ज्ञान ज्योति से मार्गदर्शन

IRICEN Journal *of* Civil Engineering

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Indian Railways Institute of Civil Engineering, Pune

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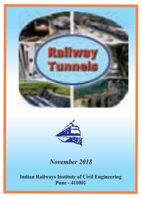

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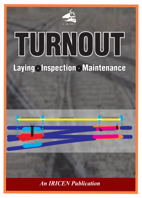

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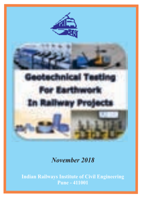

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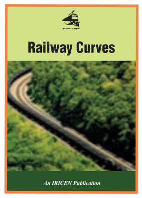

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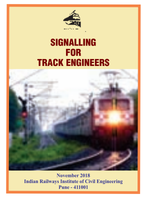

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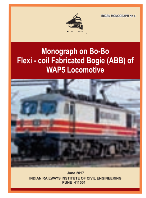

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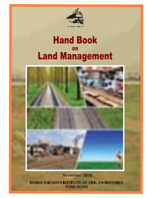

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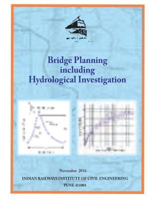

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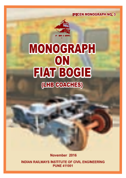

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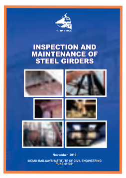

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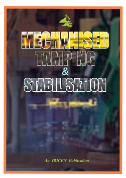

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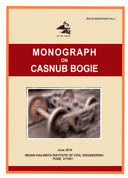

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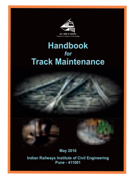

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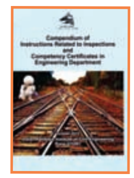

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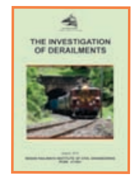

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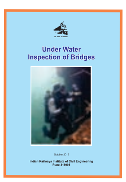

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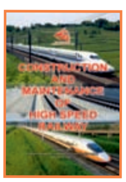

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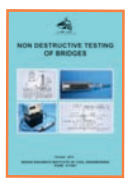

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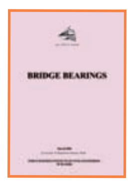

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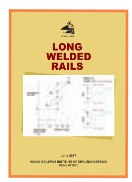

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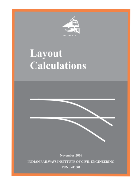

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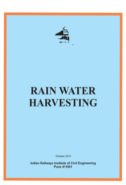

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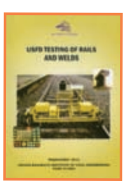

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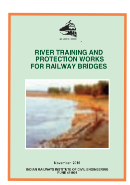

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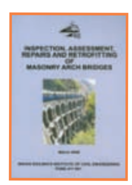

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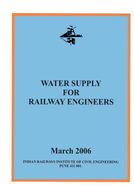

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From Director's Desk

Dear Readers,

In this edition of IRICEN Journal, a technical paper related to dismantling of existing Road over Bridge is included which is a detailed narration of challenges encountered while demolishing such a massive structures and solutions adopted by the Engineers.

Another paper is about rectification of Up-heaved portion of ballastless track in which author have discussed various possible options and the most appropriate solutions.

Next technical paper talks about the detailed mathematical background for the design of ground improvement system.

Another paper tells about the usefulness of google mapping with respect to railways affecting tasks and last but not least paper talks about the Time Dependent Stresses in Reinforced Concrete Circular columns from the point of view of Indian Railways Bridge Codes.

I sincerely hope that the readers would find these papers and other articles contained in this Journal informative and useful. I also welcome Engineers to come forward with their valuable suggestions and contributions of technical papers, news items, articles etc., for inclusion in the forthcoming issue of this journal.

Pune | (Ajay Goyal)
October 2019 | Director

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Suggestion for improvement of **IRICEN JOURNAL OF CIVIL ENGINEERING** are welcome from the readers. Suggestions may be sent to mail@iricen.gov.in

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Guidelines to contributors

Articles on the Railway Civil Engineering are welcome from the authors. The authors who are willing to contribute articles in the IRICEN Journal of Civil Engineering are requested to please go through the following guidelines :

1. The paper may be a review of conventional technology, possibilities of improvement in the technology or any other item which may be of interest to the readers. The paper should be reasonably detailed so that it could help the reader to understand the topic. The paper may contain analysis, design, construction, maintenance of railway civil engineering assets. The paper should be concise.
2. The journal is likely to be printed in a paper of size 215 mm X 280 mm. While sending the articles the author should write in 2 columns. Sketches, tables and figures should be accommodated in a 2 column set up only.
3. Author should send the original printout of photograph along with the digital copy of the photograph.
4. Soft copy as well as hard copy of article must be invariably sent to the editors of concerned subject.
5. Only selected articles will be included in the IRICEN Journal of Civil Engineering.

Railway News

Free WiFi 'RailWire' at 6,485 Railway Stations in Next 100 Days under Digital India Drive

If all goes to plans and promises, Internet will soon be like water at railway stations across the country. The Indian Railways has set itself a target — providing free WiFi at 6,485 railway stations in the next 100 days.

While they have already completed installing WiFi services at 1,603 stations, authorities claim that the balance 4,882 stations will be completed within this stipulated time frame. Previously, Google had launched a similar project that provides free WiFi across 400 railway stations in the country.

Currently, on the Western and Central Railways, 260 railway stations, which includes a majority of Mumbai suburban stations, already have this facility. In the coming days, 245 stations on CR and 281 stations on WR will also have WiFi facilities.

Railways Reach:

- **4,882** – Stations where work is still pending
- **1,603** – Installations already complete
- **400 stations** – in India that use Google's free WiFi services
- **50 Mbps** – promised speed of the WiFi services
- **30 minutes** – of free access after first login
- **260** – Stations on WR and CR that already have the free WiFi
- **245** – CR stations in line to get the service
- **281** – WR stations set aside to get the facilities
- **32L** – Mumbai commuters who used free WiFi in Dec 2018
- **725 TB** – Data used by Mumbai commuters in Dec 2018

A railway official said, "The IR's subsidiary RailTel, in a tie-up with Tata Trust, is providing the WiFi under the name RailWire. It focuses on broadband and VPN services to provide free internet connectivity. Only the halt stations where passengers are far and few in between, will not have this facility."

As per the system, the user will be automatically directed to the RailWire homepage, where they will have to enter their mobile number to get the OTP, which would be sent thereafter. Once the OTP authentication is done, they can use the free WiFi network at a speed of 50 Mbps for up to 30 minutes per day.

To get an estimate of data usage in Mumbai, the Railways measured the consumption in December 2018 and found that 32 lakh people consumed close to 725 TB of data.

Interestingly, the railway officials accept that most of these people use the WiFi for free download of movies, serials, and even adult content. Many of them also reportedly sit along the railway station premises — either on the FOBs, ticket counters or platforms — to avail this facility.

Meanwhile, the railways have also proposed a Digital Railway Corridor that will have 10 Mhz spectrum in 700 Mhz band. This will be used to improve the safety of the running trains and enhance passengers' interface with railwaymen.

Ref: http://www.railnews.co.in

Bhaupur-Khurja Section of Dedicated Freight Corridor on Allahabad Division to Open in Nov 2019

Indian Railways network eyes infrastructural boost! In a big development for the dedicated freight corridor (DFC) project, the Bhaupur and Khurja section of the Eastern Dedicated Freight Corridor (EDFC) in Uttar Pradesh (UP) will open by the month of November this year, according to a recent PTI report. This will result in the reduction of pressure and congestion on the saturated trunk route of the North Central Railway (NCR) between Kanpur-New Delhi, stated the Dedicated Freight Corridor Corporation India Limited (DFCCIL).

The DFCCIL is the organisation responsible for implementing the DFC project under the Railway Ministry. With the shifting of freight trains from the Indian Railways main lines to the DFC route, the network will be decongested, resulting in faster movement of trains.

According to the report, the decision to open the stretch was taken in a meeting between the DFCCIL and the NCR zone recently. Both the parties studied a detailed review of every aspect for the targeted project completion. Besides the targets, coordination issues among various executing agencies were also examined and finalised.

According to a statement released by DFCCIL, the DFC corridor will not only help the NCR zone run passenger train services efficiently with enhanced punctuality but will also ensure the availability of more time for the railway track and other asset maintenance.

Hence, this will enhance the overall safety in all train operations across the network. Additionally, it will also help in the non-interlocking work of Tundla, which has been long overdue. Tundla is the only main rudimentary yard on the major trunk route from the cities of Kolkata to Delhi which is operated and handled by hand lever arrangement. The entire DFC network will be completed by the year 2021, according to DFCCIL.

Meanwhile, according to an earlier report, the first traction sub-station (TSS) of 63 MVA (mega volt ampere) was charged at Biruni near Tundla on the Khurja-Bhaupur route of the EDFC on May 25. Along with this, a 2 X 25 KV overhead equipment (OHE) from Biruni to Dariyapur with a length of 66 km had been charged. This development was an important milestone for DFC because it would pave the way for the movement of freight trains which are being pulled by the electric locomotives.

Ref: http://www.railnews.co.in

Railways Mulls Inviting Private Firms to Run Trains on Some Routes; IRCTC to Operate Two Trains on Trial Basis

Indian Railways is planning to get private firms onboard to run passenger trains on some low congestion and tourist routes. Bids for the same will be invited in the next 100 days. It may be noted that railways has also proposed to corporatise units manufacturing coaches and other rolling stock. The government is seriously exploring a plan to induct private operators to run passenger trains on low congestion and tourist routes and will invite bids in the next 100 days, a Railway Board document shows.

To begin with, the railways will experiment by offering two trains to its tourism and ticketing arm, IRCTC, to operate. The ticketing and on board services will be provided by IRCTC and the railways will receive a lump sum amount in return. These trains will be run on routes which have “low congestion and connect important tourist spots”, which will be identified by the national transporter. These will run on the routes like Golden Quadrilateral and connecting major cities. The custody of rakes will be transferred to IRCTC, which will pay annual lease charges to the railways financing arm, IRFC.

Subsequently, the railways will float the expression of interests “to identify operators willing to participate in the bidding process for rights to run passenger day/ overnight train sets connecting important cities,” a communication sent by Railway Board Chairman Vinod Kumar Yadav to all members and top officials of railways on Tuesday said.

The letter said that the railways will consult trade unions while finalising the contours of inviting private players to run passenger trains on select routes.

If the pilot project is successful, railways will float expression of interest (EoI) ‘to identify operators willing to participate in the bidding process for rights to passenger day/overnight train sets connecting important cities”. As per the national daily report, the national transporter will consult trade unions before finalising everything and inviting private players.

The railways will also launch a massive campaign urging people to give up the subsidy while booking or purchasing a rail ticket. The passengers will have the option to purchase ticket with or without subsidy.

The proposed campaign asking people to give up railway ticket subsidy will be on the lines of the Ujjwala campaign where people were urged to forgo the subsidy for LPG cylinders. According to the Railways, it recovers only 53% of the cost incurred from passenger transport business.

Among other plans, the railways also proposes to corporatise units manufacturing coaches and other rolling stock. There are seven production units across the country. The proposal says that the production units, including associate workshops, will be hived off into a government-owned new entity “Indian Railways Rolling Stock Company”. Each production unit with a Chief Executive Officer will function as an individual profit centre, report to the board or chairman-cum-managing director of the new entity.

The railways will immediately start consultations with unions and come up with a Cabinet note for approval for at least one production unit to begin with, and it might be the Modern Coach factory in Rae Bareli.

The report further mentioned that railways has also proposed to corporatise units manufacturing coaches. According to the proposal, the production units, including associate workshops, will be hived off into a new government-owned entity “Indian Railways Rolling Stock Company”.

India spent the past five years contemplating ways to set up a holding company for the Indian Railways and make all its public sector undertakings (PSUs) its subsidiaries. This requires several reforms proposed by the NDA regime to take shape, but since the initial step of merging the railways budget with the country’s general budget, the NDA has developed cold feet. At present, the ministry of railways is a separate entity, but most experts believe that India, like most countries, must have a single transport ministry at the Union level. This would bring flexibility and efficiency, commercially and operationally. This step could, however, upset the unions, who see this as a backdoor entry for private players and a threat to their jobs.

Now, a big infrastructure push for dedicated freight corridors (DFCs) is being made, which would allow segregation of slow-moving freight from mail and express trains. The rolling stock and locomotives are being replaced with more efficient and modern versions and administrative reforms-restructuring of the railway board, decentralisation of decision-making-has started taking shape. The new government is in a much better position to push this reform of turning the railways into a PSU, allowing an increase in private investments in this sector.

In the past two decades, reports by panels led by eminent economists and technocrats, like Bibek Debroy, Rakesh Mohan and Sam Pitroda, have pointed to the modernisation and commercialisation of the railways.

In 2013, the challenges faced by China in this sector were identical to India's, but changes have been slow compared to our neighbour's progress. In India, the railways is also bound by the social obligations of providing a cheap transportation for the poor.

In 2017, the cabinet approved the formation of a Railway Development Authority to level the playing field for private investors to step in. If the new government wants to encourage private players to own locomotives and run commercially viable passenger and freight travel, the railways is required to make its PSUs its subsidiaries. This is a big task for the new government, but doable.

The 100-day plan

Why

- Movement of freight (24 km/ hr) and passenger (50 km/ hr) trains among the slowest compared to peer countries
- Indian freight fares are among the costliest globally, while the passenger fare is the cheapest
- The railways is unable to increase revenues to match expenditure. The operating ratio has remained above 90 per cent for many years.

How

- Appointment of a Railway Development Authority will allow price regulation of both freight and passenger trains
- Setting up of Indian Railways Rolling Stock Company Ltd.
- Opening up to private investment, to allow ownership and operations of freight terminals
- More 'Vande Mataram' coaches to replace premium trains
- More DFCs by 2019 end and more bullet trains are expected

Ref: http://www.railnews.co.in

New Signaling System to Bring Indian Railways Network to World-Standards

Indian Railways bets on big signalling upgrade! Aiming to provide safer and faster train travel to its passengers, Indian Railways has taken the big step towards signalling upgradation. The national transporter has signed an MoU with RailTel Enterprises Limited (REL) for the work of modernization of signalling system. The REL is an entirely owned subsidiary firm of RailTel Corporation of India Limited. The modernization of signalling system will include Automatic Train Protection System along with Long Term Evolution (LTE) based Mobile Train Radio Communication System (MTRC) together with the provision of Electronic Interlocking (indoor) wherever required. The project will be implemented on four sections across the Indian Railways' network.

According to a press release issued by RailTel, the project of Modern Train control system will be implemented on 165 rail kilometres on South Central Railway's Renigunta–Yerraguntla section, 145 rail kilometres on East Coast Railway's Vizianagaram–Palasa section, 155 rail kilometres on North Central Railway's Jhansi– Bina section and 175 rail kilometres on Central Railway's Nagpur–Badnera section. All these sections are some of the busiest routes on Indian Railways' network with heavy traffic.

The pilot project, which is expected to cost around Rs 1,609 crores, is likely to be completed in 2 years. According to RailTel, the Modern Train Control system based on LTE based MTRC is Indian Railways' one of the most ambitious modernization projects, which envisages upgradation of the signalling system at par with the world standards. After the successful implementation of this project, the national transporter plans to implement the same on other crucial high-density routes of its network.

The release stated that the Automatic Train Protection System and LTE based MRTC will improve safety. Also, it will help to reduce congestion in these rail networks, increase line capacity as well as improve punctuality. Additionally, it will also help to increase the average speed of trains.

According to RailTel, the Modern Train Control System will be as per Open Standard, Interoperable with Multi-vendor support. The MTRC system will be deployed using LTE backbone for communication between drivers, train running staff, control office and guards. Moreover, the system will be used for working of Automatic Train Protection System as well. RailTel also stated that this system will initially be provided on as many as 500 locomotives.

Talking about the project, Kashinath from the Railway Board, said “We need to implement modern train control system to keep pace with modern signalling and communication systems being used all around the world, to ensure safety of the passengers and increase line capacity to match with the challenges of running more trains on existing track.”

Briefing about the project, Puneet Chawla, CMD/ RailTel said “We are already on the job and team of REL has already taken inputs, feedback from industry representatives with respect to technology, interfaces, compatibility and interoperability of various sub-systems.”

The MoU was signed by Pradeep M Sikdar, executive director, signal development, railway board, Ministry of railways and AK Sablania, director and chief executive officer of REL in the presence of N Kashinath from railway board and Puneet Chawla, CMD of RailTel..

Ref: http://www.railnews.co.in

Railways Giving Final Touches to Mini ICPs, Rail-Based Ro-Ro Service to Decongest Road Traffic in Bangladesh Border

Two weeks after the Prime Minister Narendra Modi assumed office for the second term, the Centre is giving final touches to a host of new initiatives – mini ICPs (Integrated check post), rail-based Ro-Ro service to decongest road traffic in Bangladesh border and bus service between Myanmar and Manipur – to improve connectivity with Eastern neighbours.

Trade and connectivity

It was anticipated that the new government will focus its attention on trade and connectivity with BBIN (Bangladesh, Bhutan, India and Nepal) and BIMSTEC nations (Bangladesh, India, Myanmar, Sri Lanka, Thailand, Nepal and Bhutan). There is now an enhanced focus on coordination between different Departments to ensure efficient implementation of projects.

While NITI Aayog is leading on the planning part, the recently formed Logistics Department under the Commerce Ministry is ensuring that different arms of the government do not work in silos.

This is expected to improve the efficiency of various trade facilitation projects, as in Jogbani where the Land Ports Authority under home ministry is building ICP on Indian side, while the Ministry of External Affairs is funding a mirror ICP on Nepalese side and the Railways is connecting the two countries.

Innovative solutions

A better coordination is bringing some innovative solutions. For example, the Railways proposed starting a service between Kolkata and the largest land border with Bangladesh at Petrapole-Benpole to allow loaded trucks to roll in and roll out (Ro-Ro).

The Central Warehousing Corporation (CWC) is preparing an online platform so that Indian trucks can book parking spaces at the ICP Petrapole in advance.

The aim is to reduce road congestion on the 70-km stretch from Kolkata to Petrapole and end the local extortion racket at the border town of Bongaon.

An in-principle decision has been taken to upgrade 10 Land Customs stations — which barely see movement of goods due to lack of facilities — on Bangladesh and Myanmar border into mini ICPs enabling both trade and passenger movements.

This is over and above the existing ICPs (Moreh in Maniur, Petrapole in Wet Bengal and Akhaura in Tripura) and upcoming ICPs (Sutarkhandi in Assam, Sabroom in Tripura and Dawki in Meghalaya).

The proposed list for upgrade includes border gates at Nampong (Arunachal Pradesh) and Zokhawthar (Mizoram) with Myanmar. The Zokhawthar border has potential to connect India sponsored Trilateral Highway (from India to Thailand) bypassing Manipur.

On Bangladesh border, Mahendraganj (Meghalaya), Sheila Bazar (Meghalaya) and Srimantapur and Raghnabazar in Tripura are proposed to be mini ICPs.

While the list will be finalised after a detailed study, the government is looking for a suitable location to open a trade point between Nagaland and Myanmar. Nagaland does not have any trade facility.

Meanwhile, officials of India and Myanmar met last week to start a bus service between Mandalay (Myanmar) and Imphal (Manipur). To ensure quick launch and avoid the long negotiations for protocol movement, the two governments are looking to start synchronised services.

“Better coordination between ministries will help the north-eastern States the most,” an official said. Though different ministries are mandated to allocate 10 per cent of the budget to North-east; funds often remain unutilised due to lack of coordination.

Ref: http://www.railnews.co.in

First Electric Loco Freight Train Chugs on Villupuram-Cuddalore Port Stretch

For the first time an electric locomotive hauled freight train has been operated in the Villupuram – Cuddalore Port Junction energised section more than two months after the stretch was authorised by the Commissioner of Railway Safety for introduction of passenger and goods traffic.

The freight train transporting rice to Thanjavur from Villupuram was hauled by an electric loco up to Cuddalore Port Junction from where it was hauled by a diesel locomotive since the remaining stretch from Cuddalore to Thanjavur via Chidambaram, Mayiladuthurai and Kumbakonam was yet to be electrified. The Villupuram – Cuddalore single line broad gauge stretch was the first portion to be electrified in the mainline section from Villupuram to Thanjavur via Cuddalore, Chidambaram, Mayiladuthurai and Kumbakonam.

The Rail Vikas Nigam Limited (RVNL), a Government of India enterprise under the Ministry of Railway has been entrusted with the task of executing the electrification project in the 228-km mainline section from Villupuram to Thanjavur which serves as an alternate route to Chennai from Tiruchi and to Tiruchi from Chennai.

The RVNL took up the electrification work in the Villupuram – Cuddalore Port Junction stretch completing it in March this year which was followed by mandatory inspection by the Commissioner of Railway Safety, Southern Circle, Bengaluru. The Commissioner of Railway Safety authorised the newly electrified stretch for introduction of passenger and freight trains on March 25 subject to series of stipulations laid down by him after inspecting the energised stretch.

Senior RVNL and Tiruchi Railway Divisional officials said the first electric locomotive hauled freight train with 42 wagons transporting rice was operated on Sunday from Villupuram to Cuddalore Port Junction. The electric loco was detached at Cuddalore Port Junction and a diesel loco was attached with the freight wagons to continue with its onward journey up to Thanjavur, said the officials.

An RVNL official said the foundation works and mast erection as part of the ongoing overhead electrification project in the remaining stretches from Cuddalore to Mayiladuthurai and beyond was apace. Simultaneously, the RVNL would also electrify the 38-km stretch from Mayiladuthurai to Tiruvarur via Peralam, Punthottam and Nannilam as part of this project being executed at a cost exceeding ₹300 crore. The RVNL's plan is to complete the stretch from Cuddalore to Mayiladuthurai in 2020 and the last portion from Mayiladuthurai to Thanjavur by 2021, said the official.

Ref: http://www.railnews.co.in

Railways Give Red Signal to Tejas Coach Production to Ramp up Train-19

India's first fastest train – the Tejas Express – which runs between Mumbai and Goa – is about to get terminated. The Indian Railways have decided to stop production of Tejas Express. The Railways plan to upgrade their technology with Distributed Power Rolling Stock (DPRS) which is apparently better than the current one. Sources said Tejas Express will be replaced by Train 19.

Sources said that this is being done so that they can replace it with much better technology used across the world. "We have asked all the production units to stop manufacturing of Tejas Express. We are migrating to DPRS," confirmed Rajesh Agrawal, Member (Rolling Stock), Railway Board.

The officials said that the technology used in Tejas Express will be replaced with DPRS which is being tried in Train 19. This is one step further to Train 18 – Vande Bharat Express – which will have sleeper coaches. "The DPRS will allow faster acceleration and deceleration. It will happen within 1-2 minutes. At present, it takes 12-15 minutes for the same. It will save time, fuel and power," Agrawal said.

There are only two Tejas Express trains in the country – the Mumbai-Goa and Chennai-Madurai. The production of the third one is on hold. Speed wise, Tejas Express, which uses LHB coaches, had touched 180-200 kmph during trial runs. This is the same as Train 18.

The tickets of Tejas Express is available during weekdays barring weekends. This semi high-speed train departs CSMT at 5 am and reaches Karmali at 1.30pm. On the Chennai-Madurai route too there have been complaints of clogged toilets, broken windows and LCD sets etc.

There have been cases in the past where the LCDs and entertainment units inside Tejas Express were damaged. The manufacture of Tejas Express coaches is 10-15% more than a normal LHB coach used for Rajdhani and Shatabdi Express.

The railways are also working on speeds of Rajdhani Express using DPRS, which will save journey time by two hours. Train 18 or Vande Bharat Express is a flagship train of Prime Minister Narendra Modi.

On Track

- There are only two Tejas Express trains in the country – the Mumbai-Goa and Chennai-Madurai.
- The technology used in Tejas Express will be replaced with DPRS
- The Rail ministry had said that nearly 130 Train 18/19s would be manufactured in the near future.

Ref: http://www.railnews.co.in

Kangra Railway Line may get Heritage Tag

The 160-km Pathankot-Jogindernagar narrow gauge rail corridor is on the tentative list of UNESCO world heritage site.

At present, three narrow gauge corridors of India — Kalka-Shimla, Siliguri-Darjeeling and Nilgiri Mountain Railway line — have been accorded the world heritage status by UNESCO.

The final decision will be taken after the inspection of the corridor. The Railways was hopeful of getting the world heritage status for Kangra. A team of UNESCO will soon inspect the corridor and necessary amendments will be made.

Recently, Kalka-Shimla Railway was included in the list. The Kangra valley lies in the sub-Himalayan region and covers a distance of 160 km from Pathankot to Jogindernagar. The railway line was planned in May 1926 and commissioned in 1929.

Ref: http://www.railnews.co.in

South Eastern Railway to Equip Functional Railway Sidings with Advanced PWBS

Chakradharpur Division of the South Eastern Railway (SER), which handles bulk iron ore dispatch from mining belts in Odisha and Jharkhand, has decided to equip all its rapid loading system (RLS) railway sidings with the advanced pre-weighment bin system (PWBS).

The fresh move envisages to substantially enhance availability and mobility of railway rakes cutting down rake detention time.

Till June 15, two each RLS railway sidings of Tata at Noamundi (Jharkhand) and Joda in Keonjhar (Odisha) have been equipped with PWBS.

The move came after Kolkata-based SER general manager PK Mishra and Chakradharpur Divisional Railway Manager (DRM) Chhatrasal Singh closely analysed the reasons for unnecessary detention of rakes during a visit to one of the RLS sidings recently.

Senior divisional commercial manager Manish Pathak said Chakradharpur Division is the first to introduce this system in iron ore sector of Indian Railways.

In the earlier practice of weighing on motion weigh bridge (MWB), a train rake used to move at a speed of 10 km on the MWB and if there was overloading, the rake had to be returned for adjustment.

This used to inevitably detain the loaded and other rakes in the queue for 90-120 minutes. But the PWBS works on load cells hydraulically with computer command system, he added.

Pathak said a loading siding without PWBS if handles 10 rakes in a day, then it inevitably lose above 13 hours in rake detention.

But with the PWBS, two more rakes could be adjusted. The new initiative has improved rake availability with turnaround time in loading-release-loading cycle.

In near future, remaining RLS loading sidings would be equipped with the PWBS. The Indian Railways also plans to integrate PWBS-RLS sidings with Freight Operation and Information System (FOIS) under its Mission Raftaar, he added.

SER sources said in next phase, all six functional railway sidings of KMC would be equipped with PWBS-RLS facilities to improve rake movement.

Ref: http://www.railnews.co.in

Major portion of land for Bullet Train to be acquired by December

A major chunk of land required for the 508-km-long Mumbai-Ahmedabad Bullet Train project to be acquired by the National High-Speed Rail Corp Ltd (NHSRCL) by the end of this year. The land will be taken over once the tenders for the project is finalized. The NHSRCL has acquired 39% of the required 1,380 hectares of land till date. This comes to 537 hectares: 471 of 940 hectares in Gujarat and 66 of 431 hectares in Maharashtra.

The railways had floated tenders for tunneling work, including testing and commissioning of the double line high speed railway using tunnel boring machine (TBM) and New Austrian Tunneling Method (NATM) between the underground station at the Bandra-Kurla Complex in Mumbai and Shilphata in Maharashtra. A 21-km tunnel will be dug between Boisar in Maharashtra and the Bandra-Kurla Complex, 7 km of which will be under the sea.

The design and construction of civil and building works, including testing and commissioning of 237 km of length of rail line corridor between Zaroli village on the Maharashtra-Gujarat border and Vadodara in Gujarat, have been floated. The tenders to construct the stations in Gujarat's Vapi, Bilimora, Surat and Bharuch had also been floated. The construction of the Sabarmati hub in Ahmedabad to be linked to the bullet train station has started.

Ref: Master Builder, June 19, pg. 22

Vijaywada Railway Station Gets 'Gold Rating' For Green Measures

Vijayawada railway station, one of the busiest railway junctions in the country, has received Gold Rating by the Indian Green Building Council.

Council Chairman Meka Vijaya Sai presented the gold rating shield to Divisional Railway Manager R Dhananjayulu on Monday.

The IGBC-CII, with the support of the Environment Directorate of Indian Railways, developed the Green Railway Stations Rating System to facilitate adoption of green concepts, thereby reduce the adverse environmental impact due to station operation and maintenance and also enhance the overall commuter experience, a railway release said.

The rating system helps to address national priorities like water conservation, handling of waste, energy efficiency, reduced used of fossil fuel, lesser dependence on usage of virgin materials and health and well being of occupants, it said.

The DRM said that Vijayawada Railway Station was ranked fourth in the Indian Railways for cleanliness among A-1 category stations.

The station boasts of 100 per cent LED lighting,five star rated fans, pumps and motors and solar water heating systems.

Ref: Builders friend, June 2019, Pg. 20

Challenge to Arbitral Award on Policy Ground only in Exceptional Circumstances, says SC

Challenge to an arbitral award on the ground of public policy of India can be made only in very exceptional circumstances when the "conscience of the court is shocked" by violation of principles of justice, the Supreme Court has said.

A bench comprising Justices R F Nariman and Vineet Saran said this in a judgement in which the court dealt with issue related to section 34 of the Arbitration and Conciliation Act, 1996.

The section deals with application for setting aside arbitral award and the grounds on which a court may do that.

The Act says that an award is in conflict with "public policy of India" if the making of the award was induced or affected by fraud or corruption or was in violation of two other provisions of the law.

"However, when it comes to the public policy of India argument based upon 'most basic notions of justice', it is clear that this ground can be attracted only in very exceptional circumstances when the conscience of the court is shocked by infraction of fundamental notions or principles of justice," the bench said in its 90-page judgement.

The court was dealing with a petition filed by a Korean company, whose bid for construction of a four-lane bypass on national highway-26 in Madhya Pradesh was accepted in 2005 by the National Highways Authority of India (NHAI).

The bench also dealt with a question as to whether the amendments made in section 34 of the Act are applicable to applications filed under the provision to set aside arbitral awards made after October 23, 2015.

The court held that section 34 of the Act, as amended, will apply only to applications filed under the provision that have been made to the court on or after October 23, 2015 "irrespective of the fact that the arbitration proceedings may have commenced prior to that date".

"Therefore, even in cases where, for avoidance of doubt, something is clarified by way of an amendment, such clarification cannot be retrospective if the earlier law has been changed substantively," the bench noted in its verdict.

Indian construction journal June 2019 (page 58)

Simla-Kalka Mail

Dismantling of Delisle ROB at Lower Parel in Mumbai

By
*U S S Yadav **
Vilas Wadekar #

Synopsis:

On 3rd July 2018, pedestrian footpath of Gokhale ROB on the south of Andheri station had collapsed. Soon after this incidence, based on instructions from Hon'ble MR, joint audit was carried out on all the ROBs in Mumbai suburban section by Western Railway, MCGM and IIT Bombay. This Bridge was found unsafe for public use and was closed to road traffic on 24th July 2018 based on the joint inspection held on 17th July 2018.

ROB details:

The Delisle Bridge was constructed by erstwhile B. B. & C. I. Railway in 1921. The bridge connecting Curry Road/Lalbag in the east to Lower Parel in west, spanned across 5 railway tracks on which suburban local trains were running with peak hour frequency of 3-3.5 minute on each track. The length of bridge was 59.3m (across tracks) and 57.2m (along track) with bridge being in skew having skew angle of 65 degree. The total width of carriage way including footpath on both sides was 24.2m, with each footpath housing scores of utilities below it. The bridge was in skew having skew angle as 64 degree and had three spans. The superstructure consisted of steel Fascia girders of 2.15m depth at both ends, Steel Cross beams on 12 steel columns in each of the two piers parallel to track, 84cross girders of 1m depth on top running normal to fascia girders and composite slab connected to cross girders with RS Joists of 0.3m depth embedded in between slabs. The original depth of composite slab was 400 mm on which later on subsequent layers of depth 300 to 350mm were added and thus making total thickness of slab as 700 to 750mm. The carriageway consisted of a 120mm thick bitumen layer, above 700-750 mm thick concrete layer.

The work of dismantling of the bridge and its subsequent rebuilding was assigned to construction department of Western Railway. Western Railway has accordingly chalked out concept plan for dismantling with due consultation of all experts in the field considering the peculiarity of the ROB. The tendering process was completed by 16th August.

Special features of the ROB:

1. It is 4 lanes ROB with cantilever footpaths on either side and is in skew with skew angle of 65°.
2. It has a 'T' junction on West end, meeting at the abutment.
3. The load distribution arrangement is very complex. There are three spans, (two abutments of stone masonry and two piers of 12 steel columns and steel cross beams across columns). Fascia girders on both sides are supported by the column cross beams and are connected to 45 cross girders which are placed normal to Fascia girders. RCC slab of 400 mm is provided with 300 mm embedment of cross girders and further have embedded I section beams at 1m spacing running across the cross girders through slab. There is further BB layer of 150 mm and CC pavement of 200 mm over the original deck slab. Thus the concrete slab thickness was around 750 mm. The topmost layer of bitumen/asphalt also existed.
4. The concrete deck slab is not supported by steel trough from bottom and as a result, it is to be dismantled with extreme care and precaution by a method which will not allow spalling / falling of concrete on tracks / OHE below.
5. The surface area of ROB is substantial due to its skew alignment (65° skew) in comparison to normal ROB.
6. Vertical clearance of the bridge bottom from Rail level is only 4.9m to 5m.

Challenges

Among the major challenges, first challenge was to provide alternate access to approximately 1 Lakh pedestrians who were using this ROB footpaths. This was overcome by allowing them to use the two Railway Foot over bridges of Lower Parel Station.

The second challenge was planning the dismantling work in phases so that train services, including local

** CAO/C/WR*
 # CE/C/HQ/WR

trains are not affected while executing the work and the fact that final girder de-launching block of minimum duration is required on final day. Considering that the concrete slab was unsupported from bottom, removal of concrete slab without allowing any material to come on running tracks below was a challenging task. It was also to be ensured that after every night block no material is left loose as trains had to run till next night block. Any mistake in ensuring this would have caused havoc. Further the bridge was in skew with skew angle of 64 degree and concrete slab was to be cut using diamond cutters from Hilti using wire sawing system with proper marking of slab panels within two cross girders. So dismantling scheme with proper sequencing of activities with meticulous planning and execution needed to be foolproof to ensure safety of train as well as workers doing dismantling.

The third challenge was the diversion of a large number of utilities pertaining to MCGM, BEST, MTNL, BSNL, Reliance, Vodafone, Idea etc. which were serving important and sensitive establishments such as Hospitals, Traffic signals, Police stations etc. This required convincing these departments about the urgency of dismantling the bridge and also getting them on board to carry out the same within the quickest possible time.

Demolition

The bridge was closed for vehicular traffic on 24.07.2018 with the support of traffic and city police and the vehicular traffic was diverted to the adjoining bridges. Pedestrians were diverted to two FOBs of Railways located at Lower Parel station. Tender was awarded on 16th August (within 48 hours of its opening) and thereafter preparatory works were carried out. Dismantling scheme was prepared by the contractor and approved by Railways and CRS sanction was also processed and obtained on 6th September 2018 after which the dismantling began.

First, the bitumen layer was removed and core cutting was done to assess thickness of concrete slab. Thereafter, wooden planks were placed on bottom flange of cross girders to safeguard the tracks as well as running trains from any type of material which may fall during or after removal of concrete layers. Poly sheets were placed on this wooden plank from above for additional safeguard.

Position of cross girders were marked by paint on top exposed slab to enable cutting of slab panels between cross girders without cutting the cross girders. After that, diamond cutters were used to cut the slabs including embedded steel RSJs of 1 feet depth in pieces of approx. 4square metre (2m x 2m size) using wire sawing system. The entire activity of slab cutting and its removal was undertaken in traffic and power blocks of affected lines during night shutdown periods as per the prescribed cutting sequence and supporting the panels through slings of crane.

After operation of few blocks it was seen that there are difficulties in cutting of the slab panels and it was seen that there are non-homogeneous layers of concrete. Original slab was of 400mm depth only and subsequently layers were added by road authorities during road surfacing making total depth of 700-750 mm. So here improvisation in the cutting scheme was needed and accordingly floor saw diamond cutters were used to cut and remove the top layers of road concrete which was subsequently added during non-block hours. The original slab was then cut by diamond cutters using wire sawing system during traffic and power blocks. It worked well. Cutting and removal of entire 303 nos concrete slab panels within cross girders of total about 1000 cum concrete with approx.2500 MT quantity was done in total 88 night blocks.

Thereafter, balance concrete embedded in cross girders was removed using concrete breakers before starting the work of removal of total 84cross girders of 1m depth and length varying from 22m to 5 m. Out of 84nos, 28 cross girders were supporting OHE. So Final block requirement of removing 6 nos. fascia girders of 2.15 m height and 28 cross girders supporting OHE was modified by providing 10 temporary OHE Supports reducing block hour to only 11 hours.

Before the mega block, 29 night blocks were taken to remove 75 out of 84 cross girders. 9 cross girders were kept to ensure stability of fascia girder having 2.15m depth. In all, total 130 Night blocks were taken prior to mega block to remove concrete slab and cross girders and other items of work.

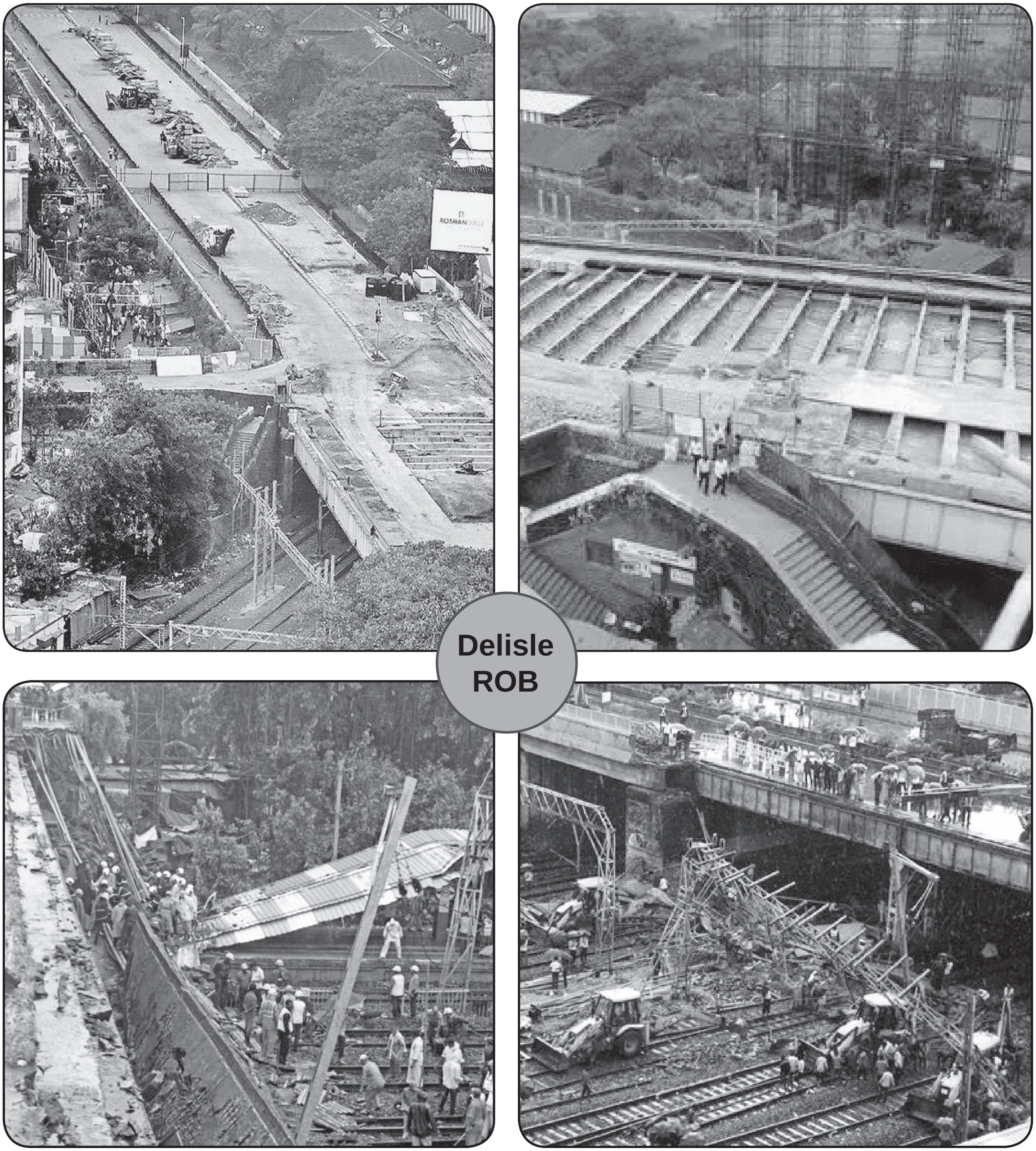
Megablock

In the Mega block, total 8 cranes were deployed for the entire dismantling work which involved 6 Nos Fascia girders, 9 Cross girders, 10 No. OHE Booms and 20 Nos OHE stools. Out of 8 cranes, 5 were of more than250T capacity, one of 110 MT Capacity and 2 small crane of 17T capacity. The cranes were suitably positioned to cater to the requirement of standby crane in case of failure of any of the crane. Approx. 40 No. skilled and 100 No. unskilled labour were deployed. 20 Gas cutters were used to cut the steel girders. Total 5 Nos. Tower Wagons, one each on each track were deployed for OHE work.

The work was carried out with clockwork precision, ensuring safety at all times. Heavy RPSF and Civil police bandobast was made on both ends as well as at the track level on either side. Doctors, both of the Railways as well as of the MCGM, along with Ambulances were stationed at both ends.

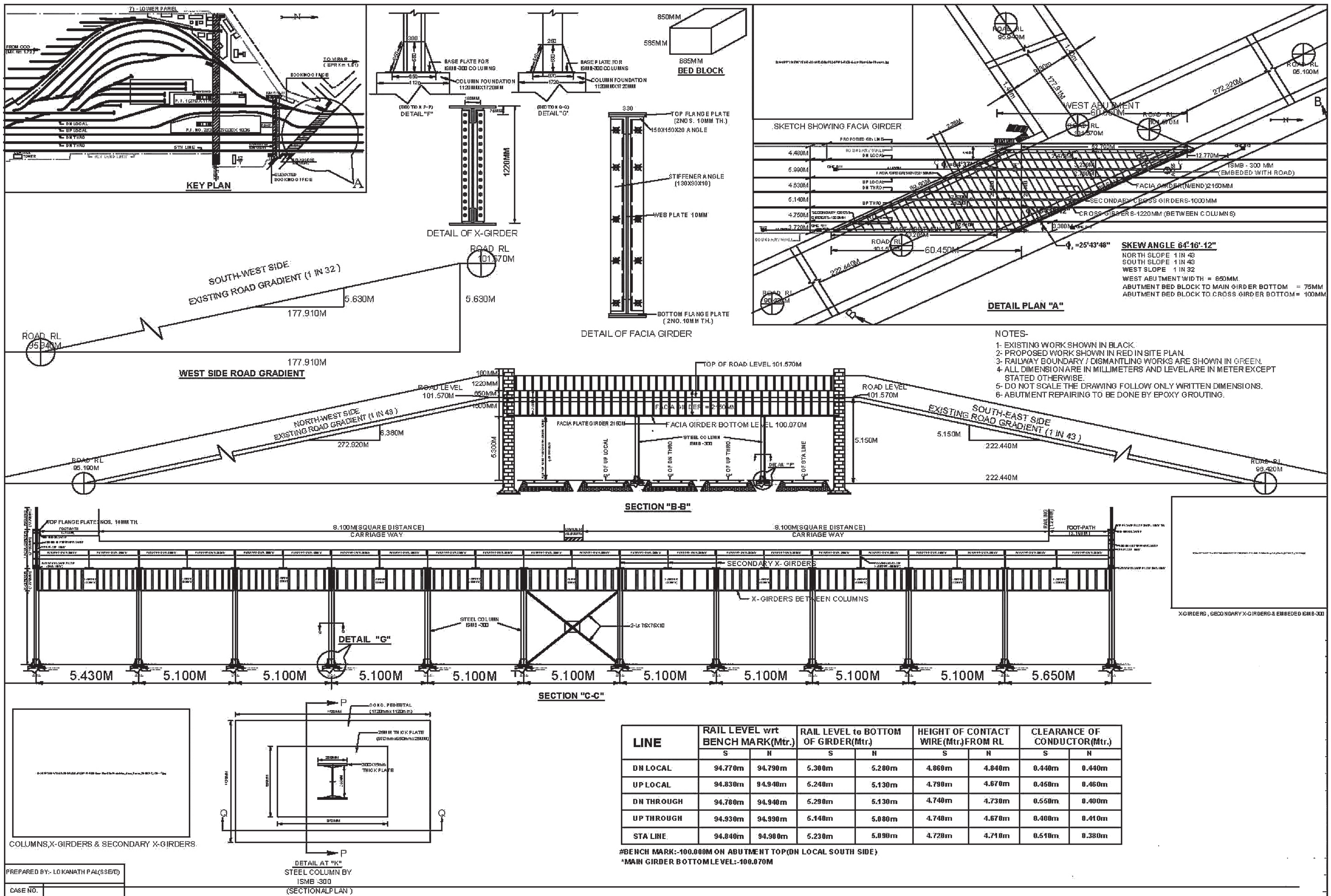
The entire work from the closure of the bridge to its dismantling was done in a short span of 5 months, which is a record in itself considering the complexities involved and the size of the bridge which was in skew. Such a mammoth job could not have been completed but for the hard work and dedication of all the concerned officers and staff and the determination on the part of Western Railway to get the work executed safely in time. This work has brought laurels to Western Railway and bears testimony to what an organization can achieve in the most difficult circumstances, given the will and the commitment of its employees, with each member performing as leaders in their respective roles.

- **Enclosed: Copies of drawings for the ROB and de-launching scheme**



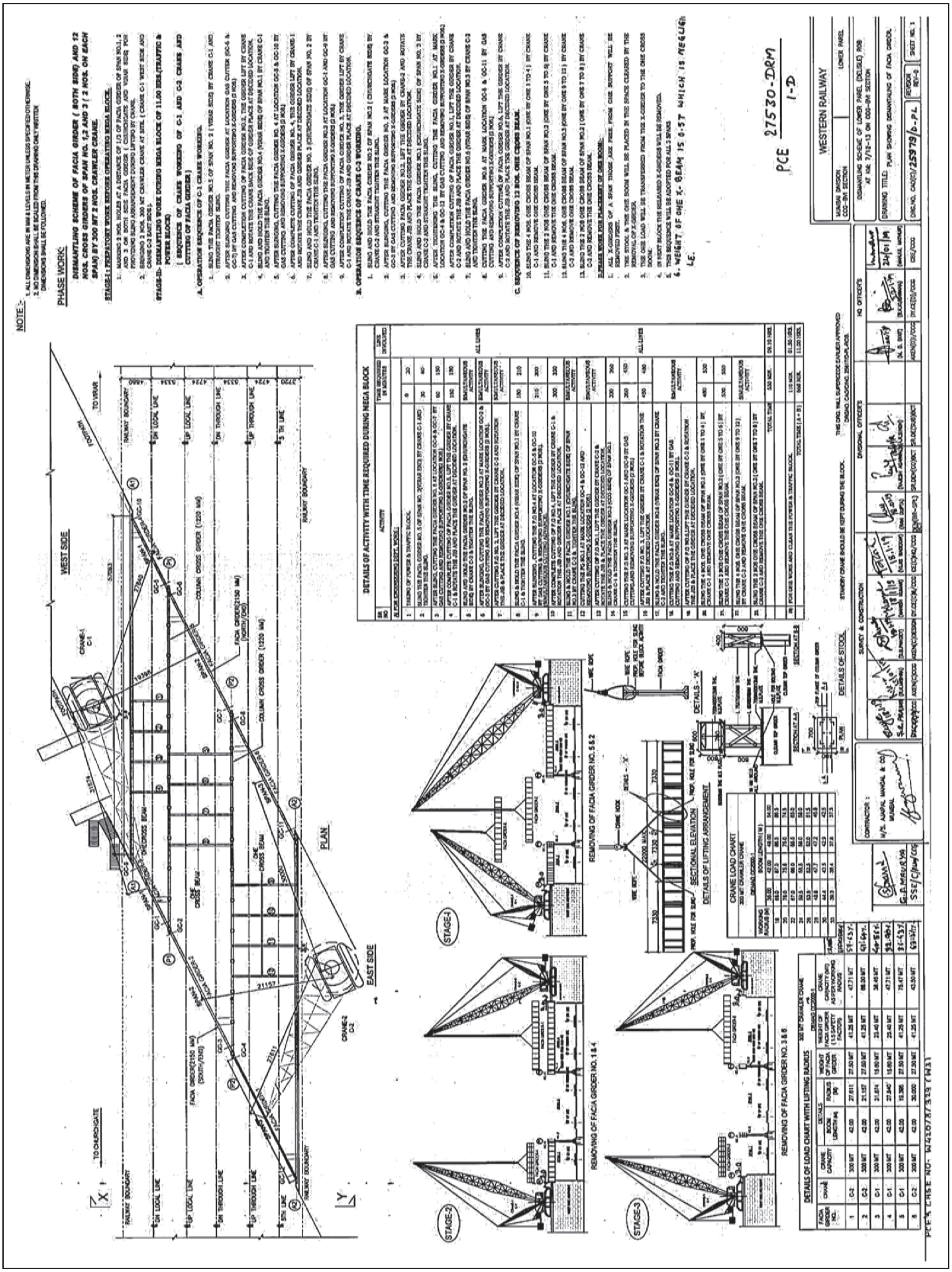
Delisle ROB

EXISTING DELISLE ROB DRAWING

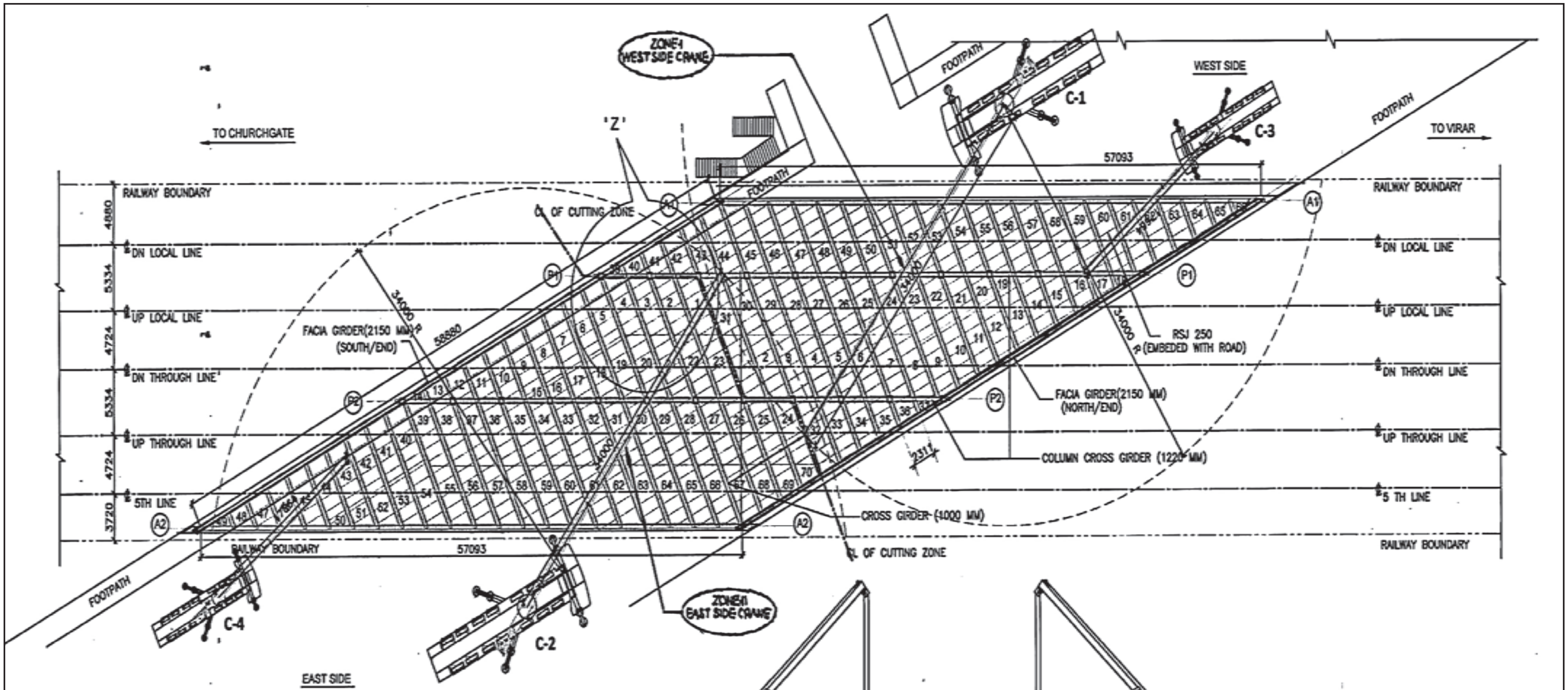


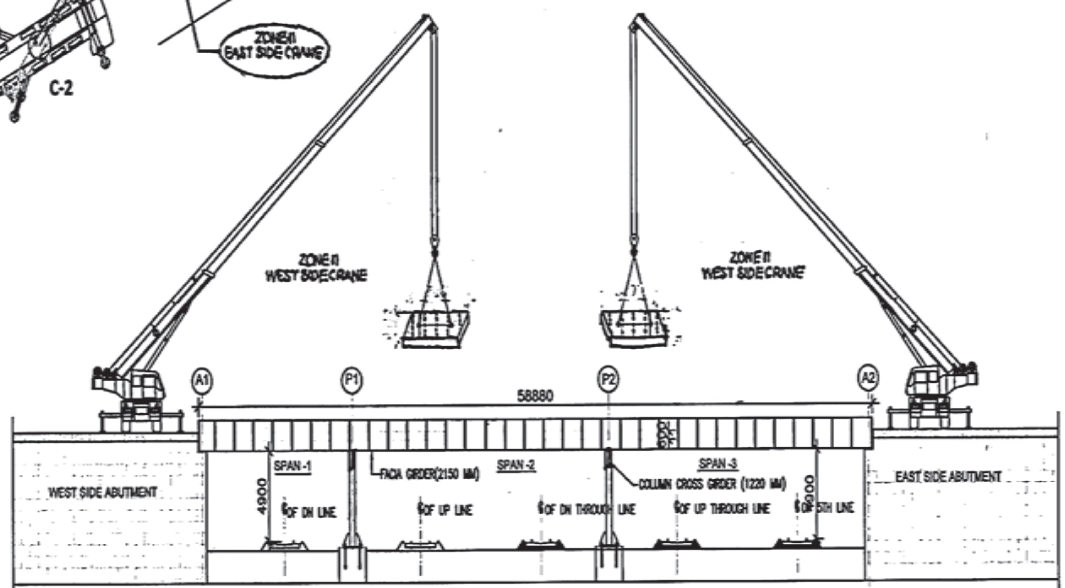
LINE	RAIL LEVEL wrt BENCH MARK(Mtr.)		RAIL LEVEL to BOTTOM OF GIRDER(Mtr.)		HEIGHT OF CONTACT WIRE(Mtr.)FROM RL		CLEARANCE OF CONDUCTOR(Mtr.)	
	S	N	S	N	S	N	S	N
DN LOCAL	94.770m	94.790m	5.300m	5.280m	4.860m	4.840m	0.440m	0.440m
UP LOCAL	94.830m	94.940m	5.240m	5.130m	4.790m	4.670m	0.450m	0.460m
DN THROUGH	94.780m	94.940m	5.290m	5.130m	4.740m	4.730m	0.550m	0.400m
UP THROUGH	94.930m	94.990m	5.140m	5.080m	4.740m	4.670m	0.400m	0.410m
STA LINE	94.840m	94.980m	5.230m	5.090m	4.720m	4.710m	0.510m	0.380m

#BENCH MARK:-100.000M ON ABUTMENT TOP(DN LOCAL SOUTH SIDE)
*MAIN GIRDER BOTTOM LEVEL:-100.070M



WESTERN RAILWAY
PHASE WORK
NOTE:-
PLAN
WEST SIDE
EAST SIDE
TO CHURCHGATE
TO VIRAR
STAGE-1
STAGE-2
STAGE-3
REMOVING OF FACIA GIRDER NO. 1 & 4
REMOVING OF FACIA GIRDER NO. 5 & 2
REMOVING OF FACIA GIRDER NO. 3 & 6
DETAILS OF ACTIVITY WITH TIME REQUIRED DURING MEGA BLOCK
DETAILS OF LOAD CHART WITH LIFTING RADIUS
CRANE LOAD CHART
DETAILS OF LIFTING ARRANGEMENT
SECTIONAL ELEVATION
DETAILS OF STOOL
CRANE-1 C-1
CRANE-2 C-2
PCE 27530-DRM 1-D





SECTIONAL ELEVATION A-A

CUTTING & REMOVING OF DECK SLAB

1. CUTTING OF DECK SLAB BY DIAMOND CUTTING USING WIRE SAWING SYSTEM AND REMOVAL BY CRANE IN BETWEEN P1–P2 ZONE–I & ZONE–II SIMULTANEOUSLY. AS SHOWN IN PLAN. POWER AND TRAFFIC BLOCK OF UP LOCAL AND DN THROUGH LINE.
2. CUTTING OF DECK SLAB BY DIAMOND CUTTING SAW AND REMOVAL BY CRANE IN BETWEEN P1–A1 ZONE–I. AS SHOWN IN PLAN. POWER AND TRAFFIC BLOCK OF DN LOCAL.
3. CUTTING OF DECK SLAB BY DIAMOND CUTTING SAW AND REMOVAL BY CRANE IN BETWEEN P2–A2 ZONE–I & ZONE–II SIMULTANEOUSLY. AS SHOWN IN PLAN. POWER AND TRAFFIC BLOCK OF UP THROUGH AND 5TH LINE.

DETAILS OF LOAD CHART WITH LIFTING RADIUS

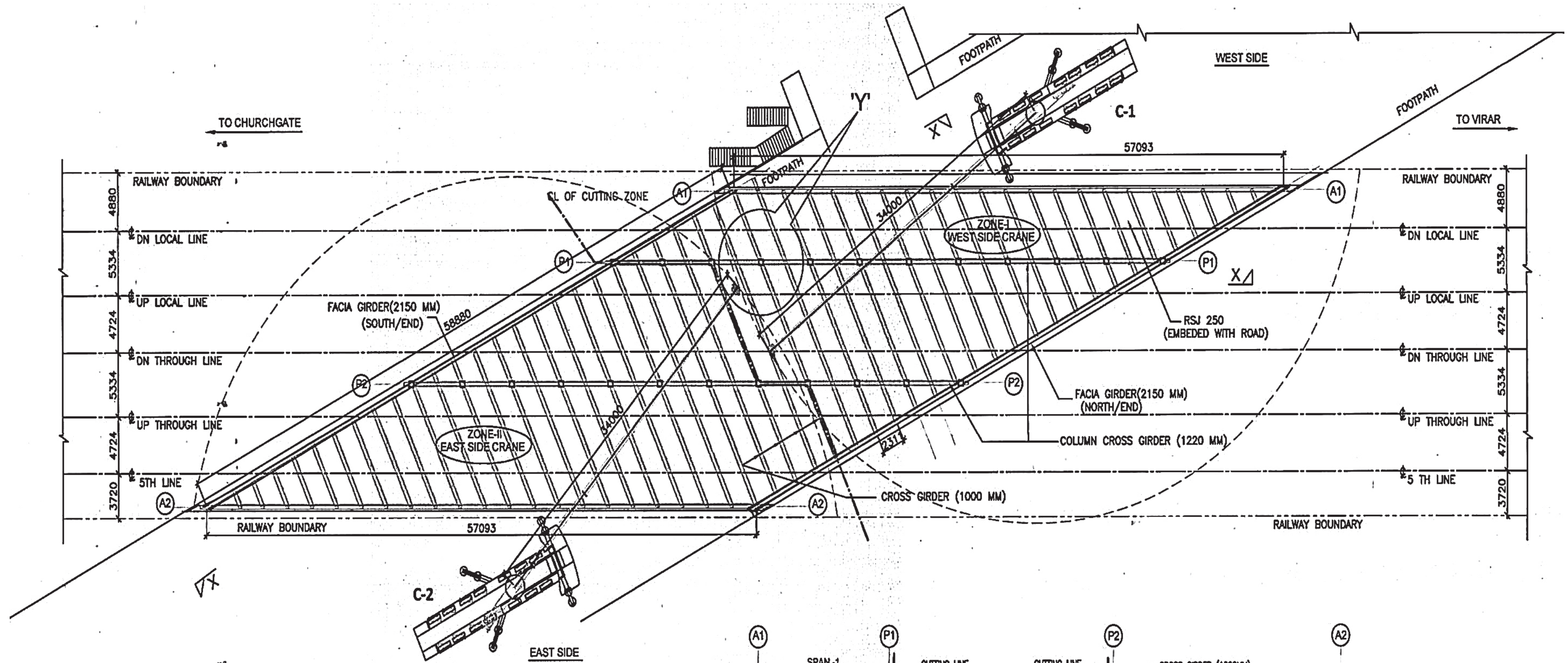
ITEM	CRANE	CRANE CAPACITY	DETAILS		CRANE CAPACITY AS PER CHART (MT)	ACTUAL LOAD APPROXIMATE	
			BOOM LENGTH (M)	MAX. RADIUS (M)		(TO BE LIFTED)	(INCLUDING 1.5 TIMES LOAD)
ZONE–1	C–1	250 MT	48.1	38.00	17.00 MT	10.00 MT	15.00 MT
ZONE–2	C–2	250 MT	48.1	38.00	17.00 MT	10.00 MT	15.00 MT
ZONE–1	C–3	50 MT	28.0	18.00	5.10 MT	3.40 MT	5.10 MT
ZONE–2	C–4	50 MT	28.0	18.00	5.10 MT	3.40 MT	5.10 MT

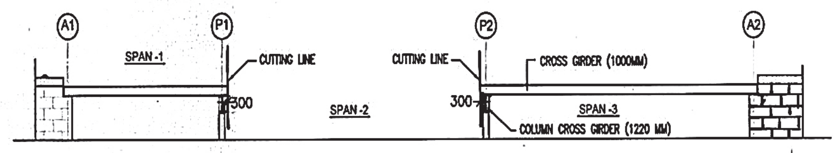
PCE 27020-DRM / 1-D -4

WESTERN RAILWAY

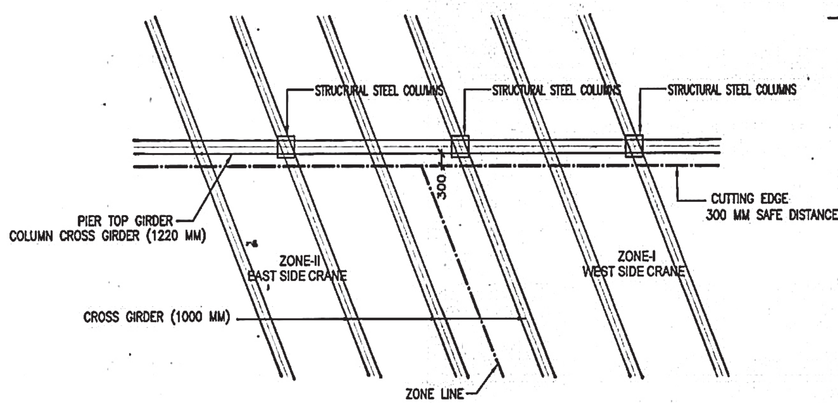
MUMBAI DIVISION
CCG–BVI SECTION

LOWER PAREL





SECTION X-X



DETAILS OF 'Y'

FOR REMOVAL OF CROSS GIRDER

DETAILS OF LOAD CHART WITH LIFTING RADIUS

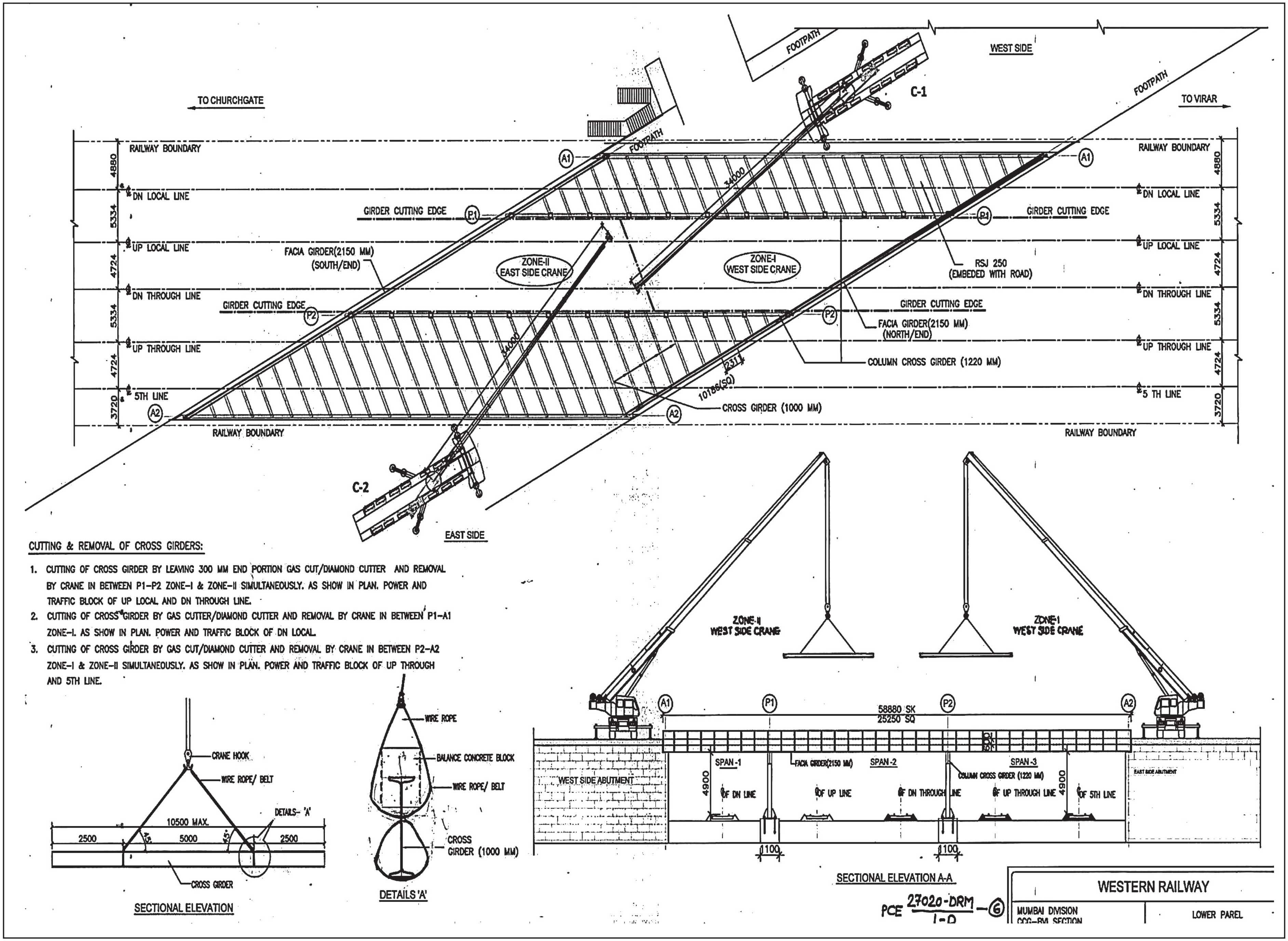
ITEM	CRANE	CRANE CAPACITY	DETAILS: BOOM LENGTH (M)	DETAILS: MAX. RADIUS (M)	CRANE CAPACITY AS PER CHART (MT)	ACTUAL LOAD APPROXIMATE: (TO BE LIFTED)	ACTUAL LOAD APPROXIMATE: (INCLUDING 1.5 TIMES LOAD)
ZONE-1	C-1	250 MT	48.1	36.00	17.00 MT	10.00 MT	15.00 MT
ZONE-2	C-2	250 MT	48.1	36.00	17.00 MT	10.00 MT	15.00 MT
ZONE-1	C-3	50 MT	28.0	18.00	5.10 MT	3.40 MT	5.10 MT
ZONE-2	C-4	50 MT	28.0	18.00	5.10 MT	3.40 MT	5.10 MT

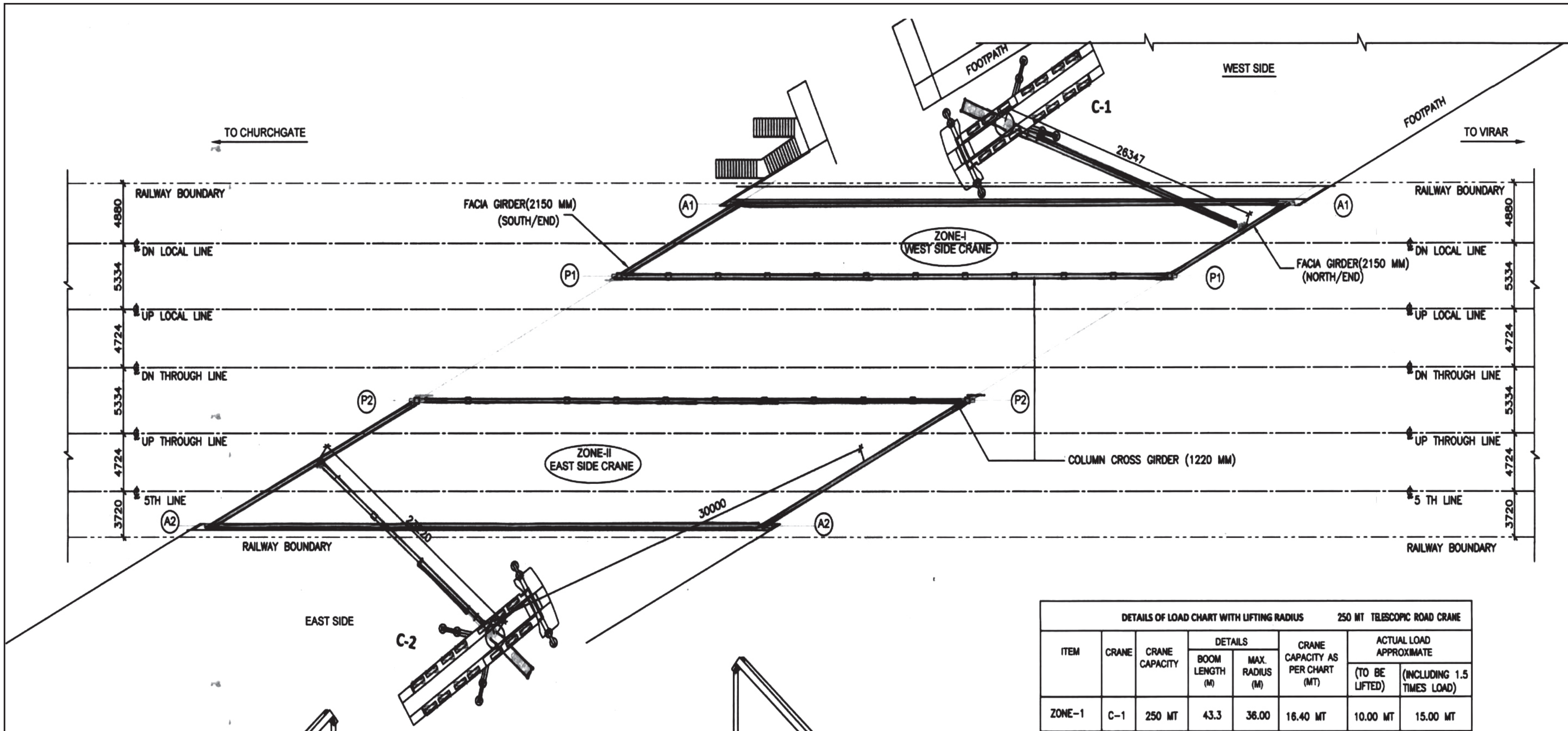
PCE 27020-DRM (5) / 1-D

WESTERN RAILWAY

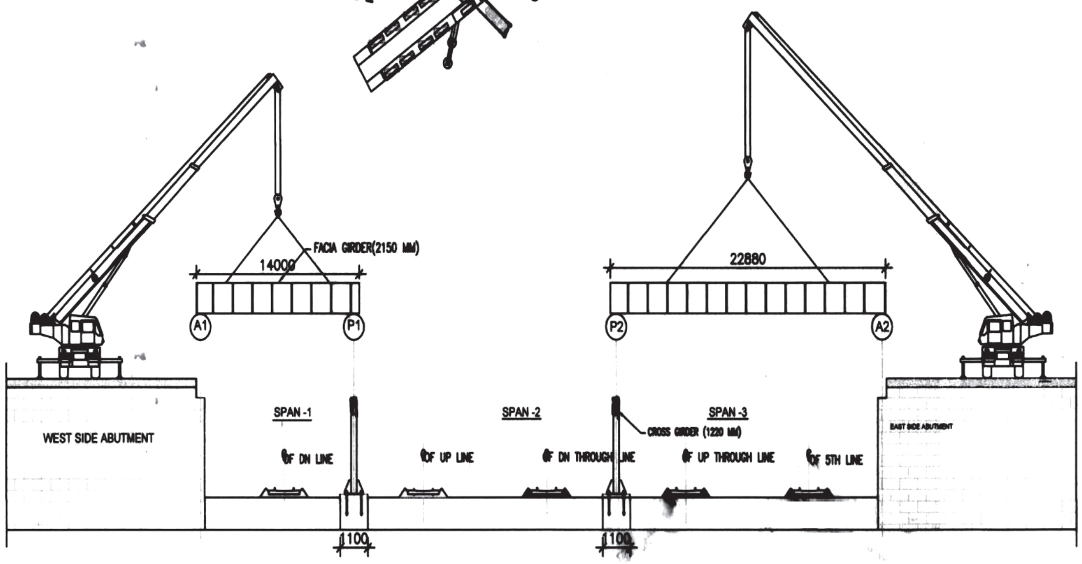
MUMBAI DIVISION
CCG-BVI SECTION

LOWER PAREL


WEST SIDE
TO CHURCHGATE
TO VIRAR
FOOTPATH
C-1
C-2
EAST SIDE
RAILWAY BOUNDARY
DN LOCAL LINE
UP LOCAL LINE
DN THROUGH LINE
UP THROUGH LINE
5TH LINE
5 TH LINE
4880
5334
4724
3720
GIRDER CUTTING EDGE
FACIA GIRDER(2150 MM) (SOUTH/END)
FACIA GIRDER(2150 MM) (NORTH/END)
ZONE-II EAST SIDE CRANE
ZONE-I WEST SIDE CRANE
RSJ 250 (EMBEDED WITH ROAD)
COLUMN CROSS GIRDER (1220 MM)
CROSS GIRDER (1000 MM)
34000
2311
10186(SQ)
A1
A2
P1
P2
CUTTING & REMOVAL OF CROSS GIRDERS:
1. CUTTING OF CROSS GIRDER BY LEAVING 300 MM END PORTION GAS CUT/DIAMOND CUTTER AND REMOVAL BY CRANE IN BETWEEN P1-P2 ZONE-I & ZONE-II SIMULTANEOUSLY. AS SHOW IN PLAN. POWER AND TRAFFIC BLOCK OF UP LOCAL AND DN THROUGH LINE.
2. CUTTING OF CROSS GIRDER BY GAS CUTTER/DIAMOND CUTTER AND REMOVAL BY CRANE IN BETWEEN P1-A1 ZONE-I. AS SHOW IN PLAN. POWER AND TRAFFIC BLOCK OF DN LOCAL.
3. CUTTING OF CROSS GIRDER BY GAS CUT/DIAMOND CUTTER AND REMOVAL BY CRANE IN BETWEEN P2-A2 ZONE-I & ZONE-II SIMULTANEOUSLY. AS SHOW IN PLAN. POWER AND TRAFFIC BLOCK OF UP THROUGH AND 5TH LINE.
CRANE HOOK
WIRE ROPE/ BELT
10500 MAX.
2500
5000
DETAILS- 'A'
CROSS GIRDER
SECTIONAL ELEVATION
WIRE ROPE
BALANCE CONCRETE BLOCK
CROSS GIRDER (1000 MM)
DETAILS 'A'
ZONE-II WEST SIDE CRANE
ZONE-I WEST SIDE CRANE
58880 SK
25250 SQ
WEST SIDE ABUTMENT
EAST SIDE ABUTMENT
SPAN-1
SPAN-2
SPAN-3
FACIA GIRDER(2150 MM)
COLUMN CROSS GIRDER (1220 MM)
4900
OF DN LINE
OF UP LINE
OF DN THROUGH LINE
OF UP THROUGH LINE
OF 5TH LINE
100
SECTIONAL ELEVATION A-A
PCE 27020-DRM / I-D -6
WESTERN RAILWAY
MUMBAI DIVISION
LOWER PAREL



DETAILS OF LOAD CHART WITH LIFTING RADIUS 250 MT TELESCOPIC ROAD CRANE							
ITEM	CRANE	CRANE CAPACITY	DETAILS		CRANE CAPACITY AS PER CHART (MT)	ACTUAL LOAD APPROXIMATE	
			BOOM LENGTH (M)	MAX. RADIUS (M)		(TO BE LIFTED)	(INCLUDING 1.5 TIMES LOAD)
ZONE-1	C-1	250 MT	43.3	36.00	16.40 MT	10.00 MT	15.00 MT
ZONE-2	C-2	250 MT	43.3	36.00	16.40 MT	10.00 MT	15.00 MT



SECTIONAL ELEVATION A-A

CUTTING & DISMANTLE OF FACIA GIRDERS FROM END SPANS:

1. CUTTING OF FACIA GIRDER BY GAS CUT/DIAMOND CUTTER AND REMOVAL BY CRANE IN BETWEEN P1-A1 ZONE-I. AS SHOW IN PLAN. POWER AND TRAFFIC BLOCK OF ALL LINES.
2. CUTTING OF FACIA GIRDER BY GAS CUT/DIAMOND CUTTER AND REMOVAL BY CRANE IN BETWEEN P2-A2 ZONE-II SIMULTANEOUSLY. AS SHOW IN PLAN. POWER AND TRAFFIC BLOCK OF ALL LINES.

PCE 27020-DRM-8

WESTERN RAILWAY

Rectification of Up-Heaved Portion of Ballast Less Track (RHEDA 2000-VOSSLOH300-IU System)

By
Sandeep Gupta*
Sumeet Khajuria**
Anirudh Bansal **

1.0 Preface

This paper gives an insight into the unique and innovative work of rectification done to solve the longstanding problem of an upheaval existing in ballastless track (RHEDA 2000- VOSSLOH 300-1U system) laid in Tunnel No. T-25 located between Udhampur and Katra section of USBRL National Project. This rectification work was unique in nature for the reason, being the first of its kind executed in a ballastless track under traffic operation on Indian Railways, which has yet nowhere been executed so far. The rectification work involved flattening of up-heaved portion of BLT by introduction of properly designed vertical curves having radii more than 1 in 4000 in the track structure without causing any destruction to the permanent concrete structure. For providing a vertical curve of 1 in 4000 radius in track structure, special fittings from VOSSLOH 300-1U Germany involving height adjustment steel and plastic plates , angled guide vanes etc were got designed ,imported and installed in order to introduce the designed vertical versine under each rail seat. This entailed precise survey of rail levels, data collation and plotting, design of best fitting vertical curves (Radii more than 1 in 4000) over the design track length and working out vertical versines for each rail seat and accordingly design, manufacture and install height and gauge adjustment plates underneath each rail seat for execution. The entire work was planned and successfully executed under three traffic block of 4 hours duration eacwh, taken for three (03) days in all. After installation, fittings were tightened for the design torque 250 N-m to achieve the designed service track parameters for gauge, cross level, twist, unevenness and longitudinal rail levels. Speed was relaxed in stages before finally a joint speed trial was done with open line to check the riding comfort over the rectified track length. No abnormality was noticed and speed was thus relaxed to sectional speed of 80 kmph on 05.04.2019.

2.0 Origin of Problem

2.1 Tunnel T25 is a 2.5 km Length tunnel, constructed in the year 2005. Ballastless Track (RHEDA 2000- VOSSLOH 300-1U system) designed by RAILONE Germany was laid inside tunnel in the year 2012- 2013.This tunnel , located between Udhampur to Katra ,was thrown open to passenger traffic in July' 2014, as a part of railway line opened between Udhampur to katra.

2.2 In Feb 2015, an unusual phenomenon of upheaval of ballastless track occurred between Ch. 66.00– Ch. 66.01 inside T25 during ongoing grouting work in the tunnel for tunnel strengthening and seepage rectification.

2.3 Grouting operation was immediately stopped and relief holes were drilled inside drains so as to dissipate the locked up pore water pressure in clayey substratum, if any, formed during grouting. As a safety measure, a speed restriction of 20 kmph was immediately imposed over the affected length to ensure safe running of trains.

2.4 It was observed that due to locked up uplift pressures, an upheaval of 95 mm occurred in the permanent structure (BLT) evenly spread over 13.3 m length of the ballastless track. As a result of this upliftment, track parameters (Gauge, cross levels, longitunallevel, twist and gradient) were checked and found distorted of its service limits.

2.5 Speed restriction of 20 kmph continued till a permanent solution to the problem was found and implemented. However to monitor any further deterioration in the track parameters due to running of trains , measurement of track parameters was regularly done for 3 years and recorded to notice residual active movements if any .It was observed that the hump had steadied with no movements noticed at any stage post occurrence in 2015.

** Chief Engineer / S / USBRL*
*** Dy. Chief Engineer / USBRL*
**** XEN / C / USBRL*

3.0 Possible solutions to the problem

3.1 To address the problem, rounds of discussions were held with the available engineering experts in the field of BLT design for arriving at a practical solution for repair of the hump. Two options to address the problem were zeroed in on :

a) Option 1: Destructive method: Dismantling of the 13.3 m upheaved track structure and re-casting with in-situ concrete. This involved a block period of 21 days i.e. stoppage of all traffic between Udhampur-Katra for three weeks.

b) Option 2: Non-Destructive method: Introduction of a properly designed vertical curve of 1 in 4000 through installation of specially designed modified vossloh fittings for each sleeper for the length of the design curve. This involved its execution under 5-6 blocks of 4-5 hrs each.

3.2 Option (2) was considered as practical and workable solution keeping in view that the work was to be executed under running traffic conditions.

4.0 Design and Planning

4.1 As per the maintenance manual for RHEDA 2000 slab track system, lateral adjustments up to +/-8mm per each fastening and height adjustments up to -4mm/+56 mm can be made up with variable vossloh 3001-U fittings. As this rectification work involved upheaval adjustments greater than the prescribed limits, RAILONE Germany being original system provider for RHEDA 2000 BLT, was engaged for seeking design of vertical curve (1 in 4000) with special modified fittings.

4.2 Precise survey , data collection using digital autolevel and total station was done for left hand and right hand rails and survey data was plotted for profile sketch (Exhibit :1).

4.3 Based on the design, three Verticle curves having radii greater than 1 in 4000 were designed for smoothening of upheaved portion thus entailing total track length of 100 mtrs (166 sleepers)((Exhibit :2) .

4.3 Shimming pattern was calculated under each rail seat for the designed vertical curves. For manufacturing , specially designed height adjustment plates of steel plates (Ap 20s) of thickness 20mm, Plastic height regulation plates (Ap 20u) of 6mm and 10mm, Rail pads (Zw692) varying 2mm to 12 mm were designed. Gauge adjustment was done through angle Guide Plates (Wfp 15U) of varying width. Dowel Screw (Ss 36) length of increased length were designed and manufactured to cater increased height adjustments . (Exhibit: 3).

4.3 Detailed shimming and regulation sheet was prepared for various track components for the designed track length .

4.4 After finalisation of the Shimming list and regulation sheet, manufacturing of these modified fittings was done in Germany, China and India, before they were finally imported to India and transported to site of the work.

5.0 Team Mock up and Work Execution

5.1 Fittings designed as per regulation sheet were enumerated assembled and synchronized as per the sleeper no. and were placed on both sides of the sleepers.

5.2 A Mock up trail for replacement of 5-6 fittings with modified ones was carried out on 17.01.19 under traffic block so as to determine its fitment at site. (Exhibit: 4).

5.3 Execution of Replacement work was planned to be done under a traffic block of four hours (12.00AM to 4.00 AM) taken for 3 days each w.e.f 20.01.19 to 22.01.19.

5.4 Sleeper screws were opened using spanners and Rails were lifted using Manual jacks. Rail seat was cleaned, already installed fittings were taken e out and modified fittings were seated under each rail seat as per the shimming and regulation sheet. The workwww was completed in two days and then sleeper screws were tightened using torque wrench of design torque to maintain track parameters.Track parameters (Gauge, Cross levels, longitudinal rail levels etc) were measured and found within prescribed service limits.The entire work was done in the presence of Open line (Exhibit :5).

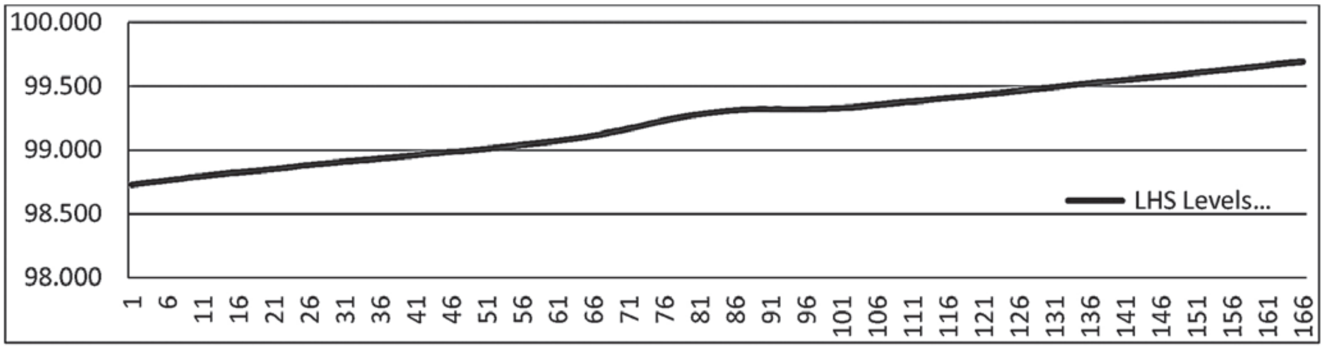
5.5 Track profile plotted pre and post rectification work is placed at Exhibit: 6.

5.6 Speed Relaxation was carried out in stages of 30 kmph and 45 kmph before a final joint speed trail was done in conjunction with open line to relax the speed restriction to sectional speed of 80 kmph.The joint speed trail done was successful and no abnormality was noticed.

6.0 Conclusion

As Indian Railways is fastly moving from ballasted to slab track system, this kind of non-destructive rectification work done has added a new dimension towards the maintenance and repair work of BLT , which can be done under running traffic conditions for such peculiar problems.

Exhibit 1: Upheaval profile (Left hand and right hand rail)



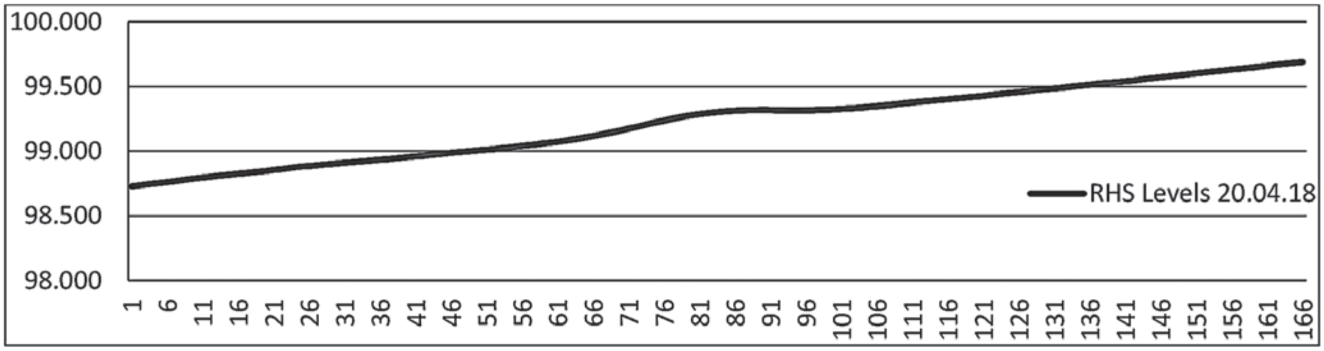


Exhibit 2: Design curves

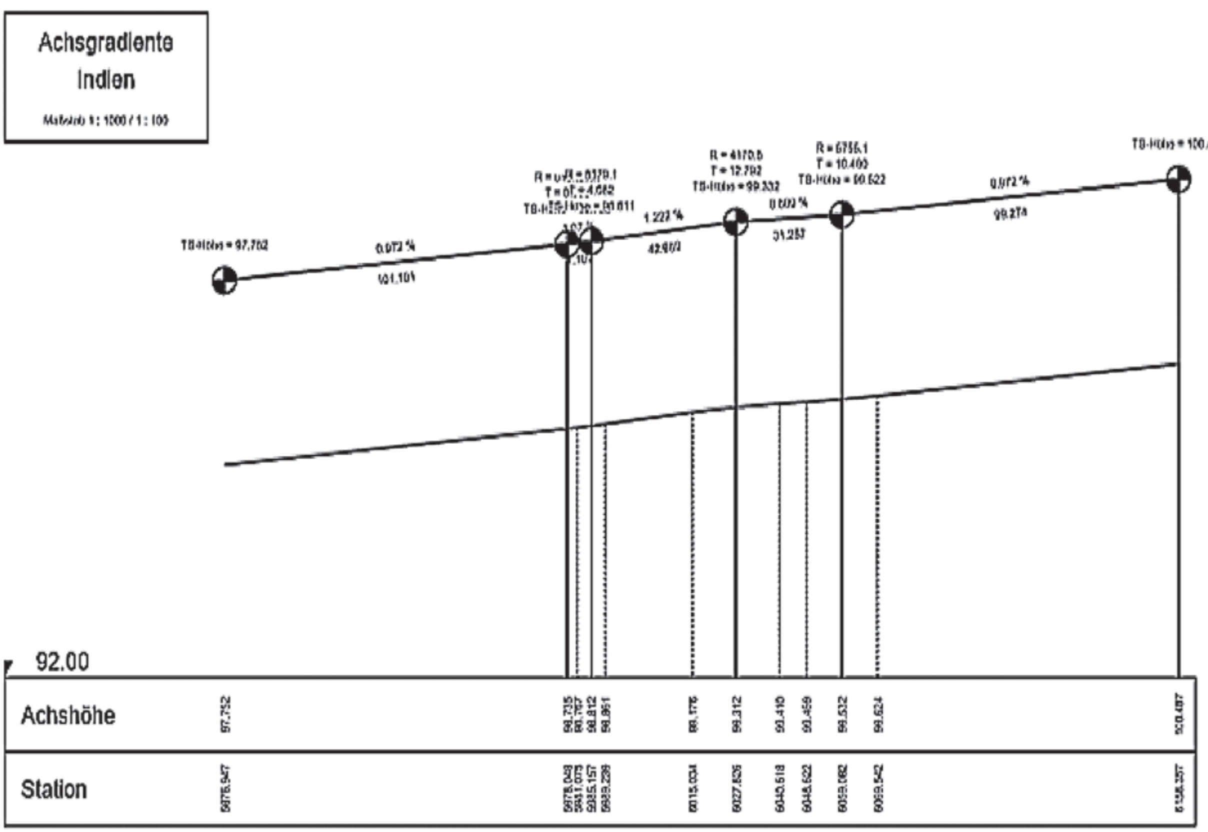


Exhibit 3 :
Modified Vossloh fittings 300-1-U

Modified fittings 300-1-U seated

Figure: Height regulation between +27 mm up to +56 mm

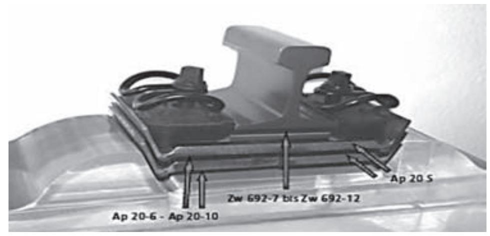


Figure: Height regulation between +57 mm up to +76 mm

Exhibit 4: Mock Up trail

Exhibit 5: Work Execution under traffic block

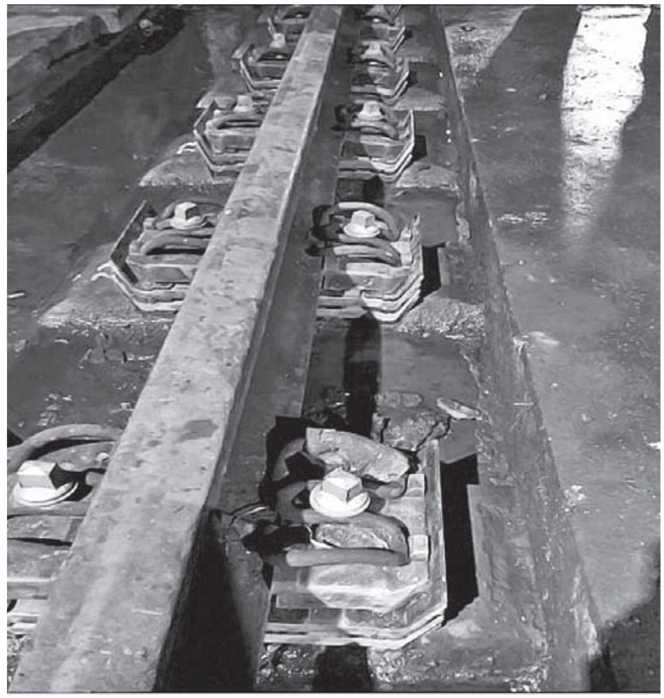
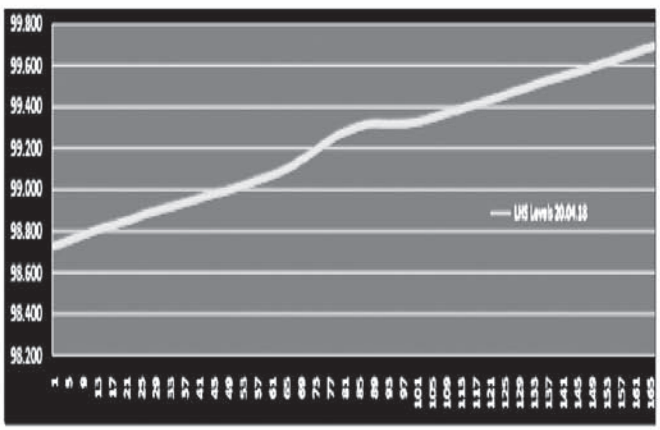
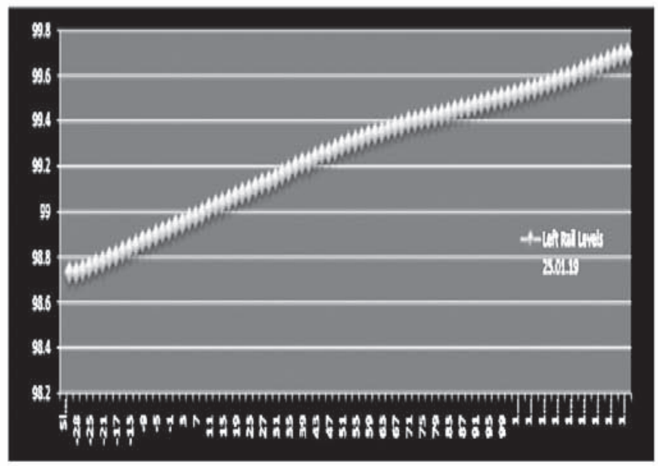
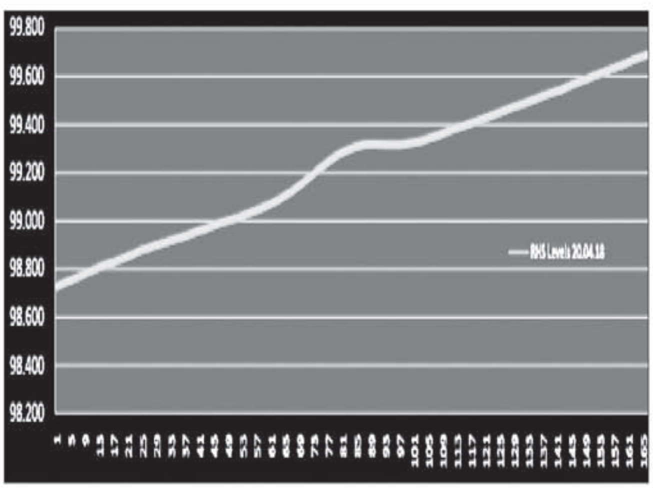
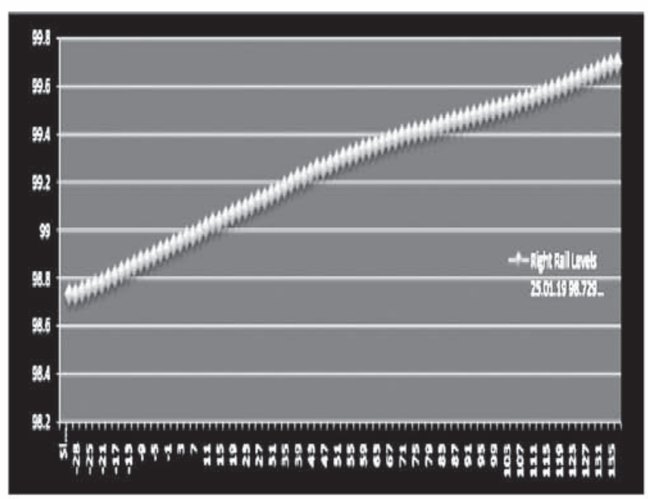


Exhibit 6: Track profile (Pre and Post rectification)









Design of Ground Improvement System for Construction of Embankment on Soft Soils

By
*R. K. Shekhawat**

Synopsis :
A simple computer program for "Design of Ground Improvement system using Vertical Drains or Pre-fabricated Vertical Drains (PVD)" has been developed and being uploaded on IRICEN's website. This technical paper explains the usage of this simple program along with complete theoretical background for understanding this program as well concept of Ground Improvement for construction of Railway Embankment on Soft soils.

1. Soft soils:

Soft soils are typically highly fine grained soils (containing more than 50% fines, passing 75 micron IS sieve), with moderate to high clay content. These soils have very high Liquid Limit and Plastic Limit values, tendency to retain moisture for longer duration thereby having high Natural Moisture Content (NMC), low material permeability, highly compressible and low shear strength.

As per RDSO Guidelines No. GE:G-14, the sub-soil (ground) is classified as "weak" or "soft" when the "N" value (determined from Standard Penetration Test) is less than 5 or Undrained Shear Strength Cu (determined from Triaxial Shear Test) is less than 25 kPa or the Elastic/Deformation Modulus of 2nd Stage Cyclic Plate Load test Ev2 (determined as per DIN:18134-2012) is less than 20 MPa. In such cases, the ground needs suitable "Ground Improvement" before for construction of a safe and serviceable embankment.

2. Consolidation of Soft Soils:

When embankment is constructed on soft soils, the water in the pores of sub-soil mass (which is at atmospheric pressure) develops excess pore water pressure due to the embankment load. Dissipation of this excess pore water pressure, in the process of consolidation, depends on the characteristics of sub-soil, the depth of sub-soil layer and drainage path available. This time may extend upto 2 or 5 years also in case of highly clayey soft soils upto considerable depth. During the process of consolidation, there will be dissipation of pore water pressure and settlement in the sub-soil layers, which will eventually reflect in the settlement of the embankment. The correlation between construction of embankment, buildup/ dissipation of excess pore water pressure and consolidation settlement are shown in Fig.01.

The shear strength of soil is given by following formulae (Coulomb):

$\zeta + C + \sigma \tan \emptyset$ ………. (1)

Where:

ζ = Shear strength of soil

C = Cohesion value of soil

σ = Normal stress on the plane of failure

$\emptyset$ = Angle of internal friction of soil

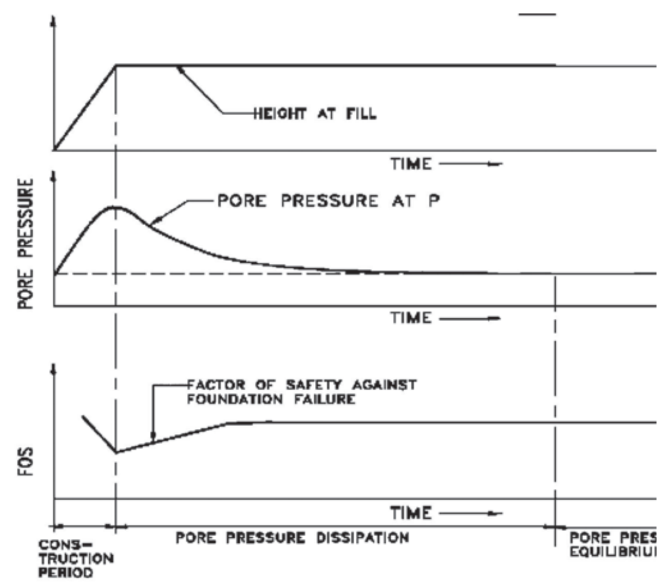


Fig.01: Variation of Pore Water Pressure and Factor of Safety

Using the concept of "effective stress", Terzaghi modified the shear strength formulae taking into account the effect of pore water pressure and gave following equation :

$\zeta = C' + (\sigma - U) \tan \emptyset'$ ………. (2)

Where:

C' = effective Cohesion value of soil

U = Excess pore water pressure

$\emptyset'$ = Effective Angle of internal friction of soil

---

* Sr. Professor (Projects) / IRICEN

3. Problems in construction of embankment on Soft Soils:

There are mainly two problems, which need to be addressed, for constructing embankment on soft soils.

3.1 Safety of embankment during construction: Embankments built on soft clays have been observed to fail along circular slip surfaces, with the slip circle passing through the sub-soil and exiting after the toe of embankment (Fig.02). Such failures are known as "base failures". In most of such cases, the failed soil mass heaves after the toe of embankment.

The factor of safety in such cases is ratio of "Resisting Forces" divided by "Disturbing Forces". The disturbing forces are components of self-weight of embankment, surcharge and any other external force, along the slip surface. The resisting forces mainly consists of the mobilized shear strength along the slip surface, as given in Eq. (2).

As seen from Fig.01, the excess pore water pressure (normally referred as pore water pressure) starts increasing from the time the embankment construction commences, reaches peak value when full height of embankment is constructed (i.e. end of construction) and then starts dissipating. Since increase in pore water pressure (U) decreases the mobilized shear strength, it reduces the factor of safety (FOS), with most critical stage being the "end of construction" when the pore water pressure is at its' peak value. Therefore, the FOS is verified at end of construction and RDSO Guidelines (GE:G-1) stipulate the minimum value of FOS "at the end of construction" to be 1.20 (whereas normally it is 1.40). Therefore, first requirement from safety point of view is to ensure that FOS against circular failure (calculated using effective stress parameters) is minimum 1.20, at the end of construction stage.

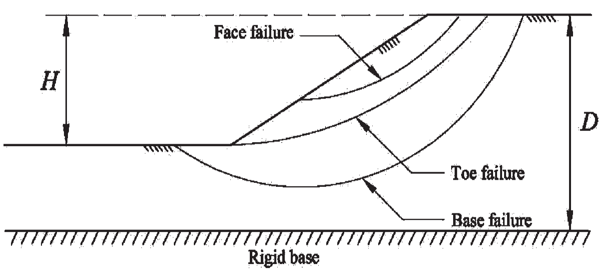


Fig.02: Base Failure of Embankments

3.2 Settlement of embankment: During the process of consolidation, there will be settlement of soft sub-soil layers, which in-turn will reflect in the top surface of the embankment. Thus, the second requirement from serviceability point of view is that first of all the embankment should be constructed for a height equal to "design height of embankment" plus "expected settlement value for the sub-soil" and secondly the settlement (or most of the settlement) of sub-soil should have been completed before commissioning the track on the embankment. The time required for achieving total consolidation (or significant consolidation) depends on the "degree of consolidation".

4. Stage construction methodology:

As explained in para 3.1 above, the safe rate of construction of embankment will depend on gain in shear strength of sub-soil with dissipation of pore water pressure. Normally this rate is very slow, which makes the construction time of high embankment on soft soil very long. Reduction of this time will either require very flat side slopes of embankment or wider sub-banks, both of which will make the construction uneconomical. One of the solution to this is adoption of "stage construction method", wherein total height of embankment is constructed in various stages. Height of any stage is decided based on the safety criteria as discussed in Para 3.1 above. After completion of any stage, a waiting period is given before commencing construction of next stage. This time is decided based on the gain in shear strength of sub-soil due to surcharge load of initial stage and consolidation in this time, which makes the embankment safe for subsequent stage construction. The safe height of every stage and waiting time to be given after very stage construction have been calculated subsequently. Due to many assumptions (associated with theory of consolidation) involved in calculation of waiting period after each stage and the sub-soil not being fully homogeneous, it is observed that actual waiting time required for safe construction of next stage varies from the theoretically calculated time. Therefore, the settlement and pore water pressures are monitored by adopting suitable instrumentation scheme and next stage of construction is taken up only when the settlement and pore water pressure readings indicate that the required degree of consolidation has been achieved, rather than merely relying on the calculated values of settlements and pore water pressure only. This approach is called "observational approach" also. The recorded values of settlement and pore water pressure in case of stage construction methodology will be in the pattern shown in Fig.03.

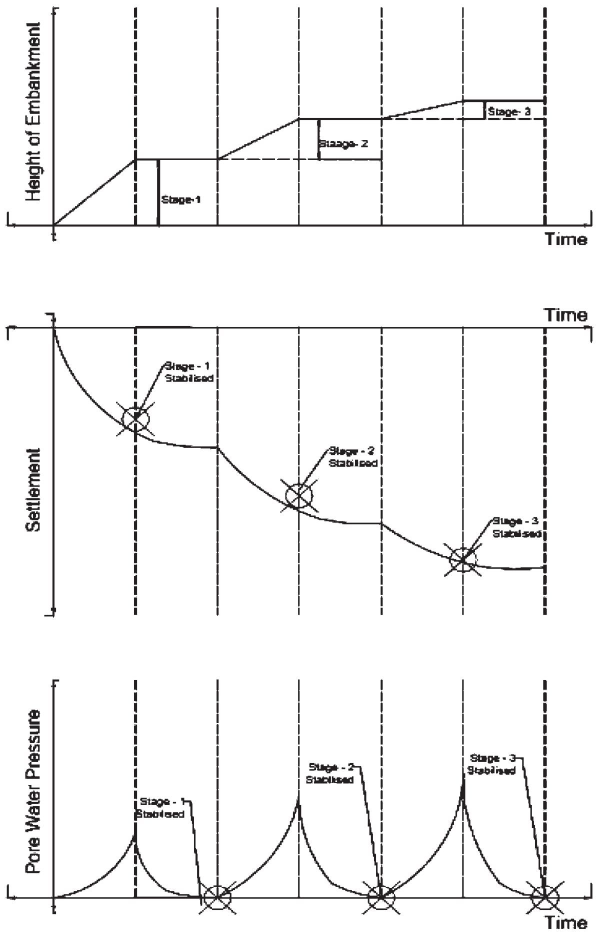


Fig.03: Variation of Settlement and Pore Water Pressure with Time

5. Time Required for Achieving Required Degree of Consolidation:

Using one-dimensional consolidation theory of Terzaghi, time required for achieving any degree of consolidation can be calculated by following formula:

$t = T_V * H_T^2/C_V$ …….. (3)

Where:

t = Time required for consolidation

T_V = Dimensionless Time factor, depending on degree of consolidation (Ref. Table.01)

H_T = Drainage Path (equal to depth of sub-soil layers in case of drainage from one face of sub-soil only and equal to half of this depth in case of drainage from both faces of sub-soil)

C_V = Coefficient of consolidation of sub-soil (determined from Oedometer test)]

Table.01:
Values of TV v/s Degree of Consolidation (U)

U (%)	Tv	U (%)	Tv	U (%)	Tv	U (%)	Tv
5	0.002	30	0.071	55	0.238	80	0.567
10	0.008	35	0.096	60	0.287	85	0.684
15	0.018	40	0.126	65	0.342	90	0.848
20	0.031	45	0.159	70	0.403	95	1.129
25	0.049	50	0.197	75	0.477	100	Infinity

From experience, it is observed that actual rate of settlements is 5-8 times faster than the theoretical value. Hence, actual time of settlement will be 5-8 times less than the calculated value of "t".

6. Degree of Consolidation Achieved in any Time Period:

Value of time factor for any given time can be calculated using Eq. (3), as following:

$TV = t * C_V / H_T^2$ …….. (4)

From the calculated value of TV, the degree of consolidation achieved (U in %) can be worked out using Table.01. The value of "t" will be divided by a factor of 5-8 before using it in the Eq.(4), due to reasons stated in Para.4 above.

7. Settlements: The Total Settlement will Consist of following Parts:

7.1 Elastic settlement: This is settlement of sub-soil layers due to their elastic deformation, when subjected to stress by embankment loading. This settlement is normally very small and can be neglected for all practical purposes.

7.2 Consolidation settlement: This is settlement caused by volume change of sub-soil by draining out of water from its' pores, due to application of embankment loading on the sub-soil. This can be determined using classical Terzaghi's theory for one-dimensional consolidation. A typical embankment with two different types of sub-soil layers and three stages of construction is shown in Fig.04:

h = Height of embankment

H = Total depth of compressible layers

a = Horizontal distance between toe of embankment and end point of the berm/cess

b = Horizontal distance between centre of embankment and end point of the berm/cess

Z = Vertical distance between the bottom of embankment/stage constructed and mid-point of the sub-soil layer under consideration

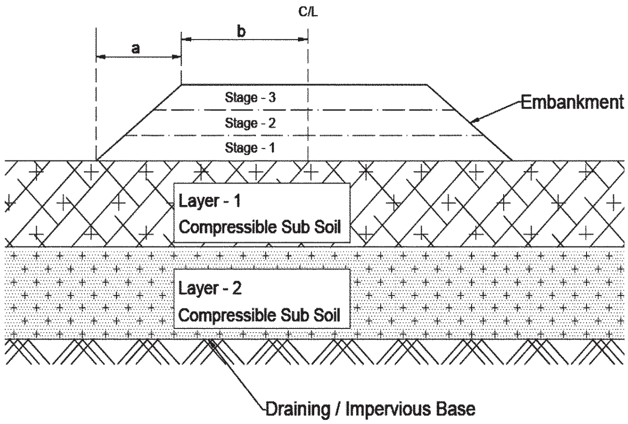


Fig.04: Details of a Typical Embankment

The effective overburden pressure at the centre of any sub-soil layer (P0) before construction of embankment/at any stage will be equal to weight of the sub-soil layers and weight of the embankment soil layer above the centre of that layer. Weight of sub-soil/embankment layers is calculated by multiplying the saturated/submerged density of the concerned layer (submerged density for the portion below water table and saturated density for the portion above water table) with the thickness of the layer concerned. The submerged and saturated density can be calculated by following formula:

$\gamma_{sat} = \{(G-1)/G\} * \gamma_d + \gamma_w$ ……… (5)

$\gamma_{sub} = \{(G-1)/G\} * \gamma_d$ ……… (6)

Where:

γ_{sat} = Saturated density of soil

γ_{sub} = Submerged density of soil

γ_d = Dry density of soil

G = Specific gravity of soil

γ_w = Density of water (= 10 kN/m^3)

Now, if an embankment is constructed, with the details as shown in Fig.04, then Influence Value (I) of half embankment loading, for the given values of a/z and b/z, can either be found from Osterberg Charts shown in Fig.05 or the formulae given in Eq.(7).

$I = 1/\pi\ [\tan^{-1}\{(a/z)+(b/z)\} + \{(b/z)/(a/z)\} * \tan^{-1}\{(a/z)+(b/z)\} + \{(b/z)/(a/z)\} * \tan^{-1}(b/z)]$ ……… (7)

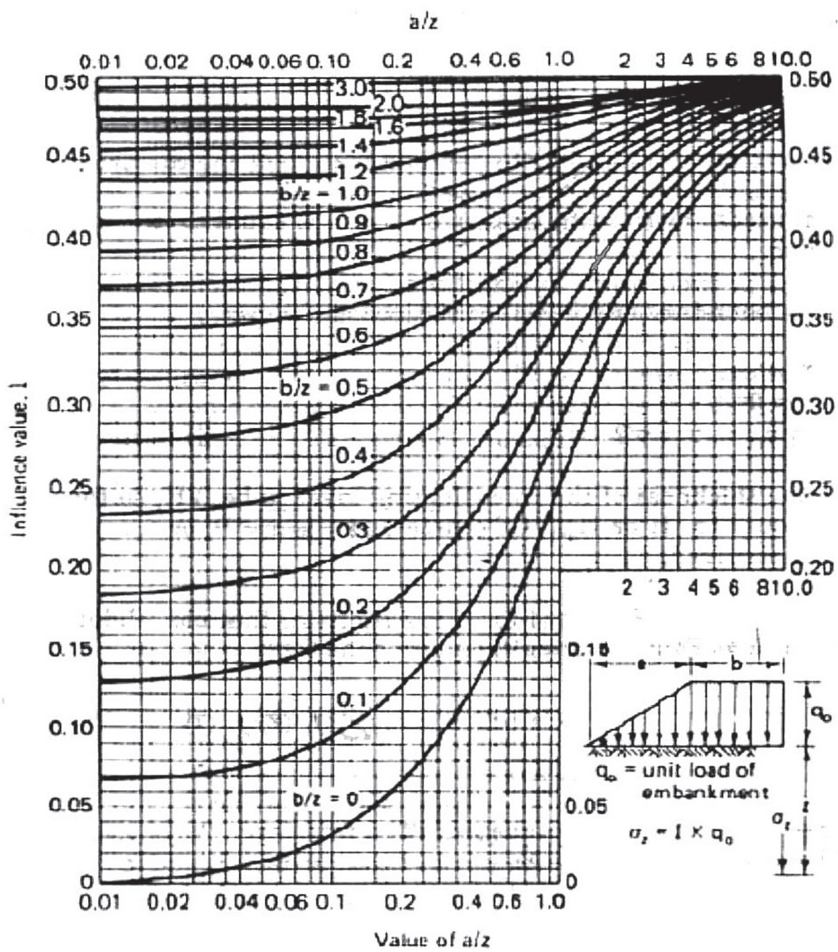


Fig.05: Osterberg Charts for Embankment Loading

For symmetrical embankment, this value of “I” will get doubled to take into account the effect of both parts. The increase in overburden pressure (ΔP_0) at the centre of any sub-soil layer will be given by the following formulae:

$$\Delta P_0 = H_b * \gamma_{bsat} * I \qquad \text{......... (7)}$$

Where:

H_b = Height of embankment constructed

γ_{bsat} = Saturated density of embankment soil

The settlement of embankment is calculated in two parts i.e. change in overburden from P_0 to P_C and then from P_C to $P_0+\Delta P_0$. Here, P_C is “Pre-consolidation Pressure” for the given sub-soil strata. The settlement due to change in overburden pressure from P_0 to P_C can be calculated from following formulae:

$$S_1 = (C_S/1+e_0) * H * \log_{10} (P_C/P_0) \qquad \text{....... (8)}$$

Where:

C_S = Re-compression/swelling index of sub-oil (From Oedometer test)

e_0 = Initial void ratio of sub-soil layer

H = Thickness of the sub-soil layer under consideration

In case, P_C is less than P_0, the value of settlement “S1” will be taken as zero.

The settlement due to change in overburden pressure from P_C to $P_0+\Delta P_0$ can be calculated from following formulae:

$$S_2 = (C_C/1+e_0) * H * \log_{10} (P_0+\Delta P_0/P_C) \qquad \text{.......... (9)}$$

Where:

C_c = Compression index of sub-soil (From Oedometer test)

In case, $P_0+\Delta P_0$ is less than P_C, the value of settlement “S_2” will be taken as zero. The total value of settlement “S” will be sum of S_1 and S_2. The total consolidation settlement will be equal to λ *S, where “λ” is a factor given by Skempton & Bjerrum and reproduced in IS:8009-1976 (Part-I) “Code of Practice for Calculation of Settlement of Foundations”, as given in Table.02 below:

Table.02: Values of “λ”

Type of Clay	Value of "λ"
Very sensitive clays (Soft Alluvial, Estuarine and Marine clays)	1.0 to 1.2
Normally consolidated clays	0.7 to 1.0
Over-consolidated clays	0.5 to 0.7
Heavily over-consolidated clays	0.2 to 0.5

8. Safety in Stage Construction Methodology:

In case, the embankment is constructed in stages, then the safety of each stage height needs to be checked by checking the value of Factor of safety (FOS) and in addition the waiting time required after each stage needs to be calculated. This is done in following manner:

8.1 Factor of Safety after each stage of construction:

Calculate values of “D/H” and “N = CU/γH”.

Where:

D = Total depth/thickness of sub-soil layers

H = Total height of embankment after construction of proposed stage height

CU = Undrained shear strength of the sub-soil

γ = Saturated density of embankment soil

For the calculated values of “D/H”, “N” and Ø (angle on internal friction of embankment soil), the FOS can be determined from the charts given in Fig.06. These charts were developed by Pilot G. and Moreau M. For any intermediate values of “N”, than those given in these charts, linear interpolation may be done.

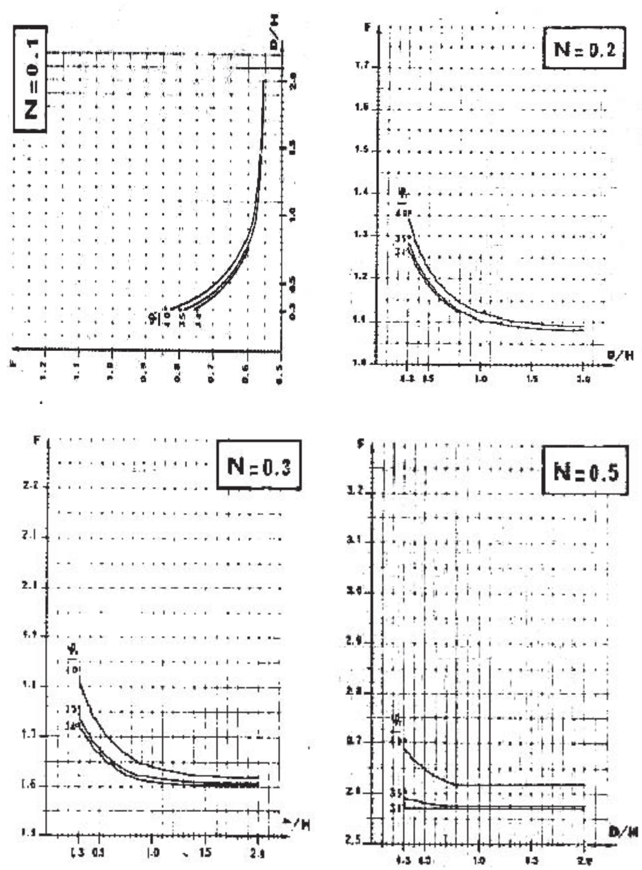


Fig.06: Stability Charts

In case, the FOS comes out to be more than the stipulated value (1.20 for Short tern end of construction stability of railway embankments), then the proposed stage height can be adopted, otherwise the stage height should be revised downwards till it is found safe. This check has to be done for each stage of construction of embankment.

This check is a fast and quick check and it is always advisable that after stage heights are fixed, final check is done using any software for slope stability analysis.

8.2 Waiting time after each stage of construction:

To start with assume some waiting time after first stage of construction. For this time, calculate the Time Factor (TV) using Eq.(4) and for this value of "TV" calculate "Degree of Consolidation (U)" from Table.01. The increase in effective stress on sub-soil due to construction of first stage of embankment ($\Delta\sigma'$) will be given by following formula:

$\Delta\sigma'$ = Stage height * Saturated density of embankment soil * U ……… (10)

The increase in undrained shear strength of sub-soil (ΔCU) due to this stress increment will be given by following formula:

$\Delta C_U = \Delta\sigma' * \tan Øs$ ……… (11)

Where: Øs is angle of internal friction for sub-soil.

Using the increased value of undrained shear strength of sub-soil after first stage construction (i.e. initial value + ΔC_U), check the FOS for the embankment for its' fitness to construct second stage by using stability charts as explained in Para 7.1 above (with value of "H" now being total height of bank after second stage construction). If the FOS value comes out to be less than the stipulated value, then increase the waiting period after first stage or reduce second stage proposed height and repeat the calculation till acceptable value of FOS is obtained. This procedure shall be repeated for calculating the required waiting time after each stage of construction.

9. Settlements with Stage Construction Methodology:

The settlement was calculated in Para.6 above, when the whole embankment was constructed in one go without any stage construction. In case of construction of embankment in stages, settlement can be calculated after each stage also, using the same methodology. It is observed that normally the total settlement calculated with stage construction is more than the total settlement calculated for single stage construction.

10. Ground Improvement using Vertical Drains:

In case the waiting time after each stage, as calculated in Para.8 above, needs to be reduced for optimizing the project completion, then suitable "ground improvement measure" has to be adopted and one of them is using "Vertical Drains". In this method, vertical drains are constructed in sub-soil in either Square or Triangular pattern, as shown in Fig.08. These drains are filled by material of high permeability (normally sand) which introduces the element of "radial consolidation" also in addition to the "vertical or one-dimensional" consolidation, thereby making the overall consolidation faster. If spacing of vertical drains is "s", then the diameter of drain influence area (de) will be equal to "1.128s" for drains in square pattern and equal to "1.05s" for drains in triangular pattern.

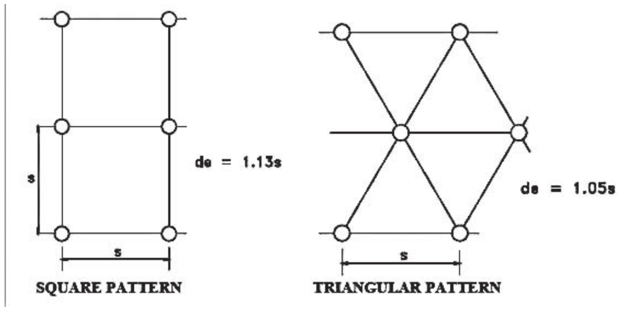


Fig.07: Pattern of Installing Vertical Drains

10.1 Smear effect:

Due to wiping action, caused by the casing or the hollow mandrel used to form the well for vertical drain when it is driven down in the ground and then pulled out, the sub-soil at the drain periphery is smeared. This reduces the horizontal permeability of the sub-soil in the smear zone and thereby reducing the horizontal flow of water to the drain. This in-turn slows down the process of radial consolidation. The extent of smear is expressed by Smear factor (S).

S = ds/dw ……… (12)

Where:

ds = Diameter of smear zone

dw = Diameter of vertical drain

10.2 Well resistance:

If during consolidation, the discharge capacity of drain is reached, the overall consolidation process is retarded and the drain presents resistance to the water flowing in them. The Well Resistance Factor (L) is calculated by following formula:

$L = (32/\pi^2) * (kh/kw) * (HT^2/dw)$ ……… (13)

Where:

kh = Horizontal permeability of sub-soil

kw = Longitudinal permeability of vertical drain fill material

HT = Drainage path for the sub-soil layers

dw = Diameter of vertical drain

10.3 Radial Consolidation:

The time factor for radial consolidation (Tr) due to vertical drain is given by the following formula:

$Tr = Cr*t/de^2$ ……… (14)

Where:

Cr = Coefficient of radial consolidation for the sub-soil

t = Time period for consolidation (same as for vertical consolidation)

The degree of radial consolidation (Ur) is calculated by the following formula:

$Ur = 1 - \exp^{(-8Tr/a + 0.8L)}$ ……… (15)

Where:

L = Length factor, from Eq. (13)

a= A Lumped parameter = A + B + C

$A = \{n^2/(n^2 - S^2)\} * \log_n (n/S)$ ……… (16)

$B = (3n^2 - S^2)/4n^2$ ……… (17)

$C = (kh/ks) * \{(n^2-s^2)/n^2)\} * \log_n S$ ……… (18)

n = de/dw

S = Smear factor, from Eq. (12)

Kh/ks = Normal horizontal permeability of sub-soil divided by horizontal permeability in smear zone of sub-soil

For the same time period (t), the time factor for vertical consolidation (Tv) is calculated using Eq. (3) and for this time factor, the degree of vertical consolidation (UV) is worked out from Table.01. The overall degree of consolidation, in three dimensional consolidation, is given by the following formula:

U = 1- {(1-UV)* (1-Ur)} ……… (19)

To calculate the reduced waiting period, with same stage height, the drain parameters (i.e. its' diameter and spacing) shall be varied to achieve the same overall degree of consolidation as was achieved in one-dimensional consolidation without ground improvement. The same procedure is repeated for each stage to calculate the reduced waiting period, with same stage heights.

11. Ground Improvement using Pre-Fabricated Vertical Drains (PVD):

In case of use of pre-fabricated vertical drains (PVD), the procedure followed is same, as in Para.9 above for vertical drains, except following changes:

(i) Equivalent diameter (dw) is given by the following formula:

$dw = 2(b + tp)/\pi$ ……… (20)

Where:

b = Width of PVD (normally in range of 95-105 mm)

tp = Thickness of PVD (normally in range of 3-6 mm)

(ii) Longitudinal permeability of PVD (kw) is calculated by following formula:

kw = Q/(b*tp) ……… (21)

Where, Q is least discharge capacity of PVD (taking into account the deformation of PVD and damage to PVD, at maximum value of earth pressure to which PVD is subjected).

12. Software for Ground Improvement Calculations:

For calculating numerous values and parameters as described above, a computer program has been developed in MS-Excel and being uploaded on the website of IRICEN. This program computes following parameters:

(i) Time required for achieving any Degree of Consolidation.

(ii) Degree of Consolidation achieved for any given Time

(iii) Total Settlement

(iv) Design of various stages for constructing the bank, without ground improvement, duly verifying Safety for each stage

(v) Settlement at the end of each stage

(vi) Reduced time for each stage of construction with use of vertical drains as ground improvement measure (including design of vertical drain)

(vii) Reduced time for each stage of construction with use of pre-fabricated vertical drains (PVD) as ground improvement measure (including design of PVD)

The generic sketch of the problem solved in this program is as shown in Fig.04. In this program it is assumed that:

(i) Embankment consists of one type of soil only.

(ii) In case of stage construction of embankment, maximum number of stages is three (3).

(iii) The sub-soil consists of two layers of compressible soils. In case sub-soil consists of more than two layer, then convert their thicknesses into two layers of equivalent thicknesses, from consolidation point of view, by the formulae given below:

$Hi = H_1 * (Cvi/Cv_1)^{0.5}$

Where:

Hi = equivalent thickness of with layer w.r.t. 1st layer

H_1 = Thickness of 1st layer

Cvi= Coefficient of Consolidation for layer "i"

Cv_1= Coefficient of Consolidation of 1st layer

(iv) The strata beneath the compressive sub-soil layers can be either free draining type (which will make the drainage path as "Double draining") or impervious type (which will make the drainage path as "Double draining")

This simple program not only performs various calculations but also explains the flow of computations in a sequential manner, thereby helping the users in developing the better understanding of the subject.

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Matheran Train

Google Mapping of Railway Affecting Tanks in Ongole Sub-Division

By
Pratham Agrawal*

Introduction :

The washout and damage due to breaches and flash floods has been one of the persistent causes of major accidents involving huge casualties and loss/ damage to railway assets. These accidents also involve long disruptions due to washout of earthwork and/ or related infrastructure.

Tanks are potential source of water streams/ flash floods affecting earthwork and tracks, creating situation leading to washout and possibility of long disruptions, if not accidents.

While as per the codal provisions, an annual inspection is to be carried out with the civil authorities to take measures to avoid such possibilities, the exercise undertaken by me has been to map the potential danger spots to railway track to create a basic information frame work which can be used to launch mitigation measures along the track alignment.

Exercise Undertaken :-

To begin with, let me throw some light on terms utilized and the associated codal provisions.

The term Railway Affecting Tank means any tank which if not constructed or maintained or operated properly may result in danger to railway line.

As per the IRPWM Para 726 Clause 3 (b) - Assistant Engineer shall jointly inspect with civil authorities (in this case, state irrigation officials) all RAT's before the monsoon every year and arrange for their safe maintenance to avoid any danger to nearby tracks and structures.

During the course of my yearly inspection which was conducted in the month of March 2017, following observations were made and instructions were passed on to the state irrigation officials -

(i) The Existing tanks may be upgraded to latest standards, wherever feasible, in a phased manner.

(ii)The tanks which have not been provided with the adequate holding capacity should be provided with additional weirs to flush out excess water.

(iii) Periodic dredging & maintenance of existing tanks may be ensured.

(iv) River training works may be strengthened for controlled movement of flood water.

Accordingly, the state officials brought to light that under the Neeru Chettu program, many works were being undertaken and are under progress.

During the course of my yearly inspection, i was finding it difficult to visualize as to how a breach in tank could affect the railway line and what are such spots on railway line and on conversing / interacting with my batch mates and seniors colleagues, i realized that they were facing similar challenges. In fact the breach in Guntur division acted as a catalyst for the work i have undertaken.

Under this i have tried to identify and locate the railway affecting tanks in Google maps there by making the process of visualization easier. I have outlined the tank boundaries, marked the location of surplus weir and traced the stream path which makes it easier to visualize as to how the breach in tank will result in danger to railway line. The above data has been collected in consultation with state irrigation officials and a certification was also received from them.

In Ongole sub division, 76 nos. of RAT's have been identified, located on Google map, their boundaries marked, breach point(s) identified, breach stream path identified and marked right up to railway tracks. This has created a basic data/ information frame work of potential danger points along the railway tracks where mitigation measures could be planned and launched to avoid situations of flash floods causing washout or track sinking. The outcome of this exercise is the Google mapping of RAT's, few photos of which is enclosed.

** Sr. ADEN / OGL / SCR*

Applications & Future Developments :–

1. With the help of historical data of rain fall and tank breach, breach proneness mapping of each tank can be undertaken to urge civil authorities to prioritize their attention and deployment of resources for repairs to the tanks.

2. The historical data as above can also form the basis of prioritizing railways attention and deployment of resources for taking **mitigation measures** at the potentially dangerous spots along the track.

3. In case of any danger or breach, further activities can be **planned in a better** manner by consulting with the map. In fact a close look at the map would indicate that pivotal tanks which are in line of **successive breaching** have been identified. The state irrigation officials have taken cognizance of the same and works are on for strengthening of those tanks.

4. This data can be shared with agencies like **NDRF** who can then initiate steps at the local level and can be **better prepared** in case of any unusual/breach. This data can be used by NDRF to integrate with their larger disaster relief plans.

In the end I would like to conclude by saying that this entire exercise was done with a goal of **"Bring an activity into the realm of PREDICTIVE FRAMEWORK"**

Time Dependent Stresses in Reinforced Concrete Circular Columns from the View of Indian Bridge Codes

By
R. Sundaresan*

Abstract :
In the year 2004, Limit States Method was introduced first time in India for the design of Railways' bridges. There after the Railway Bridges were designed for the Strength against the Ultimate Collapse and also checked primarily for the Stresses and crack width in service based on the limitations stipulated in the Indian Railways Standard Code for the Concrete Bridges. In 2011, the Indian Road Congress revised the Code for the design of Road Bridges based on the Limit States Method. This paper analyses the calculation of time dependent stresses in RC cracked columns having circular shape following the above Railway and Road Bridge Codes for the design of bridges.

1.0 Introduction:

The increase in compressive strain in concrete due to creep is well known from our experience in the past. When the stress in concrete is below one-third the characteristic compressive stress caused by the permanent loads, it is usual to consider the creep strain to be proportional to the actual elastic compressive stress. In addition to creep, the shrinkage strain acting on the compressive zone of concrete will also increase the compressive strain. Since strain caused by creep and shrinkage is not dependent on the loads acting on the structural element, the analysis for finding the additional stresses is a non-linear problem requiring a little more computational effort as compared to the linear problems. The procedure for the calculation of additional stresses in Reinforced Concrete (RC) cracked circular columns is presented below.

1.1 Significance for the Study:

In order to account for the creep and shrinkage, the Indian Railway Bridge Code[1] uses the empirical modular ratios of 280/fck, and 420/fck for the tension zone and compression zone respectively. The IRC:112-2011[2] provides for the Effective Modulus Method for calculating the time dependent stresses. Since two different approaches are used for the design of bridges, it becomes necessary to study the effects of creep and shrinkage based on the above two codes for ensuring their reliability in predicting the safe working stresses.

1.2 Limitation and extension of the work:

For this study, a cracked circular RC column subjected to an axial load and uniaxial bending is considered. Since circular column is symmetrical about the centroidal axis which is perpendicular the axis of bending, the circular columns subjected to axial load and biaxial bending can be treated as the column under axial load and uniaxial bending by replacing uniaxial moment by the resultant moment given by Eq.(1)

$$M_r = \sqrt{M_x^2 + M_y^2} \qquad \text{Eq.(1)}$$

2 Distribution of Strains and Stresses:

The cross section of RC column and distribution of strain across the depth is shown in Fig.1.

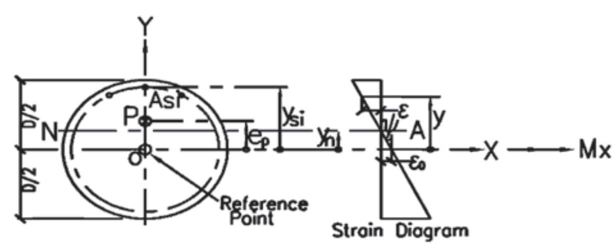

Fig. 1: Section of Column and Distribution of Strain along Depth

The strain at a distance, 'y' from the centre of circular column is given by Eq.(2)

$$\varepsilon = \varepsilon_0 + \psi y \qquad \text{Eq.(2)}$$

ε_0=strain at reference point

ψ=strain gradient

y=distance from reference point

* SSE/Drg./CAO/C/MS/SR

Hence the stress in concrete can be expressed as

$$\sigma_c = E_c(\varepsilon_0 + \psi y) \text{ if } y_n \le y \le \left(a = \frac{D}{2}\right) \qquad \text{Eq.(3)}$$

$$= 0 \text{ if } y < y_n$$

where E_c=elastic modulus of concrete

Similarly, the stress in reinforcement steel can be found as

$$\sigma_s = E_s(\varepsilon_0 + \psi y) \qquad \text{Eq.(4)}$$

where E_s=elastic modulus of rebars

3 Methodology for Calculation:

The four steps analysis suggested in Ghali A, et. Al3 is used in this paper for calculating the time dependent stresses. The detailed methodology as applied to the RC columns can also be found from Sundaresan R4.

4.0 Formulae for Calculation:

The initial and time dependent stresses in concrete and reinforcement steel bars can be calculated from the following formulea.

4.1 Calculation of neutral axis at the time of initial loading:

Taking the stress at the extreme compression fibre as, the distance of neutral axis, yn from the centre of column can be calculated from Eq.(5a) given below.

$$\frac{M_c + M_s}{P_c + P_s} - \frac{M}{P} = 0 \qquad \text{Eq.(5a)}$$

where

$$M_c = \frac{-2\sigma_c}{(a - y_n)}\left\{\begin{aligned}&\frac{y_n}{3}(a^2 - y_n^2)^{\frac{3}{2}} \\ &+\frac{a^4}{32}\left[\begin{aligned}&\sin\left(4\cos^{-1}\left(\frac{y_n}{a}\right)\right) \\ &-4\cos^{-1}\left(\frac{y_n}{a}\right)\end{aligned}\right]\end{aligned}\right\} \qquad \text{...Eq.(5b)}$$

$$M_s = \frac{\alpha\sigma_c}{(a - y_n)}\sum_{i=1}^{ns} A_{si} y_{si}(y_{si} - y_n) \qquad \text{Eq.(5c)}$$

$$P_c = \frac{2\sigma_c}{(a - y_n)}\left\{\begin{aligned}&\frac{1}{3}(a^2 - y_n^2)^{\frac{3}{2}} \\ &-y_n\left[\frac{\pi a^2}{4} - \left(\begin{aligned}&\frac{y_n}{2}\sqrt{a^2 - y_n^2} \\ &+\frac{a^2}{2}\sin^{-1}\left(\frac{y_n}{a}\right)\end{aligned}\right)\right]\end{aligned}\right\} \qquad \text{...Eq.(5d)}$$

$$P_s = \frac{\alpha\sigma_c}{(a - y_n)}\sum_{i=1}^{ns} A_{si}(y_{si} - y_n) \qquad \text{Eq.(5e)}$$

$$a = \frac{D}{2};\ \alpha = \frac{E_s}{E_c};\ ns = \text{Nos of rebars} \qquad \text{Eq.(5f)}$$

$$y_{si} = (a - d_c') \cdot \sin\left(\frac{2\pi(i-1)}{ns}\right);\ i = 1, 2, \ldots, ns$$

and $d_c' = \text{Eff. cover}$ Eq.(5g)

4.2 Sectional properties of RC circular columns at time of initial loading, :

The sectional properties of effective concrete section

alone and transformed sectional properties of the column can be calculated using the following Eq.(6a)-Eq.(6f)

$$A_c = \frac{\pi a^2}{2} - y_n\sqrt{a^2 - y_n^2} - a^2\sin^{-1}\left(\frac{y_n}{a}\right) \qquad \text{Eq.(6a)}$$

$$A_{tr} = A_c + \alpha\sum_{i=1}^{ns} A_{si} \qquad \text{Eq.(6b)}$$

$$B_c = \frac{2}{3}(a^2 - y_n^2)^{\frac{3}{2}} \qquad \text{Eq.(6c)}$$

$$B_{tr} = B_c + \alpha\sum_{i=1}^{ns} A_{si} y_{si} \qquad \text{Eq.(6d)}$$

$$I_c = \frac{a^4}{16}\left[\sin\left(4\cos^{-1}\left(\frac{y_n}{a}\right)\right) - 4\cos^{-1}\left(\frac{y_n}{a}\right)\right] \qquad \text{...Eq.(6e)}$$

$$I_{tr} = I_c + \alpha\sum_{i=1}^{ns} A_{si} y_{si}^2 \qquad \text{Eq.(6f)}$$

where 'a' is the radius of the RC circular column, Y_n is the distance of neutral axis from the centre of circular column and α is the ratio of Young's modulus of steel to the initial modulus of concrete.

4.3 Calculation of strains and stresses at the time of initial loading t_0 :

The distance of neutral axis, Y_n can be found from Eq.(5a) by using trial and error method or by using numerical method like bisection method. The sectional properties can be calculated from Eq.(6a)-Eq.(6f). The initial strain and strain ε_0 (t_0) and strain gradient,$\psi(t_0)$ can be related to the external loadings in the Eq.(7).

$$\begin{Bmatrix} P \\ M \end{Bmatrix} = E_c(t_0)\begin{pmatrix} A_{tr} & B_{tr} \\ B_{tr} & I_{tr} \end{pmatrix}\begin{Bmatrix} \varepsilon_0(t_0) \\ \psi(t_0) \end{Bmatrix} \qquad \text{Eq.(7)}$$

From the above equation, we can find the values of ε_0 (t_0) and $\psi(t_0)$ as given in Eq.(8) and Eq.(9) respectively

$$\varepsilon_0(t_0)=\frac{(I_{tr}P-B_{tr}M)}{E_0(t_0)\left[A_{tr}I_{tr}-B_{tr}^2\right]} \quad \text{Eq.(8)}$$

$$\psi(t_0)=\frac{(-B_{tr}P+A_{tr}M)}{E_0(t_0)\left[A_{tr}I_{tr}-B_{tr}^2\right]} \quad \text{Eq.(9)}$$

The stress in concrete and reinforcement steel bars can be found from Eq.(3) and Eq.(4) respectively from the above values and using the appropriate values of modulus of concrete and steel.

4.4 Sectional properties of RC circular columns at time, *t*:

The modified modulus of concrete and modified modular ratio considering the effect of creep can be found by

$$\bar{E}_c=\frac{E_c}{(1+\varphi(t,t_0))}\;;\;\bar{\alpha}=\frac{E_s}{\bar{E}_c} \quad \text{Eq.(10)}$$

The sectional properties of effective concrete alone and transformed sectional properties modified to take care of creep of concrete in the column are given by Eq.(11a)-Eq.(11f).

$$A_c=\frac{\pi a^2}{2}-y_n\sqrt{a^2-y_n^2}-a^2\sin^{-1}\left(\frac{y_n}{a}\right) \quad \text{...Eq.(11a)}$$

$$\bar{A}_{tr}=A_c+\bar{\alpha}\sum_{i=1}^{ns}A_{si} \quad \text{Eq.(11b)}$$

$$B_c=\frac{2}{3}\left(a^2-y_n^2\right)^{\frac{3}{2}} \quad \text{Eq.(11c)}$$

$$\bar{B}_{tr}=B_c+\bar{\alpha}\sum_{i=1}^{ns}A_{si}y_{si} \quad \text{Eq.(11d)}$$

$$I_c=\frac{a^4}{16}\left[\sin\left(4\cos^{-1}\left(\frac{y_n}{a}\right)\right)-4\cos^{-1}\left(\frac{y_n}{a}\right)\right] \quad \text{...Eq.(11e)}$$

$$\bar{I}_{tr}=I_c+\bar{\alpha}\sum_{i=1}^{ns}A_{si}y_{si}^2 \quad \text{Eq.(11f)}$$

4.5 Restraining stress to the free creep and shrinkage:

The restraining stress in the concrete section against the free creep and shrinkage can be written as

$$\Delta\sigma_{res}=-\bar{E}_c\left\{\varphi(t,t_0)\left[\varepsilon_0(t_0)+\psi(t_0)a\right]+\varepsilon_{cs}\right\} \quad \text{...Eq.(12)}$$

where φ $(t,t_0)$is the creep coefficient and ‘a’ is the radius of the RC circular column.

4.6 Resultant force for restraining stress due the free creep strain:

The forces induced at the reference point due to the artificial restraint to the free strain of creep on concrete section alone are given by the following Eq.(13).

$$\begin{Bmatrix}\Delta P_{crp}\\ \Delta M_{crp}\end{Bmatrix}=-\bar{E}_c\varphi(t,t_0)\begin{pmatrix}A_c & B_c\\ B_c & I_c\end{pmatrix}\begin{Bmatrix}\varepsilon_0(t_0)\\ \psi(t_0)\end{Bmatrix} \quad \text{...Eq.(13)}$$

4.7 Resultant force for restraining stress due the free shrinkage strain:

On the application of restraint to the concrete section the free shrinkage strain will induce the following force at the reference point.

$$\begin{Bmatrix}\Delta P_{shr}\\ \Delta M_{shr}\end{Bmatrix}=-\bar{E}_c\varepsilon_{cs}\begin{Bmatrix}A_c\\ B_c\end{Bmatrix} \quad \text{Eq.(14)}$$

4.8 Additional strain due to creep and shrinkage at time, *t*:

The restraining force to prevent free creep and shrinkage strain can be found using Eq.(13) and Eq.(14) respectively. On summing up the additional axial forces and bending moments due to creep and shrinkage calculated from above equations can now be applied in reversed direction to nullify the effect of restraint. The mathematical equation for the above is expressed as in Eq.(15)

$$\begin{Bmatrix}\Delta\varepsilon_0(t,t_0)\\ \Delta\psi(t,t_0)\end{Bmatrix}=\frac{1}{\bar{E}_c}\begin{pmatrix}\bar{A}_{tr} & \bar{B}_{tr}\\ \bar{B}_{tr} & \bar{I}_{tr}\end{pmatrix}^{-1}\begin{Bmatrix}-\Delta P\\ -\Delta M\end{Bmatrix} \quad \text{Eq.(15)}$$

where $-\Delta P=-(\Delta P_{crp})-(\Delta P_{shr})$, and

$-\Delta M=-(\Delta M_{crp})-(\Delta M_{shr})$

4.9 Change in stress in concrete due to creep and shrinkage at time, *t*:

The stress due to restraint to creep and shrinkage can be found from Eq.(12) using modified modulus of concrete, creep coefficient, shrinkage strain and initial strain. Adding the above restraining stress to the stress caused by the change in strain due to creep and shrinkage, the final stress in concrete can be calculated by

$$\Delta\sigma_c=\Delta\sigma_{res}+\bar{E}_c\left[\Delta\varepsilon_0(t,t_0)+\Delta\psi(t,t_0)y\right] \quad \text{...Eq.(16)}$$

4.10 Additional stress in reinforcement bars due to creep and shrinkage:

Since the concrete in tensile zone is ignored, the changes in stress in reinforcement steel bars are calculated using the following equation.

$$\Delta\sigma_s=E_s\left[\Delta\varepsilon_0(t,t_0)+\Delta\psi(t,t_0)y_s\right] \quad \text{Eq.(17)}$$

5.0 Numerical example for Cracked Circular Column Using IRC:112-2011 code:

An RC circular column having the sectional properties shown in Fig.2 is taken for demonstration of application of theoretical equations given in the previous sections to the real structural member. The above theoretical equations are derived from integration by considering an elemental strip parallel to the X-axis. The grade of concrete for the column is M35 and the grade of steel for the reinforcement bars is Fe500D. The Initial modulus for concrete taken from the code is 32308MPa. The Young's modulus of reinforcement steel considered for this example is 200000MPa. Hence, the initial elastic modular ratio of concrete can be calculated as 6.19. The creep coefficient and the shrinkage strain considered for this study are 2.5 and -0.0003 respectively.

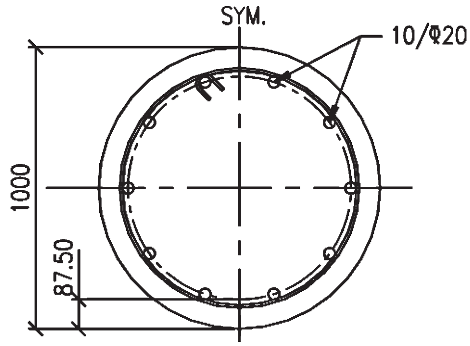

Fig. 2: Cross Section of RC Column

5.1 External loads acting on the column:

The axial compressive force acting in the positive direction of 'Y-axis' is considered to be negative for the purpose of this paper. In a similar manner, the bending moment causing compressive stress in the positive direction of 'Y-axis' is taken as negative. The column section is subjected to an external axial load of -2430kN and bending moment of -713.40kNm acting permanently due to dead and superimposed dead loads. In addition to the above, an axial load of -810kN and bending moment of -237.80kNm are acting as transient loads due to live loads.

5.2 Initial stresses for the total loads:

By substituting $\boldsymbol{P_{TOTAL}} = (\boldsymbol{P_{PERM}} + \boldsymbol{P_{TRANS}})$ in the place $\boldsymbol{P}$ and $\boldsymbol{M_{TOTAL}} = (\boldsymbol{M_{PERM}} + \boldsymbol{M_{TRANS}})$ in the place of $\boldsymbol{M}$ in Eq.(5a) and using Eq.(5b)-Eq.(5g), the distance of neutral axis, $\boldsymbol{Y_{N\ TOTAL}}$ from the centre of circular column is calculated as -39.42mm by trial and error procedure. The transformed sectional properties of the RC column section is found from Eq.(6a)-Eq.(6f). The Eq.(7) can be formulated as

$$\begin{Bmatrix} -3.24E+06 \\ -9.5E+08 \end{Bmatrix} = 3.23E+04 \begin{pmatrix} 4.50E+05 & 8.22E+07 \\ 8.22E+07 & 2.61E+10 \end{pmatrix} \begin{Bmatrix} \varepsilon_0(t_0) \\ \psi(t_0) \end{Bmatrix}$$

From the above, we find the strain and the curvature or strain gradient by the Eq.(8) and Eq.(9) respectively and stresses in concrete and reinforcement steel are calculated from Eq.(3) and Eq.(4) as

$$\varepsilon_0(t_0)_{Total} = -3.96E-05\ ;\ \psi(t_0)_{Total} = -1.00E-06$$
$$\sigma_{cTotal} = -17.50\text{MPa}\ ;\quad \sigma_{scTotal} = -72.69\text{MPa}$$
$$\sigma_{stTotal} = 70.86\text{MPa}$$

5.3 Initial stresses for permanent loads:

Similarly, the strain and strain gradient for the permanent loads can be formulated as

$$\begin{Bmatrix} -2.43E+06 \\ -7.13E+08 \end{Bmatrix} = 3.23E+04 \begin{pmatrix} 4.50E+05 & 8.22E+07 \\ 8.22E+07 & 2.61E+10 \end{pmatrix} \begin{Bmatrix} \varepsilon_0(t_0) \\ \psi(t_0) \end{Bmatrix}$$

The calculated values of strain, strain gradient and stresses in concrete and steel reinforcement bars are given below.

$$\varepsilon_0(t_0)_{Perm} = -1.48E-15\ ;\ \psi(t_0)_{Perm} = -7.53E-07$$
$$\sigma_{cPerm} = -13.12\text{MPa}\ ;\quad \sigma_{scPerm} = -54.52\text{MPa}$$
$$\sigma_{stPerm} = 53.15\text{MPa}$$

5.4 Stresses due to transient loads:

Since the principle of superposition cannot be applied for the analysis of RC column subjected to axial load and uniaxial bending, the stresses caused by the transient loads are found by the difference of stress produced by the total loads and the permanent loads as given below.

$$\sigma_{cTrans} = \sigma_{cTotal} - \sigma_{cPerm} = -4.37\text{MPa}$$
$$\sigma_{scTrans} = \sigma_{scTotal} - \sigma_{scPerm} = -18.17\text{MPa}$$
$$\sigma_{stTrans} = \sigma_{stTotal} - \sigma_{stPerm} = 17.72\text{MPa}$$

5.5 Restraining stress in concrete to free creep and shrinkage strains:

The stress in concrete due to the restraint of free creep and shrinkage can be calculated from Eq.(12) as $\Delta\sigma_{res} = 12.14\text{MPa}$

5.6 Equivalent forces for the restraining stress in concrete:

The forces likely to cause the restraining stress due to free creep strain and shrinkage strain can be found by Eq.(13) and Eq.(14) respectively as

$$\begin{Bmatrix} \Delta P \\ \Delta M \end{Bmatrix} = \begin{Bmatrix} 1.73E+06 \\ 5.52E+08 \end{Bmatrix}_{crp} + \begin{Bmatrix} 1.20E+06 \\ 2.76E+08 \end{Bmatrix}_{shr} = \begin{Bmatrix} 2.93E+06 \\ 8.27E+08 \end{Bmatrix}$$

5.7 Additional strain due to creep and shrinkage:

The modified sectional properties are calculated from Eq. (11a)-Eq.(11f) for the distance of neutral axis, $\boldsymbol{Y_{N\ PERM}}$ found for the permanent loads, modified values of modulus of concrete and modular ratio. The additional strain and curvature can be formulated in matrix form by applying the restraint forces in the reversed direction as

$$\begin{Bmatrix} -2.93E+06 \\ -8.27E+08 \end{Bmatrix} = 9.23E+03 \begin{pmatrix} 4.98E+05 & 1.02E+08 \\ 1.02E+08 & 3.75E+10 \end{pmatrix} \begin{Bmatrix} \Delta\varepsilon_0(t,t_0) \\ \Delta\psi(t,t_0) \end{Bmatrix}$$

From the above, the values of $\Delta\varepsilon_0(t,t_0)$ and $\Delta\psi(t,t_0)$ can be found as

$$\begin{Bmatrix} \Delta\varepsilon_0(t,t_0) \\ \Delta\psi(t,t_0) \end{Bmatrix} = \begin{Bmatrix} -3.32E-04 \\ -1.49E-06 \end{Bmatrix}$$

5.8 Additional stresses due to creep and shrinkage:

The time dependent additional stress in concrete and reinforcement steel bars controlling the design can be found as

$$\Delta\sigma_c = \left(\frac{12.14 + 9.23E+03^*}{(-3.32E-04 + (-1.49E-06)*500)}\right) = 1.66\text{MPa}$$

$$\Delta\sigma_{sc} = \left(\frac{(2.0E+05 - 9.23E+03)*}{(-3.32E-04 + (-1.49E-06)*392.31)}\right) = -186.06\text{MPa}$$

$$\Delta\sigma_{st} = \left(\frac{(2.0E+05)*}{(-3.32E-04 + (-1.49E-06)*(-392.31))}\right) = 38.79\text{MPa}$$

5.9 Final stresses considering creep and shrinkage:

The stresses in concrete and reinforcement steel bars can be obtained by adding the total stresses at initial loading and additional stresses due to creep on permanent loads and shrinkage strains as

$$\begin{Bmatrix}\sigma_c\\ \sigma_{sc}\\ \sigma_{st}\end{Bmatrix} = \begin{Bmatrix}-17.50\\ -72.69\\ 70.86\end{Bmatrix} + \begin{Bmatrix}1.66\\ -186.06\\ 38.79\end{Bmatrix} = \begin{Bmatrix}-15.84\\ -258.75\\ 109.65\end{Bmatrix}$$

6.0 Application of IRS concrete bridge code:

In this method, the modular ratio is calculated based on the empirical relationship given this code. The effect of creep and shrinkage is partially considered in the above relationship.

6.1 Distance of neutral axis:

The distance of neutral axis can be calculated by the following Eq.(18a) using trial and error method or any computer method involving numerical procedures like bisection method.

$$\frac{M_c + M_{sc} + M_{st}}{P_c + P_{sc} + P_{st}} - \frac{M}{P} = 0 \qquad \text{Eq.(18a)}$$

where

$$M_c = \frac{-2\sigma_c}{(a-y_n)}\left\{\begin{matrix}\frac{y_n}{3}(a^2 - y_n^2)^{\frac{3}{2}}\\ +\frac{a^4}{32}\left[\begin{matrix}sin\left(4\cos^{-1}\left(\frac{y_n}{a}\right)\right)\\ -4\cos^{-1}\left(\frac{y_n}{a}\right)\end{matrix}\right]\end{matrix}\right\} \qquad \text{...Eq.(18b)}$$

$$M_{sc} = \frac{\sigma_c}{(a-y_n)}\frac{420}{f_{ck}}\sum_{i=1}^{ns} A_{si}y_{si}(y_{si} - y_n) \text{ if } y_{si} \geq y_n \qquad \text{...Eq.(18c)}$$

$$M_{st} = \frac{\sigma_c}{(a-y_n)}\frac{280}{f_{ck}}\sum_{i=1}^{ns} A_{si}y_{si}(y_{si} - y_n) \text{ if } y_{si} < y_n \qquad \text{...Eq.(18d)}$$

$$P_c = \frac{2\sigma_c}{(a-y_n)}\left\{\begin{matrix}\frac{1}{3}(a^2 - y_n^2)^{\frac{3}{2}}\\ -y_n\left[\frac{\pi a^2}{4} - \left(\begin{matrix}\frac{y_n}{2}\sqrt{a^2 - y_n^2}\\ +\frac{a^2}{2}sin^{-1}\left(\frac{y_n}{a}\right)\end{matrix}\right)\right]\end{matrix}\right\} \qquad \text{...Eq.(18e)}$$

$$P_{sc} = \frac{\sigma_c}{(a-y_n)}\frac{420}{f_{ck}}\sum_{i=1}^{ns} A_{si}(y_{si} - y_n) \text{ if } y_{si} \geq y_n \qquad \text{...Eq.(18f)}$$

$$P_{st} = \frac{\sigma_c}{(a-y_n)}\frac{280}{f_{ck}}\sum_{i=1}^{ns} A_{si}(y_{si} - y_n) \text{ if } y_{si} < y_n \qquad \text{...Eq.(18g)}$$

6.2 Stress in concrete:

After finding the distance of neutral axis from Eq.(18a), the bending compressive stress at extreme compression fibre can be obtained from the following Eq.(19).

$$\sigma_c = \frac{P}{P_c + P_{sc} + P_{st}} \qquad \text{Eq.(19)}$$

6.3 Stress in reinforcement steel bars:

The stresses in reinforcement steel bars can be found from the Eq.(20a) and Eq.(20b)

$$\sigma_{sc} = \frac{\sigma_c}{(a-y_n)}\frac{420}{f_{ck}}(y_{si} - y_n) \text{ if } y_{si} \geq y_n \qquad \text{...Eq.(20a)}$$

$$\sigma_{st} = \frac{\sigma_c}{(a-y_n)}\frac{280}{f_{ck}}(y_{si} - y_n) \text{ if } y_{si} < y_n \qquad \text{...Eq.(20b)}$$

7.0 Numerical example for cracked column section using IRS concrete bridge code:

The example problem considered in section 5 and details of column shown in Fig.2 are used for this analysis of stresses as per IRS concrete bridge code also for the purpose of comparison of results.

7.1 Stresses for the total loads:

The distance of neutral axis is calculated as -148.00mm for the total load using Eq.(18a).The stresses in concrete and reinforcement steel bars can be calculated using Eq.(19), Eq.(20a) and Eq.(20b). The calculated values are given below.

$\sigma_{cTotal} = -16.75MPa$; $\sigma_{scTotal} = -148.03MPa$

$\sigma_{stTotal} = 84.19MPa$

7.2 Stresses for the permanent loads:

The neutral axis for the permanent load is found as -148.00mm. The stresses due to permanent loads alone are

$\sigma_{cPerm} = -12.56MPa$; $\sigma_{scPerm} = -111.02MPa$

$\sigma_{stPerm} = 63.14MPa$

7.3 Stresses due to transient loads:

The difference between the stresses due to total loads and the permanent loads are equal to the stresses caused by the transient loads. The values of the stresses due to transient loads are

$\sigma_{cTrans} = -4.19MPa$; $\sigma_{scTrans} = -37.01MPa$

$\sigma_{stTrans} = 21.05MPa$

8 Comparison of Results for Cracked Circular Column and Discussion:

The calculated stresses in concrete and reinforcement steel bars using the Road and Railway Bridge Codes are tabulated in table 1 given below. The ratio of stresses obtained from IRS concrete bridge code to the stresses obtained by IRC:112-2011 are given in column 4 of the above table. From the above ratio, it can be seen that the IRS concrete bridge code underestimates the stress in concrete by 4% at the initial stage of loading and over estimates the long term stress by 6%. The stresses in compression reinforcement steel bars are overestimated by 104% by the railway bridge code at initial stage of loading. But, the long term stresses in the above reinforcement bars are under estimated by 43% by the IRS concrete bridge code. The stress in tensile reinforcement steel is over estimated by 19% in initial loading and it is under estimated by 33% in long term loading. Since the crack width is dependent on the tensile stress in the reinforcement bars, the above under estimation of tensile stress will in turn lead to lesser crack width affecting the serviceability of the RC circular columns.

From the above study, it is clear that the time dependent stresses in concrete have not been properly addressed in the IRS concrete bridge code.

Table -1 Comparison of stresses

Stress (1)	IRC:112-2011 (MPa) (2)	IRS concrete bridge code (MPa) (3)	Ratio of stresses (4) = (3)/(2)
Total loads at initial loading stage			
σ_c	-17.50	-16.75	0.96
σ_{sc}	-72.69	-148.03	2.04
σ_{st}	70.86	84.19	1.19
Permanent loads at initial loading stage			
σ_c	-13.12	-12.56	0.96
σ_{sc}	-54.52	-111.02	2.04
σ_{st}	53.15	63.14	1.19
Transient loads at all loading stages			
σ_c	-4.37	-4.19	0.96
σ_{sc}	-18.17	-37.01	2.04
σ_{st}	17.72	21.05	1.19
Creep and shrinkage or time dep. stresses			
$\Delta\sigma_c$	1.66	-	-
$\Delta\sigma_{sc}$	-186.06	-	-
$\Delta\sigma_{st}$	38.79	-	-
Total stresses + time dependent stresses			
σ_c	-15.84	-16.75	1.06
σ_{sc}	-258.75	-148.03	0.57
σ_{st}	109.65	84.19	0.77

9 Scope for the Future Work:

Hollow RC circular columns are also used in railways if the heights of the columns are high. Hence, it becomes necessary to formulate the equations for the analysis of time dependent stresses for cracked hollow circular columns. Even though, uncracked circular and hollow circular columns are not common in railway bridges, they can also given a separate study for the evaluation of time dependent stresses using slightly a different approach.

10 Conclusion:

The following conclusions are made from the above study for the time dependent stresses in RC cracked circular columns by comparison of predictions of IRS concrete bridge code with IRC:112-2011 which is used for the design of road bridges.

1). The bending compressive stresses in concrete for RC circular columns are slightly under estimated at the initial loading time ie., before creep and shrinkage strains are developed in concrete by the IRS concrete bridge code.

2). On the other hand, the bending compressive stresses in concrete for the above columns are over estimated after a prolonged period of time ie., after creep and shrinkage strains are developed in concrete by the concrete bridge code of railways.

3). The compressive stresses in reinforcement steel bars in the RC circular columns are over estimated by more than 100% at the time of initial loading when the calculations are made using IRS concrete bridge code.

4). On contrary to the above, the compressive stresses in reinforcement steel bars in the RC circular columns are largely under estimated after a long period of time when IRS concrete bridge code is used for the prediction of stresses.

5) The IRS concrete bridge code, over estimated the tensile stress by 19% in reinforcement steel bars at the initial loading time. But the above code under estimated the tensile stress by 33% after the creep and shrinkage strains are developed in concrete.

6). Since there are large variations in stresses in concrete and reinforcement steel bars while considering the time dependent properties of concrete for the design of RC circular columns, it is required to reconsider the provisions of empirically based formula for considering the long term effect in IRS concrete bridge code.

11 Notations:

A_c =Area of concrete section in compression zone alone.

A_s =Cross sectional area of reinforcement steel bars.

A_{si} =Cross sectional area of i[th] reinforcement steel bars.

$A_{tr} = A_c + \alpha A_s$ =Transformed area of column section.

$\bar{A}_{tr} = A_c + \bar{\alpha} A_s$ =Effective transformed area of column section.

$a = D/2$ =Radius of RC circular column.

B_c =First moment of area of concrete section in compression zone alone.

B_{tr} =First moment of transformed area of column section.

$\bar{B}_{tr}$ =Effective first moment of transformed area of column section.

D =Diameter of RC circular column.

$E_c(t_0) = E_c$ =Elastic modulus of concrete at initial loading time

$\bar{E}_c(t,t_0) = \bar{E}_c = \dfrac{E_c}{\left[1+\varphi(t,t_0)\right]}$ =Effective modulus of concrete at time, t

E_s =Elastic modulus of reinforcement steel bars.

f_{ck} =Characteristic compressive strength of concrete.

I_c =Second moment of area of concrete section in compression zone alone.

I_{tr} =Second moment of transformed area of column section.

$\bar{I}_{tr}$ =Effective second moment of transformed area of column section.

M =External uniaxial bending moment acting on the axis through the mid-depth of column.

M_x, and M_y =Uniaxial bending moment acting about X-axis and Y-axis respectively.

M_r =Resultant bending moment

M_c =Bending resisting force offered by concrete alone.

M_s =Bending resistance offered by steel rebars.

M_{sc} =Bending resisting force offered by reinforcement steel in compression zone.

M_{st} =Bending resisting force offered by reinforcement steel in tensile zone.

ns =Number of reinforcement steel bars.

P =External axial load acting on column.

P_c =Axial resisting force offered by concrete alone.

P_s =Axial resisting force offered by steel rebars.

P_{sc} =Axial resisting force offered by reinforcement steel in compression zone.

P_{st} =Axial resisting force offered by reinforcement steel in tensile zone.

t =Time elapsed after Initial loading.

t_0 =Initial loading time.

y =Distance of a point under consideration from the reference axis

y_n =Depth of neutral axis from the reference axis.

y_{si} =Distance i[th] reinforcement steel bar from the reference axis.

$\alpha = \dfrac{E_s}{E_c}$ =Elastic modular ratio for concrete

$\bar{\alpha} = \frac{E_s}{\bar{E}_c}$ =Effective modular ratio for steel reinforcement bars at time, t

ε =Strain at any point under consideration.

ε_{cs} =Shrinkage strain at time, t.

$\varepsilon_0(t_0) = \varepsilon_0$ =Strain at the level of reference axis at initial loading time.

$\varepsilon_0(t_0)_{Total}$, and $\varepsilon_0(t_0)_{Perm}$ =Strain at the level of reference axis at initial loading time due to total loads and permanent loads respectively.

ε_{crp} =Creep strain at time, t.

$-\Delta M = -(\Delta M_{crp} + \Delta M_{shr})$ =Sum of the restraining bending moments to the creep and shrinkage acting in reverse direction at the reference point.

ΔM_{crp}, and ΔM_{shr} =Restraining bending moment to the creep and shrinkage respectively acting at the reference point.

$-\Delta P = -(\Delta P_{crp} + \Delta P_{shr})$ =Sum of the restraining normal forces to the creep and shrinkage acting in reverse direction at the reference point.

ΔP_{crp}, and ΔP_{shr} =Restraining normal force to the creep and shrinkage respectively acting at the reference point.

$\Delta\varepsilon_0(t, t_0)$ =Additional strain at the level of reference axis at time, t

$\Delta\sigma_c$ =Change in stress in concrete due to the creep and shrinkage.

$\Delta\sigma_{sc}$ =Change in controlling stress in compression reinforcement steel bars due to the creep and shrinkage.

$\Delta\sigma_{st}$ =Change in controlling stress in tensile reinforcement steel bars due to the creep and shrinkage.

$\Delta\sigma_{res}$ =Restraining stress in concrete to the creep and shrinkage.

$\Delta\psi(t, t_0)$ =Additional curvature at the level of reference axis at time, t

$\varphi(t, t_0) = \varphi = \frac{\varepsilon_e + \varepsilon_{crp}}{\varepsilon_e}$ =Creep coefficient.

σ_c =Compressive stress at the level of reference axis.

σ_{sc} =Controlling compressive stress in reinforcement steel bars.

σ_{st} =Controlling tensile stress in reinforcement steel bars.

σ_{cTotal}, σ_{cPerm}, and σ_{cTrans} =Stress at the level of reference axis at initial loading time due to total loads, permanent loads and transient loads respectively.

$\sigma_{scTotal}$, σ_{scPerm}, and $\sigma_{scTrans}$ =Controlling stress in compression steel reinforcement bar at initial loading time due to total loads, permanent loads and transient loads respectively.

$\sigma_{stTotal}$, σ_{stPerm}, and $\sigma_{stTrans}$ =Controlling stress in tensile steel reinforcement bar at initial loading time due to total loads, permanent loads and transient loads respectively.

$\psi(t_0) = \psi$ =Strain gradient or curvature at the level of reference axis at initial loading time

12 References:

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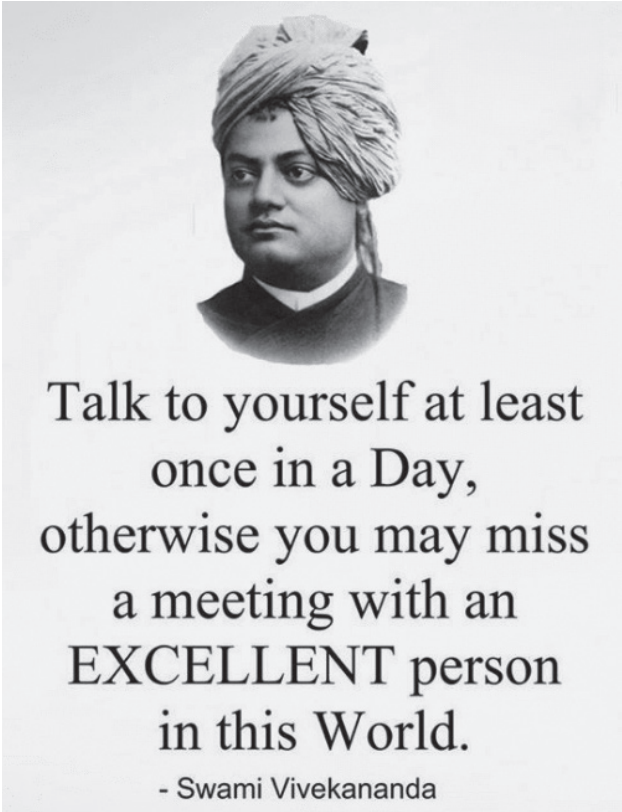

A Comprehensive Approach for Damage Assessment & Repair of Concrete Structures

By
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Job of a repair rehabilitation engineer can be compared to that of a physician. Both of them are presented with some symptom/distress/abnormality and they are expected to find the reasons behind it. Not only that, they are also expected to prescribe the cure/medicine.

The job thus needs inspection, testing, drawing inference form the results and prescribing solutions based on logical deductions using domain knowledge. This paper attempts to provide a step-by-step approach towards understanding the distress of concrete structures using different testing methods.

1.0 Introduction:

Reason for damages in concrete structure can be either due to aging or it can take place prematurely due to human error, environmental reasons or accidents. Accidental damages are mainly due to cyclone, tsunami, earthquake etc which are natural calamities or even due to fire. Using inferior quality of material, improper design or workmanship can lead to distress in structure. Often concrete structures are exposed to corrosive environment or subjected to chloride or sulphate attack. In case proper protection is not envisaged in design or construction phase, it may lead to permanent damage.

Normally concrete structures exhibit distress in the form of cracking, spalling, honeycombing, leakage etc. Defects in freshly placed concrete may be due to excessive bleeding, shrinkage and plastic settlement etc whereas that in new concrete (already set) is due to honeycombing, delamination or drying shrinkage. It is worthwhile to mention here that minor surface crack in concrete is harmless but excessive appearance of cracks can be detrimental. Moreover, too many cracks pose a serious doubt about the safety of a structure in the mind of the user who is more often than not are non-technical persons.

Cracks in old concrete can be due to sulphate attack, chloride attack, bursting pressure caused due to corrosion of reinforcement or settlement of structure and damage in joints. Loss of protective layer due to prolonged abrasion especially prevalent in bridge structures can also expose concrete to aggressive environment and thus cause distress. Continuous water ingress through a minor crack may lead to major structural problem if not tackled in the initial stage.

2.0 Preliminary Investigation/Inspection:

In most of the cases, the repair of concrete structures is carried out either in haste or without going into the details of the reason of damage. Application of preconceived repair methodologies or use of materials without understanding its properties is also very common. Indiscriminate use of epoxy products bears a testimony to the above.

A good repair scheme needs to start from the extent and probable reasons for damage. Assessing probable reasons for damage of structures start with thorough visual inspection. It needs no equipment but the eyes of the inspecting engineers. Inspecting engineers should look for cracks, it extent, location, dimension, spalling, rusting marks, disintegration, honeycombing and other relevant defects. Use of crack microscope, light hammer, magnifying glass, bore scope, endoscope etc. could be of immense help. Photographs may prove very useful in such inspections. Minor cracks or exposed reinforcements which may be missed out in naked eye can be discernible when good photographs are taken.

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Study of available drawings, its design needs to be done in order to find out any design deficiency inherently present in the structure. Analysis of such structure with present loading may point towards some design deficiency though that needs to be ensured with the application of other techniques.

Studying the present exposure condition of the structure is also important. The exposure of structure to corrosive or aggressive environment may accelerate the process of distress in the structure. Often structures are exposed to such a condition that was not envisaged during design or construction. This is mainly true for public structures where enforcement of rules and regulations leaves much than desired. Putting utility lines like electric, telephone or other types of cables, water and gas pipelines below or alongside the footpath without caring for the permission of bridge maintenance agency is extremely common and attention to these matters are drawn only when some accident happens in the bridge. Overloading in these locations can lead to early distress of the concrete structures.

The purpose of visual inspection and study of documents is to identify the root causes(s) of distress in the structure. It may not be possible to be sure about the reason but at least it would give some idea to the experienced engineers. That idea may well be the starting point of detailed investigation for finding the actual reasons of distress.

In spite of all its advantages, visual inspection suffer from the disadvantage that it is highly dependent on the skill and experience of the investigating engineer. No internal crack or defect can be detected by visual inspection.

The above shortcomings no way undermine the importance of visual inspection in identifying the assessment of damage of a structure. Non-destructive tests (NDT) or Semi Non-destructive tests (semi NDTs) could supplement visual inspection of the structure in finding the reason for damage.

3.0 Details of NDT methods:

NDT methods are based on certain mechanical or engineering properties of the material being tested. Most of the NDT methods owe their origin to the testing of metals which are homogenous in nature. Since concrete is highly non-homogeneous in nature by virtue of its method of production, NDT tests in concrete is not that reliable as it is in the case of metals. Therefore, single test result of NDT in case of concrete structure can often be misleading and needs to be confirmed by some other methods of testing. NDT in concrete can be divided in the following based on the methods used:

a) Surface Hardness Methods

b) Pulse Velocity Methods

c) Semi Destructive Methods

d) Other Methods based on Impact/Impulse etc.

3.1 Surface Hardness Methods:

Surface hardness methods are based on the fact that as the time passes, hydration reaction progresses in concrete and its strength and hardness increases. If the surface of concrete is impacted with known value energy, either it would create indentation on the surface or the mass will be rebound back. If indentation or rebound is measured and a correlation is developed between indentation/ rebound and strength, strength can be predicted. There are many tests which have been developed with the above principle like Frank Spring Hammer, Einbeck Pendulum Hammer but the most common one is Rebound Hammer or Schimdt Hammer Test.

Rebound Hammer Test works on the principle that a spring loaded mass strikes the concrete surface and its rebound value is recorded. Manufacturers normally supply a number of correlation curve and from that the strength of concrete could be predicted.

However, rebound value is likely to be affected by type of materials used, presence of coarse aggregate very near the surface, smoothness of surface, size and shape of specimen and carbonation of concrete surface. Owing to the above, even after calibration, the result of rebound hammer is likely to have a variation of 20%- 25% with the actual cube strength. Therefore, rebound hammer test can no way substitute cube test but can be used to confirm its result or to develop suitable correlation. In case of a structure in service but experiencing some distress, this can be used to predict the strength of concrete after many years of production.

3.2 Pulse Velocity Methods:

In this method, an ultrasonic wave passes through the concrete by setting the pulse generator on a point of the structure. The time of travel of wave is measured on some other surface having a known distance between them. Thus the velocity of wave pulse is calculated.

The velocity of a pulse of ultrasonic compression waves (P-wave) depends on dynamic modulus of elasticity, dynamic Poisson's ratio and density of the material. Thus it completely depends on material properties. It is believed that the denser is the concrete; higher will be the values of modulus of elasticity or Poisson's ratio. Thus in a dense concrete, the pulse velocity would be higher and the waves would take less time to travel between two points at a known distance apart. As the manufacturers provide correlation curve between strength and pulse velocity, strength of concrete can be predicted.

Often the opposite surfaces of the structures are not accessible. In those cases, the sensor is required to be placed on some other surface leading to measurement of pulse velocity through indirect methods. Moreover, pulse velocity is known to be affected by water cement ratio, size and grading of aggregate and age and temperature of concrete. Pulse velocity also increases when the pulse velocity passes through reinforcement steel. For the above reasons, predicting concrete strength solely based on pulse velocity results is not advisable.

3.3 Semi Destructive Methods:

There are other test methods which predict concrete strength by partially damaging the structure though the extent is very negligible. From that point of view, they are not exactly non-destructive testing but may be semi-destructive tests. Such methods are of two types:

a) Pull-out Test

b) Penetration Test

c) Core Test

3.3.1 Pull-out Test:

The pull out test is meant to measure the force needed to extract a 25mm diameter metal piece (or insert) that has either been embedded during casting or installed afterwards at a depth of 25mm against a counter force having 55mm diameter. Based on several experimental and theoretical analyses, a correlation has been obtained between ultimate pull-out force and the compressive strength. Moreover, it has been found that pull-out force is not much affected by type of cement, aggregate or admixture, water-cement ratio or age of concrete. It is known to be affected by the size of aggregate when the maximum size of aggregate is greater than 38mm.

The above advantages have made the pull out test as a most reliable and preferred NDT but this also have the disadvantage that an elaborate arrangement is required to be made at site. Moreover, technicians need to be trained properly for conducting this test.

3.3.2 Penetration resistance:

A probe of known diameter and length is pushed into the concrete with application of a known amount of energy. The penetration will be inversely proportional to the hardness of concrete and once the exposed length of the probe is measured, compressive strength of concrete can be found based on the available correlation curves. It is known to be affected with hardness, type and size of aggregate and age of concrete.

The test also has the following disadvantages:

- Test location should be at least 200mm away from the edge of concrete.
- Test cannot be conducted in concrete strength more than 40 MPa
- Reinforcement spacing is required to be more than 100mm
- Initial cost of equipment is too high.

- In spite of correlation curve provided by the manufacturer, a separate correlation curve needs to be developed for each site. It is not only time consuming but also adds to the cost.

The above disadvantages deter this test from becoming very popular for site application.

3.3.3 Core Test:

Concrete core test is the most common semi destructive test of concrete because of its ease of sampling and testing. It gives the exact value of compressive strength of structure.

In this method, cylindrical cores are taken from the affected part of the structure by core cutting machine. The core is usually 50 mm in diameter. Neat cement and sulphur capping is done at the end faces. Usually 3 specimens are tested under compression. The compressive strength thus found is multiplied by a correction factor based on h:d ratio of the sample. Correction factor can be read from IS: 516 (figure 1). However, if maximum size of aggregate is more than 38 mm, this test is not recommended.

3.4 Other Methods based on Impact/Impulse etc.

The following techniques are relatively new and less popular at present:

3.4.1 Impact Echo Method:

Short duration mechanical impact is created with the help of steel balls on the surface of the concrete. It produces wave that reflects back from the external boundaries as well as internal defects. When these multiple responses return to the surface, it is picked up by displacement transducers.

On analysing such response as well as with the help of given constants provided by machine manufacturer, thickness of the member as well as location of the rack could be determined.

3.4.2 Impulse Response Method:

A rubber tipped hammer is used to generate an impulse and a transducer is placed near to it. Both the hammer and the transducer are linked to the on-field computer, which analyses the response. The noted response is matched with the pre-defined pattern indicating the quality of concrete.

3.4.3 3D Tomography:

Concrete structures having depth of 2m or more can be analysed with the help of this technique. This can produce 3D image with the help of SAFT (Synthetic Aperture Focussing Technique).

3.4.4 Ground Penetrating Radar:

High duration and short frequency microwave is generated by this method. When there is dissimilar material wave is reflected back from the boundary. Response is again analysed with the help of software to find out the defects in concrete.

4.0 Short-listing of Available Repair Options:

Once the reason for damage in a structure is established, engineers need to choose among the available repair options. Engineers need to understand that with the ongoing use of structure, how long the structure could serve without failure once repair is undertaken. Engineers need to determine which of the repair schemes could give likely performance in long term at a given cost and future repair cycles. The future appearance of the structure after repair is completed also needs to be considered in great detail and discussed with the client/owner.

5.0 Choosing Correct Repair Methodology:

Based on the above, engineers need to choose the repair method. Methods should be such that it remains compatible with the further service of the structure. Needless to mention that the appropriateness of such method would be evaluated in the success it provides against the identified reason and extent of damages.

Choosing correct repair material is often neglected but the most important step in the complete process. Engineers using the repair materials should be well aware of the properties of the repair material in the given exposure and service condition. No material in this world could have all the desired properties and hence more often than not engineers are compelled to use a combination of materials. Their inter-compatibility in service condition requires to be studied in great detail. If required, suitable tests may also be carried out in the laboratory to ensure that the desired properties are really available in the chosen repair material or in the planned combination of material. Advertising and part technical specifications provided by the manufacturers may not be reliable for all cases.

Construction chemicals like Epoxy, Acrylic, SBR etc. are being used in the repair application frequently. Unfortunately study of their property in the exposure conditions of our country are not being done with due diligence. This results in failure of many repair schemes. Number of laboratories equipped with testing of properties of construction chemicals is not widely available; their availability is especially limited in the metro or tier II cities of the country making it difficult for the engineers to test its properties even when they feel its necessity. Most of the repair jobs are carried out in haste and is essentially a low cost project. Setting up of laboratory for testing of materials at site is often practically not possible.

6.0 Documentation:

Documentation of repair jobs needs to be treated with equal importance as done in projects. All requirements of a project need to be followed in case of repair works also. All such documents need to be compiled and preserved for future reference.

Original construction drawings here serve as the basic document for measuring distress. The records of inspections- visual, NDT or semi NDT needs to be kept with sketches for future reference. Photographs taken during the inspection are some invaluable documents which could be used in future. The details of the options available and the option chosen should be documented in great detail so that if similar distress reappears, the then engineers could take some clue from it. Intended properties of repair materials, their sampling as per codes and their testing reports (in fresh or hardened stage) need to be kept on record.

Preparation of contract documents needs to be done with great care. Special conditions of contract, performance criteria and BOQ have to be prepared with due application of mind and considering all possible situations that can be encountered at site as they vary greatly from work to work and site to site. Normal techniques of quantity surveying may not be applicable to repair jobs. Items like dismantling of weak concrete, grouting, guniting etc. depends on the exact condition of the structure and to some extent on individual judgement. This makes exact estimate of the repair quantities extremely difficult. Strict and regular supervision is hence must for such kind of jobs. Needless to mention, site records and payment records have their own importance in this case also.

Finished photographs and future observations should be added to the file in a chronological manner. In this way, even smaller distress can be detected by comparing with the immediate post-repair data.

7.0 Conclusion:

A structure in service develops some distress during its lifetime. Often the users notice it late and try to manage the some unskilled labours and/or by deploying primitive techniques. Users approach repair professionals quite late.

The job of a repair and rehabilitation engineer starts from identifying the reason for distress and complete after the suggested repair scheme is executed. The job can be compared to a physician who identifies the reason of illness based on symptoms and test results and prescribe appropriate medication.

8.0 Reference:

1. Indian Railway Bridge Manual-1998
2. IS: 13311 Part 1- 1992: Non-destructive Testing of Concrete- Method of Test
3. IS: 13311 Part 2- 1992: Non-destructive Testing of Concrete- Method of Test
4. ASTM C900-15: Standard Test Method for Pullout Test of Hardened Concrete
5. ASTM C803: Penetration Resistance of Hardened Concrete
6. IS: 516-1959: Methods of Test for Strength of Concrete
7. A Step Wise Approach To Non-Destructive Testing of Reinforced Concrete Structure by Parampreet Singh, Civil Engineering & Construction Review, April, 2008.

Rationalising the Index Section of Projects For Economical and Faster Execution

By
Neeraj Jain*

Abstract:

The policy of not allowing unmanned level crossings on new projects is having a huge impact on the project timelines and costs. making by and large most projects unviable. There is a need to redefine some basics like Ruling Gradients, consider road underpasses depressed below ground level. This paper addresses these issues and suggests mitigating solutions.

1.0: Statement of the problem.

1.1 A need for doing away with level crossings for all now Rail line constructions has had a huge impact on the project cost. The bank profiles being raised to accommodate the Road underpasses leading to huge increase in quantum of earthwork as also land for the project. Earlier at many LC locatins where a bank height of 1 to 1.5 metre may have been acceptable, now at corresponding Road Under Bridge locations the banks are being raised to maybe a height of 4 metres or more even for village/minor roads.

1.2 Fig. 1 graphically shows how rapidly the volume of the bank and also the toe to toe width of the bank increases with increased height of bank.

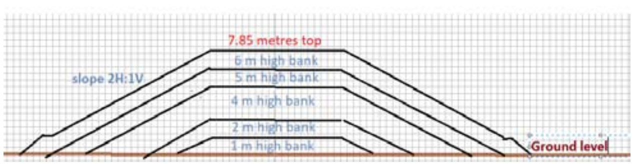


Fig. 1: Cross sections of bank with increasing height

Height of bank in metres	Vol. of bank in m3/m run	Toe to toe width in metres	Width of land acquisition in Metres
1	9.9	11.85	18.85
1.5	16.3	13.85	21.35
2	23.7	15.85	23.85
3	41.6	19.85	31.85
4	63.4	23.85	37.85
5	89.3	27.85	43.85
6	120.1	32.85	50.85
7	155	36.85	56.85

Table 1: Width of land acquisition and volume of bank for various height of bank for single line BG track.

1.3 While the land width from toe to Toe of bank increases by about 4 metres for every metre increase in bank height. Height beyond 5m may need an extra 1 metre for berm to provide bank stability. Even where there is no provision kept of borrow pits for bank construction, the Engineering code stipulates a land width beyond the toe depending on the bank height. Land has become very costly to acquire ever since the passage of the Right to Fair Compensation and Transparency in Land Acquisition, Rehabilitation and Resettlement Act, 2013. As a consequence of this act the quantum of compensation for rural agricultural land has gone up 2 to 3 times. The cost of land alone is likely to badly affect the project financial viability.

1.4 The earthwork is a time consuming process, particularly when the quantities are huge, apart from being the major component of the Civil Engineering costs of the project. A large quantum of earthwork also negatively impacts the local environment as a lot of farms/ fields may be destroyed.

2.0 Mitigating solutions suggested

Two suggestions to mitigate the problem of increasing costs and time due to large quantum

* IRSE/Retired

of earthwork and land necessitated by need to underpasses are suggested.

2.1 The first suggestion is to plan for 'sunken' subways ie subways whose floor level may go below the surrounding ground level. While this concept has been accepted and adopted for elimination of existing level crossings by ROB's, but it has not found takers for new construction, the argument being that while it is unavoidable in case of existing low banks since raising operational lines may not be feasible, however in new construction there is no constraint in raising. This leads to great increase of the banks. Of course, sunken RUBs may lead to problem of water accumulation in the RUB box. This problem can be easily addressed by providing a sump nearby where the water drains by gravity, and can be pumped at leisure, possibly by solar power. By providing a drainage sump, the apprehension of flooding of RUB due to simultaneous heavy rain and failure of pumping arrangement can be avoided.

2.2 The more radical solution being suggested here is to review the whole concept of 'Ruling Gradient' as we currently understand it. The Ruling grade is the steepest gradient on the section (duly adjusted for curves if curves and grades are concurrent in any stretch).

2.2.1 The ruling gradient is specified under the terms of reference for a project considering the contiguous sections, the mode of traction, the type of trains to run. The ruling grade is important from considerations of required powering on train, as well as consideration of braking distances. Typically, in plain sections, we are adopting a ruling gradient of 1 in 200, whereas in hill stations ruling gradients of 1 in 100 or even steeper may not be uncommon.

2.2.2 It is contended that the present concept of ruling gradient as we take into consideration is scientifically faulty. We have considered the ruling grade as the steepest of the spot longitudinal gradient. The hauling power and the braking distance are not a function of the spot gradient at any one point but rather the steepest weighted average gradient as experienced by the full length of the train. The concept of Ruling Gradient as we understand was good enough for all practical purposes in earlier times since the train lengths were short. As progressively the train lengths are being increased, the 'spot' gradient ceases to be of substantial relevance.

2.2.3 It has to be recognised that the traction effort required to haul the train is not per se the grade encountered by the leading locomotive, but the rate by which the Centre of Gravity of the complete train is being lifted for every unit of longitudinal movement of the train.

2.2.3.1 In the conceptual diagram, let the train of length L be having parts of it running on different grades. Then the Effective gradient at that instant which determines the ability of the loco to haul/accelerate/brake the train is determined by the red line AD, which is the weighted average of the grades being traversed by different portions of the train.

We can define the term EFFECTIVE GRADE (EG) as =(G1*AB+G2*BC+G3*CD)/L

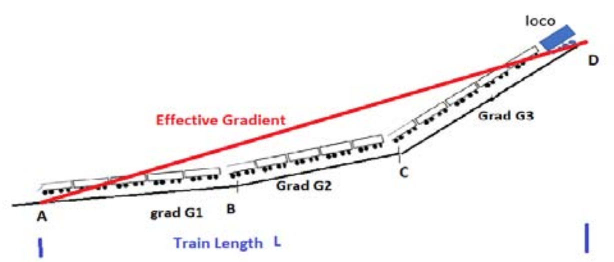


Fig 2: Explaining the concept of Effective Grade

2.2.4 It is the Effective Grade as defined above which determines the negotiability of a section by a train with a certain powering. It is suggested that we re-designate what we currently call the 'Ruling Gradient' as the 'Effective Gradient' thereby allowing steeper grades to prevail for short lengths, but ensuring that the Effective Grade over the least powered/tonne of trailing load train length is adequate for hauling as also braking within a specified distance.

2.2.4.1 Using the above concept for designing the Index Section or the vertical profile of the track would lead to the bank tops closely following the ground profile, thereby reducing the quantum of earthwork and also the project land requirements.

2.2.4.2 Other spinoff benefits would be faster construction, less maintenance cost for both track as well as bank due to shallower banks, reduced cost of RUB's due to reduction in barrel length, shorter ROB's because to shortened approach due to lower banks. Also, with lower banks, it may be possible to avoid shifting /raising of High tension Electric crossings which are crossing the alignment and violating the vertical height norms thereby avoiding the costs, effort and time involved in these activities.

2.2.5 Some apprehensions for such an approach are natural, firstly this concept may involve a greater number of change of grades, thereby leading to greater potential of train separation since it is believed that the change of grade is a factor in train separation. This apprehension if justified could be mitigated by larger radius vertical curves. Also this may lead to higher coupling forces. The longitudinal forces in a train are a complex phenomenon, hence while the longitudinal forces averaged over time would remain unchanged, instantaneous forces envelope may change, leading to changed fatigue characteristics of couplings particularly. This can be assessed only by some trials and simulation studies.

2.2.5.1 The present norms for vertical curves for Broad Gauge as per the LWR manual put the lower limit to Radius of vertical curves to 4000 metres for A routes, 3000 metres for B routes and 2500 metres C, D and E routes.

3.0 Delving into LWR technical

3.1 The LWR manual limits the steepest gradient of LWR to 1 in 100. This may put a bit of constraint on what is being suggested, because the spot gradient may then have to be limited to 1 in 100.

Calculation of factor of safety for lifting forces at Sag / hump under extreme conditions

LH gradient absolute	gl	0.02	% ie, I in		50					
RH gradient absolute	gr	0.02	% ie. 1 in		50					
Change of grade	0.04									
Let Rail temperature zone		IV			range of rail temp		76	max range in India as per LWR manual		
Change from destressing temp	40		degrees centigrade				mean of temperature range		38	
Rail section	60	kg/m					range of destressing temp = Tmin+		43	48
Steel density	7.8	gms/cc					Max temperature range for tension			48
Cross section area=	76.9	cm2					max temp range for compression			33
Coeff of thermal exp		1.10E-05		per degree centigrade						
Young modulus		2.10E+06		KG/cm2						
Compressive force on 1 rail		5.86E+04		Kg	58.64	tonnes	on two rails compressive		117.3	Tonnes
Tensile force on I rail		8.53E+04		Kg	85.29	tonnes	on two rails tensile		170.6	Tonnes
Total difference of grade		0.04	%							
Central angle of vertical		0.04	radians							

curve					
Radius of vertical curve	3000	m			
Length of vertical curve	120	metres			
Vertical component of force both rails from Both sides of sag or hump	4.69	tonnes	comp. lifting at humps	6.82	tensile lifting at at sags
Weight of each sleeper and fitting	250	Kg			
Weight/metre of track	535	Kg			
Weight of vertical curve track	64.2	tonnes			
Factor of safety	**13.7**	**compressive hump**	**at**	**9.4**	**tensile at sag**

Table 2: Calculation of factor of safety against lifting of track due to thermal stresses

This seemingly artificial constraint may also be looked into ab-initio. the presumable reason for imposing this restriction is that the track may tend to lift at the vertical curves at sags or humps due to thermal related rail tension/compression respectively.

A simple calculation shows that with a 3000 metre vertical curve, with most adverse temperature conditions, the factor of safety against lifting is of the order of 13.7 and 9.4 give a lot of assurance as the calculations in table 2 show.

Factor of Safety against track lifting on sags and humps for different radius of Vertical curves.				
	Steepest grade 0.2% or 1 in 50	Radius of vertical curve		
		2000	3000	4000
Factor of safety	At hump	9.1	13.7	18.2
	At Sag	6.3	9.4	12.5

Table 3: Factor of safety for different vertical curve radius with 1 in 50 track gradient

3.2 For a 1 in 50 gradient on either side of a sag/hump, the factor of safety against lifting are indicated in the table 3 gives a high level of assurance:

4.0 Quantifying the benefits

To get a quantitative feel of savings by adopting the concept of Effective Grade and sunken RUB, let us assume a case where there are two RUB's spaced 1 km apart on level terrain. The height at RUB is 3.5 m hence the bank height at ROB location is 4 metres. There are no intermediate vertical obligatory points like bridges. The numbers for three scenarios are worked out where the Ruling Gradient are kept at 1 in 200, 1 in 100 and 1in 50 respectively Scenarios A, B and C). In the scenarios D and E, the 3.5 metre high RUB is sunk 1 metre below ground, and hence the Bank height at RUB location is taken as 3 metres and a ruling gradient of 1 in 100 and 1 in 50 respectively assumed. It is assumed there are no intermediate vertical fixed points. The land acquired is for as prescribed in the Engineering Code with no provision for borrow pits. Vertical curves are assumed to be of 3000 metre radius. The minimum height of the bank allowed has been assumed to be 1 metre.

Scenario	A	B	C	D	E
	3.5 metre Underpass with I in 200 ruling grade	**3.5 metre Underpass with I in 100 ruling grade**	**3.5 metre underpass with I in 50 ruling grade**	**3.5 m Underpass sunken by 1 metre with I in 100 ruling grade**	**3.5 m Underpass sunken by 1 metre with I in 50 ruling grade**
Grade %	0.005	0.01	0.02	0.01	0.02
Volume of bank for I Km in cum	19536	15023	8579	7391	6969
Land area for 1 Km as per Engg code in m2	28181	25558	23611	22978	21512
Quantum of bank reduction in m3	0%	23.1%	56.1%	62.2%	64.3%
Land reduction %	0.0%	9.3%	16.2%	18.5%	23.7%
Effective gradient 750 m long train%	0.0016	0.0016	0.0040	0.0027	0.0027
Effective gradient represented as 1 in	619	625	250	375	375

Table 4 quantities of earthwork and land for various scenario

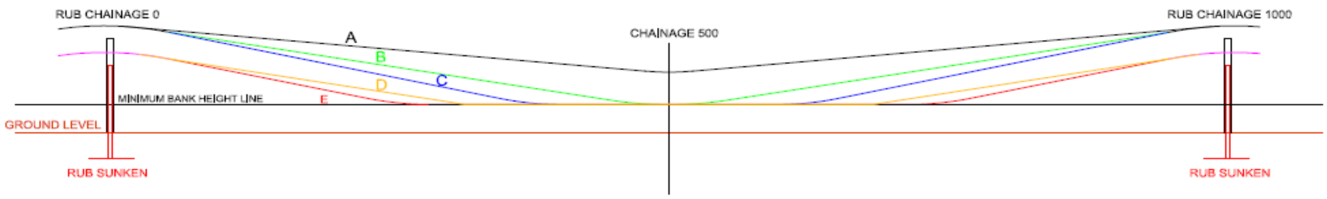


Fig 3 : Vertical profile for different scenario.

4.1 For computation of land area, the land width has been assumed to be as per Engineering code. Bank slopes assume as 2h:1v, bank top width 7.85m, width of land acquisition up to 2.7 metres high banks of height H = 5 *H+13.85, and 6 *H+13.85 for bank height beyond 2.7 metres.

4.2 As can be seen from table 4, there are substantial reductions achievable in both quantum of land as also volume of the banks (i.e. volume of Earthwork+ volume of Blanketing) if we adopt the concept of sunken underpass.

5.0 Qualifying the benefits

The savings would be still more dramatic if both concepts of sunken underpass, and 1 in 50 ruling gradient were to be adopted simultaneously. Also, savings would be higher if the radius of the vertical curve were to be reduced say to 2000 metres and further higher if the height of subway were required to be higher based on road classification.

5.1 Lest there be a misconception created that the concept being proposed is valid only for relatively flat terrains, on the contrary the concept can be more advantageous in graded terrain, where advantage is mass balancing (ie balancing cutting and filling earthwork) could lead to even more dramatic results in reduction in borrowed earth, in total quantum of earthwork as also in quantum of land required, as the sketch at Fig. 4 showing a conceptual index section would make readily apparent.

earthworks going into fillings.

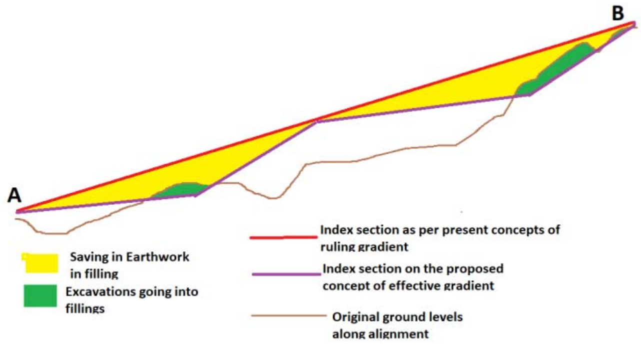


Fig 4: a conceptual sketch of savings in a continuously graded ground by adoption of proposed concept.

5.2 If the line has to go from point A to point B, as per our current concept of 'Ruling Gradient' the track would follow the red profile. However, if the concept of Ruling Gradient is replaced by an Effective Gradient, the track could follow the purple profile, greatly reducing the bank heights, and allowing a lot of balancing of earthwork by cuttings Discussion with various people have met with the whole spectrum ranging from enthusiasm to scepticism. Have tried to address the various issues raised :

5.3 However considering the complex nature of train dynamics, simulation/ trials may need to be carried out for the effect on stresses in couplings etc.

6.0 Some questions/ concerns addressed

Various colleagues have raised some issues during discussions. These are addressed here.

6.1 Q: What if the train stops, with the engine at steepest grade? Will it be able to start?

6.1.1 Ans: When the train is stationary, the ability to start will largely depend upon the force on the couplings. This force is a function of the weighted grade on which the train is standing, the weightage being the load (payload + tare) of the vehicles at each grade. Hence it is the same if the whole train is on one grade, or on different grades, provided the difference in heights from engine to last wagon is the same. Added to the function of the force in coupling, is the force to overcome the static resistance ie to initiate the rolling,. This force is essentially a function of the weight of the train, and is independent of the grade. Hence the locomotive should not have to experience any greater level of resistance in starting even if the Locomotive or for that matter any part of it were to be standing on the steepest grade (say 1 in 50).

6.1.2 Another apprehension is the loss of tractive power due to the locomotive standing on a steeper grade. While there is a loss of tractive power due to locomotive standing on grade, but the effect is almost negligible. To quantify, the loss of tractive power which is a function of force from wheel normal to the rail, and the coeff. of friction between the rail and wheel being Mu, would be effectively unaffected for all practical purposes. As Shown in the above sketch, if the wheel is standing on a grade of Φ, then the traction effort is W * Cos Φ. For Φ being of the order of .02 radians, the value of cos Φ is about .9998, ie not far from 1, hence virtually no effect on tractive power would be felt.

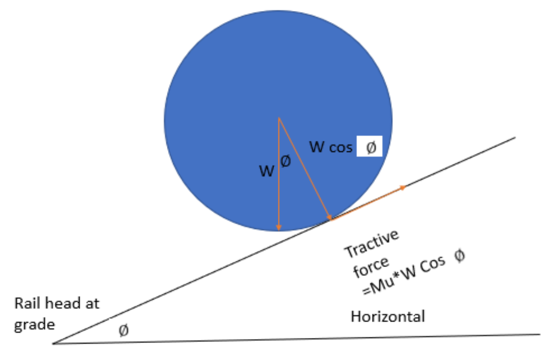


Fig 5: Effect of grade on tractive force

6.2 Q: Wouldn't the propensity of uncoupling increase with higher grades?

6.2.2 Ans: The propensity for uncoupling should be a function of the rate of change of grade, rather than the grade per se. maybe to address such concerns, the vertical curve radius could be kept at a sufficiently large value. However, since 3000 meters is the prescribed radius for B routes, this could be the standard for steep grades even on C, D and E routes. However this decision can be taken after appropriate simulations/trials.

There would not be any change in the maximum coupling forces (averaged over time) however for determining the dynamic instantaneous forces which could be much very different due to train dynamics, so that aspect needs further study. However a major factor contributing to uncoupling is the engineman ship. There may be a need to reskill the Loco drivers to cater to this sort of profile, and additional signages put on the section to caution the driver when different portions of his train are at different grades so that he can employ the appropriate driving technique.

6.3 Q: Wouldn't passenger discomfort increase with such steep grades?

6.3.1 Ans: To put matters in perspective from passenger comfort consideration, the 1 in 50 grade is not significant. The Indian railway standards allow for a cant of 165 mm, and a cant deficiency of 75 mm, and an excess likewise of 75 mm. Hence on a stationary train on a cant of 165 mm on a base of 1676mm gauge comes to a lateral tilt of 165/1675 ie almost 1 in 10. Even on a moving train, the passenger can feel a lateral force equivalent to a tilt of 75/1676 or 1 in 22.3. further more if it be argued that the lateral forces are not to be compared with longitudinal forces, the suburban trains have a peak acceleration of the order of 0.6 m/sec2 which is equivalent to a grade of 0.6/9.8 or 1 in 16. On metro trains the maximum acceleration and braking is permitted of the order of 1.1 m/sec2 equivalent to a longitudinal grade of 1 in 9. Hence the ‘toppling over ‘feeling due to longitudinal grade is negligible.

Another aspect of passenger comfort is the jerk rate, which is the time derivative of acceleration. The vertical acceleration is governed by the radius of the vertical curve. Hence should not be an issue. Also as explained earlier, since neither the traction abilities of the locomotive are effected, nor the trailing load as felt by the locomotive and its coupling, there would not be any significant change in the hauling ability of the locomotive.

7.0 Conclusion

Earthwork is a major component of the project execution timeline. A reduction is the earthwork and also the quantum of land by such large proportions by introducing the concept of “Effective Gradient’ will help in greatly reducing project timeline and costs, apart from various other benefits of reduced bank heights listed earlier. The benefits would greatly be enhanced with the progressive introduction of dedicated routes for longer heavy haul trains. However simulation trials may be needed to validate effect on train dynamics and wagon component forces.

8.0 Acknowledgements:

would like to acknowledge the insights, encouragement and inputs provided by various people in preparing this paper, Namely Mr. SS Limaye (IRSE retd.), Mr. Pankaj Jain (IRSE retd.), Mr. MK Gupta (ex ME), Mr. Anirudha Jain (IRSE retd.), and Mr. VK Raina (IRSME retd.) and Mr. Hari Bhajan Singh (IRSEE retd.) and last but not the least Mr. Maneesh Gupta (Voluntarily retired IRSE).

Glorious View of Green Building on the eve of Independence Day.

Literature Digest

The Role of Track Stiffness & Its Epatial Variability on Long Term Track Quality Deterioration

Transport Technology Center Inc.(TTCI) has investigated an application called Neural Network (NN) technique, earlier used for wheel failure prediction to predict bolt hole crack (BHC),vertical split head (VSH) and crushed head (CH) rail flaw in 136 RE rail. The analysis has shown that rail flaw can be reasonably predicted by in putting different variables. The model is based on 10 input variables pertaining to rail flaw development and rail defect type output. The model were separately proposed for BHG, VSH & CH. Of the existing failure data available for each defect in the section, 50% was used for training the model and other 50% was used for its validation.

This study done has shown sufficient degree of accuracy (more than 80%) in most situation in predicting flaw. One of the significant observation was that GMT variable was not significant to the model performance. This analysis can in future even include fastening system, track geometry etc to get even better result. This model can be further developed to give advance warning.

By: Abbe Meddah & Matt Witte

Ref. :-Journal of Railway Track & Structures, April 2019

Fatigue Crack Growth in Rail; Simulation & Full Scale Lab Tests

The fatigue crack on rail head was used in a fracture mechanics model developed by TTCI called Rail grow to analysis the influence of Fatigue Crack Growth Rate (FCGR) properties on growth of fatigue defects formed in Rail head. It has been observed that, residual stresses have a substantial influence on crack growth rate but quantifying the same was difficult. Rails with known internal defect was also subjected to cyclic loading in lab and rate of growth of defect was measured periodically.

The actual result was compared to result predicted by Rail grow model. The result has led to understanding of influence of metallurgical properties, fatigue properties and residual stress and their inter play in growth of fatigue defect.

By : Ananyo Banerjee
Ref:-Rail Track & Structure Journal, May 2019

Bridges to the Future- Modern Approaches to Research & Repair

The paper deals with modern approach to inspection & monitoring of Rail Road Bridges. U.S Federal Rail Road administration (FRA) by its document FRA 237, issued in 2010 instructed Rail Roads that owned Bridges to do four things.

1) Inspect all Bridges (at least once a year & with no more than 540 days between inspections);

2) Have a bridge management plan in place and share it with the FRA;

3) know the capacity of all bridges and include it in the plan;

4) And use experts (bridges engineers, bridge inspectors and bridge supervisors) for the above.

The instruction suddenly brought surge in maintenance input and introduction of new technology for inspection. Also it resulted in development of technically competent workforce.

The future technology that has come as a result falls in three general categories, namely.

A) See me - All part has to be seen and unmanned Aerial vehicle (UAV), the drones has been inducted to inspect the location otherwise difficult to reach.

B) Feel me -There is hitting of bridges by vehicle which can be mild to severe. Equipment that monitor impact and reports to rail road authority in real time duly classifying the level of impact has been developed. This ensures 24x7 monitoring and even controlling traffic if required in case of severe hit ensuring safe running.

C) Touch me –Some failure grow at a very small speed and are not easily detected by naked eyes but then all of a sudden the failure rate increases. Digital photography taken at different type & by superimposing them, such detoriation can be monitored at initial stage itself. Software’s has been developed for this.

The paper however mentions that the human factor still cannot be replaced completely by these technologies

By Paul colony

Ref: Journal of Railway Track & Structures, April-2019

Understanding the Impact of Train Run-throughs on Railway Switches using Finite Element Analysis

Train run-throughs on railway switches is a special issue, where a train passes through non-trailable railway switches in the wrong direction. This has the potential to cause severe damage and can lead to derailment. In order to understand the impact of train run-throughs on railway switches, a three-dimensional finite element model using explicit analysis has been developed. A detailed switch model has been developed that includes all key components: stretcher bars, supplementary drive and point operating equipment. The model was validated through a specifically designed experiment where switch run-throughs were emulated on a real switch; a good agreement was found between the experimental data and the model. The model has been used to make an assessment of the locking mechanisms. The forces in each component have been assessed and investigated, and the observations of failure location and component during run-through analysis are indicated. During a run-through, the supplementary drive rod and stretcher bar encounter a significant plastic deformation, and it is recommended that they should be redesigned in order to avoid plastic behavior

By: JY Shih , H Hemida, E Stewart and C Roberts

Ref: Journal of Rail and Rapid, April 2019

Traction Modelling in Train Dynamics

This paper presents five locomotive traction models for the purpose of train dynamics simulations, such as longitudinal train dynamics simulations. Model 1 is a look-up table model with a constant force limit to represent the adhesion limit without modelling the wheel–rail contact. Model 2 is improved from Model 1 by empirically simulating locomotive sanding systems, variable track conditions and traction force reduction due to curving. Model 3 and Model 4 have included modelling of the wheel–rail contact and traction control. Model 3 uses a two-dimensional locomotive model while Model 4 uses a three-dimensional locomotive. Model 5 is based on Model 2 and developed to simulate hybrid locomotives. Demonstrative simulations are presented for the case of longitudinal train dynamics. The results show that the consideration of locomotive sanding systems, variable track conditions and traction force reduction have evident implications on the simulated traction forces. There can be up to 30% difference in the simulated traction forces. Simulated traction forces by models that consider the wheel–rail contact are about 10–15% lower than those simulated by models without consideration of the wheel–rail contact. This is mainly due to the variable friction in the wheel–rail contact and conservative traction control schemes.

By: Qing Wu, Maksym Spiryagin, Peter Wolfs and Colin Cole

Ref: Journal of Rail and Rapid, April 2019

Effect of the Inter-Car Gap Length on the Aerodynamic Characteristics of a High-Speed Train

In wind tunnel experiments, the inter-car gaps are designed in such a way as to separate the force measurements for each car and prevent the interference between cars during tests. Moreover, the inter-car gap has a significant effect on the aerodynamic drag of a train. In order to guide the design of the inter-car gaps between cars in wind tunnel experiments, the impact of the inter-car gap length on the aerodynamic characteristics of a 1/8th scale high-speed train is investigated using computational fluid dynamics. The shear stress transport model is used to simulate the flow around a high-speed train. The aerodynamic characteristics of the train with 10 different inter-car gap lengths are numerically simulated and compared. The 10 different inter-car gap lengths are 5, 8, 10, 15, 20, 30, 40, 50, 60, and 80 mm. Results indicate that the aerodynamic drag coefficients obtained using computational fluid dynamics fit the experimental data well. Rapid pressure variations appear in the upper and lower parts of the inter-car gaps. With the increase of the inter-car gap length, the drag force coefficient of the head car gradually increases. The total drag force coefficients of the trains with the inter-car gap length less than 10 mm are practically equal to those of the trains without inter-car gaps. Therefore, itcan be concluded from the present study that 10 mm is recommended as the inter-car gap length for the 1/8th scale high-speed train models in wind tunnel experiments.

By: Tian Li, Ming Li, Zheng Wang and Jiye Zhang

Ref: Journal of Rail and Rapid, April 2019

Comparison of Non-Hertzian Modeling Approaches for Wheel–Rail Rolling Contact Mechanics in the Switch Panel of a Railway Turnout

Due to the complicated wheel–rail contact relation of railway turnouts, it is necessary to select a reasonable rolling contact model to simulate the vehicle–turnout dynamics and wheel–rail damages.

This paper mainly aims to evaluate the calculation accuracy and efficiency of different non-Hertzian modeling approaches in solving normal and tangential wheel–rail contact problems of railway turnouts. Four different non-Hertzian approaches, namely CONTACT, Kik–Piotrowski, Ayasse–Chollet, and Sichani methods are compared and analyzed. The above four models are built considering the relative motion of stock/switch rails. A wheel profile called LMA contacting with stock/switch rails (head width 35 mm) of CN60-1100-1:18 turnouts is selected as the object of analysis. The normal contact problems are evaluated by the wheel–rail contact areas, shapes, and normal contact pressures. The assessment of tangential contact problems is based on the creep curves, tangential contact stresses, and distribution of the stick/slip region. In addition, a contrast analysis is performed on the calculation efficiencies of the four approaches. It is found that the normal and tangential contact results calculated based on the Sichani method coincide well with those obtained according to CONTACT, and the calculation efficiency is about 262 times that of CONTACT. The conclusions can provide some guidance to the selection of wheel–rail rolling contact approach in the simulation of vehicle–turnout dynamics and wheel–rail damages.

By: Xiaochuan Ma, Ping Wang, Jingmang Xu and Rong Chen

Ref: Journal of Rail and Rapid, April 2019

Precise Railway Alignment Measurements of the Horizontal Circular Curves and the Vertical Parabolic Curves using the Chord Method

The chord method is generally used for the horizontal and vertical versine measurements of rails in order to maintain their alignment. In this study, the horizontal versines of the left and right rails were precisely calculated, and the vertical versine functions were exactly derived. According to the inferences, the line charts used for determining the vertical versine were created to demonstrate the proportional relationship with the grade difference and the curve length. for proximate vertical versine calculations were therefore derived. The asymmetrical chord measurements were introduced as well. The results indicated that the versine difference between the inner and outer rails should be considered when measuring the horizontal circular curve with a radius less than 300 m using a 10-m chord. According to the design criteria, a maximum vertical versine of 30 mm formed by the parabolic curves could be measured using a 10 m chord, or a maximum versine of 7.5 mm could be measured using a 5 m chord, which should be considered as the essential versines. The deviation tolerance should be based on these values for accurate rail inspections.

By: Ship-Bin Chiou and Jia-Yush Yen

Ref: Journal of Rail and Rapid, May 2019

of Deck Cracking on Prestress

The American Association of State Highway and Transportation Officials' AASHTO LRFD Bridge Design Specifications have a refined method for the calculation of time-dependent losses in prestressing force. This method estimates a gain in the prestressing force due to differential shrinkage between the precast concrete girder and the cast-in-place deck; however, it does not consider the possibility of cracking in the deck. Some state departments of transportation believe that after the deck cracks, the gain in the prestressing force is lost, and they do not include it in the prestress loss calculations. Some other state departments of transportation believe that not all of the prestress gain is lost and allow some percentage of the gain to be retained. This study focuses on the effect of deck cracking on the long-term loss of prestressing force.

A finite element software model was used to simulate three different girder lengths with varying deck concrete strengths, differential shrinkage parameters, and girder ages at the time the deck was placed. When the girder age at the time of deck placement is more than 60 days, there is a high probability that the deck will crack, though a deck placed on a girder of any age may crack, depending on the properties of the deck and girder. When the deck cracks, some of the gain of prestressing force is lost, but the percentage lost depends on the extent of cracking in the deck. The older the girder is when the deck is placed, the greater the cracking. The cracked deck still provides some restraint, and even in cracked decks, nearly 50% of the prestress gain due to differential shrinkage was retained.

By: Soumya Vadlamani, Richard A. Miller, and Gian A. Rassati

Ref: PCI Journal May –June 2019 page no. 66

OpticVyu Construction Cameras Construction Monitoring, Management & Marketing

OpticVyu Construction Cameras Construction Monitoring, Management & Marketing In recent years, advancement in construction technology has

allowed builders, contractors & project management consultants to efficiently monitor the progress of the job site and make the quick decisions to avoid costly delays. Construction camera systems are emerging as latest technology to monitor project progress, control documentation, maintain image records and to allow efficient inter-team coordination.

OpticVyu Construction Cameras Designed & Developed In India

OpticVyu Construction Camera systems incorporate high end programmable cameras to capture HD images in regular intervals. These images are immediately uploaded via cellular network to cloud based server networks and can be accessed by authorized personnel.

The cameras offer site monitoring/ management solutions & are very helpful in checking project progress, pace of construction & managing number of remote sites from single platform. These are plug and play cameras with inbuilt WiFi providing unlimited storage and 100% documentation of all the events occurred at construction sites.

OpticVyu Features:

- Time-Lapse Videos
- Custom/Event Based Time lapsing
- Monitoring – Project Planning' Integration
- Archived Images
- Photo Comparison
- Photo Annotation & Sharing
- Auto Email
- Event Tagging
- Multiple Projects/Cameras Handling
- Mobile Photo Synchronization Through OpticVyu Mobile App

Project Planning Integration

OpticVyu enable user to integrate project planning with camera monitoring by uploading planning data to OpticVyu portal. Event-based photo comparison allows user to visually verify site's status at event's planned start & end date. User can update event's actual start & end dates based on visual inspection through camera images. By analyzing events' planned & actual dates, Gantt chart shows delays. Ultimately, event-based time-lapse videos can be created to check the progress during the execution of a particular event.

Major Products:

– OpticVyu Nikon Box Camera: 24.2MP

– OpticVyu Fixed Position Camera: 5MP

How is it different from CCTV Camera?

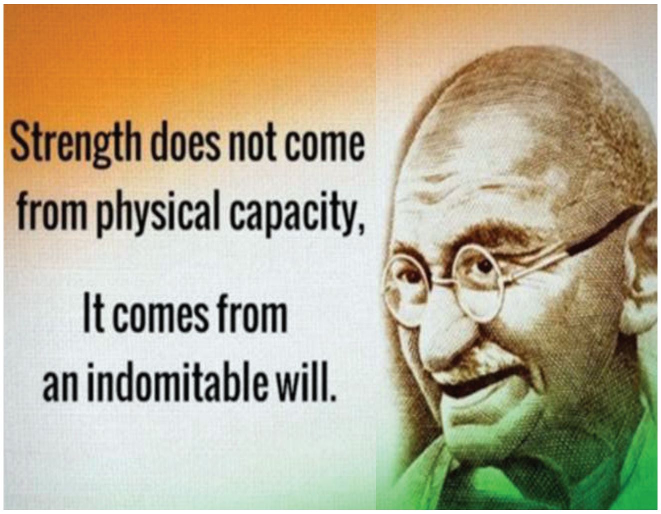
CCTV camera is an obsolete method of surveillance using low quality videos with limited storage capabilities. Construction cameras offer following advantages over CCTV cameras:

- Unlimited storage capabilities
- High image quality
- Require low bandwidth to operate on
- Works on most remote sites
- Produces HD time-lapse videos
- Plug & play systems
- Synchronization of mobile photos
- Easy and streamlined tracking of past event
- Less cameras, more coverage

Bridge Projects Undertaken

OpticVyu is currently live at Noney-Imphal Bridge (Indian Railways: World's Tallest Rail Girder Bridge), Barapullah Bridge (Delhi), Durgam Cheruvu Cable Stayed Bridge (Hyderabad), Bridge #40 & 44 (North East Frontier Railways, Manipur), Silvassa Skywalk.

Ref: Master Builder, June 19, pg. 22



Events

Discussion during Heritage Conference

Director addressing on World Environment Day

Rajbhasha Meeting

Workshop on International Yoga Day

Updates Of Codes & Manuals

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
			TRACK
1. Indian Railways P-Way Manual			
01.	94	01.06.2004	Para No 410(3) and 410(4) replaced
02.	95	30.06.2004	Para No 826 replaced
03.	96	22.07.2004	New para 607(3) added
04.	97	19.10.2004	Note of Annex. 8/3 (para 807 & 808) replaced
05.	98	17.05.2005	Para No 202(1) replaced
06.	99	14.12.2005	Para No 107(1) replaced
07.	100	21.06.2006	Para No 902(1), 907(2)(a), 919(1), 924(b)(i) replaced. Addition to sub-para 916(1)(i), 924(b)(ii), 924(c)
08.	101	21.12.2006	Para No 107(1) replaced
09.	102	30.05.2007	Para No 244(2)(e), 273(1), 275(3), 276, 506(3), 716(2), 1404 replaced
10.	103	01.06.2007	Para No 237(6) replaced
11.	104	19.07.2007	Para No 1504 replaced
12.	105	22.08.2007	Para No 277(a)(3) replaced
13.	106	14.09.2007	Para No 124(2)(a) replaced
14.	107	12.11.2007	Para No 615(3) replaced
15.	108	18.12.2007	Para No 1302(2)(h) replaced
16.	109	15.02.2008	Para No 202(1) replaced
17.	110	04.03.2008	Annexure 8/1 – Para 806(1), 8/2 – Para 806(2), 11/4- Para 1118(5) and 11/5-1120(4)(C) replaced
18.	111	23.04.2008	Para No 222, 701(2), 1301(1), 1302(1), (1) (i), (2) (g), 1307(1)(g), 1309(1), 1309(2), 1309(4) and Annexures 13/1 (2), (4), (5) & Annexure 13/6 (heading) replaced
19.	112	25.04.2008	Para No 726(3)(b) replaced
20.	113	03.06.2008	Para No 1504 replaced
21.	114	10.10.2008	Para No 206(3) and 317(3) replaced
22.	115	24.11.2008	Note given at the bottom of item No. 9(a) of Annexure-9/1 para 904 replaced
23.	116	23.04.2009	Para No 220 replaced
24.	117	19.05.2009	Para No 322, 248(1), 248(2)(a), 244(4), 263(2)(a) replaced
25.	118	30.07.2009	A new sub para 146(vii) added
26.	119	18.08.2009	Para No 220(3) replaced
27.	120	16.04.2010	Para No 151(1)(a) replaced
28.	121	12.07.2010	Para No 1302 (2)(h) replaced
29.	122	23.11.2010	Para No 238(2)(g)(iii) and 238(2)(g)(iv) added Para No 257(4), (6), (7) & Para 917 replaced
30.	123	27.01.2011	Para No 170(15) added
31.	124	14.02.2011	Para No 250(2) replaced

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
32.	125	21.02.2011	New Para 272(4)(a) added. Para No 508, 510(3) replaced
33.	126	21.06.2011	Para No 263(2)(a)(i) replaced
34.	127	28.11.2011	Para No 1302(2)(g) replaced
35.	128	05.03.2012	Para No 234(5), 238(2)(d)(i) replaced. Annexure-2/11 Para 263 replaced, Para 273 renumbered as Para 273(a), New para added as Para 273(b), Para 904 replaced, Para 916 sentence deleted, Para 918(1), 924(b)(i), 924(c) replaced, Para 924(b) iii deleted
36.	129	28.06.2012	Para No 302(1)(a) and 248(2)(a) replaced
37.	130	16.11.2012	Para No 244(4), 1001(6), 302(1)(b)(ii), 224(2)(e)(v) and annexures-2/11 & 2/13 of para 263 replaced
38.	131	11.01.2013	Para No 277(a)(7), 429 and 502 replaced
39.	132	08.04.2013	Para No 238(1)(b)(5), Para 107, Para 108(2)(b), Para 123, 124, 127, 129, 136, 139, 144, 223, 237(5)(a) replaced. Added new para 108(2)(c), Para 124(A), 139(A) deleted
40.	133	04.06.2013	Para No 202(1) modified, New sub para (c) of para 1303(1) added
41.	134	18.07.2013	Annexure-2/6(B), (C) and (D) and Annexure-2/6/1of Par 237(5) added
42.	135	07.05.2014	Para No 151(1), 910(1)(g)(i), 1007(c), 1116(c), Annexure 9/7 of para 918(2), Annexue 2/11 & 2/13 of para 263, Annexure 2/6 of para 237(5), para 1014(1)(d) replaced, New note to para 107 added
43.	136	14.11.2014	Para No 237(8)(b), 279, 406(2)(a), 421, 427(2), 502(1), Annexure 7/2 part B of para 708(1), para 804, 824, 825, 910(y), 1007(1)(l) replaced, Para 708(1) modified
44.	137	18.06.2015	Para No 254, 257(4)(b), 317(3)(b), 814(1)(a), 1304(3) replaced, New para 255(6), 310(7) added
45.	138	25.08.2015	Para No 1302(2)(g) replaced
46.	139	08.02.2016	Para No 607(4) added, Para 248(2)(a), 910(1)(j) replaced
47.	140	28.11.2017	Para No 420(2), 502(1) replaced
48.	141	18.01.2018	Para No 152(2) replaced
49.	142	27.03.2018	Para No 169, 170(5), 170(6)(b) & 1408(3) Note (v) replaced
50.	143	19.04.2018	Para No 237 and Annexure 8/3, 8/3A & 8/4of Para No 807 & 808 replaced
51.	144	08.05.2018	Para No 302(1)(e), 919(1) & Annexure 9/5 Para 916(1) replaced
52.	145	23.07.2018	Para No 228(4), New Annexure 2/7 to para no. 228(4) added, Para No replaced
53.	146	15.18.2011	The existing 'Annexure 2/4 of para 211' shall be replaced.
54.	147	28.12.2018	The existing para 248(2)(a) replaced.
55.	148	08.01.2019	The existing paras 275(1) & 275(3) replaced.
56.	149	08.02.2019	The existing para 316(2) shall be replaced.
2. LWR Manual			
01.	02	1999	Para No 6.2, 6.2.1(i), 1.18 replaced
02.	03	1999	Heading of Para No 4.5.7 & sentence at para 4.5.7.1(1)(b)(i) replaced
03.	04	1999	Para No 6.3 modified, Para 6.3.1 added, Para 6.3.1 & 6.3.2 renumbered
04.	05	1999	New para 4.4.1(i) added

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
05.	06	1999	Text substituted to para No 2.2.1, Para No 5.2.1 reworded
06.	07	2000	Changes made to Figs 4.2.1(a) to (d)
07.	08	2002	Para No 1.1 & 1.2 reworded, Text added to para 4.5.3
08.	09	2005	Para No 6.2.1(i), 4.4.1 replaced
09.	10	2006	Fig. 5.6 & Para No 6.4.1(i)(c) replaced
10.	11	2006	Text substituted to Para 3.2.1
11.	12	2009	Para No 1.16, introductory para of Annexure XA [Para-9.1.2(i)] replaced
12.	13	2010	Para No 1.11 replaced
13.	14	2011	Para No 3.4, 8.2.5, 9.1.8(i), 9.1.8(iv), Annexure-VI item 1(c)(i), Annexure-V, Table-I replaced
14.	15	2012	Para No 8.1.5(i) & Figs 4.2.1(a) to (d) replaced
15.	16	2014	Para No 4.3.3(i), Fig. 4.2.1(a) to (d) replaced
16.	17	2018	Para No 5.6.1, Fig. 5.6, Annexure XIII-A replaced. New Fig 8.2.1 added
17.	18	11.03.2019	Footnote to para 4.5.7.1 (iv) at page 12 shall be replaced
18.	19	15.04.2019	Item No. 1 of Annexure III replaced, Fig 4.4.3(a) and 4.4.3(b) replaced, Fig 4.4.3(c) modified, Item No. 2 of Annexure III deleted
3. Track Machine Manual			
01.	01	Mar. 2000	Addition of text to the end of Preface.
02.	02	Mar. 2000	Annexure 8.1 para (A) & para (B) replaced
03.	03	May 2003	Para 4.6.8 added
04.	04		Para 4.6.8(iv) added
05.	05	Sept. 2003	Text added to para 5.6
06.	06	13.01.2004	Para No 4.3.3 & 8.3.2 replaced
07.	07	07.04.2004	Para No 4.3.1 modified
08.	08	25.10.2004	Para No 5.1.3 added & existing para 5.1.3 renumbered
09.	09	20.10.2006	Annexure 8.1 replaced
10.	10	12.12.2006	Para No 2 of Annexure 5.9 replaced
11.	12	22.08.2013	Annexure 8.2 replaced, New annexure 8.3 added, Table 6.2 in Para 6.3.5 modified, Text replaced in Para 8.4.5
12.	13	25.09.2012	Para 6.2.1 added
13.	14	14.06.2012	Para No 5.3.3 replaced
14.	15	14.06.2012	Para No 4.4.3 replaced
15.	16	12.11.2013	Para 9.6(ii) modified
16.	17		Para 6.2 modified
4. Manual For Ultrasonic Testing of Rails & Welds			
01.	01	Nov. 2014	Para No 8.15.1 replaced
02.	02	Dec. 2014	Para No 8.14, 8.15.1, Annexure IIA and IIB replaced
03.	03	Mar. 2016	Para No 4.1.1(c), 5.1.2, 8.6.4, 8.7.2, 8.10, 8.14, 8.15.1, Figs. 3 & 22 replaced. New clause b)(iii) below para 8.16 added, New para 6.3.1 & 6.3.2, 10.6 added

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
04.	04	Sept. 2018	Para 6.6, 8.14, 8.15, 8.15.1 & 8.15.2 modified,
5. Indian Railways Code For The Engineering Department			
01.	50	21.09.2017	Introduction of measurement & recording of 'executed works' by the contractor' in Rly Construction Works.
02.	51	27.09.2017	Para Nos 701, 1102, 1209 should be amended
03.	52	23.10.2017	Existing para 1238 replaced
04.	53	06.11.2017	Para No 701 should be amended
05.	54	22.01.2018	Para No 1264 (e) & 1264 (f) should be amended
06.	56	05.03.2019	Para No 1264 should be amended
BRIDGE			
1. Indian Railways Bridge Manual			
01.	01	01.09.1999	Para No 1007 replaced, New para 1007(A) added.
02.	02	21.07.2000	New para 16 added
03.	03	21.07.2000	Deleted para 513(b)
04.	04	21.07.2000	Deleted para 515
05.	05	21.07.2000	Deleted para 603
06.	06	21.07.2000	Deleted para 222(1b), 222(2f)
07.	07	21.07.2000	Deleted para 618
08.	08	21.07.2000	Para No 504(4) replaced, Add new para 521, sub para 5 under para 616 and sub para 5 under para 210
09.	09	27.07.2000	Add new sub para 317 of Chapter III
10.	10	31.08.2000	Para No 604 replaced
11.	11	14.01.2003	Add para before chapter 1
12.	12	18.12.2007	Para No 217.2(a)(i) and para 217.2.(b)(i) replaced
13.	13	22.01.2008	Para No 317 replaced
14.	14	20.03.2008	Delete para 310, 312(4), 313(2) and 313(3) of chapter III,
15.	15	05.08.2008	Para No 410(2)(b), 418(5), 430 replaced, Para 3(ii) of 606 is proposed for deletion and Para 3(i) renumbered as 3
16.	16	13.08.2008	Para No 317(iii) replaced
17.	17	15.09.2008	Para 318 added
18.	18	17.12.2008	Para 224 added
19.	19	11.01.2010	Para 318 modified
20.	20	07.06.2010	Para No 1104(5) replaced
21.	21	02.07.2010	Para No 1107 (d) modified. Add para 1107(15)(i)
22.	22	28.03.2011	Para No 1107(15)(i) replaced & renumber as 1107(15)(b)(i), para 1107(15)(b) is renumbered as 1107(15)(b)(ii)
23.	23	23.08.2011	Replace existing Chapter-VIII by revised Chapter-VIII
24.	24	14.09.2011	Para 714(2), 1005(1), 1005(3), 1104, 1104(2), 1104(5), 1106(2), Page No xi (Index)1104 modified

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
25.	25	17.12.2012	New sub para 3 may be added to existing para 311
26.	26	23.08.2013	Para No 217.2(a)(ii), 217.2(c), 217.4(c), 217.4(d), 217.4(e), 217.4(l), 615 to be replaced
27.	27	03.01.2014	New para 1107 5 i), 215 A added
28.	28	20.03.2014	Chapter-X, Part B – Title of Deep Cuttings replaced and para 1010 to 1015 & Annexure 10/2 replaced by Para 1010 to 1017 and Annexure 10/2 attached.
29.	29	15.04.2014	Para No 312(2), 312(4) replaced. Add new para 313(4)
30.	30	25.11.2014	Para 102(b), 504, 505, 506, 507, 508, 509 & Annexure 5/1 deleted
31.	31	09.02.2015	Para No 617 replaced.
32.	32	12.03.2015	Para No 222 2(f) replaced.
33.	33	21.03.2016	Para No 107(1) (a) is amended and 107 (1)(f) added. Para 222 (3) is amended by adding sub para (c), (d) and (e)
34.	34	04.10.2016	Added para 224
35.	35	31.07.2017	Para No 313(2) & 313(3) replaced.
36.	36	27.03.2018	Para No 317 & 318 replaced.
2. Indian Railways Bridge Rule			
01.	47	22.06.2017	Add new para 2.8.1.2
02.	48	22.06.2017	Add new clauses
03.	49	26.12.2017	Para 2.12 deleted. New para 2.12 inserted
3. Indian Railways Bridge Substructure & Foundation Code			
01.	01	17.04.2014	Para 4.8.1, 4.9.3 replaced
02.	02	20.10.2016	Modify description & heading of contents at S.No. 7.5, Delete para 7.5.3
03.	03	22.06.2017	Modified para 4.5.9
04.	04	11.08.2017	Modified para 4.9.2 & 4.9.3
4. Indian Railways Concrete Bridge Code			
01.	01	16.12.2014	Replace table 10 of para 10.2.1
02.	02	14.01.2015	Insert para 5.4.7 & 5.4.7.2
03.	03	20.01.2015	Insert note under para 4.5.1, delete para 14.9 & replace, delete para 15.9.4.1 & replace, delete para 15.9.4.2 & replace, delete para 15.9.9 & replace
04.	04	15.11.2016	Para 14.9, 14.9.1 & 15.9.9 deleted
05.	05	13.06.2017	Para 16.4.4.4.5 modified
06.	06	27.07.2017	Para 7.1.5 modified
07.	07	26.06.2018	New para 4.5.1 added
5. Indian Railways Arch Bridge Code			
01.	07	25.09.2000	Replace para 1.1
02.	08	28.01.2015	Replace para 5.3.3
6. Indian Railways Welded Bridge Code			
01.	01	16.02.2015	Para 27.1 replaced
02.	02	11.07.2018	Para 27.1 replaced

IRICEN Calendar of Courses -2019

COURSE NO	COURSE NAME	FROM	TO	DURATION	ELIGIBILITY
JULY					
19308	EPC & PPP	01/07/2019	02/07/2019	2 Days	JAG/SG Officers
19309	CAO/C Seminar	04/07/2019	05/07/2019	2 Days	CAO/Cs of all rlys.
19204	Sr Prof Bridges	08/07/2019	02/08/2019	4 Weeks	SS/JAG/SG Officers with min. 6 years of service in Gr. A
19417	Bridge planning	08/07/2019	19/07/2019	2 Weeks	JS/SS
19701	G6 Prob. awarness course	15/07/2019	26/07/2019	2 Weeks	Probationers of Other Deptt.
	Spare Slot	15/07/2019	19/07/2019	1 Week	
19418	Construction & Maintenance of Steel & Concrete Bridges	22/07/2019	02/08/2019	2 Weeks	JS/SS
19702	G1 Prob. awarness course	29/07/2019	09/08/2019	2 Weeks	Probationers of Other Deptt.
AUGUST					
19003	IRSE 2018 Joining	05/08/2019	09/08/2018	1 Week	IRSE Prob. - 2018 Batch
19703	G2 Prob. awarness course	13/08/2019	23/08/2019	2 Weeks	Probationers of Other Deptt.
19310	PCE Seminar	16/08/2019	17/08/2019	2 Days	PCEs of All Rlys.
19419	Tunneling	19/08/2019	23/08/2019	1 Week	JS/SS/JAG/SG Officers
19704	G3 Prob. awarness course	26/08/2019	06/09/2019	2 Weeks	Probationers of Other Deptt.
19421	CA & PM	26/08/2019	06/09/2019	2 Weeks	JS/SS/JAG/SG Officers
19420	GeoTech Inv. & Survey	26/08/2019	06/09/2019	2 Weeks	JS/SS/JAG/SG Officers
SEPTEMBER					
19705	G4 Prob. awarness course	09/09/2019	20/09/2019	2 Weeks	Probationers of Other Deptt.
19422	Basics of Track Maintenance	09/09/2019	20/09/2019	2 Weeks	JS/SS
19311	SJV	09/09/2019	10/09/2019	2 Days	Officers of Rly & other Govt. Dept.
19312	CGE Seminar	12/09/2019	13/09/2019	2 Days	CGEs/Trg. Managers of All Rlys.
19103	Integrated	16/09/2019	05/12/2019	12 Weeks	Gr. B Officers
19706	G5 Prob. awarness course	23/09/2019	04/10/2019	2 Weeks	Probationers of Other Deptt.
19205	Sr Prof PWay	23/09/2019	18/10/2019	4 Weeks	SS/JAG/SG Officers with min. 6 years of service in Gr. A
19423	Advanced Track Maintenance with mechanized maintenance	23/09/2019	04/10/2019	2 Weeks	SS/JAG/JS who have attended Basic track maintenance/ Tr. Module-1
OCTOBER					
19424	Derailment investigation and advance track technology	07/10/2019	18/10/2019	2 Weeks	SS/JAG/SG/JS who have attended Adv. Tr. Mntn with mechanized maintenance
19425	MIDAS	07/10/2019	18/10/2019	2 Weeks	SSE/ABEs
NOVEMBER					
19313	IRICEN Day	07/11/2019	08/11/2019	2 Days	IRSE Officers of 1993 Batch
19004 (P)	PHASE-1	11/11/2019	03/01/2020	8 Weeks	IRSE Prob. '2018 Batch

COURSE NO	COURSE NAME	FROM	TO	DURATION	ELIGIBILITY
19506	PSU	11/11/2019	16/11/2019	1 Week	Sr Level Officers of PSU
19323	Work Shop DY/CE	18/11/2019	19/11/2019	2 Days	Dy CE/Ds of All Rlys.
DECEMBER					
19314	Seminar CE/TP	12/12/2019	13/12/2019	2 D	CE/TPs of All Rilys.
19514	Spl course for IRCON Officials	16/12/2019	27/12/2019	2 Weeks	IRCON Officials
19515	Spl course for DFCCIL Engineers	16/12/2019	03/01/2020	3 Weeks	DFCCIL Engineers
20001	Posting Exam IRSE -'17 Batch	30/12/2019	03/01/2020	1 Wk	Probationers- IRSE 2017 Batch
JANUARY 2020					
19314	Spl course for DFCCIL ENGINEERS	13/01/2020	17/01/2020	1 Week	DFCCIL ENGINEER

SSTW Calendar of Courses -2019

COURSE NO	COURSE NAME	FROM	TO	DURATION	ELIGIBILITY
JULY					
19833	Advanced Track maintenance including mechanized maintenance	08/07/2019	19/07/2019	2 Weeks	JE/SSE/P.Way
19834	Construction & Maintenance of Steel & Concrete bridges	08/07/2019	19/07/2019	2 Weeks	JE/SSE/Bridge
19910	PSU RVNL TCB2	07/08/2019	08/02/2019	4 Weeks	
19835	RWI (Rail Wheel Interaction)	22/07/2019	02/08/2019	2 Weeks	JE/SSE/P.Way
19836	Tunneling	22/07/2019	26/07/2019	1 Week	JE/SSE/Works
19837	Survey	29/07/2019	02/08/2019	1 Week	JE/SSE/Works
AUGUST					
19838	Basics of Track Maintenance	05/08/2019	16/08/2019	2 Weeks	JE/SSE/P.Way
19839	Bldg Const	05/08/2019	09/08/2019	1 Week	JE/SSE/Works
19911	PSU RVNL TCB3	08/05/2019	30/8/2019	4 Weeks	
19840	Contract Mgmt	13/08/2019	23/08/2019	2 Weeks	JE/SSE/Works-P.Way
19841	Advanced Track maintenance including mechanized maintenance	19/08/2019	30/08/2019	2 Weeks	JE/SSE/P.Way
19842	Conc-Tech	26/08/2019	30/08/2019	1 Week	JE/SSE/Works
SEPTEMBER					
19843	RWI (Rail Wheel Interaction)	03/09/2019	13/09/2019	2 Weeks	JE/SSE/P.Way
19844	Land Mgmt	03/09/2019	07/09/2019	1 Week	JE/SSE/Works
19845	TMS	09/09/2019	13/09/2019	1 Week	JE/SSE/P.Way
19846	Basics of Track Maintenance	16/09/2019	27/09/2019	2 Weeks	JE/SSE/P.Way
19847	Bridge planning	16/09/2019	27/09/2019	2 Weeks	JE/SSE/Bridge

COURSE NO	COURSE NAME	FROM	TO	DURATION	ELIGIBILITY
19848	Advanced Track maintenance including mechanized maintenance	30/09/2019	11/10/2019	2 Weeks	JE/SSE/P.Way
19849	Construction & Maintenance of Steel & Concrete bridges	30/09/2019	11/10/2019	2 Weeks	JE/SSE/Bridge
OCTOBER					
19901	PSU	14/10/2019	18/10/2019	1 Week	PSU ENGGS.
19902	TOT	14/10/2019	18/10/2019	1 Week	Instructor ZRTI
NOVEMBER					
19850	Basics of Track Maintenance	04/11/2019	15/11/2019	2 Weeks	JE/SSE/P.Way
19851	Geo-tech	04/11/2019	08/11/2019	1 Week	JE/SSE/Works
19852	Survey	11/11/2019	16/11/2019	1 Week	JE/SSE/Works
19853	Advanced Track maintenance including mechanized maintenance	18/11/2019	29/11/2019	2 Weeks	JE/SSE/P.Way
19854	Bldg Const	18/11/2019	22/11/2019	1 Week	JE/SSE/Works
19855	Conc-Tech	25/11/2019	29/11/2019	1 Week	JE/SSE/Works
DECEMBER					
19853	Geo-Tech	02/12/2019	06/12/2019	1 Week	JE/SSE/Works
19903	PSU	30/12/2019	03/01/2020	1 Week	PSU ENGGS.
19904	TOT	30/12/2019	03/01/2020	1 Week	Instructor ZRTI


दीक्षांत समारोह
इरिसेन
खेल परिसर

IRICEN Calender of courses (CoC) for 2019 (Rev 11)

Month & Year	July.2019					Aug.2019				Sept.2019					Oct.2019				Nov.2019				Dec.2019					Jan.2020			
HOSTEL CAP	105	146	105	105	105	105	65	120	95	95	100	110	110	110	105	105	40	0	101	91	116	91	91	96	101	101	98	70	20	0	0
Date (Monday)	1	8	15	22	29	5	13	19	26	2	9	16	23	30	7	14	21	28	4	11	18	25	2	9	16	23	30	6	13	20	27
Institute holidays							12&15			2				2	8			28 & 29		12						25					
IRSE 2016																					Phase-1, 19004 (Q BATCH)										
IRSE 2017	19002, Phase-																										posting exam 2017, 20001				
IRSE 2018		Joining, 19003																	Phase-1, 19004 (P BATCH)												
Integrated courses	2 Integ	IRTMTC ALD		19102 INTEGRATED								19103 Integrated					IRTMTC ALD		19103 Integrated												
Sr.Prof. courses		19204-Sr Prof. Bridge											19205-Sr Prof. Track																		
Track courses									19422-Basic track maintenance				19423-Adv. Tr. Mntn incl. mechanized tr mntn		19424-Derailment Investigation & Adv. Tr. technology					19426-Derailment Investigation for Traffic officers											
Bridge,survey , Geotech courses		19417-Bridge planning		19418- Construction & Mntn of Steel & Conc. Bridges							19420-Geo Tech &Survey																				
Contract, Arbit & PM								19419 - TNLG	19421 CA&PM																						
Software courses															19425-MIDAS																
Courses for PSU			SPL CR FOR MRIDC - 19511					SPL CR FOR MRIDC - 19512				19513DMRC													19514-IRICON						
Trainees			25					25				15													20	20					
COURSES FOR PSU																									19515-DFCCIL				19515-A,DFCCIL		
TRAINEES																									20	20	20		20		
AWARENESS COURSES			19701-G6		19702-G1		19703 G2		19704-G3		19705-G4		19706 -G5																		
HAG/SAG SEMINARS	19308					19325					S1 19312						S2		19313		19323 S3			S4 19314				S5			
Trainees	35					40					35						40		40		25			35				40			
	EPC &PPP					VIGILANCE MEETING					S. SLOT , CGE						S SLOT		IRICEN Day		WS/ DyCE'D, S. SLOT			S. SLOT, CE/TP				S SLOT			
Misc																															
Trainees																															

SSTW Calender of courses (CoC) for 2019 (Rev 11)

Month & Year	July.2019					Aug.2019				Sept.2019					Oct.2019				Nov.2019				Dec.2019					Jan.2020			
HOSTEL CAP	65	65	65	65	65	65	30	65	65	70	70	70	70	70	70	65	0	0	0	70	70	70	70	70	35	0	0	0	0	0	0
Date (Monday)	1	8	15	22	29	5	12	19	26	3	9	16	23	30	7	14	21	28	4	11	18	25	2	9	16	23	30	6	13	20	27
Track courses	Refresher course - (P.Way) - 19601			Refresher Course (P.Way) - 19602						Refresher Course-(P.Way) - 19603			Refresher Course-(P.Way) - 19604				No Trg activities					Refresher Course (P.Way) - 19607									
Trainees	35	35	35	35	35	35				35	35	35	35	35	35							35	35	35							
Track courses								RWI - 19835	TMS - 19845			19861 TI-1								RWI - 19843	TMS - 19860										
Trainees								35	35			35								35	35										
Bridge courses													Refresher Course (Bridge) - 19605																		
Bridge courses																															
Trainees																															
Works courses										SURVEY - 19837	GEOTECH - 19838									Refresher Course (Works) - 19606		SURVEY - 19852	GEOTECH - 19853								
PSU courses																PSU - 19901								PSU - 19903							
Trainees																30								35							
TOT courses																TOT-19902									TOT - 19904						
Trainees																35									35						
Spare slots	NL TECHNI	19910 RVNL TECHNICAL CR B2				19911 RVNL TECHNICAL CR B3																									
Trainees	30	30	30	30	30	30	30	30	30																						

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