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Time-Dependent Reliability Analysis for Deflection of a Reinforced Concrete Box Girder Bridge

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Abstract

Prior to casting of concrete, proper supervision and attention to camber provision in bridge construction are required. It is also critical to use an appropriate quality control manual, pay due attention to reinforcement bar placement, and have a high level of formwork design before construction begins. If these issues are not properly addressed, performance of structures will be affected. In this research, performance of a 40.5m box-girder reinforced concrete bridge which was constructed without having proper camber is studied. As camber was the most important issue of the bridge under investigation, the impact on strength and serviceability requirements is compared to the standard. A dynamic load test with an Arduino type accelerometer is performed to assess the bridge's current condition in relation to the serviceability limit requirement. The deterioration of reinforced concrete (RC) sections due to reinforcement corrosion, creep, and an increase in load intensity, as well as the corresponding statistical distributions are considered to estimate the long-term effect of bridge deflection. Time-variant analysis results showed a linear decrease in deflection reliability indices with the bridge's expected service life of 58 years. After strengthening with steel plates, its service period increased to 85 years.

Keywords: Accelerometer, box-girder bridge, dynamic load test, reinforcement corrosion, time-variant analysis

1. Introduction

Pre-camber is generally an upward deflection provided to counteract the deflection and stress produced during the life time of the structure. Bridge girders are provided with camber to resist dead load deflections, so appropriate caution should be exercised during construction and is critical [1]. If bridge defects or construction problems exist, bridge's performance will be reduced, and reconstruction of these structures requires additional resources [2].

Bridge structure deflection caused by dead loads, live loads, design errors, construction mistakes, overload, and other factors should be minimized as much as possible, and this should be addressed during design, construction and operation phases. If excessive deflection occurs it leads to a structural failure with serious economic and life losses [3]. This could also be the result of poor formwork design and construction. Therefore, proper design and construction methodology should be followed to achieve adequate cambering; failing to do so will ultimately affect the performance of bridges. In this regard, structural health monitoring (SHM) has become an essential tool for performance assessment of bridges under various conditions [4]. For such an activity, dynamic load test can be conducted to assess the behavior of the bridge. In this case, accelerometers have been extensively used for bridge monitoring by directly measuring force and the corresponding deflections of the bridges are computed analytically [5].

The core objective of this study is to investigate the current condition of Beressa bridge, to ultimately assure its safety and to check the bridge's long-term deflection because surveying data shows that the camber is nearly nil after the formwork has been removed. Furthermore, a field load test was performed to monitor the displacement of an existing bridge using accelerometer sensors. The field tests were taken at two critical locations of the bridge considering variable vehicular speed (varies from 10km/hr to 50km/hr). The verification is mainly focused to assess whether the deflection requirement of the bridge is exceeded or not. Finally, time-variant reliability analysis for deflection was performed to predict the service life of the bridge. Even if provision of camber is missed during construction, the analysis and test result show that the current condition of the bridge is safe as per strength and serviceability requirements. Considering deterioration of concrete and incremental load intensity, the service period of the bridge is estimated to be 58 years.

2. Literature Review

Deflection is one of the key performance evaluation indicators for bridge structure, and it perfectly reflects the bridge's safety and serviceability condition. As a result, it is essential to evaluate the bridge's serviceability and reliability using deflection data [6]. Camber and deflection that differ from those estimated during design may necessitate changes during construction, resulting in increased costs and longer construction period. Bridges are subjected to environmental and loading conditions during their service life, resulting in a reduction in load carrying capacity [7]. Furthermore, due to uncertainties, defection and camber requirements for bridges are challenging to predict [1].

During service period of bridges, their serviceability and durability gradually deteriorate due to the influence of various factors, which may even result in affecting safety issues. As a result, in order to provide a scientific basis for structural health diagnosis and maintenance decisions to ensure the safe operation of bridges, the service status of bridges must be evaluated throughout their entire service lives. Material properties, load conditions, geometric characteristics, and steel corrosion, among other things, exhibit significant randomness and have a significant influence on the mechanical properties of bridges [8].

Several controlled live load tests are carried out with trucks with known axle weights and configurations by varying loading positions and truck speed. Using the acceleration signal, the displacement signal is obtained analytically [9]. Selection of bridge displacement tracking methods are frequently made based on the site/bridge to be monitored. Traditional approaches, such as linear variable displacement transducers (LVDTs), can be used in a limited number of cases if a fixed reference is available to measure from. Acceleration is typically an easier method used to measure dynamic response of bridges than displacement, the fundamental issue when attempting to integrate acceleration to recover displacements is the presence of low frequency noise in the acceleration signal [10, 11]. In structural engineering, exceeding a beam's deflection limits implies failure of beams in serviceability. Such failure may result in excessive vibrations of the floor slab or beam caused by a lack of stiffness. However, it should be noted that even after excessive deflection, the beam is usually safe from structural failure due to the design, which has taken appropriate precautions to utilize the ductility of the beam as a primitive sign of impending failure [12].

3. Strength Evaluation of an Existing Bridge

3.1. Bridge Data

Beressa bridge, which is located at 128km from Addis Ababa along the main road to Debre Berhan, is a reinforced concrete box-girder bridge that has a clear span of 40m and a bearing shelf width of 0.50m for the bridge seat. The bridge consists of two decks which has a clear carriageway width of 10m each and a curb walkway width of 1.25m. It has five girders, which are spaced at 2.4m on centerline of girders. They are 2600mm in depth and have a web width of 300mm. The top deck and bottom slab are 200mm thick. Test results revealed that compressive strength of concrete is ranging from 29.5MPa to 53.50MPa and yield strength of the reinforcing bars are ranging from 460MPa to 525MPa. Fig. 1 shows elevation view of Berresa bridge.

Fig. 1. Elevation of Beressa bridge

3.2. Design Review

The design review was carried out for different parts of the bridge. The minimum requirements for girder dimensions are as per the AASHTO and ERA BDM [13, 14]. Furthermore, the number of reinforcing bars provided for flexure and stirrups for shear force are found to be adequate. Overall, the findings of the design review clearly indicate that the girders are safe against both maximum flexural and maximum shear action. Materials used in the construction met the minimum requirement stipulated in the design specification. However, since no camber was provided during construction, the requirement for camber was not checked.

3.3. Bridge Load Rating

Load rating is usually performed to assess the capacity of bridges against vehicular loading specified in bridge evaluation manuals [15] and it is computed using Eq. (1) [13, 14, 16]. In the case of rating factor calculation for deflection, ‘R_n’ is considered as the deflection limit set in bridge design manuals [16].

$$RF = \frac{\varphi R_n - \gamma_{Di} D_i - \gamma_{DW} DW}{\gamma_{Li}(L_i + I)} \tag{1}$$

Here, RF is the rating factor, φR_n is the nominal resistance, φ is a resistance factor, D_i is the effect of dead loads, DW is the effect of wearing surface, L_i is the live-load effect, I is an impact factor for the live-load effect, γ_{Di} is the dead load factor, γ_{Li} is a live load factor and γ_{DW} is a load factor for wearing surface.

3.4. Loading Condition

For the computation of effect of live load, a legal load type 3-3 with 36.4ton given in ERA bridge design manual [14] is used. The critical legal load placement (m) and axle load (kN) used for the assessment is shown in Fig. 2. In Fig. 2, CG is the location of the resultant force.

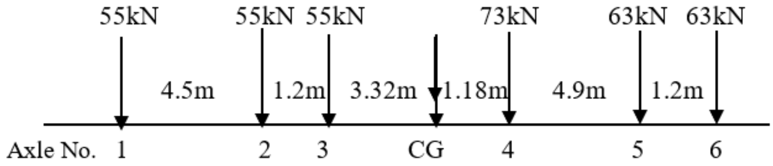


Fig. 2. Truck type 3-3 axle load arrangement

3.5. Material Properties

For sectional analysis, compressive strength of concrete for different sections is taken as; 37MPa for top slab, 35MPa for girder web and 29.5MPa for bottom slab. The yield strength of the reinforcing bars is considered as 460MPa.

3.6. Load Factors

Load factors are used in structural analysis to determine the design strength and compare it with maximum loads [13]. In this study, for the calculations of rating factors, resistance factor of 0.95 has been used. The load and impact factors used in the assessment are taken from bridge evaluation manuals [13, 14, 16]. As per Table 4.6.2.2.2b-1 and d-1 of AASHTO bridge design specification, the distribution factors for shear and moment are computed as 0.796 and 0.611, respectively and the effects of live loads are multiplied by these factors [16]. The effect of loads of the typical interior girder and the factors are given in Table 1.

Table 1. Effect of loads and different factors

Load Effects	Moment (kN-m)	Shear (kN)	Load Factors	Impact Factors
Dead load	8,420.18	830.59	1.20	-
Live load (legal)	2,780.94	298.58	1.65	1.20
Live load (design)	3,396.05	414.35	1.75	1.33

3.7. Section Capacity

Fig. 3 shows the cross section and reinforcement detailing of an interior girder. It has an effective flange width of 2400mm. The capacity of the section is analyzed using Response 2000 software (Reinforced Concrete Sectional Analysis using the Modified Compression Field Theory) and the outputs (bending moment capacity, M_{Rd}=26,345kN-m and shear force capacity, V_{Rd}=3,464.3kN) is shown in Fig. 4 (shear force versus shear strain graph is also shown in Fig. 4b) [17].

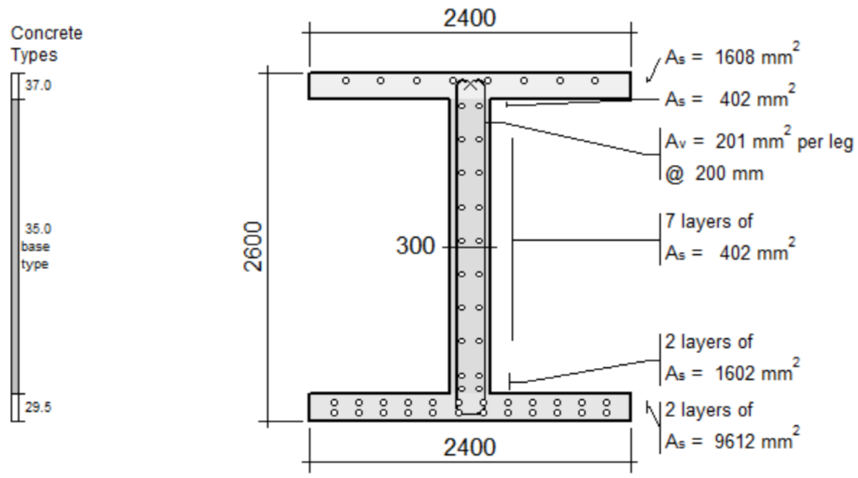


Fig. 3. Cross section of an interior girder

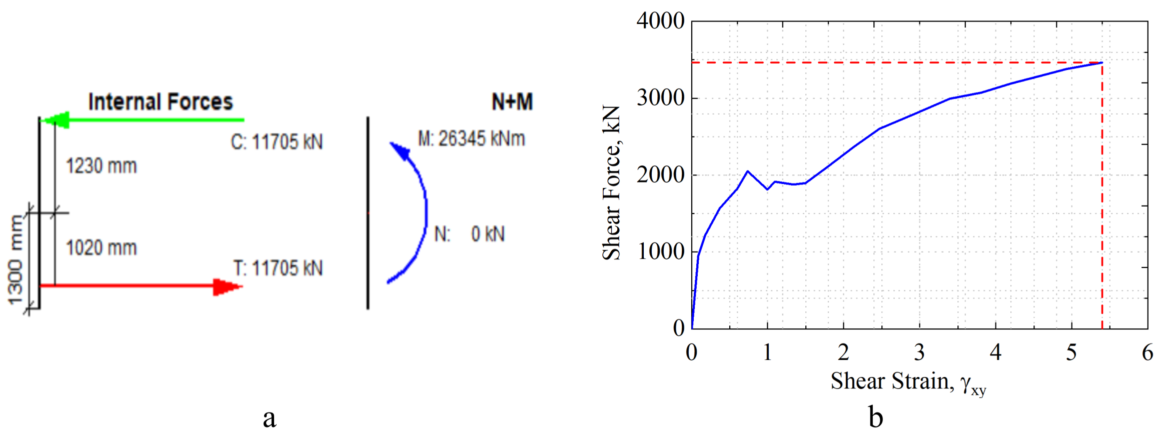


Fig. 4. Capacity of cross section a) bending moment b) shear force

3.6. Rating Factor Calculation

The rating factors of the bridge for shear force, bending moment and deflection considering legal truck type 3-3 and design loads are computed deterministically using Eq. (1). The overall deflection of the bridge due to dead and truck type 3-3 load is computed as 35.72mm which is within safe limits, L_e/800=50.6mm [14]. The analysis results show that the minimum rating factor of the bridge is computed as 2.90 and 2.94 for legal and design loads, respectively (in both cases, shear force governs). Hence, the bridge is reasonably safe against strength requirements.

The bridge also satisfies serviceability requirements as the rating factors for deflection are 4.85 and 5.38 for design and legal loads, respectively. In this case, Eq. (1) is used with the following conditions; R_n is the deflection limit (50.6mm), D_i is deflection due to dead load (31.85), L_i is the live load deflection (3.86mm and 3.48mm for design and legal loads, respectively). For the computation of live load deflection, distribution factor, impact factor and 25% of the live load (AASHTO Article 3.6.1.3.2) are considered [13].

4. Field Load Test

Load test on bridges is performed to evaluate their load carrying capacities and there are different methods. Among these, dynamic tests are important and are carried out to evaluate dynamic characteristics (natural frequency, mode shape, damping ratio, and so on) [11].

In this study, the field test is designed to assess the performance of the defective girder and solely targeted on the strength, stiffness, and geometry aspects. The equipment and instrumentation used for the field test include; loaded truck-44 ton (loaded with bulk sand) and accelerometer with simulated software. The accelerometer used in this test was an Arduino type accelerometer, MPU-6050.

4.1. Truck Load Test

The wheel arrangement of the truck is measured with a spacing of 3.80m and 1.45m from the front axle to rear ones. The corresponding loads on each of the axles are 11.25ton, 16.875ton and 16.875ton, with a total load of 44tons. The truck used for field load test is shown in Fig. 5.

The truck loading test was mainly aimed to assess the performance of the bridge (stiffness requirement) under moving load action of the loaded truck. The truck weighs 44ton and was made to pass on the bridge at different speeds (10km/hr to 50km/hr). These were considered to asses any potential change in the response of the bridge due to speed or impact. For each loading case, accelerometer measurements were recorded for each position. The sampling frequency for all tests is 10Hz.

Fig. 5. Truck used for loading test - 44ton

4.2. Instrumentation Layout

The actual layout of the bridge showing the locations of accelerometers is shown in Fig. 6. In Table 2, the location of instruments is listed. Critical locations are selected on points of maximum deflection; at interior and exterior girders. The truck loading test was conducted using the set-up presented in Fig. 7.

Table 2. Position of accelerometers

Test No. / Points	x (m)	y (m)	Speed (km/hr)	Remarks
1/A	20.25	1.20	10, 20, 30 and 50	Exterior girder, mid-span
2/B	20.25	1.90	10 and 30	Interior girder, mid-span

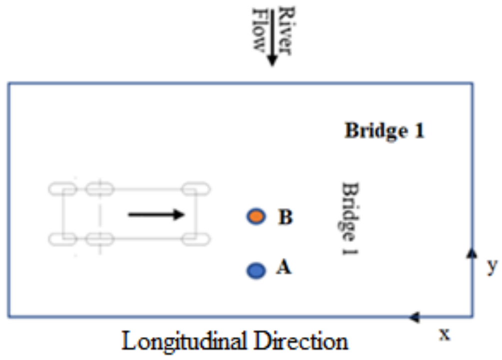
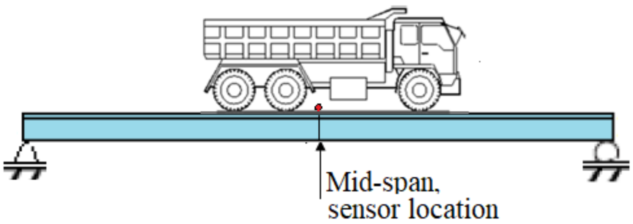


Fig. 6. Position of sensor from plan view of the bridge (not to scale)



a

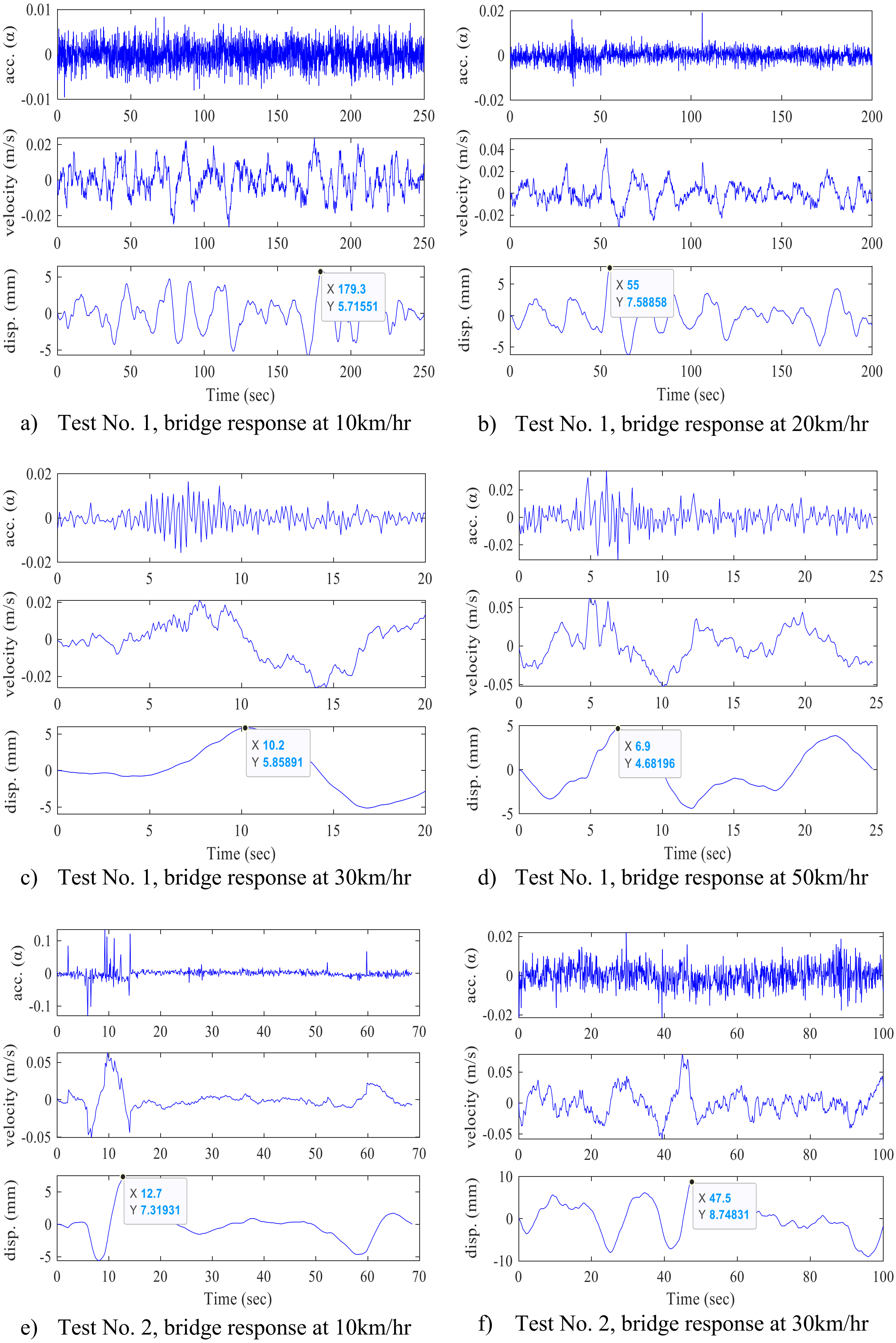
b

Fig. 7. Set-up for truck loading test and accelerometer

4.4. Truck Loading Test Results

The deflection of the bridge can be calculated by the double integral of the acceleration measured through accelerometers [6]. The recorded acceleration, computed velocity and displacement responses using Fast Fourier Transform (FFT) of the bridge at different locations consisting of a single truck traveling at different speeds are shown in Fig. 8 (a to f). In the figures, the acceleration is expressed in terms of ground acceleration ratio (α).

As can be observed from the displacement response curve, the maximum deflection of the bridge is 8.748mm, which occurred at point B with a single truck moving at 30km/hr (Fig. 8f). In contrast, the maximum allowable limit of deflection at mid span is $L_e/800$ = 50.60mm [13]. The actual recorded mid span deflection is one - sixth of the limit for deflection at mid-span, making the girder reasonably safe against serviceability requirements



a) Test No. 1, bridge response at 10km/hr
b) Test No. 1, bridge response at 20km/hr
c) Test No. 1, bridge response at 30km/hr
d) Test No. 1, bridge response at 50km/hr
e) Test No. 2, bridge response at 10km/hr
f) Test No. 2, bridge response at 30km/hr

Fig. 8. Bridge responses at critical locations with different vehicular speeds

5. Structural Reliability Analysis

5.1. Reliability Index

A reliability index is an attempt to quantify a system's reliability using a single numerical value. The requirements to the safety of the structure are consequently expressed in terms of the accepted minimum reliability index (β) or the accepted maximum failure probability. In a general case, the probability of failure P_f given in Eq. (2) is defined by the limit state function g(x)< 0 [18] and the limit state function (LSF) is defined in Eq. (2) as the boundary between safety and failure region [15].

$$P_f = P(g(x) < 0)$$
$$g(x) = R(x)\text{-}S(x) \tag{2}$$

where P_f is the probability of failure, $g(x)$ is the limit state function, $R(x)$ is the resistance and $S(x)$ is the load effect. Thus, the first order reliability index is to be computed from Eq. (3) [19]:

$$\beta = \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R{}^2 + \sigma_S{}^2}} \tag{3}$$

where β is reliability index, $\bar{R}$ and $\bar{S}$ are mean value of resistance and load effects, respectively. σ_R and σ_S are standard deviations for the resistance and load effects, respectively. For reliability analysis of the bridge under consideration, as shown in Table 3, 12 statistical random variables with five groups have been considered.

Table 3. Statistical distribution of random variables

No.	Random variables	Mean values	CoV (%)	Std. dev.	Distribution
1	Statistical distribution of material properties				
1.1	Yield strength of flexural reinforcement steel (MPa)	436	5	21.80	Lognormal
1.2	Compressive strength of Concrete (MPa)	37	10	3.70	Lognormal
2	Statistical distribution of reinforcement bars				
2.1	Longitudinal bars (mm^2)	22,507	5	1,125.4	Normal
3	Statistical distribution of force effects				
3.1	Live loads, dead loads, wearing surface	1.00	5	0.05	Normal
3.2	Analysis Variable for DL and LL	1.00	5	0.05	Lognormal
4	Statistical distribution of different factors				
4.1	Resistance factor	0.90	10	0.09	Normal
4.2	Model uncertainty, N_R	1.00	4.6	0.046	Lognormal
5	Statistical distribution of bridge dimensions				
5.1	Bridge span (m)	40.5	0.05	0.02	Normal
5.2	Web width (mm)	300	0.5	1.50	Normal
5.3	Web depth (mm)	2600	0.5	13.00	Normal
5.4	Girder spacing (mm)	2400	1.0	24.00	Normal
5.5	Top and bottom slab thicknesses (mm)	200	0.5	1.00	Normal

For reliability assessment of the defective girder, different combinations of random variables of Latin Hypercube Sampling (LHS) are used [20]. In the LHS sampling method, the cumulative distribution function of each factor is divided into intervals with equal probability, and then sampling is done only from each interval [21, 22, 23]. The different combinations of random variables are generated using a MATLAB built-in function for LHS design [24].

As deflection is the major concern of the bridge under consideration, a reliability index for deflection is calculated and compared with the minimum limit. In this scenario, 'R' is taken as the deflection limit and 'S' is the deflection of the bridge due to service loads [16]. For the current condition, the probabilistic distributions of S and M (design margin=R-S) for deflection are plotted and shown in Fig. 9. The reliability index for deflection of the bridge is calculated using Eq. (3) and found to be 6.47, i.e., β is computed by taking the deflection limit as constant ($\overline{R}$=50.6mm and σ_R =0) and the overall deflection of the bridge (dead and live load deflections) as random variable ($\overline{S}$ =35.72mm and σ_s =2.30). The reliability index of the bridge is within the acceptable standard and exceeds the safety index limit set for newly constructed bridges; which is 3.5 and above [25, 26].

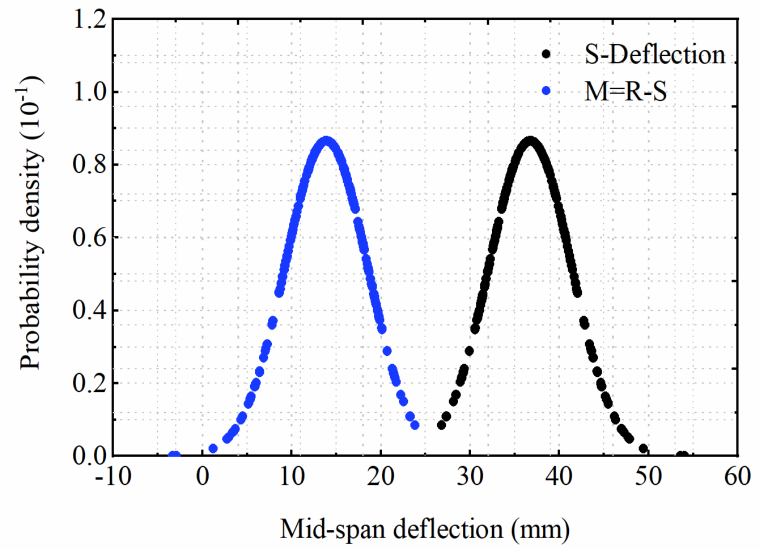


Fig. 9. Probabilistic distribution of S and M for deflection

5.2. Time Variant Analysis

As a live load acting on a bridge structure and its resistance changes with time, especially due to deterioration of the structure, the service time of deflection of the bridge (time-dependent deflection) is considered. For time dependent random variables, the limit state function given in Eq. (2) is modified to:

$$P_f = P(g(x(t)) < 0) \quad \text{for } t\ [0,T] \tag{4}$$

In Eq. (4), [0, T] denotes the reference period, which can be structural life-time or other period of interest. Time-dependent reliability analysis for deflection of the bridge due to dead and live loads are computed to predict service life of the bridge. For time-variant analysis, the followings are considered:

5.2.1. Corrosion type

Carbonation-induced corrosion with an exposure class of moderate humidity, i_{corr} of $0.10\mu A/cm^2$ [27] has been used. Time of corrosion initiation is assumed to be 30 years. The reduction in bar diameter as a function of time assuming uniform corrosion obtained from Eq. (5) is considered [7, 28, 29].

$$\begin{gathered} \phi(t) = \phi_0 - \alpha P_x(t) \\ P_x(t) = 0.0116\, I_{corr}\,(t - t_o),\ t > t_o \end{gathered} \tag{5}$$

Here, ϕ (t) is residual diameter at time t (mm), ϕ_0 is the initial bar diameter (mm), a is equal to 2 (for carbonated concrete), $P_x(t)$ is the average value of the attack penetration (decrease of bar radius) at time t, in mm, t_o is the time of corrosion initiation (years), t is elapsed time (years) and I_{corr} is the corrosion rate ($\mu A/cm^2$). As bridge stiffness is deteriorated due to concrete cracking caused by reinforcement corrosion and external load, reinforcement cross-sectional area reduction and bond degradation need to be considered in the analysis. To characterize the influence of various adverse factors caused by reinforcement corrosion on bridge stiffness, the empirical equation given in Eq. (6) can be used [30].

$$I_{ce} = \gamma(\rho)\, I_e \tag{6}$$

In Eq. (6), I_{ce} is an effective bending moment of inertia of the bridge after reinforcement corrosion, $\gamma(\rho)$ is the correction coefficient expressed in Eq. (7) and ρ is the corrosion rate of reinforcement.

$$\gamma(\rho) = \begin{cases} 1 & \rho \leq 0.05 \\ 1.22 + 12.88\rho^2 - 5.05\rho & \rho > 0.05 \end{cases} \tag{7}$$

5.2.2. Time-variant load

In this study, a linear (α=1) time-variant load increment given in Eq. (8) is applied for legal truck-type 3-3 with an incremental rate of 0.004 (assuming that over 75 years, the live load intensity increases by 30%) [31].

$$\mu_2(t) = \mu_2(0) \times (1 + a t^{\alpha}) \tag{8}$$

Here, $\mu_2(t)$ is the load intensity at time t, $\mu_2(0)$ is the initial load intensity, α is time-variant load increment and a is the scale factor or annual live load increment (1/year).

5.2.3. Creep coefficient

For time-dependent creep coefficient prediction, the creep function given in Eq. (9) is used and the reduced modulus of elasticity of concrete is computed accordingly [32, 33].

$$\varphi\,(t, t_0) = \frac{(t - t_0)^{0.6}}{10 + (t - t_0)^{0.6}}\,\varphi\,(t_0)\gamma_c \tag{9}$$

where, φ (t, t_0) is creep coefficient at time t due to a load at time t_0, φ (t_0) is the ultimate creep coefficient =2.35 [32], t is age of concrete in days, t_0 is initial time of loading in days and γ_c is creep correction factor for non-standard conditions found in [32].

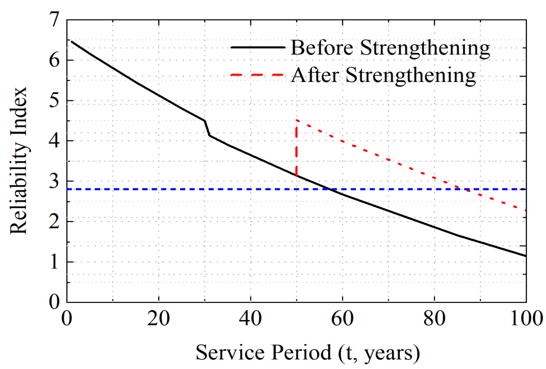
For time-variant reliability analysis, corrosion rate, load effects, time of corrosion initiation and creep coefficients are considered. Statistical parameters of time-variant random variables with their distributions are shown in Table 4 [20, 25, 34, 35, 36]. Hence, based on the variability of random variables, different combinations have been generated using a MATLAB built-in function for LHS design [24]. Furthermore, the reliability indices of deflection are calculated for 100 years of bridge at various service years at 5-year incremental and the evolution of the reliability indices is obtained.

Table 4. Statistical distribution of time-variant random variables

No.	Random variables	Mean values	CoV (%)	Std. dev.	Remarks
1	Corrosion rate (μA/cm^2)	0.10	0.30	0.0003	Lognormal
2	Attach penetration, $P_x(t)$	Eq. (5)	0.02		Lognormal distribution
3	Time variant load effects	Eq. (8)	0.35		Extreme type I
4	Scale factor or load increment (a)	0.004	0.30	12×10^{-6}	Normal distribution
5	Time of corrosion initiation (years)	30	0.20	0.06	Normal distribution
6	Creep coefficient, φ (t_0)	2.35	10	0.23	Normal distribution

5.3. Life Time Prediction and Strengthening

As shown in Fig. 10a, the structural-life of the bridge without the need of any maintenance intervention is predicted as 58 years as the minimal recommended value for evaluation is 2.8 for RC bridges corresponding to a rating factor of 1.0 [25, 33, 37]. After approximately 58 years of service, the reliability index falls below the target value, indicating that the bridge may exhibit greater deflection than expected, necessitating maintenance or speed control actions. Under normal conditions, to carry out maintenance activities, a three-year maintenance plan before the performance of the bridge reaches to the minimum target strength is required [38]. Here, t=50 years is considered as the maintenance intervention period for the Berresa bridge. If strengthening of the reinforced concrete girder bridge using external steel plates (with a thickness of 8mm, overall depth of 1200mm and a steel grade of 235MPa) are proposed as shown in Fig. 10b, the reliability index curve for deflection will be improved and the predicted service period of the bridge will be extended to 85 years, as shown in Fig. 10a. The stiffness of the girder is computed considering the attached steel plates that reduce deflection [39, 40] and reliability indices for deflection are computed accordingly.



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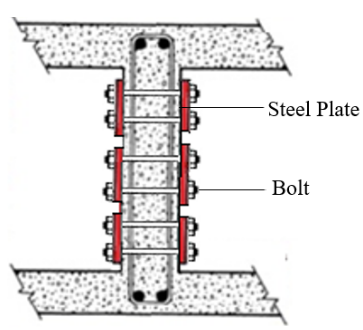

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Fig. 10. a) Time-dependent reliability index for 100 years b) cross section of the proposed strengthening method for longitudinal girders

6. Conclusions

The performance of the bridge is evaluated through various approaches including design check, strength evaluation through design loads, legal loads and field loading test. The current performance of the bridge satisfies both strength and service limit requirements owing to the fact that the corresponding rating factor and reliability index limits are fulfilled. The numerical and field test results revealed that even if camber was not provided, the bridge is safe from structural failure because of the Strength I design requirements stipulated in design specifications. Time-dependent reliability analysis result; considering time variant loads, possible future reinforcement corrosion and creep effects, shows that the service time of the bridge is predicted as 58 years. The service period of the bridge extended to 85 years if steel plates are attached to longitudinal girders at t=50 years. To ensure the bond between RC girders and external steel plates, mechanical technique of shear connection is required. Regular inspection accomplished with truck load test and strength evaluation is recommended before maintenance intervention is made for strengthening.

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