Journal of Engineering and Applied Sciences 5 (1): 4-13, 2010
ISSN: 1816-949X

Probabilistic Evaluation of Aluminum Bridge Decks Design Criteria on Steel Girders

A.O. Agboola, O.S. Abejide and A.T. Olowosulu
Department of Civil Engineering, Ahmadu Bello University, Zaria, Nigeria

Abstract: The probabilistic evaluation of aluminum decks in flexure between girders, using AASHTO LRFD specification is presented. The results show that the evaluated deck is in conformity with AASHTO LRFD specification as regards the target safety value for bridges of 3.5 for load resistance of 1.0, since its safety value is higher than the code prescription that is 4.44 for LRF of 1.0 and 3.39 for LRF of 1.1. These results indicate that aluminum decks as suggested by Alumadeck™ met the requirement for bridge decks and have the ability to carry more live loads than conventional reinforced concrete decks. Also, a comparative study of safety margins of composite action between aluminium decks on steel girders and concrete decks on steel girders gave results that show that Alumadeck™ as proposed is more superior from the view point of strength and serviceability. This result of Alumadeck™ to conventional concrete decks gave a safety index value of 17.33 for the full composite action while that of concrete is only 11.83.

Key words: Structural safety indices, aluminum deck systems, composite concrete bridges, Alumadeck™, Nigeria

INTRODUCTION

The majority of present day bridge decks are in questionable conditions (Hadipriono, 1985). The conventional highway bridge decks systems deteriorate over time as a result of environmental or climatic factors such as changes in temperature, moisture content fluctuation and freeze-thaw cycle (Dobmeier *et al*., 2001). These activities are results of what we see in damaged concrete decks today. Further more, ageing, inadequate maintenance, increase in load spectral and environmental contamination has contributed to this present problem of deterioration (Konig and Nowak, 1999). These damages or deficiency are often corrected by limiting the live carry capacity of the bridge (Nowak and Collins, 2000).

Bridge decks required major repair or replacement every 15-20 years while the substructure and the superstructure tend to last 40 year or more (Wesley and John, 2006).

Eventually, some day the bridge deck will need to be replaced. This has spirited the search for alternative decks that can resist environmental factors without any protective coating (Davis, 1993), reduce highway closure time and make retrofitting that meet the current design specifications possible while retaining the super structure and sub-structure.

The suggestion made by Reynolds Aluminium Company has given way to a feasible alternative to conventional reinforce concrete decks (Dobmeier *et al*., 2001). This deck is new and therefore requires a close study of its characteristics structural behavior and compatibility to the current codes of practice for the design of bridges.

The loads on a bridge at any time depend on many factors, such as the number of vehicles on the bridge, weight of vehicles and their approach speeds. Neither the fact that the details of the number of vehicles passing over a bridge may not be ascertained nor the number of vehicles on the bridge at any time space means that there are some uncertainties about the loads, total loads on the bridge and the bridge resistances (Nowak and Collins, 2000).

But engineering structures should be designed on the basis of relevant codes, which should be satisfactory by engineering judgment and previous experience (Surahman and Rojiani, 1983), rather than understanding the uncertainties that influence the strength and loads on the structures (Rosowsky *et al*., 2002). The traditional component-based approach often does not allow revealing the actual load capacity (Nowak, 2004). Accumulation of research in the field of bridge evaluation as indicated the justification of using reliability index for the measure of safety (Ferhat and Dan, 2004). The traditional component-based approach often does not reveal the exact safety margin of the load carrying capacity of the structure. Therefore reliability methods can be used as an important tool in the evaluation of existing or proposed structures (Melchers, 1987).

New advances have taken place in bridge evaluations in the area of probabilistic methods (Nowak, 2004). In

Corresponding Author: A.O. Agboola, Department of Civil Engineering, Ahmadu Bello University, Zaria, Nigeria

reliability analysis, failures in structural components are usually estimated based on particular performance criterion. In recent times, reliability methods have been derived for example as in the first and second order reliability methods to solve structural problems using computer algorithms. These advance methods provide accurate results (Rosowsky *et al*., 2002; Nowak, 2004).

In this presentation the proposed aluminum bridge deck is evaluated for systems I and II only. Systems I stresses represent the longitudinal bending of the composite aluminium deck and steel girder. System II stresses are represented by the transverse bending of the top deck flanges due to wheel loads. Other systems of stresses are not considered due to continuity across the deck splices (Dobmeier *et al*., 2001).

The Load Resistance Factor (LRFD) design specification is employed herein for the evaluation of aluminum decks on steel girders. The computer algorithm employed is the First Order Reliability Methods (FORM) as proposed (Augusti *et al*., 1984; Melchers, 1987; Ditlevsen and Madsen, 2005) (Fig. 1). A comparison of the full composite action on bridge decks is used as a basis of

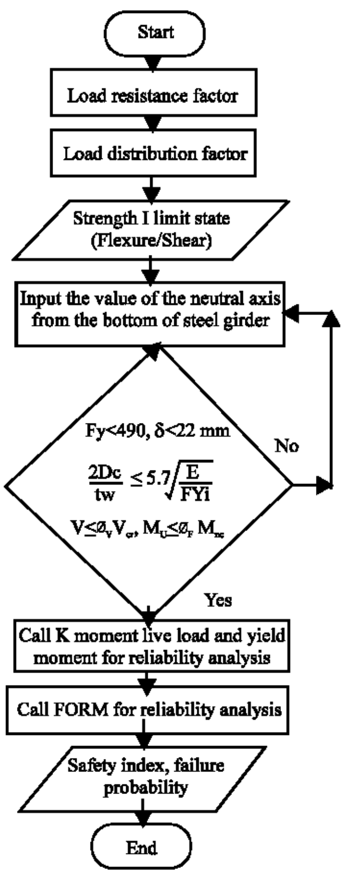

Fig. 1: Flow chart for reliability analysis using the computer algorithm in FORTRAN

comparison between types of decks in order to assess the structural safety of the aluminum deck systems. The scope involves the application of the algorithm for reliability calculations of two voided 6063T6 extrusion of aluminum decks at their design strengths using LRFD method as the limit state function. The result obtained broadens the horizon on the durability and use of aluminum decks as a better alternative to reinforced concrete bridge decks.

MATERIALS AND METHODS

Limit states: The developed theoretical work of reliability and safety checking of structural members has been presented in many textbooks (Ang and Tang, 1975; Melchers, 1987; Nowak and Collins, 2000). Reliability analysis can be performed using iterative procedures (Nowak and Collins, 2000). Limit states are boundaries between the desired and undesired performance of a structure. A bridge structure is said to be functionally obsolete when its resistances are below the load effects. The modes of failure in bridges can be in any of the following ways; such as cracking, corrosion, excessive deformations, exceeding ultimate moment capacity for shear or bending moment, local buckling and so on.

Briefly, it is clear (AASHTO, 2004) that there are four limit states applicable to the design of I-girders. Strength or Ultimate Limit States (ULS) are mostly related to the moment carrying capacity, shear capacity and overall stability. Serviceability Limit State (SLS) has to do with satisfactory performance and user comfort. For steel members, the objectives are intended to be satisfied by limiting the maximum level of stress permissible. Fatigue and Fracture Limit State (FLS) is concerned with crack growth on repeated loading. This is accommodated by limiting the allowable stress range to which the girders are subjected. Extreme Event Limit State (ELT) has to do with accidental collisions, earthquakes or floods. These are only considered on special occasions. This study is focused on the ULS of the bridge deck since it is the criterion that is most critical in decks.

Reliability index: The concept of a 'limit state' is often used to help define failure in the context of structural reliability analyses. Melchers (1987) defines the concept of limit state as a formalization of the criteria under which the structure can be considered to have failed. For example, a girder fails if the load exceeds its resistance. Now, let the variable affecting the strength be denoted by R where R is a function of several other stochastic variables such as the modulus of Elasticity E, area of the cross-section, A or strength in bending f_b and the load effect be designated by R which arises from a variety of loads such as wind loading, traffic loading, self-weight etc. A limit state can be represented mathematically as:

$$g(R, S) = R\text{-}S \tag{1}$$

The limit state corresponds to the boundary between the desired and undesired performance for which the limit state equation g = 0. So that when $g \geq 0$, the structure is safe (desired performance) and when $g \leq 0$, the structure is not safe (undesired performance) and the limit state is violated. The probability of failure (P_f) of the structural element can be expressed in terms of the performance function as:

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

Thus for a simple strength-load (R-S), system the failure probability can be represented as:

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} F_R(x) f_S(x)\,dx \tag{3}$$

If the F ( ) denotes the Cumulative Density Function (CDF) and f ( ), the Probability Density Function (PDF), then the safety margin can be represented as:

$$G = R\text{-}S \tag{4}$$

The mean value of G is given by:

$$\mu_G = E(G) = E(R) - E(S) = \mu_R - \mu_S \tag{5}$$

while the variance if R and S are independent variables is defined by Eq. 6:

$$\sigma_D^2 = D(G) = \sigma_R^2 + \sigma_S^2 \tag{6}$$

The safety index, β is the number of deviations the mean value of G is from the failure surface which implies the point where G = 0. The design point is the point where the design load effect, S and the material strength, R are equal and produces the required value of safety index, β. This Cornell (1969) safety index, β_C is given by:

$$\beta_C = \frac{E(G)}{D(G)} = \frac{\mu_G}{\sigma_G} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_S^2 + \sigma_R^2}} \tag{7}$$

Also if R and S are normally distributed, it is possible to relate the safety index to the probability. Then an estimate of the failure probability is obtained as:

$$P_f = \Phi(-\beta) \tag{8}$$

Where:

Φ = The cumulative Gaussian distribution of the standard normal law

β = The reliability index according to Hasofer and Lind (1974). The precision of the approximation depends on the non-linearity of the failure surface

However, according to Thoft-Christensen and Baker (1982), when R and S are jointly normally distributed with correlation coefficient ρ, Eq. 7 still holds but σ_G in this case will be given as:

$$\sigma_G = \left(\sigma_R^2 + \sigma_S^2 + 2\rho\sigma_R\sigma_S\right)^{1/2} \tag{9}$$

But if R and S are both log-normally distributed, then, P_f may be expressed as in Eq. 10, where, V represents the coefficient of variation.

$$P_f = \Phi\left[\frac{L_n\left[\frac{\mu_S}{\mu_R}\left[\sqrt{\frac{V_R^2+1}{V_S^2+1}}\right]\right]}{L_n\sqrt{\left[(V_R^2+1)(V_S^2+1)\right]}}\right] \tag{10}$$

When a limit state is not a hyper-plane, it is not possible to calculate the expected values and the variance of the safety margin, G, solely from the expected values and variance of the basic variables, X (Ditlevsen and Madsen, 2005). The resulting safety margin is arbitrary and affects the corresponding safety index. One of the methods to avoid this is to use transformation of the basic variables suggested by Hasofer and Lind (1974), so that we can use the design point, x^*, as the expansion point (Madsen *et al*.,1986; Nowak and Collins, 2000).

Iterative procedure (Nowak and Collins, 2000) adopts a linearization procedure of the limit states surface at the design point. If the checking point $y^{(1)}$ is poorly chosen, the condition of perpendicularity between the tangent (hyper) plane at $y^{(1)}$ and β direction will not be satisfied.

Therefore if y^m is the iteration of the mth approximation of the design point at g (y) = 0 from the origin, then $y^{(m+1)}$ is a better approximation of ym. Thus, the design point of ym in the y-space is given by:

$$y^{(m)} = -\alpha^{(m)}\beta^{(m)} \tag{11}$$

where, $\alpha^{(m)}$ is the sensitivity factor or the directional cosine at the iteration point and is given as:

$$\alpha^{(m)} = -\frac{g_Y{}^{m}}{(g_y{}^{mT} g_y{}^{m})^{1/2}} \tag{12}$$

$g_y^{(m)}$ in Eq. 12 denote the gradient vector that is

$$g_Y{}^{(m)} = \left[\frac{\partial g}{\partial y_1} y^{(m)} \ldots \frac{\partial g}{\partial y_n} y^{(m)}\right] \tag{13}$$

When n is the number of basic variables, a relationship between $y^{(m+1)}$ and $y^{(m)}$ can be obtained from first Taylor series expansion of $y^{(m+1)} = 0$ about $y^{(m)}$.

$$y^{(m+1)} = \alpha^{m}\left[\beta^{(m)} + \frac{g(y^{(m)})}{(g_y{}^{mT} g_y{}^{m})^{1/2}}\right] \tag{14}$$

This iteration procedure expressed in Eq. 14 is continued until convergence at a point of $y_i^{(m)}$ or β, then:

$$y_i^{(m)} = \alpha^{(m)} \beta \text{ and } y_i^{(m)} = 0 \tag{15}$$

The procedure for determining the safety indices and probability of failure has been programmed in FORM5 (Gollwittzer *et al.*, 1988) and calculations are carried out using the computer algorithm.

Bridge load model: The loads considered in this research are dead and live load component. The load model used is based on the AASHTO LRFD (2004) specifications. The basic statistical parameters considered are the bias factor, λ and coefficient of variation, V.

The dead load components used include factory-made member weight (girders), cast in place member (deck slab) and wearing course. The bias factor λ = 1.03 and coefficient of variation V = 0.08 were used for factory-made components while λ = 1.05 and V = 0.10 for cast-in-place components and λ= 1 and V = 0.25. The reliability analysis was considered for both aluminium decks and conventional reinforced concrete decks. The aluminum deck in this study is assumed to behave similar to reinforced concrete based on experimental evaluation conducted by Matteo *et al.* (1997).

Live load parameters are derived from AASHTO (2004) which is designated as HL-93. The models used considered various positions for both single and multiple lane loads to obtain maximum moment on the deck for transverse truck locations. Figure 2 shows the truck location, which is 0.15 m close to the center line.

In the case of the composite action of the aluminium deck and girder, the live load components used for the reliability analysis was determined using design truck

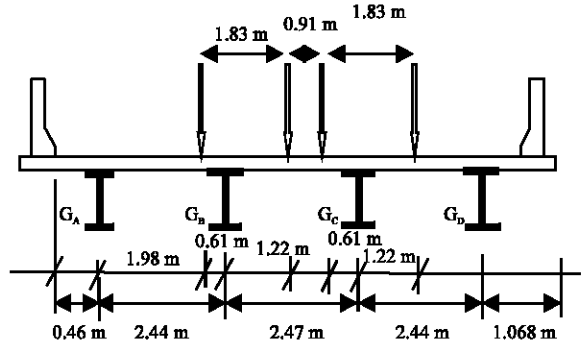

Fig. 2: Typical transverse section of bridge showing HL-93 loading location (AASHTO, 2004)

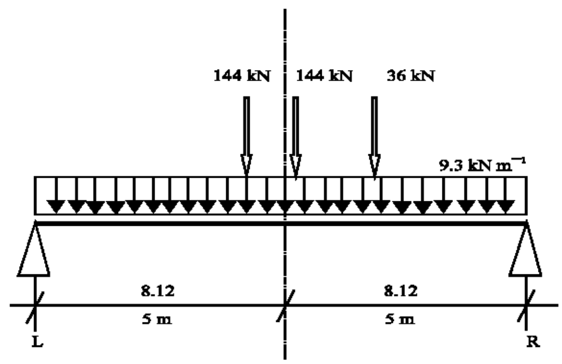

Fig. 3: Truck position for the maximum moment and lane load

occurring simultaneously with a uniformly distributed lane load. The maximum possible moment, M_i, occurred when the first rear axle is 0.055 m from the center of the girder span as the truck move from eastward on the bridge (Fig. 3). The moment obtained were multiplied with bias factors to obtain the expectation used in reliability analysis, as given in Eq. 16:

$$M_i = \lambda_i . S \tag{16}$$

Design model: The statistical parameters of load carrying capacity of the deck are based upon the components based approach. The resistance parameters are calculated by analytical methods. Also, the moments for dead load and live load components are multiplied by the statistical parameters of the component resistance that is the bias factor and coefficient of variation. The results are used in the AASHTO (2004) limit state equation as stated in Eq. 18.

$$\eta_D \eta_R \eta_I \sum \gamma_i S_i \leq \varphi R_n \leq R_r \tag{17}$$

Where:

η_D = Ductility factor
η_R = Redundancy factor
η_I = Operational importance factor
γ_i = Load factor
S_i = Force effect
φ = Resistance factor
R_n = Nominal resistance
R_r = Factored resistance

The load factors from the specifications of AASHTO (2004) per Article 3.4.1 are γ_{DC} (1.25), γ_{DW} (1.50) and γ_{LL} (1.75). The results of the three factors to a load modifier, η are limited to a range between 0.95 and 1.00. This general limit state equation is simplified as:

$$\eta\sum_{i=1}^{n}\gamma_i S_i \le \varphi R_n \quad i = 1, 2, 3, 4.......n \tag{18}$$

BS 5400 loading: The proprietary aluminum deck was equally studied in accordance with the load specification of BS 5400: Part 2, 1978. The reliability analysis of the bridge for combination 1 is stated in clause 4.4.1 of BS 5400. This includes the permanent loads together with the appropriate primary live loads.

Load component: The dead load includes the self weights of all permanent structural members of the bridge system. These loads are factored irrespective of whether these parts have adverse or relieving effect, as 1.1 for steel at ultimate limit state and 1.0 at serviceability limit state. While that of concrete is taken as 1.2 at ultimate limit state and 1.0 at serviceability limit state.

Superimposed dead load: The superimpose dead load includes weight of filling imposed upon the deck of the bridge such as wearing surface made of asphalt. Clause 5.2.2 of BS 5400: part 2, 1978 specified the factor to be applied to all parts of superimposed dead load is 1.75 for ultimate limit sate. Other load combination will not be considered in this study.

Live load: Highway bridge live loads are designated as HA and HB loading in BS 5400. The HA represent normal traffic in Great Britain. The BS 5400 considered loading of a closely spaced vehicle of 24 t laden weight in each of the two traffic lane for load length up to 30 m.

Primary live load: This is the static live loads due to the mass of traffic on the highway.

Secondary live load: These are live loads due to change in speed or direction of vehicle traffic. e.g., lurching, nosing, centrifugal, longitudinal, skidding and collision loads.

The load considered in this study is the primary live load. The bridge under consideration is one lane loaded in accordance with clause 3.2.9.3 of BS 5400 which limits carriageway width <4.6 m as one lane loaded. This obviously governs the design.

Impact factor considered for vehicles on highway bridge in BS 5400 is 25% on one axle or pair of adjacent wheel has been incorporated in the HA loading. The impact factor considered for minimum number of unit of type HB loading is usually 25 but this number may be increased up to 45 if so directed by the appropriate authority (BS 5400).

The limit state equation is represented as:

$$1.2(DL_{conc.}) + 1.75(DL_{WC}) + 1.1(DL_{SG}) + 1.5(LL) \tag{19}$$

$$1.2(DL_{conc.}) + 1.75(DL_{WC}) + 1.1(DL_{SG}) + 1.3(LL) \tag{20}$$

Equation 19 above represent the limit state equation for HA loading this is used for normal design consideration. Equation 20 represents a limit state equation for HB loading. The reliability indices for HA and HB loading are discussed later.

Type HA loading is taken as 30 kN per linear metre of notional lane for loaded length up to 30 m. For load length in excess of 30 m, Eq. 22 is used to derive the required load on the bridge (BS 5400, 1978).

$$W = 151\left(\tfrac{1}{L}\right)^{0.475} \text{ but not} > 9 \tag{21}$$

Where:

L = The loaded length (in m)
W = The load per metre of lane (in kN)

The distribution of the UDL and KEL shall be taken to occupy one notional lane, uniformly distributed over the full width of the lane.

Load application: The highway bridge is design for HA loading and checked using HB loading. The live load moment considered in this study was obtained from Table 1 (Raina, 2003). Since the bridge is simply supported, the value in Table 1 is used for the purpose of comparison between the ASHTTO and BS 5400. The quantitative comparison by Raina (2003) for road live load from various countries was presented on the follow bases:

Table 1: Statistical parameter of load and resistance factor of aluminium deck and reinforced concrete deck on steel girder

Material type	Properties	Distribution type	Covariance(V)
Aluminium*	Yield limit state	Normal	0.08
	Ultimate	Normal	0.08
High steel plate**	N/A	Normal	0.08

*Nowak and Collins, 2000; **Melchers, 1987

- The maximum bending moment and shear force that would be caused by live load in simply supported spans. Since simple span were more common and are also indicative of what is likely to happen in other type of construction
- As the impact allowance defer from one country to another, it is added to the calculated value of bending moment and shear force

The bending moments present in Table 2 below was extracted from Table 1 and of Raina (2003). The value obtained is based on simple span; the value was therefore adopted for the Little Buffalo Creek Bridge under consideration. Also, value used in reliability analysis was based upon the value obtained by interpolation for a span of 16.25 m, which is the span of the bridge under consideration. The results obtained are 1546 kN-m for HA loading and 3912 kN-m for HB loading.

Resistance model: The probabilistic distributions of the basic variables in Table 3 are obtained from the limit state equation as given earlier (Nowak, 2004). These are based on summary statistics available, while the assumption is made that the data upon which these statistics are based, perfectly suit the distribution and statistics parameter presented in the Table 1.

The yield moment, that is the moment which causes the first yield is computed using Eq. 22. This computation method for the yield moment recognizes that different stages of loading (e.g., composite dead loads, non-composite dead loads and live loads) act on the girder when various girder section properties are applied. The yield moment is determined by solving for M_{AD}, where M_{D1}, M_{D2} and M_{AD} are the factored moments applied to non-composite, long-term composite and short-term composite sections, respectively.

$$F_{yt} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \quad (22)$$

The standard deviation can be obtained from Eq. 23 given as:

$$SX = (COV).(EX) \quad (23)$$

Table 2: Comparison of Bending Moment between AASHTO and BS loading

*Span	**HS 20-44 loading of AASHTO@	BS loadings Type HA	Type HB
5	231	243	756
10	573	694	1863
15	1073	1336	3331
20	1552	2175	5654
25	2022	3156	7862

*Simply supported span versus maximum bending moment for one lane; **Maximum bending moment for one lane, including impact allowance (kN-m). @ Based on standard HS truck loading. Otherwise Standard lane loading governs

Table 3: Probabilistic model for properties of aluminum and steel plates

Random variable	Distribution function	Bias factor λ_i	Coefficient of variation COV
Factory-made member load	Normal	1.03	0.08
Cast in place member load	Normal	1.05	0.10
Wearing course load	Normal	1.00	0.25
live load	Normal	N/A	0.18
Resistance R_r	Lognormal	1.12	0.10

RESULTS AND DISCUSSION

Results of reliability analysis

Safety indices of system II: The safety of the aluminum decks on steel girders was evaluated at various Load Resistance Factors (LRF) from 1.0-1.5. It can be observed that the safety or reliability index of 4.4 was far above the safe limit for the aluminum deck systems at a LRF of 1.0. This obviously exceeds the specifications of AASHTO (2004) safety index rating of 3.5 at a specified LRF of 1.0. Also from the analysis, it was observed that as the LRF increases, there is a decrease in the safety of the aluminum decks and giving a safety index of 3.5 at a LRF 1.08. As the value of the load resistance factor increases, the safety of the aluminum decks is jeopardized as indicated by the decreasing values of the safety indices and increase in the probability of failure. However, the AASHTO (2004) specification is that the safety index for any bridge deck should not be <3.5 at a Load Resistance Factor (LRF) of 1.0. Now, since the safety index evaluation of the aluminum deck system is 4.4 at LRF of 1.0 and 3.5 at LRF of 1.08, there is at least 8% increase in the load carrying capacity of the aluminum decks as proposed for highway bridges, when the proprietary aluminum deck is in flexure between the steel girders. The safety indices as obtained in the results for aluminum decks are shown in Fig. 4.

Safety indices of system I: The safety of composite action of the aluminum deck on steel girder sections were also evaluated at various load resistance factors ranging from 1.0-1.5. This range was used to model the suitable safety levels between the proprietary aluminum deck and

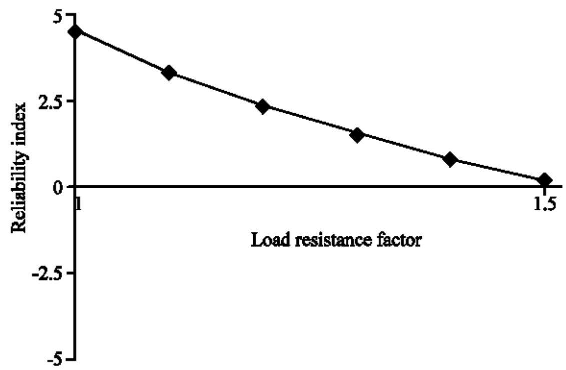

Fig. 4: Safety of LRF for aluminum decks

equivalent conventional reinforced concrete decks on steel girders. The results obtained for system I mode safety indices and their corresponding load resistance factors are presented in Fig. 5-14 for both aluminum decks and concrete decks systems. Figure 5-9 show aluminum decks at various percentages of composite actions with steel girders and these indicate reasonable results. The resulting safety level for full composite action shown in Fig. 5 is obtained as 17.3 and it far exceeds the target value of 3.5 specified by AASHTO (2004) for load resistance factors of 1.0. At higher resistance factors, the safety indices as indicated are above or equal to this safety target level of 3.5 up to a LRF of 1.21. Thus this result guarantees a good structural performance capability of the full composite action between the proprietary aluminum decks on steel girder systems in sustaining higher live loads on bridges than the code's specifications.

The percentages of composite action were done in such a way as to reduce the area of the deck in composite with the steel girders. This is to simulate the behavior of the bridge decks system under the expected maximum combination of loads. Figure 6-9 show reliability or safety indices obtained for various combinations of percentages in composite action.

The results obtained in Fig. 6-9 indicate safety indices above the target safety level of 3.5 for load resistance factors of 1.0-1.1 from 20-80% composite action with steel girders.

The results of the safety or reliability analysis on conventional concrete decks are also presented in Fig. 10-13. The condition of composite action of concrete decks from full composite to 20% composite action between concrete decks and steel girder sections were considered so as to compare results obtained to that of aluminum decks in order to clearly establish a perfect replacement of the concrete with aluminum decks.

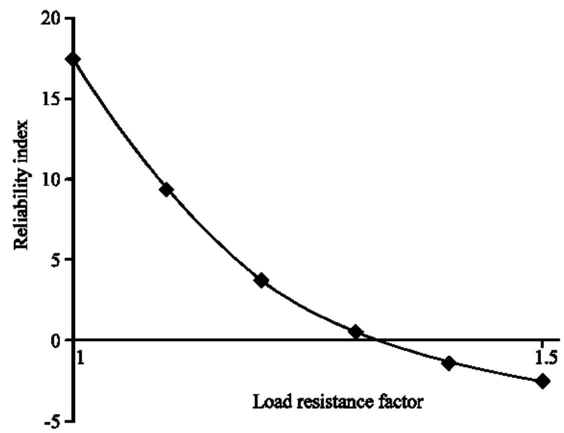

Fig. 5: Safety of LRF for full composite action of 100% aluminum deck and steel girder

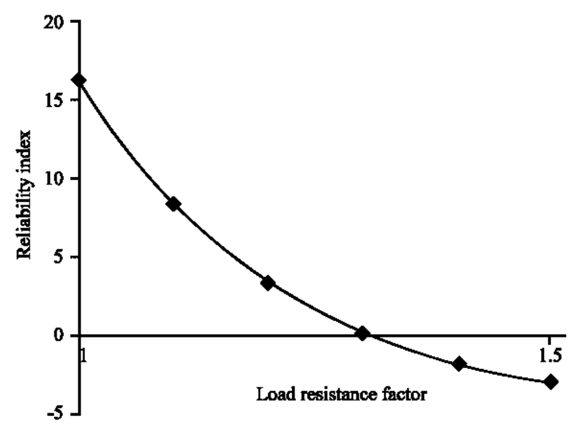

Fig. 6: Safety of LRF for 80% composite action of aluminum deck and steel girder

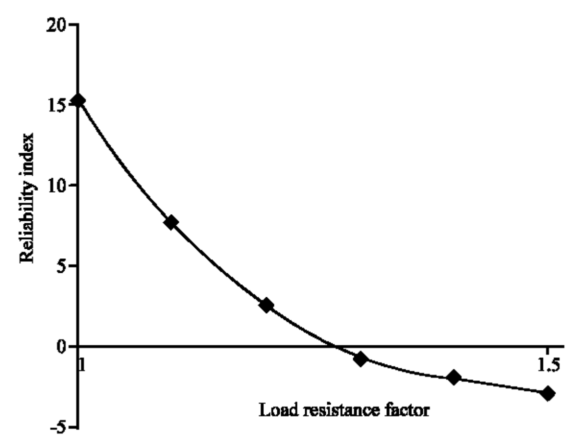

Fig. 7: Safety of LRF for 60% composite action of aluminum deck and steel girder

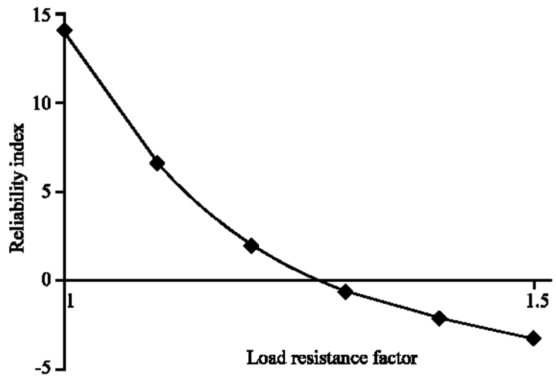

Fig. 8: Safety of LRF for 40% composite action of aluminum deck and steel girder

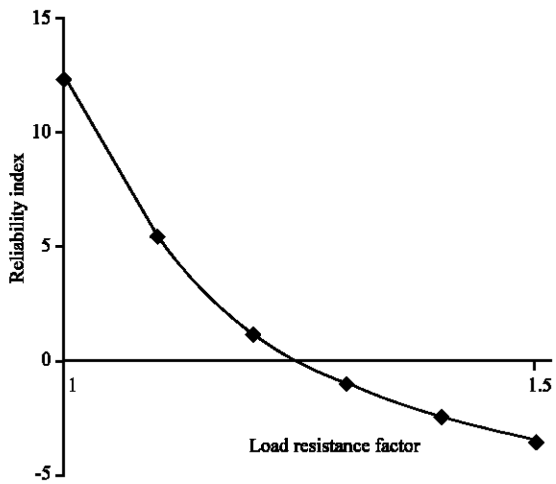

Fig. 9: Safety of LRF for 20% composite action of aluminum deck and steel girder

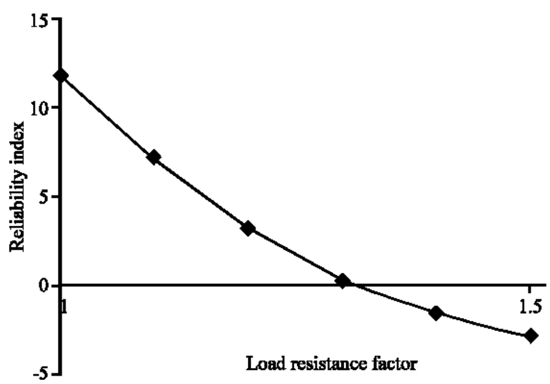

Fig. 10: Safety of LRF for 100% composite action of concrete deck and steel girder

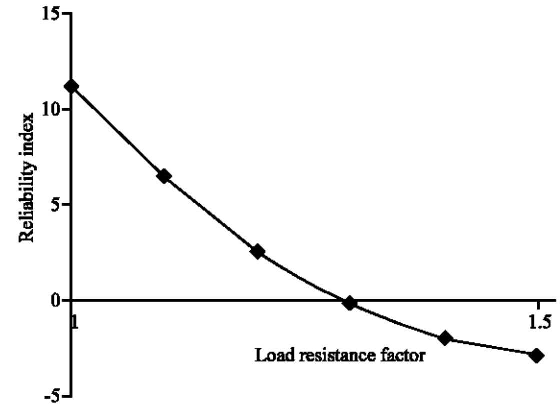

Fig. 11: Safety of LRF for 80% composite action of concrete deck and steel girder

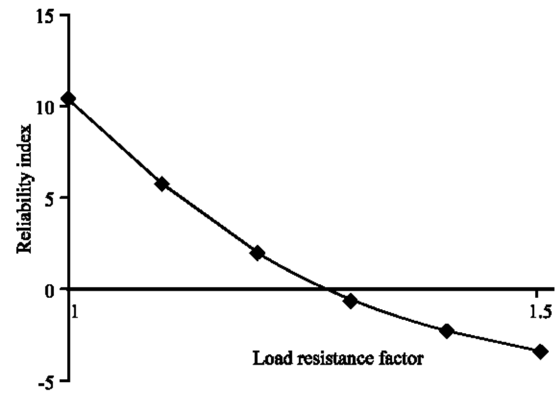

Fig. 12: Safety of LRF for 60% composite action of concrete deck and steel girder

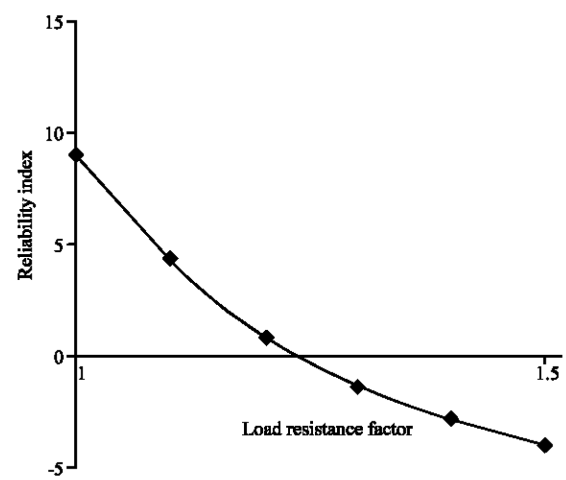

Fig. 13: Safety of LRF for 40% composite action of concrete deck and steel girder

Figure 10 shows the result of full composite action of concrete decks on steel girders. It is observed that the safety level of 11.83 at LRF of 1.0 obtained is above the target safety level of 3.5 in the code; while the

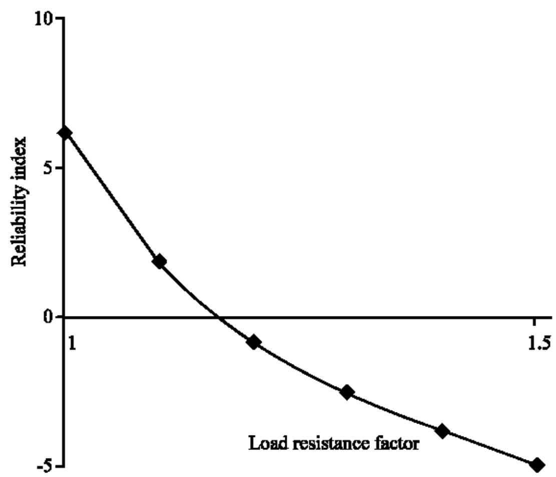

Fig. 14: Safety of LRF for 20% composite action of concrete deck and steel girder

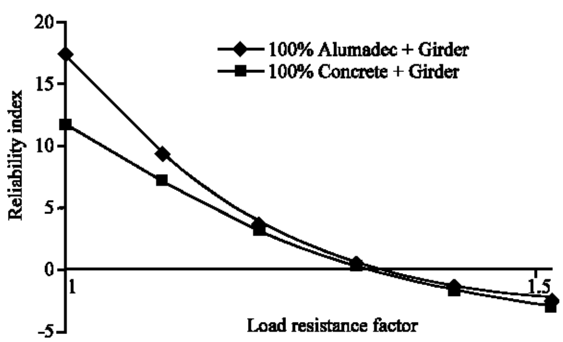

Fig. 15: Comparison of safety of LRF of full composite aluminum and concrete decks on steel girders

corresponding LRF for the specified safety level in the code is just 1.17. This gives about 24% below the codes specifications. A similar trend was generally observed for 20, 40, 60 and 80% composite action of concrete on steel girders for LRF of 1.0 with a maximum safety index of 6.2 at 20% composite.

The 20% composite action shown on Fig. 13 was found to be acceptable at LRF of 1.0, while the code's specification is only achieved at a LRF of about 1.05.

Figure 15 shows the combined results obtained from the evaluation of aluminum and equivalent concrete decks on steel girders as a basis for comparison. This indicates that aluminum decks are still very much structurally safe at an LRF of 1.21with a safety index of 3.5, while concrete decks gave a safety index of 3.5 at LRF of 1.17. Thus aluminum decks will sustain about 24% more loads than the conventional concrete decks. Hence, Fig. 15 clearly demonstrates the superiority in structural strength and durability of the proprietary aluminum decks over conventional concrete decks in addition to other advantages.

Reliability indexes for BS 5400 loadings: The reliability assessment of the aluminium deck using BS 5400 HA loading gives a safety indices of 8.8 while HB loading gave 9.1 for composite action of the bridge system which herein is referred to as system I.

CONCLUSION

From the modeling of safety of aluminum and concrete decks, the following conclusions are made:

It is shown that the proprietary aluminum deck (ALUMADECK™) system is in conformity with the AASHTO (2004) specifications which give the target value of safety or reliability index for members in flexure as 3.5 for load resistance factor of 1.0.

The results of the composite action of the aluminum deck and steel girders for LRF of 1.0-1.21 at full composite action indicate safety indices that are within the acceptable limits. This indicates that aluminum decks can withstand more live loads on the bridge due to composite action. Furthermore, results obtained indicate acceptable safety values for 20% composite action aluminum decks on steel girders at LRF of 1.0.

The percentage composite action between concrete decks on steel girders gives acceptable reliability indices for LRF of 1.0-1.17, while the LRF for aluminum decks is up to 1.21. This clearly demonstrates the superiority of aluminum decks over conventional concrete decks on steel girders in addition to the weight of the deck that is about 80% lighter than conventional concrete decks.

REFERENCES

AASHTO, 2004. LRFD Bridge Design Specifications. 3rd Edn., American Association of State Highway and Transportation Officials, Washington DC, USA.

Ang, A.H. and T.H. Tang, 1975. Probability Concept in Engineering Planning and Design. Vol. 1, John Wiley and Sons, New York, ISBN-13: 9780471032007, pp: 424.

Augusti, G., A. Baratta and F. Casciati, 1984. Probabilistic Methods in Structural Engineering. Routledge, UK., ISBN: 978-0-412-22230-6, pp: 556.

BS 5400-2, 1978. Steel, concrete and composite bridges. Specification for loads. British Standards Institution uncontrolled copy, UK. http://shop.bsigroup.com/en/ProductDetail/?pid=000000000000073235.

Cornell, C.A., 1969. A probability-based structural code. J. Am. Concr. Inst., 66: 974-985.

Davis, J.R., 1993. ASM Speciality Handbook Aluminium and Aluminium Alloys. ASM International, Materials Park, OH, USA., ISBN-10: 87170-469-X, pp: 605.

Ditlevsen, O. and H. Madsen, 2005. Structural Reliability Methods. 1st Edn., John Wiley and Sons, New York, ISBN: 0-471-96086-1.

Dobmeier, J.M., F.W. Barton, J.P. Gomer, P.J. Massarelli and W.T. Jr. McKeel, 2001. Analytical evaluation of bridge deck. J. Performance Constructed Facilities, 15: 68-75.

Ferhat, A. and M.F. Dan, 2004. Reliability correction: Comprehensive study for different bridges types. J. Struct. Eng., 130: 1063-1074.

Gollwittzer, S., T. Abdo and R. Rackwitz, 1988. First Order Reliability Method: Form Manual. RCP-GMBH, Germany, pp: 134.

Hadipriono, F.C., 1985. Analysis of events in recent structural failures. J. Struct. Eng., 111: 1468-1481.

Hasofer, A.M. and N.C. Lind, 1974. An exact and invariant second moment code format. J. Eng. Mech. Division, 100: 111-121.

Konig, G. and A.S. Nowak, 1999. Bridge Rehabilitation. Ernst and Sohn, Berlin, Germany.

Madsen, H.O., S. Krenk and N.C. Lind, 1986. Methods of Structural Safety. 1st Edn., Prentice Hall Inc., Englewood Cliffs, New Jersey, ISBN 10: 0135784757, pp: 403.

Matteo, A.D., P.J. Massarelli, J.P. Gomez, W. Wright and J. Cooper, 1997. Preliminary Evaluation of an Aluminum Bridge Deck Design for Highway Bridges. Structural Fault and Repair, Edinburgh, Scotland.

Melchers, R.E., 1987. Structural Reliability, Analysis and Prediction. 1st Edn., John Wiley, New York, USA., ISBN:0-85312-930-4, pp: 401.

Nowak, A.S. and K.R. Collins, 2000. Reliability of Structures. 1st Edn., McGraw-Hill Co. Inc., USA., ISBN: 0-07-048163-6, pp: 388.

Nowak, A.S., 2004. System reliability models for bridge structures. Bull. Polish Acad. Sci., 52: 321-321.

Raina, V.K., 2003. Concrete Bridge Practice: Analysis, Design and Economics. 2nd Edn., Tata McGraw-Hill Co. Ltd., New Delhi, ISBN: 0-07-462362-1, pp: 756.

Rosowsky, D.V., M.G. Steward and D.C. Eparachchi, 2002. Structural reliability of multi-storey building during constructions. J. Struct. Eng., 128: 205-213.

Surahman, A. and K.B. Rojiani, 1983. Reliability based optimum design of concrete frames. J. Struct. Eng., 109: 741-757.

Thoft-Christensen, P. and M.J. Baker, 1982. Structural Reliability Theory and its Application. Springer-Verlag, Berlin, pp: 267.

Wesley, J.M. and J.M. John, 2006. Load testing and load distribution response of missouri retrofitted with various FRP systems using a non-contact optical measurement system. Proceedings of 85th Annual Meeting, Jan. 22-26, TRB, Washington DC, pp: 2-2.