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Effects of Superstructure Flexibility on the Dynamics of Steel Girder Bridges

Abdulhakeem O AGBOOLA, Abdulfatai A MURANA*, Samuel O ABEJIDE

ABSTRACT [ENGLISH/ANGLAIS]

The probabilistic evaluation of the effect of superstructure flexibility of a conventional deck on steel girders using the current Load and Resistance Factor Design format is presented. The results demonstrate how superstructure flexibility affects the safety of bridges. The effect of decreasing the superstructure flexibility on safety of bridges for reinforced concrete decks on steel girders is achieved by modelling the slab thickness between 180mm to 300mm while acting as a composite structure. The specific results obtained are 4.0, 3.8, 3.5 and 3.0 for 180mm, 220mm, 260mm and 300mm respectively. The safety index of 4.0 which is obtained for slab thickness of 180mm is in conformity with the code specification which recommends a reliability index of 3.50. These safety indices in relation to the dead load component of the bridge deck show that there is a decrease in safety of the bridge as dead load component of decks increases when the live load remains constant. Similarly, the safety of the bridge decreases as the moment of resistance of the superstructure increases as a result of increases in overall dead load. However, the moment of resistance shows no appreciable increase for slab thicknesses of 260 mm to 300 mm.

Keywords: Reliability, deck systems, composite concrete bridges, dynamic response

RÉSUMÉ [FRANÇAIS/FRENCH]

L'évaluation probabiliste de l'effet de la flexibilité superstructure de pont classique surune des poutres d'acier en utilisant le courant de charge et le format de conceptionrésistance facteur est présenté. Les résultats montrent comment la flexibilitésuperstructure affecte la sécurité des ponts. L'effet de diminuer la flexibilitésuperstructure sur la sécurité des ponts pour les dalles de béton armé sur poutres en acier est obtenue par la modélisation de l'épaisseur de la dalle entre 180 mm et 300 mm tout en agissant comme une structure composite. Les résultats spécifiques obtenus sont 4.0, 3.8, 3.5 et 3.0 pour 180mm, 220 mm, 260 mm et 300 mm respectivement. L'indice de sécurité de 4,0 qui est obtenue pour épaisseur de la dalle de 180 mm est en conformité avec les spécifications du code, qui recommande un indice de fiabilité de 3,50. Ces indices de sécurité par rapport à la composante de charge morte du spectacle tablier du pont qu'il ya une diminution de la sécurité du pont en tant que composant de charge morte des augmentations de ponts lorsque la charge en directreste constante. De même, la sécurité du pont diminue à mesure que le moment de la résistance des augmentations de la superstructure à la suite de l'augmentation descharges mortes globale. Cependant, le moment de la résistance montre pas d'augmentation appréciable pour des épaisseurs de dalle 260 mm à 300 mm.

Mots-clés: Fiabilité, les systèmes de pont, composites ponts en béton, la réponse dynamique

Affiliations:

Department of Civil Engineering, Ahmadu Bello University, Zaria. NIGERIA

Address for Correspondence/ Adresse pour la Correspondance: fatinoyi2007@yahoo.co.uk

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INTRODUCTION

Recent developments in bridge designs have shown a trend towards superstructure flexibility resulting from lighter decks, increased traverse girder spacing and longer spans [1].

Accumulation of research in the field of bridge evaluation has indicated the justification of using reliability index for the measure of safety [2]. In this presentation, the conventional concrete deck on steel girder is evaluated for composite action between the deck and the girder (acting as one component), as a case study. It is intended to study the dynamic force response of the bridge based on increasing superstructure flexibility. This is demonstrated here by reducing the thickness of the concrete deck at determined dynamic amplification factors. The results focus on how this helps in increasing the life load carrying capacity of bridges while retaining the substructures and the super structures.

The (LRFD) specification [3] is employed herein for the evaluation of concrete decks on steel girders. The computer algorithm employed is the First Order Reliability Methods (FORM) as proposed [4]; [5]; [6]; [7]. The scope involves the application of the algorithm for reliability calculations of conventional concrete decks of various thicknesses at a specified design strengths using LRFD method as the limit state function.

LIMIT STATES

The developed theoretical works of reliability and safety checking of structural members have been presented in many textbooks (e.g., [6]; [8]; [9]. 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 [3] 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. This paper is focused on the ULS of the bridge deck since it is the criterion that is most critical in decks.

SAFETY INDICES

The concept of a 'limit state' is often used to help define failure in the context of structural reliability analyses. [6] Defines the concept of limit state as a formalization of the criteria under which the structure can be considered to have failed. A limit state can be represented mathematically as (where R is the resistance and S the applied load):

$$g(R,S) = R - S \qquad (1)$$

is a function of several other stochastic variables, such as modulus of elasticity, cross section area, bending strength, etc.

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:

$$P_f = P(R - S < 0) = P(g < 0) \qquad (2)$$

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

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

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. 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:

$$P_f = \Phi(-\beta) \qquad (4)$$

where Φ represents the cumulative Gaussian distribution of the standard normal law and β is the reliability index according to [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 [7]. 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 [10], so that we can use the design point, x^*, as the expansion point [11] 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.

The procedures for determining the safety indices and probability of failure has been programmed in FORM5 [12] and calculations are carried out using the computer algorithm.

BRIDGE LOAD MODEL

The loads considered in this work are dead and live load components. 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 conventional reinforced concrete decks with various thicknesses ranging from 180mm to 300mm as shown in figure 1.

The live load components adopted for this analysis was derived using the [3] load specifications and determining the corresponding moments for the loading conditions. Values obtained for truck positions for maximum moment and lane load are given in Table 1.

The moment obtained were multiplied with bias factors to obtain the expectation used in reliability analysis, as given in equation 5.

$$M_i = \lambda_i . S \quad (5)$$

Where:

M = maximum moment in girder; w = uniformly distributed load; L = girder span (say 14.5m). The unit weight of asphaltic concrete is 22.43 kN/m^3

Therefore, the applied loads are derived thus:

Slab: w = 2.18*0.18*24kN/m^3= 9.42kN/m.

M = 9.42*14.5^2*0.125= 247.51kN-m.

Girder: w = 1.65kN/m (self weight)

M = 1.65*14.5^2*0.125= 43.36kN-m

DC 1= concrete slab + girder

Concrete parapet (DC2) = 24 kN/m^3 (self weight)

M = (6.32*0.5)*14.5^2*0.125 = 83.04 kN-m

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 [3] limit state equation as stated in equation (6).

$$\eta_D \eta_R \eta_I \sum \gamma_i S_i \le \varphi R_n \le R_r \quad (6)$$

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 [3] 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,

$$\eta \sum_{i=1}^{n} \gamma_i S_i \le \varphi R_n \qquad i = 1, 2, 3, 4 \ldots\ldots n \quad (7)$$

RESISTANCE MODEL

The probabilistic distributions of the basic variables in Table 2 are obtained from the limit state equation as given earlier [13]. 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 Table 3.

The yield moment, that is, the moment which causes the first yield, is computed using equation (8). 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.

$$F_{yt} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \quad (8)$$

The standard deviation can be obtained from Equation (9) as:

$$SX = (COV).(EX) \quad (9)$$

RESULTS OF RELIABILITY ANALYSIS

The results of reliability estimates are shown in Figures 2 to 4. These results clearly demonstrate how super structure flexibility affects the safety of bridges. The safety index of 3.5 was obtained for slab thickness of 180mm which is in conformity with the AASHTO 2004 LRFD specification which recommended a reliability index of 3.5.

Table 1: This table shows values used in the reliability calculation

S/No	Slab thickness (mm)	Wearing course (kNm)	DC1 moment (kNm)	DC2 moment (kNm)	Live load moment (kNm)	Resistance moment (kNm)
1.0	180	96	291	83	1023	3219
2.0	220	96	346	83	1023	3249
3.0	260	96	401	83	1023	3263
4.0	300	96	456	83	1023	3260

Table 2: This table shows comparison of Bending Moment between AASHTO and BS loading

Simply supported span versus maximum bending moment for one lane Maximum bending moment for one lane, including impact allowance (kN-m)			
Span (m)	HS 20-44 loadings 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

Table 3: This table shows statistical Parameter of Load and Resistance Factor of Reinforced Concrete decks on steel girder

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

Figure 2: This figure shows effect of Safety Index on Slab thickness

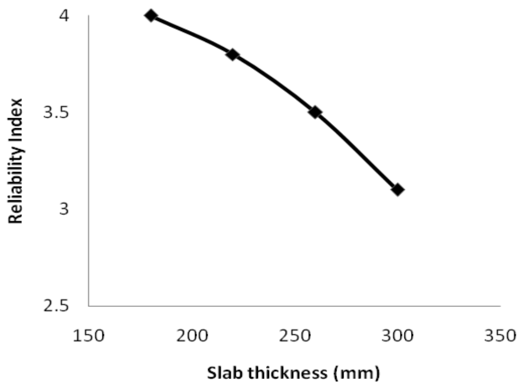


Figure 3: This figure shows effect of Dead Load on Superstructure Safety

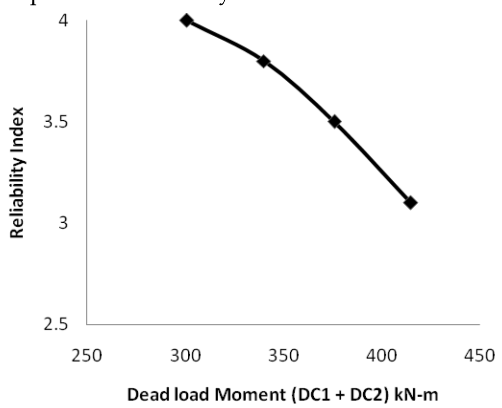
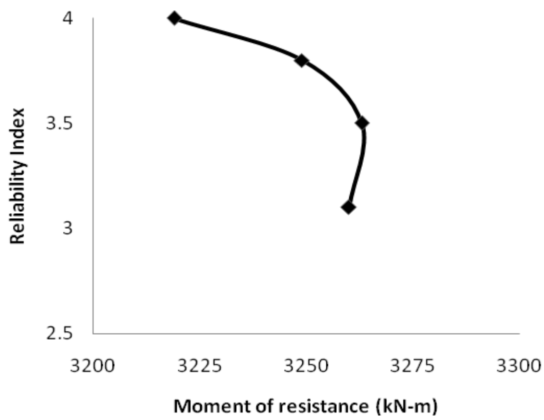


Figure 4: This figure shows implication of Moment of Resistance on Deck Safety



The effect of decreasing the super structure flexibility on safety levels of reinforced concrete on steel girder is achieved by modeling the slab thickness between 180mm to 300mm acting as a composite structure. The composite action between the deck and the girder is then re-designed and the reliability index re-calculated.

The results obtained are 4.0, 3.8, 3.5 and 3.1 for 180mm, 220mm 260mm and 300mm respectively. The reliability index for slab thickness of 300mm is obviously below AASHTO LRFD Specification. See Figure 2.

The relationship of reliability index to dead load component shows that there is a decrease in safety level as structural dead load component is increased. This is shown in Figure 3.

Similarly, the safety level decreases as the moment of resistance of the structure increases as a result of increases in overall dead load. However, the moment of resistance shows a decreasing trend for slab thicknesses of 260 mm to 300 mm. See Figure 3.

CONCLUSION

The reliability or safety estimate obtained in this study for the bridge superstructure indicates that the structural dead load (girder + concrete deck) plays a significant role in the overall safety of a bridge system. The more flexible the super structure is the safer the bridge becomes and the more live load it can sustain. Also, in composite concrete deck on steel girders bridge design, it is better to limit slab thicknesses between 180mm and 260mm so as to ensure good performance of the bridge structures. And finally, the increase in the resisting moment due to combined effect of the long and the short term loading condition does not necessarily guarantee structural safety. This is because when the deck thickness is more than 260mm, the moment of resistance decreases likewise the structural safety of the composite deck system.

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Nil

CONFLICT OF INTEREST

No conflicts of interests were declared by authors