RELIABILITY ANALYSIS FOR A 100-YEAR-OLD REINFORCED CONCRETE FRAMED BRIDGE

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SUMMARY

Existing bridges, all over the world, carry gradually increasing in weight and number vehicular traffic. The objective of this study is to determine reliability index of a 100 year old reinforced concrete framed bridge. Geometric data about the structure was obtained with usage of destructive and non-destructive methods. Material data was collected from field tests and available literature on evaluation of existing structures. The most harmful load configuration was established in a recent study on weigh-in-motion data for the state of Alabama. Using finite element numerical method, a three dimensional model of the bridge was developed. The statistical parameters of resistance were obtained using Rosenblueth 2k+1 method. The reliability analysis was demonstrated on an example of one of the spans.

Keywords: *Evaluation of existing structure, concrete slab, flat slab bridge, reliability analysis, finite element modelling, arching action, weigh-in-motion.*

1. INTRODUCTION

The road infrastructure is exposed to an increasing number of vehicles and heavier loads. Existing bridges often carry trucks that are significantly heavier than the original design loads. There is not enough money to strengthen or replace deficient structures. To save limited resources, there is a need for accurate evaluation of the bridges, to determine what is the actual load carrying capacity or resistance. Knowledge of the resistance as well as the predicted maximum expected loads, can serve as a basis for important decision about prioritization for repair or replacement. Therefore, the state departments of transportation that are responsible for maintenance of roads and bridges, can benefit from having efficient bridge evaluation procedures. The objective of the present study sponsored by the Alabama Department of Transportation (ALDOT), is to develop an approach for evaluation of a reinforced concrete rigid frame bridge without any prior technical documentation.

2. CONSIDERED STRUCTURE

The considered structure is an 11-span flat slab reinforced concrete bridge, with no existing technical drawings nor other details that can be used to perform a load rating. The bridge was constructed between 1914 and 1916, and ALDOT's "Bridge Card" showed that it was widened by approximately 1.20m (4”) in 1930. Visual inspection of the bridge indicates that the bridge was widened twice.

Fig. 1. *Side view of the bridge.*

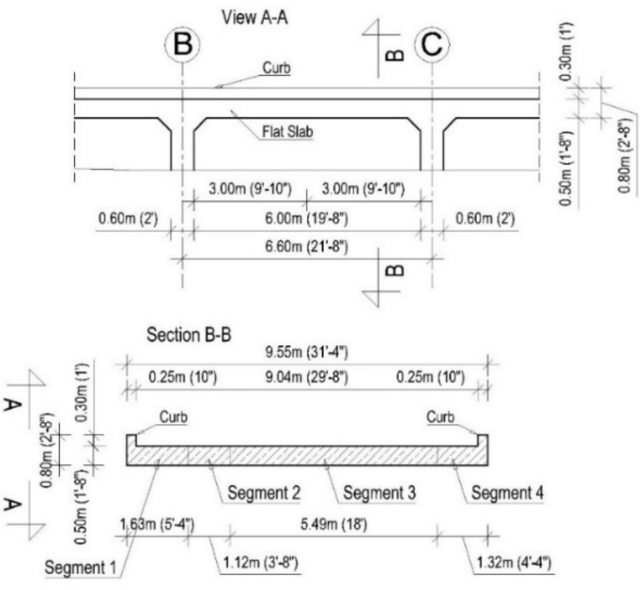

Fig. 2*. Detailed drawing of the bridge.*

Currently the bridge carries unrestricted traffic. This is allowed by the AASHTO *Manual for Bridge Evaluation* [1] in cases where a reinforced concrete bridge of unknown details has carried unrestricted traffic without developing signs of distress. However, because the structural details of the bridge are unknown, ALDOT has problems with issuing permits to overweight, non-standard trucks as it requires analytical justification.

In order to determine some of the structural parameters, the bridge was inspected and measured using field testing instruments, involving a series of destructive and non-destructive tests described in the following parts of this paper.

All 11 spans are equal and the center-to-center span length is 6.65m (21'-10"), while the total width is 9.53m (31'-4"). Pier wall thickness is 0.61m (2'). Total width for each span of the bridge consists of four segments: the original one and three additions. The width of the oldest segment (segment 3) is 5.49m (18'). First, the bridge was widened by 1.12m (3'-8") on the East side (segment 2). Then it was widened on both sides at the same time, by 1.63m (5'-4") on the East side (segment 1) and 1.32m (4'-4") on the West side (segment 4).

3. MATERIALS

The location and size of existing reinforcement of the bridge was investigated using advanced sensing/detecting devices. Bottom surface of the bridge was scanned with Proceq Profometer PM-630, an instrument using electromagnetic pulse induction technology. The Profometer precisely detected rebars location and measured their diameters and cover thickness. A number of scans showed an identical rebar distribution for all the spans. All the transverse line-scan readings where thoroughly processed and analysed, confirming that the bottom longitudinal reinforcement is extended into the supports in all the segments. Summary of the bottom reinforcement is presented in Table **1**.

Table 1. *Details of the bottom reinforcing bars.*

Segment No.	Rebar size [mm/US size]	Cover [mm/in]	Number of rebars in segment
1	Φ25 / #8	32.0 / 1.25	10
2	Φ22 / #7	32.0 / 1.25	9
3	Φ25 / #8	32.0 / 1.25	53
4	Φ25 / #8	32.0 / 1.25	7

The AASHTO Manual [1] specifies yield strength for reinforcing bars by considering the date of construction. For unknown steel constructed prior to 1954, the yield strength f_y is given as 227 MPa (33 ksi).

Top surface of the bridge is a 0.05m (2") layer of asphalt, and it was investigated using the Ground Penetrating Radar (GPR). The GPR provided information on the top reinforcement and detected transverse discontinuities between spans. Therefore, simple support conditions were assumed and top reinforcement was neglected entirely.

Three different concrete mixes were used in the bridge. Due to restriction on number of cores, three concrete samples were taken. One core was drilled in segment 3 (Fig. 2), over the support, in the oldest concrete. Additional two cores were taken from segment 1 with the newest concrete, at over support and mid-span locations.

Table 2. *Compressive test results for concrete cores.*

Segment No.	Drilling location	Compressive strength [MPa/psi]	Mean compressive strength for analysis [MPa/psi]
1	Over support	13.36 / 1937	12.76 / 1850
1	Midspan	12.13 / 1760	
3	Over support	23.03 / 3340	17.24 / 2500

Concrete cylinder compressive strength values obtained in ALDOT's material laboratory are presented in Table 2. For the newest concrete, the mean compressive concrete core strength is an average of two values. For the oldest concrete, AASHTO Manual [1] recommends a minimum compressive strength value of 17.2 MPa (2500psi) - for superstructure components constructed prior to 1959. Therefore, conservatively, this value was taken as strength of concrete in segment 3. It is assumed that the value of compressive strength for segment 1 applies to segments 2 and 4 and it is referred to with subscript 'NC' (new concrete). For the oldest concrete, a subscript 'OC' is used in further notation.

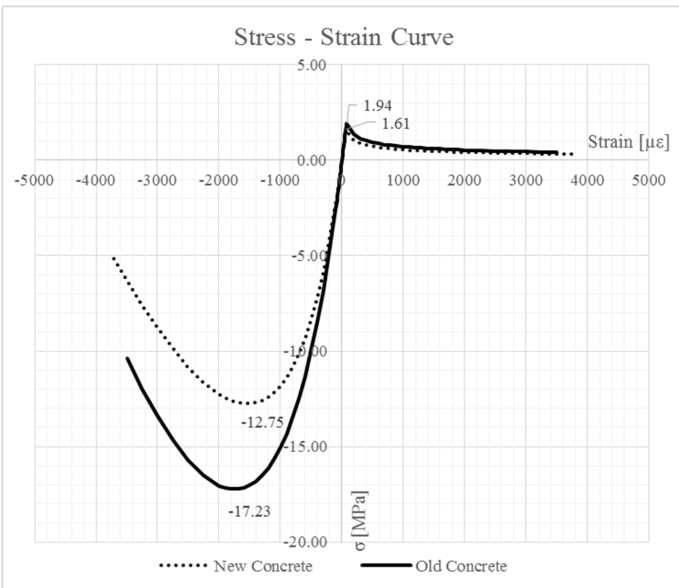

Fig. 3. *Stress-Strain curve for New and Old concrete (1MPa=145psi).*

In order to build a proper material model, the obtained mean values of compressive strength were fitted into typical stress-strain curve using approximate equations. The compressive stress-strain relationship was obtained using formulas specified in EC-2 [2]. The tensile strength of concrete was calculated using EC-2 formulas [2], while the stress-strain curve was developed using a modified Wang & Hsu formula [3]. Figure 3 shows the resulting curves for both concretes considered.

4. FINITE ELEMENT MODEL

A three dimensional Finite Element (FE) model was developed in Abaqus CAE 6.14 Software. Concrete elements (curbs, slab segments, piers) were modelled with 8-noded linear brick elements with reduced integration (C3D8R). The element type used for reinforcing bars is a 2-node linear beam element (B31). The mesh study showed that the most effective mesh size, in terms of accuracy and computing time, is 0.10x0.10x0.125m (4"x4"x5") for the brick elements and 0.10m (4") length for the beam elements. Static wheel loads on the bridge are modelled as flat rigid load transferring plates with a uniform load applied.

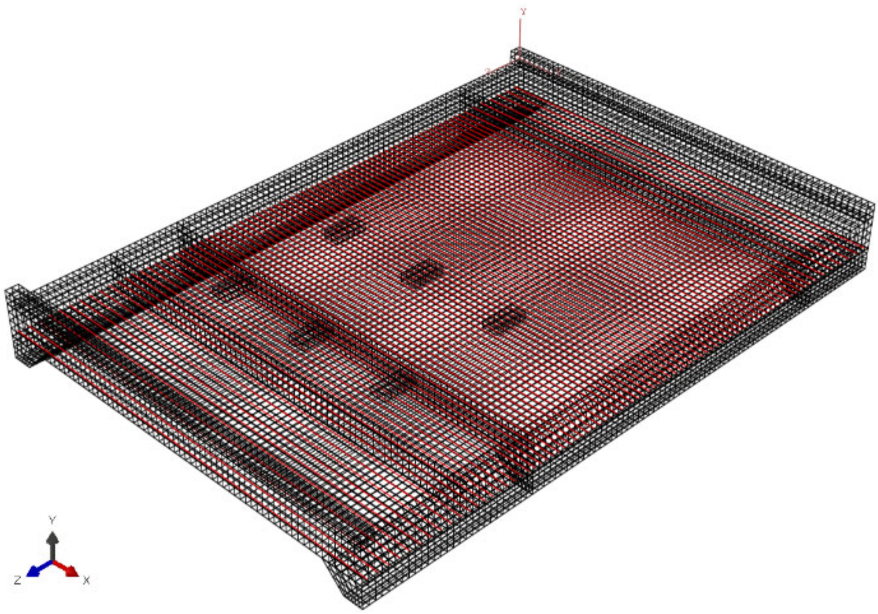

Fig. 4. *3D Finite Element Model of one span of the bridge.*

These load transferring elements, imitating tires contact surfaces, have dimensions recommended by AASHTO of 0.25x0.50m (10"x20").

The most conservative support conditions are pin supports at both ends of the span. Such support conditions are achieved by restraining the displacements in all directions for both bottom XZ surfaces as well as for both, back and front, YZ surfaces of the piers. For nodes on these surfaces the rotations are allowed. Allowed displacement *dy* at front and back YZ surfaces of the slab immitates the discontinuity of the concrete slab due to transverse cracks detected over the supports.

Contact conditions specified in the model are as follows: full bond of reinforcing bars with concrete in all segments, full connection to the side surfaces of the adjacent segments, pressure transfer interaction between tire footprint elements and concrete segments.

The material models used are the concrete damaged plasticity model for concrete, and elasto-plastic model for steel.

5. LOAD MODEL

For the probabilistic analysis of the bridge, a weigh-in-motion (WIM) data had to be processed. WIM data provided by ALDOT was collected at twelve measuring stations across Alabama state between years 2006 and 2014. ALDOT's WIM data contained of regular legal traffic as well as permit vehicles. For the considered bridge, the most harmful are closely spaced heavy axles. It was determined that the maximum effect is produced by sets of three closely spaced axles (tridems). Therefore, a tridem set, with axle spacing of 1.47m (4'-10") and wheel-line spacing of 1.93m (6'-4"), was used as the load in the reliability analysis.

In the analysis, the load was applied as a set of static forces distributed over a tire contact area of 0.25x0.50m (10x20"). Two load configurations were considered – with a tridem in the right traffic lane and in the left traffic lane, at midspan. It is assumed that the probability of occurrence of the tridem in the right lane is 67%.

The total applied load was calculated using statistical data for axle loads, provided in [4]. The statistical parameters were taken for 75 year time period and Average Daily Truck Traffic (ADTT) of 1000. The bias factor λ, which is the ratio of mean to nominal value, was taken as 1.48. The coefficient of variation V, the ratio of standard deviation to mean value, used in the live load model is 0.12. For the AASHTO HL-93 load configuration, the single axle load is 111.2kN (25 kip). Multiplication of the HL-93 single axle load by the bias factor and the number of axles in the tridem, results in the total tridem load of 493.8kN (111kip). This value of the total load represents the maximum expected weight of the tridem to occur in 75 years.

6. RELIABILITY ANALYSIS

If Q is the total load and R is resistance, then the structure is safe as long as

$$Q \le R\,. \tag{1}$$

Probability of failure P_f and the corresponding reliability index β depend on the statistical parameters of Q and R. The reliability index β can be calculated from the following equation [5]:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{{\sigma_R}^2 + {\sigma_Q}^2}}\,, \tag{2}$$

where μ_Q - mean load, μ_R - mean resistance, σ_R - standard deviation of load and σ_R - standard deviation of resistance. The probability of failure, P_f, is related to the reliability index, β:

$$P_f = \Phi(-\beta), \tag{3}$$

where Φ is the standard normal distribution function.

The statistical parameters of resistance can be determined using Monte Carlo simulations. However, the non-linear analysis of the reinforced slab is quite complex therefore, instead of Monte Carlo, the Rosenblueth's 2k+1 point estimate method is applied [5]. If resistance can be described by a function of several random variables:

$$Y = f(X_1, X_2, ..., X_k), \tag{4}$$

where X_i are random variables representing resistance parameters such as strength of materials, modulus of elasticity and so on. Statistical parameters of the resistance can be estimated from *2k+1* resistance calculations. The first step is to establish the value of resistance y_0, which is a value of eq. (4) when all input variables are equal to their mean values. For additional *2k* points, the resistance is evaluated for each random variable X_i at two values of $\mu_{Xi}+\sigma_{Xi}$ and $\mu_{Xi}-\sigma_{Xi}$, while all other variables are assumed to be equal to their mean values. Using mathematical notation,

$$y_i^+ = f\left(\mu_{X_1}, \mu_{X_2}, ..., \mu_{Xi} + \sigma_{X_i}, ..., \mu_{X_k}\right), \tag{5}$$

$$y_i^- = f\left(\mu_{X_1}, \mu_{X_2}, ..., \mu_{Xi} - \sigma_{X_i}, ..., \mu_{X_k}\right). \tag{6}$$

For each random variable, the following parameters are calculated based on y_i^+ and y_i^-:

$$\mu_{y_i} = 0.5(y_i^+ + y_i^-), \tag{7}$$

$$V_{y_i} = \frac{y_i^+ - y_i^-}{y_i^+ + y_i^-}. \tag{8}$$

The mean value and coefficient of variation of the resistance Y are then calculated as follows:

$$\mu_Y = y_0 \prod_{i=1}^{k} \left(\frac{\mu_{y_i}}{y_0}\right) \tag{9}$$

$$V_Y = \sqrt{\left[\prod_{i=1}^{k} \left(1 + V_{y_i}^2\right)\right] - 1} \tag{10}$$

7. RESISTANCE MODEL

The load carrying capacity of the bridge, or resistance *R,* is expressed in terms of the total load of a tridem *P*, that results in the concrete strain in compression of 0.003 [6]. It is assumed that the total load of a tridem *P*, is a function of three variables: effective depth *d* and compressive strengths for both concretes $f_{c.NC}$,$f_{c.OC}$.

Table 3. *Statistical parameters of resistance function's variables.*

Variable	Mean value, μ	Coefficient of variation, V	Standard deviation, σ
Effective depth, d	47.0cm (18.5")	0.034	1.6cm (0.63")
Compressive strength of New Concrete, $f_{c.NC}$	12.76MPa (1850psi)	0.180	2.30MPa (333psi)
Compressive strength of Old Concrete, $f_{c.OC}$	17.24MPa (2500psi)	0.180	3.10MPa (450psi)

Statistical parameters of resistance are calculated using the Rosenblueth 2k+1 method, for which: the mean values of concrete compressive strength are listed in Table 2 and the mean value of effective depth *d* is calculated for rebar details specified in Table 1. Coefficients of variation of the resistance function's variables are taken from previous studies [7] and are presented in the table below.

8. RESULTS

Statistical parameters of resistance are obtained from the FE analysis results. The results for both load configurations are shown below:

Table 4. *FEA results. Load configuration: right travel lane.*

Iteration i	Total tridem load, P_0 [kN/kip]	Total tridem load, P_i^+ [kN/kip]	Total tridem load, P_i^- [kN/kip]
0	6716 / 1510	-	-
1	-	6550 / 1472	6677 / 1501
2	-	7209 / 1621	6151 / 1383
3	-	6996 / 1573	6653 / 1496

Table 5. *FEA results. Load configuration: left travel lane.*

Iteration i	Total tridem load, P_0 [kN/kip]	Total tridem load, P_i^+ [kN/kip]	Total tridem load, P_i^- [kN/kip]
0	6368 / 1432	-	-
1	-	6261 / 1408	6768 / 1521
2	-	6526 / 1467	6223 / 1399
3	-	6875 / 1546	5817 / 1308

Parameters for total load effect Q and resistance R are presented in Table 6.

Table 6. *Statistical parameters of total load effect Q and resistance R.*

Load Configuration	μ_Q [kN/kip]	V_Q	σ_Q [kN/kip]	μ_R [kN/kip]	V_R	σ_R [kN/kip]
Right Lane	494 / 111	0.12	59.3 / 13.3	6453 / 1451	0.083	536.9 / 120.7
Left Lane	494 / 111	0.12	59.3 / 13.3	6498 / 1461	0.095	617.9 / 138.9

The reliability indices and corresponding probability of failure are presented in the table below:

Table 7. *Reliability analysis results.*

Load Configuration	Reliability index, β_i	Final reliability index of the bridge, β	Probability of failure of the bridge, P_f
Right Travel Lane	11.0	10. 6	1.73E-26
Left Travel Lane	9.7		

9. CONCLUSIONS

The failure criteria considered in this paper is compressive strain of concrete. The obtained reliability index is very high due to arching action.

For some of the unknown parameters, conservative assumptions were made. Therefore resulting reliability indices are also representing conservative values.

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