plug at Whole Section of Excavation at Face to Prevent the Ground to Flow CableL Cable Lacing - Applicable for Controlling Rock Burst in Deep Underground Spaces
CF	Collar Filtration - Filtration of Drainage Holes' Outlet to Stop Debris/Fines Discharge ConeB Cone Bolts - Oriented/Radial Cone Bolts with L = 1/2D
CPS	Crown Periphery Spiling - SN Umbrella at 5-30 deg with L = 2/3D D Width or Horizontal Span of Underground Opening
DD	Drilling Depth
DH	Drainage Hole/s - Upward Angled NX Size Hole/s (with/without casing) with L = 3/2H and Spacing Determined based on Ground Water Condition
DIC	Digital Image Correlation
DnB	Drill and Blast - Controlled/Smooth
DR	Design Remark/s
DWall	Diaphragm Wall
EC	Exploratory Coring - Single Hole Coring Parallel to Axis of Underground Space Using NX Size Hole/s with L = 3D ElFootR Elephant Foot Rib - Applicable in Case of High Vertical/Passive/Dead Load above Crown of Tunnel as a Very Stiff/Rigid Support System
ERT	Electrical Resistivity Tomography - A non-destructive geophysical method for characterization of ground ET Excavation Technique/s
FaceB	Face Bolting - Fibreglass/SDA Bolts Parallel to Axis and Perpendicular to Face with L = 1D
FaceButt	Face Buttress - Keeping Part of Face in Place with a Nose Length Equal to L = 1/4D (Only if D ≥ 6 m) as a Buttress to Absorb Face Pressure and Thrust as Part of Face Stabilization
FaceP	Face Plug - Application of 300 mm Shotcrete at Face to Plug the Outlet of the Debris Discharge FibreD Fibreglass Dowel/s
FF	Full Face Excavation
FP	Fore Poling - Umbrella using Perforated/ Blind SDA with L = 1D FRC Fibre Reinforced Concrete
Freezing	Solidification Technique for Underground and Surface Openings Prior Excavation FRFS Fibre Reinforced Face Sealing
FRS	Fibre Reinforced Shotcrete
FT	Forecast Technique/s
GD	Gravity Driven - Flowing Ground Class with Fully Plastic Behaviour H Height of a Slope/Trench/Opening/Buttress
HEAM	High Energy Absorption Mesh - Protective Mesh against Dynamic Loads or Mesh Over Shotcrete HnB Top Heading and Benching
IncM	Inclinometer/s
IndiB	Individual Bolting - Oriented and in Very Limited Number IT Instrumentation Technique/s
JetG	Jet Grouting -Application in Underground or Surface Metro Station Construction
L	Length of ADH, ConeB, CPS, DH, FaceB, FaceButt, FP, BH, EC, PR, RDH, SRH, SysA, SysB, SysDB, SysHB, SysLB, SysN, VPH, WDH, WH, and YieldB
LG	Lattice Girder
LoadC	Load Cell/s
LSC	Longitudinal Stress Controller - Rubber/ Spring

MASW Multichannel Analysis of Surface Waves - A non-destructive geophysical method for characterization of ground ME Mechanised Excavation - TBM/ Roadheader/Excavator/Hammer

MicroP Micro Piles - Applicable for Elephant Rigid Ribs to Distribute the Concentrated Load to a Wider Footing Area MultiRodE Multiple Rod Extensometer - Measuring Points @ 2, 4, and 6 m Recommended

OC Open Cut

PatchHEAM Patch High Energy Absorption Mesh (Protective Mesh against Dynamic Loads) PatchPS Patch Plain Shotcrete

PatchWeldM Patch Weld Mesh - Applicable as Protective Mesh in Surface/Underground Openings to Prevent Spot Rock Falls PCC Plain Cement Concrete

PD Pull Depth

PH Probe Hole Probing Using Blind Hole Drilling with 100+ mm Dia or Exploratory Coring Using NX Size Hole/s PL Pull Length - Advance Length

PostG/I Post-excavation Grouting/Injection - Consolidation/Solidification PPV P e a k Particle Velocity

PR Pipe Roofing - With/Without Grouting/Injection using Perforated/Blind Pipe with L = 1D PreF Pre-Excavation Freezing of Face or Excavation Line/Periphery

PreG/I Pre-excavation Grouting/Injection - Cement/Mineral/Chemical-base PreS Pre-excavation Splitting

PressC Pressure Cell/s

ProbH Probe Hole/s - Blind hole/s or Coring to Predict Ground Ahead of Current Face in Underground Openings PS P l a i n Shotcrete

PSD Partial-Sequential Digging - Small Scale Partial Digging in Several Sequences e.g. Small Pilots, Considering Stand-up Time and Maximum Unsupported Span

PSE Partial-Sequential Excavation - Small Scale Partial Excavation larger than Digging Scale in Several Sequences e.g. Pilot and Enlargement, Considering Stand-up Time and Maximum Unsupported Span

PSFS Plane Shotcrete Face Sealing - Application of 50 mm Plain Shotcrete at Face for Hazards and Disintegration Prevention PT Prevention Technique/s

RCC Reinforced Cement Concrete (Conventional)

RDH Radial Drainage Hole/s - NX Size Radial/ Axial Holes (with/without casing) with L $\leq$ 1D and Spacing Determined based on Ground Water Condition

RigidR Rigid Ribs - H Section Heavy Beams RingC Ring Closure

RWall Retaining Wall

S Spacing related to ConeB, CPS, FaceB, SRH, SysA, SysB, SysDB, SysHB, SysLB, SysN, or YieldB; S = 1/3L Scng Scaling - Removal of Loose Fragments/ Blocks

SDA Self-Drilling Anchor

SecP Secant Piling

SFL Structural Final Liner

SingleRodE Single Rod Extensometer - Measuring Point @ 3 m Recommended SolP Soldier Piling

SPL Structural Primary Liner

SpotA Spot Anchoring

SpotB Spot Bolting - Oriented with Limited Number

SRH Stress Release Holes - Long Radial Necked Holes at Minimum 100 mm Dia with L = 1D (Invented and developed by the author in 2015 at USBRL Project; Client: Northern Railway, Engineer in Charge: Konkan Railway Corporation Limited)

SRT Seismic Refraction Tomography - A non-destructive geophysical method for characterization of ground SS Support System

SSH Squeezing/Swelling/Heaving

StrainG Strain Gauge/s

StrainM Strain Meter

SurS Surface Structure - It includes Surface and Subsurface Structure/s or and Mine/s in general comprising of but not limited to Slopes, Trenches, Foundations, Open Cuts, Open Pits, Cut and Covers, Surface Power House Openings, and Shallow Stations

SysA Systematic Anchoring - Perpendicular

Anchors to the Face of Slope with L = 1/2H SysB Systematic Bolting - Radial Direction with L = 1/2D

SysDB Systematic Dynamic Bolts - Oriented/ Radial Dynamic Bolts with L = 1/2D SysHB Systematic Horn Bolting - Only above SPL at 30 - 45 deg with L = 2/3D SysLB Systematic Long Bolting - Radial Long Rock Bolts with L = 2/3D

SysN Systematic Nailing - Radial Bolts/ Anchors with L = 1D = 1H TanP Tangent Piling

TD Time Dependent - Ground Condition with Time Dependent Shearing Behaviour such as Squeezing/Swelling/Heaving Behaviour, or even Creep

TSP Tunnel Seismic Prediction

UC Universal Column as per Australian Standard (i.e. 150UC23 and 200UC46)

UndS Underground Structure/s - It includes Underground Shallow and Deep Structures/Openings/Mines comprising of but not limited to Tunnels, Wells, Shafts, Caverns, Stopes, Underground Power Houses, and Underground Stations

VP Visco-elasto-Plastic - Ground condition as visco-elasto-plastic to fully plastic behaviour that contains elastic component/s together with viscous component/s, which gives the ground strain rate dependence on time; however, due to losing energy during static/dynamic loading cycle, its behaviour converts to fully plastic and may flows like a viscous substance

VPH Vertical Probe Hole - Blind/Coring Exploration Using Vertical NX Size Hole/s Drilling with L = 1/2H

WeldM Weld Mesh - Conventional Weld Mesh used in Reinforcement of Shotcrete or as Mesh Over Shotcrete in Rock Burst Condition

WDH Wing Drainage Holes - NX Size Wing Shape with Umbrella Patterned Holes (with/without casing) at 30 - 45 deg applicable in Underground Openings to Drain the Water from sides and ahead of Face to Reduce the Pore Hydrostatic Pressure with L

$\leq$ 2D and Spacing Determined based on Ground Water Condition

WH Weep Holes - Upward Angled NX Size Weeps (with/without casing) with L = 1H and Spacing Determined based on Ground Water Condition

WPM Waterproofing Membrane - An Elastic/ Flexible Impermeable Geotextile or Fibre Reinforced Geomembrane or Composite to be used for Sealing

X1 One Row

X2 Two Rows

YieldB Yielding Bolts - Oriented/Radial Yield Bolts with L = 1/2D

YieldFRC Yield Fibre Reinforced Concrete - Cast Fibre Reinforced Concrete with Embedded LSC YieldFRS Yield Fibre Reinforced Shotcrete - Cast Fibre Reinforced Shotcrete with Embedded LSC YieldR Yield Ribs - Sliding Ribs using T-H Profile or any Profile Capable of Sliding Function

MICROPILE FOUNDATION FOR BRIDGE OVER RIVER SHYOK IN LADAKH REGION – FIRST BRIDGE OF ITS KIND IN THE COUNTARY - A CASE STUDY

ANIL KUMAR and MANISH KAPIL
BRTF

ABSTRACT

This particular paper deals with construction of 10x140 ft TBR Extra-wide class-70R Bailey Bridge on micropile foundation in Ladakh region. It is the first bridge of this kind in the country which has been constructed on micropile across a riverine bed that too at an altitude of 14,500 ft across river Shyok in Ladakh region on road Darbuk – Shyok - Daulat Beg Oldi (D-S-DBO) which cuts across the treacherous route along the Shyok River at km 150.400. The location being very critical and important, required to be bridged urgently to ensure smooth flow of traffic. Upon analyzing the technical viability, ease and speed of construction, it was decided to construct 10 x 140 feet TDR CL 70R EWBB over micropile foundation which is a small diameter pile, generally less than 300 mm, drilled and grouted pile that is typically reinforced.

The design for the substructure using micro pile foundation was arrived at after carrying out a detailed subsurface Investigation and carrying out vertical and horizontal load test. The design of micro pile foundation comprises of 21 m X 273 mm diameter perforated casing with designed reinforcement.

1. INTRODUCTION

1.1 Road Darbuk – Shyok - Daulat Beg Oldi (D-S-DBO) cuts across the treacherous route along the Shyok River which is literally known as "The River of Death" and flows through northern Ladakh in India and the Ghangche District of Gilgit–Baltistan of Pakistan spanning some 550 km. The Shyok River, a tributary of the Indus River, originates from the Rimo Glacier, one of the tongues of Siachen Glacier. The river widens at the confluence with the Nubra River.

The alignment of the Shyok river is very unusual, originating from the Rimo glacier, it flows in a south-easterly direction and, joining the Pangong range, it takes a north-western turn, flowing parallel to its previous path. The Shyok flows in a wide valley, suddenly entering a narrow gorge after Chalunka, continuing through Turtuk and Tyakshi before crossing into Pakistan. The similarity in the courses of these two important rivers probably indicates a series of paleo fault lines trending NW-SE in delimiting the upper courses of the rivers. The importance of the Indus and the Shyok rivers is in the deposition of the thick Quaternary sediments—a treasure trove for geology researchers.

Fig.1: Overview of The Roadsector

1.2 Daulat Beg Oldi, which is in discussion at times, lies near the easternmost point of the Karakoram Range in a cold desert region. The place is named after Daulat Baig, a 16th-century Yarkandi nobleman who is supposed to have died at this place after descent from the Karakoram Pass. Daulat Baig Oldi literally means, "Daulat Baig Died". The trade route via the Karakoram Pass was used by caravans traveling between Leh and the Tarim Basin. DBO is believed to have been a stopping point for the caravans but not known to have any permanent civilian population. Temperature in the region plummets as low as -55 ^{0}C in the winters. The weather deteriorates frequently with strong icy winds and the region has very little vegetation or wildlife.

1.3 During 2001, it was decided to construct a motorable road from Leh to its destination at Daulat Beg Oldi to provide connectivity to the extreme border areas. Initially temporary bridges of 24 R load classification where launched however subsequently it was decided to replace/construct class-70R bridges. As the river has very wide water way at this place, it was decided

to construct a multi – span bridge at Km 150.400 on this road.

1.4 As this bridge has multiple span (i.e 10 span) aspect of foundation for intermediate pier has been the major issue of concern. Sinking of well in such regions having limited working period, sub-zero condition is a tedious work which was not found feasible. Use of conventional pile due to problem of conveyance of conventional pile rig to such location because of tough terrain conditions and geomatrics of the road Manali – Leh or Srinagar –Leh was also not found a feasible pre-position. Because of excessive dewatering efforts and continuous flow in the river and rise in water level during summer, open foundation was also not found practically feasible to construct.

1.5 The bridge site is a deserted zone at an elevation of 14,500 feet with glacier valleys all around and during summer season glacier melts which increases the water level of Shyok River and damages the road which causes problem in traffic movement.

Fig. 2 : Over View of Br Site

This phenomenon is more pronounced at the selected location. No well-defined course is available at upstream side. The river has witnessed frequent occurrence of

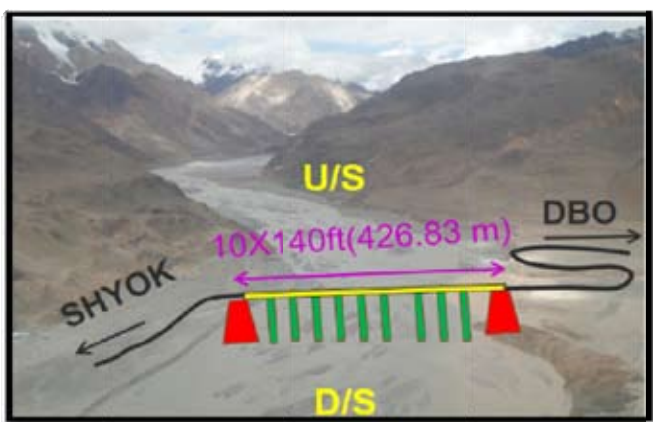


Fig. 3 : Panoramic View of Bridge Site

Glaciated Lake Outburst Floods (GLOF) and the effluent increases due to global warming.

1.6 Keeping these issues in view and after detailed deliberation on various construction techniques available to speed up the bridge construction works in such region, it was decided to provide micro pile foundation for this bridge at KM 150.40 on D-S-DBO road for piers and as also for the abutments.

2.2 The river bed consists of Bouldary, Gravel and pebble strata. The bridge location is very remote and severely under developed. The working season available for construction is of only 4–5 months. Efficiency of machinery and manpower also drops down drastically due to high altitude, adverse climate and lack of oxygen.

2. FINALIZATION OF BRIDGE SCHEME

2.1 The River Shyok over which the bridge has been proposed, is fed by snowmelt water from numerous glaciers. The summer temperature in the valley exceeds 30 °C and the winter temp drop to – 42 °C. The river flow and the sediment load that the river carries are the highest from June to Sep.

2.3 Upon analyzing various alternatives, considering their technical viability, feasibility of induction of machine, availability of contractors for execution, ease and speed of construction and discussion held at high level committee, Construction of a 10 x 140 feet Triple Double Reinforced (TDR) CL 70R Extra Wide Bailey Bridge (EWBB) was planned at this site on micropile foundation with following mode of execution:-

- Phase I: - Carrying out of SSI, Micro piling load test, design of substructure and construction of sub structure on micropile foundation (up to bottom of the pile cap) through an execution contracton the basis of design and construction system.
- HFL- 4378.106 m
- OWL- 4375.650 m
- Effective Linear Water Way- 513.054 m
- Deck Level- 4380.30 m
- Founding RI -4369.50 m
- Scour Level for Pier- 4371.818
- Phase II: - Construction of balance sub structure departmentally.
- Scour Level for Abutment- 4374.962 m
- SBC- 40 T/Sqm

- Phase III: - Procurement of Bailey bridge equipment and launching of 10x140ft TDR Cl 70R EWBB from original manufacturer
- Super structure - 10x140 ft Triple- Double Reinforced Load Class 70 R, Extra wide Bailey Bridge 2.4 The salient features of this bridge are as under:-
- Design Discharge - 3225.87 cum/sec
- Foundation Micropile (273 mm dia, 21mtr deep)
- Piers – 49 Nos
- Abutment -69 Nos

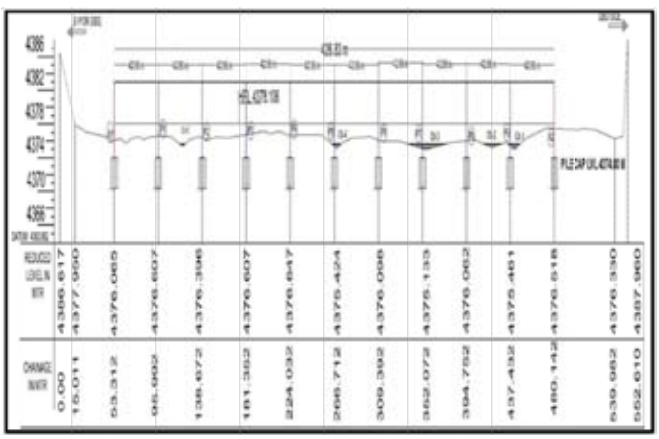
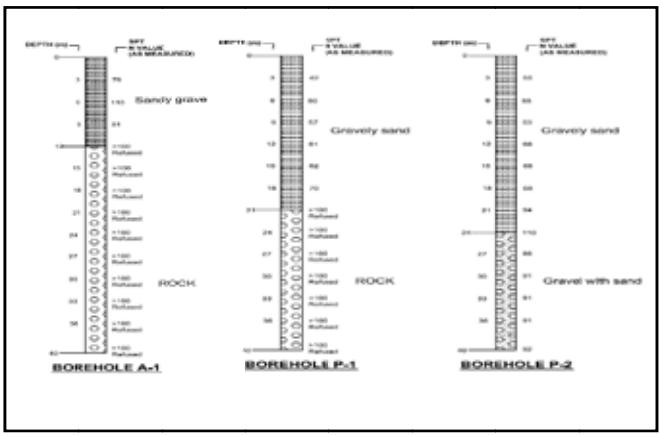


Fig. 4 : General Arrangement Drawing of The Bridge

3. Sub-Surface Investigation

3.1 For the design of micropile foundation a detailed investigation were carried. In all, nine boreholes had been carried out. The boreholes had been explored to a depth ranging from 24 m to 40 m. The strata at the site was observed to consist of variable depths of Gravelly Sand followed by highly weathered Granite type material (UCS 20 to 60 MPa) up to the explored depth of 40.00 m in the various locations assessed. It is also found that the strata at site top layers being sandy gravel to

Fig. 5 : Samples Collected During Ssi



gravelly sand with high SPT N values is competent to support the micro piles. The sandy gravel and gravelly sand layers in the strata is also found to be ideally suited for permeation grouting, i.e. replacement of pore water with cement slurry which is again ideal for micro piles, as well as formation of bulb.

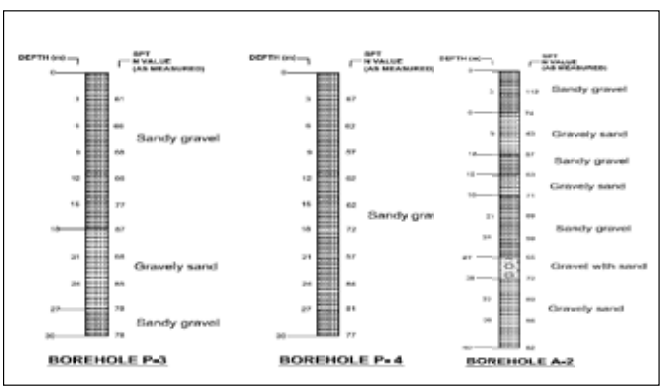


Fig. 6 : Soil Strata at Borehole Locs

4. Method Statement for Micropile Foundation:-

4.1 Micro Piles are deep foundation elements, constructed using high strength, small diameter steel casing. The micro pile casing is advanced to the design depth by drilling. The steel casing has perforations to allow flow of grout into the surrounding strata for consolidation and increasing the strength of the soil.

Re-drilling is carried out after the grout attains minimum strength. Reinforcement cage is lowered into the drilled hole and grouting is done from bottom to top. The outer diameter of micro piles is 273 mm. With 8 mm thick casing, the inner diameter is 257 mm.

4.2 Micropiles were planned to be executed with a unique execution strategy. Tube-a-manchette (TAM) grouting was taken into consideration for the purpose of their installation. On rigorous analysis and design, it is concluded to put vertical as well as raker piles (with 1:6 configuration) of 21.00 m depth.

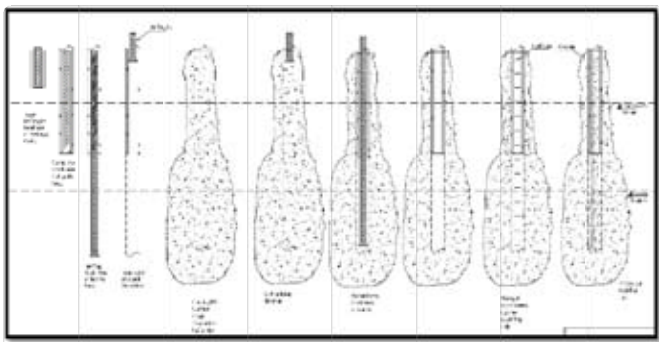


Fig. 7: Method Statement For Micropile

Fig. 8 : Installation of Micropile At Bridge Site

5. LOAD TEST:-

5.1 Vertical Load Test:- All the load tests have been carried out on individual vertical piles. The targeted design pile capacity was 40 ton. All the piles have been tested up to a load of 80.00 ton. On individual piles vertical load tests- three nos. for Abutments and 10 nos. for Piers have been carried out. In order to simulate the scour condition, the soil up to 3.0 m depth was excavated from pile cap bottom level. The results obtained from Vertical load test have been presented at table. The piles have behaved exceptionally well, also because of the presence of the casing pipe and the grout body around the pile.

Fig. 9: Vertical Load Test

Table -1 : Vertical Load Test Result

S/No.	Test No.	Settlement of Load of 40 t (mm)	Settlement of Load of 80 t (mm)
1	P1	2.86	6.13
2	P2	2.76	6.19
3	P3	2.2	4.71
4	P4	3.05	6.26
5	P5	3.01	6.33
6	P6	3.2	7.09
7	P6(ii)	2.15	5.9
8	P7	2.75	5.85
9	P8	3.92	8.12
10	P9	3	7.76
11	A1	1.6	3.12
12	A1(ii)	1.47	3.57
13	A2	3.72	8.9

5.2 Lateral Load Test:- The lateral load tests have been carried out on individual vertical piles on Piers P1 to P3, P6 to P9, Abutments A1 & A2 at a free head condition for a design load of 1.25 Ton and a test load of two times the design load, i.e. 2.5 Ton.

Fig.10 : Lateral Load Test

Table -2 : Lateral Load Test Result

S/No.	Test No.	Displacement at Design Load of 1.25 T (mm)	Displacement at Design Load of 2.5 T (mm)
1	P1	1.4	2.35
2	P2	1.03	1.64
3	P3	1.27	2.22
4	P6	1.38	1.97
5	P8	1.38	1.78
6	P9	1.25	2.18
7	A1	1.83	3.25
8	A2	1.42	2.28

5.3 Lateral Load Test Result on Pile Group:- At three pier locations- P1, P2, P8, pile group consisting of one vertical and one raker pile (1 Horizontal to 2.74 Vertical; 20 degrees to the vertical) were tested for lateral load for a design load of 11.00 Ton; test load of 22.00 Ton (Twice as the design load).

Table -3 : Lateral Load Test Result on Pile Group

S/ No.	Test No.	Displacement at Design Load of 1.25 T (mm)	Displacement at Design Load of 2.5 T (mm)
1	P1	1.51	2.31
2	P2	1.95	3.57
3	P8	1.54	2.45

5.4 In case of vertical piles subjected to lateral load test the soil up to 3.0m depth was excavated to simulate scour condition. As per IS 2911, even for driven piles, the raker should be normally restricted to 1 horizontal to 6 vertical. So the same has been adopted for the execution.

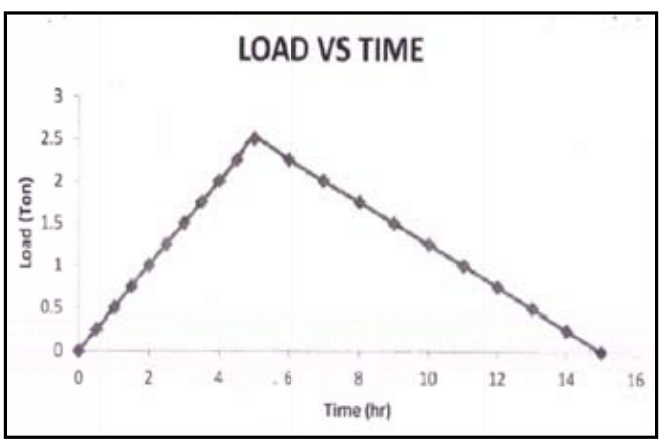


Fig-11 : Load Vs Time

5.5 Based on the satisfactory test results on micropiles, the superstructure loads were taken into account to pursue the design of sub-structure and micropile. The severe and rare design load combinations with seismic conditions as per codal provisions were considered in serviceability limit state for evaluating loads coming on the sub-structure and foundation to achieve the selection of proper foundation characteristics. On analysis and computations the axial capacity of 40.00 Ton and lateral capacity of 1.25 Ton was achieved for the individual micropile considered for use. The combination of vertical as well as raker micropiles were considered for achieving safe lateral support to the bridge structure.

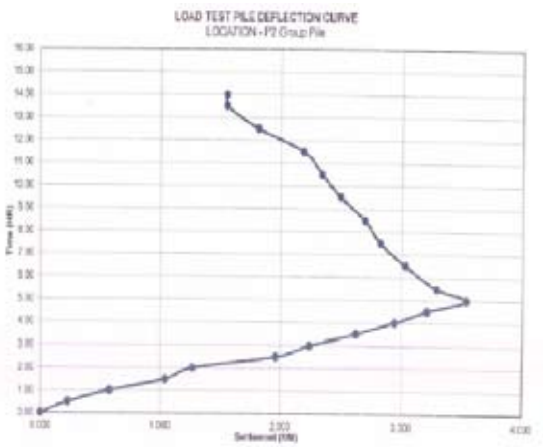
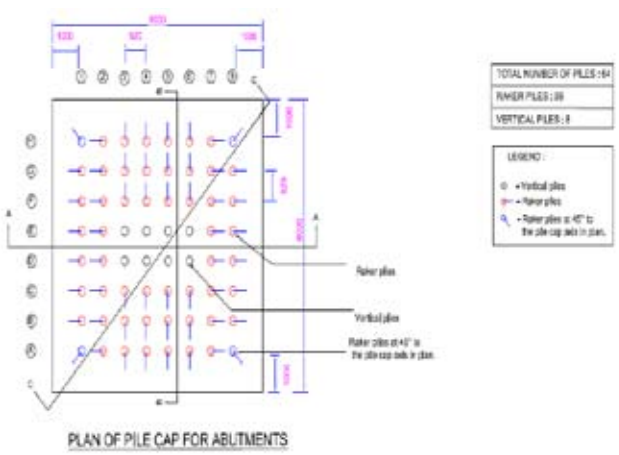


Fig-12:

5.6 A pile group consisting of 69 micro piles is proposed for the abutments and 49 nos for the piers. The diameter of the micro pile is 257 mm. The length of the micro piles is 20 m below the bottom of pile cap which penetrate 1 m into the pile cap. The figures as placed opposite shows the plan and section of the micro pile group for the sections. The load sharing between the vertical and raker piles depends on the relative stiffness of these



piles in the vertical and horizontal directions, which is particularly critical in case of horizontal loads.

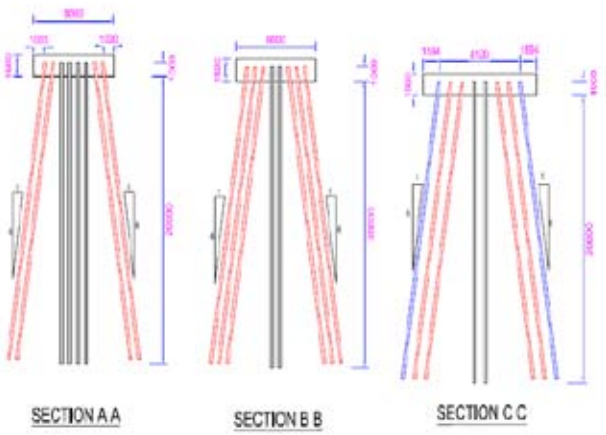


Fig-13: Layout Plan of Micropiles

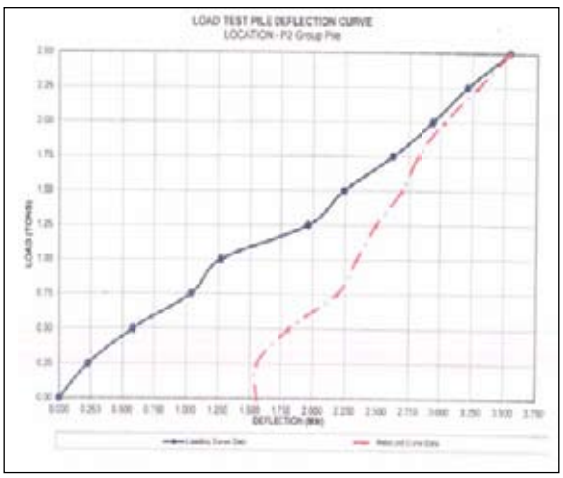


Fig -14 : Micropile Foundation During Construction

Fig -15: Reinforcement Scheduling for Pile Cape

6. DESIGN AND LAUNCHING OF SUPERSTRUCTURE

6.1 The design and launching of ten spans of 140 ft Cl 70 TDR EW bailey bridge was an another challenge as this configuration was decided as per strategic requirement, quick launching for reducing construction time and maximum span available in India for Cl 70 bailey bridge. The contract for fabrication and launching of the bridge as per Memorandum of Understanding was given to Br & Roof company, Kolkata. The design of this bailey bridge is standard in nature, however launching of the same was carried out as continuous bridge by having a special arrangement of link reinforcement at each pier location. The launching of this superstructure has been completed in only one working season.

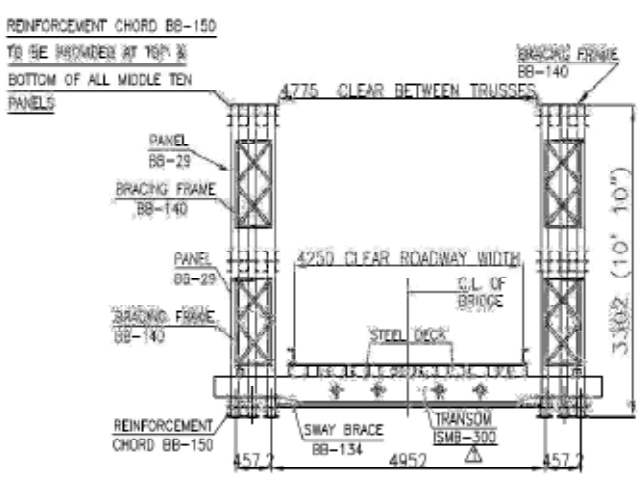


Fig-16 : Details of Truss Arrangement

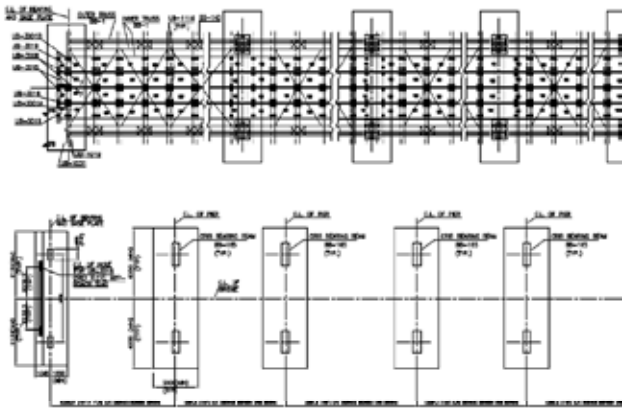


Fig -17 : Details of Superstructures Arrangement

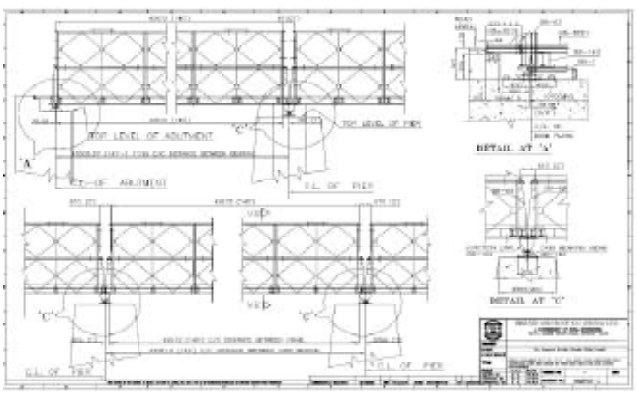


Fig -18 : Details of Bearing Arrangement

Fig -19 : Superstructure Launching Under Progress

Fig -20 : Completed View of Bridge From Shyok Side

Fig-21 : View of Bridge with Approach Road

7. CONCLUSION

7.1 The Ladakh region where induction of heavy machineries is not feasible due to difficult terrain enroute, the working period is very limited, the efficiency of man and machines drops considerably, the concept of micropiles as used for this project as the foundation work though has been a deviation from Codal provisions but has been found very effective and bridge could be built in four working seasons including super-structure, which is a big achievement at such an altitude and inclement weather conditions. Further, optimization of structural capacity of pileswill enhance the applicability of this concept manifold as micropiles in India for riverine bridge has been used for the first time. Though micropiles are extensively being used for tunneling projects, buildings, bridges in urban area, slope stabilization but it is the first project where it has been used for riverine bridges. Micropiles may behave well under higher axial and lateral loads for bridge structures provided, the techniques and strength characteristics are applied to the fullest extent.

In the current project, the micropiles were adopted as bridge foundations and it behaved exceptionally well in all the scenarios including worst as well as rare load combinations. A new execution technology of TAM (tube-a-manchette) grouting has been adopted successfully for installing micropiles. This type of foundation has been found a quick and feasible execution strategy in remote and inaccessible places of work. This arrangement therefore need further studies and optimization of design to be adopted at large scale for riverine bridges where induction of heavy equipment or well foundation is not feasible.

7.2 The bridge has finally been inaugurated by Shri Rajnath Singh Hon'ble Defence Minister on 21 Oct 2019 and dedicated to the Nation. The bridge has been named as Colonel Chewang Rinchen Setu

for his significant contribution to the Nation while serving in Defence force.

Inauguration of Colonel Chewang Rinchen Setu by Shri Rajnath Singh Hon'ble Defence Minister

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Shri Anil Kumar* is currently the Addl Director General Border Roads (NW) Chandigarh. The officer joined Border Roads Organization in 1983 on completion of graduation and post-graduation from IIT, Roorkee. He has vast experience of 35 years on planning, design and construction of Roads and Bridges including Tunnels in North and North-East part of the country having difficult terrain and adverse climate conditions. In 35 years of illustrious career, the officer held various command and staff appointments and served the organization with distinction and exceptional commitment at each level.Planning of 8.080 Km long highway tunnel project across Rohtang Pass and other highway tunneling projects in higher passes of J&K, Himachal Pradesh& Uttarakhand, and construction of fencing along Indo-Bangladesh border in Tripura, Planning design and construction of more than 9000 m length of bridges in North and North-East part of the country. Construction of double lane road Jawai-Churaihari-Agartala, double lane of erstwhile NH-52 in Arunachal Pradesh and double lane of road Zozjila-Kargil-Leh in Ladakh Region have been his major contribution. In present appointment the officer has responsibility of planning, design and construction of Roads, Bridges and Tunnels in the North-West Border Area of our Country.

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CROSSING OF TUNNELS BETWEEN THE PILES OF OPERATIONAL METRO STATION

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ABSTRACT:

This paper is a case study of crossing of Delhi Metro's Line-7 tunnels between the piles of operational Line-6 Lajpat Nagar Metro Station. The pile foundations of existing station which were falling within the zone of influence of the tunnels were likely to get affected by the tunnelling and construction activities. To keep lateral deflection of pile and ground settlement under acceptable limit, it was necessary to reduce the effect of vibrations generated during TBM operation and movement of piles induced due to tunnelling. To reduce the mentioned effects, contiguous RCC piles were casted in front of existing piles of Lajpat Nagar metro station. The casted curtain of RCC piles reduced the lateral deflection and ground settlement under acceptable limit. Also to reduce vibra-tions, TBM speed was reduced while passing existing station. An extensive instrumentation monitoring system was placed adjacent to piles, piers and station structure to ensure safe working. This monitoring was covered on a 24 hour, 7 days basis by two shifts of moni-toring team.

1 INTRODUCTION

The total length covered by metro after the completion of phase-III of project assigned to Delhi Metro Rail Corporation is 329 km. A total of 140 km of the total length is added to the existing network of 189 km in phase-III. In this 140 km, underground twin tunnels of length 53 km (106 km tubes) are constructed in phase-III. The current work discusses about the challenges faced and process used in constructing these underground tunnels.

The network from Mukundpur-shiv vihar also known as line-7, constituted of 58.596 km. Line-7 has 39.479 km of elevated network and 19.117 is km underground network. There are total 38 stations out of which 26 are elevated and 12 are underground. The tunnels between South Extension and Lajpat Nagar metro stations of line-7 were cross-ing between the piles of operational line-6 Lajpat Nagar Metro Station. As shown in Figure 1, the depth of line-7 tunnels from the platform of existing Lajpat Nagar station was 10.5m and the minimum clearance from the existing piles was 2.4m. The pile foun-dations of existing Lajpat Nagar Metro station which were falling within the zone of influence of the tunnels were likely to get affected by the tunnelling and construction activities.

1.1 Ground Condition

The urban area soil in Delhi consists of a layer of alluvium, silt with contents of sand and clay including pockets of coarse material, on a rock bed of quartzite with varying weathered state according to depth. The construction of tunnels of line-7 below existing Lajpat Nagar station was carried above the water table. The ground condition at this location is below in Table 1.

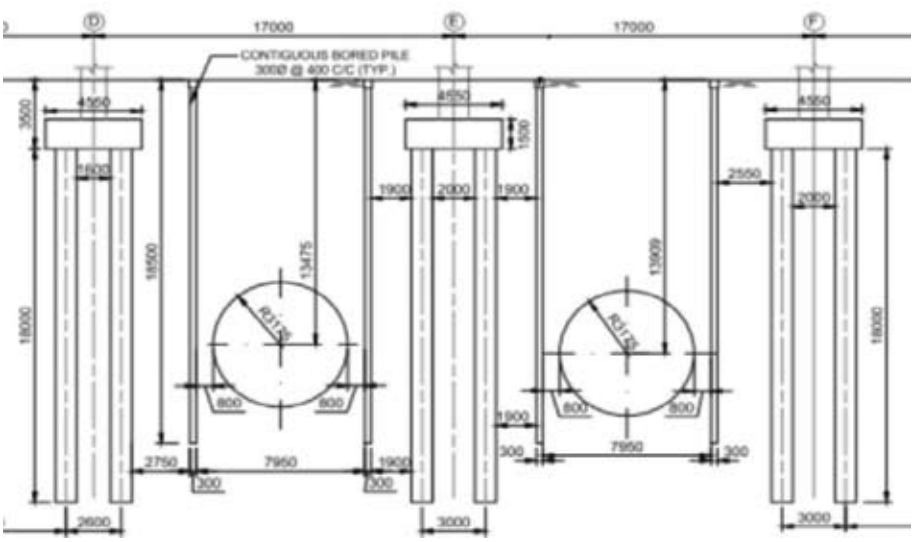

Fig. 1: Section showing distance between piles and tunnels

1.2 Assessment of Existing piles

A typical pile-tunnel configuration used for the assessment is shown in Figure 2. Depending on the location of the tunnel centre line with respect to the pile toe level, the pile can be assessed as either short pile (Lp/H < 1) or long pile (Lp/H > 1).

The stress-strain development mechanism induced by tunnel excavation is different for buildings founded on piles. It is not appropriate to consider green field settlements for build-ing risk assessment for buildings founded on piles because the building will not settle along the ground settlement trough. Building settlement is the result of pile settlement as sown in Figure 3.

Tunnelling-induced effects on existing pile foundations was assessed under long pile cat-egory since Lp/H > 1. To overcome this care has been taken to restrict the ground settlement with in 5mm.

1.3 Potential Risk to Existing piles

Due to tunnelling induced ground movements and vibrations generated during TBM operation, existing pile foundation of existing Lajapat Nagar Station falling within the zone of influence of tunnels was subjected to

Table 1. Soil Parameters

Layer #	Depth (m)	N60	CN	(N1) 60	Ko = (1-sin Φ') OCR0.5	Permeability (x10-6) cm/s	Design Parameters				
							Bulk Density	Strength Parameters		Stiffness Parameters	
							γb	c'	Φ'*	E'	υ'
							kN/m3	kPa	deg	MPa	
Layer -1	0–5	14	1.22	17	0.53	3	18	-	28	10	0.30
Layer -2	5–15	46	0.85	39	0.46	8	19	-	33	32	0.30
Layer -3	15–28	78	0.55	43	0.44	24	20	-	34	55	0.30

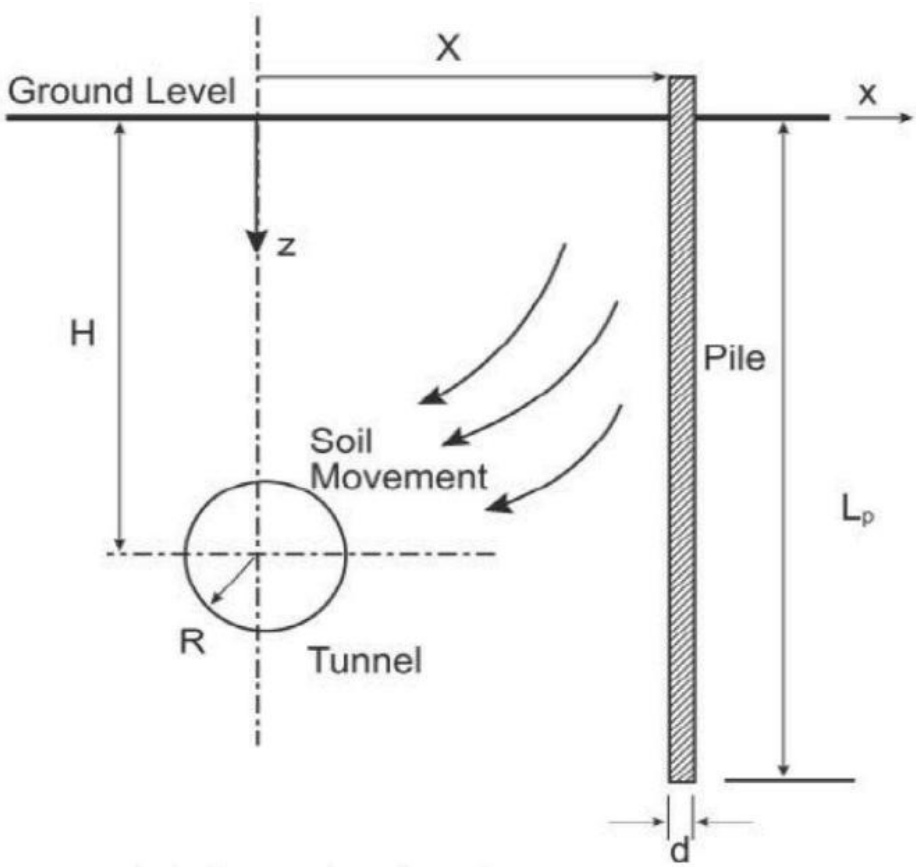

Fig. 2: Typical pile-tunnel configuration

high lateral deflection during twin tunnel excavation. By analyzing the behavior of piles during tunnel excavation in PLAXIS-2D, it was observed that Lateral deflection in few of the piles was more than 5mm, which was not acceptable. The vertical settlement of ground observed at above the twin tunnel locations was in the range from 23mm to 95mm, which was too high. Ground settlement adjacent to the pile also induced negative skin friction on pile. This negative friction added extra load on the pile.

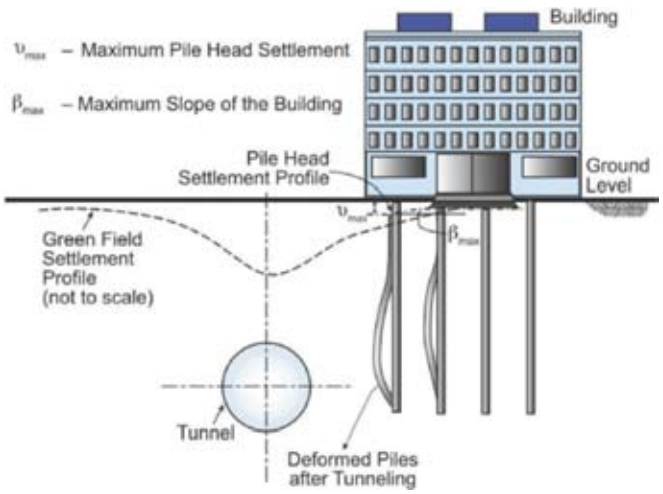

Fig. 3: Schematic diagram of pile settlement profile and ground settlement

Also, the vibrations generated during tunneling operation was to be minimized as the same would have resulted more lateral deflection and ground settlement in the proximity of twin tunnels.

Considering the safety of commuters and importance of the existing Lajpat Nagar Station, protection measures were required for the structure to reduce the impact of tunnelling on the pile foundations and on the station structure.

2 PROTECTION MEASURES

Considering the safety of commuters and importance of the existing Lajpat Nagar Station, protection measures were required for the structure to reduce the impact of tunnelling on the pile foundations and on the station structure.

2.1 Contiguous piles

To control the lateral deflection of existing piles and vertical settlement of ground, it was considered to drive the contiguous piles in between existing piles and tunnels. In order to reduce the lateral deflection of existing piles and vertical settlement of ground as shown in Figure 4, 300mm diameter of contiguous piles with 400mm c/c spacing was driven to reduce the volume loss of soil. The distance between contiguous piles and tunnel was kept 800mm.

The actual site photographs showing the machinery used for carrying piling work and the progress of work is shown in Figure 5.

The insertion of contiguous bored pile formed a curtain in front of existing piles (as shown in Figure 6) and from PLAXIS analysis, it was observed that there was drastic reduction in vertical ground settlement and lateral deflection of pile after driving the contiguous pile at and around the twin tunnel surface and deflections of existing piles analyzed after insertion of contiguous piles were within the permissible limit.

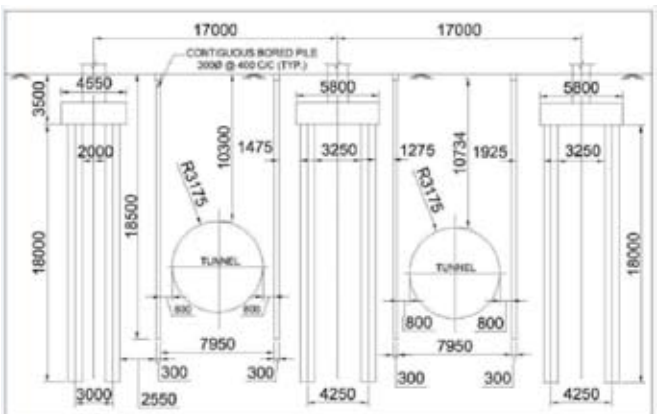

Fig. 4: Insertion of contiguous bored pile

Fig. 5: Machinery used for carrying piling work and the progress of work

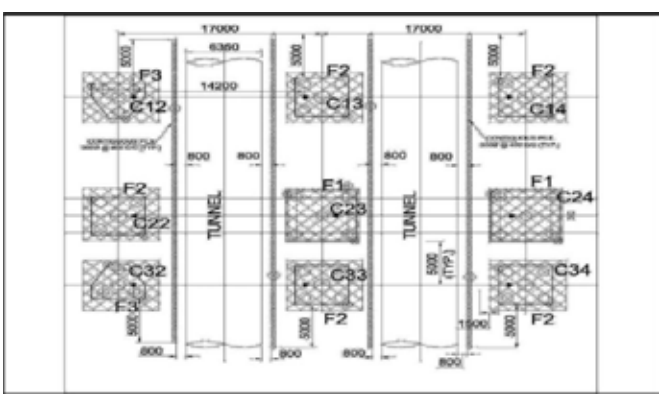

Fig. 6: Plan showing pile cluster and curtain of contiguous bored pile

The movement of ground before and after insertion of contiguous piles is shown in Figure 7.

The comparison of lateral deflection of existing piles and vertical settlement of ground before and after the drive of contiguous piles for the most severely affected piles has been shown in Table 2 and Table 3 respectively. The ground settlement at most critical location at the time of excavation of twin tunnels has been shown in Table 4.

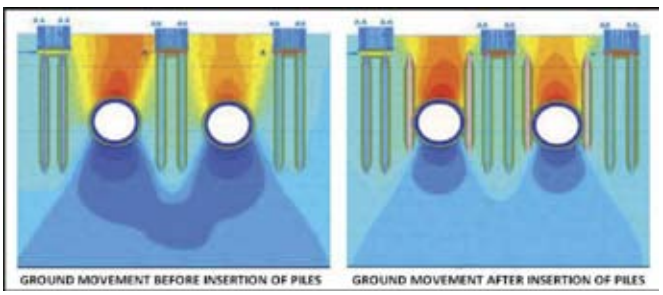

Fig. 7: Comparison of ground movement before and after contiguous piles

Table 2. Lateral deflection of existing piles

Column/ Footing	Pile	Length of Pile	Lateral deflection (mm)	
			Before contiguous piles	After contiguous piles
C12/F3	P3	18	10.664	2.683
	P4	18	10.669	2.681
C13/F2	P5	18	6.774	3.022
	P6	18	6.018	3.026
C14/F2	P7	18	7.477	3.082
	P8	18	3.917	2.267

Table 3. Vertical deflection of existing piles

Column/ Footing	Pile	Length of Pile	Vertical deflection (mm)	
			Before contiguous piles	After contiguous piles
C12/F3	P3	18	57.338	3.473
	P4	18	57.049	3.146
C13/F2	P5	18	51.505	4.237
	P6	18	51.867	4.568
C14/F2	P7	18	48.799	2.041
	P8	18	48.658	1.688

Table 4. Ground settlement during tunnel excavation

Stage	Ground Settlement (mm)	
	Before contiguous piles	After contiguous piles
Excavation of Twin Tunnel	93.62	19.95

3 INSTRUMENTATION MONITORING

Instrumentation monitoring by precise leveling was undertaken to measure actual ground movements. This monitoring was carried out to verify that the actual ground/building settle-ments were as predicted, which ensured structural movements and damages to existing piles and buildings was under the considered limits.

Monitoring of buildings and structures within the zone of influence of tunnel was carried by using following instruments:

a. Building settlement marker - Monitoring by precise leveling was carried to measure actual ground movement on all buildings

b. Optical survey targets - On the piers, at height optical targets were placed and 3-D moni-toring was carried

c. Ground Settlement marker - To monitor the settlement of ground level, ground settlement markers was used and precise leveling was carried to monitor the movement of ground

d. Tilt Meters - To monitor the movements of piles, tilt meter was placed on piers and pile caps.

To ensure safety of commuters using the operational line, it was necessary mitigate any damage to the existing metro line. For the same at track level of existing metro line, 24 hours real time monitoring was carried with the help of beam sensors.

3.1 Frequency of monitoring

Based on the instrumentation type and location, the construction progress and criticality of activity, frequency of monitoring adopted is mentioned in Table 5.

3.2 Review levels

Instrumentation monitoring review levels was defined in accordance with the contract and acceptable parameters. Trigger Values was be established as approximately 70% of the Design

Table 5. Frequency of monitoring

Instrument	Monitoring frequency		
	Prior to tunnelling	During tunnelling and within 10m boundary region of station location	After tunnelling
Building Settlement	Weekly	Every 6 hours	Weekly
Markers			
Optical Survey targets	Weekly	Daily	Weekly

Ground Settlement Markers	Weekly	Every 6 hours	Weekly
Tilt meters	Weekly	Daily	Weekly
Beam Sensors	24 hours real		
	time monitoring		

Table 6. Review levels

Instrument	Trigger level	Allowable level	Design level
Building Settlement Markers	7mm	9mm	10mm
Optical Survey targets	3.5mm	4.5mm	5mm
Ground Settlement Markers	17.5mm	22.5mm	25mm
Tilt meters	1/4000	1/2550	1/2000

Values (limiting values) and allowable levels as 90% of design values. The Limiting Values will be set as stated in Volume 4 of tender document. The design value for the review levels were selected based on the actual calculation done on the PLAXIS model. The review levels for different instruments are shown in Table 6.

4 CONCLUSION

The project was challenging and allowed us to learn quite a lot of things. Firstly, it was observed that vibration can significantly increase in case the TBM speed was high. Hence, a bare minimum speed was kept. The reduction in the speed of TBM also helped in controlling the deflection and settlement. Further, isolating structures were found more suitable than going for the ground stabilization. Also, instrument monitoring is a rigorous process which was religiously followed for identifying any movement of ground structure. After the insertion of contiguous piles, the deflection of most critical pile was also brought under acceptable limit i.e. below 5mm. The ground settlement also reduced to below 20mm.

The exhaustive instrumentation monitoring explained above and by taking protection meas-ures, the tunnels were successfully constructed below the operational Lajpat Nagar metro sta-tion. No damage has been observed on the structures coming in the zone of influence of tunnels and no change has been observed in the track level of existing metro line.

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TUNNELING CHALLENGES IN THE BUSINESS CAPITAL OF INDIA

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ABSTRACT:

Underground Tunneling has always been daunting when it comes to some of the oldest and busiest cities of the world. A lot of challenges await the construction team start-ing from the soil diversity to selection of most appropriate TBM's, maintaining safety of exist-ing structures and residents of the area being the topmost priority. In the present scenario, demand of Tunnel Boring Machines has suddenly increased worldwide to construct the Metro tunnels in Metropolitan cities, but issues related to vibrations caused by use of TBMs to nearby structures have been a matter of concern. TBM sometime based upon the soil profile are the source of vibrations that causes distortion due to its impact and displacements of material under the foundation of structures.

1 INTRODUCTION

The problem of tunnel-induced stresses produced at the time of construction and related risk assessments of buildings damages has attracted attention of investigators over the last many years and so these all have to be taken into consideration and analyzed at all times.

In this paper, we will go through the on-site challenges faced during tunneling of one of the tunnel drives from Mumbai Metro Line 3 (MML3) project considering few case studies.

2 PROJECT BACKGROUND OF MUMBAI METRO LINE 3

2.1 Geological Overview of Mumbai

Mumbai is located in the western coast of India, by the bank of the Arabian Sea. Back in the 16th century, Mumbai was known by the name "The seven islands of Bombay" because it is made from the cluster of 7 islands comprising of Isle of Bombay, Colaba, Old Woman's Island (Little Colaba), Mahim, Mazagaon, Parel and Worli. By 1845, the islands had been merged into one landmass by means of multiple land reclamation projects.

2.2 Regional Geology

Mumbai region is located in the great volcanic formation building up the Deccan plateau.

A wide variety of Basalts and associated rocks such as volcanic breccia, black tachylytic Bas-alts, red tachylytic Basalts etc. occur in the area covered by Deccan trap Basalts. Most Basalts are either compact i.e., with no gas cavities, or amygdaloidal with gas cavities filled with second-ary minerals, and vesicular Basalts with empty gas cavities. Zeolites are the commonest second-ary minerals filling gas cavities, though silica, calcite and chlorophacite also occur as infillings.

2.3 Need of the Project

Mumbai, the financial capital of India, has witnessed phenomenal growth in population and employment and the trend is expected to continue in the future. Four-fold growth of popula-tion since 1951 has been largely accommodated in the suburbs while the highest concentration of jobs has remained in the Island City. The physical characteristics of the City are such that the suburbs have been constrained to spread northwards only, and all transport facilities are concentrated within three narrow corridors. Today's major challenge is to provide connectiv-ity and promote growth by providing additional infrastructure, which would improve the quality of life of the residents.

2.4 About Mumbai Metro Line 3

Mumbai Metro line 3 connecting Colaba- Bandra-SEEPZ is the first underground Metro line in Mumbai metro system having 26 underground stations and 1 at grade station. The total length of MML-3 is 33.5 kms and the total cost of the project is $ 3.19 billion with 57.2% soft loaned from Japan International Cooperation Agency (JICA) and rest 42.8% from Government of India/Government of Maharashtra/Others. It is further bifurcated into seven contract packages namely UGC01 to UGC07 and is implemented by The Mumbai Metro rail Corpor-ation Limited (MMRCL). Once completed, Mumbai Metro Line 3 will cater to 1.39 million daily ridership by 2021 as estimated.

The case studies in this paper pertain to package UGC04 of MML3 corridor which comprises of 3 underground

stations named Siddhivinayak, Dadar and Sitladevi and associated bored tunnels (cumulative tunneling length of 10992 meters), cross passages and NATM tunnels at Sitladevi station. The project is being executed by a Joint venture of Continental Engineering Corporation, ITD Cementation India Limited, and Tata Projects Limited (CIT JV).

2.5 Brief Description and Alignment of UGC 04 of MML3

Fig. 1. Tunnel alignment for UGC 04.

Table 1.	Project Brief
Project	Mumbai Metro Line 3 – UGC 04
Client	Mumbai Metro Rail Corporation
GC	MAPLE Consortium (Joint Venture of AECOM, Padeco, LBG Inc. and Egis Rail)
Scope	Design & Construction of Underground Section including Three Underground Stations at
	Siddhivinayak, Dadar & Sitladevi & Associated Tunnels together
Location	Worli End to Dharavi Start
Tunnel	Inner Dia – 5.80 M
	Outer Dia – 6.35 M
	Segment Width – 1.4 M (Universal Type)
	Segment Thickness – 275 mm.
Stations	Siddhivinayak (250m L)– Bottom Up with secant pile
	Dadar (324m L) – Bottom Up with secant pile
	Sitladevi (240m L) – Bottom Up with secant pile

2.6 Geological profile along package UGC-04

Rock type Basalt (about 75%) and Volcanic Breccia and Tuff (about 25%) are encountered along the alignment with some amount of intertrappean shale (interbedded with Basalt or Volcanic Breccia). The ballast rock generally having high intact strength varies between 5 to 73 MPa. Rock is mostly within weathering grade II to III followed by grade I. Thin bands of tuff are present beneath the soil layer having a weathering grade of V and IV. The bored tunnel section consists mostly of Basalt in weathering grades I to III except in the tunnel por-tion after Nayanagar launching shaft where Breccia of grade I to III is encountered. Basalt rock does not have very high abrasivity, but locally intact rock strength could be high. Perme-ability values are generally low to moderate.

We were dealing with face conditions with rock mass varying from grade 1 to 4 throughout the alignment.

3 SELECTION OF TUNNEL BORING MACHINES

After the detailed soil analysis, selection of TBM was most important, as tunneling was going to be executed in the city that had undergone various land reclamation projects and any sur-prises may await the execution team, so several criterions were considered before selecting the appropriate TBM and its specifications.

Based upon the geology pertaining to UGC package 04, Earth Pressure Balancing Shield (Herrenknecht AG) was chosen as the best-suited TBM w.r.t. required progress and safety of Tunneling Operations & existing building structures (EBS).

Fig. : Tunnel Boring Machine used in UGC-4.

3.1 Cutter Head design

The cutter head was designed with an opening ratio of 33% and for the rock strength of UCS upto 150 Mpa. This anticipated rock strength was based upon the available geological data from the tender document that was subsequently verified with respect to detailed geological investigation conducted in the stretch.

4 CHALLENGES WHILE TUNNELING

While we tried to cover all the known risks through the selection of TBM and design of permanent structures but we still faced the following challenges presented in the form of case studies:

1. Launching of TBM.
2. Tunneling below old buildings.

4.1 Launching of TBM

The major tunneling activities for this package were exclusively focused on the construction of the TBM launching shaft at a place called Nayanagar, in Dharavi area of Mumbai as almost 50% of the total tunneling of the package was planned to be executed from this shaft.

As launching of 2 TBMs and 4 drives were to be executed from this shaft, so its construction on time was one of the critical activities for the project. The following challenges were faced as briefly described below:

4.1.1 Land for TBM Launching Shaft Construction

The land selected for the shaft construction was amidst a densely populated slum area, Dharavi which is the 2nd largest slum in Asia and the 3rd largest in the world. 404 hutments were to be removed to clear the space for shaft construction.

For land acquisition and hutment removal all the families living in the are ahad to be relocated; so a special resettlement and Rehabilitation policy for Project Affected Person (PAP's) was framed by the State Government.

4.1.2 Resettlement and Rehabilitation of Project Affected Person (PAPs)

i. State Govt. approved to make Mumbai Urban Transport Project Resettlement and Rehabilitation (MUTP R&R) policy applicable to the project PAPs GR dt.03 March 2014.

ii. Around 2800 families are affected by project.

iii. Social Impact Assessment (SIA) report was prepared by General consultants as per Japan International Cooperation Agency's (JICA) Socio Economic and Environmental guide-lines in Jan 2016 after incorporate changes in impacts.

iv. Public consultation were conducted with all affected slum dweller families during year 2016.

v. Availability of tenements; Hon'ble Chief Minister had approved 4000 nos. of tenements for ML3 PAPs from Slum Rehabilitation Authority scheme at Andheri, Chakala, premier compound Kurla (W), Bhandari metallurgy Kurla (E) and Mahul. SRA handed over 500 tenements at premier Kurla. SRA also handed over 34 tenements to MMRCL at Antop Hill Wadala.

Fig. 3.TBM Launching Shaft Area - Nayanagar.

vi. TBM launching shaft for package 4 of is constructed on a plot of 3355 sq. m by clearing encroachment 404 hutment/slum of Govt. Land.

vii. Baseline Socio Economic Survey (BSES) was conducted by General consultants as a part of SIA to decide the eligibility for Resettlement and Rehabilitation of Project Affected Person (PAPs) on the said plot at Nayanagar.

viii. After detailed scrutiny of BSES forms and SIA report list of eligible PAPs was prepared by R&R cell of MMRC and accordingly public consolations was conducted at MMRC office with Eligible PAPs.

ix. Alternative options were discussed with PAPs regarding location of resettlement, residential commercial etc. and PAPs agreed for Premier Kurla Resettlement sites.

x. Tenements were allotted to eligible PAPs through Computerized lottery system.

xi. After collection of all requisite documents, allotment letters were issued to PAPs for tene-ment and 1-week time was given for vacating the hutments at Nayanagar in October 2016.

xii. Transportation arrangement were provided by MMRC/Contractor for shifting of home luggage of PAPs to Kurla premier site.

4.1.3 Space Constraint for TBM Operations

Fig. 4. Shaft Area after TBM Arrival.

As 2262.42 sqm (Excluding the Shaft Area) space was available for setting up of two TBM back up services like Grouting Plants, Cooling Towers, Mucking Gantries, Muck Pits and TBM Power supply which was a challenging task w.r.t. movement of heavy machineries and cranes etc along with maintaining the planned tunneling progress.

This challenge was tackled by carefully planning the gantry crane setup for parallel operations of two TBMs and optimized utilization of the available land.

4.2 Conclusion

The first ring for TBM1 (named as Krishna 1) was erected on 06 November 2017 beginning Initial drive of the first TBM of Mumbai Metro Line 3 from Nayanagar Shaft towards Dadar Station, in the presence of Honorable Chief Minister of Maharashtra Mr. Devendra Fadnavis. It is also notable that UGC04 has the longest stretch for tunneling among all the seven pack-ages of MML3 and so timely start of tunneling drive was necessary.

4.3 Case Study 2: Tunneling below Old Buildings/ Dilapidated Buildings

This case study takes few examples to define the challenges that were faced while tunneling below some of the dilapidated buildings and the major steps taken for safely executing the project.

Many buildings in Mumbai are currently unfit as a habitat. Some of such buildings in the influence zone* of our TBMs and Station area, have to be analyzed for the impact of the construction activities. Therefore, a pre-construction condition survey was carried out and based upon which a protection plan consisting of monitoring and structural support plan was framed for these buildings.

*Influence Zone: The 50 meters distant area parallel to the TBM centerline on both sides is defined as the Influence Zone of the TBM.

4.3.1 Pre-Construction Condition Survey

After conducting the Pre-Construction Condition Survey of all the buildings of the entire stretch lying in the influence zone, the reports were submitted to the DDC (Detailed Design Consultants for Contractor) for analysis and categorization of buildings was done according to the degree of estimated damage to the buildings. After studying the survey reports of all the buildings, it was concluded that most of the buildings did not require any structural repairs and were competent to bear the effects of tunneling activities but for the dilapidated buildings, special provisions were made to take care of the buildings and their residents.

4.3.2 Building Protection Plans

Structural audits were also conducted by an MCGM (Municipal Corporation of Greater Mumbai) authorized agency and reports were submitted to DDC for incorporating the necessary provisions to protect the buildings in the building protection plan. Some of the special provisions included:

- Structural repairs of various structural members of the buildings.
- Real- Time Monitoring.
- Increasing the frequency of monitoring of buildings in the bubble zone of the TBM.

*Bubble Zone: The zone lying 100 metres behind the TBM and 50 metres ahead of the cutter head.

4.3.3 Tunneling below Dilapidated buildings (St. Xavier Institute of Engineering)

The problem with tunneling in Mumbai under dilapidated buildings was that for many of the buildings the as built drawings of structures were not available and even if it was available, the actual geometry of the buildings had been altered by various repair works undergone over due course of time. As the tunneling activities started, our upline TBM had to pass below 2 such buildings just 110 metres away from the Nayanagar Shaft; St. Xavier's institute of Engineering (G+3; slightly damaged) and St. Xavier's Technical institute of Engineering (G+5; Severely damaged)

4.3.3.1 ST. XAVIER'S INSTITUTE OF ENGINEERING

The first building we encountered lying above the Tunnel alignment was St. Xavier Institute of Engineering (G+3).

- The building was assessed to be in category 2 (Slightly damaged) according to the BCS Report.
- In addition, the impact of Tunneling over the building was also found to be in category 2 (Slight Impact) by the Construction Impact Analysis Report.
- The architectural and structural details of the buildings were unavailable.
- At first, it was inferred that it has isolated RCC footings of depth 2.5 metres.
- Accordingly, the DDC prepared a monitoring plan considering the foundation to be iso-lated RCC footing.

- Further deeper research with the owners and local body it was confirmed that the building is resting on pile foundation.

After this revelation, a joint meeting among the stakeholders was conducted to carry on necessary research and analysis.

Initially we could not find any drawings for the building foundations but after digging in deep a blue print was obtained which had details about the piles of the building and it proved out to be very critical factor in analyzing the tunneling impacts on the building.

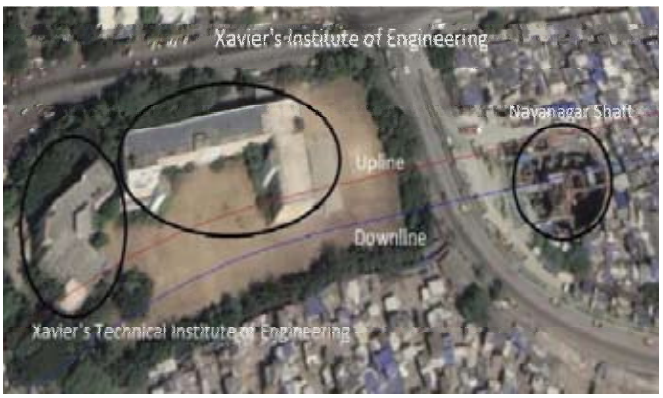


Fig. 5. Satellite view of Nayanagar Shaft and St. Xavier Institute of Engineering.

(1) Preliminary analysis:

- Preliminary analysis included Ground settlement analysis, Analysis of loads and Bending Moment and Building Condition Survey (BCS).
- Construction Impact Analysis and Building Damage Assessment were to be done for any concrete outcomes regarding the piles of the building.
- Numerical Modelling of the building Structure was conducted by DDC to analyze the Ground Settlement and loads on the structure.
- Monitoring of the structures using various instruments at regular intervals started.

(2) Further Investigation of building:

- After analyzing the pile layout, it was observed that the four columns along the edge of the building were coming above the tunnel alignment.
- It was mentioned in the pile drawings that the pile shall be embedded into the hard rock/stratum to a depth equal to the diameter of the pile. Any deviation in length of embedment shall be with the specific instructions in writing from the consultant.
- The pile foundation can become sensitive to the effects of tunneling. Piles risk a reduction of their end bearing capacity and skin friction resistance due to the displacements and ground stress redistributions that occur as a result of tunneling.
- Thorough study of the substructure and superstructure was conducted.
- The cross section across the tunnel alignment at the location of the building is modelled in RS2 (A Finite element software).

(3) The following reports were generated regarding the assessment of the structure. a) Ground Settlement Analysis Report:

Construction impact assessment is done considering the ground movements at 13m below ground level where the pile termination is expected. Whereas, the building will not undergo settlement to this extend since the skin friction along the pile will be mobilized and the vertical settlement is restricted.

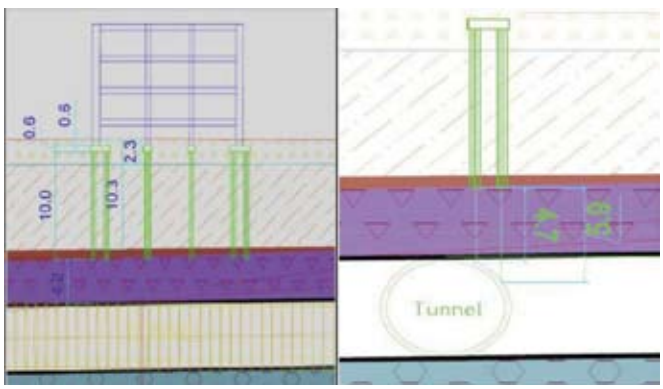


Fig. 6. Piles of St. Xavier Institute of Engineering with respect to the Tunnel Alignment.

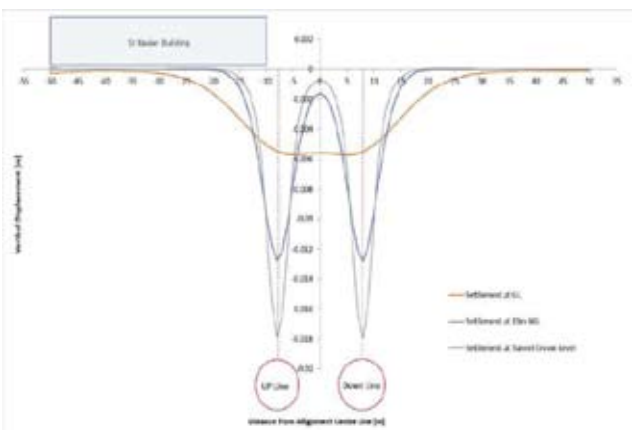


Fig. 7. Ground settlement description.

b) Analysis of Bending Moment:

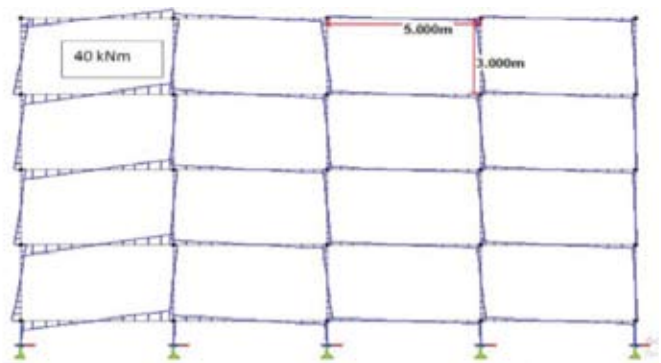


Fig. 8. Bending Moment Description.

The STAAD analysis of one bay of the building with differential settlement obtained from Finite Element Method at pile toe is applied as load which shows that the build-ing is capable of sustaining the additional forces coming on it. Also, the analysis shows that the bending moment produced in the columns is negligible and can be dealt by the skin friction of the pile and the rock surface.

c) Aftermath of the research done by DDC and CIT JV:

The following conclusions are drawn from the calculations and submitted designs: -

1. Based on the data from additional probe holes conducted near the institute building, rock is encountered at a depth of 11m to 12m from the ground. Rock is considered from 12m depth for the pile capacity calculations.
2. From the pile capacity calculations, it was noticed that the pile load was well within the pile capacity even after ignoring end bearing capacity of the pile.
3. It was noticed from Construction Impact Analysis that the expected settlement of the ground at the level of pile toe due to the TBM operation does not impact the structural integrity of the structure since it falls in slight category.
4. None of the piles were coming directly over the central axis of tunnel which is the highest point of boring. In fact, the pile away from the tunnel axis and towards the building side would have an even larger cushion of rock available between the pile toe and tunnel.
5. It should be noticed that these calculations are carried out for the two center columns. The piles for the edge columns will have an even lesser load.
6. It should also be noted that the calculations are done for a pile length of 13m where it terminates above the tunnel crown. Even if the pile extends beneath the tunnel crown, the skin friction alone will be sufficient to take up the load on to the pile even after applying the reduction factor. However, extreme caution should be taken during the TBM drive due to the close proximity of piles to the excavation and the uncertainty involved in the length of piles.

d) Recommendations:

The following recommendations were made in a joint meeting between the stake-holders after studying the results of the conducted tests to safely carry out the task: -

1. First 2 bays of the columns (approx. 15m from the edge of the building) near the building edge close to the tunnel alignment were evacuated. This was a preventive measure.
2. It was recommended to advance the TBM with slower rate and with closed mode to minimize the settlements.
3. Control of TBM face pressure – in order to minimize the settlements near the pile foundations of the building ahead of the cutter head, it was recommended to increase the face pressure a slightly higher (20-40 kPa) than the recommended face pressure. The recommended face pressures during TBM drive at the location of St. Xavier Institute of Engineering are as given below:

   At tunnel crown – 130 kPa

   At tunnel center line – 163 kPa At tunnel invert – 196 kPa
4. As a contingency measure, it was recommended to stack the props at site and readily available for propping if required
5. It was recommended to install 2 numbers of vibration sensors near the column locations of the building at ground level for the vibration monitoring due to TBM drive.
6. It was recommended to install some additional ground settlement markers along the alignment of tunnel centreline and also between the location of building and current TBM location for monitoring of surface settlements
7. The instantaneous filling of the “annulus”, which is created behind the segment lining at the end of the shield tail, was an operation of paramount importance. Its main goal was to minimize surface settlements due to the unavoidable over-excavation generated by the passage of the TBM. Therefore, it was recommended to carry out the tail grouting soon.
8. It was recommended to install 2 numbers of additional 3D optical targets on the building face towards tunnel alignment.

9. The frequency of the monitoring should be increased to every 2 hours. Any visible sign of distress in the building should be monitored and reported. The field instrumentation and monitoring reports shall be prepared and circulated to DDC/GC Design team on daily basis.
10. The vibration at the Ground Level due to the TBM operation shall be limited to not more than 4 mm/s.
11. The building should be monitored continuously using settlement markers, crack meters, tilt meters and 3D targets as per the submitted drawing and the additional instruments recommended as mentioned above (points 5 and 8).
12. Extreme caution should be taken during the TBM drive in the vicinity of the building due to the close proximity of piles to the excavation and the uncertainty involved in the length of piles.
13. Forward probing from TBM should be carried out to confirm the absence of piles.

e) Monitoring done by Instruments at site:

The following Instruments were installed at St. Xavier buildings:

(a) Crack Meters

(b) Ground Settlement Markers

(c) Building Settlement Markers d) Tilt Plates

(e) Vibration Meters

(f) Stand pipe Piezometer

(g) Bi- Reflex Targets

(h) Inclinometers

(i) Multi Point Borehole Extensometers (MPBX)

(4) Propping at vulnerable locations:

Fig. 9. Supporting Arrangements installed inside classrooms of the St. Xavier's Technical Institute.

4.4 CONCLUSION

The St. Xavier Institute building was the first uncertain building whose construction data was not available. Since the building's damage category was slight it was not posing a threat but continuing without ascertaining the pile location was never an option.

Similar analysis was conducted for the 2nd building i.e. St. Xavier Technical Institute of Engineering which was much more critical than the first building and it required extra prop supports considering the critical nature of the building.

Therefore, in the absence of proper foundation details, tunneling was carried out by proper analysis of the ground conditions, impact analysis, strict instrumentation monitoring, perfect operations and hence tunneling was completed without any damage to the sub structure or superstructure of the buildings.

TUNNEL FIRE PROTECTION OF THE LATEST GENERATION

High-tech detection systems for guaranteed fire-safety in tunnels

Great value is placed on safety. Schrack Seconet (LISTEC) has made it their duty to develop systems and solutions for people to increase their safety by offering products of the highest technical standard.

Proven know-how & the most modern technology

Reliable fire detection in environments with extreme conditions such as heat, frost or humidity places the highest demands on the technology. Detecting a fire at the earliest possible time and alerting the emergency services is the top priority at all times. The latest generation of Integral IP fire alarm control panels from Schrack Seconet successfully combines proven system features with trendsetting technology: Complex fire alarm networks are set up on the basis of Ethernet, thus enabling the integration of existing network structures.

The LIST (Linear Sensing of Temperature) fire alarm system, manufactured by LISTEC GmbH and supplied by Schrack Seconet AG, has been deployed for over 30 years for continuous temperature monitoring and the precise localisation of fires in tunnels. More than 500 tunnels, with nearly two million meters of sensor cable, are now monitored worldwide. The reliability of the system has been proven in many fire tests and also by the system's correct behaviour in the event of realfires. The LIST controller features the most modern technology with a clear text display and open communication.

The result is the greatest possible repeat accuracy of temperature measuring and very precise localisation of the fire, even at higher wind speeds.

Highest level of reliability thanks to full redundancy

Schrack Seconet's modular Integral IP fire alarm control panels are redundantly constructed: two independent systems constantly work parallel to one another. If a fault occurs in the active half of the system, then the system automatically switches over to use the part of the system that is still functioning properly. Consequently, all the functions in the entire system, such as the fire control function, remain fully available without any restrictions.

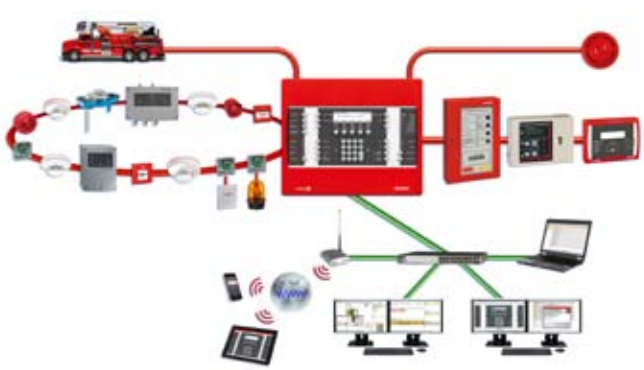
As with the Integral IP fire alarm system, the full redundancy can also be set up in the LIST system: with separate controller units, self-communicating and an individual uninterruptable power supply, there are redundant transmission paths available.

Quality "Made in Europe"

Schrack Seconet fire alarm systems are developed in Austria and manufactured in Germany and have been used for decades all over the world. LISTEC GmbH also develops and manufactures in Germany. All important stages of production are carried out and monitored with Swiss micro technology having guaranteed the high durability and reliability of the sensors since 1990.

Fire safety for tunnels from a single supplier

The strengths of Schrack Seconet (LISTEC) are combined in an overall fire detection system for particularly safe and high-quality early fire detection in tunnels. Maintenance and repair costs are low. The results are reliable and cost-optimized products and services for tunnel operators all over the world.



Line-type heat detector LIST

Keeping fire risks superior under control

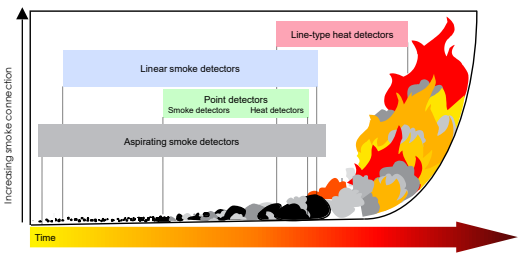
FIRE ALARM

SCHRACK SECONET

Line-type heat detector LIST

SCHRACK SECONET

Sensitivity overview of fire alarm detectors



2

Line-type heat detector LIST

SCHRACK SECONET

Typical applications

- Road tunnels
- Railway and metro tunnels
- Long cable and supply tunnels
- Long conveyor belts

3

Line-type heat detector LIST

SCHRACK SECONET

Overview of the Listec product range

Tunnel system

- Sensor cable - SEC 20
- Evaluation unit LISTcontroller
- Remote display RDU 316

4

Line-type heat detector LIST

SCHRACK SECONET

Sensor cable SEC 20

- Individually addressable temperature measurement points
- Sensor spacing: free selectable
- Sensor spacing: 2, 4, 5, 8 or 10 m
- Measures temperature from -40° to +120°C, for short periods +200°C
- Temperature measurement resolution: 0.1°C
- Measurement points marking by cable imprint
- Monitored distances of up to **3,200 metres**
- Maximum **350 sensors**
- VdS certified in accordance with EN 54-22 class A1N, A2N (G213072)

5

Line-type heat detector LIST

SCHRACK SECONET

Sensor cable SEC 20

- Simple installation and assembly
- All round aluminium shield against external electrical influences
- Robust construction for industrial systems ⇨ can be used in extreme conditions
- Sensor cable route can be branched and fits to any requirement

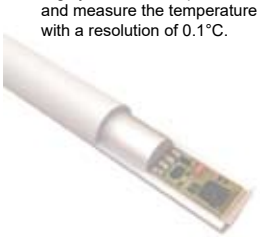
6

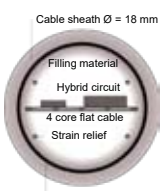
Line-type heat detector LIST

SCHRACK SECONET

Sensor cable SEC 20

Highly sensitive temperature sensors are integrated within a sealed cable and measure the temperature along a distance of up to several kilometres with a resolution of 0.1°C.





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Line-type heat detector LIST

SCHRACK SECONET

Evaluation unit LISTcontroller

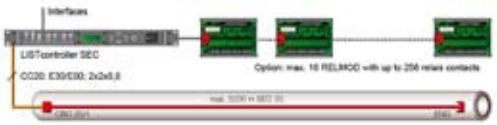
- Central cable terminal processor with one central processor
- Maximum 2,800 m sensor cable with 350 sensors or
- Maximum 3,200 m sensor cable with 320 sensors
- Measuring cycle: every 10 seconds
- Up to 254 alarm sections programmable
- Clear text display
- Navigation and function buttons
- USB interface

8

Line-type heat detector LIST

Evaluation unit LISTcontroller SEC

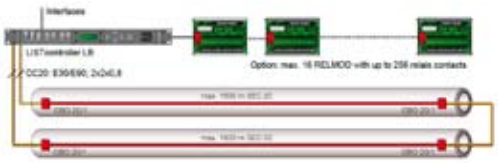
- Stub installation



Line-type heat detector LIST

Evaluation unit LISTcontroller LB

- Loop-back installation



Line-type heat detector LIST

Evaluation unit LISTcontroller LB

- Redundant RDT installation

Line-type heat detector LIST

Evaluation unit LISTcontroller – Technical specification

	LISTcontroller
Operating voltage	24 VDC
Power consumption	max. 5 W
Outputs	1 relay for each alarm, pre-alarm and frost alarm
Ambient temperature	-5° to +70°C
Case material	Aluminium
Case dimensions	43.6 x 482.6 x 315.5 mm (HxWxD)
Weight	2.6 kg

Line-type heat detector LIST

Remote display RDU 316

- Detached clear text display
- 3 lines, each with 16 characters
- Coloured background lighting
- Menu keys
- Collective LED display
- For connection to up to one LISTcontroller via RS-485 interfaces
- Allows “individual detector recognition”, i.e. displays the relevant sensor in the event of an alarm

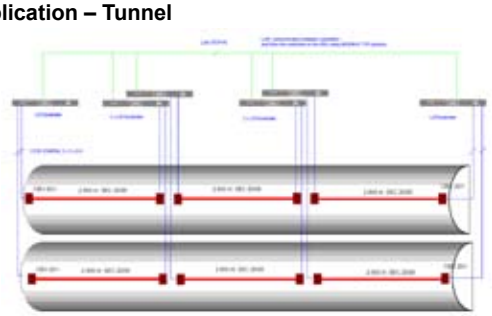
Line-type heat detector LIST

Connection boxes CBO 20/0, CBO 20/1 and CBO 20/3

	CBO 20/0	CBO 20/1	CBO 20/3
Connecting cord	2 sensor cables SEC 20	1 CC cable 1 sensor cable SEC 20	3 sensor cables SEC 20
Ambient temperature	-30° to+90°C		
Dimensions	244 x 164 x 100 mm (HxWxD)		
Protection class	IP 66		
Case material	Fibreglass reinforced polycarbonate		

Line-type heat detector LIST

Application – Tunnel



Line-type heat detector LIST

Advantages of LIST

- Precise detection
- High response speed
- Stable operation
- Long lifetime
- Robust
- Optimised maintenance costs
- Could be repaired in case of damage
- The system is full functional even when a sensor is defective
- Branches are possible

⇨ suitable for all applications in tunnels!

TAI NEWS

PUNE METRO CONTINUES WITH ONE LESS TBM

Work on the new metro line

Maharashtra Metro Rail Corporation (Maha Metro) has decided to complete the Pune Metro project using three TBMS instead of the previously planned four TBMS. This after observing two TBMS already working on the project performing much better than assumed.

"The two TBMs that were operational from Agriculture College were to cover 8m per day, but they were able to cover 12m per day. The work schedule is further ahead than planned and it has reached Shivajinagar now," stated Brijesh Dixit, Chairman of Maha Metro. He added the 2 TBMs are supposed to keep the same pace of work.

The distance to be covered by two TBMs starting from Agriculture College would now become larger. This means TBMs starting from Swargate towards Agriculture College should cover less distance, explained Dixit. Therefore Maha Metro is now able to only use one TBM which will construct both twin tubes starting from Swargate.

Source : Metro Rail News India, Feb 29, 2020

INDIA ALLOCATES $14 BILLION TO TUNNELLING

Mr. Nitin Gadkari, India's Minister for Road Transport & Highways, has said that works worth over $14 billion will be undertaken for tunnels at strategic locations in the next five years to ensure all-weather connectivity. The minister added that competent contractors, whether small or big, need to be given opportunity and there is a need to relax parameters including technical and financial qualifications for bidders. Mr. Gadkari announced this after holding a round-table discussion with senior officials, stakeholders and industry players. He said the need of the hour is to reduce cost and improve quality and added that departments need to work in a comprehensive manner.

Source : The Economic Times, Feb 13, 2020

KOLKATA EAST-WEST METRO TO START SERVICES

Work on the tunnel underneath the river

The first underwater metro project in India and the second metro project in Kolkata will begin truncated services from February 13. Services along the East West Metro line will start along the 5km route from Salt Lake Sector V to the Salt Lake Stadium. The 16.6 km East West Metro line will connect the Howrah Maidan on one side of the Hooghly river with Rajarahat Newtown on the other side. The elevated corridor is 5.8 km long while the underground stretch measures 10.8 km. Tunnelling about 13m underneath the Hooghly river bed was successfully completed in 2017. The foundation stone of the project was laid in February 2009 and the project was scheduled to be completed by October 2014. The delay of more than five years can be attributed to an alignment change. Work was stalled from 2012 to 2015 due to realignment. Questions about the fate of the project had surfaced again in September 2019 when several houses collapsed during underground tunnelling in the Bowbazar area. The TBM hit an aquifer resulting in subsidence. The last 2.45km from Bowbazar to Sealdah still needs to be bored.

Source : The Hindu, Feb 04, 2020

WORK STARTS ON BANGALORE METRO PHASE II

Tunnelling work for Phase II of Bangalore Metro is expected to start in February. The vertical shaft at Shivajinagar is being made ready to accommodate two TBMs. The two machines will start their journey from Shivajinagar and move towards Vellara Junction, boring 2.76km of twin tunnels. The TBMs are expected to excavate 5m per day,

Site preparations at Shivajinagar

depending on the geological conditions. Two additional TBMS will be deployed to bore a 2.88km stretch from Shivajinagar station to Tannery Road station. These machines are expected to arrive later this year. During Phase I, the TBMs faced hard rock due to which the project was delayed by many years. Project manager BMRCL had taken more than four years to complete this entire underground tunnel network of 8.8km.

Source : The Hindu, Jan 26, 2020

NORWAY OPENS RECORD LENGTH AND DEPTH SUBSEA LINK

Ryfylke subsea highway opens to traffic

Norway opened the longest, and one of the deepest, subsea road tunnels in the world in December 2019. At 14.4km long and up to 290m below sea level at its deepest, the twin-tube Ryfylke Tunnel is the main structure in the 19.5km-long Ryfast road project, developed by the Norwegian national roads authority Statens Vegvesen to pass under the Hidlefjord to Solbakk on the far shore. The drill+blast excavation of the win tubes began in 2012 and the first hole-through was achieved in late 2017. Since then efforts switched to fit-out of the mechanical and electrical systems and lighting. The fjord crossing features some cCavern enlargements with extra lighting to help give visual breaks to drivers on the long subsea journeys. Construction of the full Ryfast project began in late 2012 and main excavation started in 2013,with 2019 as the target opening year. The second part of the link is the 5.5km long, twin-tube Hundvåg Tunnel, which runs at shallower depths and links in to the heart centre of Stavanger. The Hundvåg portion connects with the 3.7km long Eiganes Tunnel on the E39 Coastal Highway. Both are due to open to traffic in February 2020. As the tunnels join together, both Hundvåg and Eiganes are opening at the same time. From Stavanger, the E39 travels north to the site of the next major subsea road project, the planned 26.7km twin-tube Rogfast link under the Boknafjord which is larger in its scale than the Ryfast project. Pocurement of the three main lots of the project stalled after high tender prices led the first large lot contracts to be cancelled. Statens Vegvesen is considering its options to proceed.

Source : Statens Vegvesen News Release, Jan 15, 2020

DUBAI METRO TO USE DRONES FOR INSPECTION

Al Tayer visiting the metro depot

Mattar Mohammed Al Tayer, Director of the Board of Executive Directors of Roads and Transport Authority (RTA), revealed RTA's intention to start a trial-run of drones in inspecting Dubai Metro tunnels extending 14km on both the Red and Green Lines. "This initiative uses custom-made drones fitted with multiple cameras enabling pictorial monitoring of tunnels via high-definition lenses. It also enables the inspection of tunnel walls through infrared radiation," said Al Tayer. "The technology generates instant reports easing the assessment of the condition of assets, monitoring the maintenance works and detecting faults. The process is linked with RTA's maintenance department and provides a paperless digital inspection platform that curbs potential human errors."

Source : Government of Dubai news release, Jan 14, 2020

PHASE II OF CHENNAI METRO BEGINS IN JUNE

Chennai Metro

Work on phase II of Chennai Metro is set to begin on a massive scale in June. Tunnel boring will be carried out on two stretches — a 9km stretch from Venugopal Nagar in Madhavaram to Kellys and another 12km stretch from Kellys to Taramani Road Junction. According to Chennai Metro Rail (CMRL) officials, they have called for tenders and the contractors will be known in about two months. Two TBMs will bore the twin tunnels simultaneously. Usually they would complete 6m to 10m a day but this may vary, depending on soil conditions. “We have checked soil samples every 25m so that we know how to handle the machines and operate accordingly without causing any damage,” an official said. In a few weeks, CMRL will call for tenders to build stations along the 21km stretch. It decided not to hand over the tunnelling and station construction work to the same contractor in phase II. “Depending on one firm for the whole construction could delay the entire project,” the official said.

Source : The Hindu, Jan 07, 2020

MUMBAI METRO LINE 3 PROGRESSES

First breakthrough in March 2019

The Mumbai Metro Rail Corporation (MMRC), which is currently executing the fully-underground Metro line 3, has completed 74% of the tunnelling work for the corridor. As of December 26, MMRC has completed 40km of the total 54.5km twin-tunnel line. MMRC has also completed 24 out of 32 planned tunnel breakthroughs. All tunnelling works should be completed by September 2020.

While the first phase was initially set to be operational from December 2020, MMRC is looking at making the line partly operational by June 2021. MMRC will also be inviting bids for the operation and maintenance of the corridor in February.

Source : Hindustan Times, Dec 28, 2019

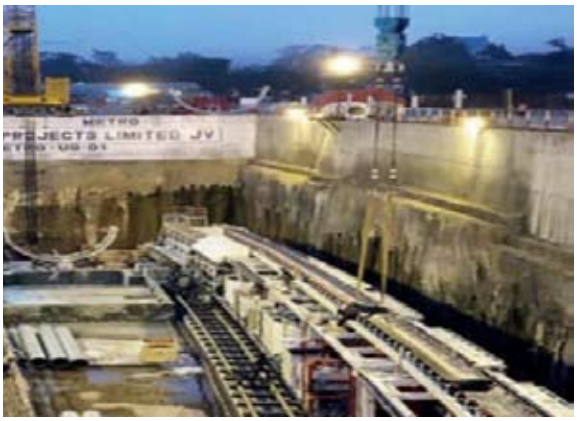
ANOTHER BREAKTHROUGH AT CHINA-LAOS RAILWAY

Luang Prabang

The construction of tunnels along a new railway connecting China with Laos has reached a new milestone. On December 27 a breakthrough was achieved for the longest bored tunnel of the line, in the mountains of northern Laos. Seven months ahead of schedule, PowerChina Sinohydro finished excavating the 9,384m-long Ban Sen No. 2 tunnel, south to the ancient Lao capital of Luang Prabang. Construction started in March 2017. The tunnel crosses the provincial border of Luang Prabang and Vientiane in tropical mountains with steep terrain, complex geology and abundant groundwater. This lead to difficult construction and transportation conditions. So far about 50 out of the 76 tunnels, with a total length of 198km, have been excavated along the line. The 414km Vientiane–Boten railway is part of the China one Belt and one Road Initiative and the Laos strategy to convert from a landlocked country to a land-linked hub. The project started in December 2016 and is scheduled to open in December 2021.

Source : Xinhua, Dec 27, 2019

PUNE METRO STEADILY PROGRESSING



Parts of the second TBM at the site

Officials are planning to start the construction of a second tube for a 6km twin tunnel stretch of Pune metro. A TBM will start excavating the tunnel in January. Parts have arrived at the construction site at Agriculture College, from where it will bore its way towards Swargate. Another TBM started boring the other tube last week. Project owner MahaMetro said eventually four TBMs would be involved in the tunnelling works. A second launch shaft will be constructed at the Swargate site. Earlier this year manufacturer Terratec was selected by main contractor Gulermak-Tata Projects JV to deliver the 6.61m diameter EPBMs for the metro tunnels. MahaMetro officials stressed that the work was progressing according to schedule. Construction of underground metro stations has also started. The 6km twin tunnels are expected to be operational by December 2020.

Source : Times of India, Dec 18, 2019

BANGLADESH WANTS MORE WATER CROSSINGS

The Karnaphuli river

The government of Bangladesh is weighing plans to construct more underwater tunnels instead of bridges to cross the country's rivers in order to save the environment. "We do not want to construct bridges on big rivers like the Padma and Jamuna," stated Planning Minister MA Mannan on December 12. "Bridges damage the environment as they cause sediment to settle and fill the rivers," he said. Bangladesh is currently constructing a tunnel under the Karnaphuli river in Chattogram at an estimated cost of around $1 billion. Prime Minister Sheikh Hasina inaugurated the boring work for the 3.4km tunnel in February. The government is planning another tunnel to the north of the Jamuna river to link Kurigram with Jamalpur, according to Mannan.

Source : BD News 24, Dec 12, 2019

HIMALAYAN ROAD LINK AWARDED TO APCO INFRA

Kargil

Apco Infratech has won the main contract for the Z-morh road tunnel in Jammu & Kashmir, India. It is a first step in restarting an under-construction project which was put up for rebidding following uncertainty over its completion date. Upon completion the 6.5km-long Z-morh tunnel will provide all-weather connectivity between Srinagar and Kargil. The project, initially awarded to the now-troubled Infrastructure Leasing & Financial Services (IL&FS), had been in stasis for about a year. Amid the uncertainty regarding its progress, state-run National Highways & Infrastructure Development Corporation (NHIDCL) had terminated IL&FS' contract and invited fresh bids for it in June this year. Apco emerged as the lowest bidder for financial bids that opened on August 29.

The tunnel will be constructed on a build-operate-transfer model. Work on the project had been stuck since July 2018. IL&FS had completed about 30% of the work. The remaining work on the tunnel is expected to be completed in 3.5 years.

Source : Construction Week India, Sep 03, 2019

FIRST HYPERLOOP MIGHT BE BUILT IN INDIA

The XP-1 pod in Nevada

The world's first Hyperloop is likely to be built in India and construction will start at the end of next year, Virgin Hyperloop One CEO Jay Walder told Yahoo Finance.

"One of the big projects that we have been working on right now is in India. It is to build a route from Mumbai to Pune," Walder explained. "That is about a 120km route. The first phase of that project will be 12km long. I think there will be shovels in the ground by the end of 2020." The Hyperloop system is a futuristic mode of transportation that would put passengers in pods that would magnetically levitate and travel through vacuum tubes across states, or even countries. The mode of transportation would move with "1080km per hour," Walder said. Aside from the first actual track that is going to be built in India next year, the company has already built a test track in Nevada, near Las Vegas. The pod that was used in that test, XP-1, was on display in New York this recently.

Source : Yahoo Finance, Sep 30, 2019

HIMALAYAN 8.8 KM ROAD LINK TO OPEN IN MAY

Work site at the tunnel portal

The 8.8 km-long Rohtang tunnel being built on the Leh-Manali highway in India may become operational within the next six months, an official spokesperson said. Indian Defence Minister Rajnath Singh informed Himachal Pradesh Chief Minister Jai Ram Thakur during a meeting on November 22 that efforts would be made to make the tunnel operational by May next year, the spokesperson added.One of the longest road tunnels in the country, it is being built at an elevation of 3.1 km under the Rohtang Pass in the eastern Pir Panjal range of the Himalayas. When operational it will reduce the distance between Manali and tribal district Lahual-Spiti's administrative centre Keylong by around 45 km.

Source : NDTV, Nov 22, 2019

BIG PLANS FOR WATER DIVERSION IN MAHARASHTRA

Wainganga River in Bhandara, Maharashtra

In order to provide relief to drought-hit districts in the Indian state of Maharashtra, Chief Minister Mr. Devendra Fadnavis on August 15 said the state government has planned four river-linking projects and a 480km-long tunnel to divert water. He said that the state cabinet had sanctioned the proposal to raise loans for four projects that would inter-link rivers across the state and divert water to the Marathwada region. He also said the state has planned a tunnel that would lift excessive water from Wainganga dam and channel it to five districts. "Although no policy decision has been taken yet, the state government has a specific plan for the tunnel for water lifting. This is the best solution to solve the water crisis in these districts," said an official from the water resources department.

Source : Hindustan Times, Aug 17, 2019

ITA-AITES WTC 2020 15 - 21 May 2020 Kuala Lumpur, Malaysia
World Tunnel Congress 2020

IEM The Institution of Engineers, Malaysia

ITA AITES

Geneva, Kuala-Lumpur

February 26th, 2020

Dear All

The health and safety of our colleagues is of the utmost importance to the ITA and IEM (the Malaysian Tunnelling Technical Group – ITA Member Nation in Malysia).

The WTC 2020 Organizing Committee, in cooperation with ITA, Sponsors, Exhibitors, lecturers, reviewers, ITA Member Nations, ITA Working Groups and Committees have been working hard during the last 3 years to bring every-body a successful World Tunnel Congress.

After considering various possibilities, the development of the Coronavirus (Covid-19) in Asia-Pacific and also in other parts of the world leads us to shift the date of the WTC and associated events from15-21 May, 2020 to 11-17 September, 2020.

Indeed, it is not possible at the moment to predict the evolution of this virus and many people could have been forced not to be able to participate. On the other hand, for the time being the World Health Organization recommends limiting mass gatherings.

We know that, for some of you, this change of date may not be ideal; however, it seemed to us the best solution to be able to organize this major event for the entire tunnelling industry.

The congress venue has been secured for this change to September 2020 and the WTC 2020 Organizing Committee is working with the hotels and all associated venues to make all the necessary changes accordingly. We will communicate with all to make these changes as smooth as possible.

If you need specific documents, notably to change your flights and hotel booking, we will do our best to provide you with the requested documents.

We all stay at your disposal

With our best regards

Dr Teik Aun Ooi
WTC 2020 Organizing Committee Chair

Olivier Vion
ITA Executive Director

World Tunnel Congress 2020 - c/o IEM Training Centre Sdn Bhd - 21, Jalan Selangor, P.O. Box 224 (Jalan Sultan) - 46720 Petaling Jaya, Selangor, Malaysia -
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OBITUARY

Dr. Harvey Weston Parker, Former ITA President

DECEMBER 23, 1936 – MAY 5, 2020

Dr. Harvey Weston Parker, of Bellevue, WA, died peacefully at his home on May 5th, 2020. He battled Ampullary Cancer for almost two years with his family always by his side.

Harvey was born in Panama City, Panama on December 23rd, 1936. He had the privilege of being raised by four mothers, which included his mother, his grandmother and his two aunts - Norma Harrington and Muriel Neill, while living in the Panama Canal Zone, Colorado, Alabama, and Buenos Aires, Argentina. He attended the Alabama Polytechnic Institute (now called Auburn University) where he graduated with a Bachelor of Civil Engineering (BSCE) in 1957. He received his M.S. in Civil Engineering from Harvard in 1967 and then his Ph.D. in Civil Engineering with a Minor in Geology from the University of Illinois Urbana-Champaign in 1976.

He made significant civil engineering contributions and publications for over 55 years. From his time as Chief Engineer of a US Navy Destroyer, to his pioneering work on steel fibre shotcrete in 1972, and to his innovative approaches to civil engineering projects including the Mount Baker Ridge Tunnel in Seattle, Washington (the largest soil tunnel in the world) and metro systems across the United States including Los Angeles, Washington D.C., New York, Boston and Chicago.

He also gave back to the industry through leadership and service. He was Chairman of several organizations including the US National Committee on Tunnelling Technology, The Underground Technology Research Council and the Underground Shotcrete Subcommittee for the American Concrete Institute Committee. He served as president of the International Tunnelling Association & Underground Space Association (ITA) from 2004-2007. In 2019, he was awarded the Lifetime Achievement Award from the ITA.

One of Harvey's greatest loves was traveling with his wife Karen. They travelled throughout the Americas, Europe, Africa and Asia and made lifelong friends everywhere they stayed. He also enjoyed spending quality time with his extended family and grandchildren.

He was preceded in death by his father Harvey Parker II and his mother Barbara O"Shaughnessy. He is survived by his loving wife of 57 years, Karen of Bellevue, WA, son Neil (Diane) Parker of Tucson, AZ, and daughter Erika (Chris) Price of Edmonds, WA. He is also survived by his four grandchildren, Ryan, Stephen, Sydney and Andrew.

In lieu of flowers, the family requests that donations be made in his name to the Pancreatic Cancer Action Network (pancan.org), Seattle Cancer Care Alliance (www.seattlecca.org), the University of Illinois, Grainger College of Engineering (www.grainger.illinois.edu/giving) or to Evergreen Health Hospice Services (www.evergreenhealthfoundation.org, please specify "to support the Hospice program".

Due to the Covid-19 pandemic, Harvey's memorial service will be held at a future date. The family will be in touch with friends and family as information is available.

FORTHCOMING EVENTS

S No	Description	Date	Country
1.	NATWeek: A Virtual Experience … Right Where You Are!	June 8-12,	
2.	WTC 2020	September 11-17	Kuala Lumpur, Malaysia
3.	BTS Conference and Exhibition …	October 6-7, 2020	London, UK
4.	Cutting Edge 2020 …	November 9-10, 2020	Dallas TX, USA
5.	Australian Tunnelling Conference,	29 November to 2 December,	Melbourne Australia
6.	REAL CORP 2020	15-18 September 2020,	Aachen
7.	4th Intenational Conference on Transporation Geothechnics		Chicago Illinois USA
8.	16th International Conference of the Intenational Association for Computer Method and QAdvances in Geomechanics	1 Jul-4 Jul 2020	Toronto Itly
9.	Tunnelling fundamentals practice and innovations	October 19-22, 2020	Golden, Colorado USA
10.	Trenchless ASIA	17-18 Nov, 2020	Kuala Lumpur Malaysia
11.	ASEAN RAIL 2020	July 23-24, 2020	Minh City, Vietnam
12.	Tunnel Safety & Ventilation Conference	December 1-3, 2020	Messecongress Graz, AUSTRIA
13.	Under ground construction	June-29-1 July, 2020	Cambridge UK
14.	Samoter	October 21 – 25,2020	Verona, Italy.
15.	TAC Confernece	Oct 19-21, 2021	Toronto, Canada
16.	69 th Geo mechanics colloquium	Oct 7-9 2020	Canada
17.	No Dig North	Oct 19-21, 2020	Canada
18.	EUROCK 2020	July 15-19, 2020	Norway
19.	Asia Pacific Rail Conference	Nov 2020	Thailand, Bankok
20.	7th European Geosynthetics Conference	Sep 6-9,2020	Poland, Warsaw
21.	TAC 2020 Conference	Oct 19-21, 2021	Toronto,Canada
22.	Buma CHINA 2020	Nov 24-27, 2020	Shanghai China
23.	ACUUS 2020	February 2-4, 2021	Helsinki, Finland
24.	AFTES 2021	September 6 – 8, 2021	Paris, France
25.	WTC 2021	May 16-19, 2021	Copenhagen
26.	InnoTrans	April 27-30-2021	Berlin, Germany
27.	underground construction technology (UCT)	26-28 January -21	Nashville TN
28.	Middle East Rail	10-11 March, 2021	Dubai, UAE
29.	No Dig Live ,	Tuesday 23rd March – Thursday 25th March 2021	East of England Arena & Events Centre
30.	RETC 2021	Jun 13-16, 2021	Las Vegas USA
31.	RO-KA-TECH 2021	21- 23 April The bi-yearly	Kassel Exhibition Center
32.	Railways 2021	August 30 - September 2	Mallorca, Spain

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Delhi Metro Rail Corporation Limited

Shri Rajeev Verma
Project Manager – 3A
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Shri Daljeet Singh
Director (Works)
Delhi Metro Rail Corporation Limited

Shri Umesh Billore
Manager/Civil (Underground Works)
Delhi Metro Rail Corporation Limited

Shri Shivam Singh
Manager/Civil
Delhi Metro Rail Corporation Limited

Shri Saurabh Sharma
Dy. General Manager/Civil
Delhi Metro Rail Corporation Limited

Shri Ashwani Kumar
Chief Project Manager-3
Delhi Metro Rail Corporation Limited

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Manager (Civil)
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Manager/Civil
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6th EDITION OF THE ITA TUNNELLING AWARDS 2020

After the success of the 5 first editions of the ITA Tunnelling and Underground Space awards, in Switzerland, Singapore, France, China and USA, the ITA Awards 2020 will take place at the end of the year. This year, the Awards conference will be held as a virtual Conference. It will be a new challenging experience allowing everyone from all over the globe to attend and discover the most interesting projects. The winners will be announced live during the Australian Tunnelling Conference.

By the time being, it is important to continue celebrating success in our industry and this can be done through the ITA Awards. For this edition, some changes are proposed in the categories with notably a category “Overcoming the challenge” for projects which overcame either foreseen or unforeseen extreme challenges during excavation and construction and another one named “Oddities of the Underground” celebrating the creativity, ingenuity, and imagination of the human spirit in its endeavors underground.

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