

# Studies on Various Theories and Models for Assessing the Remaining Life of Damaged Railway Bridges-Review (Fatigue and Fracture Mechanics Approach)

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**Abstract:-** This paper elaborates the present state of art about various theories and models for evaluating the remaining life of homogeneous and heterogeneous material used for various structures and bridges. Analytical methods for arriving the remaining area of steel in damaged rcc beams, natural and excited frequency for beams are detailed herewith. A brief method to calculate the fatigue load for the fatigue stress in the damaged bar, is described. The concept of actual bridge analogues to the specimen beam tested and the variation of remaining life in years with respect to stress range and Gross Million Tons(GMT) are explained including the experimental setup and further research aspects.

**Keywords:** Life prediction theories, models, remaining area of damaged steel, natural frequency, fatigue load and stress, Analogues Bridge, Specimen beam, Gross Million Ton, Experimental set up.

## 1. INTRODUCTION

This paper presents a proposal and objectives for evaluating the remaining service life of damaged RC beams of bridges with respect to size effect, the ratio of effective cover-to-effective depth (**depth ratio**), crack depth-to-side or diameter of reinforcement (**rebar size ratio**), for the cyclic loadings.

A review of the existing stipulations of fatigue design for railway RC beams of bridges as per IRS Concrete Bridge Code (1997) [1], Bridge Manual (1998), Bridge Rule (2008) [2], BS-5400 (1999) and Euro codes (2003) incorporating fracture mechanics parameters, has been made to arrive a beam with long life and minimum weight.

## 2. STATE-OF-ART

Computerized structural design and analysis of continuous prestressed concrete box girder bridges provide output as cross-section properties, section force, for working and

ultimate load conditions, based on loads, form works, methods of construction, profile of cable and area of section for arriving durable designs.[3]

In the reliability analysis of girder bridges, the live loads and compressive strength of concrete increases with time. This reliability equation is more conservative and depends on above distribution and time relationship [4]

The dynamic response of highway prestressed concrete bridges are affected by length of span, damping surface roughness, vehicle speed and impact. Power density function is used for road surface roughness. Above factors are studied for PSC girder bridges and compared with AASHTO specified values [5].

Fully prestressed concrete girder bridges are 1.5 to 2 times more reliable than partially prestressed bridges, while load is increasing rate. Capacity is determined using non-linear finite element program[6].

The effect of cyclic loading on bridge frames subjected to seismic loading has been simulated in experiments and relationship between horizontal force and deflection, ductility, energy dissipation with increased effective prestress are studied[7].

This paper describes about cumulative fatigue damage theories for metals and alloys. The principles considered in the various life prediction models are fully as per Linear damage rules, Non-linear damage curve, Two stage linearization approach, Life curve modification methods, Approaches based on crack growth concepts, Continuous damage mechanism models (Energy based theories). And also, this paper describes about the life prediction theories for homogeneous materials only. The service life evaluations of heterogeneous materials like reinforced cement concrete are not explained herewith. [8].

The compressive strength, modulus of elasticity, tensile strength, modulus of rupture, shrinkage and creep of concrete, absorption potential and freeze-thaw durability are studied in prestressed concrete girders, which are made up of high strength concrete using Portland cement, micro silica, fly ash with different types of coarse aggregates exposed to different temperatures [9].

Resistance models for DL, LL, impact loads based on statistical data, material tests and simulations are developed for developing codes and evaluation of structures [10].

In addition to the current design provisions, adopting the good fatigue design and construction practice, based on the research studies on fatigue response of slab like concrete beams, there is a need to improve the prediction of service life of RC girder bridges [11].

Euro code is more conservative than Spanish Norma IAP and AASHTO LRFD, and LRFD is most permissible code for reliability level of prestressed concrete girder bridges [12]. Inspection and condition rating of any bridge by the professional engineer for alerting the damages, fractures, and material failures can improve the maintenance [13].

The corrosion effect on bridge girders can lead to reduction of cross-section, while fatigue induces nucleation and propagation of cracks in the steel reinforcement. The coupled action of corrosion fatigue on RC structures reduces the expected life time. For RC T-girders subjected to cyclic loading, path dependent in plane-constitutive laws of cracked reinforced concrete structures are utilized for material modeling for analysis. An RC zoning method is adopted for two dimensional analysis. [14] .

Use of stainless bar in concrete bridges add 10% of total project cost compares to black bar but reduces cumulative costs by 50% over 120th year bridge life in the case of coastal bridges, chloride profiling is the tool to analyze corrosion. Thermal sprayed Zinc anodes in impressed current and galvanic cathodes protection, thermal sprayed titanium, Al<sub>12</sub>Zn-0.21n and Zinc-hydrogen anodes, are used in corrosion protection of coastal bridges to improve life [15].

In the case of composite girder bridge, using flat shell strip and penalty function method, the slab, girder and connections are modeled by considering the thickness of slab, modular ratio, stress range and intended service life.

The fatigue behavior is analyzed using LEFM-approach [16].

This paper presents various aspects pertaining to alignment, shape and size in straight/skew/curved, material specifications for bridges along with salient database, field studies and AASHTO methods for analysis, for predicting the remaining life of beams subjected to diagonal tension cracks. The S-N curve for shear stirrups is detailed for the bridge elements. The fatigue effect in RC beams in Flexural Region and on how to ascertain remaining life of RC beams of bridges with Fracture Mechanics Approach are not described [17].

A prestressed girder was strengthened using prestressed CFRP plates and subjected to testing. Another reference psc girder was strengthened with non-prestressed CFRP plates. The load carrying behaviors of both girders are studied. It is observed that the psc girder with prestressed CFRP plates had shown decreased deflections and less strain, with increased load carrying capacity around 45% higher than that for psc girder with non-prestressed plates [18].

Deterministic and reliability analysis of prestressed concrete bridge girders based on various codes of practice are compared. The strands required as per Hong Kong codes (2004) are higher than Chinese (2004) and AASHTO LRFD codes (2005). Flexural capacity based on the service limit state requirements is greater than strength limit state requirements in all three codes [19].

Dynamic behavior of long span box girder bridges subjected to moving loads are validated experimentally and numerically (using four node Langrangian and Hermits finite elements) for various position of moving loads and results are in good agreements [20] .

Using sampling method, partial rank correlation coefficient, standardized rank regression coefficient are examined to quantity the sensitivity of the outputs of each input variables and to obtain a method of uncertainty analysis and sensitivity analysis of effects of the creep and shrinkage in prestressed box girder bridges to reduce the uncertainties of prediction of time dependant effects due to creep and shrinkage and to improve long-term serviceability [21].

For crack width prediction in precast reinforced concrete slabs, under repeated loadings, fracture mechanics approach has been used to analyze the crack depth at initial and the fatigue crack growth rate, at 10Hz load

frequency with respect to steel stress ratio, and reinforcement ratio [22].

The connection between number of cycles vs. crack width, and crack width vs. life time, are developed for arriving crack width and spacing of reinforcement. For a simply supported beam which is subjected to cyclic loading. A slab with V-notch type crack at midspan are adopted in three point experimental support. The Paris-Erdogan equation is used to predict the number of cycles for the crack width observed. The analytical expressions for arriving the life time, using the crack width and number of cycles are formulated. But this paper considers the road loading and not for railway loading. The size of crack in rebar and size effect are not considered in this paper [23]

Providing pot bearing for prestressed concrete girders proved that the fatigue effect at the bearings is minimum under traffic and impact loads, and temperature effect [24].

Application of carbon glass fiber reinforced polymers (CFRP/GFRP) in epoxy resin matrix increases the strength of T-girder subjected to cyclic loading of more than  $10^6$  cycles [25].

Pit-depth data for prestressing wires depends on the spatial timer dependant reliability analysis of corroding pretension prestressed concrete bridge girders. This is achieved using non-linear finite element analysis and then probabilistic models of corrosion initiation. It is possible to evaluate the flexural strength and service ability considering material and geometrical properties of the prestressed box girder bridges [26].

A simplified model to predict the strain at failure in FRP and ARAMID strengthened prestressed segmental box-girder and monolithic RC continuous girders, are validated by experiments and analytical models [27].

Remaining life of reinforced concrete railway bridges and fatigue safety assessment of railway bridges are studied by reviewing the various models, methods of structural analysis, material properties, FEM modeling and experimental observations. This study is based on non linear analysis. However, the size effect, crack in reinforcement, depth of rib in rebar and square rebar sections are not considered [28].

The Euro norms describes about  $\lambda$ -coefficient and damage method for predicting the life of railway bridges. But, the present IRS concrete bridge code does not contain

sufficient stipulations for the service life RCC bridges. The Euro code explains about the fatigue effect on service life of bridges. Hence, the fatigue effect is to be considered in the design of rcc railway bridges. This will impart RCC bridges with optimum weight bridge elements [29].

One problem which is treated by Euro code is the prediction of the fatigue life of reinforced concrete structures. Fatigue loading failure is depicted by a fracture in a particular part of a structure which is subjected to variable cyclic loading. This loading is due to traffic, wind, and waves etc. The fatigue life of a reinforced concrete bridges mainly depends the stress range and the number of cycles of loading. The purpose of this study is to arrive an empirical equation for predicting fatigue life of rcc bridge using the norms of Euro code. In Euro code there are two alternative methods i.e, the Cumulative Damage Method, and the  $\lambda$ -Coefficient Method. Both methods consider the loading during the lifetime of a structure but in different manners. The Cumulative Damage Method calculates a fatigue damage factor which expresses the actual damage occurred in the structure in relation to the design fatigue life. The  $\lambda$ -Coefficient Method simply checks if the structure fulfils the demands for a given service life [30].

The size of member, crack depth in concrete, number of cycles and fracture toughness are the various parameters considered in the assessment of remaining life of bridges subjected to dynamic loading. It is observed that the stress for failure of beams under cyclic loading is less than that of monotonic loadings. The stress intensity factor varies with respect to  $d_0$ -size effect factor and fracture toughness. The Paris-Erdogan equation with size effect factor are applicable for assessing the remaining life of bridges. However, the effect of cover, rebar diameter and depth of ribs are yet to be considered in the equation of remaining life model [31].

### 3. OBJECTIVES AND SCOPE

This study exhibits the use of linear damage theory for metals and alloys in the parts of Road and Railway Bridges. But the provisions of codes do not explain about fatigue life of RC bridges subjected to cyclic loading as per the damages in the beams.

However, Remaining life equation for RC beams of girder bridges with reinforcement tied with binding wire are yet to be formulated. These bridges are designed for Modified

**Broad Gauge 1987 loading as in Indian Railway Standards Bridge Rule and Concrete Bridge Code.**

It is possible to bring down to the required level of remaining service life by developing an appropriate equation, for higher ratio of dead load to live load and dynamic impact with minimum weight viable to construct easily and to transport in case of precast units with less erection stresses (*IRS Bridge Manual 1998*). The aim of the present review is the prediction of remaining service life of railway RC girder bridges damaged.

**4. METHODOLOGY FOR ARRIVING SERVICE LIFE OF DAMAGED RCC BEAMS**

a).RCC beams with different damage in tension steel for various grade of concrete and steel are cast. The diameter of main bar is 16 mm and The maximum size of coarse aggregate is 20 mm.

b).These beams are subjected to fatigue loading with frequency of 3 Hertz. The calculation of fatigue load range and frequency are detailed in the following pages and figures.

c).The beams are subjected to static loading using the following loading arrangement. At first ,static readings are observed up to first crack, using DMEC gauges and LVDT at mid and quarter span, side face ,end face and top side at midspan.The deflections at mid and quarter span,

d).With this first crack load and minimum load as per fatigue load range calculation. Fatigue load is applied at a frequency of 3 hertz. The deflection at mid span and quarter span, displacements in other points as described above, are observed. When the crack propagates above 0.4xd, of beam, fatigue load range is varied. All the readings for number of cycles and crack depth in concrete are observed. When the crack propagates above 0.8xd, of beam, fatigue load range as arrived already, are applied. All the readings for number of cycles and crack depth in concrete are observed.

e).Based on the experimental results, It is possible to determine the unknown coefficients of the modified equation for fatigue cycles (Nij) in terms of various parameters cited in table for comparison of models. The final equation for this beam shall be extended to the railway slab bridges. [30,31,32]

**5. ACTUAL SLAB BRIDGE ANALOGUES TO SPECIMEN BEAM TESTED IN THE LAB**

The actual railway bridge slab is subjected to dynamic loading. The slab has to withstand the load due to impact which is caused by the moving loads running at different velocity, in addition to live load and self weight. Hence total load on the actual bridge is depending on EUDL, Effective span(L),earth (d<sub>earth</sub>) and ballast cushion (d),running velocity (V<sub>r</sub>),permissible velocity (V),width of dispersion(B),density of ballast , density of earth and density of concrete. Hence total load (W<sub>AB</sub>) on a bridge is equal to

$$W_{AB} = \left[ \left( L \times B \times D \times 2.5 + B \times (d_{earth} \times \text{density}_{earth}) + B \times (d_{ballast} \times \text{density}_{ballast}) + \left( EUDL \times (1 + CDA) \times \left( \frac{V_r}{160} \right) \right) \right) \right] \text{ - Equation(1)}$$

The actual BM at mid span per m width of slab

$$M_1 = \left[ \frac{w \times L^2}{8 \times B} \right] = \left[ \frac{W_{AB} \times L}{8 \times B} \right] \text{ - Equation(2)}$$

The mid span BM for the width (b) of specimen beam tested = M<sub>1</sub> × b.

Actual moment of resistance of the damaged beam = M<sub>CRD</sub>.

$$\left[ \left( L \times B \times D \times 2.5 + B \times (d_{earth} \times \text{density}_{earth}) + B \times (d_{ballast} \times \text{density}_{ballast}) + \left( EUDL \times (1 + CDA) \times \left( \frac{V_r}{160} \right) \right) \right) \times \frac{L}{8} = (M_{CRD} / 0.25) \right] \text{ - Equation(3)}$$

For the present specimen beam considered for testing,

$$\left[ \left( 2.4 \times 4.8 \times 0.4 \times 2.5 + 4.8 \times (d_{earth} \times 1.8) + 4.8 \times (0.3 \times 2.2) + \left( EUDL \times (1 + CDA) \times \left( \frac{V_r}{160} \right) \right) \right) \times \frac{L}{(4.8 \times 8)} = (M_{CRD} / 0.25) \right] \text{ - Equation(4)}$$

$$CDA = \left[ \left( 2 - (d_{earth} + 0.3) \right) / 0.9 \right] \times 0.5 \times (0.15 + 8 / (6 + L)) \text{ for } (d_{earth} + 0.3) \leq 0.9$$

$$CDA = [(3.9 - (d_{earth} + 0.3))/3] \times (0.15 + 8/(6 + L)) \text{ for } (d_{earth} + 0.3) \geq 0.9 - \text{Equation(5)}$$

On satisfying the above equation (5), it is possible to arrive the details of actual slab bridge analogues to the specimen

beam tested in the lab, as given below. And also, the fatigue stress in the tension bar of specimen beam and analogues slab bridge are similar. The specimen beam and analogues slab bridge must be of "under reinforced sections".

Table-1: Analogues Railway Slab Bridge Details.

Specimen beam tested			Representative slab bridge analogues to the specimen beam ,for the IRS loading standard:MBG-1987.(RAILWAY BRIDGES)						
Beam No	% of damage in tension steel	Breadth of beam (b) in m	Span in m	Breadth in m	Depth in m	Ballast cushion (d <sub>ballast</sub> ) in m	Earth cushion (d <sub>earth</sub> ) in m	EU DL in t	Running velocity of train in KMPH.
B1	27.813	0.25	2.4	4.8	0.4	0.3	0.115	43	100.00
B2	38.429	0.25	2.4	4.8	0.4	0.3	0.500	43	75.05
B3	49.045	0.25	2.4	4.8	0.4	0.3	0.500	43	56.25
B4	31.352	0.25	2.4	4.8	0.4	0.3	0.830	43	80.00
B5	27.813	0.25	2.4	4.8	0.4	0.3	0.900	43	86.87
B6	38.429	0.25	2.4	4.8	0.4	0.3	0.543	43	76.69
B7	49.045	0.25	2.4	4.8	0.4	0.3	0.500	43	58.50
B8	31.352	0.25	2.4	4.8	0.4	0.3	1.000	43	81.12
B9	27.813	0.25	2.4	4.8	0.4	0.3	0.900	43	90.47
B10	38.429	0.25	2.4	4.8	0.4	0.3	0.502	43	80.39
B11	49.045	0.25	2.4	4.8	0.4	0.3	0.500	43	60.70
B12	31.352	0.25	2.4	4.8	0.4	0.3	1.000	43	84.36
B13	17.834	0.25	2.4	4.8	0.4	0.3	1.200	43	109.62
B14	21.019	0.25	2.4	4.8	0.4	0.3	1.000	43	102.58
B15	41.967	0.25	2.4	4.8	0.4	0.3	0.500	43	73.80
B16	00.000	0.25	2.4	4.8	0.4	0.3	0.918	43	140.00
B17	00.000	0.25	2.4	4.8	0.4	0.3	0.619	43	137.00
B18	00.000	0.25	2.4	4.8	0.4	0.3	0.727	43	139.00

**6.REVIEW OF VARIOUS LIFE PREDICTION THEORIES AND MODELS BASED ON THE FOLLOWING LOGICS IN SINGLE OR COMBINATIONS FOR HOMOGENEOUS MATERIALS**

[8]. i.e,“CON,conceptual;PHE,Phenomenological;EXP,experimental;EMP,empirical;ANA,Analytical;sANA,semi analytical;LDE,linear damage evolution;LLD,load level dependant;LIA,load interaction accountable; SC, small amplitude cycle damage accountable;Appl,applications;S,simple;G,general;C,complicated;the suffix ‘n’ stands for ‘not’ or ‘non’.

Models	Basic principle & Parameters considered	Equation Developed for the Model
Corton and Dolon (1956)	Number of damage nuclei is taken into account for life prediction. Other approaches are nLDE, LLD, LIA, SC, few APP.G.	$D = \sum m_i X p_i X n_{ih}$
Grovers two stage damage theory(1960)	Crack initiation & propagation, two stages linear evolution is considered. Other approaches are Two stages LDE, LLD, NIA, Nsc, fewApp.S.	$\sum_{\alpha_i N_i}^{n_i} = 1 \text{ for initiation stage}$ $\sum \frac{m_i}{(1 - \alpha_i) X N_i} = 1 \text{ for propagation stage}$
Mansion et al 1966 Double Linear Damage Rule	Crack initiation & propagation, two stages linear evolution are considered. Other approaches are Two stage LDE,LLD,Nlia,SC,someApp.S	$\sum_{N_i}^{n_i} \frac{n_i}{(N_r - P X N_r^{0.6})} = 1 \text{ for Phase -I}$ $\sum_{N_i}^{n_i} \frac{n_i}{(N_r - P X N_r^{0.6})} = 1 \text{ for Phase -II}$
Shanley,(1952)	Crack growth and crack length as damage measure. Other approaches are nLDE, LLD, n LIA, n SC, few App .G..	$D = \left(\frac{a_0}{a_c}\right)^l X r_i$
Valluri,(1961)	Crack growth and crack length dislocation fracture mechanics as as damage measure. Other approaches are , n LDE, LLD, some App .G..	$\frac{da}{dN} = C X f(\sigma) X a$
Scharton et al(1966)	Crack growth and fracture mechanics of damage measure. Other approaches are n LDE, LLD, some App .G..	$\frac{da}{dN} = a^{(m+1)} X f(\sigma_{ij})$

Mansion et al (1981).Damage Curve Approach	Effective micro Crack growth. Other approaches are n LDE, LLD, n LIA, n SC, some App .G.	$D = \sum r_q \text{ with } q = \left(\frac{N_i}{N_r}\right)^{0.4}$
Miller et al(1977) Double Exponential law I version	Two Stage Crack growths. Other approaches are n LDE, LLD, n LIA, n SC,a few App .C.	$N_{l,1} = N_{f,1} X \left(\frac{r_1 + r_2 - l}{r_2}\right)$
Ibrahim et al (1981).Double Exponential law II Version	Two Stage Crack growths. Other approaches are n LDE, LLD, n LIA, n SC, some App .C.	$r_2 = (1 - r_1) \left(\frac{1}{1 - r_{l,1}}\right) \ln \left(\frac{a_{l,1}}{a_f}\right) / \ln \left(\frac{a_0}{a_f}\right)$ $D = \frac{a}{a_f} = \left(\frac{a_i}{a_f}\right)^{\frac{(1-r)}{(1-r_l)}}$
Miller (1982).Short Crack Theory,	MSC, PSC, E-P, fracture mechanics. Clear physical basis ,it is difficult to determine the micro parameters involved	$\frac{da}{dN} = A(\Delta\gamma)^\alpha (d - a) \text{ for MSC}_S: a_0 \leq a \leq a_1$ $\frac{da}{dN} = B(\Delta\gamma)^\beta a - C \text{ for PSC}_S: a_1 \leq a \leq a_f$
Ma-Laird (1989)	Crack population. Other approaches are LIA, few App .not universal G.	$D = \sum \left(\frac{P_i}{P_{critical}}\right) = K \sum n_i \left[\left(\frac{\Delta\gamma_p}{2}\right)_i \alpha_i - \left(\frac{\Delta\gamma_p}{2}\right)_{limit}\right]$
Vasak et al (1991)	Micro crack kinetics, equivalent crack length. Other approaches are Two stage nLDE, LLD, n LIA, n SC,few App .C.	$D = 2D_C\gamma \text{ for initiation : } D \leq r \leq \frac{1}{2}$ $D = D_C + \frac{D_C}{m} [e^{m(2r-1)} - 1] \text{ for propagation : } \frac{1}{2} \leq r \leq 1$
Radhakrishnan (1978)	Crack growth is related to plastic energy. Other approaches are Another energy version of LDR,few application of G.	$r_m = 1 - \sum_{i=1}^{m-1} \frac{W_{fi}}{W_{fm}} r_i$
Topper et al (1990).Over load Damage Model,	Crack opening and closure. Other approaches are LLD, LIA, SC,some App .G and E.	$D = \sum D_{dl} + \sum D_{ss} + \sum D_{nl}$

**7.REVIEW ON VARIOUS LIFE PREDICTION MODELS FOR HETEROGENEOUS MATERIALS [23,28,31,35.].**

Models	Basic and Derived equations	Parameters considered and conclusions
Plos et al 2007	$N_{ij} = \frac{1}{D \times \alpha \times \pi^{n/2} \times Y^n \times \Delta\sigma^n \times a_i^\alpha} \left[ 1 - \left( \frac{a_i}{a_j} \right)^\alpha \right]$ $\frac{da}{dN} = D \times (\Delta K)^n$ $\Delta K = Yc \times \Delta\sigma_c \times (\pi \times a)^{0.5}$	Material constants (D& n), stress range in steel, crack in steel, diameter of steel. Based on steel strength
Bazant 1992	$N_{ij} = \left[ \frac{\frac{1}{D \times \alpha \times \pi^{n/2} \times Y^n \times \Delta\sigma^n \times a_i^\alpha} \left[ 1 - \left( \frac{a_i}{a_j} \right)^\alpha \right]}{K_{jm}^n} \right]$ $\frac{da}{dN} = \frac{D \times (\Delta K)^n}{K_{jm}^n}$ $K_{jm} = K_{ic} \times \left( \frac{d}{\bar{d}_o + d} \right)^{\frac{1}{2}}$	Material constants (D& n), stress range in concrete, crack depth in concrete, effective depth of concrete, aggregate size effect factor $\bar{d}_o$ . Based on concrete strength
Sohard jeno et al 2006	$\frac{da}{dN} = D \times (\Delta K)^n$ $K_c = Yc \times \Delta\sigma_c \times (\pi \times a)^{0.5}$ $w = (0.74 - 0.0024 \times fs) \times 10^{-5} \times ((As \div fs) \div (1 + n \times p)) \times e^{(0.013435 \times s)}$ <p>Case 1  <math>1 &lt; N &lt; N_o, w = w_o</math> (initial width of notch) at mid span  <math>N_o = (C \times (fs \div fy) + H)</math></p> <p>Case 2  <math>N_o &lt; N &lt; N_f, w = (A \times e^{(B \times fs \div fy)} \times (N - C(fs \div fy) - D)) + w_o</math></p> <p>Case 3  <math>N_f \leq N, w = ((E \times (fs \div fy) + H) \times w_o)</math>  <math>N_f = (G \times (fs \div fy) + H) \times w_o</math></p>	Material constants (D& n), stress range in steel, crack width and depth in concrete, percentage of rebar, stress ratio. Based on concrete strength
Present modified model	<p>M1 – Modified Equation for Number of Cycle Range to Fracture <math>N_{ij}</math></p> $= \left[ \frac{1}{\left[ \left( \frac{9}{2} \right) - 1 \right] \times D_s \times a_i^{\left[ \left( \frac{9}{2} \right) - 1 \right]}} \right] \times \left[ 1 - \left\{ \frac{a_{ii}}{a_{jj}} \right\}^{\left[ \left( \frac{9}{2} \right) - 1 \right]} \right] \times \left[ \frac{1}{\pi^2} \right] \times \left[ \frac{1}{\Delta\sigma^{\left( 9 \frac{a_{jj}}{a_{ii}} \right)}} \right]$ $\times \left[ \frac{1}{\left\{ \frac{(f_{ckr} \times b \times d \times \left( 1 + \frac{d}{d} + \frac{\bar{d}_o}{d} \right) \times a_{jc}^9) X \frac{1}{m} + (f_{yr} \times A_{st} \times \left( \frac{s}{z} \right) \times a_{js}^{10.6}) + \left( \frac{E_c \times I_c \times \delta}{L^2 \times (e - e')} \right)^{\frac{9}{10.6}}}{GMT} \right\}} \right]$	Material constants(D& n), stress range in steel and concrete ,crack width and depth in concrete and steel, percentage of rebar, stress ratio, size of beam, grade of concrete and steel,damage,Gross million tons of loads shape and size of rebar with ribs, geometrical proportion of beam. Based on concrete and steel strength.



Table-2: Details of Damaged Beam Specimens of Size , 2400 X 400 X 250 mm.

Beams	Grade of concrete (MPa)	Grade of Steel (MPa)	Natural Frequency, maximum and minimum fatigue loads			Diameter of tension reinforcement (mm)	Remaining Area of tension steel in mm <sup>2</sup>	Depth of notch (ac) = k x radius of tension bar (r), (mm)	Coarse aggregate size (d <sub>a</sub> ) (mm)
			Natural Frequency in hertz	P min in Ton	Pmax in Ton				
B1	25	250	4.49	0.80	6.79	16	435.2	5.00	20
B2	35	250	4.84	0.69	5.83	16	371.2	6.50	20
B3	45	250	5.13	0.57	4.85	16	307.2	8.00	20
B4	55	250	5.40	0.77	6.52	16	413.8	5.50	20
B5	25	415	4.49	0.82	6.95	16	435.2	5.00	20
B6	35	415	4.84	0.70	5.95	16	371.2	6.50	20
B7	45	415	5.13	0.58	4.96	16	307.2	8.00	20
B8	55	415	5.40	0.78	6.66	16	413.8	5.50	20
B9	25	500	4.49	0.84	7.14	16	435.2	5.00	20
B10	35	500	4.84	0.72	6.11	16	371.2	6.50	20
B11	45	500	5.13	0.60	5.08	16	307.2	8.00	20
B12	55	500	5.40	0.80	6.83	16	413.8	5.50	20
B13	45	500	5.16	0.96	8.13	16	495.36	3.50	20
B14	45	500	5.16	0.92	7.80	16	476.16	4.00	20
B15	45	500	5.14	0.68	5.76	16	349.8	7.00	20
B16	45	500	5.17	1.16	9.83	16	602.8	0.00	20
B17	45	250	5.17	1.11	9.41	16	602.8	0.00	20
B18	45	415	5.17	1.13	9.61	16	602.8	0.00	20

8.DESIGN CALCULATIONS METHOD TO ARRIVE FOR REMAINING Ast OF DAMAGED BAR,NATURAL FREQUENCY ,EXCITED FREQUENCY AND FATIGUE LOAD RANGE OF DAMAGED BEAMS.

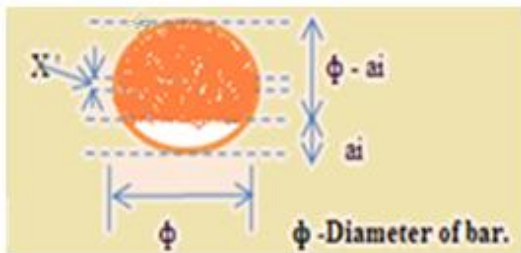


Figure-1: Damaged Bar

8.1.Calculation of Remaining Area of Damaged Steel Bar (Ast-remaining) for Shaded Portion .

Area = A<sub>st-remaining</sub> = B X  $\phi^2$ , Gross moment of inertia is ,  $I_G = C X ((\frac{\phi}{2})^4)$ , Centre of gravity of shaded area is ,  $X' = A X \frac{\phi}{2}$ , initial depth of crack is a<sub>i</sub>. A, B and C are the coefficients from paper [36].

8.2. Calculation of Natural Frequency and Excited Frequency on Beams.

Eoung's modulus of concrete is  $E_c = X 5000 \times \sqrt{F_{CK}}$ . Permissible bending compressive stress is  $\sigma_{cbc} = 0.34 F_{CK}$ . Modular ratio is  $m = \frac{280}{3 \times \sigma_{cbc}}$ . Equivalent Eoung's modulus is  $E_{equ} = E_c \times (1 + m \left(\frac{A_{st}}{bd}\right))$ . Moment of inertia of beam is  $I_G = \frac{250 \times 400^3}{12} = 1333333333 \text{ mm}^4$ . Mass (m') of beam is  $2.4 \times .25 \times .40 \times 25 \times 1000 = 6000 \text{ N}$ . The

stiffness is  $k = \frac{48 \times E_c \times I_G}{L^3}$ . Natural frequency is  $\omega = \sqrt{\left(\frac{k}{m}\right)}$ . Excited frequency is  $p' = \left[\left(\frac{486 \times E_c \times I_G \times X_g}{15 \times P \times 1000 \times L^3}\right)^{0.5}\right]$ , Where L is effective span in m [37].

**9. PROCEDURE FOR FATIGUE LOAD RANGE CALCULATIONS:**

Based on the size of beam (L X B X D), Grade of steel ( $F_y$ ) and concrete ( $F_{CK}$ ), remaining area of steel ( $A_{st\text{-remaining}}$ ), extent of damage ( $a_i$ ) in steel, Diameter and area of tension and compression bars, effective depth, effective cover, effective depth for remaining section of beam, Eoungs modulus of steel, shear span, yield stress of steel, test results of concrete cube and steel, minimum stress in steel ( $0.1 \times F_y$ ), maximum stress in steel ( $0.85 \times F_y$ ), Find strain in steel,  $\epsilon_{s\text{-min}} = \frac{E_s}{0.1 F_y}$ ,  $\epsilon_{s\text{-max}} = \frac{E_s}{0.85 F_y}$ ,  $E_c = X 5000 \times \sqrt{F_{CK}}$ .

1. Arrive the value of  $\epsilon_{c\text{-min}}$  and  $\epsilon_{c\text{-max}}$  from the equation  $\frac{X_U}{\epsilon_c} = \frac{d - X_U}{\epsilon_s}$  and stress in concrete,  $F_{c\text{-min}}$  and  $F_{c\text{-max}}$ , using the equation,  $F_c = \epsilon_c \times E_c$ .

2. By altering the value of  $X_u$ , ensure that  $C = T$ , Where Compression in concrete  $C = \frac{(0.80984 \times F_c \times X \times X_U)}{1000}$  and  $T = \frac{(F_y \times A_{st\text{-remaining}} \times \text{No. of bars})}{1000}$  for maximum and minimum stress ranges in steel and concrete respectively.
3. Get Moment,  $M_1 = C \times \text{Lever arm}$ , where Lever arm  $= d_{eff} - \frac{3 \times X_U}{8}$ .
4. Calculate, Fatigue load  $P_{fatigue} = \left[\frac{M_1}{a_{sh}}\right]$ , where Shear span ( $a_{sh}$ )  $= \frac{\text{Eff. span}(L)}{3}$
5. Arrive Fatigue load P max and P min as described above.

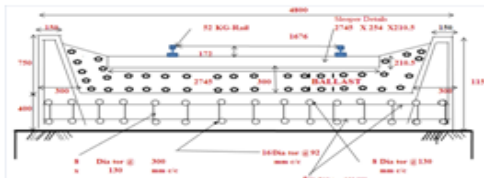


Figure-2: Cross Section of Slab Bridge

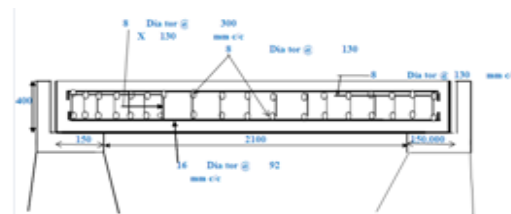


Figure-4: Longitudinal Section of Slab

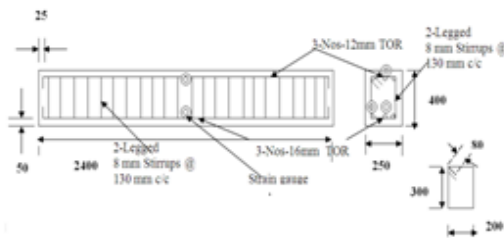


Figure-3: RCC Detailing of Specimen Beam

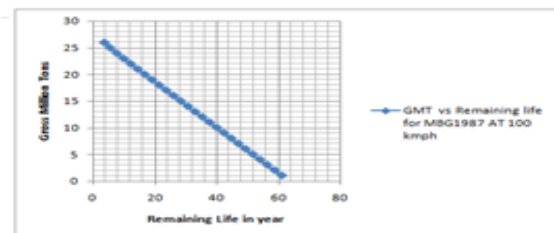


Figure-5: GMT vs Remaining Life

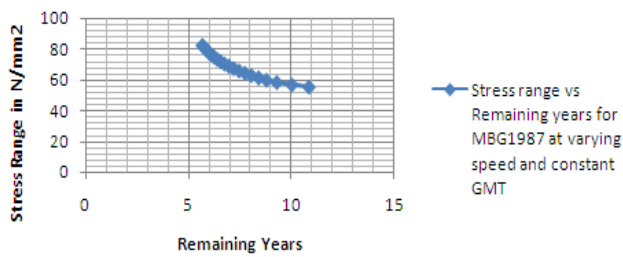
**10. DETAILS OF EXPERIMENTAL PROGRAMME.**

A RCC beam of size 2400 x 400 x 250 mm with damages varying from 5 mm to 8 mm in the rebar at midspan for

various grades of concrete and steel are tested. These beams are subjected to variable fatigue load ranges and deflections, deformations in concrete, strains in rebar are

measured using dial gauges at midspan and LVDT at other locations as shown in the test set up. All the experimental readings are under detailed study for analysis. Based on the findings from test data, the present model for life equations shall be modified ahead [32,33,34.]. The upgraded life equation model will be disclosed in the additional journals.

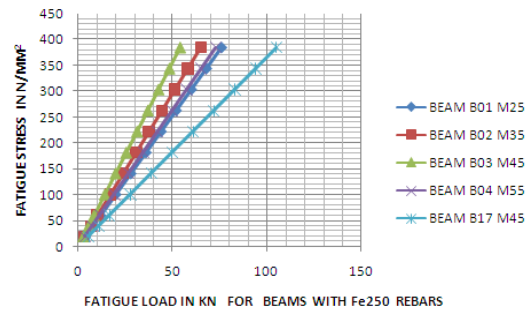
**Stress range vs Remaining years for MBG1987 at varying speed and constant GMT**



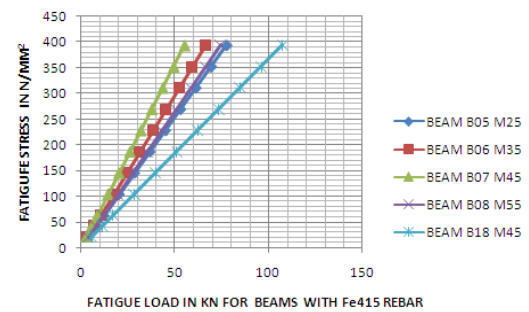
**Figure-6: Stress range vs. Remaining Years for MBG1987 at varying speed and constant GMT**



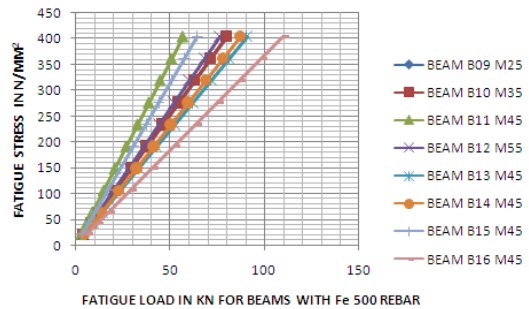
**Figure-7: Fatigue Experiment Setup**



**Figure-8: Fatigue Stress vs Fatigue Load for Fe250 Group.**



**Figure-9: Fatigue Stress vs Fatigue Load for Fe415 Group.**



**Figure-10: Fatigue Stress vs Fatigue Load for Fe500 Group.**

## 11. CONCLUSION

The material modification factor accounting for, workmanship, quality, relaxation time, temperature and previous loading history for concrete ( $a_{jc}$ ) and steel ( $a_{js}$ ) introduced in the service life equation are evaluated from the experiment. This is based on simulating the cyclic loading on RCC beams for varying stress ranges, effective cover, effective depth (depth ratio), crack depth, side or diameter of reinforcement (rebar size ratio), and percentage of steel reinforcement. Developing the empirical equation to predict the remaining service life of RCC beams of slab/girder bridges for the GMT of different standards of railway loadings (Figure 2.) and velocities are feasible to arrive for good fatigue design method of slabs/girders with long life and optimum weight.

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