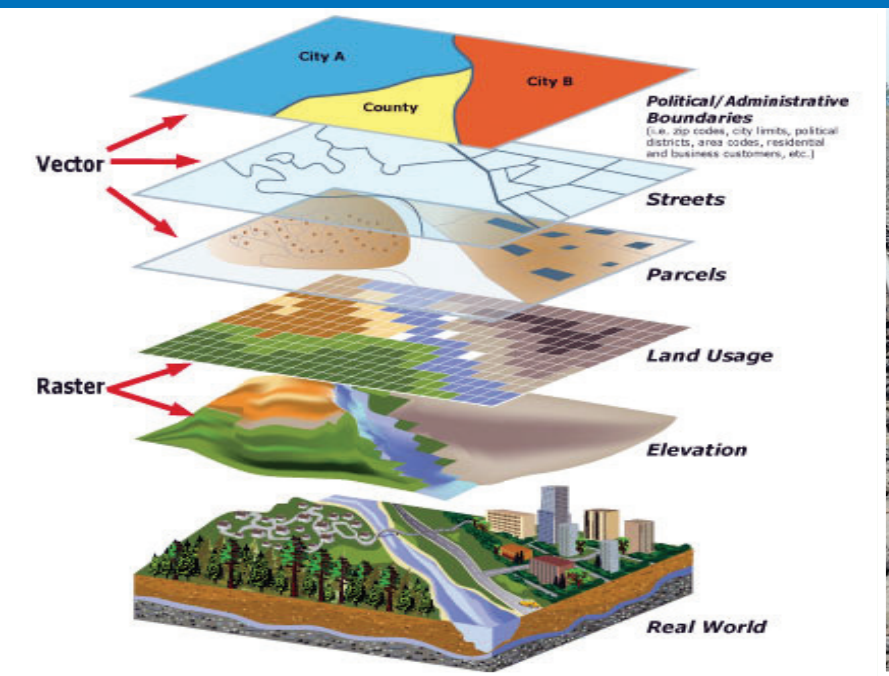
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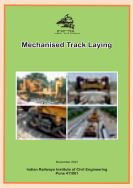


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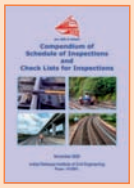
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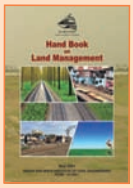
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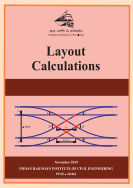
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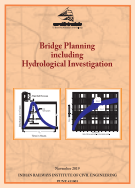
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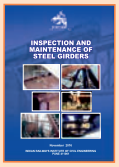
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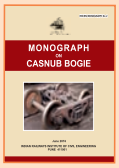
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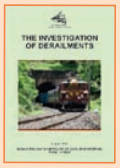
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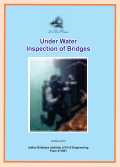
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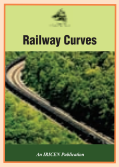
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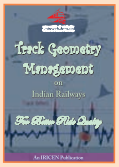
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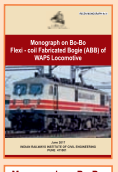
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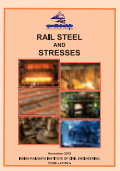
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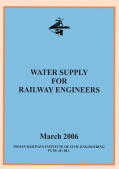
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From Director General's Desk

Dear Readers,

In this edition of IRICEN journal, papers have been included on diverse topics. There is a technical paper on Geographic Information System (GIS) for assessment of catchment area properties. Another paper is on 'TM Connect' software, to connect Track Machines operators to the higher level of management on real time basis. The paper has been included here for spreading the awareness amongst all Railway Engineers.

We have also included one paper dealing with the Design of Reinforced Concrete Pier Caps by Strut-and-Tie Method. Another paper is on Maintenance of Track on Bridge Approaches.

There is a paper on Prediction of Time Dependent Deflection, Camber and Prestress Loss in PSC bridges including a paper on Assessment of vehicle dynamics and its impact over track degradation.

I hope that the readers would find the papers and other articles contained in this edition relevant and useful. The suggestions and contributions of technical papers, news items, articles, etc. are welcome from field engineers for inclusion in the forthcoming issues of this journal which would lead to sharing of knowledge and experience amongst engineering fraternity.

Pune
November 2021

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Guidelines to contributors

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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.

Literature Digest

Field Testing and Dynamic Response of Full-Scale GFRP-Reinforced Concrete Guideway under Monorail Train

Abstract: In this study, field testing is conducted on two full-scale guideway reinforced concrete (RC) beams, one reinforced entirely by glass fiber–reinforced polymer (GFRP) and the other by conventional steel, under 450 passes of two-car monorail trains at various speeds and vehicle loads. The two beams were part of a 1.86-km test track traveled by trains for 4.5 months. The objective of the study was to evaluate GFRP bars as a sustainable reinforcement for transit infrastructure, particularly for monorail trains. The study showed that cracking and stiffness degradation in both beams occurred during train loading with one-loaded and one-empty cars, after 79–132 cumulative train passes. Cracks stabilized after 149 accumulative passes. Deflection in the GFRP-RC beam, measured during train passes at fully loaded cars, was 36%–47% higher than that in the steel-RC beam. After 200 accumulative passes, stiffness was reduced to approximately 43% and 57% of the original uncracked value for GFRP- and steel-RC beams, respectively. By the end of 450 passes, stiffness in the same beams appeared to have stabilized at 38%, and 50% of the uncracked value. The GFRP-RC beam satisfied all serviceability limits imposed in national codes. A twodimensional finite-element analysis was also performed on the test beams, showing good correlations with test results of deflections and cracking patterns. Of the six analytical equations that were used to estimate the effective moment of inertia and subsequently beam's deflection, only those by Branson and Bischoff seemed to yield acceptable predictions.

By : Nikolaus Wootton; Amir Fam, F.ASCE; Mark Green, M.ASCE; Akram Jawdhari, M.ASCE; and Salah Sarhat

Ref. : Journal of Rail and Rapid Transit : September 2021

Aerodynamic Behavior of a High-Speed Train with a Braking Plate Mounted in the Region of Inter-Car Gap or Uniform-Car Body: A Comparative Numerical Study

The continuous increase in train speed has brought serious challenges to train braking safety. Aerodynamic braking technology can effectively improve the braking effect of trains at high speeds. In this study, an aerodynamic braking device installed in the inter-car gap region (ICG) of a high-speed train is proposed and the aerodynamic performance of the high-speed train with an aerodynamic braking device is assessed by improved delayed detached eddy simulation (IDDES) based on the j-x turbulence model. The results show that the opening of the plate significantly changes the aerodynamic performance of the train, thereby greatly increasing the aerodynamic forces of the train and their fluctuation degree. The effect of the opening of the plate increases the turbulence of the downstream flow field around the tail car. The affected area is mainly concentrated in the flow field around the location of the plate for the pressure field and the whole flow field behind the plate for the velocity field. The effect of the plate mounted on the uniform-car body region (UCG) on increasing the aerodynamic drag is better than that at the ICG, though the aerodynamic fluctuation and the influence on the surrounding flow field will also be great.

By : Jiqiang Niu, Yueming Wang, Rui Li

Ref. : Journal of Rail and Rapid Transit – September 2021

A Numerical Study of Aerodynamic Characteristics of a High-Speed Train with Different Rail Models under Crosswind

The CFD (Computational Fluid Dynamics) numerical simulation method with the DES (detached eddy simulation) approach was adopted in this paper to investigate and compare the aerodynamic performance, pressure distributions of the train surface, and flow fields near the train model placed above the subgrade with non-rail, realistic rail, and simplified rail models under crosswind. The numerical methods were verified with the wind tunnel tests. Significant differences in aerodynamic performances of the train body and bogie were found in the cases with and without a rail model as the presence of the rail model had significant impacts on the flow field underneath the vehicle. A larger yaw angle can result in a more significant difference in aerodynamic coefficients. The deviations of the train aerodynamic forces and the pressure distribution on the train body with the realistic and simplified rail models were not significant. It was concluded that a rail model is necessary to get more realistic results, especially for large yaw angle conditions. Moreover, a simplified rectangular rail model is suggested to be employed instead of the realistic rail and is capable to get accurate results.

By : Zhiwei, Jiang, Tanghong Liu, Houyu Gu, Zijian Guo

Ref. : Journal of Rail and Rapid Transit – September 2021

Optimal Track Geometry Maintenance Limits Using Machine Learning: A Case Study

The aim of this study has been to determine the optimal maintenance limits for one of the main railway lines in Iran in such a way that the total maintenance costs are minimized. For this purpose, a cost model has been developed by considering costs related to preventive maintenance activities, corrective maintenance activities, inspection, and a penalty costs associated with exceeding corrective maintenance limit. Standard deviation of longitudinal level was used to measure the quality of track geometry. In order to reduce the level of uncertainty in the maintenance model, K-means clustering algorithm was used to classify track sections with most similarity. Then, a linear function was used for each cluster to model the degradation of track sections. Monte Carlo technique was used to simulate track geometry behavior and determine the optimal maintenance limit which minimizes the total maintenance costs. The results of this paper show that setting an optimal limit can affect total annual maintenance cost about 27 to 57 percent.

By : Ahmad Kasraei, Jabbar Ali Zakeri and Arash Bakhtiary

Ref. : Journal of Rail and Rapid Transit – September 2021

Experimental Investigation Into The Lateral Resistance of Y-Shape Steel Sleepers on Ballasted Tracks

Track lateral resistance ensures the track stability under operation and during its service life in lateral and longitudinal directions. Numerous methods and techniques have been considered to enhance track lateral resistance. Since sleeper, as one of the railway track superstructure components, has a significant contribution in providing the lateral resistance, using different types of sleepers, like Y-shape one, would alter the track lateral resistance. The utilization of Y-shape steel sleepers in railway tracks needs further investigation to comprehend the structural behavior. Hence, in this paper, the lateral resistance of the Y-shape steel sleeper was experimentally investigated by performing the lateral track panel loading test (LTPT) and single tie push test (STPT). The longitudinal resistance force (LRF) in STPT depends on the loading direction. The LRF was specified as 16 kN in Y-top direction and 13 kN in Y-bottom direction for a single sleeper. The LRF in STPTwas enhanced by 90% on average compared to the conventional concrete sleeper. The LRF in LTPTwas also obtained as 18 kN, and its increasing percentage was measured as 12.5%.

By : Jabbar Ali Zakeri, Yaser Bahari and Kaveh Yousefian

Ref. : Journal of Rail and Rapid Transit – September 2021

The Structural Integrity of Flash-Butt Welded Premium Rail Steel – Evaluation of Strength, Microstructure and Defects

Experimental testing of flash-butt welded premium rail steel samples was undertaken to quantify the variation in strength through the cross-section of the weld, with results showing a general correlation between the width of the heataffected-zone and the ultimate tensile strength. Microstructural examination and fractography revealed the presence of defects in some samples. Large defects can be identified by non-destructive testing methods as part of routine structural integrity assessment; however, small defects may escape detection but could propagate under cyclic loading in service leading to catastrophic failures. This work demonstrates the requirement for stringent control of heat input to prevent the development of defects in premium rail steels with high alloying content.

By : Anthony Micheletto, John Cookson, Yong Pang, Bernard Chen and Peter Mutton

Ref. : Journal of Rail and Rapid Transit – September 2021

Analysis on the Track Quality Evolution Law of Polyurethane-Reinforced Ballasted Track In High-Speed Railway

Polyurethane-reinforced ballasted track (PRBT) can improve the integrity of ballasted track structures and satisfy the high-stability requirements of high-speed railways. In this study, the quality evolution law of PRBT structures after being launched into train service was analyzed, and a reference for structural optimization and maintenance operation was provided. The track geometric state of the PRBT test section of a high-speed railway was measured and monitored for nearly one year after it was launched into operation, and the ballastless track of the adjacent section was selected as a reference. The geometric states of the tracks were evaluated and compared using various parameters, including sliding standard deviation, average standard deviation, and track irregularity spectrum density.

Results show that the track quality indexes of the test section, which were in operation for nearly one year, were slightly over the limit. Moreover, the fastener can be finely adjusted for the high-value index sections to further improve the ride comfort. The application effect of PRBT in the subgrade fracture zone was good, which could satisfy the requirements of high-quality transportation as well as the normal operation and maintenance of high-speed railway.

By : Jie-Ling Xiao, Pu Jing, Si-Xin Yu and Ping Wang

Ref. : Journal of Rail and Rapid Transit – September 2021

Repair & Management of Deteriorated Concrete Structures: New Dmrb Document & Specification

The repair and management of Britain's concrete highway structures has significantly improved since the publication of specific guidance in 1990 in the UK Design Manual for Roads and Bridges (DMRB) This guidance was updated in 2020 to include an improved approach to managing deteriorated structures. This paper focuses on the additional requirements or significant modifications in the updated guidance and explains their rationale. Key changes include the overall management strategy of deteriorated structures, repair strategies and deterioration mitigation measures, including control of the incipient anode effect and concrete injection to improve the safety and durability of a repaired structure. Internal deterioration of concrete and thaumasite sulfate attack are also covered. The related UK Manual of Contract Documents for Highway Works also provides an updated specification for concrete repairs. Predominantly performance-based, it covers proprietary concrete repair products complying with British standards and other materials and construction methods for maintenance, repair or strengthening.

By : Jessica Sandberg, Chris Mundell, Homayoon Sadeghi Pouya and David Hickman

Ref. : ICE Bridge Engineering – September 2021

Shear Friction and Strut-and-Tie Modeling Verification for Pier Caps

Abstract: This study reports on experimental test results of reinforced concrete pier caps with different shear spans to effective depth ratios (a/d) of 0.5, 1, and 1.5. Each test specimen is then designed theoretically using both shear friction (SF) and strut-and-tie modeling (STM) approaches, according to Section 16.5 and Chapter 23 of ACI 318-14, respectively, and the results are compared with the pier caps experimental test results. The cracking load, failure load, deflection, crack pattern, crack width, steel reinforcement strains, concrete surface average strains, and failure modes are observed, recorded, and discussed. The experimental load capacities are compared with the theoretical load capacities of SF and STM. Experimental test results indicate that both STM and SF are conservative approaches and STM is more conservative than SF. The reason for this is because they do not take secondary reinforcement into direct consideration. That is why, a model is proposed, modifying STM, for estimating the ultimate capacity of pier caps based on calculating the strength of concrete and secondary reinforcement separately that gave more realistic results.

By : Asala Asaad Dawood and Khattab Saleem Abdul-Razzaq

Ref. : ASCE Bridge – September 2021

Half-Joint, Deck Hinge and Post-Tensioned Concrete Bridges: Risk Management and Assessment

The documents for risk management and assessment of concrete bridges with half-joints, concrete bridges with deck hinges and post-tensioned concrete bridges in the Design Manual for Roads and Bridges (DMRB) have been rewritten and updated as part of a project to update the entire DMRB suite of documents; they have also been given the new document codes CS 465, CS 466 and CS 467. In bringing these documents up to date there have been significant technical changes introduced, including: a change to the risk management process for half-joints and deck hinges; an update of assessment methods for evaluating half-joints and deck hinges; and improvements to the risk management of post-tensioned bridges. This technical paper outlines the technical background to the key changes and highlights some of the improvements and efficiencies that can be delivered using the new documents.

By : Eleanor McDaid, Guan Sing Loh, Jon Shave, Donald Pearson, John Bennetts and Pierfrancesco Valerio

Ref. : ICE Bridge Engineering – September 2021

Using Community Data to Improve Rural Water Infrastructure in India

India continues to struggle with the combined effects of water stress alongside poor access to water and sanitation. However, data-driven infrastructure planning is helping to bring a more integrated, sustainable approach to water management and the provision of water, sanitation and hygiene in rural areas. Working

with non-governmental organisations in Chhattisgarh and Madhya Pradesh, a project was undertaken to introduce consistent digital data collection, analysis and sharing processes for water resources and water demand data. Key outputs from the project included a toolkit and mobile application, the wider adoption of which could transform rural water access.

By : Hamish Hay and Philip Songa
Ref. : ICE Civil Engineering – September 2021

Procurement and Technological Innovations of India's New Dedicated Freight Railways

Indian Railways is investing INR 814.590 billion (£8.15 billion) in two new dedicated freight corridors to decongest its heavily used national railway network. Due for completion in 2022, the new 1839 km eastern corridor runs fro. DFCCIL, Delhi to Kolkata while the 1499 km western corridor, capable of carrying double-stack containers, link. DFCCIL, Delhi to Mumbai. This paper describes the procurement process and the new technologies being adopted on the project. These include the first use in India of 32.5 t axle loads, continuous welded track, twin 25 kV overhead electrification, and automatic signalling and centralised train control for 100 km/h line speeds. The project is expected to shift a significant amount of freight transport from road to rail, saving an estimated 455 Mt of carbon dioxide emissions over 30 years of operation.

By : Kamal Kishor Sharma and Veer Narayan

Ref. : ICE Civil Engineering – September 2021

Making A Film to Share Traditional Engineering Techniques to Tackle The Global Water Crisis

Civil engineers in the UK have teamed up with a water charity to make an educational film that helps communities in semi-arid climates manage their water. The team learnt how to construct simple dams, known in India as 'johads', that harvest seasonal rainfall, reducing flood and drought risk. They then built a new dam and restored another in Rajasthan district, recording the process to enable others to replicate the work. A language-neutral educational film was then produced and tested with other communities in India and Colombia to ensure applicability to a variety of geographies, cultures and environments. Educating communities on water management in this way will help to achieve the United Nations sustainable development goal 6: clean water and sanitation for all.

By : Louise Bingham and David Hetherington

Ref. : ICE Civil Engineering – September 2021

Design and Construction of Mumbai Metro Line 3 in India

Completion of the US$3.6 billion Mumbai Metro Line 3 in December 2021 will increase the system's capacity five-fold to 1.6 million passengers a day. With 26 underground stations and one depot, the new 33.5 km line will significantly improve connectivity between the northern and southern areas of India's largest city. It is being built directly under temples, residential areas and an airport. Close collaboration with stakeholders contributed to successful delivery and launch of the project's 17 tunnel boring machines. For rail systems, innovative technologies have been adopted to facilitate redundancy and ensure the safety and comfort of passengers.

By : Mohana Killada, Raju V. R. Gottumukkala and Ankush Tikhe

Ref. : ICE Civil Engineering – September 2021

Vehicle-Based Cryogenic Rail Cleaning: An Alternative Solution to 'Leaves on The Line'

Contamination of rail lines with leaves and other organic matter and oxides can affect the traction of train wheels and cause safety issues, delays and schedule changes. The main solution in the UK is to use specialist rail-head-treatment trains that clean rails with high-pressure water jets. But these trains cannot cover all UK mainline infrastructure due to limited availability and gauging issues. As such, there is a need for a rapid-response mobile solution to expand this capability. This paper reports on successful field trials of a small road-to-rail vehicle fitted with dry-ice blasting equipment, and the effect this has on both cleaning rails and improving train braking.

By : Peter Krier, Ben T. White, Paul Ferriday, Mike Watson, Luke Buckley-Johnstone, Roger Lewis and Joseph L. Lanigan

Ref. : ICE Civil Engineering – September 2021

Use of Dimension Limestone Sludge as Filler in Asphalt Mix

This study investigates the feasibility of using waste dried sludge dust from the Indian dimension limestone (namely, Kota stone (KS)) industry in the place of conventional stone dust as filler in asphalt concrete mix. First, physical, morphological and chemical characteristics of both fillers were determined by

relevant standards. Second, asphalt concrete mixes including both fillers were designed and their performance against primary occurrences of distress (rutting, cracking, long-term ageing, ravelling and moisture susceptibility) was evaluated. The KS mix had lower optimum asphalt content than conventional mixes, which was attributed to the lower porosity of KS dust. KS mixes also displayed superior resistance against permanent deformation, cracking and ravelling due to the finer nature of KS. Although the KS had a relatively high active clay content, KS mixes displayed good active and passive adhesion as well as satisfactory resistance against moisture permeation.

By : Jayvant Choudhary, Ankit Gupta and Brind Kumar

Ref. : ICE Construction Material – September 2021

Bonding Agents to Mitigate Interfacial Strength Loss During Multi-Layer Concrete Casting

Self-consolidating concrete (SCC) is very sensitive to interruptions or delays during the placement process. Current literature converges that interfacial bond strength between successive casting lifts could drop by up to 60%, compared with monolithic casting. In this study, two series of SCC mixtures were tested to assess the suitability of bonding agents to mitigate the drop in interfacial bond strength resulting from successive casting lifts. The slabs were cast in two layers separated by 60 or 120 min rest intervals; different bonding agents, including styrene– butadiene rubber (SBR), waste latex paint and cement/water and cement/SBR slurries were compared. The results showed that the application of polymeric bonding agents on top of the first cast is efficient to improve tensile strength and restore the integrity of the hardened member. This is attributed to the polymer particles that coalesce and fuse to form continuous close-packed films between the first and second SCC layers. The cement/water slurry did not lead to considerable improvement in pull-off strength between cast layers. Series of charts enabling the prediction of the drop in bond for a given SCC composition and rest interval as a function of the polymer dosage to the area of the cast slab are presented.

By : Joseph J. Assaad and Yehia Daou

Ref. : ICE Construction Material – September 2021

Interaction Between Infill Walls and Reinforced Concrete Frames After Column Removal

The progressive collapse resistance of reinforced concrete (RC) frames in which a middle column has been removed has been investigated by many researchers. However, previous experimental and computational studies have focused mainly on the catenary action provided by longitudinal reinforcement in the beams spanning the missing column. This study investigated the contribution of infill walls. Finite-element analysis was used after calibrating a reference model using test results reported by other researchers. Models were developed for a RC frame with and without infill walls. An equivalent strut was used in the infill walls to allow the removed column load to be transferred through the struts instead of developing catenary action in the beams. A simplified model of the progressive collapse resistance of frames with infill walls was then derived to estimate the peak strength provided by the infill wall under the scenario of middle-column removal.

By : ICE Structures and Buildings – September 2021

Ref. : Amer Alkloub and Rabab Allouzi

Towards Efficient Structural and Serviceability Design of High-Strength Concrete T-Beams

Ten high-strength concrete (HSC) T-beams of total length 1700 mm, web breadth 100 mm, height 250 mm and different flange dimensions were experimentally tested until failure. The beam parameters studied were the flange dimensions and transverse and longitudinal reinforcement ratios. A numerical model was developed to predict the flexural behaviour of the tested beams. The results indicated that increasing breadth, depth and flange reinforcement ratio led to a reduction in the overall deflection and an enhancement of the ultimate load capacity. An empirical equation developed earlier by the authors for predicting the effective moment of inertia of HSC T-beams was successfully used to predict the load–deflection relationships of the studied beams. The load–deflection curves predicted by the numerical model, those predicted by this equation and those predicted by the Branson equation, which is currently used in design codes, were compared. Close agreement was found between the experimental results, numerical results and those obtained by the authors' equation. This research thus highlights the need for updating the Branson equation used in design codes for deflection design of HSC T-beams by adopting the equation proposed by the authors.

By :Ibrahim G. Shaaban and Tarek S. Mustafa

Ref. : ICE Structures and Buildings– September 2021

Using Geographic Information System (GIS) for Assessment of Catchment Area Properties

*R. K. Shekhawat**

SYNOPSIS

The conventional method which was hitherto mostly being used in Indian Railway for assessing Catchment Area Properties (i.e. using Topo Sheet, marking catchment area and water streams on the topo sheet and then working out the Catchment Area Properties) suffer from many limitations and disadvantages. This can be improved upon by using Geographic Information Systems (GIS) and other Information Technology (IT) tools, for assessing the Catchment Area Properties in a faster and more accurate manner. This Technical Paper describes one such methodology/approach which has been developed by the author using a GIS Software (QGIS) and other IT Tools/Software platforms, available freely, for assessing Catchment Area Properties.

1 Need for Assessing Catchment Area Properties

1.1 As part of planning a bridge over any water stream (for deciding width of waterway, span arrangements, type & depth of foundation etc.), one of the most relevant parameters is Design Discharge (for a specified Return/ Recurrence Period) passing through the water stream. Sometimes, the Design Discharge may be required to be estimated for exiting bridges also, if there are reports of water overflowing the bridge or breaches etc.

1.2 The most preferred way for estimating design discharge for a waterway shall be based on actual hydrological observations at the bridge location and meteorological observations in the concerned catchment area. But, these hydro-meteorological details, that too for the required Recurrence Period (which is of the order of 50 to 100 years), may be available only for very few important water streams and that too at very few locations. At other locations, if flow records (yearly peak discharges) are available for less than the desired recurrence interval, but of sufficient number of years to permit reliable statistical analysis, the design discharge can be computed statistically for the desired recurrence interval. Where flood records are not of sufficient number of yrears to permit reliable statistical analysis, but rainfall pattern & intensity records are available for sufficient length of time and it is possible to carry out at least limited observations of rainfall and discharge, Unit Hydrographs can be developed based on such observations and design discharge of the desired recurrence interval can be computed by applying appropriate design rainfall on them.

1.3 But in a very large number of cases, the details mentioned in Para-1.2 are also not readily available. In such cases, there are various approaches/ methodologies for estimating the Design Discharge in a water stream at any given point. The Methodologies specified in Indian Railway "Bridge Substructures and Foundations Code", which are similar to the approach followed by other organizations also, is as under:

(a) For small size catchment (less than 25 km^2 area): Design discharge is estimated based on methodology given in RDSO Report No. RBF-16, "Flood Estimation Methods for Catchments Less Than 25 Km^2 in Area", using "Modified Rational Formula". The input parameters required for this formula are Runoff Coefficient "C" (which depends on the type of catchment area and the values are given in RBF-16), Rainfall Intensity/Pattern Records (from the details given in RBF-16), Catchment Area "A" and Length-Elevation Profile of the water stream from the bridge to the farthest point on the catchment.

(b) For medium size catchment (25 km^2 or more but less than 2500 km^2 area): Synthetic Unit Hydrograph (SUH) is plotted using the equations given in "Flood Estimation Reports" published by Central Water Commission (CWC), for the concerned hydro-meteorological sub-zone. Subsequently, design discharge is computed by imposing the design storms (rainfall) on the concerned Synthetic Unit Hydrograph. The input parameters required for this are Catchment Area "A", Centroid of the Catchment Area, Length-Elevation Profile of the water stream from the bridge to the farthest point on the catchment and Rainfall Intensity/Pattern Records (from the

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CWC's Flood Estimation Reports).

1.4 Therefore, in most of the cases, the properties of the catchment area (i.e. Catchment Area "A", water stream profile, Centroid of the Catchment Area and Length-Elevation Profile of the water stream from the bridge to the farthest point on the catchment) are required to be assessed for estimating the design discharge. Hitherto this was mostly being done by the conventional method wherein the Topo Sheets in hard copy were used for marking catchment area on them and then working out various Catchment Area Properties. This method suffers from many limitations and disadvantages, as under:

(i) Extracting topographic features from Topo Sheets is very cumbersome and time-consuming process, especially when large size catchments are involved.

(ii) Quantifying the flow in the water stream is not possible. Thus, it is not possible to delineate only those contributories (to the water stream) which have significant flow.

(iii) Calculation of catchment area, length of water stream, centroid of catchment and the length- elevation profile of the water stream is cumbersome and not very accurate.

(iv) It requires a very experienced person.

(v) Visual representation of features being examined is not available.

1.5 To overcome these limitations and bring further improvements, a methodology/approach has been developed by the author using GIS based Software and other IT Tools/Software platforms, available freely, for assessing Catchment Area Properties.

2 Geographic Information System (GIS):

Geographic Information System (GIS) can be defined as a system designed to capture, analyze, store, manipulate, present and manage all types of geographical data. In the present time, with easy and cheap availability of very high data processing speeds and storage capacities, GIS based solutions are being used for multifarious activities. The Geographical data in a GIS is stored in various forms as shown in *Fig.01*.

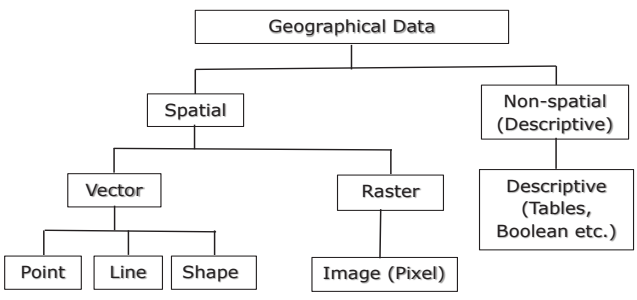


Fig. 1 . Forms of data in GIS

Various forms of data in GIS are stored in the form of layers as shown in *Fig.02*.

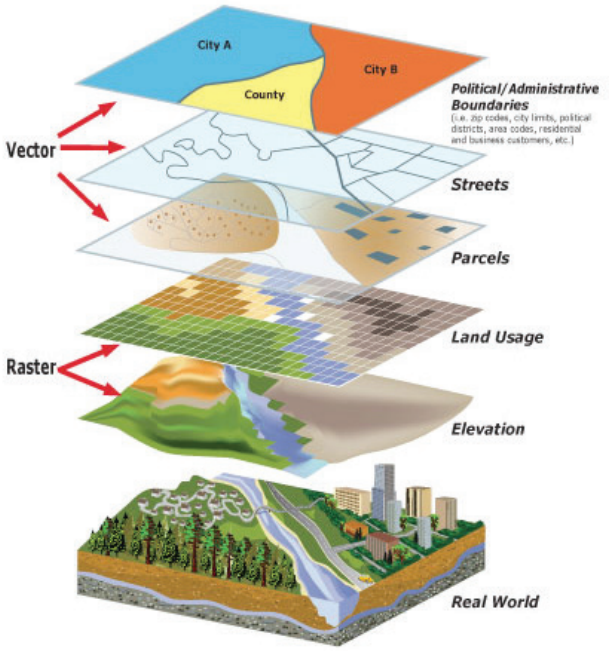


Fig. 2 . Various layers of data in GIS

There are many software for analyzing, storing, processing, presenting and managing the data in GIS environment (e.g. QGIS, ArcGIS and RemoteView etc.). In the present methodology QGIS (Version 3.12.3 "Bucuresti" or later) can be used which is a free and open-source software.

3 Steps involved in the proposed methodology

The steps involved in the present methodology, which are elaborated in the subsequent paras, are as following:

(i) Identify/confirm the Bridge Location

(ii) Download DEM(s) for the Catchment Area

(iii) Process the DEM(s)

(iv) Derive the Water Streams

(v) Delineate the Catchment

(vi) Find the Area of Catchment (A)

(vii) Mark the Centroid of the Catchment

(viii) Measure L (Length of Longest stream)

(ix) Measure LC

(x) Plot Length-Elevation Profile of Longest Stream

The details elaborated in the subsequent paras, as an example, pertain to the following bridge:

- Subzone - 2(A): North Brahmaputra Sub-zone
- Alipurduar - Bongaigaon Section, Alipurduar Division, N. F. Railway
- Name of Tributary: Gangia
- Bridge Location given: Longitude – 90^0 07'

10” E (90.11944444 E)

Latitude – 26^0 22’ 10” N (26.38611111 N)

4 Identify/confirm the Bridge Location

4.1 The location of bridge is generally denoted in Longitude and Latitude (in Degrees, Minutes and Seconds). Convert them to real numbers with decimals (up to 8th decimal place or so). By putting this location, the location of bridge, railway track and other surrounding features can be seen in “Google Earth”. This gives a feel of bridge location which is not possible in conventional method. If some refinement is required in bridge location, it can be done and the correct Longitude and Latitude of the Bridge can be noted (Fig.03). For example, in the present case, the refined location is Latitude: 26^0 23’ 19.27” (26.38868611 N) and Longitude: 90^0 06’ 50.05” (90.11390278 E).

4.2 Convert these Geographic Coordinates to UTM (Universal Transverse Mercator) Coordinates in WGS84 (World Geodetic System) datum. This can be done using “https://awsm-tools.com” or another similar website. In the present case, the UTM coordinates are- X Coordinate (UTM E): 212049.06540, Y Coordinate (UTM N): 2921954.6790 and UTM Zone: 46N. Note them for further reference.



Fig. 3 . Bridge location on Google Earth

5 Download DEM(s) for the Catchment Area

5.1 Digital Elevation Model (DEM) is a 3D representation of terrain’s surface. They are created from terrain’s elevation data and can be generated by any one of the following methods:

(a) Ground Survey of the terrain

(b) Differential Global Positioning System (DGPS) Survey of the terrain

(c) Digitization of Contour Maps

(d) Stereo Photogrammetry

(e) LiDAR Survey

(f) Satellite Images

5.2 DEMs of 1 Arc-second accuracy (about 30m distance at equator) can be downloaded, free of cost, from any of the following sources:

(a) Cartosat-1 satellite data from Bhuvan Site [https://bhuvan.nrsc.gov.in] of ISRO. In the present case, the DEMs were downloaded from this source.

(b) Earth Explorer Site [https://earthexplorer.usgs.gov]

(c) Shuttle Radar Topography Mission (SRTM) Downloader of NASA. This is available as a Plugin in QGIS Software also.

(d) Advanced Spaceborne Thermal Emission and Reflection Radiometer (ASTER) platform developed jointly by Japan and NASA.

5.3 Open the Bhuvan website. On the 2D Data Tab, the location of Bridge can again be confirmed, along with water stream *(Fig.04)*. See the bigger Catchment Area for the bridge location (shown in dark blue colour) and note down the coordinates of 4 extreme points of this catchment area, as under:

	90.01E, 27.74N	
89.89E, 26.71N		90.22E, 26.45N
	90.12E, 26.32N	



Fig. 4 . Bridge and Water Stream locations on Bhuvan Website

5.4 Open the Data Archive Tab on the Bhuvan website and select the DEM(s) covering bigger Catchment Area for the bridge location (shown in dark blue colour), in terms of Longitudes and Latitudes (26N to 28N and 89E to 91E in this case). Download the DEM tile(s) covering this range (Toposheet nos. G45L, G46G, G45F and G46A in this case, Fig.05). Each DEM tile will be a Zip file, from which GeoTiff (.tif) file may be extracted for further use.

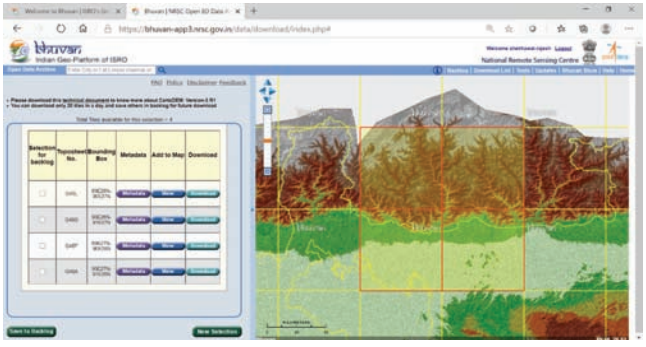


Fig. 5 . Downloading DEM tiles from Bhuvan Website

6 Process the DEM(s)

6.1 Load DEMs: Open the QGIS software and load Raster files of the concerned DEM Tiles (.tif files). Load "Open Street map (OSM) Standard" also as a background, from the "Quick Map Services" of QGIS *(Fig.06)*.

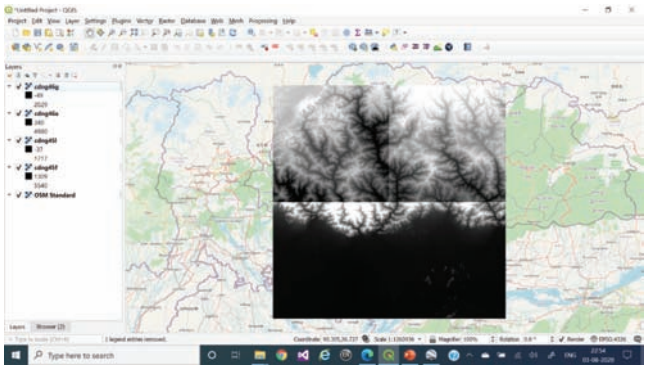


Fig. 6 . DEMs loaded in QGIS

6.2 Mosaic DEMs: Mosaic (combine) the DEM tiles and save the file/layer (named "DEM_Mosaic" in this case) as a virtual file (.vrt) as shown in *Fig.07*.

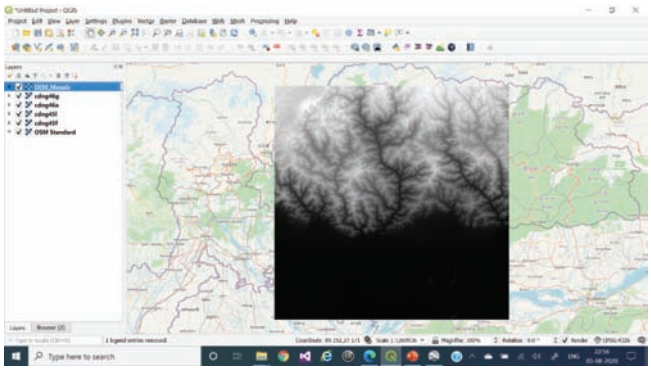


Fig. 7 . "DEM_Mosaic" layer in QGIS

6.3 Re-project DEMs: Every DEM is related to a datum or projection level e.g. WGS84 datum in this case. For every datum, there is an EPSG (European Petroleum Survey Group) code, which is used to bring all the DEMs on a common global datum/projection platform. For WGS84, the EPSG code is EPSG:4326. The DEMs are reprojected to the EPSG code of the UTM Zone in which the bridge is located. The EPSG code for any location can be found from the website "http://www.spatialreference.org". For the example in this case, the UTM Zone is 46N and the EPSG code for this UTM zone is EPSG:32646. The DEMs loaded are in EPSG:4326 code (WGS84 datum). Re-project these DEMs to EPSG:32646 (file named as "DEM_Reprojected" in this case) as shown in *Fig.08*.

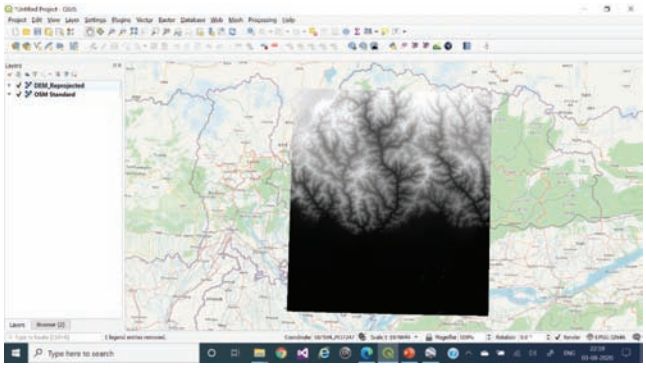


Fig. 8 . "DEM_Reprojected" layer in QGIS

6.4 Subset (Clip) the DEM: In order to reduce the processing time, the area of the reprojected DEM (which is bigger than the catchment area of the water stream) is clipped. The reprojected DEM is also made a vertical rectangle in shape, as shown in Fig.09, and the clipped raster file is saved (named as "DEM_Subset" in this case).

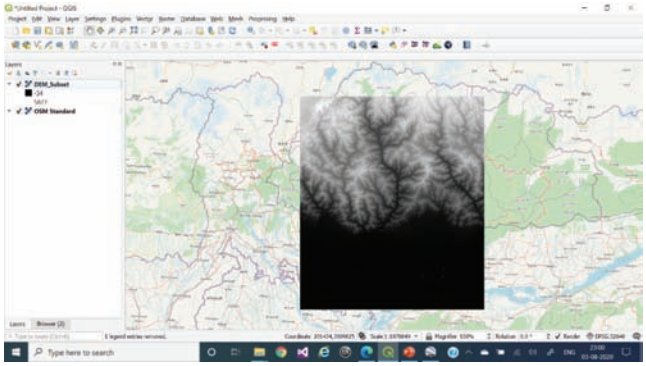


Fig. 9 . "DEM_Subset" layer in QGIS

6.5 Fill Sinks/ Remove Spikes: The DEMs contain some features like depressions (sinks) or spikes, due to data acquisition or processing process. These features need to be removed before the DEM can be used for any hydrological analysis. In QGIS, this is done using one of the algorithms from the "Toolbox". The algorithm developed by Wang and Liu (2006), which fills the localized depressions and preserves a downward slope along the flow path, is found to be most useful. The filled DEM raster layer is saved (named as "DEM_Filled" in this case) and then styled. Styling is basically a process of coloring to add 3D effects in a 2D picture. The styled DEM_Filled raster layer will be as shown in *Fig.10*.

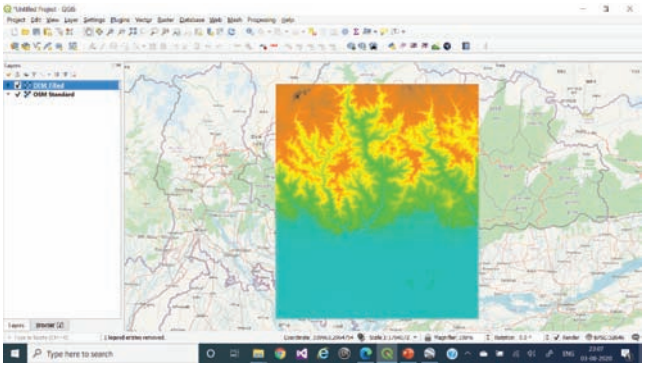


Fig. 10 . Styled "DEM_Filled" layer in QGIS

7 Derive the Water Streams

7.1 Strahler Order: As there are numerous tributaries/streams with water flow ranging from trickle to heavy flow, it needs to be determined as to what shall be considered as a stream. For this purpose, the concept of "Strahler Order" is used. The higher is this order, the bigger the stream. In QGIS, this is done using the "Strahler Order" tool from the "Toolbox". After suitable styling, the Strahler Oder raster layer (named as "Strahler" in this example) will look like as shown in *Fig.11*.

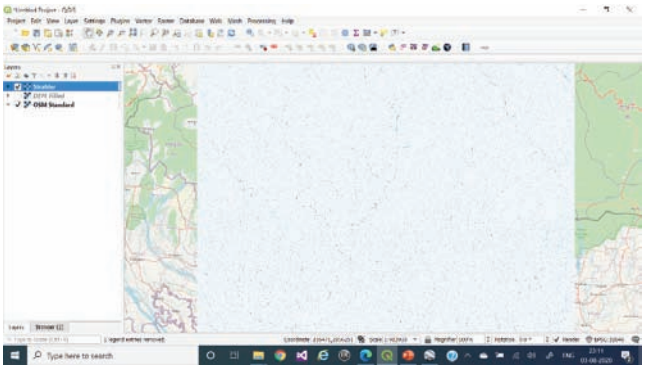


Fig. 11 . "Strahler" layer in QGIS

7.2 Threshold Strahler Order: The Strahler layer will show numerous water streams, thereby filling the complete area with blue colored water streams, and it becomes very difficult to make anything useful out of it. Therefore, the next step is to apply a calibration procedure to determine which Strahler Orders are to be considered as water streams. For this, using "Raster Calculator" command of the QGIS, Boolean layers are created showing water streams equal to or larger than a threshold Strahler Order only. The threshold Strahler order can be started from 2 or 3 and increased till the water streams in the Strahler Order layer correspond best with the water streams in "Open Street Map" displayed in the background. This Strahler Order layer is saved after suitable styling (named as "Strahler5", for threshold value of 5, in this example). The Strahler5 layer shows only those water streams which have Strahler Order of 5 or more *(Fig.12)*.

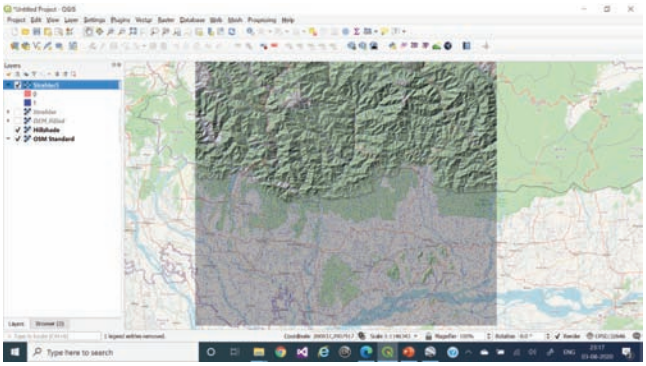


Fig. 12 . "Strahler5" layer in QGIS

7.3 Derive Channels Network and Drainage Basins: The next step is to derive the Channels Network and Drainage Basins, based on the derived threshold Strahler order (5 in this case). This is done using "Channels Network and Drainage Basins" tool in the "Toolbox". The Channels Network file (.shp shape file), containing all the flow channels (including outside the relevant catchment area also) is saved after suitable styling (named as "Channels" in this example, *Fig.13*). The thickness of the streams in this layer correspond to the Strahler Order of that stream.

The Drainage Basin file (.shp shape file), containing all catchment areas/drainage basins (including the portion which is not contributing to the flow at the bridge location also) is saved after suitable styling (named as "Drainage Basins" in this example, *Fig.14*). Different Drainage Basins (or a part of them) are shown in different colours.

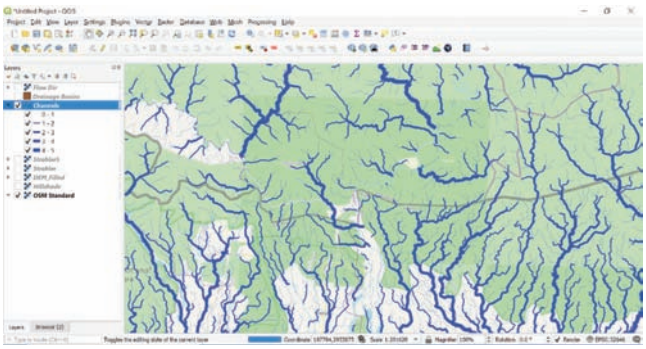


Fig. 13 . Styled "Channels" layer in QGIS

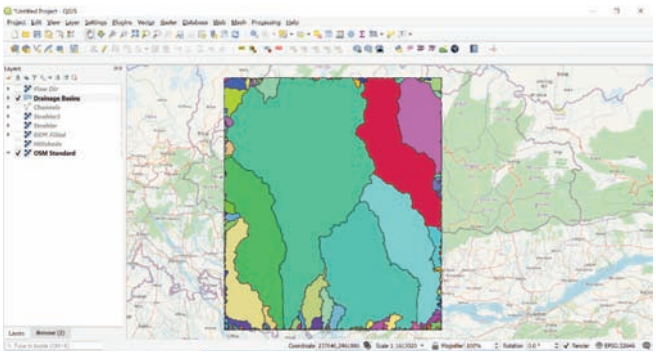


Fig. 14 . Styled "Drainage Basins" layer in QGIS

8 Delineate the Catchment

8.1 Define Outflow Point: To delineate the Catchment Area which is contributing the flow to the bridge location, it is necessary to first define the bridge location (Outgrow Point). Using the UTM coordinates of the bridge as guide, zoom the "Channels" layer to maximum extent and confirm the location of Outflow Point on it *(Fig.15)*. Using "Coordinate Capture" plugin, capture the UTM Coordinates of this location. The coordinates of the bridge location should be captured from "Channels" layer only and not from OSM Map shown in background. There may be some minor variation from the coordinates decided initially (in this example X: 211779.779 and Y: 2922051.340).

8.2 Delineate the Catchment: The coordinates of the bridge location are used to delineate the upstream area (i.e. catchment) that contributes to the water flow at this point. This is done using "Upslope Area" tool from the "Toolbox" and the derived shape file (.shp file) is saved (named as "Catchment" in this example). For overlaying the catchment boundary with other data, this file is converted to vector (polygon) format, styled suitably and saved as shape file (.shp file, named as "Catchpoly" in this example) as shown in *Fig.16*.

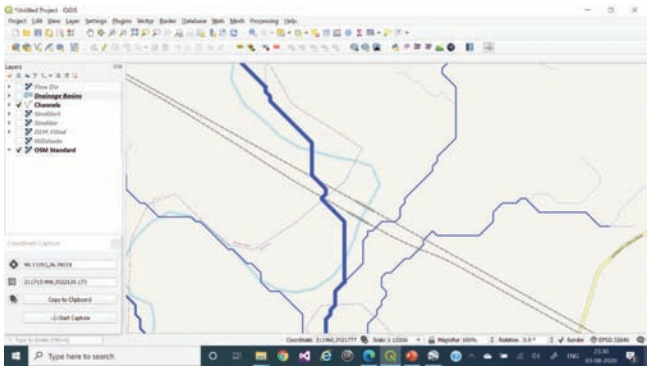


Fig. 15 . Capturing "Outflow Point" in QGIS

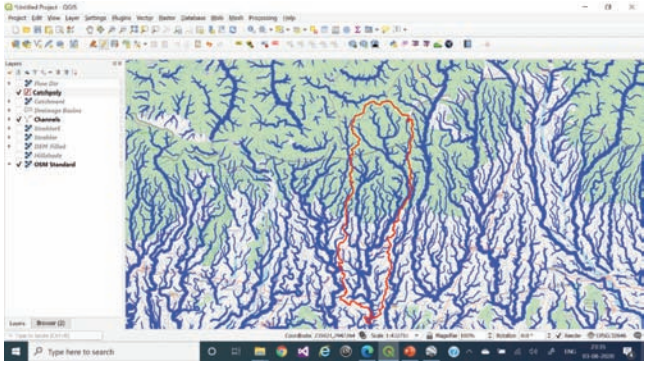


Fig. 16 . "Catchpoly" layer in QGIS

As the "Catchpoly" layer contains all the water streams, including those outside the catchment area contributing to flow at the bridge location, it is clipped to only show the water streams which are inside the relevant catchment area. The clipped shapefile layer is styled and saved (named as "Channels_Clipped" in this example, *Fig.17*).

The "DEM_Filled" layer is also clipped similarly (to contain the data for the relevant catchment area only), styled suitable and saved (named as "DEM_Clipped" in this example).

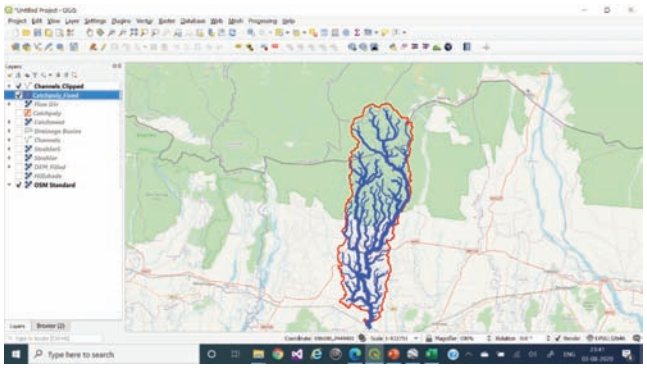


Fig. 17 . "Channels_Clipped" layer in QGIS

9 Find the Area of Catchment (A)

The area of "Catchpoly" shapefile can be determined directly using the "Field Calculator" of QGIS. This is Area of Catchment (A) for the given bridge location. In present example, it comes out as 549000216.40 m^2 C:\Users\cc1863\Downloads\3B_Lower Narmada and Tapi\Screen Shots\SS_20_Area.png or 549 km^2.

10 Mark the Centroid of the Catchment

The Centroid of the Catchpoly shapefile can be obtained directly in QGIS and the shape file layer is saved (named as "Centroid" in this example, *Fig. 18*).

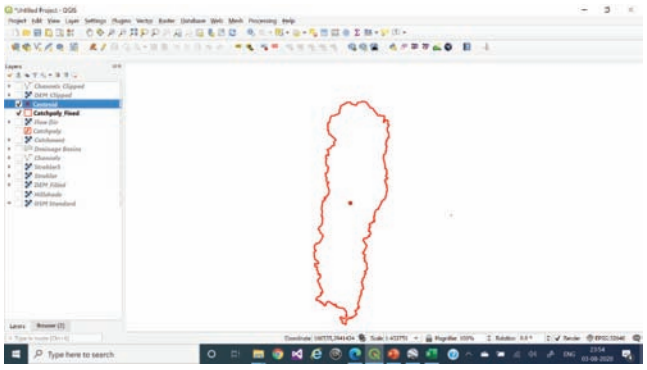


Fig. 18 . "Centroid" layer in QGIS

11 Measure L (Length of Longest stream)

Using the "Selection Tool" of QGIS, the segments forming longest stream are selected from the "Channels_Clipped", starting from the farthest point of the catchment area and tracing it up to the bridge location. These segments are exported as ESRI shapefile, styled suitably and saved (named as "L Stream_Segments" in this example). Using the "Dissolve" geoprocessing tool of QGIS, these segments are merged in one layer, which is then suitable styled and saved (named as "L Stream_

Final" in this example). This is the longest stream from the bridge location to the farthest point on the catchment area (Fig.19). The length of this water stream (L) can be determined directly using the "Field Calculator" in QGIS. In present example, it comes out as 76.559 km.

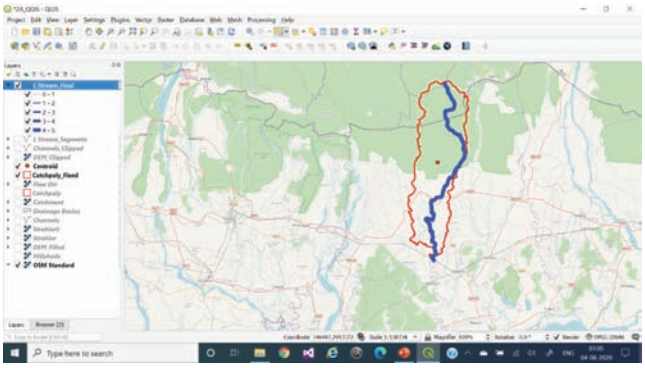


Fig. 19 . "L Stream_Final" layer in QGIS

12 Measure LC

12.1 LC is the length of longest stream from the bridge location to the point nearest to the centroid of the catchment area.

12.2 From the centroid of the catchment area, draw a straight line in such a way that it intersects the longest stream and length of this line between the centroid and the intersection point is least. This line is saved as a shapefile layer (named as "CG Line Layer" in the present example).

12.3 Split the longest water stream in two parts, on both sides of its' intersection point with "CG Line Layer", using "Vector Overlay–Split with Lines" tool of the "Toolbox" in QGIS. Delete the portion from the intersection point to the farthest point on the catchment area. The second portion, from intersection point to the bridge location, is "LC Stream". Style this layer suitably and save it (named as "LC Stream" in the present example, *Fig.20*).

12.4 The length of this water stream (LC) can be determined directly using the "Field Calculator" in QGIS. In present example, it comes out as 40.507 km.

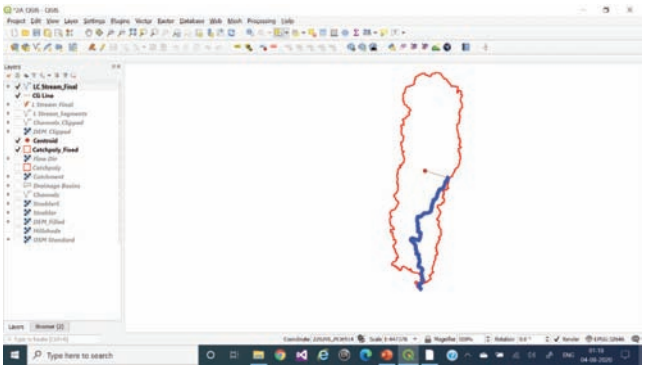


Fig. 20 . "LC Stream" layer in QGIS

13 Plot Length-Elevation Profile of Longest Stream

13.1 Using the "Profile Tool" Plugin of QGIS along with the "DEM_Clipped" layer and "L Stream Final" layer, the Length_Elevation Profile of the longest stream can be plotted. This can be saved also as a picture file (.png) for reference (named as "Elevation_Profile" in the present example) as shown in Fig.21.

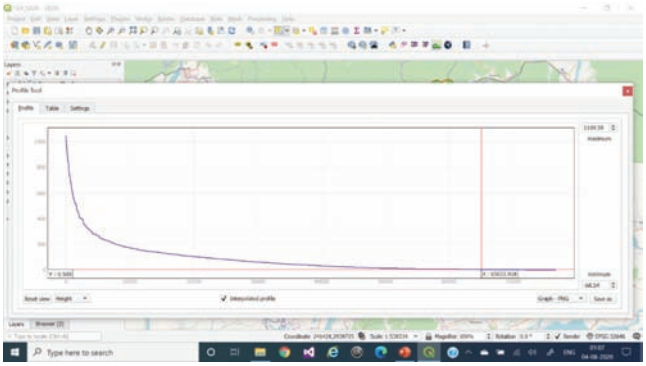


Fig. 21 . "Elevation_Profile" layer in QGIS

13.2 The "Length of Stream from farthest point on the catchment area to "any Point" & "Elevation at that Point" can be obtained by keeping the cursor at that point. These two parameters are noted down for all the points on this plot where the value of slope of the stream is changing significantly, and using this data, the "Equivalent Stream Slope (S)" can be calculated using the formula given in the "Flood Estimation Reports" of CWC.

14 Conclusion

14.1 Using the above methodology, the Catchment Area properties (A, L, LC and S) are assessed very quickly and more accurately as compared to the conventional method. This was done for the catchment areas of 20 nos. of examples given (out of total 21 nos.) of "Flood Estimation Reports" of CWC. Their comparison with the corresponding values assessed conventionally in the CWC reports can be seen in the Annexure. The calculations were not done for one subzone (No. 1F: Middle Ganga Plains) as no bridge could be located at the location given in the CWC Report. The values assessed are quite close to the values assessed in CWC report and it is opined that the differences are due to inaccuracies in the method adopted in CWC report. This opinion is based on the fact that in cases with large differences, even location of bridges in the CWC reports are not correct (as confirmed visually on "Google Earth").

14.2 Using these properties of catchment areas, the Design Discharge was calculated as per the data/approach given in each of the CWC report. For this, a separate MS-Excel based calculation

software has been developed. The comparison of the Design Discharge calculated, with the values arrived at in CWC Reports, is also given in the Annexure. The gap for the Design Discharge values further narrows down and becomes very less/inconsequential for use in the bridge substructure/foundation planning.

15 Further developments

The methodology presented in this paper is based on use of SAGA (System for Automated Geoscientific Analyses) Tools along with QGIS. There are other alternative methods also like using GRASS (Geographic Resources Analysis Support System) Tools along with QGIS, using WhiteBox Tools with QGIS and burning the Streams in DEM and then using GRASS Tools along with QGIS. The author has assessed Catchment Area Properties using these alternative methods also, and they will be presented separately.

16 References

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2 Flood Estimation Methods for Catchments Less Than 25 Km2 in Area (Report No. RBF-16). Issued by: Research Design and Standards Organisation, Lucknow, India.

3 "Flood Estimation Reports" for various hydro-meteorological sub-zone in India (total 21 nos. of reports). Published by: Central Water Commission.

4 QGIS for Hydrological Applications. By: Hans van der Kwast and Kurt Menke. Publisher: Locate Press LLC, USA.

5 Videos on this subject on the YouTube Channel of Mr. Hans van der Kwast, IHE Delft University, Netherlands.

6 Mastering QGIS. By: Kurt Menke, Dr. Richard Smith Jr., Dr. Luigi Pirelli and Dr. John Van Hoesen. Publisher: Packt Publishing Limited, Birmingham, UK.

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COLABA_STATION_1920

ANNEXURE

Comparison of Calculations/Assessment as per CWC Report vis-à-vis using GIS System

SN	Zone	Method	Bridge Location						Catchment Area				Q50 (Cumecs)
			Longitude			Latitude			A	L	Lc	S	
			Deg	Min	Sec	Deg	Min	Sec	(sqm)	(km)	(km)	(m/km)	
1	1a_Luni	CWC Report	73	27	20	25	45	15	414	52.8		3.178	**1741.14**
		GIS System	73	27	19.9	25	45	12.34	414.46	59.881		2.687	**1893.72**
2	1b_Chambal	CWC Report	77	10	8	24	31	0	361.05	38.62		3.005	**2093.24**
		GIS System	77	9	54	24	30	59.88	357.95	40.281		2.33	**1902.71**
3	1c_Betwa	CWC Report	79	28	3	35	17	30	154.62	23.35		2.615	**862.69**
		GIS System	79	27	53.1	25	17	22.15	162.12	22.442		2.157	**841.56**
4	1d_Sone	CWC Report	80	51	53	24	44	12	340.64	34.94		3.702	**1082.07**
		GIS System	80	51	49.6	20	43	59.14	340.34	33.784		2.863	**964.45**
5	1e_Upper Indo Ganga Plains	CWC Report	76	44	0	30	24	0	1126	81.42		5.14	**2715.28**
		GIS System	76	43	44.1	30	24	0.71	1388.63	99.828		3.72	**2636.19**
6	1g_Lower Ganga Plains	CWC Report	79	26	0	19	39	0	389.86	38.29	18.5	9.371	**1742.69**
		GIS System	85	2	27.4	23	39	53.76	390.34	33.775	19.089	7.797	**1799.22**
7	2a_North Brahamputra	CWC Report	90	7	10	26	23	10	595.7	75.62	47.14	1.701	**1281.09**
		GIS System	90	6	50.1	26	23	19.27	549	76.559	40.507	2.375	**1381.64**
8	2b_South Brahamputra	CWC Report	93	4	48	25	47	0	470	56.35	31.4	2.019	**1136.44**
		GIS System	93	4	32.3	25	47	2.37	482.114	53.327	22.478	4.717	**1178.83**
9	3a_Mahi and Sabarmati	CWC Report	74	22	0	22	52	0	136.36	33.5		3.263	**1049.62**
		GIS System	74	21	49	22	51	51.7	139.829	36.018		3.16	**1037.59**
10	3b_Lower Narmada and Tapi	CWC Report	76	8	0	20	54	0	285	34.45	14.45	2.479	**1225.84**
		GIS System	76	7	38.2	20	54	18.8	288	35.6	15.347	2.473	**1258.78**
11	3c_Upper Narmada and Tapi	CWC Report	78	21	56	22	45	25	70.18	23.1	12.88	4.03	**597.96**
		GIS System	78	21	42.5	22	45	24.60	66.961	22.72	10.848	2.522	**575.60**
12	3d_Mahanadi	CWC Report	83	21	0	20	4	36	194	39.36	15.13	4.359	**1077.58**
		GIS System	83	20	49.9	20	4	23.9	193.396	38.045	20.413	3.894	**999.15**

SN	Zone	Method	Bridge Location						Catchment Area				Q50 (Cumecs)
			Longitude			Latitude			A	L	Lc	S	
			Deg	Min	Sec	Deg	Min	Sec	(sqm)	(km)	(km)	(m/km)	
13	3e_Upper Godavari	CWC Report	70	0	4	90	40	0	2227.39	96.6		1.54	**4125.8**
		GIS System	76	4	0.86	19	39	54.7	2243.36	113.45		1.177	**3388.7**
14	3f_Lower Godavari	CWC Report	79	26	0	19	39	0	242	27.7	11.2	3.875	**1592.76**
		GIS System	79	26	2.19	19	39	6.13	242.959	29.453	12.656	3.222	**1455.45**
15	3g_Indravati	CWC Report	80	48	0	20	25	0	325	31.37	16.09	1.195	**2029.05**
		GIS System	80	47	52.9	20	25	3.67	313.683	36.438	18.652	1.356	**1726.88**
16	3h_Krishna and Pennar	CWC Report	75	18	0	15	13	0	270.6	35.4	13.84	1.276	**889.43**
		GIS System	75	15	58.9	15	14	31	264.620	40.930	15.656	1.235	**829.44**
17	3i_Kaveri	CWC Report	78	30	18	12	24	0	294	43.47	22.72	51.3	**832.61**
		GIS System	78	30	16	12	24	4.09	299.14	50.654	28.405	4.4	**675.38**
18	4a, 4b, 4c_Eastern Coast	CWC Report	79	27	30	13	25	0	785	52	24.71	4.12	**2224.91**
		GIS System	79	34	39.3	13	18	37.9	775.881	53.656	28.211	3.482	**1970.73**
19	5a, 5b_West Coast, Konkan and Malabar	CWC Report	75	34	40	11	52	25	176	38.48		4.21	**964.03**
		GIS System	75	34	38.4	11	52	38.4	180.769	35.468		4.901	**1114.98**
20	7_Western Himalayas	CWC Report	76	28	15	32	4	55	103.6	21.32	11.58	69.004	**1133.8**
		GIS System	76	28	1.28	32	4	52.3	114.110	21.399	14.387	48.996	**1129.99**

"TM Connect" – A Software to connect Track Machines

Abhijit Roy and Pashupati Nath Chatterjee***

SYNOPSIS

Earlier there were small numbers of track machines due to which it was possible to manage their details by collecting information over telephone and also by inspecting the machines at site. However, with the number of machines increasing over the last couple of years, it has become very difficult to remain updated about the machines. This has given rise to serious problems in planning proper upkeep of the machines including safety issues. In view of this it was felt to develop a mobile based application which can be updated by the machine operator himself and the details will be visible to all concerned responsible to keep the machines in proper condition. This has led to the development of the mobile application named "TM Connect", which will connect the machine operators to the higher level of management on real time basis.

This software has two parts, one is run through mobile app, which can be easily downloaded from a single source to any smart phone. The App is basically for machine operators, who can directly post their progress, breakdowns, support needed, machine details, etc. The second part is web based, which can be operated through any web browser. This part is for administrators, who can monitor progress, machine positions, staff position, details of machine, camping coach & staff, support needed by the machines and details of safety items available with the machines in a tabulated statement.

The App and its Web based software has been developed and commissioned by the Track Machine Organisation of South Eastern Railway, with technical support from M/s Intellio Services LLP, Kolkata to share ladder-less as well as real-time information about the various aspects of Track Machines working in the zone to all the Engineering officials simultaneously.

Salient technical features of the application are:

- ✓ Application has been developed using three tier architecture: front end in ASP. Net using C#, back end in MS SQL Server 2012 R2.
- ✓ Service layer has been used for secured data access.
- ✓ No usage of third party tool or application. Prevention of cross domain access.
- ✓ Mobile: Developed in IONIC deployable in Android mobile.

The app can be accessed through the mobile phones of field officials and the computers/mobile phone of the Engg. Officials at the HQ, Divisions and Sections, which can help in swift identification and prompt maintenance/ rectification/restoration of the Track Machines conveniently, thus augmenting the efficiency manifold.

Main objectives of the software are:

- Get daily progress directly from track machines.
- Get current location of track machines in Map View.
- Obtain day to day update and analyse idle times, if any.
- Eliminate manual and non-standardized processes.
- Online support system for breakdown with image proof.
- Monitor stock of safety items in Track Machines.
- Notifications and reporting.
- Avoid unproductive communications and e-mails, thus saving man-hours.
- Get critical information like POH/ IOH of the machines.
- Get detailed staff allocation in each machine.

The Mobile App and the Web part of the software are discussed below –

** Chief Engineer/TMC/SER*
*** AXEN/TM/HQ/SER*

A) Various aspects of "Mobile App"

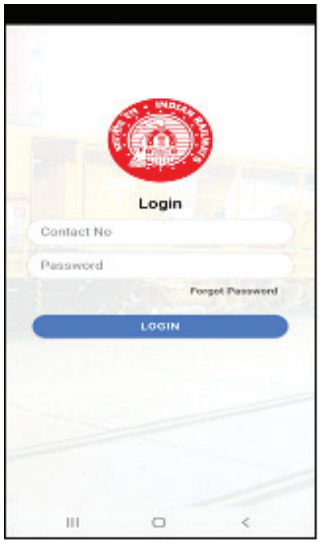


Fig.1.



Fig. 2.



Fig.3.

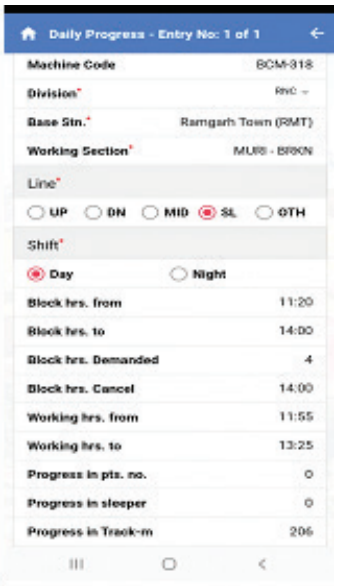


Fig. 4.

After putting credential in login page (Fig. 1), the index page as shown in Fig. 2 shall appear which has five menu items.

1 **Daily Progress :** By tapping on Daily Progress menu, the administrator gets the list of all the machines as shown in Fig. 3. He can choose the desired machine and date to display the progress filled by the machine operator as shown in Fig. 4. All the relevant data regarding progress like major section, block section, block hours demanded, block hours allowed, actual working time, progress, reasons for less progress etc are available in this page. However, if any machine operator logs in with his credential, he will only find the details of the machine assigned to him.

2 Machine Master:

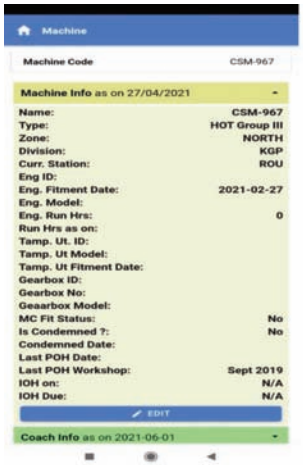


Fig. 5.



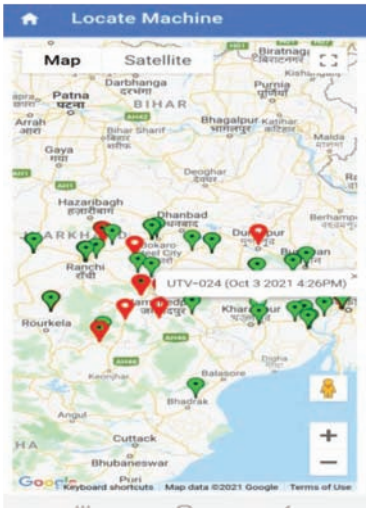
Fig.6.



Fig. 7.

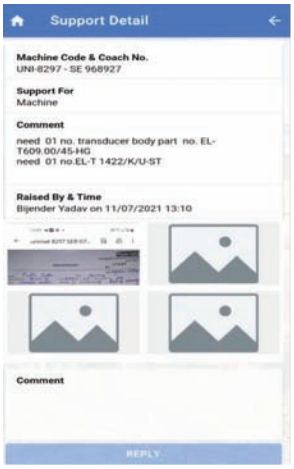
In Machine Master section, there are 3 sub-menus, namely Machine Info (Fig. 5), Coach Info (Fig. 6) and Staff Info (Fig. 7). All related data regarding machine, coach and staff can be edited/entered by the machine in-charge or Divisional SSE and everyone using the software can see it. To know these details there is no need to call for registers and other records. The date of last entry/edit is also visible in all the sub-menu pages. In staff section there is provision for uploading competency certificate of operators issued by Dy. CE/TM, from which it can be easily known which operator is to be sent to which training next.

3 **Locate Machine:** In locate machine section, location of the machine can be seen in Google Map. Location of the machines is shown in two colours, Red and Green. If the progress of a particular machine has not been uploaded in the last two days then its colour will be red otherwise it will be green. Position of the machine is displayed as the location from where its progress was uploaded. The



time & date of the last update is also displayed when the particular location balloon is tapped.

4 Support Details:



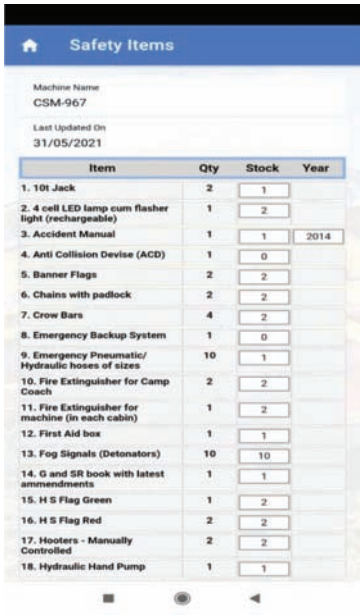


In support section, an operator can ask for support for any breakdown, parts requirement, etc., with relevant photographs. Photographs can be taken either by using the mobile phone or sharing from photo gallery and the maximum number of photographs is limited to 4 nos. per ticket.

After viewing requirement of support the Divisional SSE, AXEN/TM/Line, Officers at HQ can take necessary actions immediately to solve the problems. The action taken can also be recorded by the persons concerned which will remain logged into the system.

This section saves time in rectification of machine related problems and reduce breakdown time. This will also help in creating database of breakdown of the machines.

5 Safety Items: In this section the requirement of safety items by each individual machine has been provided which can be changed by the administrator as per requirement. The machine operator will give input on the items actually available in the machine which can be viewed by the higher management. The list of safety items has been gathered from the IRTMM and as per requirement felt by the administration. The year of publication of various Safety Manual/Books to be kept in the machine can also be entered by the machine operator. This will give a real time availability of the safety items and manuals in the machine.



B) Monitoring through Web Browser:

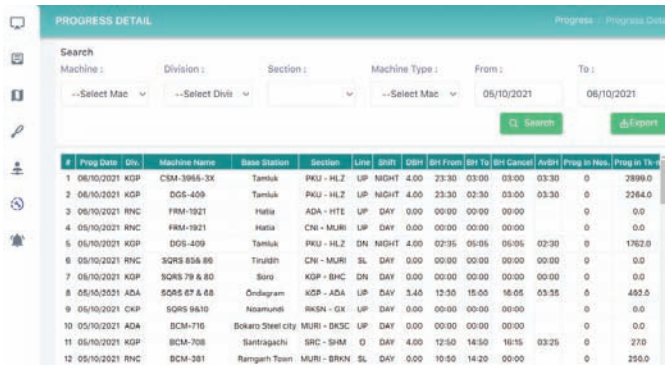
When the software is opened in web browser, its dashboard will show the machine positions in a nutshell. From there, any section can be seen and necessary editing can be done by the admin, if required.



On the top of the screen all relevant data like total number of machines, active/inactive machines in the last 2 days, machines with less safety items, total number of staff, total block hours availed, total unresolved supports, etc. can be seen.

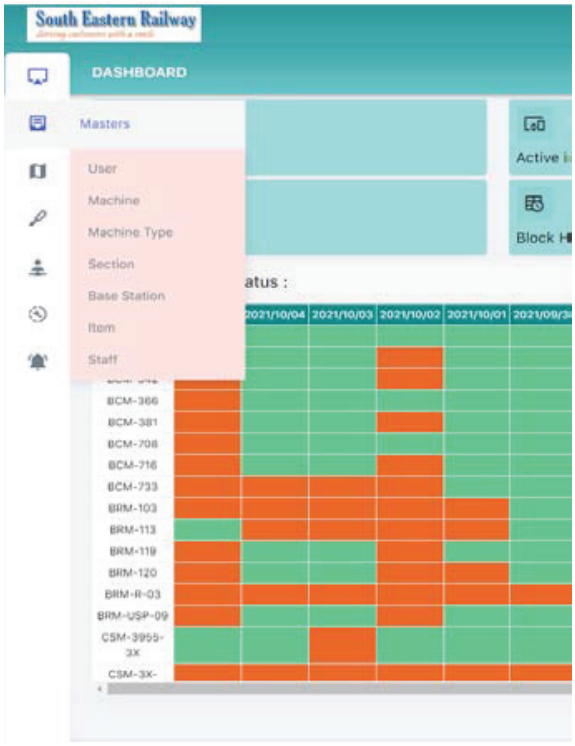
On the bottom of the screen, activity status of machines is furnished date wise. Green is for machines active upto last two days and Red colour is for machines inactive in the last two days.

From this dashboard all sections can be visited. As an example, if progress is selected, the screen will show following details:



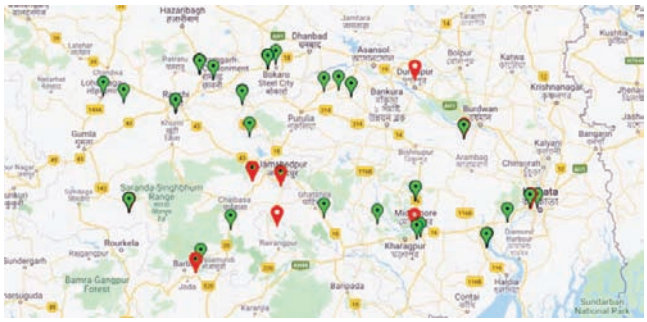
Apart from the dashboard items there are five other menu items in the opening page viz. Masters, Machine Current position, Machine Item Mapping, User Machine Mapping, Progress Details and Support Section. These are discussed below individually:

a) Masters: It has seven sub-items viz. User, Machine, Machine Type, Section, Base



Station, Safety Items and Staff. Here the admin can add/ delete/edit all the relevant details which will be reflected in the mobile app.

b) Machine Current Position :



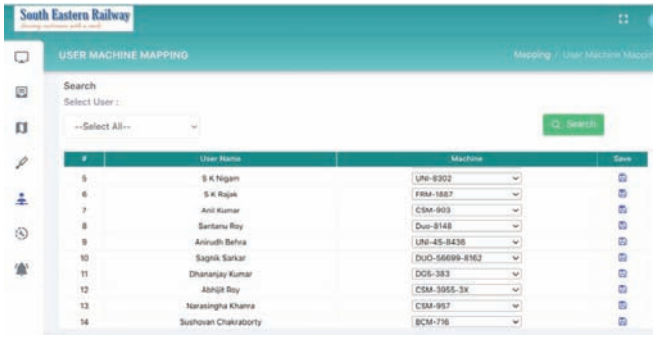
This is the Google Map view of the machines at the time of last entry made for progress of the machine. By clicking on the location balloon, one can see the details of last entered progress of the machine.

c) Machine Item Mapping :

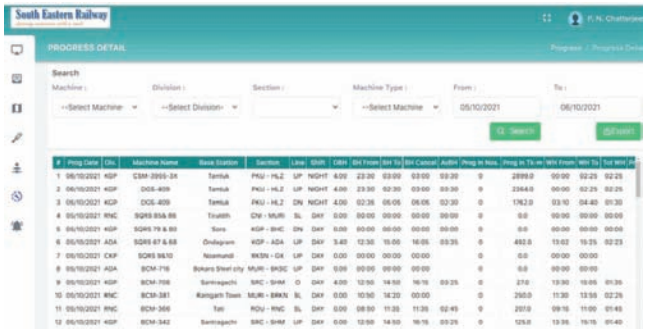
On this page the admin can add/delete/edit the list of safety items for individual track machines. Its contents can be exported to excel also.



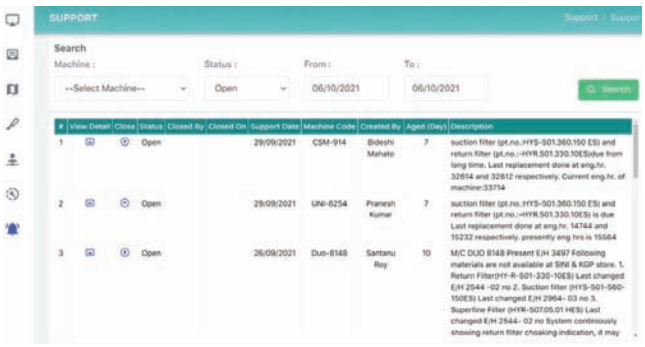
d) User Machine Mapping : In this page, machine can be assigned to users. In case an operator is transferred from one machine to other, necessary editing can be done here.



e) Progress Details : This page displays the progress related details of each machine, which can be filtered across various criteria such as date, division, machine, machine type, etc. All data can be exported to Excel for further analysis.



f) Support : This page displays the list of support requested by the machine operators. The information can be filtered across machine, status of support (open/close) and period of time. Its contents can be exported to Excel also. The user will be able to know all the open/close support items of all the machines over any period of time and can take required managerial decision.



Future Scope:

This software can further be developed in the following ways:

- If GPS data of all assets in Railway Track like Signals, Level Crossings, Points & Crossings etc. are available, then this software can alert operators of approaching track features.
- Oil contents of engine, Machine Temperatures and other parameters can be monitored by inserting sensors in the machines.



MAHATMA GANDHIJI TRAVELLED EXTENSIVELY BY TRAIN DURING THE INDEPENDENCE STRUGGLE.

Design of Reinforced Concrete Pier Caps by Strut-and-Tie Method

Avinash Kumar and G.S Yadav***

SYNOPSIS

Reinforced concrete Pier caps are like deep beams whose behaviour is dominated by shear deformation. The Strut-and-Tie method (STM) can be used to design reinforced concrete pier caps where the conventional method of design for shear and flexure cannot be used due to nonlinear distribution of strains. The designers generally adopt either sectional method or design method applicable to corbel for designing the pier cap, which is not rational. This paper presents the applicability of STM for the analysis and design of reinforced concrete pier caps of Highway as well as Railway bridges. The provisions for the use of STM is given in various international codes Like ACI, AASTHO and fib but the Indian codes for Railways and Highways are silent over it. Because of the lack of a well-ordered design process, many practitioners are reluctant to use STM. This paper presents a proper method to analyse of pier caps by formulating an appropriate strut-and-tie model.

1 Introduction-

The reinforced concrete pier caps of bridges support the loads of the girders and transfer them to single or multiple piers. In most of the cases, the pier caps behave like deep beams as the shear span to the depth ratio is relatively small. The Bernoulli's theory of bending, which assumes linear distribution of strain, is not strictly valid and can't capture the true behaviour of deep beams and hence can't be considered an appropriate method of analysis and design of pier caps.

The ultimate strength of the pier cap is primarily influence by the shear-span to depth (a/d) ratio, where a is the distance between the centre of bearing and vertical face of pier and d is the effective depth just below the centre of bearing as shown in figure 1. The pier cap is subjected to heavy concentrated loads from superstructure, hence the depth requirement is high, resulting a/d ratio less than 2 and sometimes even less than 1. When the pier cap has a/d less than 1, it is treated as corbel and the designer typically apply a semi-empirical method based on concept of shear friction developed between the cracks. When a/d is greater than 1, the pier cap is designed conventionally as a cantilever beam using the sectional approach. This is not correct approach if a/d is less than 2 as the strain distribution in disturbed region is nonlinear.

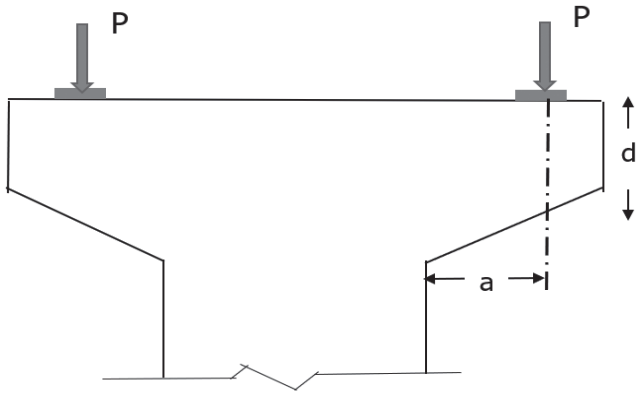


Fig. 1 . Typical pier cap subjected to concentrated load from bridge girders

The strut-tie-method is an effective tool to design such RC members where the whole section is disturbed section and subjected to non-linear strain. STM is formulated based on the actual flow of stress from bearings to supports. It involves the idealisation of the structure as combination of compression struts and tension ties, which meet at nodes to form a truss capable of transferring the load from bearings to the supports. The compression struts are formed within the concrete section and reinforcing bars act as tension ties. This method can be effectively used for the analysis and design of any size and shape of reinforced concrete pier cap.

2 Strut and Tie model

Strut and Tie modelling (STM) is a valuable tool for the analysis and design of concrete structures, especially for the regions where plane sections do not remain plane after bending. It is based on the lower bound

** Prof Track-II/IRICEN*
*** Prof Bridge-II/IRICEN*

theorem of plasticity, according to which a stress field is in equilibrium with the applied loads, and not violating the yield criteria at any point, provides a lower bound estimate of capacity for elastic-perfectly plastic materials. In other words, the resistances everywhere are greater than or equal to the internal forces. The basis of load-carrying mechanisms consists of struts, ties and nodes and the basic principle of this method is that compression is resisted by “concrete struts” and tension is resisted by “steel ties".

The Strut-and-Tie Model is a generalisation of the truss analogy method, which is used to idealise the force flow in a cracked concrete beam. During the past few decades, many experiments have been conducted to refine and develop the model. The following assumptions are made in STM: a) the reinforcement and concrete are adequately anchored, b) the concrete is assumed to carry no tension after cracking, c) the shear reinforcement yields before struts crush and d) forces in struts and ties are uniaxial.

2.1 Elements of Strut and tie Model

(i) Strut

Struts are the elements within strut-and-tie models that carry compressive stresses. The geometry of a strut varies widely and depends upon the force path from which each individual strut arises. There are mainly three basic type struts that is prismatic, fan-shaped and bottle shaped strut as shown in figure-2. Prismatic struts have a uniform cross section over their length. Such a strut can occur in beam bending where the compressive stresses are confined by the neutral axis. A compression fan shaped strut can develop when distributed loads flow into a support. A bottle shaped strut can form when the flow of compressive stresses is not confined. In this case, the force is applied to a small zone and the stresses disperse as they flow through the member. As the compression disperses, it changes direction forming an angle to the axis of the strut. To maintain equilibrium a tensile force is developed to counteract the lateral component of the angled compression forces. A bottle shaped strut can be modelled by a collection of struts and ties to account adequately for the tensile force.

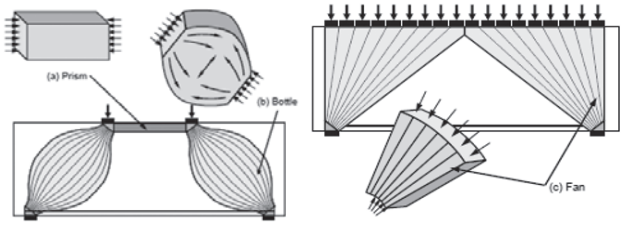


Fig. 2 . Common types of struts

(ii) Tie

Ties are the elements within a strut-and-tie model that carry tension and are generally confined to reinforcing or prestressing steel. The geometry of a tie is therefore much simpler. The tie is geometrically confined to elements that can carry high tensile forces, and the allowable force is generally given as a fraction of the yield force.

(iii) Node

Nodes form where struts and ties intersect. Nodes are described by the types of elements that intersect at the node. For example, a CCT node is one which is bounded by two struts (C) and one tie (T). Using this nomenclature nodes are classified as CCC, CCT, CTT, or TTT. A CCC node is expected to have a higher strength than any of the other types due to the effect of confinement. Each of the other node types has some tensile stresses acting upon it due to the presence of the ties. Tensile stresses can cause cracking within the nodal zone and reduce the strength.

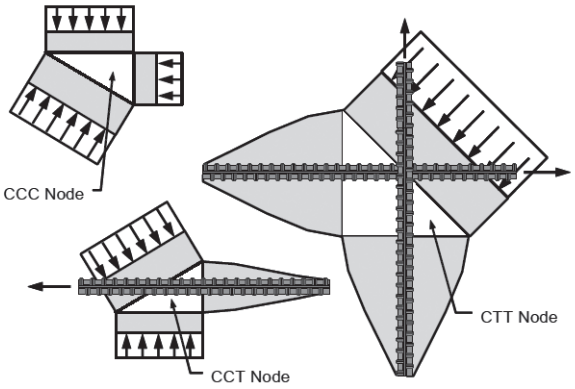


Fig. 3 . Schematic depictions of nodes (Thompson et al. 2003)

2.2 Geometry and dimensioning of Strut and Nodes

The geometry of the idealized truss should be consistent with the dimensions of the struts, ties, nodal zones, bearing areas, and supports. The basic strut and tie model of a deep beam for three-point loading is illustrated in figure-4. The width of strut, Ws, used to calculate the area of strut is the dimension perpendicular to the axis of the strut at the ends of the strut. This strut width is depicted in figure 5. If two-dimensional strut-and-tie models are appropriate, such as for deep beams, the thickness of the struts may be taken as the width of the member except at bearing supports where the thickness of the strut must be equal to the least thickness of the member or supporting element.

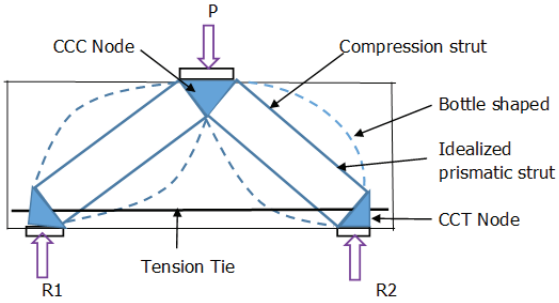


Fig. 4 . Description of Strut-and-tie Model

In design, struts are usually idealized as prismatic compression members. If the area of a strut differs at its two ends, due either to different nodal zone strengths at the two ends or to different bearing lengths, the strut can be idealized as a uniformly tapered compression member. The width of the strut and dimension of node depend on the width of bearing plate and the cover provided to tension reinforcement. The dimensions of the node and width of compression strut shall be calculated at illustrated in figure 5.

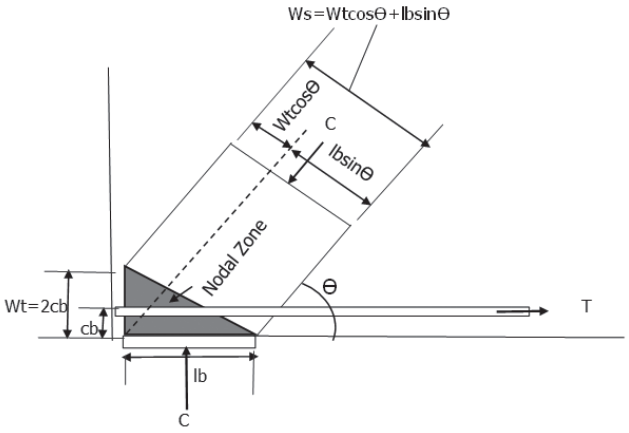


Fig. 5 . Nodal zone and compression strut at CCT Node

3 Provisions of ACI 318R-14

The ACI 318R-14 provide estimations of the strength of the struts of a strut-and-tie model as a fraction of the specified compressive strength of the concrete

$$f_{ce} = 0.85\beta_s f_c' \qquad (1)$$

Where:

β_s = the strut efficiency factor

f_c' = the concrete compressive strength (Cylinder strength)

f_{ce} = Effective compressive strength

The efficiency factor, β_s, is based on the type of strut. There are five classes of struts listed in ACI 318. The first category is a strut with uniform cross section over its length as would occur normally in the compression zone of a beam. Bottle-shaped struts are the most prevalent and general of the classes. A bottle-shaped strut is one with a varying cross-sectional area. Typically, if a force is applied to a small region of a concrete element, the stresses will disperse in the lateral direction as they flow through that element.

It is of note that there are two efficiency factors associated with bottle-shaped struts as shown in table 1. These two factors are based on the reinforcement within the strut. As the compression spreads out from the support, tension is developed. When the induced tensile stress exceeds the tensile strength of the concrete, a crack parallel to strut axis will form. Without any horizontal reinforcement, the strut could split causing a brittle failure. If sufficient transverse reinforcement exists, brittle failure can be avoided, and the strut can continue to carry load beyond the cracking load.

Table 1 . Strut Coefficient β_s

Strut geometry and Location	Reinforcement crossing a strut	β_s
Struts with uniform cross sectional area along length	NA	1.0
Struts located in a region of a member where the width of the compressed concrete at mid length of the strut can spread laterally (bottle shaped struts)	Minimum Reinforcement provided	0.75
	No Reinforcement provided	0.60
Struts located in tension members or the tension zones of members	NA	0.40
All other cases	NA	0.60

ACI 318 also places limits on the allowable stresses at the faces of the nodes. The nodal efficiency factors are based on the elements that intersect to form the node and are listed in *Table 2*.

The Effective Compressive strength of concrete at a face of nodal zone, f_ce shall be calculated by

$$f_{ce} = 0.85\beta_n f_c' \qquad (2)$$

β_n shall be in accordance with table 2.

Table 2 . Nodal Zone Coefficient β_n

Configuration of nodal zone	β_n
Nodal zone bounded by struts, bearing areas, or both	1.0
Nodal zone anchoring one tie	0.80
Nodal zone anchoring two or more ties	0.60

The strength of a strut must be checked at its minimum cross-sectional area. For a strut, especially a bottle-shaped strut, the minimum area will occur at the ends of the strut where it intersects a node. Using the efficiency factors,

the strength of the strut will control the strength of that interface except in the case of a CTT node. It is only in a CTT node that the nodal efficiency could be less than the efficiency of struts framing into that node. ACI 318-14 also provides one more restriction on the modeling process. The angle between the axes of any strut and tie entering a common node may not be less than 25 degrees. This provisions stems from the idea that struts will lose capacity as they approach the direction of a tie. Clearly a strut that is coincident with a tie will have no compressive capacity. The angle of 25 degrees is chosen to eliminate potential problems with struts that form a slight angle with a tie.

4 AASHTO LRFD Bridge Design Specifications

The AASHTO LRFD Bridge Design Specifications, like ACI 318-14, place limits on the allowable stress at the faces of the nodes and struts. However, the AASHTO approach for the allowable stress in a strut is based on Modified Compression Field Theory (MCFT) (Collins and Mitchell 1991) rather than the reinforcement ratios used by ACI. The MCFT based equations are presented below:

$$f_{cu} = \frac{f_c'}{0.8+170\epsilon_1} \leq 0.85f_c' \quad (3)$$

$$\epsilon_1 = \epsilon_s + (\epsilon_s + 0.002)cot^2\alpha_s \quad (4)$$

Where:

α_s = the smallest angle between the compression strut and the adjoining tie.

ϵ_s = the tensile strain in the concrete in the direction of the tension tie

f_c' = specified concrete strength (cylinder strength

f_{cu} = usable compressive strength

Rather than using the amount of reinforcement that crosses the anticipated crack as in ACI, AASHTO bases the efficiency on an average strain in the concrete at the location of a tie. The strut strengths as presented in AASHTO were developed based on tests of reinforced concrete panels. Panels were subjected to a planar loading scheme, in which stress was applied to the four edges of the panels. That stress consisted of both a normal and shearing components. This experimental program (Vecchio and Collins 1982) allowed the researchers to examine concrete under complex stress states and develop mechanics-based models based on that data. The model, MCFT, was then converted into a design tool. In order to apply the design form of MCFT, one must choose an appropriate tensile concrete strain (ϵ_c).

The angle between the strut and any adjoining tie is explicitly considered in the AASHTO LRFD STM provisions. Therefore, no limit is placed on that angle as in ACI. For the nodal stress limits, AASHTO, like ACI, simply specifies factors based on the type of node (Table 3). It should be noted that the AASHTO nodal efficiency factors are multiplied f_c' directly rather than $0.85f_c'$ as specified in ACI 318.

Table 3 : Nodal Zone Coefficient β_n as per AASHTO

Configuration of nodal zone	β_n
Nodal zone bounded by struts, bearing areas, or both	0.85
Nodal zone anchoring one tie	0.75
Nodal zone anchoring two or more ties	0.65

5 Finite Element Analysis

Finite Element analysis of a hammer head reinforced concrete pier cap subjected to heavy concentrated load as shown in figure 1 is done. The stress path of principal compressive stresses is shown in figure 6. The load is directly going to the support from bearing through compressive struts inside the pier cap in the disturbed region. The compressive stresses are maximum just below the bearings and at the corners of pier cap and pier stem. The principal tensile strains inside pier cap are shown in figure 7. The maximum tensile stresses occur between the bearings. These tensile stress are taken by the reinforcement ties.

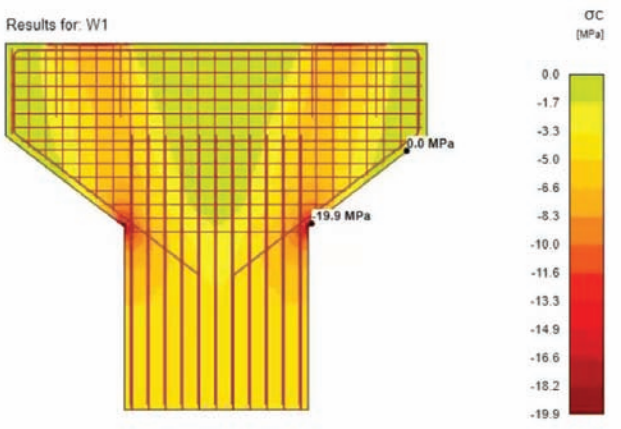


Fig. 6 . Principal compressive stress in reinforced concrete pier cap

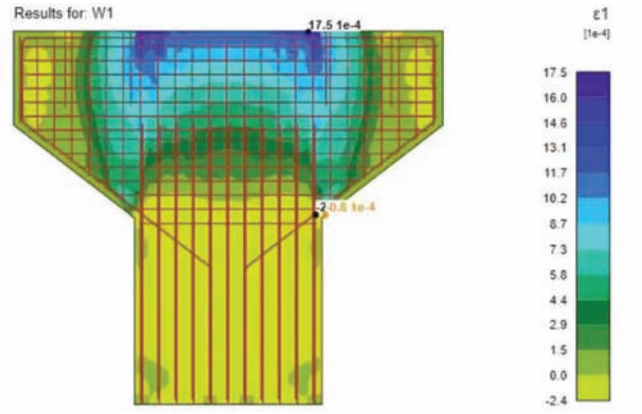


Fig. 7 . Principal Tensile strain in RC pier cap

Utilisation ratio of concrete and steel in compression and tension respectively is shown in figure 8. This also gives an idea of the distribution of stresses in the concrete and steel in compression and tension respectively. The steel provided at the top between the bearings are subjected to tensile stresses and the utility ratio is maximum in that region. If the same pier cap were analysed by sectional method, the requirement of reinforcement between the bearings would be zero. Whereas the strut and tie model concludes that the tensile stress in the reinforcement (tension tie) between the bearings should be same throughout its length. The same result is found in the finite element analysis as depicted in figure 7 and figure 8.

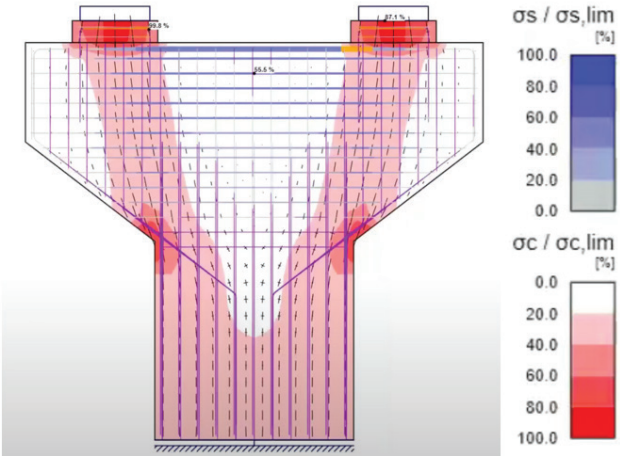


Fig 8 . Utilisation ratio of concrete and steel in compression and tension

The strut and tie model which represents the true behaviour of pier cap of figure 1 under concentrated loads can be developed as shown in figure 9. The compression struts and tension ties are compatible with the stress pattern shown in the results of finite element analysis as depicted in figure 6 to 8. Once the strut and tie model is developed by visualizing the flow of stresses within the pier cap, the forces in different members and nodes can be calculated and compared with the strength of the members and the nodes.

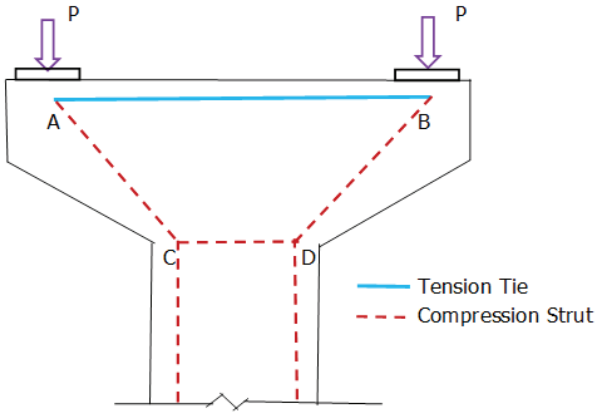


Fig. 9 . Strut and tie model for typical pier cap

Similarly, strut-and-tie model can be developed for pier cap having large numbers of bearings and have larger cantilever part. In such cases, additional nodes between two bearings can be selected as shown in figure 10. The compression struts and tension ties are shown. The vertical shear reinforcement equal to the force of vertical tie shall be uniformly distributed to take care of shear force. The horizontal reinforced placed at top will act as tension tie. This reinforcement shall be anchored sufficient length behind the bearings to develop required tension.

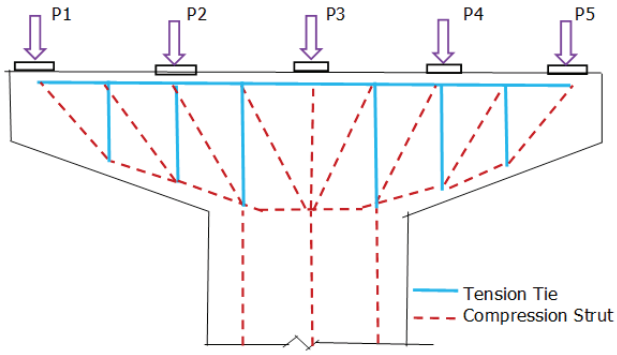


Fig. 10 . Strut-and-tie model for larger width pier cap

6 Modelling and Analysis of Strut-and-Tie

The development of Strut-and-Tie models is not straight forward. Hence, a design strategy is required for the development of a STM. The procedure is listed as follows:

a) Define the disturbed region from geometry and load conditions.

b) Visualize the actual flow of stress within pier cap from loads to supports.

c) Develop a truss model with represents

the actual flow of stresses. The axis of struts and ties shall be chosen so that they approximately coincide with the compression and tension fields respectively (Fig 9 and 10)

d) Calculate the forces in all struts and ties. Calculate the forces on different faces of nodes.

e) Define the dimensions of struts, ties and nodes as recommended by ACI, AASHTO or any other international codes.

f) Calculates the stress in strut, tie and nodes. The stresses should not be more than the permissible stress.

g) If the stresses are more than the permissible stress, resize the pier cap and/or revise the reinforcement.

h) Provide adequate anchorage to steel tie reinforcement.

The process is usually iterative as the truss geometry depends on the dimensions of the nodes which in turn depend on the member forces.

7 Conclusions and recommendations

1 Most bridge pier caps are deep beams and exhibit nonlinear strain distribution. Method of analysis capable of capturing the true behaviour of deep beam is required to be performed for strength prediction.

2 The stress flow pattern of deep pier cap obtained from FE analysis suggests that the pier cap can't be analysed by sectional method. The strut-and-tie method (STM) provides a viable analysis method for strength prediction of deep pier cap.

3 The strut-and-tie model shall be formulated which is compatible with the actual flow of stresses inside the pier cap. For dimensioning of struts, ties and nodes, recommendations given in ACI 318, AASHTO or any other standard international code can be referred.

4 Provision related to formation of strut-and-tie model, dimensions of struts, ties and nodes and prediction of their strength should be included in IRC-112 and IRS Concrete Bridge Code.

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6 Pappu Baniya, Serhan Gunar, Specialized Strut-and-tie method for rapid strength prediction of bridge pier, Engineering Structures 198 (2019)

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CST-1888H

Maintenance of Track on Bridge Approaches

B. Rama Rao and Shashank Yadagiri***

SYNOPSIS

Ten out of ten girder bridges have this problem—the approaches are either low or high. Difference in resilience of track often leads to low approaches. And, either ramping in or ramping out during tamping results in high approaches. Both thesituations are vulnerable for rough running. The approach track, therefore, should be either lifted or lowered to make it level.

At the bridges approaches under study, more OMS peaks and less TQI values (70) were recorded due to unevenness caused by either low or high approaches. At bridge 628, lifting of approach track with UNIMAT(no ramming out in the approach) and, at bridge 612, lowering of approach track with BCM were done. The method of execution is discussed in this paper.

Case1: Approach track is lower than the bridge proper

Tamping is usually stopped well before a bridge approach. After many such tampings, sag is left in the approach. At these locations, tamping should be continued till the bridge by feeding lifts, calculated based on levels, in the front tower of tamping machine. Detailed procedure of one such case (Bridge 628 UP) is discussed here:

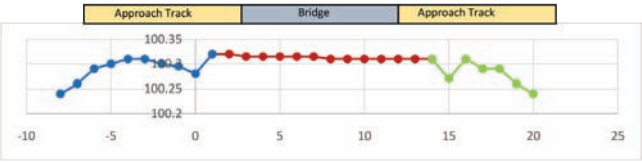


Fig. 1 : Existing levels of approach trackand bridge

1 Existinglevelsofthe approach track and bridge proper are taken at an interval of 10m.

2 Graph *(Fig.1)* is plotted using these initial levels.

3 The amount of lift (Annexure 1) at each station is calculated.

4 The lifts are written on sleepers so that these values can be fed in the front tower of the tamping machine.

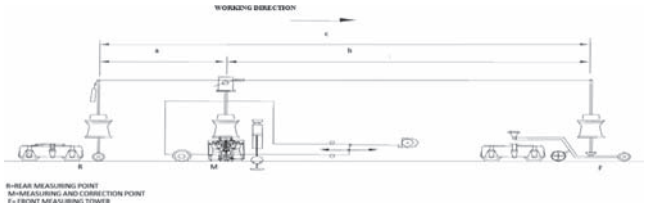


Fig. 2 . Typical layout of Tamping Machine

5 Starting point of tamping when machine works towards the bridge:

In the present case, 55mm lift is required to make a level-track up to 90m from the bridge. Thus, to get 1 in 1000 ramp, the machine should start working from 55m (55mm x 1000) behindi.e 145m from the bridge. With the sleeper spacing of 600mm, number of sleepers to be tamped in 55m are 92 (55000/600). Lift per sleeper is 0.6mm (55/92). Table 1 below lists the values of lifts to eliminate the sag.

Table 1 . Values of lifts at each station

Station	RL	Lift	Design RL	Station	RL	Lift	Design RL
TBM	100			-1	100.28	40	100.32
-9	100.24	55	100.29	0	100.32	0	100.32
-8	100.26	50	100.31	1 - 14	100.32 to 100.31	Bridge 628	
-7	100.29	25	100.31	15	100.27	40	100.31
-6	100.3	20	100.32	16	100.31	0	100.31
-5	100.31	10	100.32	17	100.29	20	100.31
-4	100.31	10	100.32	18	100.29	20	100.31
-3	100.3	20	100.32	19	100.26	50	100.31
-2	100.295	25	100.32	20	100.24	70	100.31

6 During traffic block, machine should be positioned such that tamping bank of machine is at the point of zero lift (shown in *Fig 3*) i.e. 145m from the bridge.

7 At this position, front measuring tower of the machine will be at “b” (12.82 m for UNIMAT3S) distance ahead of tamping bank and rear tower will be at “a” distance (5.58 m for UNIMAT 3S) behind the tamping bank.

8. When tamping is started from 1st sleeper, front trolley will be on 21st sleeper. Thus lift should be fed from 22nd sleeper. Since rear trolley is behind the 1st sleeper by a length of 5.58m (10 sleepers),

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*** ADEN / BPP*

a lift of 18.6 mm has to be achieved in 10 sleepers i.e. from 22nd sleeper to 31st sleeper to get the ramp of 1 in 1000 right from the first sleeper. After tamping these 10 sleepers, machine completely enters the ramp.

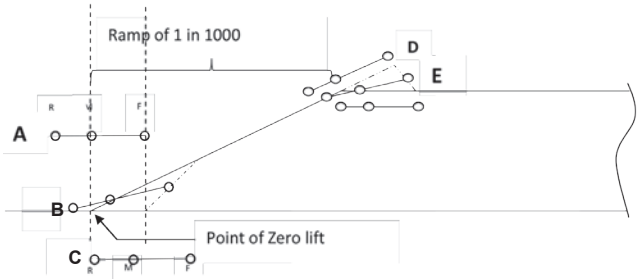


Fig. 3 . Movement of tamping machine in the ramp portion

R: Rear trolley M: Measuring trolley

F: Front trolley

Position A: Initial position of Machine ("M" is at point of zero lift)

Position B: Position when part of the machine is on 1 in 1000 gradient

Position C:When full machine is on 1 in 1000 gradient

Position D: When "M" is at exit of ramp

Position E: When "R" is still in the ramp

9 When the whole machine enters the ramp i.e. when the rear trolley (R) comes tothe point of zero lift, the lift values corresponding to 1 in 1000 gradient should be fed in the front tower.

10 Similarly, at the end of the 1 in 1000 ramp (Point 'D" in *Fig. 3*), a correction should be applied to the lifts to be fed in the front tower.Corrections are shown in the below (table 2).

Table 2. Marking of lift values on sleepers

Sleeper	Lift(mm) required as per 1 in 1000 ramp	Lift(mm) to be fed in the front tower at the entry of ramp for a length of 5.58m (Position B)	Lift(mm) to be fed in the front tower at the entry of ramp when complete machine is in ramp portion (Position C)	Lift(mm) to be fed in the front tower at the exit of Ramp until tamping bank reaches the design lift position (55mm) (Position D)	Lift(mm) to be fed in the front tower at the exit of Ramp after tamping bank reaches the design lift position (55mm) (Position E)
Start	0.0				
1 to 21	0.6 to 12.6 @ 0.6 per sleeper	-	-	-	-
22	13.2	1.86	-	-	-
25	15	7.44	-	-	-
28	16.8	13.02	-	-	-
31	18.6	18.6	-	-	-
32 to 92	19.2 to 55.2 @ 0.6 per sleeper	-	19.2 to 55.2 (Same as column 1)	-	-
93 to 113	55.8 to 67.8 @ 0.6 per sleeper	-	-	55.8 to 67.8 (Same as column 1)	-
114	-	-	-	-	66.52
115	-	-	-	-	65.24
116	-	-	-	-	63.96
117	-	-	-	-	62.68
118	-	-	-	-	61.4
119	-	-	-	-	60.12
120	-	-	-	-	58.84
121	-	-	-	-	57.56
122	-	-	-	-	56.28
123	-	-	-	-	55

Note for sleeper 114 to 123: Since the distance between "M" and "R" is 5.58m, the lift 67.8mm has to be brought down to 55mm in a length of 5.58m. It needs 10 sleepers (5.58/0.6=9.3). Hence, reduction of lift per sleeper is 1.28mm [(67.8-55)/10].

11 Further, tamping should be continued by feeding the lift values of sag written on the sleepers. When the front tower enters the bridge, sag will end, and hence lift will also be zero at the front tower. After this point, tamping should be continued with zero lift till the last ballasted sleeper i.e. start of the bridge. Since the front tower and rear tower are on level track, the track within the machine-length will be lifted automatically.

12 In this method, approaches are lifted without stopping the tamping before the bridge.

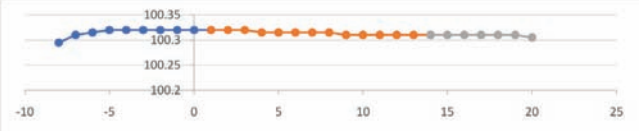


Fig. 4 . Final levels of approach track & bridge

Case 2 : Approach track is higher than the bridge proper

Lowering—may be for a length of 250m—can remedy this issue. As manual lowering is difficult, loweringwithBCM can be the best option.Many a times, required-lowering may not be possible due to inadequate cushion. In such situations, the slope of approach track can be at least flattened by lowering the track to the extent possible.

At Bridge 612 DN, the approach gradient, as per index plan, should be 1 in 1000. But the existing gradients are 1 in 333 at one end and 1 in 373 at the other end. Field survey has revealed that these slopes can be flattened by lowering. Here is the procedure for lowering of approach trackwith BCM:

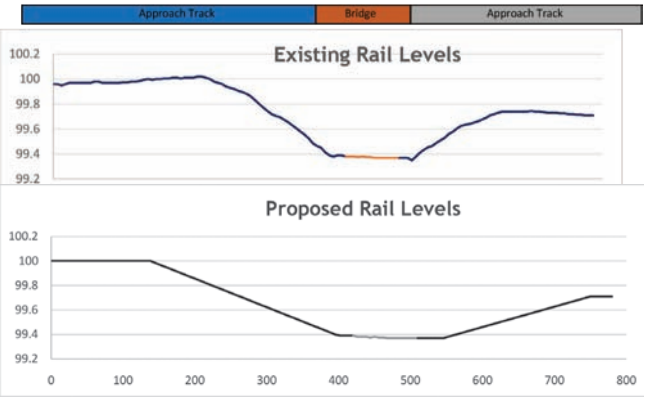


Fig. 6 . Proposed rail levels



Fig. 7 . Marking of proposed levels on pegs.

2 Proposed rail levels are calculatedconsidering possible amount of lowering based on the available cushion. This is to avoid cutting of formation.

3 The proposed levels (Table3) are marked on the pegs driven on cess at a spacing of 30m.The proposed levels are further reduced by 20mm to compensate the lift to be given during tamping.

4 Two BCMs—one machine at each approach—are deployed under traffic block of 2 hr 30 minutes. Two such split-blocks are availed due to high density of traffic. Otherwise, one block of 4 hrs will suffice.

5 In fact, there is no mechanism in BCM to measure the amount of lowering. The gap between top of the cutter bar and bottom of the sleeper (Fig 8) is measured at every 6m to ensure the amount of lowering. Small fluctuations of lowering are inevitable.



Fig. 8 . Measurement of amount of lowering (Picture taken at maximum lowering point: 217mm)

6 Levels aretaken again after lowering and amount of lift calculated to get the desired gradient. The lift-values are marked on sleepers; maximum lift is 40mm.

7 The track is tamped in design-mode by feeding these lifts. After tamping, levels are taken one

more time, for second tamping.

8 After second tamping, the approach gradients are 1 in 475 in place of 1 in 333, and 1 in 800 in place of 1 in 373.

Limitation : Adequate ballast cushion is prerequisite so that lowering can be done without cutting the formation.

Concluding remarks : Bridge approaches: Lowering of bridge approaches can be done with BCM. On the other hand, if lifting of bridge approaches is to be done with CSM or Unimat, it needs careful adjustment of lifts in the front tower of tamping machine.

Ride index is improved from 3.1 to 2.0 after these attentions.

A tip on GRPs of P&C:



Fig. 9 . T5156 GRP at SRJ

GRPs—at the toe of crossing, heel of crossing, SRJ, and welds of P&C—are displaced frequently due to dynamic loads. T5156 GRPs, which are designed with vertical projections for channel sleepers, can be used as an alternative. The vertical projections check the displacementcompletely.

Table 3 . Proposed Levels

Sl.no	Distance	RL (m)	Proposed RL (m)	Fall (mm)	
1	0	100	100	0	BCM end
2	6	99.995	99.986	9	
3	12	100	99.972	28	
4	18	100	99.958	42	
5	24	100.005	99.944	61	
6	30	100.005	99.93	75	
7	36	100.01	99.916	94	
8	42	100.01	99.902	108	
9	48	100.005	99.888	117	
10	54	100.01	99.874	136	
11	60	100.01	99.86	150	
12	66	100.01	99.846	164	
13	72	100.02	99.832	188	
14	78	100.02	99.818	202	
15	84	100.01	99.804	206	
16	90	100	99.79	210	
17	96	99.98	99.776	204	
18	102	99.97	99.762	208	
19	108	99.96	99.748	212	
20	114	99.94	99.734	206	
21	120	99.93	99.72	210	
22	126	99.92	99.706	214	
23	132	99.905	99.692	213	
24	138	99.895	99.678	217	
25	144	99.88	99.664	216	
26	150	99.86	99.65	210	
27	156	99.83	99.636	194	
28	162	99.8	99.622	178	
29	168	99.77	99.608	162	
30	174	99.745	99.594	151	
31	180	99.72	99.58	140	
32	186	99.705	99.566	139	
33	192	99.695	99.552	143	
34	198	99.675	99.538	137	
35	204	99.655	99.524	131	
36	210	99.63	99.51	120	
37	216	99.605	99.496	109	
38	222	99.58	99.482	98	
39	228	99.555	99.468	87	
40	234	99.525	99.454	71	
41	240	99.485	99.44	45	
42	246	99.465	99.426	39	
43	252	99.45	99.412	38	
44	258	99.415	99.398	17	
45	264	99.39	99.39	0	
46	270	99.38	99.39	-10	
47	276	99.39	99.39	0	BCM Start
48	282	99.39	99.39	0	
49	288	99.38			
50	294	99.38			
51	300	99.38			
52	306	99.375			
53	312	99.38			
54	318	99.375			

55	324	99.375			
56	330	99.37			
57	336	99.37			
58	342	99.37			
59	348	99.37			
60	354	99.37			
61	360	99.37			
62	366	99.37			
63	372	99.37	99.37	0	
64	378	99.35	99.37	-20	BCM Start
65	384	99.38	99.37	10	
66	390	99.41	99.37	40	
67	396	99.435	99.37	65	
68	402	99.455	99.37	85	
69	408	99.465	99.37	95	
70	414	99.49	99.38	110	
71	420	99.51	99.39	120	
72	426	99.53	99.4	130	
73	432	99.56	99.41	150	
74	438	99.58	99.42	160	
75	444	99.605	99.43	175	
76	450	99.625	99.44	185	
77	456	99.635	99.45	185	
78	462	99.64	99.46	180	
79	468	99.65	99.47	180	
80	474	99.66	99.48	180	
81	480	99.675	99.49	185	
82	486	99.69	99.5	190	
83	492	99.71	99.51	200	
84	498	99.72	99.52	200	
85	504	99.73	99.53	200	
86	510	99.74	99.54	200	
87	516	99.74	99.55	190	
88	522	99.74	99.56	180	
89	528	99.74	99.57	170	
90	534	99.74	99.58	160	
91	540	99.74	99.59	150	
92	546	99.74	99.6	140	
93	552	99.745	99.61	135	
94	558	99.74	99.62	120	
95	564	99.74	99.63	110	
96	570	99.735	99.64	95	
97	576	99.73	99.65	80	
98	582	99.73	99.66	70	
99	588	99.73	99.67	60	
100	594	99.725	99.68	45	
101	600	99.725	99.69	35	
102	606	99.72	99.7	20	
103	612	99.72	99.71	10	
104	618	99.715	99.71	5	
105	624	99.715	99.71	5	
106	630	99.71	99.71	0	
107	636	99.71	99.71	0	
108	642	99.71	99.71	0	BCM end

Time Dependent Deflection and Prestress Loss in PSC Bridges

G S Yadav and Avinash Kumar***

SYNOPSIS

In PSC bridges, the prestressing force causes the concrete to be subjected to sustained compressive stresses even without the presence of live loads. Resulting creep in concrete combined with other time dependent phenomenon such as shrinkage in concrete, change in modulus of elasticity of concrete, and relaxation in prestressing steel result in time dependent deflection and loss of pre-stress in PSC bridge decks. In composite PSC-RCC bridge decks the problem becomes more involved due to presence of different grades of concretes in girders and deck slab and due to different construction stages. Interaction of time dependent properties of materials with each-other further complicates the problem. The time dependent deflection affects the Camber in PSC bridge decks. Knowledge of time dependent deflection is important to enable the maintenance engineers to derive meaningful conclusions from measurement of Camber. Presently this parameter is not calculated for PSC bridges on Indian Railways (IR). In this paper the phenomenon of time dependent deflection and loss of prestress is explained, and calculations of time dependent deflection and prestress losses are presented for a typical design used on IR.

1 Camber in prestressed girder bridges

Camber is the common word to describe the upward deflection of eccentrically prestressed bridge girders. The amount of initial Camber is governed by the combined action of the prestress force and self-weight of the girder/deck. Camber and/or deflection during service is function of time dependent material properties such as *modulus of elasticity of concrete* (Ec), *creep* and *shrinkage* of concrete and *relaxation* of prestressing steel. Unlike steel bridges, the Camber in prestressed bridges may *increase or decrease* with time, depending upon stress distribution across depth of the girder under sustained loads.

In pre-cracking stage, the deflection of prestressed concrete girders can be predicted with *good precision* as prestressed concrete is a, by and large, homogeneous elastic body which obeys the laws of mechanics quite closely. Normally, prestressed girder bridges are designed to remain crack free under working loads, hence calculation of deflections is a straightforward process, provided *time dependent material properties and their interdependency* are known.

Figure 1 shows the deflection in uncracked and cracked prestressed girders.

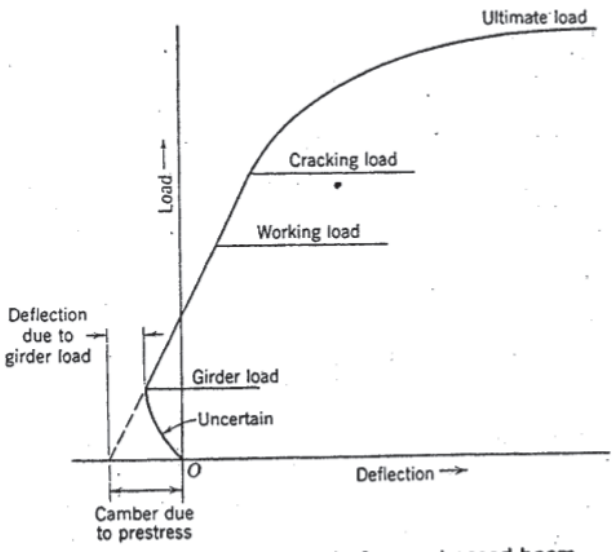


Fig. 1 . Load Deflection Curve of a prestressed beam (Design of prestressed concrete structures: T. Y. Lin and Ned H. Burns.

The time dependency of material properties introduces fair bit of complications in estimation of Camber during service, mainly in three ways:

i Firstly, it is difficult to determine the value of Ec within an accuracy of 10% or even 20%. Moreover, Ec of structural components like girders may differ from that of concrete cubes by significant margin. Besides, the value of Ec varies for different stress levels and changes with the age of concrete.

ii Secondly, the creep and shrinkage of concrete and loss of prestress in steel have profound

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effect on deflection during service. The creep and shrinkage depend upon large number of factors, hence, introduces greater uncertainty in the estimation of Camber.

iii Thirdly, not only the material properties are time dependent, they are inter-dependent also, thus complicating the issue further.

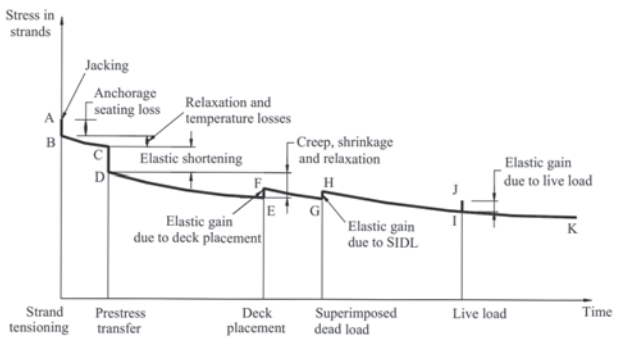
As per present state of knowledge it is possible to estimate deflections of prestressed beams within accuracy of about 20% if proper quality control measures are observed in production of prestressed concrete.

A frequent design requirement is to estimate what Camber may be expected at the time of prestressing of girders, as well as later during service. It is a good practice to balance the downward deflection resulting from dead load by the upward deflection produced by prestress whenever possible. When this is achieved, the flexural creep and the highly variable value of Ec will have little effect on the time dependent Camber or deflection. Frequently, designer can put slight Camber in the girder so that flexural creep tending to Camber the beam will just about balance the downward deflection resulting from the loss of pre-stress. In the context of Railway bridges, the change in Camber (loss or gain) beyond a certain value can cause serious overstressing of fasteners in ballast less Railway tracks. In case of ballasted tracks, the effects, however, get dissipated during maintenance operations.

If the estimated Camber at different times during service life of the bridge is known with some accuracy, actual level of prestress in PSC beams can be estimated by measuring Camber during service. This appears to be the objective behind instructions in Indian Railways Bridge Manual (IRBM) para 1107/b/ii. This para stipulates that 'Camber in prestressed bridges shall be measured within one year of construction and once in five years during subsequent inspections. Variation in Camber from estimated value may indicate loss of prestress more than the estimated value.' However, in IRBM it is not explained what the initial Camber will be and what are the expected values during service. In absence of such guidelines field officials are not able to derive any meaningful deductions from their inspections.

2 Prestress losses in prestressed girders

Components of prestress losses are illustrated in Figure 2 and explained below:



Stress vs time in the strands in pre-stressed concrete girder (NCHRP Report 496)

(a) Loss due to relaxation between initial tensioning and transfer (in pretensioned girders), and temperature change from that of the bare strand to temperature of the strand embedded in concrete.

(b) Instantaneous prestress loss at transfer due to anchorage seating/slip and effect of elastic shortening (prestress loss) and downward deflection due to self-weight (prestress gain).

(c) Prestress loss between transfer and deck placement due to shrinkage and creep of girder concrete and relaxation of prestressing strands.

(d) Pre-stress gain due to deck weight on the non-composite section and superimposed dead loads (SIDL) on the composite section.

(e) Long-term prestress loss after deck placement due to shrinkage and creep of girder concrete, relaxation of prestressing strands, and gain due to differential shrinkage of deck.

Total losses TL are losses due to Friction and slip FS, elastic shortening ES, creep of concrete CR, shrinkage of concrete SH, and relaxation of tendons RE.

TL = FS+ES + CR + SH + RE

Total prestress losses can range from about 15% to 25% of prestress before transfer (PCI design Handbook).

Accurate prediction of prestress losses requires accurate estimation of the long-term properties of concrete and prestressing steel, which is a very complex phenomenon because of the uncontrollable variables involved.

Instantaneous pre-stress loss (due to elastic shortening, anchorage set/slip and friction) can be calculated with fair degree of accuracy. However, estimation of time dependent prestress loss is complicated due to uncertainty in estimation of material properties and interaction between creep, shrinkage, and relaxation. The interaction between creep, shrinkage and relaxation takes place as explained below:

i As concrete creeps and shrinks, the prestressing steel shortens and loses some of its tension.

Consequently, concrete creeps less than in the case of sustained constant compression.

ii Also, as concrete creeps and shrinks, the prestressing strands relax at a slower rate than they would if they were stressed and kept stretched between two fixed points Thus, the "reduced" relaxation is less than the "intrinsic" relaxation that occurs in a constant-length laboratory test.

3 Factors influencing material properties

3.1 Modulus of Elasticity of Concrete (Ec):

The Ec is affected by stiffness of the cement paste, porosity, composition of the boundary zone between paste and aggregates, stiffness and porosity of the aggregates, and proportion of the concrete constituents. However, concrete strength and unit weight are used in prediction formulae to capture the effect of fundamental underlying factors. This, in part, explains the inaccuracy in the current prediction of the modulus of elasticity of concrete. In early-age concrete, the strength of the cement paste is the primary contributor to the strength while at later stage the stiffness of the coarse aggregates is the primary contributor to the modulus of elasticity. Accurate estimation of the value of elastic modulus at different times allows for accurate prediction of the initial Camber and initial elastic prestress loss and helps improve the accuracy of the prediction of creep loss. The modulus of elasticity increases approximately with the square root of the concrete compressive strength. Empirical equations have been developed to estimate the modulus of elasticity based on the compressive strength of the concrete.

3.2 Shrinkage of Concrete:

Shrinkage depends on many variables, including water-to-binder ratio, moisture, relative humidity of the environment, ambient temperature, aggregate properties, and size and shape of the structural member. It is assumed to be independent of loading and results primarily from shrinkage of the cement paste. Because aggregates tend to restrain the shrinkage of the paste, the stiffness and proportion of aggregates influence shrinkage.

3.3 Creep of Concrete:

The creep of concrete depends on many factors, such as volume content of hydrated cement paste, relative humidity, volume to surface area ratio, type and volume of the aggregate, the age of the concrete at the time of loading, the stress level, the duration the concrete is stressed, and the geometry of the member. The size and shape of concrete member can significantly influence the rate and the magnitude of creep. The rate and magnitude of ultimate creep are substantially smaller for larger members.

3.4 Relaxation in prestressing steel:

Relaxation loss in prestressing steel depends upon stress at transfer and duration the steel is stressed. For modern low relaxation stress relieved prestressing steel, the loss due to relaxation is about 30% of the loss in ordinary stress relieved steel.

4 Prestress Loss Calculation Methods:

Estimating prestress loss requires an accurate prediction of material properties and of the interaction between creep and shrinkage of concrete and the relaxation of steel. In addition, prestress losses are influenced by composite action between the cast-in-place concrete deck and the precast concrete girders. Methods of estimating prestress losses can be divided into the following three major categories, listed in descending order of complexity and accuracy:

(a) Time-Step methods (used in FEM based computer programs)

(b) Refined methods

(c) Lump-Sum methods

IRC-112 and international design codes have included detailed procedures for calculating time dependent properties of the materials.

5 Calculation of time dependent properties of materials:

Indian Railway's Concrete Bridge Code does not stipulate any methodology for estimation of time dependent properties of materials. It simply gives ultimate values of creep, shrinkage and Ec. Provisions given in AASHTO – LRFD Bridge Design Specifications -9th edition-2020 are the latest codal provisions amongst all international codes. However, the equations given in AASHTO code are based on FPS system. These equations, being empirical, can't be used without conversion to equations compatible with SI units. IRC-112 (which is almost a replica of Euro code-1992/part2) gives detailed guidelines for estimation of time dependent properties of concrete. The analysis is performed by using provisions of IRC-112, Euro code-1992/part2

and CEB-FIP 2010.

It is important to note that mean concrete strength parameters are used for analysis of time dependent deformations. Characteristic strength is used for determination of capacity of structural components. As per IRC-112, 10 MPa is added in the characteristic strength of concrete to obtain mean strength of concrete.

5.1 Time dependent compressive strength and modulus of elasticity of concrete:

Variation of mean compressive strength of concrete with time is given by

$f_{cm}(t) = \beta_{cc}(t).f_{cm}$;

With, $\beta_{cc}(t) = e^{(s(1-28/t)0.5)}$

Where,

$f_{cm}(t)$ = mean compressive strength of concrete at age t days

f_{cm} = mean compressive strength at 28 days

s = factor depending upon type of cement. It is 0.25 for Ordinary Portland Cement

Variation of modulus of elasticity of concrete with time is given by

$E_{cm}(t) = (f_{cm}(t)/f_{cm})^{0.3}).E_{cm}$

Where E_{cm} is the secant modulus of concrete related to its mean strength

5.2 Creep of concrete :

The creep coefficient at time t is calculated from:

$\phi(t,t_0) = \phi_0.\beta c(t,t_0)$

Where,

ϕ_0 is ***notional creep coefficient*** and can be estimated from $\phi_0 = \phi_{RH}\beta(f_{cm}).\beta(t_0)$

ϕ_{RH} is a coefficient to allow for the effect of ***relative humidity*** on notional creep coefficient

$\phi_{RH} = 1+ (1-RH/100)/ (0.1h00.33)$ for fcm < 45 Mpa

$\phi_{RH} = 1+ (1-RH/100)/ (0.1h00.33)$ for fcm < 45 Mpa

RH is relative humidity of ambient environment in %

$\beta(f_{cm})$ is a coefficient to allow for the effect of concrete mean strength on notional creep coefficient

$\beta(f_{cm}) =16.8/ (f_{cm})^{0.5}$

$\beta(t_0)$ is a coefficient to allow for the effect of **concrete age at loading** on notional creep coefficient

$\beta(t_0) = 1/(0.1+t_0^{0.20})$

$h_0 = 2A_c/u$

A_c is the cross sectional area

u is the perimeter of the member in contact with atmosphere

$\beta c(t,t_0)$ is a coefficient to describe the development of creep with time after loading and may be estimated from the Expression :

$\beta_c(t,t0) = ((t-t_0) / (\beta_h+t-t_0))^{0.3}$

t - age of concrete in days at the moment of consideration

t_0 - is the age of concrete at loading in days

β_h - is a coefficient depending upon relative humidity (RH in %) and the notional member size (h_0 in mm). It may be estimated from:

$\beta_h = 1.5*(1+(0.012RH)^{18}h_0 + 250 \leq 1500$ for fcm $\leq$ 45 Mpa

$\beta_h = 1.5*(1+(0.012RH)^{18}h_0 + 250\alpha_3; \leq 1500\alpha_3$ for fcm $\geq$ 45 Mpa

α_1, α_2, α_3 are coefficients to consider the influence of the concrete strength

$\alpha_1 = (43.75/f_{cm})^{0.7}$ $\alpha_2 = (43.75/f_{cm})^{0.2}$

$\alpha_3 = (43.75/f_{cm})^{0.5}$

Note : Above equations are applicable for Ordinary Portland Cement. For other type of cements the age at loading t0 is adjusted as described in IRC 112 (Annexure_A)

5.3 Drying shrinkage of Concrete:

Shrinkage of concrete consist of two components. Autogenous shrinkage which is the result of chemical reactions taking place in the cement paste; and drying shrinkage which is the result of evaporation of water from the concrete.

The basic drying shrinkage strain $\acute{\varepsilon}_{cd,0}$ is calculated from

$\acute{\varepsilon}_{cd,0} = 0.85(220+110\alpha_{ds1}) e^{(-\alpha ds2 . fcm/12.5)} * 10^{-6} *\beta RH$

$\beta RH = 1.55(1-(RH/100)^3)$

α_{ds1}, α_{ds2} are coefficients which depend upon type of cement. Their value is 4 and 0.12 respectively for Ordinary Portland Cement.

The development of drying shrinkage strain with time is given by:

$\acute{\varepsilon}_{cd}(t) = \beta ds(t,ts).K_h. \acute{\varepsilon}_{cd,0}$

Where

K_h is a coefficient depending on the notional size h0 according to table-1 below

Table 1

h_0	k_h
100	1.0
200	0.85
300	0.75
$\geq$ 500	0.70

$\beta ds\ (t,t_s) = (t-t_s) / (t-t_s)+0.04.h_0^{1.5}$

where,

t - is the age of concrete at the moment considered in days

ts - is the age of concrete(days) at beginning of drying shrinkage. Normally it is at the end of curing.

5.4 Autogenous shrinkage of concrete:

The autogenous shrinkage strain follows from:

$\acute{\varepsilon}_{ca}(t) = \beta_{as}(t)\, \acute{\varepsilon}_{ca}(\alpha)$

Where,

$\acute{\varepsilon}_{ca}(\alpha) = 2.5(fck-10)10^{-6}$

and

$\beta_{ds}(t) = 1- \exp(-0.2t^{0.5})$

5.5 Relaxation in Prestressing Steel:

Relationship between Relaxation loss at time t in hours (ρ_t) and that at 1000 hours (ρ_{1000}) is given by the equation:

$$\rho_t = \rho_{1000}(t/1000)^{k}$$

Where, k is given by (ρ_{1000} /ρ_{100})

However, above provisions in IRC-112, overestimate the relaxation loss for low relaxation steel.

Euro code defines the relaxation loss for low relaxation steel as below:

$\Delta\sigma p_r = \sigma pi * 0.66\, \rho_{1000}\, e^{9.1\mu}\, (t/1000)^{0.75(1-\mu)} * 10\text{-}5$

(3.3.2(6) Euro Code-1992/part 2)

Where,

ρ_{1000} = relaxation loss (in %) at 1000 hours at a mean temperature of 200C

= 2.5 % for low relaxation steel

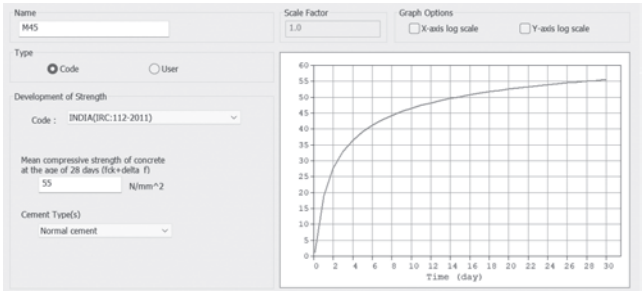
$\Delta\sigma p_r$ = relaxation loss of prestress at time t hours

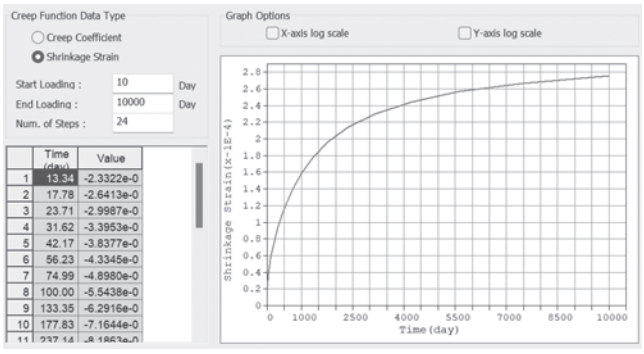
σp_i = initial prestress just after transfer

μ = σ_{pi}/f_{pk}

f_{pk} = characteristic strength of prestressing steel

t = time after tensioning in hours





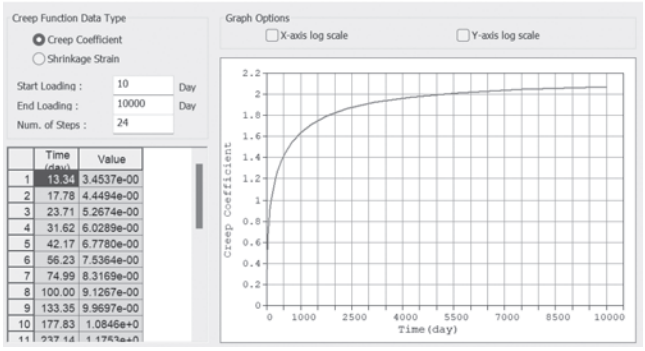


Fig. 3 . Variation of time dependent properties (Compressive strength, Creep, and Shrinkage)

6 Defining Construction Stages for Composite Bridge Deck:

For composite bridge decks it is important to precisely define the construction stages. The loads applied at different stages and the material properties at the time of loading are important considerations in the estimation of time dependent deformation in composite bridge decks. The construction stages for a typical PSC-RCC composite bridge deck (PSC-I girders for18.3 m span with 2 stages of stressing) as per RDSO standard drawing RDSO-B/10273 are defined in Table-2 below:

Table 2 .

Construction Stage (CS)	Activated Structural members	Activated Loads	Remarks
CS1	Girders	Self-weight of girder + Prestressing load of stage 1 (Tendon 1 and 3)	Unpropped construction method is assumed
CS2	Girders	Loads in CS1 + Weight of wet concrete from deck slab and diaphragms	

CS3	Girders and deck slab acting as composite deck	Loads in CS2 + Prestressing loads from stage 2 of prestressing (tendon 2 and 4)	Unpropped construction method is assumed
CS4	Same as above	Loads in CS3 + SIDL (wearing course +ballast retainers + footpath)	
CS5	Same as above	Loads in CS3 + Track	

7 Analysis Model:

Structural analysis software Midas Civil is used for modelling the bridge deck. Following are the salient features of the structural model:

(i) RDSO standard span for 18.4 m PSC-I girder deck for 25 T loading vide drawing number RDSO-B/10273 is used for creating the analysis model

(ii) Grillage method is used for generating the analysis model

(iii) Girders are modelled as longitudinal grillage members (beam elements). Longitudinal stiffness of the deck slab is considered as part of longitudinal grillage members.

(iv) Transverse grillage members (beam elements) are used to model transverse stiffness of the deck slab. Dummy material with properties of concrete of deck slab but with zero-unit weight is defined. This is done to avoid double counting of self-weight of the deck slab.

(v) All loads other than self-weight of girders and deck slab are converted into element loads and applied to appropriate members.

(vi) Construction stages are defined as mentioned in Table-2. Additional stages are defined to determine deflection and prestress loss at an interval of five years up-to 75 years age of the bridge.

(vii) Appropriate boundary conditions are defined. The girders are supported on temporary bearings up-to CS-3. After this stage the bridge deck is transferred onto the elastomeric bearings (modelled with elastic links). The elastic links are connected to fixed supports representing firm-unyielding substructure. The girder insertion points are connected to the elastomeric bearings by using rigid links. Rigid links provide structural connection between bridge deck and the substructure through elastomeric bearings (elastic links). Connection between girders and deck by using rigid links is technically valid as the deck end is very stiff due to provision of end diaphragm.

(viii) Indian Railway Concrete Bridge Code doesn't define time dependent material properties except specifying ultimate creep and shrinkage values for concrete and ultimate relaxation loss in prestress steel. IRC-112 is, therefore, used for analysis. 2011 edition of IRC -112 is used as the 2017 edition of the code is not part of the Midas Civil software available at IRICEN. However, the provisions about time dependent material properties are same in 2011 and 2017 editions of IRC-112.

(ix) The analysis is also carried out as per Euro Code-2 and CEB-FIP 2010 to compare the results obtained as per IRC -112.

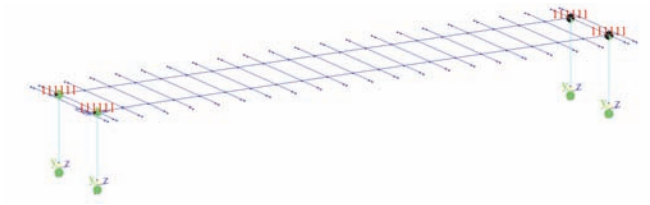


Fig. 4 . Grillage model with required boundary conditions.

8 Check for uncracked section:

PSC bridge decks are generally designed to remain uncracked even under application of live loads. The deflection is calculated based on gross concrete area of the deck. It is important to check whether this assumption of section remaining uncracked is true or not. After prestress loss the most critical part of the bridge deck for check for presence of tensile stresses is bottom of girder. The stress diagram for stresses at girder bottom after completion of 75 years of age is posted below:

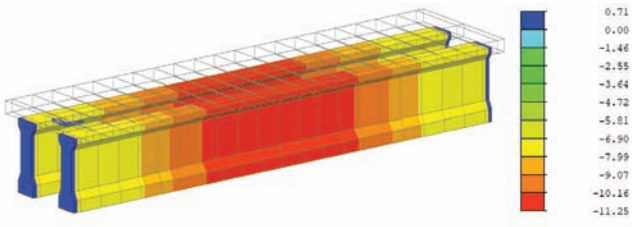


Fig. 5 . Combined Stresses at Girder Bottom under Permanent loads only at 75 years

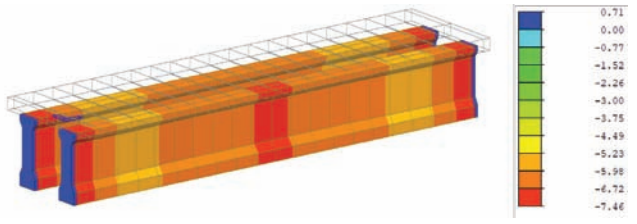


Fig. 6 . Combined Stresses at Girder Bottom under

Permanent and Live loads at 75 years

There is tensile stress of 0.71 MPa at end of the girder. This is since there is un-prestressed concrete cover of 150 mm after end anchorages of the girders. This is insignificant. Rest of the girder remains under compressive stresses up to 75 years of age. Hence the analysis of deflection based on assumption of uncracked section holds good.

9 Discussion of Results and Conclusion:

(i) Analysis is performed to obtain time dependent deflection and prestress losses for an 18.3 m Standard PSC-I girder deck used on Indian Railways. Analysis is done as per three design codes viz. IRC-112-2011, CEB-FIP 2010, and Euro Code-1992-Part2 (EN-2/part2). The results are tabulated in Table-3 below.

Tabel 3

Time After Con-struction	Deflection(mm) at mid-span due to Permanent load (Camber)			Prestress Loss (%)		
	IRC-112	EN-2 (part2)	CEB-FIP-2010	IRC-112	EN-2 (part2)	CEB-FIP-2010
1 year	9.00	10.98	10.04	13	12	12
6 years	9.50	12.03	10.96	17	16	16
11 years	9.53	12.17	11.06	18	17	17
16 years	9.50	12.18	11.05	18	18	17
21 years	9.47	12.18	11.04	19	18	18
26 years	9.46	12.18	11.03	19	18	18
31 years	9.44	12.19	11.03	19	18	18
36 years	9.39	12.19	11.03	20	19	18
41 years	9.33	12.18	11.03	21	19	19
46 years	9.27	12.19	11.03	22	19	19
51 years	9.23	12.19	11.03	22	19	19
75 years	9.25	12.19	11.03	22	19	19
Camber Gain	**0.53**	**1.21**	**1.02**			

(ii) As predicted by theory, there is camber gain over the life span of the bridge, though the values are insignificant for the span analysed.

(iii) It is important to note that the predicted PSC loss of about 20-22% is taking place, but there is no loss of camber.

(iv) This is since in a PSC bridge deck, under sustained loads, the lower part of the bridge deck is subjected to higher compressive stresses as compared to upper part of the deck. The creep will be higher in the lower part of the bridge deck as creep is dependent on applied stress. This will make the bridge deck to deflect upward. However, the loss in prestressing force will cause the deck to deflect downward. Thus, the time dependent deflection due to creep and prestress loss will be compensating each other

(v) The differential shrinkage of deck slab with respect to the girders will cause downward deflection. The creep, shrinkage and relaxation effects are not only time dependent, but they are interdependent also. In general creep dominates other two causes. Thus, in a PSC bridge deck, if the pressure line is below neutral axis, the bridge deck is likely to gain camber during service.

(vi) Change in camber more than that predicted by analysis may indicate excessive loss of prestress.

(vii) Indian Railway bridges are inspected in detail every five years. First inspection is done within one year of completion of the bridge. Camber measurement is part of the inspection. However, time dependent deflection values of PSC bridges are not available for comparing the values obtained during inspection.

(viii) The time dependent deflection should be worked out for each bridge deck and should be part of bridge completion and handing over documents.

10 References:

1 IRC-112-2011 Code of practice for Concrete Road Bridges

2 IRC 22 -2015 Design code for limit state design of composite construction

3 CEB-FIP model code 2010

4 Euro code 1992-Part2-2005 (EN-2/part2)

5 IR Concrete Bridge Code

6 AASHTO LRFD Bridge Design Specifications 2020

7 AASHTO LRFD Bridge Construction Specifications 2017

8 PCI handbook for PSC design

9 RDSO drawing no RDSO-B/

10 Design of pre-stress concrete structures by T.Y.LIN and NED H. BURNS

11 NCHRP repost no. 496

12 Research Project no. 2010-05 on 'Predicting Deflection, Camber and Loss of Prestress' in prestress concrete members: by North Carolina state University

13 Improving the prediction of time-dependent effects on pre-stress concrete bridges PHD Thesis by A. H. Gheitanbaf at IOWA state university

Assessment of Vehicle Dynamics and It's Impact Over Track Degradation - A Case Study for Gadag (GDG) – Ballari (BAY) Section of South Western Railway

Nitish Kumar Ranjan

SYNOPSIS

Rail wheel interaction is a complex phenomenon which defines the dynamics of vehicle and its impact. In view of requirement to increase the axle load and speed, assessment of vehicle dynamics and its impact becomes most important. Further it is also required to get the results of assessment in a form which can be understood and used as initial information for maintenance planning at field level otherwise complex results of this complex phenomenon will lead to complexity in maintenance. Different types of instrumentations are in use to understand the dynamics of vehicles in terms of load transferred. But none of the instrumentations presently done in Indian Railways explain completely the vehicle dynamics and its impact on track degradation. However one of such instrumentation of track known as Wheel Impact Load Detector (WILD) gives the impact of vehicle dynamics in terms of Impact Load Factor and Impact Load. In this article it is tried to understand vehicle dynamics and its impact based on the results given by WILD. System of monitoring & utilisation of WILD results, present constraints of assessment by WILD, degradation of track due to vehicle dynamics and future development & changes required to make the assessment more realistic in view of increase in speed and axle load is also deliberated in this article. For the assessment purpose, WILD results of Hubballi division of South Western Railway are used, which is installed on one of the busiest iron ore route, Vasco de gamma–Ballari, of Indian Railway. To understand the track degradation USFD results of Gadag- Ballari section of same division is also used.

1 Introduction:

In present scenario Indian Railways is facing tough competition from other mode of transportation to attract more numbers of travellers as well as more business in Goods transportation. It is imperative for railway not only to improve the track structure to have higher speeds but also making our track available for traffic movement for longer period in a day. Keeping our track available for longer period for traffic movement implies that reducing the maintenance time, which further implies the controlling and monitoring the situations which leads to premature renewals of assets like rails, sleepers etc;.

Railways use the low resistance of movement between wheel and rail, in order to be an energy efficient mode of transport. The higher speed or heavy axle load trains affects the vehicle-track dynamic interaction. Vehicle-track interaction includes ride comfort and safety, vehicle stability, wheel-rail forces, wheel-rail corrugation, wheel out of-roundness etc., and is influenced by a variety of factors. Wheel sets are regarded as fundamental components of railway vehicles; they support the vehicle during rolling, guide it and transfer longitudinal forces at traction and braking. The wheel-rail interface is the most important parameter in the dynamics of railway vehicles and their condition. This interface is where most of the cost for maintenance on both railway vehicles and track occurs. The change in rail profiles is a major maintenance cost. The profile change on wheels can also be significant, especially in curves. Damage mechanisms such as wear and plastic deformation are the main contributors to profile change. Track is designed for carrying specific load at specific speed but the impact of load as well as speed over track lead to imperceptible damage based the condition of rolling stocks. Hence to make the track the available for longer period with higher speed at

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higher axle load cannot be done in isolation. Condition monitoring and predictive maintenance of rolling stocks are equally important. It is imperative to assess and understand the vehicle dynamics and its impact on track degradation so that quality of maintenance of rolling can be improved or design of track can be improved based on the loading pattern.

WILD(Wheel Impact Load Detector) is a technology that is widely used in Global Railways to monitor the health of rolling stock to ensure safe operations with proven accuracy and reliability. WILD scan hundreds of wheels per day and wheel defects like with spalling (chipping), shelling, and flat or out of round are identified. If these are left undetected, it can cause damage to tracks and can cause derailment also. Here the extent of damage done over track by rolling stocks and system of monitoring of WILD generated alarm is discussed through a case study of various results/data of WILD and track features in Gadag-Ballari section of South Western Railways.

2 How WILD helps in assessment of vehicle dynamics?

WILD is an unmanned intelligent trackside data acquisition system that measures the dynamic impact load of wheels on the rail. It has been developed jointly by RDSO and IIT Kanpur. This system provides audio visual signals in the case of passing of abnormal wheel having higher impact load due some defects which can cause damage to the track as well as rolling stock. This system has facility for automatic detection of wheels having higher impact load and recording of rolling stocks passing over the instrumented zone. Indian Railways have installed nine such systems over different zones.

WILD uses instrumented tracks in which rails are wired with strain gauges to measure the load pattern of the wheel on the rail. The track consists of 12 sleepers – strain measuring zones. Each zone has a full bridge consisting of 4 Rosette type strain gauges. The rail length of 12 sleepers is arrived to capture two full rotation of the wheel on rail. The space between the sleeper and any load that appears in the effective zone is considered. Various such measuring zones are formed along the track. As wheel rolls over the rail, various portions of the wheel fall in the effective zone and dead zone (sleepers). As a result various diameters of wheel have different coverage. The instrumented portions are fixed in such a way to give best coverage for the wheels of interest. Once installed the system's wheel coverage pattern doesn't change.

As the Train passes over the instrumented rail, data is collected and sent to the system in real time. The data collected is tested, all extraneous noise removed and the pure raw data analyzed. Analyzed data is converted to a report form and sent through GPRS network to central Server. A Computer in C&W Control room (or anywhere as desired) continuously scans for report every 2 minutes/lesser and displays the newest report as and when released by the system. If defective axles found, an audio visual alarm is activated. All reports can be viewed and consolidated period wise in the dedicated website.

3 Criterion of WILD alarm

After analysis of information received WILD generates two types of alarm.

1) Maintenance Alarm 2) Critical alarm

A detail of limits and action to be taken based on those alarms is tabulated (Table-1) below:

Alarm	Max Dynamic Load for a wheel	Impact Load Factor for a wheel	Action to be taken
Maintenance	≥ 20 T	≥ 2.0 < 4.5	i) Coaching stock- intimation to be given to the primary/secondary depot for attending the defects. ii) Freight stock- intimation to be given to rake terminating point for attending the defects.
Critical	≥ 35 T	≥ 4.5	Thorough examination by TXR at TXR point for checking any damageable defect and to decide for detachment or to allow the stock to run upto train terminating point.

4 WILD alarms generated on Gadag-Ballari section:

Maintenance and critical Alarm (Table 2)

Year	Month	Wheel	Nos. of maintenance alarms generated	Nos. of critical alarms generated	Nos. of trains passed	Nos of trains generated maintenance and critical alarm
2017	Nov*	89176*	237*	4*	262*	103*
2017	Dec	394712	1355	6	1085	474
2018	Jan	438408	1150	4	1239	527

Maintenance and critical Alarm (Table 2) Contd…

Year	Month	Wheel	Nos. of maintenance alarms generated	Nos. of critical alarms generated	Nos. of trains passed	Nos of trains generated maintenance and critical alarm
2018	Feb	356648	1371	4	1037	445
2018	Mar	355200	2461	2	1109	584
2018	Apr	342812	1638	1	1068	445
2018	May	395988	1250	4	1180	547
2018	Jun	374876	1210	2	1122	488
2018	Jul	338624	882	2	1005	391
2018	Aug	201060	608	1	624	266
2018	Sep	416864	1303	2	1240	487
2018	Oct	472812	1197	2	1346	524
2018	Nov	461896	1196	1	1306	494
	TOTAL	**4639076**	**15858**	**35**	**13623**	**5775**

*only one week data is available for Nov'2017

To study the level of maintenance and condition of rolling stock, data generated by WILD installed at km 131/0-1 near Ginigera station of Hubballi division is taken for analysis. (Table-2)

As WILD was made functional in the month of November'2017 (it was defective for almost one and half year), data from one year i.e. from Nov '2017 to Nov' 2018 is taken to understand the results. It is also relevant to mention that Hospet station is the next TXR point (nearly 12km from the WILD location in one direction) and Hubballi is another TXR point (nearly 132km from the WILD location in other direction). Distance of TXR point from the location of WILD is one aspect which need relook considering the damaging effect of defective rolling stock.

4.1 Two major observations can be made out from the details tabulated above:

4.1.1 Observation: From Nov'17 to Nov' 2018 (say in one year time span) total 4639076 numbers of wheels have passed over WILD. These 4639076 nos. of wheels have generated 15893 nos. of alarms. It means 0.34% of wheels have generated alarm.

Comments: Before permitting the CC+8+2 loading on identified route, installation of WILD in those routes is one of the recommendation done by CRS.As per Annual Report 2007-08 issued by Chief Commissioner of Railway Safety(CCRS) it is mentioned by CCRS that there is no definite co-relation being available between the critical alarm and the condition of the rolling stock from WILD systems recently installed, more comprehensive studies are necessary for monitoring the health of rolling stock particularly on routes where higher axle load trains are running. The percentage of critical and maintenance alarms based on available data from WILD studies being in the range of 0.02 and 0.1% respectively of total wheel population, a system of reducing their number by improving maintenance and monitoring methods should be evolved. In summary, Commission recommend installation of better monitoring systems than the one currently available.

Paragraph above clearly brought out the serious concern of CCRS even for the percentage range

of 0.02 and 0.1% alarm but actual % generation of maintenance alarm is 0.34% i.e. 3 times more than what was projected, which is a serious matter.

4.1.2 Observation: From Nov'2017 to Nov' 2018 total 13623 nos. of trains have passed over the WILD location and 5775 nos. of trains have generated alarms.

Comments: It shows that 42% of trains have generated alarms which further imply that almost every alternate train have maintenance issues of rolling stock. This is high not only in terms of number but also in terms of percentage and exposes the level of stress under which assets are put in service. This is biggest challenge as far as handling of quantum of operational constraints raised out of alarm generated is concerned. So far no concrete system is put in place to take required action on such alarms.

5 Action taken on alarms in Gadag-Ballari section:

In the one year of span total 15858 nos. of maintenance alarm and 35 nos. of critical alarm is generated. Practically no action is being taken on maintenance alarm. So far it has not been figured out how to work over this information to take some action in track as well as rolling stock maintenance and ultimately it has become just a record. Even there are hardly any action is being taken on critical alarm also. In most of the critical alarm cases defective vehicle was allowed to run over system for more than 1000kms or more than weeks even after repeated critical alarms by same vehicle. It is not possible to discuss all the cases but few cases are tabulated (Table-3) here to understand the pathetic scenario of cluelessness on action required for critical alarms.

S. N.	Vehicle Nos.	Date of critical alarm	Magnitude of alarm	Action taken	Observations
1	SC GSCN 96286	29.11.17	ILF=5.23 IL=33.76	Coach was detached at Tirupati station on 30.11.17 and one wheel was replaced	i) Coach was allowed to run at sectional speed for almost 450kms. ii) One weld failure took place in the section.
2	SC GSCN 14241	03.12.17	ILF=3.81 IL=35.95	Coach was detached at Kakinada port station on 06.12.17 and wheel was replaced.	i) Coach was allowed to run at sectional speed for almost 1000km. ii) One weld failure took place in the section.
3	SC GS 09443	14.01.18	ILF=4.6 IL=47.46	No action was taken	i) Coach was allowed to run at sectional speed for four days. Even after second critical alarm it ran for 450km. ii) One weld failure took place in the section.
		18.01.18	ILF=2.83 IL=35.00	Coach was detached at Tirupati station on 19.01.18	
4	SC WGSCN 99241	16.01.18	ILF=2.38 IL=35.56	No defect noticed.	Defective coach was allowed to run at sectional speed for 27 days. It was detached only after fourth critical alarm is generated.
		03.02.18	ILF=3 IL=38	Skid marks was found on 4th wheel at TXR point	
		09.02.18	ILF=3.13 IL=37.88	Skid marks was found on 4th wheel at TXR point	
		11.02.18	ILF=3.12 IL=35.57	Coach was detached at Tirupati station	
5	CR WGSCN 97239	07.04.18	ILF=4.79 IL=39.58	Shelled thread was found	Coach was allowed to run even after defective noticed in visual inspection.
6	CR BOXN 25010106687	19.11.18	ILF=2.15 IL=40.00	Metal deposition was found over wheel	Coach was allowed to run even after defective noticed in visual inspection.

Following important points need to be discussed in connection with observations made in above table:

5.1 Presently in Indian Railway best grade of rail in use is 60kg 90 UTS rails on operational routes. Permissible limit of stress in this rail is 46.8kg/mm2 .CC+8+2 routes are permitted with 22.9t axle load. For generation of critical alarm, value for a wheel is set for more than 35t, which is almost equal to 300% dynamic augment. This much magnitude of load will create stress upto 55.28kg/mm2, which is much more than permissible 46.8 kg/mm2 in 60kg 90 UTS rails. Situation will be even worse in 52kg rails. The permissible criteria for maintenance alarm generation 20t and for critical alarm 35t, is almost 75% and 200% respectively higher than the permissible axle load on IR.

5.2 At SN.3 in table above, coach number SC GS 09443 was not detached even after critical alarm of magnitude 47.46t was noticed. Dynamic augment for such load is almost 310 % which indicated heavy dynamic forces. Such dynamic forces of repeated nature will lead to reduce fatigue life of rail/weld that may cause sudden failure also and hence standard of maintenance of rolling stock needs to be checked as there is limitation in design of track structure.

5.3 At SN.4 in table above, it can be seen that coach number SC WGSCN 99241 was detached only after fourth critical alarms, although skid marks were noticed on wheels in two alarms. Either it raises a big question mark over our skills to detect the defects or it indicates complete failure of decision makers to decide the fate of defective vehicle.

6 Impact over track degradation

Track is kept in service under continuous dynamic forces. Due to impact of those repeated dynamic forces degradation in track structures takes place. Different components of track structures like rails, ERCs, sleepers, ballast etc; gets degraded due to stresses. Even track parameters frequently get disturbed due to dynamic forces. There is no confirmed correlation available to relate the level of dynamic forces and degradation in track structures because there are various factors which contribute significantly in track degradation like vehicle condition, thermal stress, over loading, quality of track components, track geometry etc;. Rail is most important component of track structure and it is the only component which has direct interaction with rolling stock. Effect of dynamic forces can be represented in true sense by assessing the degradation in rails. Wear, rail/weld failures, IMRs, scabbing etc; are few parameters which can be used to show the degradation in rails due to dynamic forces. But in most of these parameters, level of track maintenance and geometry of track has major role and these parameters are quantitative in nature. A variation in USFD results is other parameter to shows the deterioration of health of track over a period of time. As it is seen in above paragraphs that the almost 42% of trains running in system generates maintenance/critical alarms. Level of impact load is very high as compared to designed stress of rails. Due to this plastic deformations took place in rails and rail/weld becomes weaker by repeated hammering under such load. It can be very well assessed by USFD results of Gadag- Ballari section of Hubballi division shown in the Table-4. (Data used for comparison here is taken from March'17 to March'18 as from March'18 to Nov'18, approximately 30km of CTR (P) work was done and approximately 18km of double line section is opened. Hence data after March'18 cannot be considered for comparison).

Comparison of USFD Test Result (Table 4)

SN	Rail testing		Weld Testing		Total	Month/Year	Remarks
	OBS(R)	OBS(W)	DFW(O)	DFW(R)			
1	98	395	--	--	493	June-17	Almost 64% increase in defects in USFD Rail Testing between four rail testing in a span of 10 months.
2	185	357	--	----	542	Sep-17	
3	203	477	--	----	680	Dec-17	
4	232	578	---	---	810	Mar-18	
5	----	----	2778	240	3018	2017-18 (Mar-18) (3 years testing)	Almost 43% increase in defects in USFD Weld Testing from previous testing.
6	----	----	1974	137	2111	2014-15 (Mar- 15)	

7 Discussion and Recommendations:

7.1 Discussion-1:

During 2005 to understand the dynamic augment of various rolling stock RDSO has done some field trial, which was published as C-100 report. In that report

there is mention of one ICF coach trial which was having flat of size 18x25x3mm on one wheel. It is found that the flat of that size on a wheel produces dynamic augment of 257% at speed of 96kmph as compared to normal dynamic augment of 57.4%. It can be seen that the small flat size in a wheel (say 15-20mm) can produce the instantaneous wheel load of such a magnitude that stresses in rails crosses it's permissible stresses limit.

Recommendation-1:

Presently permissible limit of flat on wheel tread is 50mm for coaching stock/locomotives and 60mm for Goods stock. It is understood that up to 50/60mm flat will be available on high number of wheels. As per discussion above, by keeping permissible limit so high, rails are kept under stresses most of the time in its plastic limit. Increasing UTS of rails will not help as it will make rails more brittle and hence difficult to handle. Therefore, permissible limit of flat on wheel tread need to revise as higher speed means higher dynamic augment and further it means much higher stress in rails.

7.2 Discussion-2:

As discussed previously that in WILD maintenance alarm is permitted from 20t to 35t in a wheel. With permissible stress limit of 46.8 kg/mm2 for 60kg 90 UTS rail ,permitted load on a wheel comes out around 10-12t,which in line with 22.9t axle load. For maintenance alarm vehicles are not being even checked at next TXR point, leave aside its detachment. It means that upto 35t impact load on a wheel; vehicle is allowed to run with maximum speed, which is 300% dynamic augment. Again with these criteria; rails are officially allowed to be stressed upto plastic limit without any check.

Recommendation-2:

Both criterions for maintenance and critical alarm need to be revised as with increase of speed and axle load, stresses in rail will lead to sudden fractures.

7.3 Discussion-3:

High impact load in wheels are caused by many defects like eccentricity, discrete defects, periodic non-roundness, non-periodic non-roundness, corrugation, roughness, flats, spalling, shelling. Many of these defects are not visually identifiable. As almost all of the examinations after stopping of trains are visual and the technicians are only looking for known visually identifiable defects. Many of the defects which can generate high wheel impact loads / high ILF are not classified as defects in the regular maintenance instructions presently in use on the IR as these are based on visual inspection of wheel treads.

In Indian Railways there is system of identifying and removing defective wheels based primarily on visual inspections. This filters out all bad wheels with known and visually identifiable defects. Therefore any wheel identified as defective by a WILD system is very unlikely to have any visual defect. Hence a large number of cases of false alarms have been reported by field unit and no action is being taken even on critical alarm due to limitations of visual examination. Dependency on visual examination is major impediment to improve the quality of maintenance of rolling stock and reducing its impact on track at higher speed with higher axle load.

Recommendations-3:

Many railways, advanced in technology have already done enough research to identify the defects other than visual examination. WILD captures data in a snapshot manner. Latest research over the world has found that it is possible to use this data for condition monitoring and predictive maintenance of rolling stock. Association of American Railroads (AAR) has created the InteRRIS service which today is a benchmark for use of trackside detectors integrated into maintenance and operations. This now being improved further under the AAR ATSI (Advanced Technology Safety Initiative) Programme as an Equipment Health Monitoring Service (EHMS).

With the help of such technology condition monitoring & predictive maintenance can be done duly removing dependency on visual examination for maintenance.

7.4 Discussion-4:

As discussed in previous paragraphs about the capabilities of WILD to assess the vehicle dynamics and its impact on track. But WILD can assess impact only in terms of vertical load. Because of these limitations, WILD is laid on straight tracks only. With increase in axle load and speed, assessment of thrust on rails at curves is very important to arrive at the maintenance requirement of track as well as rolling stock.

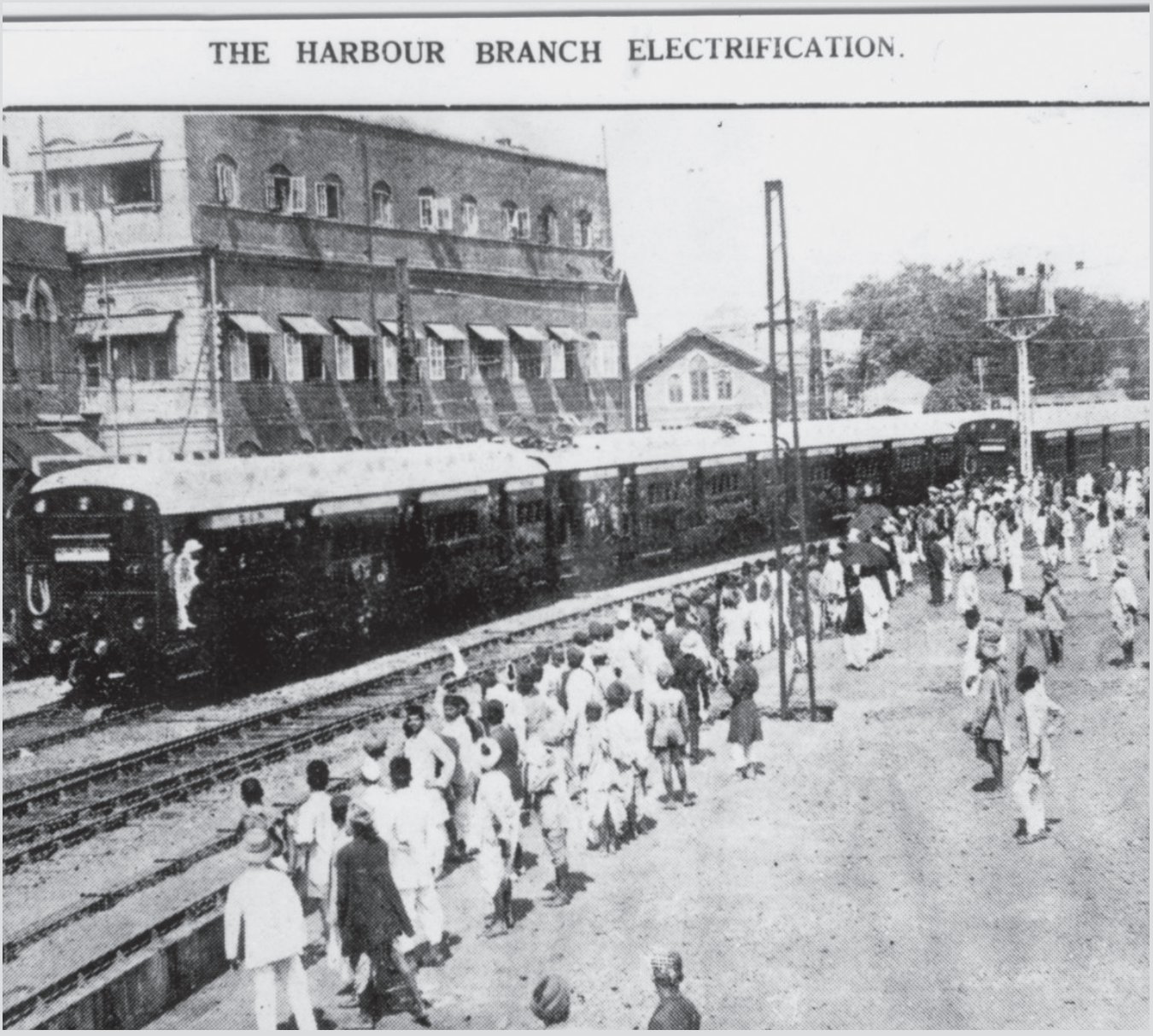
Recommendation-4:

Truck Performance Detector is another wayside detector which can measure the lateral load as well as vertical load of each wheel. This detector is already in use in many countries.

8 Conclusion:

Increase in vehicle speed and axle load is depended on vehicle dynamics. Stability of vehicle and track at higher speed and axle load is more important to ensure safety & riding comfort. With the minor redesigning of few components of track, improvement in layout and improved maintenance practices, a limited increase in speed of the vehicle and increase of axle load can be achieved but it will not be as significant as it

is desired. There is a limitation also in redesigning of track structure. Conditioning monitoring and preventive maintenance of rolling stock is major important activities which are required to meet the desired improvement. Existing standard of maintenance and monitoring of rolling stock will not be enough. Many permissible limits of maintenance & monitoring are also required to be changed so the dynamic forces can be brought down so that safety and life of assets can be enhanced. Presently degradation of track is being analysed in isolation. It is being done as more about understanding the pattern of degradation rather than contributory factors for degradation. 64% increase in defects during USFD rail testing (as shown in Table-4) cannot be just because of thermal stresses, poor track maintenance practices. Impact of vehicle dynamic forces are much bigger contributory in degradation of track, hence stipulation of life for rails should be on the basis of “peak tonnage” rather than just on tonnage. GMT needs to be revised as GMPT (GROSS MILLION PEAK TONNAGE).It is also seen that track maintenance unit never analyse gravity of critical/ maintenance alarm. Status of action taken & findings of defects after critical alarms by coach & wagon unit never discussed. Rise of USFD defects should be seen as flagging of degrading track condition. Overall better coordination among rolling stock & track maintenance unit to be done on real time basis by sharing the details and integration of their respective monitoring system of maintenance to sustain the impact of increase in speed and axle load in an economical way also.



Some of the crowd watching the first Electronic Train carrying H. E. the Governer of Bombay and party, leaving Victoria Terminas Station.

Updates Of Codes & Manuals

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
TRACK			
1. Indian Railways P-Way Manual			
01.	01	July 2021	Para No 915 replaced
02.	02	Aug. 2021	Para 636(2)(e), 702(1)(d), 702(2), 717(1)(b), 718 replaced Para 702(1)(2) renamed as Para 702(1)(b)
03.	03	21.10.2021	Heading of Para 228 replaced New para 228(4) added
04.	04	03.11.2021	Para 408(2)(e) replaced
2. LWR Manual			
3. Track Machine Manual			
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
04.	04	Sept. 2018	Para 6.6, 8.14, 8.15, 8.15.1 & 8.15.2 modified,
05.	05	Jan. 2021	Para 10.6 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,

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
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
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.
37.	37	09.10.2019	Para 1102.2(iv) replaced, Insert 11/2a proforma, Para 1107(15)(b)(i) & 1107(15)(b)(ii) modified
38.	38	14.01.2020	Para 317 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
06.	06	04.11.2019	Modified paras 1.2, 1.5 I (f), 5.12.1, 5.12.2 (a), (b), (c), 5.12.3, 5.12.5, 5.12.6, 5.12.7, 5.12.8, 5.16.2.7 (b).
07.	07	11.11.2019	Para 5.10
08.	08	11.11.2019	Para 3.1 of Appendix V (Clause 6.9.3) modified
09.	09	06.07.2020	Para No. 1.5 (I) - (c) & (g) modified

S.NO.	ACS NO.	DT OF ISSUE	REMARKS
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, existing para 7.1.5 modified
08.	08	23.05.2019	Replace the clause 7.2.6.4.2.4.1, Replace clause 7.2.6.4.2.4.2 and 12.3.2, Replace clause 15.4.2.2.1, Appendix B (Table B1), Appendix B (Table B2), Appendix B (Clause B-7.1), Appendix B1 (Fig B1-2),
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
03.	09	19.11.2019	Replace paras 2.1.2, 5.3.2, 5.3.4, 5.3.5, 5.3.5.1, 12.1.1, 12.1.2, 12.2, 12.3
6. Indian Railways Welded Bridge Code			
01.	01	16.02.2015	Para 27.1 replaced
02.	02	11.07.2018	Para 27.1 replaced
Works			
1. 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
07.	57	08.01.2020	Para No 1829 should be amended



Byculla1853

IRICEN Calendar of Courses -2021

COURSE NO	COURSE NAME	FROM	TO	DURATION	TIMINGS	ELIGIBILITY
NOVEMBER						
21314	IRICEN Day	08.11.21	10.11.21	3 DAYS		
21785	BASIC TRACK MAINTENANCE	08.11.21	12.11.21	1	BN	All officers
21786	Concrete Technology- mix design, RMC, and durability	08.11.21	12.11.21	1	AN	All officers
21231	WEBINAR	10.11.21	10.11.21	1 DAY		All officers
21787	Bridge Planning and bridge foundations	15.11.21	19.11.21	1	BN	All officers
21788	AT and FB welding of Rail	15.11.21	19.11.21	1	AN	All officers
21205	SR. PROF. P. WAY	22.11.21	17.12.21	4	CAMPUS	JAG, Selection Grade, SAG
21415	BENTLEY	22.11.21	26.11.21	1	CAMPUS	All officers
21789	Laying and maintenance of LWR	22.11.21	26.11.21	1	BN	All officers
21790	Modern surveying for railway projects	22.11.21	26.11.21	1	AN	All officers
21232	WEBINAR	24.11.21	24.11.21	1 DAY		All officers
21503	PSU COURSE SLOT	29.11.21	31.12.21	5	CAMPUS	PSU TRAINEES
21416	RAIL WHEEL INTERACTION	29.11.21	03.12.21	1	CAMPUS	All officers
21791	Formation- construction including ground improvement and geotechnical aspects	29.11.21	03.12.21	1	BN	All officers
21792	Track Tolerance and Track monitoring	29.11.21	03.12.21	1	AN	All officers
DECEMBER						
21417	USFD	06.12.21	10.12.21	1	CAMPUS	JS, SS, JAG
21793	Advance track Maintenance	06.12.21	10.12.21	1	BN	All officers
21794	Construction of PSC bridge superstructures	06.12.21	10.12.21	1	AN	All officers
21418	Points & Crossing/ Layout Calculation	13.12.21	17.12.21	1	CAMPUS	JS, SS, JAG
21795	Steel fabrication and quality Inspection and launching of girders	13.12.21	17.12.21	1	BN	All officers
21796	TMS	13.12.21	17.12.21	1	AN	All officers
21233	WEBINAR	15.12.21	15.12.21	1 DAY		All officers
21315	CAO SEMINAR	20.12.21	21.12.21	2 DAYS	CAMPUS	CAOs OF ALL RLYS.
21206	SR. PROF. BRDGE	20.12.21	14.01.22	4	CAMPUS	JAG, Selection Grade, SAG
21419	MIDAS	20.12.21	24.12.21	1	CAMPUS	AEN/Design, Design Assistants
21797	Turnouts and layout calculations	20.12.21	24.12.21	1	BN	All officers
21798	Dispute resolution and arbitration	20.12.21	24.12.21	1	AN	All officers
21316	SPARE SLOT	27.12.21	28.12.21	2 DAYS	CAMPUS	
21799	Tenders and contracts	27.12.21	31.12.21	1	BN	All officers
21800	Design, Laying and Maintenance of Curve	27.12.21	31.12.21	1	AN	All officers
21420	Mechanised Track Maintenance	27.12.21	31.12.21	1	CAMPUS	All officers
21234	WEBINAR	29.12.21	29.12.21	1 DAY		All officers
21317	SPARE SLOT	30.12.21	31.12.21	2 DAYS		

SSTW Calendar of Courses -2021

COURSE NO	COURSE NAME	FROM	TO	TIMING	Duration Weeks	ELIGIBILITY
NOVEMBER						
21624	Referesher Course P-Way	08.11.21	26.11.21	CAMPUS	3	SSE/JE/P-Way
21820	Geotechnical engineering and construction of formation	08.11.21	12.11.21	CAMPUS	1	SSE/JE/Works
21983	Mechanized Tamping	08.11.21	12.11.21	BN	1	SSE/JE/P. WAY
21984	Land Management	08.11.21	12.11.21	AN	1	SSE/JE/WORKS
21625	Referesher Course Works	15.11.21	26.11.21	CAMPUS	2	SSE/JE/Works
21985	Important aspect of Track monitoring and TMS data entry	15.11.21	19.11.21	BN	1	SSE/JE/P. WAY
21986	Water supply, Sewerage system and Waste water recycling system.	15.11.21	19.11.21	AN	1	SSE/JE/WORKS
21987	DEEPSCREENING & Track Laying - Mechanised & Manual	22.11.21	26.11.21	BN	1	SSE/JE/P. WAY
21988	Water proofing and Termite treatment of building – During construction and maintenance.	22.11.21	26.11.21	AN	1	SSE/JE/WORKS
21626	Referesher Course USFD	29.11.21	03.12.21	CAMPUS	1	SSE/JE/USFD
21821	SURVEY	29.11.21	03.12.21	CAMPUS	1	SSE/JE/Works
21989	LWR : Laying, maintenance and Rail welding	29.11.21	03.12.21	BN	1	SSE/JE/P. WAY
21990	Construction & maintenance of passenger amenity works and provisions of SOD.	29.11.21	03.12.21	AN	1	SSE/JE/WORKS
DECEMBER						
21627	Referesher Course P-Way	06.12.21	24.12.21	CAMPUS	3	SSE/JE/P-Way
21822	Concrete technology	06.12.21	10.12.21	CAMPUS	1	SSE/JE/Works
21991	IRPWM provisions of Routine track maintenance and safety precautions	06.12.21	10.12.21	BN	1	SSE/JE/P. WAY
21992	Preparation of Estimate, schedule and Contract management- Execution aspect	06.12.21	10.12.21	AN	1	SSE/JE/ALL
21823	PSC/Concrete Bridge construction and Quality control.	13.12.21	17.12.21	CAMPUS	1	SSE/JE/Works-BRI.
21993	Curve- Speed potential & Realignment. Level crossings	13.12.21	17.12.21	BN	1	SSE/JE/P. WAY
21994	Steel/Composite girder fabrication and HSFG bolts	13.12.21	17.12.21	AN	1	SSE/JE/BRIDGE
21628	Referesher Course Works	20.12.21	31.12.21	CAMPUS	2	SSE/JE/Works
21995	Track Renewal works: TRR, TFR, TSR, TWR and TBR	20.12.21	24.12.21	BN	1	SSE/JE/P. WAY
21996	Building: Planning, Construction, Important NBC Provisions and colony maintenance.	20.12.21	24.12.21	AN	1	SSE/JE/ ALL
21824	Rail wheel interaction and Derailment investigation	27.12.21	31.12.21	CAMPUS	1	SSE/JE/P-Way
21997	Turnouts, Layout Calculation and Yard drainage	27.12.21	31.12.21	BN	1	SSE/JE/P. WAY
21998	Launching of PSC/Steel girder and temporary arrangements.	27.12.21	31.12.21	AN	1	SSE/JE/WORKS



Calender of Courses -IRICEN 2021

Month & Year	July.2021				Aug.2021					SEPT.2021				Oct.2021				Nov.2021					Dec.2021				Jan.2022			
HOSTEL CAP	75	50	70	65	65	95	85	70	60	65	65	50	95	95	95	95	95	70	95	95	70	85	60	60	90	85	15	15	0	0
Date (Monday)	5	12	19	26	2	9	16	23	30	6	13	20	27	4	11	18	25	1	8	15	22	29	6	13	20	27	3	10	17	24
IRSE 2018/19													21005/21006 IRSE PHASE-II																	
Trainees													70	70	70	70	70	70	70	70										
AWARENESS COURSES																		DIPAWALI												
Trainees																														
Integrated courses	21102 INTEGRATED									21103 INTEGRATED																				
Trainees	25	25	25	25	25	25	25				25	25	25	25	25	25	25		25	25	25	25								
Sr.Prof. courses	21202 Sr.Prof.		21203 Sr. Prof. P.Way					21204 Sr.Prof.Bridge													21205 Sr. Prof. P.Way				21206 Sr.Prof.Bridge					
TRAINEES	15		15	15	15	15		15	15	15	15										15	15	15	15	15	15	15	15		
CAMPUS COURSE				21408 RWI	21409 USFD	21410 P & C - LAYOUT	21412 TUNNELING				21414 MECH MAINT											21416 RWI	21417 USFD	21418 P & C - LAYOUT		21420 MECH MAINT				
Trainees				25	25	25	35	35			25											25	25	25		25				
Software courses	21406 BRIDGE DESIGN		21407 MIDAS			21411 BENTLY				21413 STAADPRO											21415 BENTLY				21419* MIDAS					
Trainees	35		30			30				30											30				30					
ONLINE COURSE BN	21751 Building - planning, constructi	21753 Mech Maint. of track	21755 Bridge Planning and bridge	21757 Basic track maintena nce	21759 Advance concepts of bridge	21761 Advance track Maintena	21763 Tenders and contracts	21765 Laying and Maint. of LWR	21767 Steel fabricatio n and	21769 T/Outs and Layout calc.	21771 Formation- constructi on	21773 D'ment Investigati on	21775 Bridge- inspection ,	21777 USFD of rails and welds	21779 Building - planning, constructi	21781 Mech Maint. of track	21783 Advance concepts of to		21785 Basic track maintena nce	21787 Bridge Planning and bridge	21789 Laying and Maint. of LWR	21791 Formation- constructi on	21793 Advance track Maintena	21795 Steel fabricatio n and	21797 T/Outs and Layout	21799 Tenders and contracts				
ONLINE COURSE AN	21752 Bridge Rehab - case studies	21754 Quality Assurance in Execution	21756 Land Managem ent	21758 Const. of PSC bridge superstruc tures	21760 AT and FB welding of Rail	21762 Conc Tech- mix design, RMC, and	21764 TMS	21766 Modern survey for RLY PROJ.	21768 Track Tole. & Track monitorin	21770 DisputeRes olution and Arbitration	21772 Design, Laying and Maintena nce of	21774 Bridge bearings including maintena	21776 Laying and Mechanis ed Renewal	21778 Formation- maint. & Rehabilit ation	21780 Bridge Rehab - case studies	21782 Quality Assurance in Execution	21784 Land Managem ent		21786 Conc Tech- mix design, RMC, and	21788 AT and FB welding of Rail	21790 Modern survey for RLY PROJ.	21792 Track Tole. & Track monitorin	21794 Const. of PSC bridge superstruc tures	21796 TMS	21798 DisputeRe solution and Arbitratio	21800 Design, Laying and Maintena nce of				
WEBINARS	WEBINAR	21223 WEBINAR		21224 WEBINAR		21225 WEBINAR		21226 WEBINAR			21227 WEBINAR		21228 WEBINAR		21229 WEBINAR		21230 WEBINAR		21231 WEBINAR		21232 WEBINAR			21233 WEBINAR		21234 WEBINAR				
Courses for PSU								21502 PSU COURSE SLOT														21503 PSU COURSE SLOT							
Trainees								20	20	20												20	20	20	20	20				
HAG/SAG SEMINARS		21307					21308		21310			21312	21309				21311	21314 IRICEN DAY				21315	21316	21317						
Trainees		25					25		25			25													25	25				
		CTE					CBE/ TP	CE-		EPC- PPP/SLOT			SLOT/ CE-TMC												CAO	SLOT / SLOT				
Spare Capacity available	19	44	24	29	29	-1	9	24	34	29	29	44	-1	-1	-1	-1	-1	24	-1	-1	24	9	34	34	4	9	79	79	94	94

Calender of Courses -SSTW 2021

Month & Year	July.2021				Aug.2021					SEPT.2021				Oct.2021			Nov.2021					Dec.2021			
HOSTEL CAP	50	40	50	50	50	40	50	50	50	50	40	45	45	45	0	55	30	0	50	50	50	40	50	55	50	45
Date (Monday)	5	12	19	26	2	9	16	23	30	6	13	20	27	4	11	18	25	1	8	15	22	29	6	13	20	27
Track courses			21617 Refresher (USFD)	21618 Refresher Course (P WAY)			21814 RWI	21619 Refresher Course (P WAY)			21621 Refresher (USFD)	21622 Refresher Course (P WAY)				21817 RWI	21819 SPL COURSE	NO TEACHING ACTIVITY. IRICEN DAY, DEEPAWALI		21624 Refresher Course (P.Way)		21626 Refresher (USFD)	21627 Refresher Course (P.Way)			21824 RWI
Trainees	30	20	30	30	30	20	30	30	30	30	20	30	30	30		30	30		30	30	30	20	30	30	30	25
Works courses			21813 GEO. ENGG/FO					21620 Refresher Course (Works)		21815 SURVEY	21816 CONC TECH								21820 GEO. ENGG/ FORM ATION	21625 Refresher Course (Works)		21821 SURVEY	21822 CONC TECH		21628 Refresher Course (Works)	
Trainees	20		20					20	20	20	20								20	20	20	20	20		20	20
Bridge courses		21812 BR. CONST. /										21623 Refresher Course (Bridges)				21818 BR. INSP. MAINT.								21823 BR. CONST. / QLTY CTNL			
Trainees		20										15	15	15		25								25			
Online courses BN	21951 MECH. TMPNG	21953 TRACK MONI/T	21955 LWR	21957 IRPWM PROVISIO	21959 DEPSCR NING &	21961 Curve	21963 Track Renewal	21965 T/O, LAYOUT	21967 MECH. TMPNG	21969 TRACK MONI/TMS	21971 L W R	21973 IRPWM PROVISIONS FOR Track Maint.	21975 DEEPSCRNING & Track Laying - Mechanised & Manual	21977 Curve		21979 Track Renewal Works	21981 T/O, LAYOUT & YD. DRNG		21983 MECH. TMPNG	21985 TRACK MONI/TMS	21987 DEEPSCRNING & Track Laying - Mechanised & Manual	21989 LWR	21991 IRPWM PROVISIONS FOR Track Maint.	21993 Curve	21995 Track Renewal Works	21997 T/O, LAYOUT & YD. DRNG
Online courses AN	21952 LAND MGMT.	21954 Water Proofg/	21956 WATER SUPPLY/S	21958 Pass. Amenitiy	21960 Estimate/ Schedule	21962 GIRDER Launchin	21964 STL/Com posite	21966 BLDG. PLANNG.	21968 LAND MGMT.	21970 Water Proofg/ Termite Treatment	21972 WATER SUPPLY/SEWA GE SYS.	21974 Pass. Amenitiy Works/SOD	21976 Estimate/Schedule& Contract MGMT.	21978 STL/Composit e GRDR Fab		21980 GIRDER Launching	21982 BLDG. PLANNG. CONST.		21984 LAND MGMT.	21986 WATER SUPPLY/ SEWAGE SYS.	21988 Water Proofg / Termite Treatment	21990 Pass. Amenitiy Works/SOD	21992 Estimate/Schedule & Contract MGMT.	21994 STL/Composite GRDR Fab	21996 BLDG. PLANNG. CONST.	21998 GIRDER launching
PSU courses				21509 PSU COURSE SLOT																						
Trainees				20	20	20	20																			
Spare capacity available	0	10	0	0	0	10	0	0	0	0	10	5	5	5	50	-5	20	50	0	0	0	10	0	-5	0	5

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