PROBABILISTICASSESSMENT AND FIELD TEST VERIFICATION FOR STRENGTH EVALUATION OF BRIDGE WITH DEFECTIVE GIRDER

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ABSTRACT

The use of high-quality materials, regular inspection and testing of constituent materials, appropriate curing processes, and an acceptable overall construction methodology are all required for bridge construction. Failure to handle the aforementioned ways causes the bridge to operate poorly, jeopardizing its load-carrying capacity. As a result, determining the load carrying capacity of the bridge and assessing its safety are critical. This study investigated the performance of a defective girder of a Reinforced Concrete (RC) bridge (located at km 69+937 along the Agulae-Berahle road segment)caused by failure of concrete to achieve the desired compressive strength. Various assessment methods were used to assess the strength of the existing defective girder. Following the uncertainty of random variables, a probabilistic assessment approach was used. A field load test was also used to correlate and verify the numerical result. The findings reveal that the bridge under investigation is safe against the design and legal loads specified in ERA Bridge Design Manual.

Keywords: RC, deterministic approach, defective girder, probabilistic assessment, strength evaluation, uncertainty, random variables, load test

1 INTRODUCTION

The construction of a transportation network is critical for a country's development and requires significant investment. Bridges and culverts account for a significant portion of a highway project's cost. In line with this, proper construction of minor and major structures is critical, and special attention should be paid to avoid further damage throughout the operation phase. Bridge structure deficiencies are caused by design problems, poor construction quality, aging, excessive loads, maintenance negligence, and other factors. Assessing the safety of a bridge structure is a critical duty in bridge management that requires extra attention and caution. If bridge defects or construction problems are observed, load capacity of bridges will be reduced and reconstruction of these structures will consume extra time and money [1].

This study looked into performance assessment of a 20.50m defective reinforced concrete girder (bridge at km 69+937, Agula-Barahle road segment) caused by construction fault by which concrete of a lower grade than that specified in the specification has been used. The bridge was built in 2014, and a performance evaluation was conducted four years later as there was a defect on one of the interior girders (problem with concrete quality).The objective of this research was to investigate the safety level of a defective girder using numerical analysis and bridge load test. In the numerical analysis, rating factors and safety indices of the bridge were computed using deterministic and probabilistic approaches, respectively and the results were checked against the standard. For the probabilistic approach, uncertainties of variables were taken from experience and specifications. A field load test was also performed to verify the serviceability requirements of the bridge. The analytical result was correlated with field load test and the results show that the bridge can carry the design and legal loads specified in the country's bridge design manual.

2 METHODS

Methods of Structural Assessment

Data acquisition and structural analysis are procedures for gathering necessary information about the structure's condition, which is used to access and evaluate the safety and serviceability margin where the failure zone can be estimated [2].

The deterministic approach is the most commonly used method of defining safety. It is entirely based on experience, and the safety measures are empirical in nature. Deterministic verification is characterized by simplifications and associated with those by conservative safety measures [2]. In the case of deterministic approach, all the mean values of the variables with appropriate factors stipulated in the codes are taken.

In practice, material properties, dimensions, loads, and so on, used in structural assessment have uncertainties. As a result, there is a need to consider the statistical variations of these random variables, which includes a probabilistic approach (reliability is the most reasonable approach), which allows for a more systematic determination of structural reliability [2].

In a probabilistic assessment, uncertainties and analysis variables for dead and live loads, as well as model uncertainty (NR) that accounts for error in the modeling process must be considered [3]. Furthermore, reduction of uncertainties using past experience, use of load and resistance calculation technique is possible to make the necessary decisions so that the assessment work can be completed properly [4]. 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 (P_f). In a general case, the probability of failure P_f is defined by the limit state function, $g(x) < 0$ and it is given in Eq. (1) [5]:

$$P_f = P\,(g(x)<0) \qquad (1)$$

where:

P_f is the probability of failure
$g(x)$ is the limit state function, design margin=$R(x)$-$S(x)$
$R(x)$ is the resistance of the section and $S(x)$ is effect of loads

For a given limit state function, the reliability index can be calculated using a direct integration method. However, determination of the safety index using the direct integration method becomes complex, especially if a number of random variables are involved (the probability integration is multidimensional). First Order Reliability Method (FORM) and the Second Order Reliability Method (SORM) are commonly used to ease the computational difficulties [5]. In most cases, empirical equations are used. In this study, as either of the random variables has log-normal distribution, the reliability index and the multiplication factor are estimated based on the expressions given in Eqs. (2) and (3), respectively [6].

$$\beta = \frac{\mu_R\left(1 - k\frac{\sigma_R}{\mu_R}\right)\left[1 - ln\left(1 - k\frac{\sigma_R}{\mu_R}\right)\right] - \mu_s}{\sqrt{\left(\mu_R\left(1 - k\frac{\sigma_R}{\mu_R}\right)\left(\frac{\sigma_R}{\mu_R}\right)\right)^2 + \sigma_S^2}} \qquad (2)$$

$$k = \frac{\bar{R}^e - r^*}{\sigma_R^e} \qquad (3)$$

where:

β is the reliability index
μ_R and σ_R are mean and standard deviation for the resistance, respectively
μ_s and σ_S are mean and standard deviation of total-load effect, respectively
k is a multiplication factor of the standard deviation
$\bar{R}^e$, σ_R^e aremean and standard deviation for the resistance of the approximating normal distributions (equivalent normal parameters), respectively
$r*$ is a design point on the failure boundary

Load Tests on Bridges

For existing bridges with large uncertainties, analytical methods have limitations, bridge load test is commonly used. It helps to determine issues that cannot be easily resolved by simple analysis [7]. A proof load test is one type of field test in which a load equal to the factored live load is applied. If the bridge can carry this load without signs of distress, the proof load test is found to be successful. A supplementary load test is also used for the assessment process and it is preferred as it involves applying a known load to the bridge [8].

Table 1 Bridge data

Description	Values
Bridge span, L	20.50m
Area of main reinforcing bars	18 Φ32 in four layers
Transversal reinforcements	Φ12 c/c 130mm
Cylindrical compressive strength of concrete	16MPa
Yield strength of steel	400MPa
Girders spacing	2450mm
Web width	500mm
Girder depth	1450mm
Slab thickness	200mm

Strength Evaluation

The strength evaluation of a reinforced concrete bridge with defective girder is discussed hereunder. Bridge data used in the assessment was obtained from the construction drawing (see Table 1).Design checking was also performed to ensure that the reinforcing bars used in the construction were adequate.

The overall depth of the girder was checked with the design specification of AASHTO and ERA bridge design manuals (=0.07×20,500=1,435mm) and it was found out that it satisfied the minimum requirement. Web width of 500mm was used (greater than the minimum requirement, 250mm). The slab thickness was 200mm which was greater than the minimum requirement of 185mm [9, 10]. The reinforcement for the longitudinal interior defective girder using lower concrete strength was computed and is shown in Table 2.

Table 2 Reinforcements in the interior girder

Design review result	Actual bridge detailing
17 ϕ32 in four layers (Main reinf.)	18 ϕ 32 in four layers
ϕ 12 c/c 150mm (stirrups)	ϕ 12 c/c 130mm

Overall, the findings of the design review clearly indicate that the defective girder was safe against both maximum flexural and maximum shear action with sufficient margin of safety. The adequacy of the defective girder bridge for the proposed load was comprehensively assessed and the summary report is presented here below. The design compressive strength of concrete was C30/25, however laboratory test result showed that the actual concrete grade was C20/16.

Strength evaluation of bridge is expressed in terms of rating factor (RF). It is the ratio of the available capacity of the bridge to the effect produced by the live load being investigated and it is given by Eq. (4) [8-10]:

$$RF = \frac{\varphi R_n - \gamma_{Di} D_i - \gamma_{DW} DW}{\gamma_{Li}(L_i + IM)} \tag{4}$$

where:

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

If the rating factor for legal loads exceeds 1.0, the bridge is said to be satisfactory for the legal loads and it is within the acceptable range for safety verification [8-10]. For the computation of effect of live load, a legal load type 3-2 with 32.5ton (Figure 1**)** given in ERA bridge design manual was used [9].

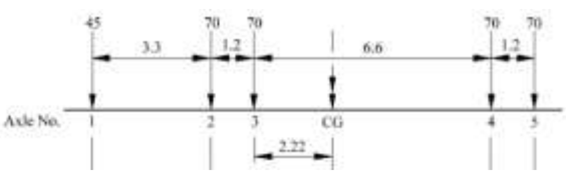

Figure 1 Truck type 3-2 axle load arrangement

The critical legal load placement (m) and axle load (ton) used for the assessment was obtained by influence line analysis. Using Response 2000 software (a reinforced concrete sectional analysis software using the modified compression field theory), the resistance of the defective girder in terms of bending moment was computed (f_y=400MPa and f_c'=16MPa). Figure 2 shows the cross-section of the defective RC girder.

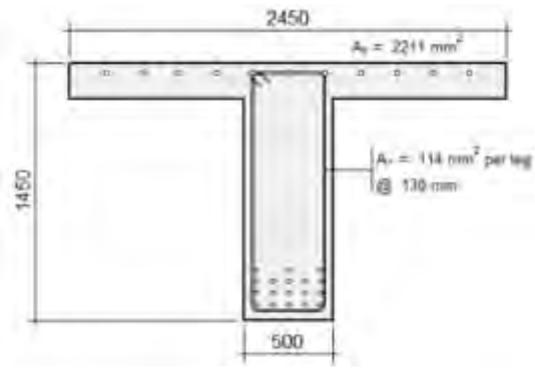

Figure2 Cross section of an interior RC girder

In order to further investigate the load-deflection response, a nonlinear analysis was carried out on the defective girder. The analysis was conducted on COM3 platform with path and time dependent constitutive laws rooted in four-way fixed crack-approach for reinforced concrete. To reduce the computational time, the girder was modeled as half symmetry with the moving truck axle load over the bridge.

Using simple static calculations, nearly 60 % of the truck load was resisted by the defective girder for the considered truck position. As load distribution based on simple static analysis overestimates the load share of the defective bridge; however, FEM analysis was conducted to see the extreme condition to assure safety.The3D finite model of the girder is indicated in Figure 3. Here, it must be noted that, the differing values of compressive strength for the deck and the web regions was explicitly considered.

Rating factors for shear and moment considering design truck load and legal truck type 3-2 stipulated in ERA Bridge Manual [9] were calculated deterministically using Eq. (4). For such calculations, resistance factor of 0.9 was used. The load and impact factors used in the assessment were taken from bridge evaluation manuals [8-10]. Impact factors of 1.33 and 1.10 were used for design and legal load ratings case, respectively. As per Table 4.6.2.2.2b-1 and d-1 of AASHTO bridge design specification, the distribution factors for shear and moment were found to be 0.867 and 0.667, respectively and the effects of live loads were multiplied by these factors [10].

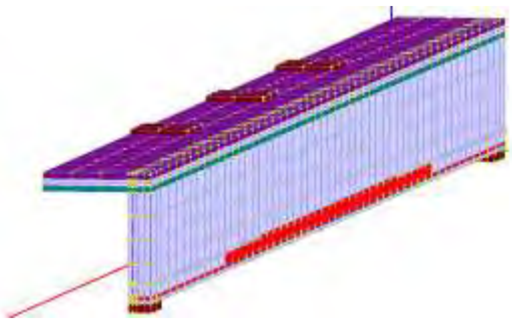

Figure 3FE model; half-symmetry

3 RESULTS AND DISCUSSIONS

3.1. Deterministic Assessment

Figure 4 shows the section capacity of the defective RC girder (output was obtained from Response 2000 software). The result of the 3D finite model (Figure 5) shows that the maximum shear capacity of the defective girder is 1540 kN (=2×770 kN).

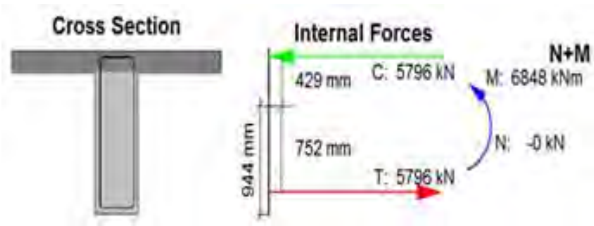

Figure 4 Section capacity

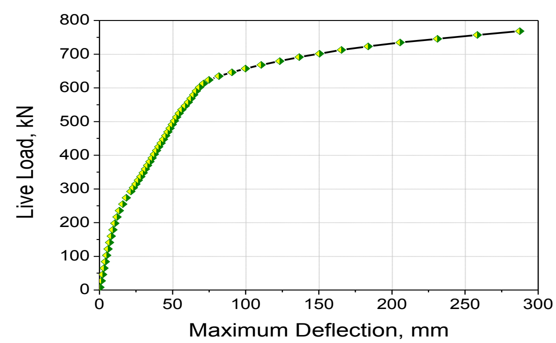

Figure 5 Load-displacement diagram

Tables 3 and 4 show the rating factor of the defective girder. The rating factors of the bridge due to design and legal loads became 1.26 and 2.64 (shear force governs), respectively. As these rating factors are greater than one, the bridge under consideration is safe against design and legal loads. The corresponding available capacity of the existing bridge is 40 ton (=2.64×32.5).

Table 3 Rating factors for shear of the defective girder

Rating		Dead load shear (kN)		Live load shear (kN)	Load factors			RF
		concrete section	wearing surface		concrete section	wearing surface	live load	
Design Load Level	Inventory	273.31	61.05	279.91	1.25	1.50	1.75	1.26
	Operating			95.33*	1.25	1.50	1.35	1.63
Legal Load	Truck Type 3-2			236.54	1.20	1.20	1.65	2.64

Table4 Rating factors for moment of the defective girder

Rating		Dead load moment (kN-m)		Live load moment (kN-m)	Load factors			RF
		concrete section	wearing surface		concrete section	wearing surface	live load	
Design Load Level	Inventory	1,395.42	312.87	1,286.29	1.25	1.50	1.75	1.43
	Operating			488.54*	1.25	1.50	1.35	1.86
Legal Loads	Truck Type 3-2		80.32	993.96	1.20	1.20	1.65	3.42

* Effect of lane load

3.2. Probabilistic Assessment (Structural Reliability)

The deterministic values considered above were taken as mean values and the statistical distribution of random variables with corresponding coefficient of variations were obtained from standards, codes and manuals [2, 3, 12]. In this study, the shear capacity of a structure with deterministic value was considered as the reduction in shear capacity of the member by deterioration was very small [3]. For the bridge under consideration, as shown in Table 5, 17 (*n*) statistical random variables with five groups have been considered.

Table5 Statistical distribution of random variables

No.	Random variables	Mean values	CoV (%)	Std. dev.	Distribution
1	Statistical distribution of material properties				
1.1	Yield strength for flexural reinforcement steel (MPa)	400	5	20.00	Lognormal
1.2	Cylindrical compressive strength of concrete (MPa)	16.0	10	1.60	Lognormal
2	Statistical distribution of reinforcement bars				
2.1	Longitudinal bars (mm^2)	14,470	5	720.90	Normal
3	Statistical distribution of force effects				
3.1	Live loads	1.00	25	0.25	Normal
3.2	Dead loads	1.00	5	0.05	Normal
3.3	Overlay (wearing surface)	1.00	20	0.20	Normal
3.4	Analysis Variable for DL	1.00	5	0.05	Lognormal
3.5	Analysis Variable for LL	1.00	5	0.05	Lognormal
4	Statistical distribution of different factors				
4.1	Distribution factor for moment	0.667	2.5	0.016	Normal
4.2	Distribution factor for shear	0.867	2.5	0.022	Normal
4.3	Resistance factor	0.90	10	0.09	Normal
4.4	Model Uncertainty, N_R	1.00	4.6	0.046	Lognormal
5	Statistical distribution of bridge dimension				
5.1	Bridge Span (m)	20.5	0.05	0.01	Normal
5.2	Web width (mm)	500	0.5	2.50	Normal
5.3	Web depth (mm)	1450	0.5	7.25	Normal
5.4	Girder Spacing (mm)	2450	1.0	24.50	Normal
5.5	Slab thickness (mm)	200	0.5	1.00	Normal

For reliability assessment of the defective girder, 256 combinations of random variables of Latin Hypercube Sampling (LHS) were used [2]. For the computational analysis, a MATLAB code was prepared which enabled to compute the cross-section resistances, effect of loads, design margin, rating factors. It was also used to plot the corresponding probability density curves. The relationship between resistance and effects of loads for both bending moment and shear force are shown in Figures 6 and 7, respectively. The RF for each case was computed and the scattered plot of RF for shear force and bending moment is shown in Figure 8. The probabilistic assessment results are summarized in Table 6. The mean values of rating factors for shear force and bending moment were found to be 3.71 and 6.41, respectively, which are 40% to 80% higher than those obtained using the deterministic approach. Furthermore, the probabilistic distributions of bending moment and shear force are shown in Figures 9 and 10, respectively.

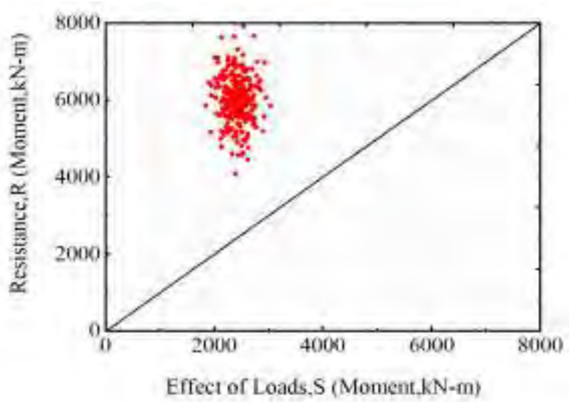

Figure 6 Distribution of moment resistance and effect of loads

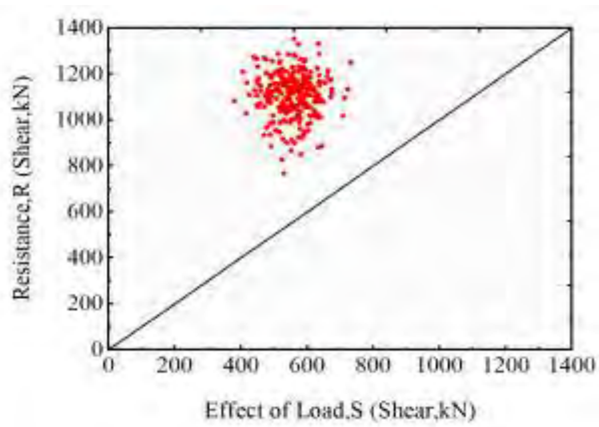

Figure 7 Distribution of shear resistance and effect of loads

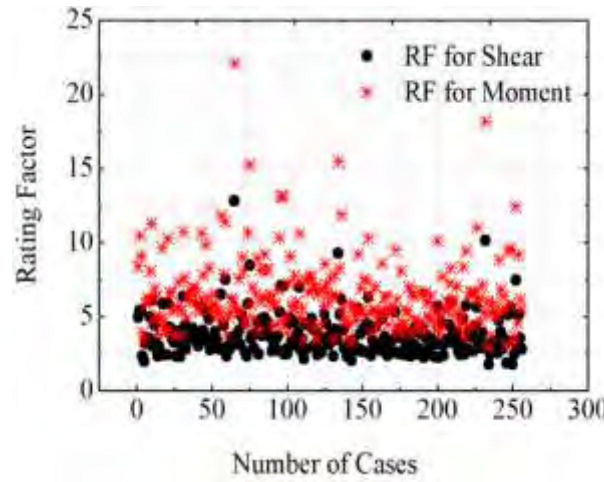

Figure 8 Rating factors for shear force and bending moment

As shown in Table 6, the bridge's safety index was 5.08 (indicating that the unsafe region or failure region is about five standard deviations away from the mean) with a failure probability of 10^{-7} [2]. The result indicates that the bridge meets the standard (the safety index limit of 2.5) [10] and satisfies the minimum safety index limit set for newly constructed bridges (3.5 and above) [3]. Furthermore, the safety index of the bridge is greater than 2.80, corresponding to a rating factor of 1.0, indicating that the bridge is safe [3].

Table 6 Design margin and safety index for shear and moment

Criteria		Resistance (φR_n)	Load (S)	Design Margin	Rating Factor	Safety Index
Shear	Mean	1,104.79	556.13	548.66	3.71	5.08
	Std. dev.	92.35	59.03	108.42		
	CoV (%)	8.36	10.62	19.76		
Moment	Mean	6,019.01	2,423.67	3,595.34	6.41	6.85
	Std. dev.	618.77	203.33	656.79		
	CoV (%)	10.28	8.39	18.27		

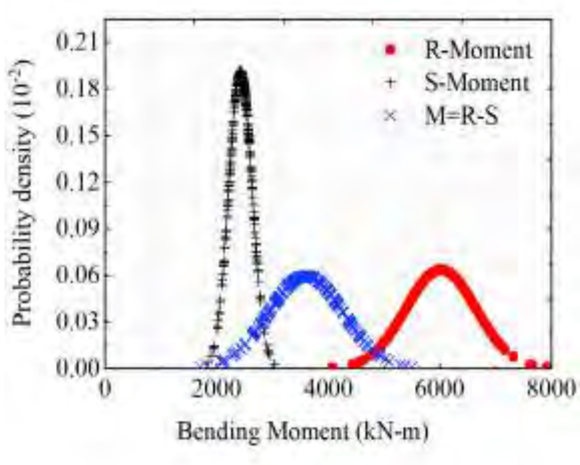

Figure 9 Probabilistic distributions of R, S and M for Moment

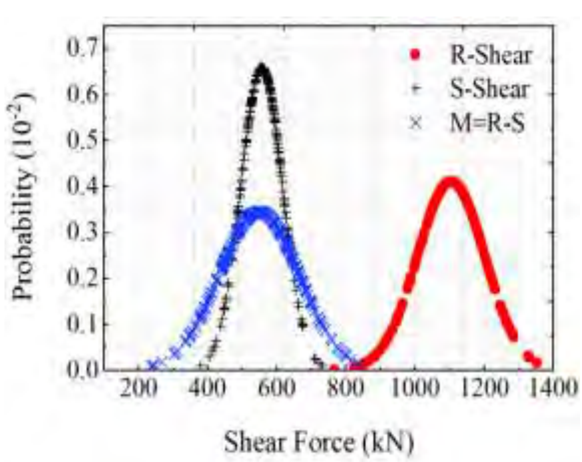

Figure10 Probabilistic distribution of R, S and M for Shear

3.3. Verification by Field Test

To verify the results obtained through the design review and a nonlinear Finite Element simulation of the defective girder, field test was conducted to ultimately assure the safety of the bridge. The verification included strength and durability aspects.

3.3.1. Field Test - Method

The field test was designed to assess the performance of the defective girder and solely targeted on the strength, stiffness, and geometry aspects. Test equipment (data Logger-for digital data acquisition-500 data points per second, 3 transducers, 3 dial gages, UPS-power storage, generator and so on) were mobilized.

3.3.2. Truck Loading Test

The truck loading test was mainly aimed to assess the strength and stiffness of the defective girder under moving load action of the loaded truck. The truck weighed 60.6 tons (Figure 11) and was made to pass on the bridge with its central axis aligned with the defective bridge. Three different speeds (5km/hr, 20km/hr and 40km/hr) were considered to assess any potential change in the response of the defective girder due to speed or impact. The load arrangement of the truck used in the load test is shown in Figure 12. The axle loads are given in tons and the axle spacings are given in meter.

For each loading case, measurements of deflection of the defective girder with its counterpart non-defective girder were recorded. Furthermore, the truck was made to pass with its edge wheel positioned at a distance of forty percent of girder spacing (=0.92m) from the curb [8, 9] and similar records were made as well. In addition to the deflection measurements, any possible formation, opening/closure of flexural and shear cracks were observed.

Figure 11 Truck used for loading test

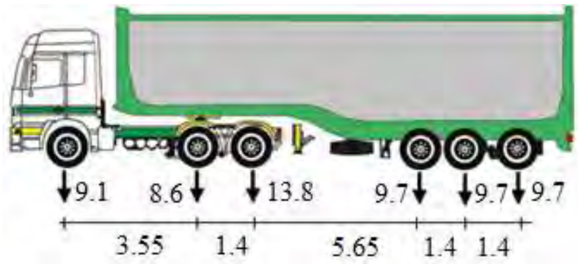

Figure12 Axle load and spacing

The recorded mid-span deflection value while the truck was passing with its central axis coinciding with the alignment of the defective girder is shown in Figure 13. Mid-span deflection under moving load passages at 5km/hr; 20km/hr; 40km/hr; edge position with 5km/hr (non-sustained and sustained loads) is shown in Figure 14.

The three initial curves indicate the response for the running speeds of 5km/hr, 20km/hr and 40km/hr, respectively. The fourth curve indicates the response for the girder deflection with the trucks edge wheel positioned at 0.92m from the curb and at a running speed of 5km/hr. However, during the return of the third set of curves, the truck was allowed to stop when maximum mid span deflection was observed. This step was intentionally made in order to observe the effect of sustained load. The truck was sustained at this fixed position for nearly 60 seconds.

Figure 13 Set-up for truck loading test

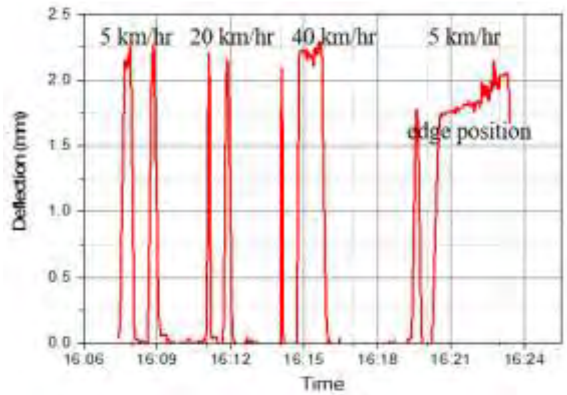

Figure 14 Mid-span deflection under moving load passages

As can be observed from the mid-span deflection of the defective girder (Figure 14), the maximum deflection was 2.3mm. For comparison, mid span deflection of the non-defective adjacent interior girder was simultaneously measured and the maximum mid-span deflection recorded was 1.9 to 2.0 mm. The last loading passage was made towards the edge of the curb and the corresponding deflection of the defective girder was observed to be 1.8mm. In addition, the truck was made to stop at the center and sustained for 60secs, resulting in a deflection of 2.2mm.

Overall, the defective girder is safe against stiffness and strength limits, with a maximum deflection of 2.3mm under the 60.6tons truck load, compared to the limit of 25.75mm. The actual recorded mid-span deflection was one-eleventh of the mid-span deflection limit, making the girder reasonably safe against serviceability and strength requirements.

3.4. Correlating Numerical Values with Field Load Testing

In some cases, prediction of load carrying capacity of bridges using conventional analytical load rating procedures possesses a lesser value than the load test result and depends on many factors. In line with this, uncertainty of analytical results depends on estimation of material properties, load distribution and impact factors, etc. Hence, conducting field test on bridges and correlating the result with the analytical value is necessary [7].

According to recent researches, when performance evaluation of bridges is determined based on field load test, the bridge load rating through field load test results can be estimated following AASHTO specification (analytical rating factor given inEq. (4)) and an adjustment factor is used to modify the rating factor [13]. Equation (5) provides an adjustment factor based on field test results [13].

$$RF_T = RF \times K \tag{5}$$

where:

RF_T is load rating factor based on field test
RF is rating factor from Eq. (4)
K is an adjustment factor (without a load test, K=l. If the load test results agree with the analytical value, then K=1)

The adjustment factors were calculated using Eqs. (6) and (7) [13]:

$$K = 1 + K_a \times K_b \tag{6}$$

$$K_a = \frac{\varepsilon_c}{\varepsilon_T} - 1 \tag{7}$$

where:

ε_T is maximum member strain measured during load test

ε_C is the corresponding theoretical strain due to the test vehicle and its position on the bridge

K_a is accounts for both the benefit derived from the load test

K_b accounts for the relationship between load test results and theoretical predictions

Equation (8) below gives the K_b factor:

$$K_b = K_{b1} \times K_{b2} \times K_{b3} \tag{8}$$

where:

K_{b1} ,K_{b2}, K_{b3} account for the type and frequency of follow-up inspections, the presence of special features like non-redundant framing and fatigue-prone details.

During the field load test, as strain gauges were not attached to the girder, the K_a factor was calculated based on deflections of the defective girder obtained by the test vehicle (T) and the rating vehicle (W). According to elastic analysis, the deflection of the defective girder caused by the rating vehicle (W) was computed as 6.81mm (in this case, gross moment of inertia of a section, I=294.2×10^9mm^{4}was used) and the maximum deflection of the bridge due to the test load was taken as 2.3mm (Figure 14). Using Eq. (7), the value of K_a was calculated to be 1.96.

K_b values were obtained from tables provided in [13] and read as K_{b1}=1.0 (for T/W>0.7), K_{b2}= 0.9 (routine inspection between 1 to 2 years) and K_{b3}= 0.7 (fatigue control without redundancy). Hence, the value of K_b became 0.63 (=1.0×0.9×0.7). Upon substitution, the value of$K$ became 1.24 (=1.96×0.63).

Thus, for the legal load, the modified rating factor, RF_T, for the bridge under consideration became 3.26 (=2.64×1.24). The modified rating factor of the bridge based on field test is in good agreement with the probabilistic method (RF=3.71). This shows that the bridge with defective girder is safe.

4 CONCLUSIONS

The load carrying capacity of the defective girder was evaluated through various approaches including design check, non-linear analysis, strength evaluation through legal load and truck load test. The deterministic approach was found as a conservative method of verification. The field test revealed that the bridge under investigation was safe against the test load with no damage or risk of collapse.

The results of the analysis indicate that the defective girder was safe against flexure as well as shear even if the compressive strength of concrete was reduced. The numerical evaluation of the bridge was also verified through load test and the modified rating factor was in good agreement with the probabilistic assessment. The safety index computed for the defective bridge satisfies the requirement for new bridges.

CONFLICT OF INTEREST

The authors declare that there is no conflict of interest.

ACKNOWLEDGMENTS

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