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INDIAN INSTITUTION OF BRIDGE ENGINEERS

BRIDGE

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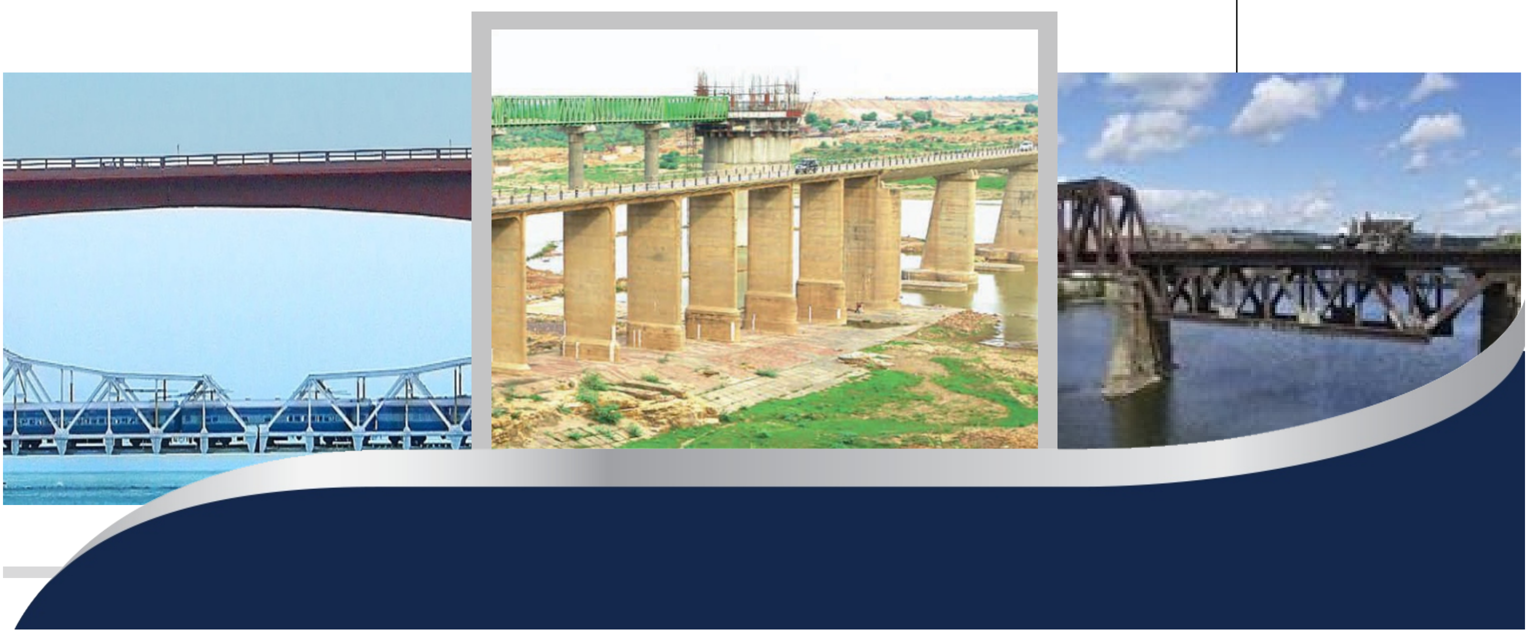
CONTENTS



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Indian Institution of Bridge Engineers (IIBE) is now 25 years old. Before IIBE, the bridge engineers had no exclusive forum to exchange ideas and knowledge of bridge engineering. We have Institution of engineers which is conglomerate of all engineering. civil engineering division is quite important but it caters all branches and rarely technical program for bridge engineers are held.

Engineering is considered as one part of larger topic of roads and Highways. Indian Railways give considerable importance to Bridge engineering and number of technical programs are held but only for the Railway Engineers but now Indian Institution of Bridge Engineers had provided exclusive platform to Bridge Engineers for dissemination of knowledge / new developments in bridge engineering to all Bridge Engineers.

MISSIONS:

- Bridging the Gaps - Inter-Continent & Intra-Continent
- Building Bridges of Camaraderie, Understanding, Friendship and Co-operation among Engineers not only of India but of the World

HIGHLIGHTS:

- 33 state & local centers spread all over India.
- 6300 individual members spread all over India and in many countries around the world.
- 169 Institutional Members
- 29,200 pages of Technical Literature published by IIBE in 25 Years since inception.

Held Ten Most Outstanding Bridge National Award competitions annually & awarded trophies in glittering functions.

ABOUT IIBE

Er. Madhav C. Bhide
Founder & Hon. Director General, IIBE

IIBE was established in year 1989 by **Late Shri Er. M. C. Bhide (Chief Eng. Railway)** with the aim to promote bridge engineering in the Country. After sad demise of **Shri. M. C. Bhide** on 7th July, 2018, IIBE will continue to promote Bridge Engineering with same enthusiasm and passion. IIBE is the only professional body, exclusively dealing in bridge engineering. It provides a common platform among the stake holders to disseminate knowledge, ensure continual development of bridge engineering and learn the use of new innovations and products.

Spread over 33 state & Local centers with about 7000 strong members force and 169 institutional members, IIBE has created laurels for the Country. It has also published 29,200 pages of Technical Literature. IIBE holds Outstanding Bridge National awards competition, annually.

Er. D. G. Diwate
Newly Elected Director General, IIBE
Principle Chief Engineer, Indian Railway - Retd.

Er. Deepak G. Diwate Graduated in civil Engineering From Collage of Engineering Pune in 1974. Through 1975 UPSC Exam he joined Indian Railway Services of Engineers (IRSE) as Class I Officer and Retired as Principle Chief Engineer of Indian Railways in 2011. During his service he worked on many important Railway Infrastructure Projects Viz.

He occupied the important post of Principle Chief Engineer NE Railway and South West Railway, Director (Board Member) Konkan Railway Corporation, Chief Engineer Konkan Railway Project, Chief Project Managerfor Tunnels of Mumbai - Pune Expressway, Project Director of Katara to Loole portion of Udhampur - Baramula J&K Railway Project.

Recently he was on Mumbai Metro 3 - 33.5KM long fully underground Metro Construction Project as CGM/Advisor (Civil). Presently he is working as Arbitrator and Dispute Adjudication board Member on many Infrastructure Projects.

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BRIDGE - 2018

About Conference BRIDGE - 2018 :

Conference on **"Innovative Technologies for Bridges"**

Indian Institution of Bridge Engineers successfully conducted two day's conference at Vishweswaraiya Bhavan, Lucknow on 25th & 26th May, 2018 supported by National Highways Authority of India, Public Works Department Uttar Pradesh, U.P. State Bridge Corporation Ltd. and Lucknow Metro Rail Corporation Ltd.22 Speakers presented their paper in the conference and 309 delegates was present.

THEMES Of BRIDGE 2018 :

- **Innovative Bridge Technologies**
- **Cable Supported Bridges**
- **Special Bridge Devices**
- **Bearings, Expansion Joints & Special Seismic Devices**
- **Bridge Foundations**
- **Speedy Construction of Bridges**
- **Economization of Bridge Cost with Modern Design**
- **Health Monitoring & Evaluation of Bridges**
- **Rehabilitation of Bridges**

Modern Technology for Repairs, Rehabilitation of Pavements-White Topping

Dr.Vishal R. Thombare
MCGM

At present many metropolitan cities in the country including Mumbai face the problem of ad-dressing severe pavement distresses in the old flexible (bituminous)pavements after each monsoon incurring huge expenditure. Though cement concrete pavements, because of their long life and superior performance, greatly reduce the maintenance expenditure, often due to initial investment constraints this option is not chosen. There is need for development of new and innovative pavement repair and strengthening strategies which are cost effective and less maintenance Intensive. One such innovative pavement strengthening strategy is to provide Thin or Ultra-Thin white topping overlay. Ultra-Thin White topping is a rehabilitation technique inwhich a 50 to 100 mm thick layer of high strength, fiber-reinforced, cement concrete is placed on a milled surface of rutted and/or cracked bituminous concrete pavement. Thin white topping can be used for thicknesses more than 100 mm. Thin and Ultra-Thin White topping deliver the performance which is almost equivalent to that of a conventional concrete pavement at a competitive price. Thin and UTWT overlays have been successfully utilized as pavements for roads with low to medium traffic. This study provides the details of investigation, structural design, mix design, construction, and instrumentation carried out for an actual thin white topping pavement overlay constructed over an existing flexible pavem-ent for a road in Greater Mumbai. Through thermocouples installed in the pavement the temperature differential between the top and bottom ofthe TWT have been measured and interpreted. With the use of a specifically designed high early strength concrete it was possible to open the rehabilitated road within a week's time. Thorough the analysis of temperature data, it was found that the temperature differential between top and bottom of the thin white topping layer is much less than the one specified in IRC guidelines.

Key words: Thin White Topping, Ultra-Thin White topping, Bituminous Concrete, Fast track construction, High early strength concrete, Thermocouple

1.0 Introduction

Although concrete pavements staged a comeback in Mumbai in1990's and are now consider as a viable option to the bituminous pavements for certain set of conditions, the share of concrete in the total paved roads in the Mumbai is still low. Generally the thickness of con-crete pavements in Mumbai ranges between 280 mm to 350 mm. Though the design life of concreter pavement is 30 years, it has been found that quite a few of them are in good ser-vice condition beyond their design life. Unlike concrete pavements, their bituminous counter parts required repeated repairs and maintenance. Further, the design life of these pavements is assumed to be around 10 years, after which they needed upgradation and strengthening to cater to higher traffic. In case serious deterioration of the pavements occurs, earlier in service life, in the form of excessive rutting, cracking, stripping, settlements, potholes, etc., as is the case on many occasions, the rehabilitation of the pavements needs to be taken up prior to the expiry of the design life. For both repairs & strengthening of the bituminous pavements, the current practice in Mumbai and many other cities in India is to again goin for the same basic materials of construction, that is, bitumen. The higher initial cost of the concrete pavements and the reluctance to adopt the life cycle cost approach limits the videuse of concrete pavement. Looking to repetitive repairs and maintenance in bituminous pavement and higher initial cost of concrete pavement, one viable strengthening/rehabilitation option that reduces the deterioration and higher initial cost is thin and Ultra-Thin White topping (UTWT) pavement over the existing bituminous pavements.

The design life of Thin White Topping (TWT) is more than 20 years as compared to 10 years of bitumen pavement. With the use of higher grade of concrete the design thickness comes down to around 100 mm to 150 mm, which is almost half the conventional concrete pavement thickness. Moreover, the pavement can be opened to traffic at early age. Tayabji (2010), says well designed and well-constructed concrete overlays require low maintenance and can have low life-cycle cost. Fast trackpaving UTWT projects use high early strength concrete mixes, to satisfy the specific needs of special applications such as durability, Modulus of elasticity and flexural strength.

Many Researchers have studied aspects relating to fast track paving concrete, analysis and behavior of UTWT, performance of UTWT through instrumentation, design of UTWT, etc.

Recent experience in the U.S.A. and some other countries however indicates that thin concrete overlays in place of hot mix aspha lt may prove to be viable and cost effective solution for pavement rehabilitation.

From the critical appraisal it is clearly seen that, most of the work on white topping has been carried out outside India for their local conditions. White topping is a new concept in India. It is being implemented on experimental basis in India. Thus, there is need of a detailed study on white topping design methodology, considering Indian conditions.

In this paper, as a part of research, the use of TWT for the structural strengthening of flexible pavements in Mumbai Metropolitan region by constructing experimental sections with 100 mm to 150 mm TWT has been explored. The intent of this research study is to design and construct an appropriate TWT pavement for the selected section of road and evaluate its performance through appropriate instrumentation. The objectives include, i) carryout investigation for identifying the design parameters, ii) designing of appropriate TWT pavement for the selected section, iii) designing of appropriate high and early strength concrete mix for TWT, iv) construction of TWT with appropriate instrumentation for monitoring temperature and strain values at typical locations and v) analyze the temperature gradients across the TWT pavement.

The details of the test site and the investigations carried out at thesite are provided in the next section. The structural design details of UTW that is implemented are provided in Section 3. The design details and the characteristics of the concrete mix used for the construction of TWT are also provided in this section. The process of actual construction and the details on instrumentation are described in Section 4. Section 5 provides the analysis on the tempera-ture gradient data collected across the thickness of TWT pavement. The paper is concluded in Section 6.

2.0 Test Site and Investigation

The study stretch is Vithalbhai Patel Road located in Mulund, a suburb of Greater Mumbai. The existing pavement of this road is flexible with 120 mm of bituminous layers. The length of road is 850 m and the width of carriage way is 6 m catering to light to medium traffic.

This road stretch has storm water drains and sidewalks on both sides.

The condition survey of existing pavement was made when the potholes, bad patches, cracks and undulations were noticed. Trial pits were taken to determine the thickness of existing pavement layers. The subgrade obtained from these test pits was evaluated for its CBR. Fig.1 shows the existing crust of the pavement.

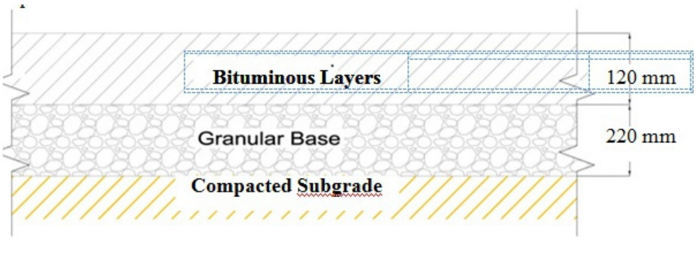


Fig. 1 Existing Crust of the Pavement

2.1 Subgrade Evaluation

The subgrade samples collected from the test pits were tested for their California Bearing Ratio (CBR) in the laboratory followingthe standard procedure. The soaked CBR value of the subgrade was obtained as 7.90%. The plastic limit and the liquid limit of the soil was found to be 21% and 27% respectively. Therefore, the plasticity index is 6%. The filed moisture content of the soil was found to be 14%

2.2 Structural Evaluation

The Pavement stretch was visually inspected before the actual deflection measurements using the Benkelman beam. The deflection measurements were taken at a distance of 1.5 m from the pavement edge at 50 m interval, in staggered manner. The deflection measurement was taken only in one direction (as the carriage width was only 6 m) of traffic under standard test conditions in accordance with IRC: 81 (1997). The pavement temperature in the interior was measured during the deflection measurements using digital thermometer.

The rebound deflection of the pavement surface under standard axle load was measured by Benkelman Beam method and structural strength of pavement was assessed. After correcting for temperature and moisture the characteristic rebound deflection was found to be 1.83 mm.

3.0 Structural Design of TWT and Concrete Mix Design

For the design of the UTWT and TWT overlays, procedures and models have already been evolved. The commonly used method of design for the UTWT is the one developed by the Portland Cement Association (PCA) and for the TWT by the Colorado Department of Transportation, both from the USA. As with any pavement design, the thickness of the new concrete pavement is determined by the type and volume of expected traffic, the strength and the condition of the existing pavement and the material properties of the concrete to be used. For the experimental stretch at Vithalbhai Patel road, Mulund in Mumbai, the design parameters for TWT required were found out based on the investigations carried out as mentioned earlier. Table 1 shows the design parameters for TWT.

The design of Thin White Topping was done as per the guide lines of IRC SP-76, 2008. The effective modulus of subgrade reaction (k) on the top of the milled surface was worked out based on two concepts: i) based on the k value of the subgrade and the thickness of the granular and asphalt layers of the existing pavement, and ii) based on the characteristic deflection obtained from Benkelman beam deflection study. It was found that the effective k value found based on deflection is significantly less than the one found based on the subgrade k value. Therefore this lower value obtained from deflection study has been used for the design of TWT.The design data finally used for the TWT for Vithalbhai Patel Road is shown in Table 1. Based on this design data following the procedure prescribed in IRC: SP: 76-2008 a thickness of 150 mm for the white topping was found to be appropriate.

**Table 1 Parameters considered for design of Thin White topping*

Parameter	Value	Unit
Design life	20.00	Years
Commercial traffic	1249	Commercial Vehicles per day (CVPD)
Grade of concrete	M 60	-
Flexural Strength of Concrete	5.5	MPa
Elastic Modules of Concrete(E)	3x105	kg/cm2
Coefficient of Thermal Expansion of concrete	10 × 10-6/	0C
Temperature Differential	14.60	0C
Rate of increase of traffic	7.5	%
Cumulative repetitions in 20 years	19741954	
Design edge Traffic on edge	4935489	CVPD
CBR (%)	7.90	-
Effective *k* value on top of Asphalt layer	5.30*	kg/cm2/cm
Spacing of joints in both directions	1.0	m

*based on characteristic Benkelman beam rebound deflection

3.2 Concrete Mixes for TWT

The concrete mix selected for a particular project is decided basedon the traffic and loading conditions and the requirement of early opening to the traffic. Concrete mix design for white topping includes cementious materials (cement, micro silica and fly ash), coarse aggregate, fine aggregate, air entraining agent, admixtures (water reducers and/or plasticizers), synthetic fibers (as specified),and a low water cement ratio. Fast-Track Paving white topping projects use high early strength concrete mixes.

Since the TWT overlays are intended for bituminous pavement rehabilitation and need to be constructed without causing much disruption to traffic, use of "fast-track" construction requiring high-early strength concrete is desirable. It is possible, today, to produce such mixes using a low water-binder ratio with the incorporation of a superplasticizer and a variety of mineral admixtures such as silica fume, fly ash, high reactivity metakaolin,etc. Such mixtures have greater potential for shrinkage and hence proper care is essential to avoid shrinkage cracking. Incidentally, with a view to contain the shrinkage cracking and enhance the toughness and residual strength of concrete, fibers have been used in many UTW and TWT mixtures.

For the present TWT, certain laboratory investigation was carried out for arriving at an appropriate mix design. The attempt was made basically to arrive at concrete mixes that can be used for fast track construction. The governing criteria was to achieve flexural strength of around 4 MPa at the age of at least 3 days. For this purpose it was found essential to use both mineral and chemical admixtures in the mixes. After a number of initial trials a total of 6 mixes viz., VTMS07, VTMF02, VTOF02, VTMS08P, VTMF03P, and VTOF03P were finally selected.

The details of these mixes are given in **Table 2**. It may be mentioned that further optimization of the mixes is possible and whatever is presented in **Table 2** included the results of initial trials.

It can be seen that the flexural strength of 4 MPa at the end of 3 days can be achieved with the use of silica fume and superplasticizer. As an alternative, ASTM type F fly ash was also tried and it was found that the flexural strength of 4 MPa can be achieved with 20 percent addition of fly ash and superplasticizer and polypropylene fibers (mix VTOF03P of Table 2). For this mix, the 28-day flexural strength was 6.2 MPa. As a part of quality control, the actual compressive and flexural strength of concrete received at the site during construction of the experimental stretch at Vithalbhai Patel Road in Mulund, Mumbai is shown in Table 3.

Table 2 Parameters of Laboratory mixes for Thin White Topping

Ingredient	VTMS07	VTMF02	VTOF02	VTMS08P	VTMF03P	VTOF03P
Cement, kg	450	380	420	450	380	420
Micro-silica, kg	35	30	0	35	30	0
Fly ash, kg	0	100	105	0	100	105
Total Binder, kg	**485**	**510**	**525**	**485**	**510**	**525**
Polypropylene fibres, kg	0	0	0	0.900	0.900	0.900
Aggregate, kg: 20 mm	633	607	601	644	639	622
Aggregate, kg: 10 mm	422	405	400	430	426	415
Natural sand, kg	497	497	491	486	463	470
Crushed Sand, kg	332	331	328	324	309	313
Water, lit	121	143	147	121	143	147
superplasticizer, lit	9.70	5.30	5.25	9.70	6.63	6.56
W/B Ratio	0.25	0.28	0.28	0.25	0.28	0.28
Slump, mm	65	80	80	55	60	75
	Comp Strength, MPa					
1 Day	35.11	19.73	19.87	36.31	24.8	26.8
3 Days	51.47	32.18	34.89	54.84	38.44	37.95
7 Days	69.02	45.51	47.78	68.71	56.31	48.71
28 Days	**94.30**	**69.07**	**69.56**	**90.42**	**72.26**	**70.44**
	Flexural Strength, MPa					
1 Day	3.6	-	-	4.00	-	-
3 Days	-	4.20	4.00	-	4.20	4.2
7 Days	6.80	4.40	4.40	7.00	5.60	4.6*
28 Days	**9.2**	**7.80**	**6.00**	**9.00**	**8.00**	**6.20**

***at the end of 6 days**

Table 3 Observed compressive strength and flexural strength of concrete samples of TWT for Vithalbhai Patel Road

S. No.	Date of Casting	Strength at the end of 7 Days	
		Flexural Strength	Compressive Strength
1	1/13/2010	7.02	-
2	1/15/2010	6.96	-
3	1/21/2010	6.99	66.73
4	1/29/2010	6.99	65.73
5	1/30/2010	7.26	66.58
6	2/2/2010	6.99	66.27
7	2/9/2010	6.99	66.93
8	2/10/2010	6.93	66.37
9	2/11/2010	6.96	67.91
10	2/13/2010	7.08	66.36
11	2/16/2010	6.84	65.42
12	2/18/2010	6.99	66.74
13	2/20/2010	7.08	66.92
14	2/25/2010	6.93	65.35
15	3/3/2010	7.02	66.22
16	3/9/2010	7.23	65.75
17	3/12/2010		66.22
18	3/17/2010	7.14	65.11
19	3/19/2010	-	66.73

4.0 Construction and Instrumentation

4.1 Pre-Overlay Repair

Before the overlay is laid, repair of the existing bituminous pavement is essential to provide necessary uniformity to the support system. The techniques to be adopted for the repair would depend upon the condition of the existing pavement which includes pothole patching, crack sealing, milling, shoving, leveling, etc. Out of these, the technique of milling is most commonly used and strongly recommended as it helps in establishing better bond between the bituminous layer and the concrete TWT. For TWT, milling is not mandatory unless the concrete-HMA bond is assumed in the design. If the rutting thickness is more than 50mm, milling becomes a necessity. Incidentally, excessive milling should be done carefully as it is considered essential to have at least a 75-mm thick bituminous layer below the concrete overlay. Fig. 2 shows the milled surface of the experimental stretch.

Fig. 2 Milled bituminous surface of the experimental stretch

4.2 Concreting Operations

Concrete of required characteristics as per design was produced in a commercial ready-mixed concrete plant, as shown in Fig. 3.Before undertaking this, it was highly essential to optimize the concrete mix after detailed initial trails as explained in the previous section. All other operations such as transportation, placement, vibrations, finishing and curing were done in conventional manner. If a large stretch of the overlay is available for construction, it may be advisable to use the automated concrete paver having necessary attachments

Fig. 3 Pouring of Ready Mix Concrete from the Transit Mixer

4.3 Saw Cutting of Joints

Both UTW and TWT are characterized by their shorter joint spacing, which is usually 12 to 18 times the thickness of the slab.Such joint spacing reduces the curling stress, and combined with the adequate concrete-bituminous layer bond, also reduces the flexural stresses in the concrete panels. One, however, needs to ensure timely joint cutting as sawing too early may lead to ravelling of the joints while late sawing may allow stress built up resulting in random cracking in slabs. For both the UTW and TWT works, it is reported that “early entry" saws are commonly used. These saws have been specially developed specifically for early-age sawing, minimizing the raveling of the concrete at the saw cuts. The large number of joints will obviously increase the cost, however, it has to be weighed against the cost of thicker slab. Incidentally, the total cost of the overlay would be the most significant decision-making criterion. Here, it would be advisable to adopt the life cycle cost analysis approach. For this work the joint cutting was done within 8 -12 hours of placing concrete to form 1 m × 1 m panels. Fig. 4 shows the photograph of the actual TWT pavement surface of the experimental stretch showing the saw cut joints forming 1 m × 1 m panels. In Fig. 4 the textured surface of TWT can also be seen which was obtained using a manual broom brush.

Fig.4 TWT Pavement with Saw Cut Joints and Textured Surface with Broom Finish

Fig. 5 shows the photograph of the completed road stretch with TWT pavement overlay. This photograph has been taken after 5 years of servicelife of the pavement. The performance of the pavement in the first 5 years has been found to be excellent without any distresses. The pavement is expected to serve without much maintenance interventions for 20 years.

Fig.5 Photograph of the Vithalbhai Patel Road with TWT Overlay

4.4 Instrumentation for Thin White Topping

Thermocouples and strain gauges were used to measure internal temperature and strains in concrete respectively. Vibrating wire type strain Gauges were used for strain measurement. When the gauge is embedded in concrete the strain resulting from the stresses developed in the concrete alters the natural frequency of oscillation of the wire. The strain in the concrete is proportional to the square of the frequency of oscillation of the wire, and the gauge thus provides a sensitive system for measuring internal strains.

The following instrumentation was deployed for this experimental stretch:

1. Vibrating Wire type strain Gauges-55 mm gauge length.
2. K-Type thermocouple with required length of compensating cable.
3. 10-way cable termination & switch box.
4. Data Logger-Data Taker make.
5. GSM Radio modem with antenna at transmitting end.
6. GSM Radio with antenna at receiving end.
7. PC for Data recording/displaying data.
8. Multi-core cables for connecting sensors to the Data loggers through cable termination boxes.

Fig. 6 shows the schematic of the instrumentation and the data acquisition employed for this experimental stretch of TWT.

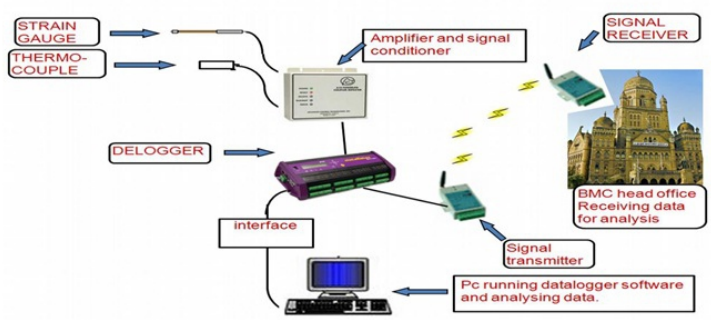
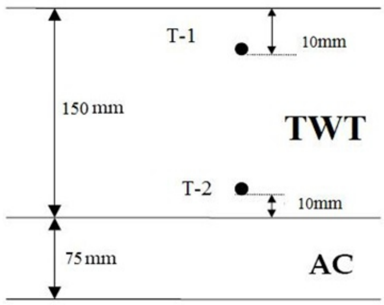


Fig. 6 Schematic Diagram of Data Processing, using Sensors

The Sensors (VW strain gauges & Thermocouples) were installed by manually keeping them in-place while pouring concrete, and by manually compacting the surrounding areas. Care was taken to route the cables carefully as per the cable route plan, preferably in a heavy duty PVC/MS conduit pipe to a centralized location, where the cable termination data logger boxes were placed. Fig. 7 shows the position of thermocouples at top and bottom in the TWT cross section.



AC: Asphaltic concrete; T-1: Thermocouple at top;
T-2: Thermocouple at bottom; TWT: Thin White Topping

Fig. 7 Position of Thermocouples (K Type)

5.0 Temperature Variation Studies for TWT

The stresses in TWT pavements are result of various factors such as wheel loads, cyclic changes in moisture and temperature, and volumetric changes in supporting layers. These loads and changes tend to deform the slab, causing stresses of widely varying intensity. The present design methodology of concrete pavements considers the combined effect of wheel load and temperature stresses along with their variation over time. Temperature differential between the top and bottom of cement concrete slab is an important parameter that determines the extent of curling stress induced into the slab. The temperature differential between top and bottom of the concrete pavements is a function of solar radiation received by the pavement surface at the location, losses due to wind velocity and thermal conductivity of concrete.Temperature stresses aredue to curling developed in the slab due to variation in temperature. The variation in the temperature across the depth of the slab is caused by the daily variation, whereas overall increase or decrease in slab temperature is caused by the seasonal variation in temperature.

During the day the top of the slab gets heated under the sunlight when the bottom remains relatively cold. The maximum difference in temperature between top and bottom of the pavement slab may occur at some period after midnoon. This causes the slab to curl or bend and this is resisted by the self-weight of the slab. This resistance induces the curling stress in the slab. The stress is also induced during evening due to reversal of the curling of slab.

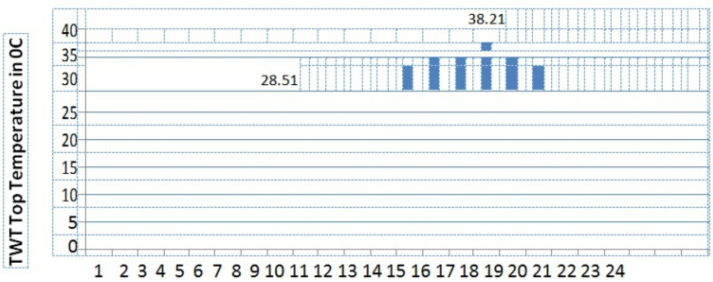
Thus the daily variation in temperature causes the curling stresses in the corner, edge and interior regions of the slab. The seasonal changes in temperature induce the frictional stresses in the concrete slabs which aretaken care along with shrinkage stresses by saw cutting of joints. (Belsheet al, 2011). In the present study, for the experimental stretch, investigation into the temperature gradient across the thickness of TWT was carried out.

5.1 Experimental studies for Temperature variation

For experimental studies, on entire stretch of concrete bay, total eight no of square slabs of dimensions 1m x1m of grade M60 with thicknesses of 150 mm were selected in north side and south side of the road. Thermocouples and Strain gauges were embedded in concrete during construction of thin white topping of slabs as shown in Fig.7. The slabs were cast and cured for 7 days. In each slab two thermocouples were installed, one at top and another at bottom for the purpose of accurate measurement of temperature. The Sensors (VW strain gauges & Thermocouples) were installed as explained in section 4.4. Initial data of the sensors was recorder immediately after its installation and immediately after the concrete has set which was used for future references. After the collection of this initial data the, the monitoring and collection of temperature and strain gauge data was done on daily/weekly basis. This data was recorded remotely through wireless network in a personal computer. It was possible to collect and monitor the data under different loading and environmental conditions. In this paper, however, only the analysis and results relating to the temperature variation has been presented.

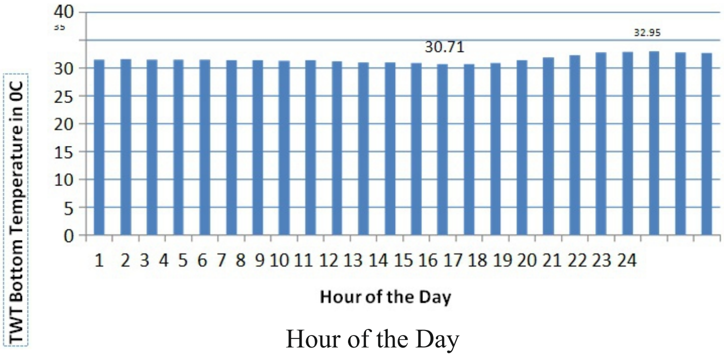
5.2 Temperature Data and Analysis

The variation of temperature on the top and bottom of TWT for the experimental stretch is shown in Fig.8 and Fig.9 respectively. The top temperature varied from a minimum of 28.5 0C to a maximum of 38.2 0C.Whereas, the bottom temperature varied from a minimum of 30.7 0C to a maximum of 33.0 0C. The corresponding maximum and minimum ambient temperatures were respectively 34.3 0C and 25.1 0C. It can be observed that the variation of bottom temperature over 24 hours of the day is very much less when compared to that of the top temperature. It is also to be noted that the bottom temperature has remained above the minimum ambient temperature and minimum top temperature throughout the day.



Hour of the Day

Fig. 8 Temperature Variation at the Top of TWT



Hour of the Day

Fig. 9 Temperature Variation at the Bottom of TWT

The instrumented 1 m × 1 m panels were monitored for understanding the relation between the maximum top temperature and the ambient temperature. Fig. 10 gives shows the relation between the maximum top temperature of TWT and the ambient temperatures. The same relation is also shown in Table 4. These data were collected during the month of June of Year 2010.

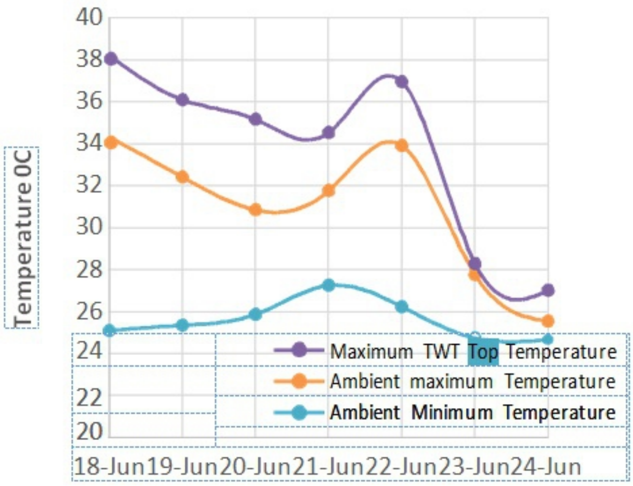


Fig. 10 Relation between Top Temperature of TWT and the Ambient Temperature

Based on the temperature data observed over a period of time the following relation between the maximum top temperature of TWT (*T2*) and the maximum ambient temperature (*T1*) emerged.

$$T_2 = 1.1\, T_1 \qquad (1)$$

Table 4 Relation between Top Temperature of TWT and the Ambient Temperature

Day of the Year	18-Jun	19-Jun	20-Jun	21-Jun	22-Jun	23-Jun	24-Jun
Ambient Maximum Temperature, 0C	34.33	32.42	30.86	31.78	33.92	27.77	25.55
Ambient Minimum Temperature, 0C	25.1	25.36	25.88	27.26	26.24	24.75	24.67
Maximum TWT Top Temperature, 0C	38.21	36.11	35.18	34.54	36.96	28.30	27.03

The R2 of this Eq. (1) is 0.99 indication excellent goodness-of-fit As per the general observations for the pavements in open areas, it was reported that the pavement temperature is generally found to be more than the ambient temperature by at least 20%. In this case, however, as this experimental stretch is in a built-up area with the presence of moderately high-rise buildings on both sides and presence of trees providing shade, the maximum temperatures of TWT were found to be only 10% more than the maximum ambient Temperature.

Moreover the presence of water table at shallow depth is also preventing heating of the pavement surface.

On a typical day in the month of June, the observed variation of the temperature difference between top and bottom of the 150 mm thick TWT panel is taken into consideration. The maximum temperature differential has reached a value of 7.3 0C resulting in a temperature gradient of 0.049 0C per mm thickness of slab. The maximum temperature differential that was observed during the monitoring period was 8.2 0C which corresponds to a temperature gradient of 0.055 0C per mm thickness of slab. Huang (2004) states that the maximum temperature gradient can be reasonably assumed within the range of 0.055 – 0.077 0C per mm thickness of slab. However, the temperature differential suggested for 150mm thick cement concrete pavement by IRC:58 (2011) for the coastal regions bound by hills, in which the test site falls, is 14.6 0C. This corresponds to a temperature gradient of 0.097 0C per mm thickness of slab. This high gradient may be appropriate for pavements located in open areas with ambient temperatures reaching over 40 0C. The present pavement located in built-up area with the presence of trees, has shown only half the temperature differential suggested by IRC:58 (2011).

If this actual observed temperature differential is used in design, there willbe reduction in the thickness of the TWT due to the reduction in curling stresses.

6.0 Summary and Conclusions

This paper shared the experiences and insights of a thin white-topping pavement overlay constructed on one of the road stretches located in Mumbai. The investigations carried out for obtaining the design parameters were detailed and the design data that was used for the design of the TWT overlay were presented. The laboratory attempts for the design of an appropriate concrete mix for the TWT that can achieve early high strength were explained. Also the construction and instrumentation of TWT was briefly explained. The analysis of temperature data collected through the embedded thermocouples was presented and discussed.

Following conclusions were drawn from this study.

1. Thin White-Topping pavement overlay can be advantageously carried out over existing bituminous pavements resulting in road pavements with prolonged service life with negligible maintenance interventions for even medium to moderately heavy traffic.

2. During the investigation for obtaining the design parameters for the design of TWT, it was found that the effective *k* value found based on deflection criteria was found to be much less than that found based on the *k* value of the subgrade and the thickness of pavement layers. Therefore while designing the thin and ultra-thin white topping overlays it is more appropriate to find the effective *k* value based on Benkelman beam deflection.

3. The concrete mix with 20% ASTM type F fly ash and appropriate quantity of polypropylene fibers and superplasticizer achieved a flexural strength of 4 MPa at the end of three days. Such mixes can be efficiently used for opening the TWT overlaid road within a week's time to traffic.

4. The maximum TWT pavement temperature has been found to be only 10% more than the maximum ambient temperature. This prompts for separate guidelines relating to temperature stresses for concrete roads in built-up areas with high rises and trees on both sides.

5. The temperature measurements revealed that the temperature gradient across the thin white topping is only one half the value suggested by IRC:58 (2011). Though based on this isolated study it cannot be generalized, in view of this observation it may be inferred that the temperature differentials that are used in the design are on the highly conservative side.

Acknowledgement

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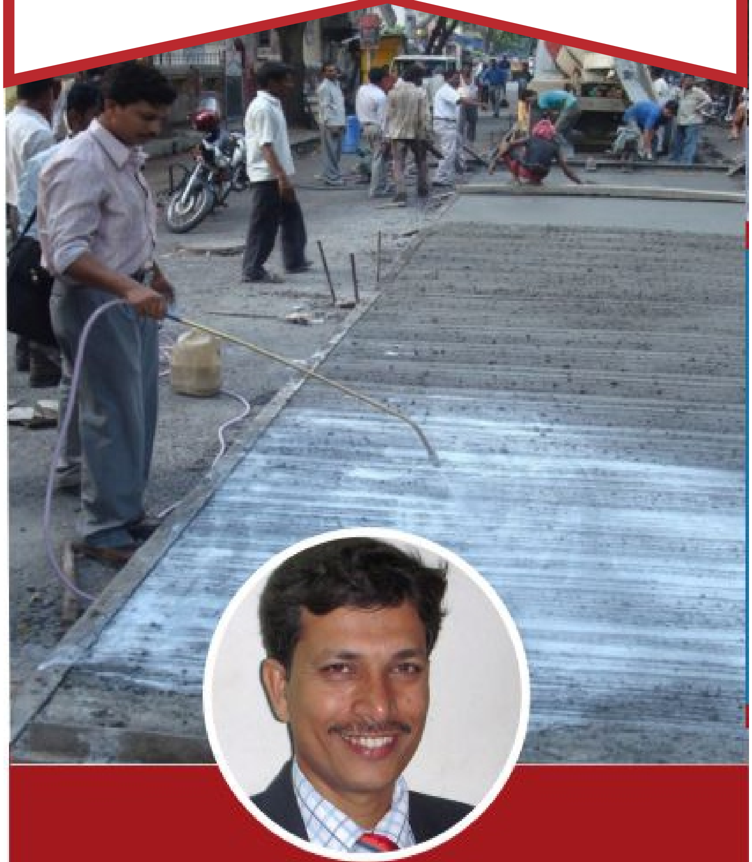

"Modern Technology for Repairs, Rehabilitation of Pavements"

by

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Municipal Corporation of Greater Mumbai

Retrofitting and Rehabilitation of PSC Girder Bridge with CFRP: Case Study

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Director
Dhirendra Group of Company (DGC)

Abstarct

This paper aims at the topic of strengthening of existing old deteriorated bridges with the help of case study of retrofitting of half century old Indian railway bridge. The structure has monitored before and after strengthening. The two response parameter has considered for the monitoring i.e. deflection and frequency, because deterioration and cracks affect the stiffness of structure and hence these responses. The result shows significant change in response.

Keywords: *Retrofitting, Frequency, Stiffness, PSC I girder, FRP.*

1.0 Introduction

Deterioration of ageing bridges has been well noted worldwide. Retrofit of deteriorated infrastructure has become a major challenge for governments in developed and under developing countries in the last decades. In response, there has been an escalating world-wide tendency to select Fiber Reinforced Polymer (FRP) composite retrofit systems as an alternative to traditional bridge rehabilitation schemes. Accordingly, several design codes were developed to standardize the bridge strengthening process using FRP systems. This paper covers the effectiveness of external FRP strengthening of bridge elements such as I-girders with help of case study.

2.0 Description of case study

Western Railway has proposed rehabilitation of PSC I girders on Bridge of Godhra-Ratlam section of Ratlam Division of Western Railway, India. The Bridge has been observed to require immediate strengthening. The bridges were constructed during 1958-60 and as such no detailed design & drawings are available. The bridges have composite PSC I girders supported on neoprene bearing. The main reason for retrofitting of the PSC I girders on the bridges was due to development of cracks on the girders. As the cracks are propagating with time strengthening of these girders are required immediately to arrest further deterioration.

3.0 Preliminary Structural Assessment

The overall evaluation included a thorough field inspection, measurement of different load response parameter such asdeflection and frequency forstatic and dynamic load and a structural capacity analysis. Existing construction and operati-onal documents for the bridges were reviewed, including the design drawings, project specifications, as-built information, and past repair documentation. Assuming that the PSC girder beams behave as simple reinforced girder beam with no prestress. As the existing reinforcement details are not known, an analysis is done on the girder to find the reinforcement required for the section. Based on this reinforcement and section details the capacity is found out.

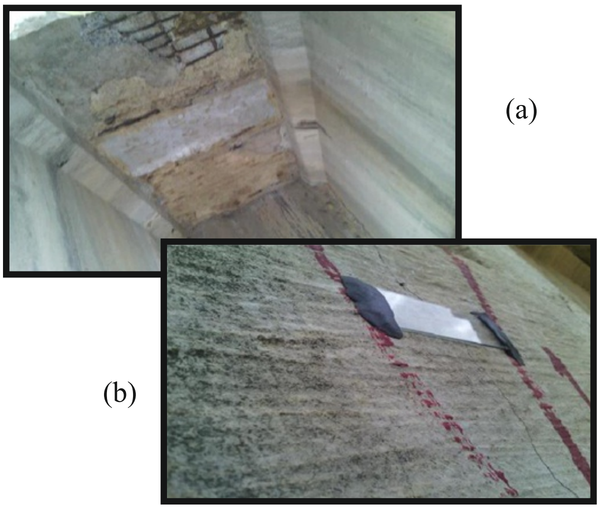
Table 1. Dimension Details of Bridge of PSC I Girder.

Sr. No.		Details
1	Type of superstructure	PSC-I girders
2	Span	1 x 18.29 m
3	Clear span	18.15 m
4	Effective span	19.17 m
5	Overall length of girder	19.67 m
6	Overall length of deck slab	20.20 m
7	Diaphragm details	6 (4 Intermediate and 2 End) size-1880x1450x275 mm
8	Name of river	Nakdi river
9	Year of casting and launching of Bridge	1958-60
10	Total weight of girder (per span)	145 T (Girders +Deck slab+Diaphragms)
11	Ballast cushion	300 mm
12	Details of track	60 Kg, Sleeper-PSC
13	Bank height in approaches	10.5 m
14	Dimensional details of girder cross section	Overall depth-2130 mm Width of Top and bottom bulb-620,Thickness of web-310 mm, Depth of web-1435 mm
15	Thickness and width of deck slab	Thickness-150 mm,width-4300 mm
16	Substructure type and material type	Stone masonry in cement mortar, gravity type substructure

1)Visual Inspection

The PSC I girders had been severe cracks. Discolouration due to deterioration with time had seen in the overall structure. Spalling/ delimitation of concrete had been observed in diaphragm bottom. Cracks at the girder bottom at bearing locations had been observed. Termite attack had observed in the girders. Surface deterioration had been observed throughout the structure. Severe spalling and corroded reinforcement exposure had been observed at the bottom slab. This may be attributed to the carbonation taken place in the structure. It is expected that such a distress will be reflected through a change of stiffness of the girder.

As the stiffness of the girder changes, the natural frequency will also change. Hence, it may be possible to monitor the health of a bridge girder in a non-destructive in-situ manner by measurements of the natural frequency at regular intervals. It has decided to monitored natural frequency and deflection of the bridge before and after strengthening.



(a)

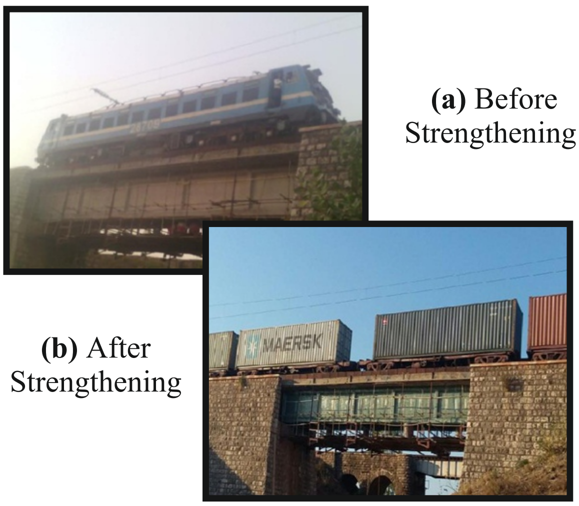
(b)

Fig 1. Deterioration of structural members of Ratlam Railway Bridge. (a) Cover delamination and corroded reinforcement exposure observed in deck slab bottom (b) Severe cracks seen in girder.

Fig 2. Structural health monitoring of Ratlam railway bridge with help of sensor

2) Pre-strengthening Monitoring for deflection and frequency

The bridge girder has monitored with the help of LVDT and accelerometer. The frequency has measured for dynamic forces and deflection has measured for static as well as dynamic forces.The un-cracked and undamaged girder has also monitored for reference.



(a) Before Strengthening

(b) After Strengthening

Fig 3. Actual load has applied to bridge for monitoring

3) FRP Strengthening of PSC I Girder of Railway Bridge

The strengthening design for the PSC I girders was proposed with the help of Prestressed carbon laminates and carbon fiber wrapping.It has assumed that there is no prestress in the PSC girder beams and that it is acting as RCC girder beam.

Material Properties

***Table 2.** Properties of concrete and Prestressing wire*

Grade of Concrete of RCC deck Slab	M20
Grade of Concrete of PSC Girders	M35
No. of cables in each girder	6
No. of Strands in a cable	8
Diameter of one strand	8mm
Tensile Strength of Pre-stressing Steel	1600 Mpa

***Table 3.** Properties of CFRP System*

Width of CFRP laminate	100 mm
Thickness of CFRP laminate	2.4 mm
Modulus of Elasticity of CFRP laminate,	E165 GPa
Tensile Strength of CFRP laminate	1800 MPa
Rupture Strain of CFRP laminate	0.012
Pre-stressing Force Applied on one Laminate	80 kN

4) Flexural Strengthening

In order to increase the flexural capacity and stiffness, reduce the distribution and width of the flexural cracks and improve the performance of the RC members under the service load conditions, the FRP material can be epoxy-bonded to the areas under tension while the fibers are oriented parallel to the principal stress direction. This type of strengthening can increase the ultimate flexural strength of the strengthened members from 10% to 160% (2). For the railway bridge 3 numbers of prestressed laminate has provided at bottom of I girder and 2 number of non-prestressed laminate has provided at side face of bottom flange of I girder (as shown in figure) to increase the stiffness and the flexural capacity of the girder. The numbers of prestressed and un-prestressed laminate has calculated through retrofitting design calculation.

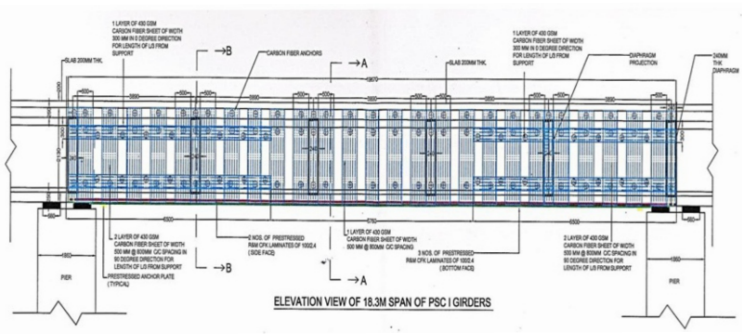


Fig 4. Elevation View of 18.3m Span of Ratlam Railway Bridge PSC I Girders.

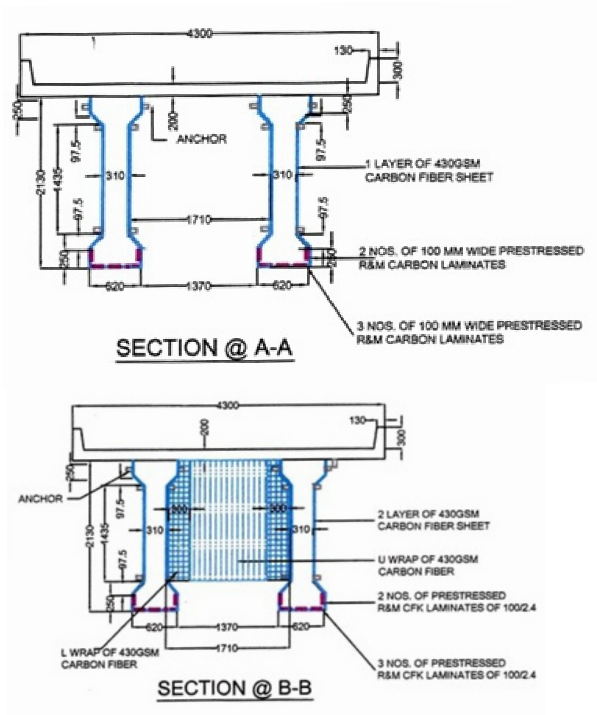


Fig 5. Sectional elevation of Ratlam railway bridge PSC I girders

5) Shear Strengthening

The shear strength of an RC beam is attributable to aggregate interlock, compressive zone concrete, dowel action, and transverse steel reinforcement, and can be increased significantly by bonding the FRP composites externally to the RC member, having the fibers crossing the shear cracks and parallel to principal tension stresses. Thereby, the beam will fail in flexure and the brittle shear failure can be avoided. To this end, the FRP composite is bonded to the beam covering either only the two sides of the beam (side bonding), or the two sides together with the tension face (U-jacketing). It is noteworthy that, covering the whole cross-section (closed wrapping) is possible only in bridge columns (and not the girder), because of girder's being integral with the slab. For the railway bridge girder FRP ply has applied as U wrap (as shown in figure) as it is most efficient.

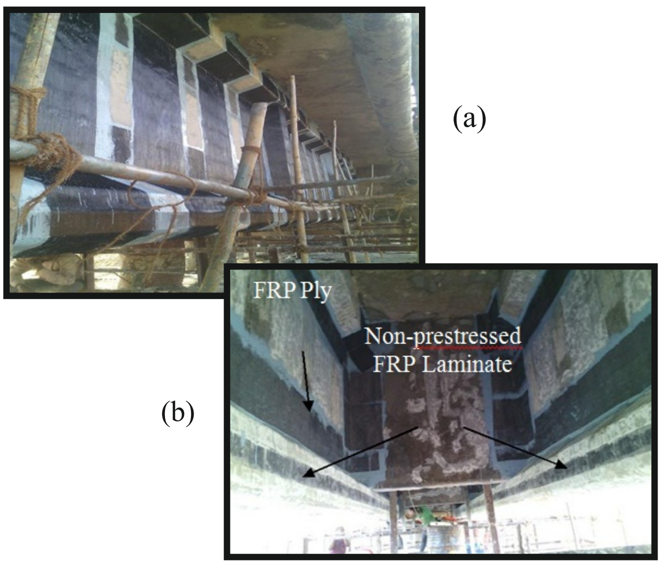


Fig 6. FRP Retrofitting of Ratlam railway bridge girder**.**
(a) U- Wrap FRP applied to PSC I girder. **(b)** FRP ply and non-prestressed laminate applied to inner side face of PSC I girder

6) Parametric investigation and experimental result

The frequency of any structural member is depend on the stiffness and the mass of member.

$$\omega = \sqrt{\frac{k}{m}}$$

$$2\pi n = \sqrt{\frac{k}{m}}$$

$$n = \frac{1}{2\pi}\sqrt{\frac{k}{m}}$$

$$n \propto k \;\ldots\ldots\ldots \qquad \textit{for constant mass}$$

Where,

ω = Circular frequency

k = Stiffness of the member

m = Mass of the member

n = natural frequency

The natural frequency will be directly proportional to stiffness of structural member if its mass keeps constant. In the retrofitting of railway bridge the stiffness of the girder increases with increase in negligible mass, which is one of the important feature of FRP retrofitting.

Similarly, the deflection in any structural member depends on the load acting on it and the stiffness of the member. The deflection in the girder of railway bridge can be reduce by increasing its stiffness.

$$\delta = \frac{P}{k}$$

$$\delta \propto \frac{1}{k} \;\ldots\ldots\ldots \qquad \textit{for constant P}$$

The deflection and the frequency of the bridge girder has measured before and after strengthening.

The change in the response of one of the bridge girder for its retrofitting is as follows.

Table 4. *Comparison of pre-strengthening results with post-strengthening results for natural frequency.*

Sr. No	Girder	Loading Condition	Natural Frequency in Un-Cracked Girder	Natural Frequency in Cracked Girder	Natural Frequency in Post Strengthened Girder at 20kmph	% Increase in the Natural Frequency as compared to the un-cracked girder at 20kmph
1	Span 2	Dynamic	9.574	8.6435	9.2885 Hz	97.02 %

Table 5. *Comparison of pre-strengthening results with post-strengthening results for Deflection*

Sr. No.	Girder	Loading Condition	Deflection in Un-Cracked Girder (mm)	Deflection in Cracked Girder (mm)	Deflection in Post Strengthened Girder (mm)	% Reduction to the excessive deflection compared to Un-cracked girder
1	Span 2	Static	2.728	3.9656	3.08	71.48 %
2	Span 2	Dynamic	2.644	3.9055	3.01	70.49%
Average % Reduction to the excessive deflection of cracked Span No.2 girder as compared to the sample un-cracked Span No.1 girder						70.99%

Observations

1. The deflection in the un-cracked girder is less than the deflection observed in the cracked girder indicating that stiffness of un-cracked girder is more than cracked girder.

2. Natural Frequency of un-cracked girder is more than the natural frequency of cracked girder indicating that uncracked girder has enhanced stiffness compared to cracked girder.

3. After strengthening there is an average 70.99 percentage reductions in excessive deflection in the cracked girder of span 2 as compared to the un-cracked girder of span No.1. Also, the reduction in excessive deflection is within 25% of the un-cracked girder.

4. The natural frequency of the cracked girder is within 25% of the natural frequency of un-cracked girder.

5. After strengthening the average percentage of the natural frequency of the cracked span girders is increased to 97.02 percentage of the un-cracked girder of span No.1.The natural frequency of cracked girder has been improved and brought closer to that of un-cracked girder.

6. With the strengthening measures the stiffness of the cracked girder has been improved as indicated by the reduction in deflection and improvement in the natural frequency.

Conclusion

1. Retrofitting of the bridge with prestressed and non prestressed laminate has increased the flexural strength of the girder.

2. CFRP retrofitting reduced the excessive deflection in cracked girder about 70% compared to the deflection in un-cracked girder.

3. The cracked girder got stiffened after retrofitting and hence its natural frequency has increased almost nearly to un-cracked girder i.e. about 97% of un-cracked girder.

4. The retrofitting with FRP not only improves the performance of bridge reduced by crack in girder but also cease the further propagation of active crack up to considerable extent.

5. The retrofitting proves effective and successful method for increasing the life span of deteriorated bridge.

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"Modern Technology for Repairs, Rehabilitation of Pavements and Bridges"

"Rehabilitation of Bridges"

by

Dr. Gopal Rai

Director, Dhirendra Gorup of Company

How Do We Construct Bridges For Long Service Life

Dr. S. C. Gupta
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SCG Consultancy Services & Pinnacle admix

WHAT IS SERVICE LIFE?

1. Service Life: As per AASHTO (2006) - The period of time that the structure is expected to be in operation without major repairs.

2. The new bridges are designed and being built with a service life target of upto 300 years in extreme exposure.

3. Designing for a very long service life has considerable economic and community benefits and is a means of maximizing the return on community investment in infrastructure.

4. Delaying replacement and minimizing maintenance costs and the disruption caused by maintenance activities is an aim of asset owners.

WHAT IS DESIGN LIFE?

1. Design Life: A service life performance must be formulated into specific design criteria.

2. The codes usually define a design life that implies a particular service life regarded by society as acceptable.

3. AS5100 (2004) adopts a design life of 100 years. BS 5400. 1 (1988) assumes a design life of 120 years and AASHTO (2006) is based on a design life of 75 years.

4. It is usual to simply apply these codes and standards that simply a particular assumption of service life performance. These are based on current materials and technologies.

5. Present codes inhibit achieving very long service lives through the use of new materials or combinations of materials and new technologies.

GREENHOUSE GASE

1. Greenhouse gas is carbon dioxide and, during the 20th century, its concentration in the environment has risen by 50 percent.

2. Carbon dioxide is a major by-product in the manufacturing of the two most important materials of construction: Portland cement and steel.

ISSUE BEFORE THE SOCIETY

1. How future infrastructural needs can be met without further increases in the production of cement and steel.

2. Conservation of these materials through enhancing the durability of structures is one of the ways by which the construction industry can become a part of the solution to the problem of sustainable development

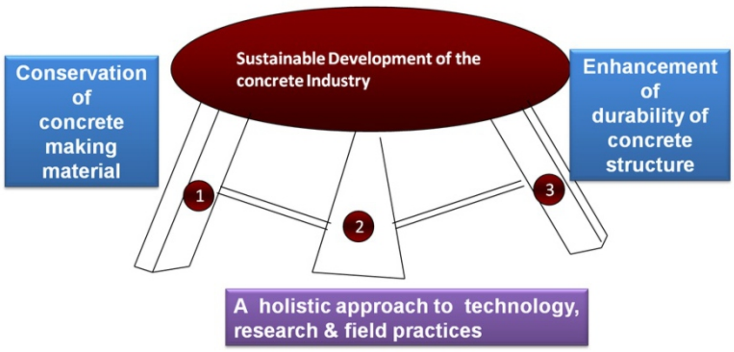
Role of Concrete Industry:

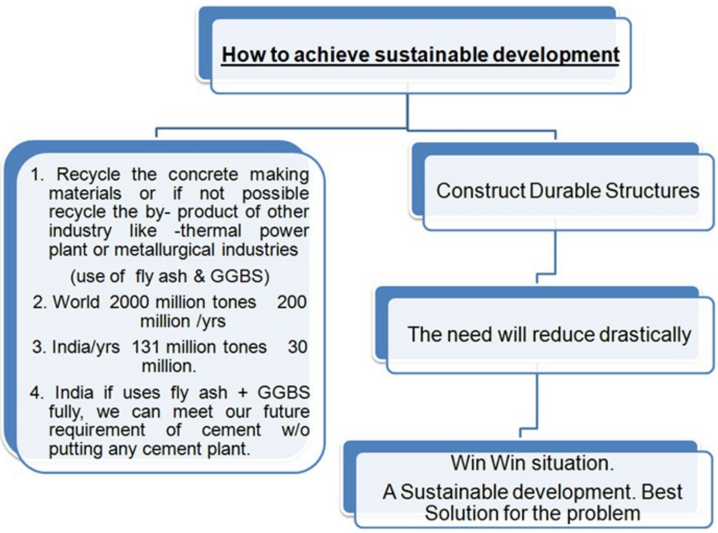
The most Important Player in:

1) Infrastructure Development &;

2) Major Consumer of limited natural resources is Concrete Industry. It has thus an obligation to incorporate environmentally sound technologies .

❖**Environmental Friendly Concrete Technology :**





OLD Vs. NEW Structures

1. Some 2000-year-old structures - made of slow-hardening, lime-pozzolan cement, are in excellent condition.

2. While the 20th century (after 1960s in India and 1930s in US & Europe) reinforced concrete structures that are constructed with portland cement are quickly deteriorating.

3. When exposed to corrosive environments like seawater, serious durability problems have occurred in bridges and other marine structures in less than 20 years.

Reasons for failures of bridges in the past

1. In the past, it was generally found that neither structural design nor materials were responsible for the lack of durability.

2. In most cases, it was the construction practices that turned out to be the culprit.

3. Inadequate consolidation or curing of concrete, insufficient cover for the reinforcement, and leaking joints are examples of poor construction practice.

Present Scenerio

1. A serious issue now is the growing evidence of premature deterioration in recent structures that were built in conformity with the state-of-the-art construction practice.

2. This means that the premature deterioration of concrete structures will continue to occur at unacceptably high rates unless we take a closer look at the current construction practice to understand and control the primary causes that adversely affect the durability of concrete.

Mechanism of deterioration

1. Deterioration, such as corrosion of reinforcing steel and sulphate attack, occurs when water and ions are able to penetrate into the interior of concrete.

2. This penetration happens when interconnections between isolated micro cracks, visible cracks, and pores develop.

Cracks in concrete - main culprit

1. Therefore, deterioration is closely associated with cracking.

2. The causes are many; however, there is one cause that has emerged as the most predominant factor in the cracking of concrete structures at early ages, namely, the use of high-early strength cements and high concrete mixtures to support the high speed of modern construction.

Durability depends on cracking

Concrete industry in the 20th century, while responding to calls for higher and higher strength from consumers, inadvertently violated a fundamental rule in materials science that there exists a close connection between cracking and durability.

Paradigm shift – for sustainability

To pursue the goal of building environmentally sustainable concrete structures, a paradigm shift in certain beliefs and construction practices is needed.

History in US & Europe

- Pre 1930
- 1930 – 1950
- 1950-1980
- 1980 – present

Pre 1930 (India pre 1960)

1. Deterioration of concrete had occurred either due to crumbling (possibly from exposure to freezing and thawing or sea water) or due to leaching from leaking joints or poorly consolidated concrete.

2. No cases of cracking-related deterioration were reported.

3. It is known that concrete with pre-1930 Portland cements developed strengths at a very slow rate because they were coarsely ground 180 cm^2/gm, Blaine specific surface and contained a relatively small amount (less than 30%) of Tricalcium silicate C_3S.

In 1930 in US

1. Users demanded higher early strength.

2. The ASTM specification was changed to permit more finely ground portland cement with higher C_3S.

3. The cement blaine fineness increased from 180 Cm^2/g to 300 cm^2/g.

GENERAL OBSERVATION

1. A survey of 3 to 14 years old bridges was done in US in 1944 and it was found that bridges built after 1930 are not proving as durable in service as bridges built before 1930.

2. 67% bridges built before 1930 were found to be in good condition as compared to only 27% of the post 1930 bridges.

3. Construction technology had remained essentially the same after 1930, it was only the:
   i.Fineness; &
   ii.C_3S of cement which had changed and was concluded the cause of deterioration of bridges.

4. Because the construction technology had remained essentially the same, engineering scientists concluded that the change in the cement fineness was the probable cause of the problem.

5. It was reported that, in 1930, as a result of users' demand for higher early strength, the ASTM specification was changed to permit more finely ground Portland cement.

Effect of fineness and high C3S

Deterioration due to cracking occurred when cement manufacturers started manufacturing faster-hydrating portland cements by raising the fineness and the C_3S content.

1950 – 1980

1. Since 1950, several important changes have taken place in the concrete construction practice.

2. Changes such as rapid development of the ready-mixed concrete industry, placement of concrete by pumping and consolidation by immersion vibrators triggered the need for high-consistency concrete mixtures which, before the advent of high-range water-reducing admixtures in 1970, were made by increasing the water content of fresh concrete.

3. Increasing the water content of fresh concrete.

4. Consequently, to achieve sufficiently high strength levels at early ages for the purpose of maintaining fast construction schedules.

5. Further increases were made in the fineness and the C_3S Content of the general-purpose portland cement.

6. By 1970 the C_3S content of the ASTM Type-I portland \ cements in the U.S. had risen up to 50 percent and the Blaine fineness to 300 cm^2/gm.

Impact of change of specification

1. The impact of this drastic change in the composition and hydration characteristics of general-purpose portland cement on durability of concrete can be judged from the fact that with the 1945 cements, a 0.47 water-cement ratio concrete gave 30 MPa strength at 28 days.

2. With the cements available in 1980, it was possible to achieve the same strength with a lower cement content and a much higher w-c of 0.70.

3. Being more permeable, this concrete naturally proved less durable in corrosive environments.

1987 survey in US

1. Another study was done in 1987 and it was concluded that concrete bridge subjected to wetting-drying, heating-cooling, freezing-thawing, mostly built after the 1940s, were suffering from an epidemic of durability problems.

2. It was estimated that 253,000 bridge, some of them less than 20 years old, were in varying states of deterioration and that the number was growing at the rate of about 35,000 bridges every year.

Study – Inference

There are reasons to believe that the acceleration of bridge durability problems since 1974 is directly attributable to cements and concrete mixtures possessing relatively higher strength at early ages.

Neville Says

1. Neville has also stated that the deterioration of concrete increased because cement specifications did not have limits on fineness, C_3S, and early strength. Today, ASTM Type I and II cements can be found with more than 60 percent C_3S and higher than 400 cm^2/gm fineness.

2. "It appears that the general property of moderate heat of hydration as a defining characteristic of cement has been lost over the years"

Cement in 1994 in US – (2000 in India)

1. Compressive strength of ASTM Type-I portland cement has doubled, from 17 to 30 MPa during the last 70 years.

2. In regard to ASTM Type II cements, In 1953, at least 50 percent of the cements had less than 20 MPa at 7 days, whereas in 1994, none had such a low strength.

3. Now a days ASTM specified minimum 28 days strength is achieved in 3 to 7 days .

Another study of 20000 bridges in 1996 in US & Canada

1. The report showed that more than 100,000 concrete bridges had developed transverse cracks soon after construction.

2. This was attributed mainly to thermal contraction. Usually, the cracks were full depth and spaced 1 to 3 m apart along the length of the bridge.

3. It was concluded that, under adverse environmental conditions, the crack growth reduced the permeability of concrete and accelerated the rate of corrosion of steel and deterioration of concrete.

4. "When high cement content, HRWR admixtures (super plasticiser) and silica fume are used, one-day moist-cured compressive strengths of 27.6 to 55 MPa have been achieved.

4. These concretes would have 1 day modulus of elasticity of 28.8 to 35.8 Mpa - values 3 to 7 times those of a nominal 20 MPa concrete used before 1974.

5. These very high strength concrete also have significantly reduced creep potential.

6. The brittleness relates to dramatically reduced creep potential and the observed early cracking or other unusual cracking that is not consistent with engineer's experience with more conventional concrete".

Something wrong with the codes.

1. There is a close relationship between cracking and deterioration of concrete structures exposed to severe exposure conditions.

2. Premature deterioration of concrete structures has occurred even when state-of-the-art construction practice was followed.

3. This shows that there is something wrong with the current durability requirements for concrete in our codes.

Lab results can be erroneous

1. When considering service life of actual structures, the results of laboratory tests on concrete durability should be used with caution because the cracking behaviour of concrete is highly dependant on the specimen size, curing history, and environmental conditions.

2. Laboratory specimens are small and usually not re-strained against volume change.

3. Laboratory tests of rich mixtures containing a fast-hydrating cement may yield low permeability values.

4. The same concrete mixture when used in an actual structure may not prove to be durable if exposed to frequent cycles of wetting-drying, heating-cooling, and freezing-thawing.

5. Under similar circumstances, inadequately cured concrete containing a high volume of fly ash or slag will also crack and deteriorate in the field, whereas well-cured specimens may have given excellent performance in a laboratory test on permeability.

Durability requirements in codes

1. ACI laid down in 1989 that when durability requirements are important, the selection of mixture proportions shall be governed primarily by the durability considerations.

2. Although the goal is well-intentioned, the recommended practice to pursue this goal has become counterproductive from the standpoint of building durable and environmentally sustainable concrete structures.

3. To illustrate this point, an analysis of how the ACI 318-99 durability requirements would affect the mix proportion of concrete of a RCC structure with 20 Mpa strength and exposed to sea water is prescribed.

Cement content

1. According to the code, a maximum 0.40 w/c and a minimum 35 MPa concrete mixture shall be specified.

2. Generally, the average concrete strength will be from 5 to 10 MPa higher than the specified strength, depending on whether or not field strength test data are available to establish a standard deviation.

3. For a 25 mm maximum-size aggregate and 100 mm slump, the ACI 211 tables for non air-entrained concrete recommend 195 kg/m^3 water content.

4. A normal water- reducing admixture, by reducing the water requirement 7 to 8 percent, will bring down the water content to 180 kg/m^3.

5. Thus, at the maximum permitted 0.40 one shall need 410 kg/m^3 cement which is too high to give crack free, durable structure.

Water content

1. ACI 318 controls the water content by specifying a maximum limit on w-cm.

2. As shown above, this approach is unsatisfactory when the cementitious material happens to be exclusively or mostly portland cement.

3. From standpoint of durability, it is apparent that a direct control on the maximum allowable water content in the concrete mixture is essential.

Mineral admixtures

1. Mineral admixtures, such as ground granulated blast-furnace slag and flyash, are highly effective in reducing the heat of hydration, strength, and elastic modulus of concrete at early age.

2. This is why properly cured concrete mixtures containing high volumes of slag or flyash (50 percent or more by mass of the cementitious material) are generally less crack-prone and therefore less permeable in service, which is an important factor in controlling the deterioration of concrete from causes such as reinforcement corrosion, alkali- aggregate expansion, and sulphate attack.

3. The construction codes should incorporate guidelines on the use of a high volume of mineral admixtures in concrete structures designed for durability.

Crack width and durability

1. There are no clear guidelines in the ACI Manual of Construction Practice on the relationship between crack width and durability of reinforced concrete structures exposed to different environmental conditions.

2. Although ACI 224R-98 suggests 0.15 and 0.18 mm as maximum tolerable crack widths at the tensile face of reinforced concrete structures exposed de-icing chemicals or seawater, respectively.

3. For a designer to exercise engineering judgement on the extent of needed crack control, at least some understanding of the effect of cracks and micro-cracks (less than 0.1 mm) on the permeability of concrete is necessary. A brief summary is presented herein.

4. Generally, at the interfacial transition zone between the cement mortar and coarse aggregate or reinforcing steel, a higher than average w/c exists, which results in higher porosity, lower strength, and more vulnerability to cracking under stress.

5. Thus, when a structure or a part of the structure is subject to extreme weathering and loading cycles, an extensive network of internal micro cracks may develop.

6. Under these conditions, the presence of even a few apparently disconnected surface cracks of narrow dimensions can pave the way for penetration of harmful ions and gases into the interior of concrete.

7. Thus, when a structure or a part of the structure is subject to extreme weathering and loading cycles, an extensive network of internal micro cracks may develop.

8. Under these conditions, the presence of even a few apparently disconnected surface cracks of narrow dimensions can pave the way for penetration of harmful ions and gases into the interior of concrete.

Paradigm shifts… cont…

1. We have reached a point in time when some sacrifice in the speed of construction seems to be necessary if it is important to pursue the goal of durable and sustainable concrete structures.

2. This, obviously, will require a change in the mindset of owners, builders, and designers. Some of the badly needed paradigm shifts in the current construction practice are given in subsequent slides.

Myth No. 1

1. The belief that society is being well served by high-speed construction is questionable due to dramatic changes during the 20th century.

2. We do not have a labour shortage, but we do face a serious problem of man-made climate change which brings into the limelight the construction materials like steel and concrete that are being produced at a great cost to the environment.

Therefore, conservation of materials, not the construction speed, should be the new emphasis of the concrete industry in the 21st century

Myth No. 2

1. The belief that the higher the strength of concrete, the more durable will be the structure, is not supported by field experience.

2. High-early strength concrete mixtures are more crack-prone and deteriorate faster in corrosive environments.

3. Codes should be amended to stress this point adequately.

Myth No. 3

1. By ignoring the cracking- durability relationship and by overemphasising the relation between strength and durability, ACI 318 is not helping the cause of constructing durable and environmentally sustainable concrete structures.

2. A paradigm shift to a holistic approach to control cracking in concrete structures is necessary to create a much closer working relationship between the structural designer, materials engineer and construction personnel than exists today.

Myth No. 4

1. The belief that the durability of concrete can be controlled by controlling the w/c is not correct because it is not the w/c but the water content that is more important for the control of cracking.

2. A reduction in the water content will bring about a corresponding reduction in the cement content at a given value of strength, which in turn, will reduce thermal contraction, autogenous shrinkage and drying shrinkage of concrete.

Method of Concrete Mix Design must change

1. Therefore, to achieve durability, the standard practice for selecting concrete mix proportion will have to undergo a fundamental change.

2. Note that a change in emphasis from the w/c strength relation to the water content durability relation will provide the needed incentive for a much closer control of the aggregate grading than is customary in the current construction practice.

How to Reduce water content

1. A substantial reduction in water requirement can be achieved by using a well-graded aggregate.

2. Additional reductions in the water content of concrete mixtures can be realized by the use of midrange or high-range water-reducers, high-volume fly ash or slag cements, and coarse-ground portland cements.

Prescriptive to Performance- based standard specifications for materials

To serve the goal of materials conservation, a paradigm shift is needed from prescriptive to performance based standard specifications for materials.

Present Necessity

1. There should be restrictions on the composition and fineness of the cements; the fineness and the C_3S content of modern Portland cement will have to be controlled. This can be achieved by making a coarse -ground, low C_3S Portland cement or by blending normal Portland cement with a high volume of fly ash or GGBS.

2. Such cement is expected to be less crack-prone.

Conclusions

1. In the 20th century, the concrete construction industry, driven primarily by the economics of higher and higher speeds of construction, increasingly used cements and concrete mixtures possessing high-early strength.

2. Consequently, the field experience with many modern concrete structures shows that they are crack-prone and those exposed to severe environments tend to deteriorate much faster than their anticipated service life.

3. To build environmentally sustainable concrete structures, it is clear that instead of strength, the 21st century concrete practice must be driven by considerations of durability.

4. The transition can be achieved by major paradigm shifts in the selection of materials, mixture proportions, and construction practice as outlined in this presentation.

My suggestion to have Cement like this

Sr. No.	Property	Limit
01	Initial Setting Time	Minimum 60 minutes
02	Final Setting Time	Between 300 to 400 min
03	Total Cl^- content in cement	< 0.05 %
04	SO_3 content in cement	< 3.0 %
05	C_3A Content	Between 5 – 8 %
06	C_3S Content	less than 35%
07	Fineness	<=280 m^2/kg
08	Heat of hydration at 7 days	Less than 270 KJ/kg

My suggestion…. Cont…

Sr. No.	Property	Limit
09	Heat of hydration at 28 days	Less than 320 KJ/kg
10	Concrete properties:	
a)	low heat special cement	about 40 to 50%
b)	Flyash	about 25 to 30%
c)	GGBS	about 20 to 25%
d)	Micro Silica Fumes	about 08 to 10%
11	W/ Cementitious materials ratio (maximum)	0.35
12	Binder Maximum	550 KG/M^3
13	Binder Minimum	450 KG/M^3

Sr. No.	Property	Limit
14	Permeability Maximum	15 to 25 MM
15	RCPT	Maximum 1500 Coulombs
16	Coefficient of Chloride diffusion at 56 days	1.2 x10-12 m^2/s
17	Crack Width (Maximum)	0.2 mm for RCC and 0.1 mm for PSC works
18	Temp. of Concrete	max^m 30 ^{0}C
19	Curing	45 Days
20	Use of Admixture	Compulsory

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"Why do have premature death of bridges in India? and how do we construct for long service life in future."

by
Er. Subhash Chandra Gupta
Retired Chief Engineer
Central Railway
SCG Consultancy Services
Mumbai

EVENTS 2018

EVENTS 2017

Lecture on "Sustainable Bridges In Urban Environment"on 3rd March 2017
By Prof. Mahesh Tondon, Director , Tondon Consultant Pvt. Ltd. Delhi

Lecture on "The Yamuna Bridge In Delhi & Other Cable Stayed Bridges "By SBP on 10th JULLY 2017 By Dr. Mike Schlaich (SBP Germany)

Lecture on “Uses of LARSA 4D and LUSAS 4D models for fast track implementation of Cable Stayed bridge-experiences and overview” on 18th August, 2017 by Er. Rajesh Prasad, ED Metro RVNL, Kolkata

STUDENTS CHAPTER

**Lecture on “Fascinating Metro Projects in India”And “Art & Science of Bridge Engineering” by Er. Vinay Gupta , Director & CEO Tondon Consultants Pvt. ltd.
13th September, 2017 & 14th October, 2017**

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