

A stochastic framework for the assessment of horizontally curved aluminium bridge decks on steel girders

Abdulhakim Adeoye Shittu^{a,b,*}, Samuel Abejide^c, Samuel Chukwuemeka Olisa^d, Ali

Mehmanparast^e, Athanasios Kolios^e

^aFaculty of Engineering, Federal University Wukari, 200 Katsina-Ala Road, Wukari, P.M.B.

1020, Taraba State, Nigeria

^bSchool of Water, Energy and Environment, Cranfield University, Bedford, MK43 0AL United

Kingdom

^cCivil Engineering Department, Walter Sisulu University, Butterworth Campus, 4960, South

Africa

^dSchool of Aerospace, Transport and Manufacturing, Cranfield University, Bedford, MK43 0AL

United Kingdom

^eFaculty of Engineering, University of Strathclyde, Glasgow G1 1XQ, United Kingdom

*Corresponding author at: Faculty of Engineering, Federal University Wukari, 200 Katsina-Ala Road,

Wukari, P.M.B. 1020, Taraba State, Nigeria

E-mail address: adeoyeshittu@fuwukari.edu.ng (A.A. Shittu).

Abstract:

A purpose-developed structural reliability assessment (SRA) framework for the evaluation of horizontally curved aluminium alloy bridge decks on steel I-girders of centre subtended angle, $\theta \leq 34.4$ with precinct on the American association of state highway and transportation officials (AASHTO) load resistance factor design (LRFD) specification is presented. The finite element analysis (FEA) simulations were performed in the ABAQUS® CAE, whilst the probabilistic assessment model was developed using the first-order reliability method (FORM). Besides performing detailed design checks

and a validation exercise, the developed SRA framework was used to examine the structural behaviour of the bridge assembly in the presence of stochastic design truck axle loads, structure's self-weight, among other loads, whilst varying geometric properties. The most critical structural responses chosen at salient points obtained via FEA simulation is applied in deriving the limit state function (LSF), which is then substituted into the stochastic model within the purpose-developed iterative FORM algorithm to calculate the reliability index (RI), β of the structure. It is shown that the proprietary AlumadeckTM system conforms with the LRFD specification, which stipulates that the target value of RI is 3.5 for resistance factor (RF) of 1.0 (assuming 80% composite action compression flange). The result also reveals that the RI shows a strong dependence on the composite action between the deck and the girder for RF of 1.0 full composite action (considering the failure of the bottom flange), indicating the safety index is within acceptable limits. Furthermore, it is revealed that the minimum composite action for safety is 40%, corresponding to a safety index of 1.16. Hence, it can be inferred from the foregoing that the AlumadeckTM can withstand the stochastic axle load it is subjected to considering the HL-93 load design condition and satisfies all design criteria considered from a stochastic perspective based on the AASHTO LRFD guidelines, provided the minimum stiffener thickness $t_{stiffner}$ of 7mm is adopted (based on the FEA simulation results). A case study conducted herein established that the structural configuration selected (i.e. depth at 2.4m, flange thicknesses at 21mm, flange width at 50cm and web thickness at 16mm) demonstrates the structural safety and durability of the bridge system coupled using the AlumadeckTM.

Keywords: Structural reliability assessment (SRA), FORM, FEA, AlumadeckTM, curved bridge decks

1 Introduction

Curved I-girder (CI-G) bridges possess unique properties which enable them to change direction within each span; making them ideal interchange structures for highways or to connect existing road sections where abutments cannot be relocated for physical or economic reasons [1]. CI-Gs in recent times are gaining widespread use as highway infrastructure as well as its use for; rebuilt atop existing structures to handle increasing traffic volumes or new interchange geometries within the context of urban settings

[2]. CI-G are most advantageous for constructing roadways in areas with serious geographical or manmade impediments.

Majority of existing bridge decks are a subject of grief for engineers resulting from a lack of innovative maintenance programs as well as alternative measures to mitigate the risk of failure. The conventional highway bridge deck system deteriorates over time due to changes in microclimate such as variations in temperature gradient, moisture content fluctuation, freeze-thaw cycle and effect of alkaline minerals; usually a common problem with bridge structures. Corrosion of the reinforcing steel has been identified as the major cause of the deterioration in concrete; this remains the most critical factor responsible for the large majority of structurally deficient bridges.

Most bridge decks may require replacement after a cycle of 15 years minimum or after 20 years, depending on the design criteria, while the substructure and superstructure tend to last 40 years or more [3]. Eventually, someday 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 [4], reduce highway closure time and make retrofitting that meet the current design specifications possible while retaining the superstructure and substructure.

Improving the design service life of bridges can be achieved using alternative materials, such as fibre-reinforced composites or aluminium. Since composites and aluminium are: lighter than steel and concrete, do not rust nor need painting or protective coatings, and have shorter fabrication as well as erection time; thus cheaper, they have a distinct advantage over other construction materials. Aluminium will often be more affordable than concrete when whole-life costs are calculated. The suggestion made by Reynolds aluminium company (manufacturers of the Alumadeck™) has given way to a feasible alternative to conventional reinforced concrete decks. This deck is new and requires a close study of its characteristic structural behaviour.

Savings as high as 40% on steel material could be realised utilising an aluminium deck and steel girder system. These savings are attributed to the decrease in the dead load that aluminium provides and the fact that the steel girders do not have to support large construction loads required during the curing of a reinforced concrete deck. Principally, because it is light, durable, and can be created into a wide variety

of structural forms. These properties have been widely exploited in aerospace, railway carriage and architectural applications; they are also useful for bridgeworks. The low self-weight can be invaluable for handling during fabrication and construction and in the final design. The durability of aluminium alloys is very good, and it is one of the most underestimated virtues of the material. Most designers have often been confronted with negative perceptions – cost, corrosion, deflection and fatigue being the main concerns that abound. Many believe that aluminium alloys will not be fit for a highway bridge. In the main, these perceptions arise from examples of poor design. An aluminium structure will compete with any other material on cost with the right approach and will outperform most in service.

Consequently, when fabrication, erection and treatment costs are taken into account, there is little difference for the completed structure. Though the initial cost of aluminium alloys is higher compared to other traditional concrete, which is the barrier to their implementation in the construction industry, aluminium will often be cheaper than concrete when whole-life costs are calculated [5]. Aluminium is the third most abundant element in the earth's crust, occurring mostly as aluminosilicates. The abundance of raw material and ease of recycling is such that resource depletion will not be an issue. It is extracted by electrolysis of alumina from bauxite fused with cryolite.

Structural analysis using the finite element method (FEM) has found a wide application for rigorous and accurate engineering response predictions and investigates structural behaviours difficult to experimentally study. Mathematical models used to evaluate the failure response of structures can either be one-dimensional (1-D) beam models or three-dimensional (3-D) finite element analysis (FEA) models. In most cases, the 3-D FEA mathematical models are more advantageous since they can analyse high-level stress distributions and predict structural responses more accurately. Hence, owing to the high fidelity, (3-D) FEA models have been widely applied to model similar structures as studied herein [6,7], and this study adopts the FEA approach to simulate these structural responses [8–12].

Stochastic finite element methods (SFEMs) have recently become an active area of research. As the name suggests, researchers in this field attempt to combine two crucial methodologies developed to deal with the complex problems of modern engineering: FEA and stochastic (or probabilistic) analysis. Stochastic analysis refers to the explicit treatment of uncertainties in any quantity entering the

corresponding deterministic analysis at a specific period of time. The exact values of these quantities are usually unknown because they cannot be precisely measured [13,14]. It describes the merging of advanced reliability methods with the FEM to obtain probability estimates for predefined performance criteria.

In order to apply the finite element (FE) technique, the region of interest is discretised by a FE mesh. The basic idea of the mean-based, second-moment analysis as used in stochastic FEA is to expand, via Taylor series, all the vector and the matrix stochastic field variables typical of deterministic FEM about the mean values of the random variables, to retain only up to second-order method terms and to use in the analysis only the first tier statistical moments. In this way, equations for the expectation and cross-covariance of the nodal displacement can be obtained in terms of the nodal displacement derivatives with respect to the random variables [15]. A comprehensive review of the capabilities and limitations of SFEMs has been discussed in [8,10]. Given the inherent drawbacks, such as this method being too intricate and computationally expensive, which could lead to fatal errors in programming such algorithms, a simple novel SFEM technique is developed herein, which offers a more efficient structural reliability assessment (SRA) framework capable of predicting the safety of engineering structures in a cost-effective time scale.

This study aims to investigate an integrated FEA and First Order Reliability Method (FORM) to ascertain the durability and safety levels of the proposed curved aluminium deck and the corresponding supporting steel CI-G under wheel loads. To achieve this aim, this is broken down into the following manageable steps: First, to analyse the horizontally curved bridge deck using FEM for the system I and II stress, and second to obtain a suitable single equation based on the requirement of [16,17] that will serve as the limit state function (LSF), next to perform a reliability evaluation of a horizontally curved bridge structural system using the HL-RF FORM to determine the safety indices and last to propose apposite geometric dimensions. In Siwowski [3], research was conducted to develop and implement an aluminium bridge deck system that would be applicable and realisable in domestic conditions. This involved several load tests on prefabricated deck panels in order to examine and evaluate the panel behaviour under standard truck load and when loaded to failure. In Siwowski [18] demonstrated a

computational finite element modelling procedure to characterise the static behaviour of the aluminium bridge deck panel and for predicting the performance of panel under service load. In Dobmeier et al. [19] the service-load behaviour of an aluminium deck panel was evaluated by developing finite element models that predict panel responses; static load tests were conducted in the laboratory and measuring the responses of these load tests. In Agboola et al. [20], a probabilistic model for the evaluation of the Alumadeck™ in flexure between girders using AASHTO LRFD specification was developed. Also, a comparative study of safety margins of composite action between aluminium decks on steel girders and concrete decks on steel girders was conducted to ascertain the superiority of the Alumadeck™ from a strength and serviceability perspective. Consequently, in all these studies, straight aluminium deck panels were investigated. Having completed an extensive literature study [3,4,8,21–29], to the best of the authors' knowledge, the work performed herein is the first to study the SRA of a complex curved bridge structural system employing a purpose-developed integrated FEA-FORM based SRA framework to evaluate the safety levels of the structure in terms of reliability index (RI), β .

Well-detailed and properly erected structures designed by the limit state method will have acceptable probabilities that they will not become unfit for their intended purpose. Thus, a design engineer should strive to achieve good design and be creative while at the same time appreciating the dangers inherent in revolutionary concepts. Ample experience in the past and in recent times has shown that uncommon designs or unfamiliar constructional methods and materials do increase the risk of failures such as encountered in this study [22].

The remaining parts of this paper are structured as follows. Section 2 presents the theoretical background on bridge dynamics; Section 3 presents the methodology applied in the present study; Section 4 presents the results and discussion, followed by the conclusion in Section 5.

2 Bridge dynamics

Structural response of bridge components to dynamic and cyclic loading is critical to bridge evaluation. Structural resistance moments and serviceability depend on the load configuration. However, a bridge's actual service life response may be different from the initial loading configuration proposed. The actual

load distribution characteristics and dynamic amplification of responses in a bridge have been studied using SRA [17,20,30], and this will be adapted for this study. During construction, the steel girder must support the weight of the deck and steel weight in addition to other construction loads. There have been large relative deflections observed between girders on CI-G bridges during placement of concrete decks, making it challenging to maintain the specified final camber and superelevation and form and key in the construction joints [30]. The Alumadeck™ is shop fabricated and, therefore, will not provide the same issues as reinforced concrete decks during construction.

2.1 Description of the bridge deck (Alumadeck™)

Figure 1(a) refers to the two-dimensional view of an aluminium deck proposed for evaluation in this work. As shown in Figure 1(b), the Alumadeck™ comprises the top and bottom surfaces and vertical and inclined stiffeners.

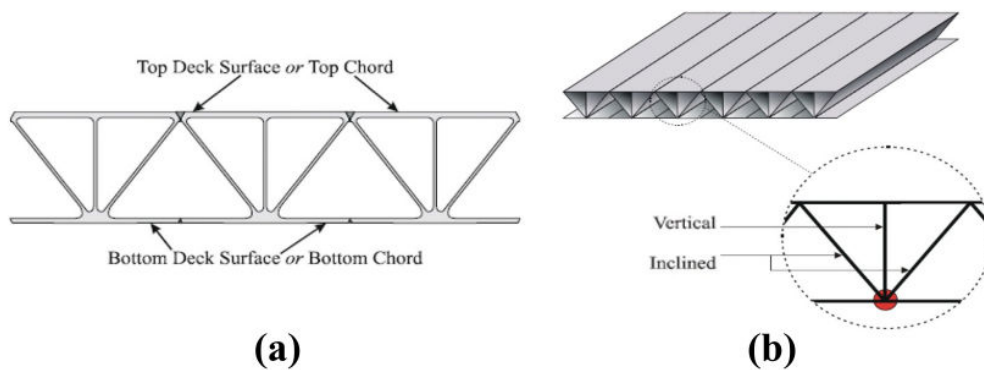


Figure 1. Nomenclature of: (a) top and bottom deck surfaces (b) stiffeners [3].

2.2 Bridge design concepts

Research by [17,31,32] has shown the types of loadings that act on bridges. The load-carrying capacity of bridges must meet two safety criteria, namely: unrestricted use by vehicles and restricted use by heavier vehicles. Two types of loads are considered in the design of bridges: First are the permanent loads known as the dead load, superimposed dead loads, loads due to filling materials, differential settlement, and loads due to creep and shrinkage. The dead load carried by a bridge member consists of its self-weight, portions of the superstructure's weight, and any fixed loads supported by the member. Secondly are transient loads, defined as wind loads, temperature loads, exceptional loads, erection

loads, centrifugal loads, braking, skidding and collision loads. The load-deformation of a curved plate is influenced by the flexural and torsional rigidities of the slab. These deformations are expressed in the mathematical form of differential equations (DEs). The DEs of flexure for curved orthotropic plates are expressed relative to the radius and angle of rotation with other stiffness parameters like modulus of elasticity and rigidity, moments of inertia and torsional constant.

2.2.1 Bridge load model

The load model applied in this research are the dead and live load components derived from AASHTO [17] specifications. The dead load components employed include factory made member weight (girders), deck slab and wearing coarse. Live load parameters are derived from AASHTO [17], 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 0.15m close to the centre line [20].

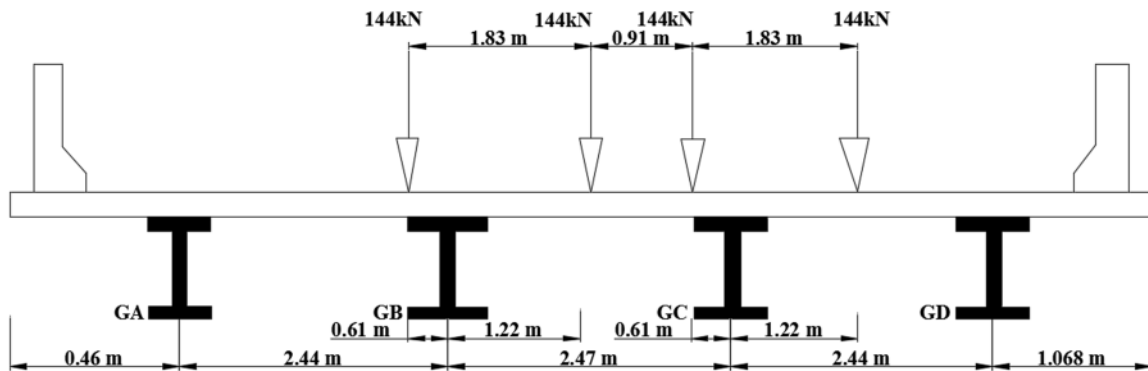


Figure 2. Typical transverse section of the bridge showing HL-93 loading location.

2.2.2 Bridge deck design

According to AASHTO [17], the first principle design of a bridge deck slab is perpendicular to the centre line of traffic (i.e. the direction of vehicular loading). The basic partial DE (PDE) governing the behaviour of a curved plate is derived from the assumptions that external loads are normal to the surface of the deck plate, the plate acts as a monolithic unit, the deck plate is homogenous, isotropic and elastic, the plane surface initially perpendicular to the middle surface of the deck plate remains plane and

perpendicular to the middle surface during bending, and the deflections are small in relation to the thickness of the orthotropic plate [33].

2.2.3 The AASHTO flexural resistance equations

It is crucial to recall the following structural mechanics' expression in Eq (1) as this will be extensively applied in the present study:

$$\sigma = \frac{Mc}{I} \quad (1)$$

where σ represents stress, M the bending moment, c the distance to the structural element's neutral axis (NA) along the height of the cross-section, and I is the centroidal moment of inertia.

The Eq (2) gives the formulation for the AASHTO [16] flexural resistance equations for members in which the flexural resistance is expressed in terms of stresses and addresses the combined effects of major-axis bending and flange lateral bending:

$$f_{bu} + \frac{1}{3}f_l \leq \phi_f F_n, \quad (2)$$

whilst Eq (3) gives the formulation for members in which the flexural resistance is expressed in terms of moments,

$$M_u + \frac{1}{3}f_l S_x \leq \phi_f M_n, \quad (3)$$

where: f_{bu} = flange major-axis bending stress, f_l = flange lateral bending stress, $\phi_f F_n$ = factored flexural resistance in terms of the major-axis bending stress, M_u = member major-axis bending moment, S_x =elastic section modulus about the major-axis of the section to the flange under consideration taken as $\frac{M_{yf}}{F_{yf}}$, and $\phi_f M_n$ = factored flexural resistance in terms of member major-axis bending moment. The plastic moment capacity is in **Appendix ((A1))**.

A skewed bridge is one in which the major axis of the substructure is not perpendicular to the longitudinal axis of the superstructure. The effects of skew and grade are primarily geometric, with some effects on shears and moments. Larger skew angles also have some effects on live load distribution. The effects of curvature are both structural and geometric. Structures with very sharp curvature, such as freeway on or off-ramps, may require the use of specially made box beams. Straight

AASHTO I-beams are normally used on curved bridges with shorter spans or on longer spans with larger radii because the offset between arc and chord is small.

The proposed aluminium deck on steel girders, evaluated using FEM and SRA, and the formulations of AASHTO [16,17] with parameters such as stiffener thickness, percentage composite action, etc., are studied herein to determine the possibility of use in horizontally curved alignments and sometimes in skewed situations whilst the safety of doing this is also evaluated. The methodology employed is discussed in the next section.

3 Methodology

3.1 Structural reliability

Sorensen [34] defined the reliability of structural systems as the probability that the structure under consideration has a proper performance throughout its lifetime. Structural reliability (SR) could also be defined as the structure's ability to fulfil its design purpose for some specified reference period [10,35]. The study of SR is concerned with calculating and predicting the probability of an LS violation for engineered structures at any stage during their life. In particular, the study of structural safety concerns the violation of the ultimate or safety LS for the structure. The “violation” of an LS is the attainment of an undesirable condition for the structure [36].

3.1.1 Limit state functions (LSF)

Bridges can fail in many ways (modes of failure), by cracking, corrosion, excessive deformations, exceeding carrying capacity for shear or bending moment, local or overall buckling, and so on. There are three types of limit states (LS), according to several sources [6,9,16,17]. Ultimate limit states (ULS) are mostly related to the bending capacity, shear capacity and stability. Serviceability limit states (SLS) are related to gradual deterioration, user's comfort or maintenance costs. For further details, see [37].

According to several sources [9,10,38], the ULS (ultimate limit state) defines the ability of the structure to resist yielding. In terms of the ULS, the maximum stress in the structure $\sigma_{VM,max}$ should not exceed the allowable stress limits $\sigma_{VM,allow}$. The Eq (5) gives the formulation for the LSF for the Von Mises criterion:

$$g_u(x) = \sigma_{VM,allow} - \sigma_{VM,max} \quad (4)$$

Eq. (6) gives the formulation for allowable stress $\sigma_{VM,allow}$:

$$\sigma_{VM,allow} = \frac{\sigma_y}{\gamma_m} \quad (5)$$

where σ_y is the yield strength, and γ_m is the material safety factor. This concept will be applied later in the current study.

3.2 Concept of the first-order reliability method (FORM)

In this study, having determined the stochastic variables and obtained the LSF, the RI, β , is then calculated using the iterative Hasofer Lind Rackwitz-Fiessler (HL-RF) FORM. This method incorporates several uncorrelated standard normal random variables. Independent of whether each parameter has been defined as Normal, Log-Normal, Gumbell, etc. distribution, the original variables, which may, in general, be correlated and non-normal, are transformed to the u-space (or equivalent standard space with zero mean and unit variance) using well-established transformations such as Rosenblatt's transformation. The original limit state surface (LSS) is then mapped onto the new LSS. The exact failure probability is the integral of the joint probability density function over the failure domain $g(u) < 0$. The first-order Taylor series expansion of the LSS $g(u) = 0$ is applied at the point with the shortest distance from the origin in the u-space. An RI, β can be referred to as the shortest distance from the origin to the LSS in this space. The point at which the distance from the origin is minimum to the LSS denotes the worst combination of random variables and is thus called the design point or most probable failure point (MPP). The HL-RF algorithm can be employed in recursively searching for the MPP. This technique can be easily extended to non-linear LS and has a reasonable balance between ease of use and accuracy [39].

Having defined the LSF $g(u_1, \dots, u_n)$ in the u-dimensional Euclidean space, the FORM aims to approximate the failure domain $F = \{\mathbf{u}; g(\mathbf{u}) < 0\}$ by a halfspace by replacing the LSF by a linear tangent hyper-plane at the point $\mathbf{u}^*$. The LSS $G = \{\mathbf{u}; g(\mathbf{u}) = 0\}$ has the nearest geometric distance to

the origin, which means that the PDF is maximal there since it is proportional to $-|\mathbf{u}|^2$. The HL-RF algorithm revolves around finding a point $\mathbf{u}^*$ for an LSF $g(\mathbf{u})$ at the normal standard space such that

$$|\mathbf{u}^*| = \min_{g(\mathbf{u})=0} |\mathbf{u}| = \min_{g(\mathbf{u}) \leq 0} |\mathbf{u}| \quad (6)$$

i.e. with the shortest distance to the origin. Basically, this involves linearising the LSF at an initial point, calculating the design point for the linearised LSF and then proceeding recursively, always again linearising until convergence is achieved. Further advanced information on this concept can be found in [8]. See Figure 3 as follows:

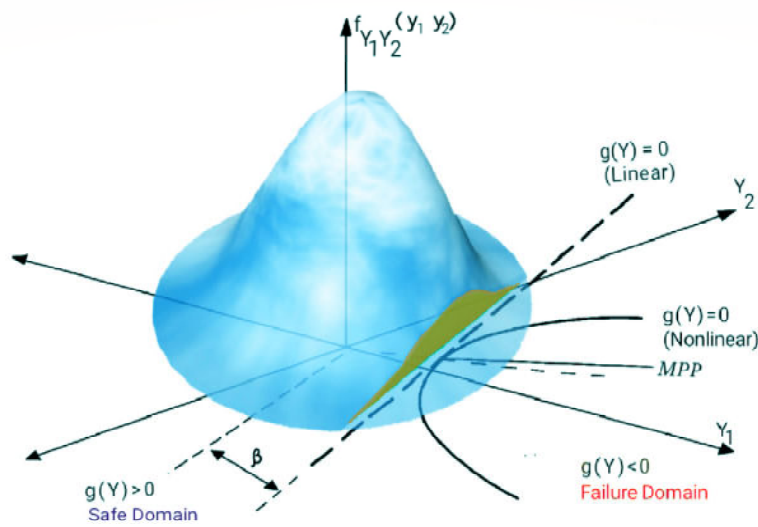


Figure 3. Illustration of the first-order reliability method (FORM) approximation.

3.2.1 Calculating the Safety Index

1. Define the limit state function (LSF) for the corresponding LS, eg. ultimate limit state (ULS), serviceability limit state (SLS), etc.
2. Let the mean value point be the initial design point, i.e. $x_{i,k} = \mu_{x_i}$ $i = 1, 2, \dots, n$, and evaluate the gradients $\nabla g(X_k)$ of the LSF at this design point. Where, $x_{i,k}$ represents the i^{th} element in the vector X_k of the k^{th} iteration, and μ_{x_i} is the mean value of the i^{th} element;
3. Compute the initial RI, β adopting the mean-value approach, i.e. $\beta = \frac{\mu_{\bar{g}}}{\sigma_{\bar{g}}}$ and its direction cosine α .

$$\beta = \frac{\mu_{\tilde{g}}}{\sigma_{\tilde{g}}} = \frac{g(\mu_X)}{\left[\sum_{i=1}^n \left(\frac{\partial g(\mu_X)}{\partial x_i} \right)^2 \cdot \sigma_{x_i}^2 \right]^{\frac{1}{2}}} \quad (7)$$

$$\alpha_i = - \frac{\left(\frac{\partial g(X^*)}{\partial x_i} \right) \cdot \sigma_{x_i}}{\sqrt{\sum_{i=1}^n \left(\frac{\partial g(X^*)}{\partial x_i} \sigma_{x_i} \right)^2}} \quad (8)$$

4. Calculate a new design point X_k and U_k , function value, as well as gradients at this new design point.

$$x_{i,k} = \mu_{x_i} + \beta \sigma_{x_i} \alpha_i \quad (9)$$

$$u_{i,k} = \frac{x_{i,k} - \mu_{x_i}}{\sigma_{x_i}} \quad (10)$$

5. Compute the RI β and direction cosine α_i using Eqs. (9) and (10), respectively.

$$\beta = \frac{g(U^*) - \sum_{i=1}^n \frac{\partial g(U)}{\partial x_i} \sigma_{x_i} u_i^*}{\sqrt{\sum_{i=1}^n \left(\frac{\partial g(U^*)}{\partial x_i} \sigma_{x_i} \right)^2}} \quad (11)$$

$$\alpha_i = - \frac{\left(\frac{\partial g(X^*)}{\partial x_i} \right) \cdot \sigma_{x_i}}{\sqrt{\sum_{i=1}^n \left(\frac{\partial g(X^*)}{\partial x_i} \sigma_{x_i} \right)^2}} \quad (12)$$

Iterate steps (4) to (5) until the values of the RI β converge. The dz and error tolerance of the FORM is 10^{-3} and 10^{-2} respectively. The failure probability P_f can then be calculated from the following:

$$P_f = \Phi(-\beta) = 1 - \Phi(\beta) \quad (13)$$

Where $\Phi(\cdot)$ is the standard normal cumulative distribution function (CDF) given as

$$\Phi(\beta) = \int_{-\infty}^{\beta} \frac{1}{\sqrt{2\pi}} e^{-\frac{1}{2}z^2} dz \quad (14)$$

3.3 Theory of curved decks

According to several sources [33,40], the formulation for DE for flexure of a curved plate is detailed in **Appendix (A2) – (A6)**. The general solution to **Appendix (Error! Reference source not found.A2)** determined by Heins and Hails [33] can be expressed in Appendix (A7) – (A8). The curved plate moment DEs are expressed in Appendix (A9) – (A12): Appendix (A9) and (A10) are expressed in terms of the general solution of Appendix (A6) for the radial moment and radial shear. Since the angular moment and shear are much smaller than the radial moment and shear, the angular moment and shear shall be neglected. See **Appendices (A13) and (A14)** for their expressions.

The above concept will be applied herein in the upcoming Section 4. The flowchart of the developed SRA framework for this study is depicted in Figure 4.

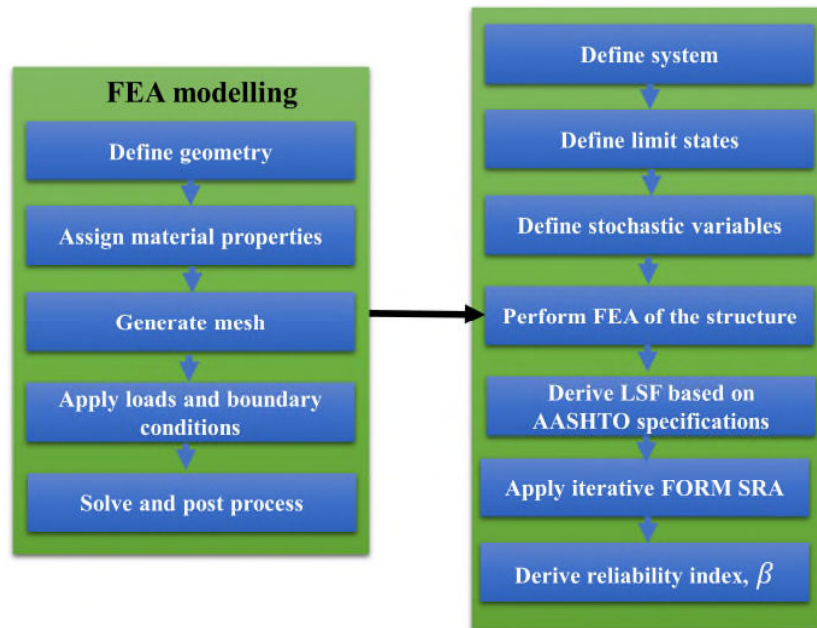


Figure 4. Flowchart of the purpose-developed SRA framework.

4 Results and discussion: FEA, design and stochastic optimisation

4.1 FEA results

The FEA results to calculate the critical structural responses, the maximum displacement, U_{max} (mm) and maximum critical stress, σ (kN/m²) employing the system I and II stress concept is presented in this Subsection. This will enhance the calculation of the structural response of interest using the various AASHTO LS design criteria. The structural behaviour of the deck was studied while varying salient normal and centrifugal stresses at various geometric properties.

4.1.1 FEA modelling on ABAQUS©

The deck was analysed using FEA software ABAQUS©. Figure 5(a) depicts the part; the case study considered - a horizontally curved aluminium alloy deck - Alumadeck™ – radius = 100m, subtended angle = 34.4° while Figure 5(b) shows the 3D zoomed display of vertical and inclined stiffeners. The mesh generated was selected, taking into account a trade-off between computational cost and precision.

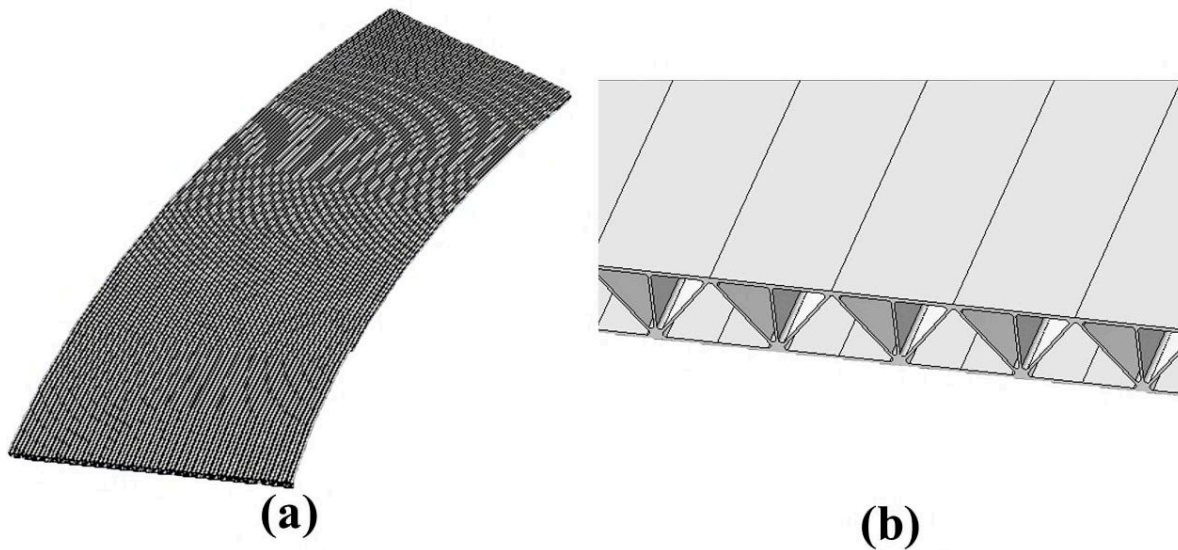


Figure 5. The developed horizontally curved bridge deck model in the ABAQUS© environment: (a) Built 3-D model (b) 3-D display of the vertical and inclined stiffeners.

As these figures depict, the Alumadeck™ and the section as built-in ABAQUS© are elucidated. The hexahedral solid element feature within the ABAQUS© configuration settings was adopted. The material properties of the Alumadeck™ were assigned in the model thus: (1) The Young's modulus assigned was $68.9GPa$, and the Poisson's ratio was 0.33. (2) The density and the yield stress were also assigned. The loading configurations entered into the model applied on the deck are according to HL-93 load AASHTO specifications.

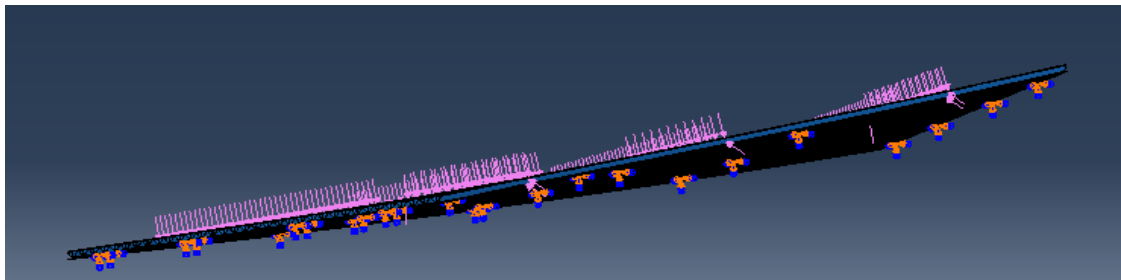


Figure 6. Loading and Boundary Conditions

The loadings and boundary conditions are depicted in Figure 6. The finite element analysis was performed in Abaqus. A high-performance computer (HPC) was used for the simulation, which ran for 5minutes.

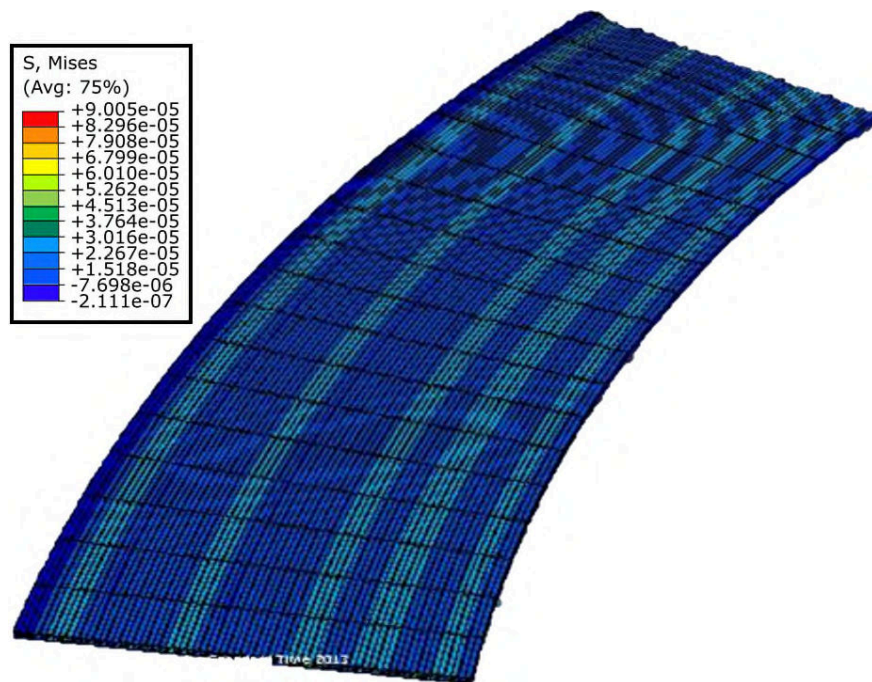
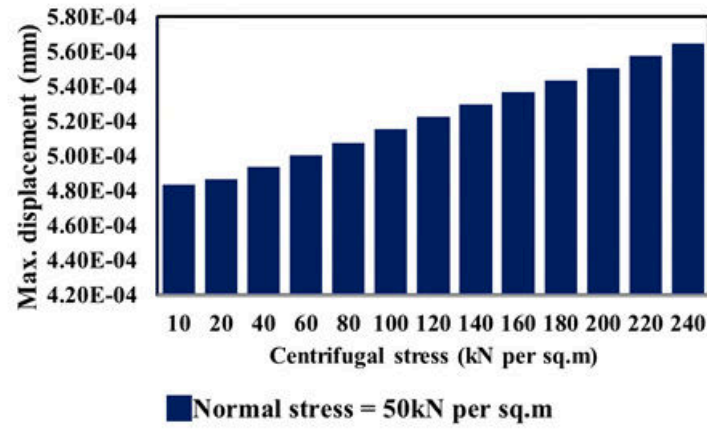
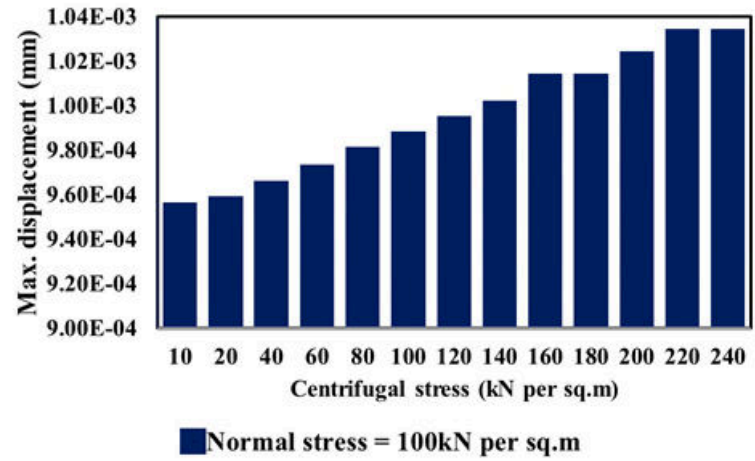


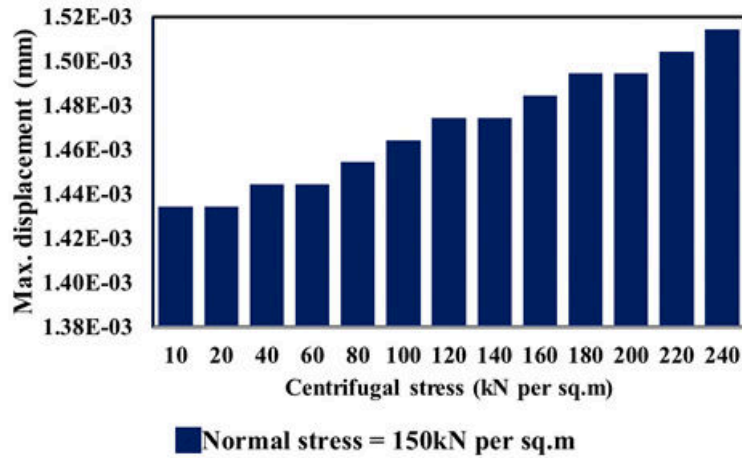
Figure 7. 3D ABAQUS© model result display the contours of Von Mises stress



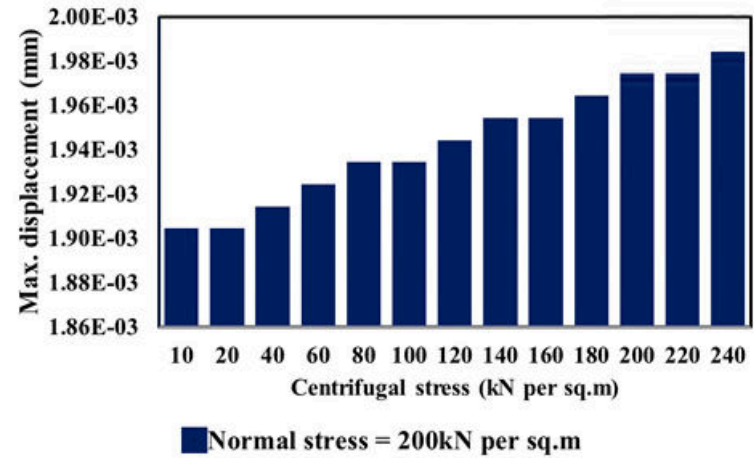
(a)



(b)

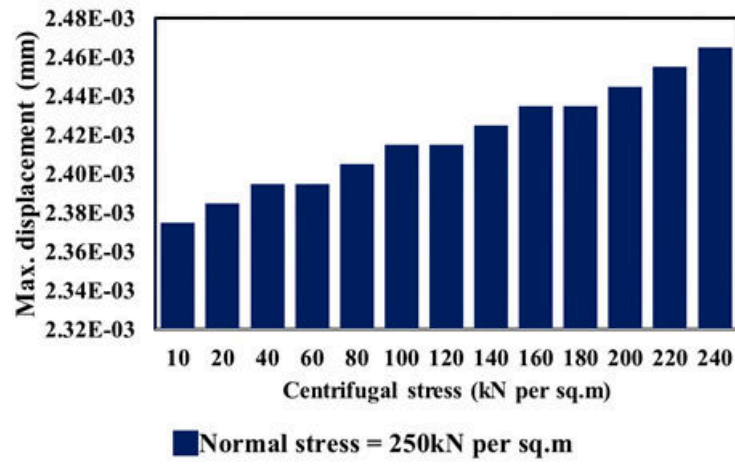


(c)

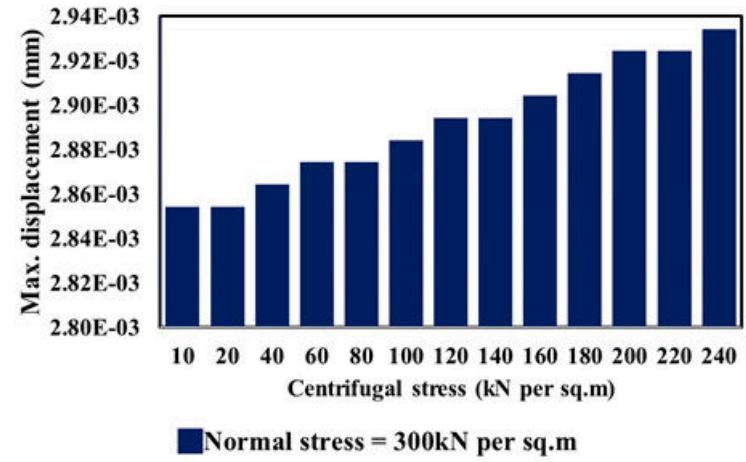


(d)

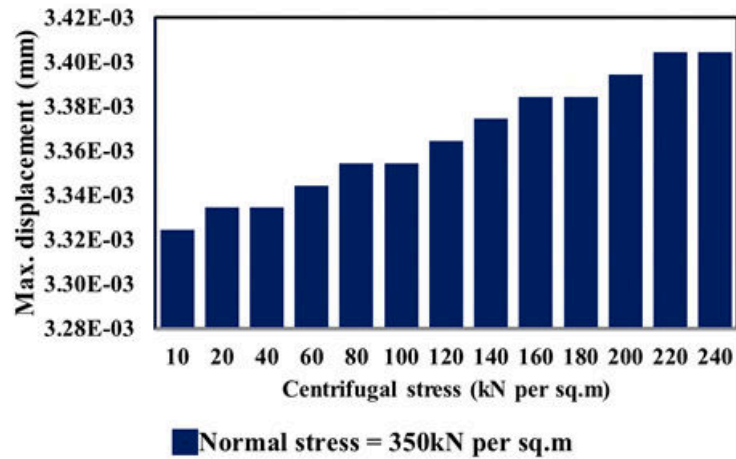
Figure 8. Variation of maximum displacement (mm) against centrifugal stress (kN/m^2) – $t = 5mm$



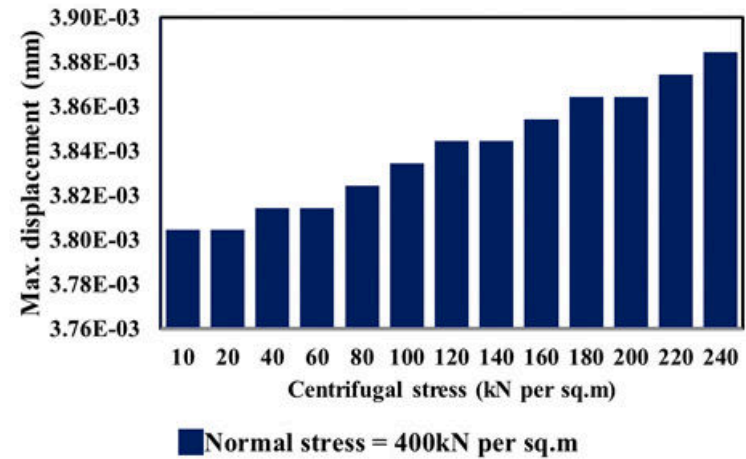
(a)



(b)

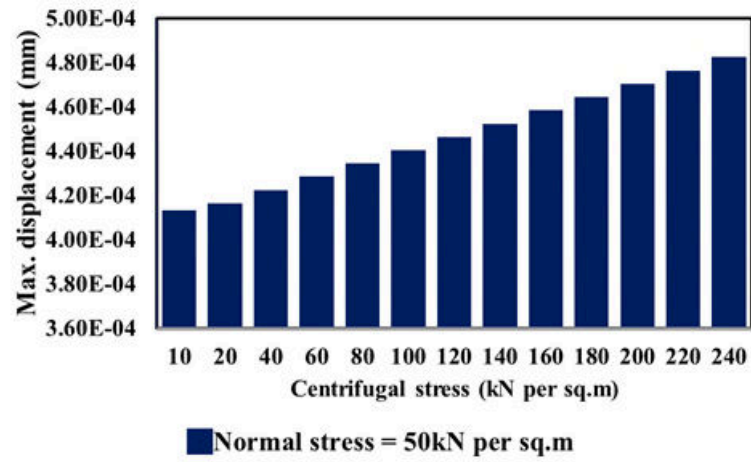


(c)

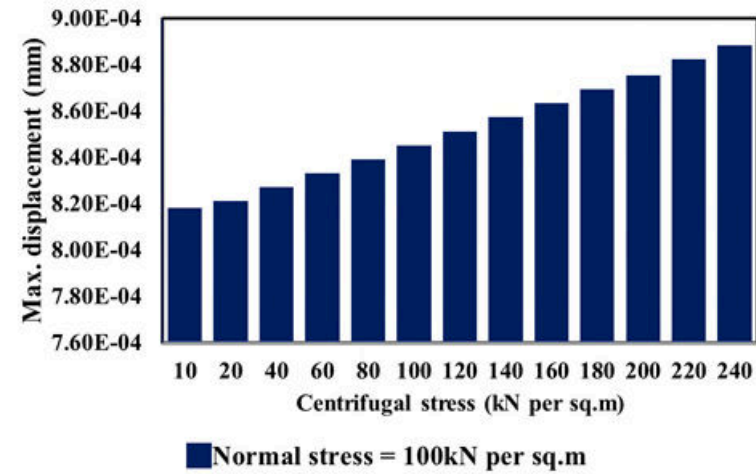


(d)

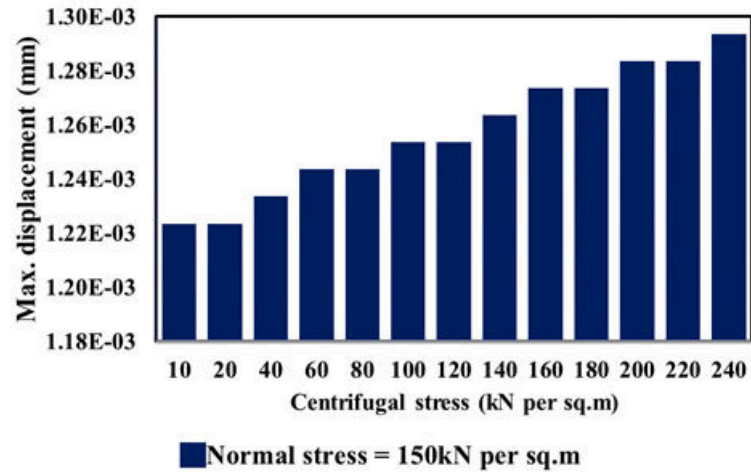
Figure 9. Variation of maximum displacement (*mm*) against centrifugal stress (kN/m^2) – $t = 5mm$



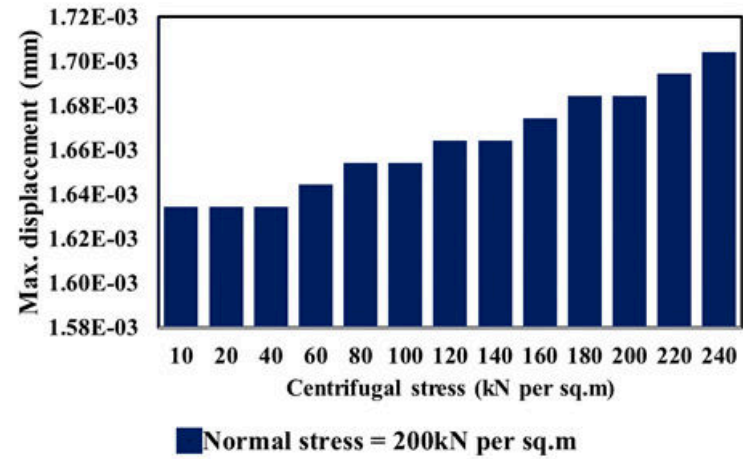
(a)



(b)



(c)



(d)

Figure 10. Variation of maximum Von Mises stress (kN/m^2) against centrifugal stress (kN/m^2) - $t = 5mm$

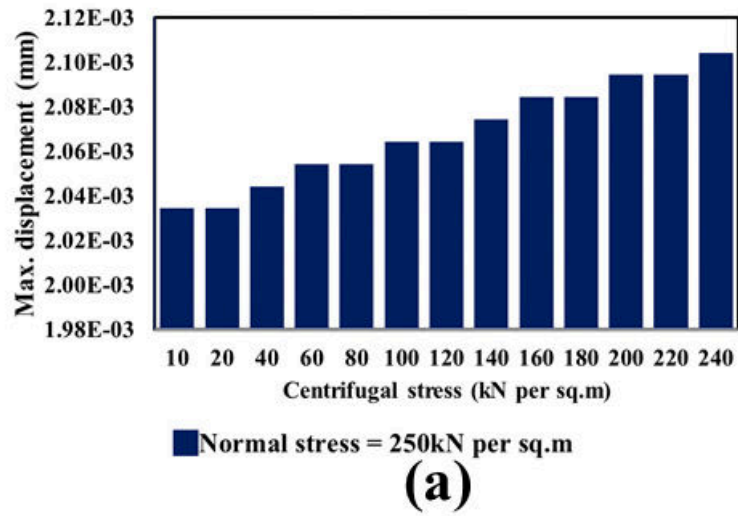
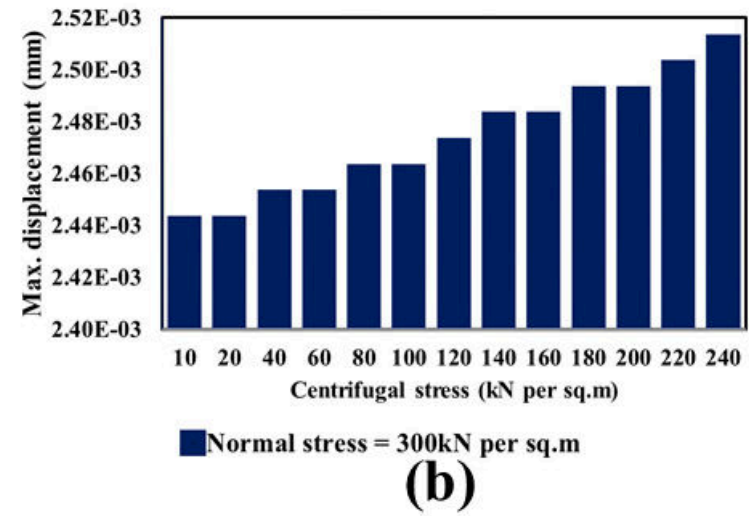
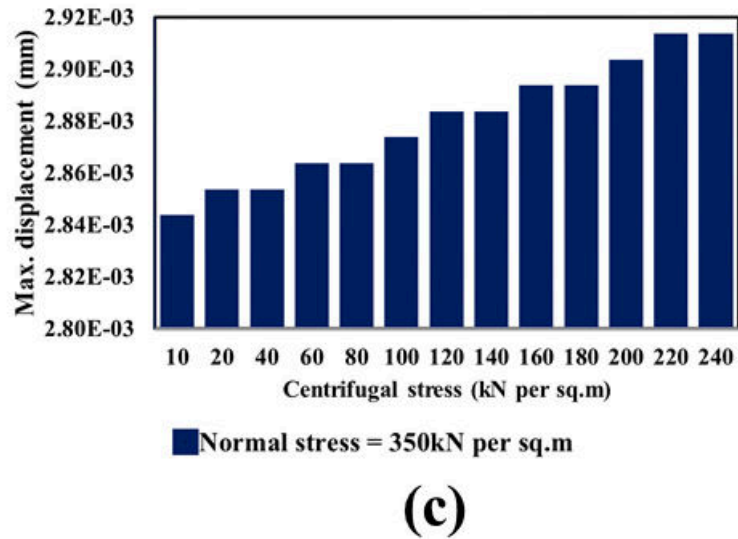
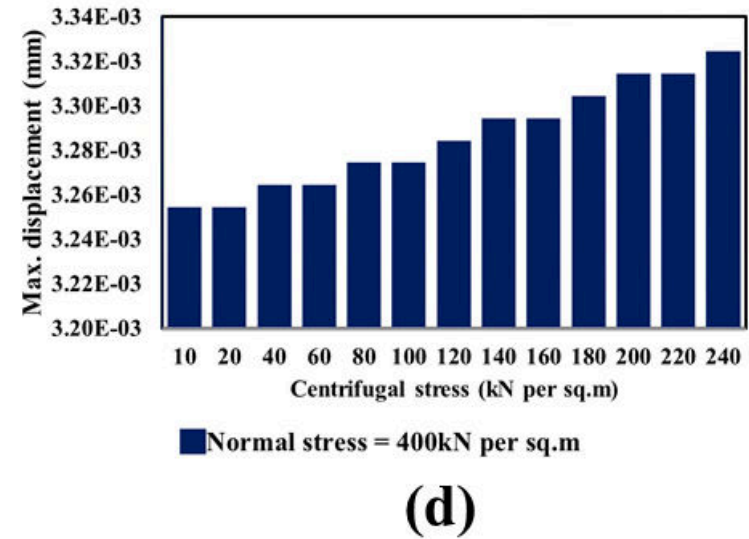


Figure 11. Variation of maximum Von Mises stress (kN/m^2) against centrifugal stress (kN/m^2) - $t = 5mm$

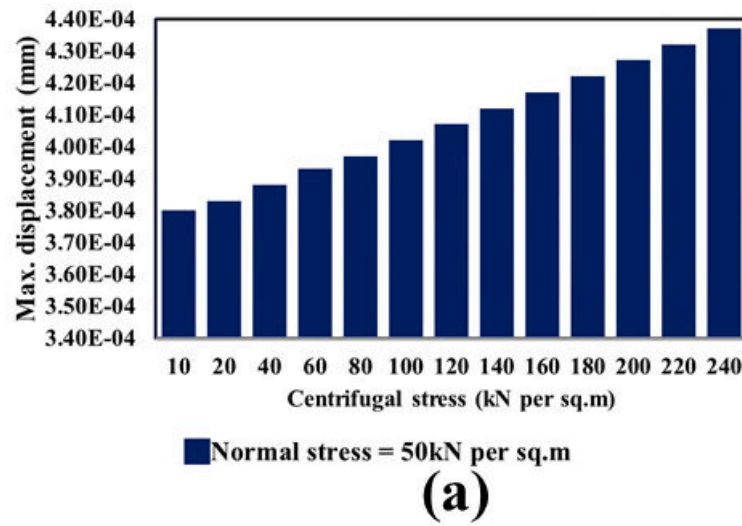
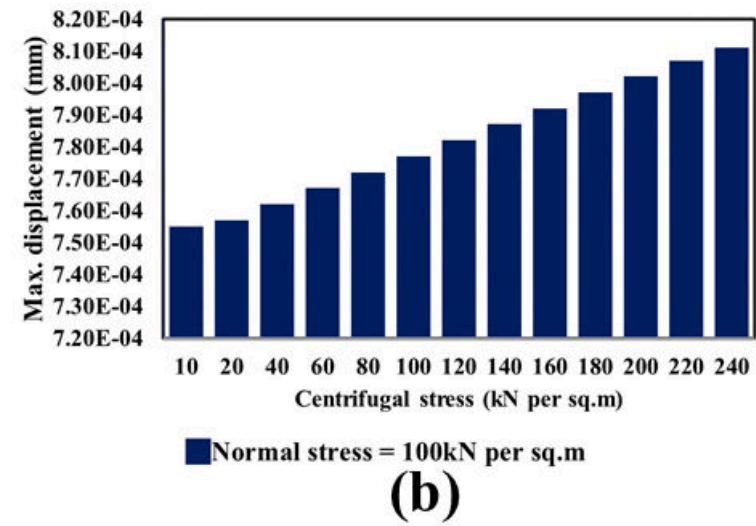
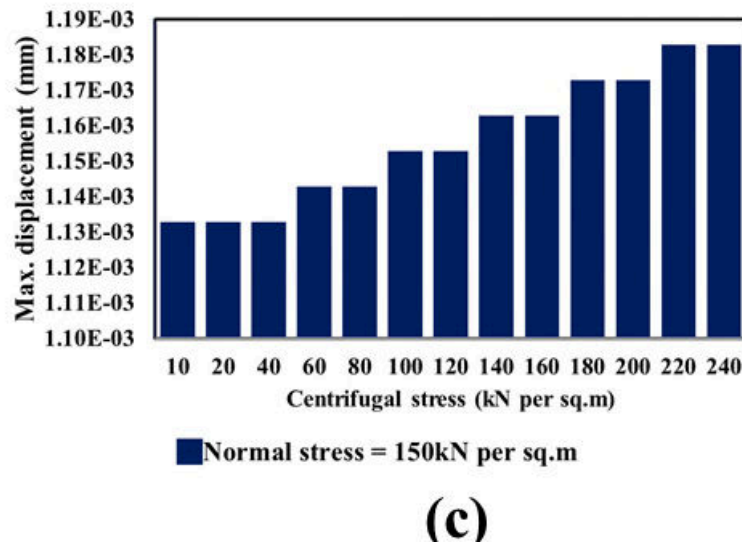
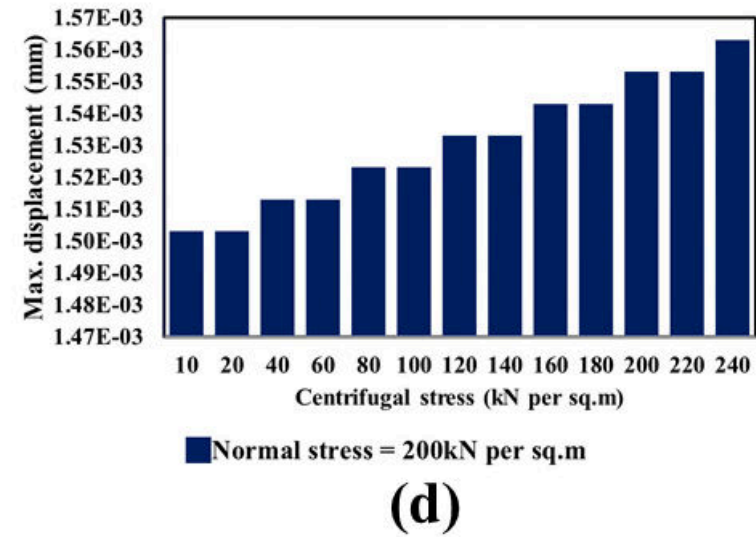
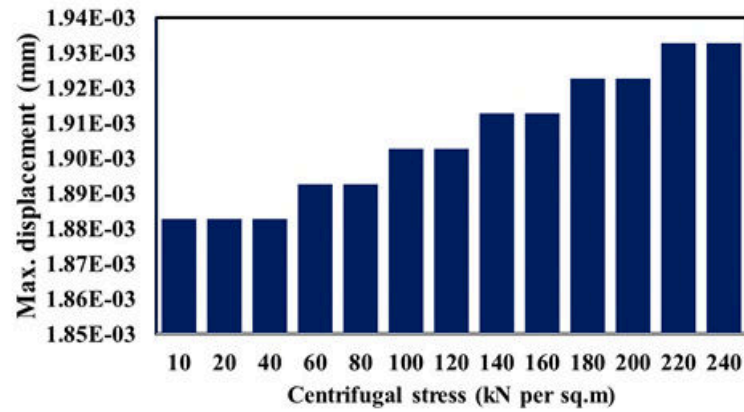
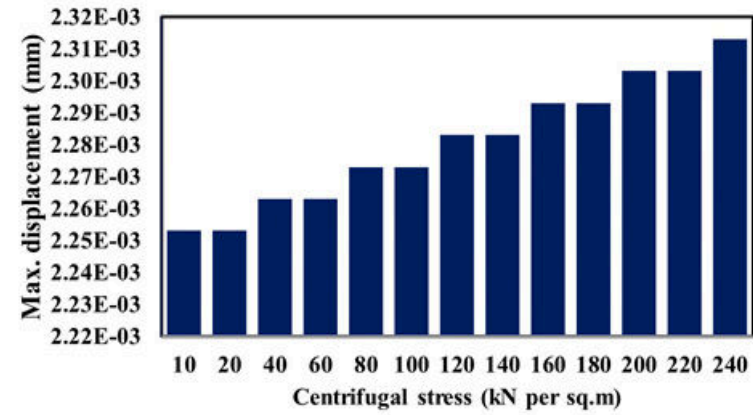


Figure 12. Variation of maximum displacement (mm) against centrifugal stress (kN/m^2) - $t = 7mm$



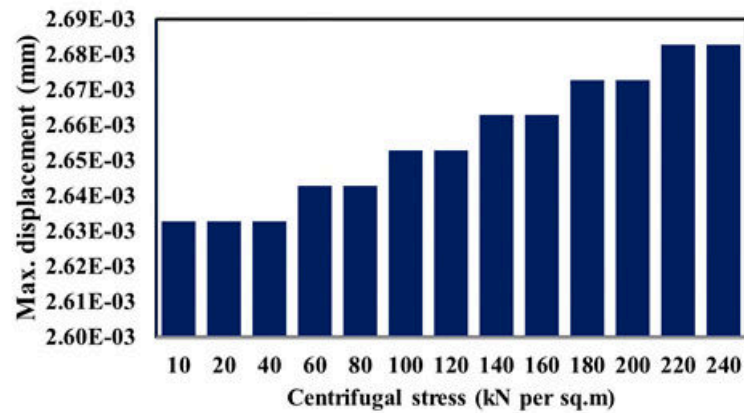
■ Normal stress = 250kN per sq.m

(a)



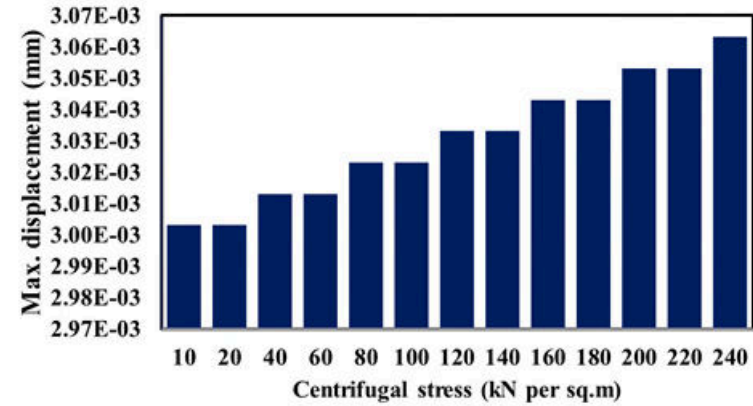
■ Normal stress = 300kN per sq.m

(b)



■ Normal stress = 350kN per sq.m

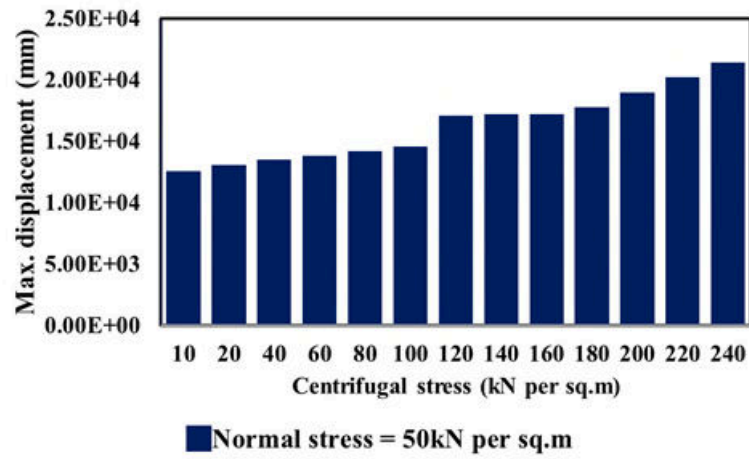
(c)



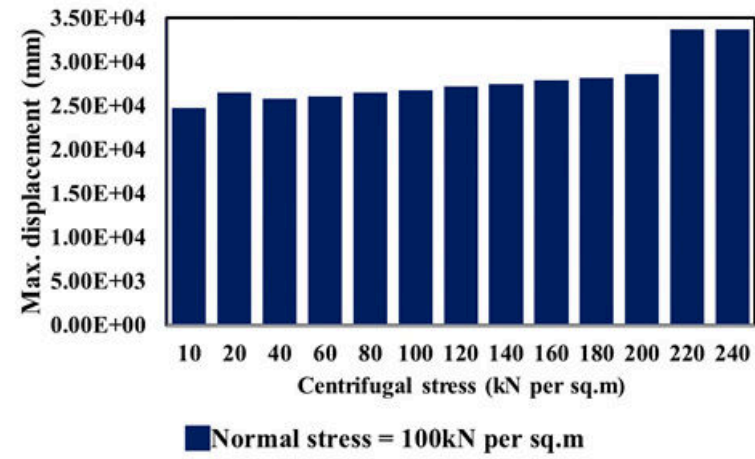
■ Normal stress = 400kN per sq.m

(d)

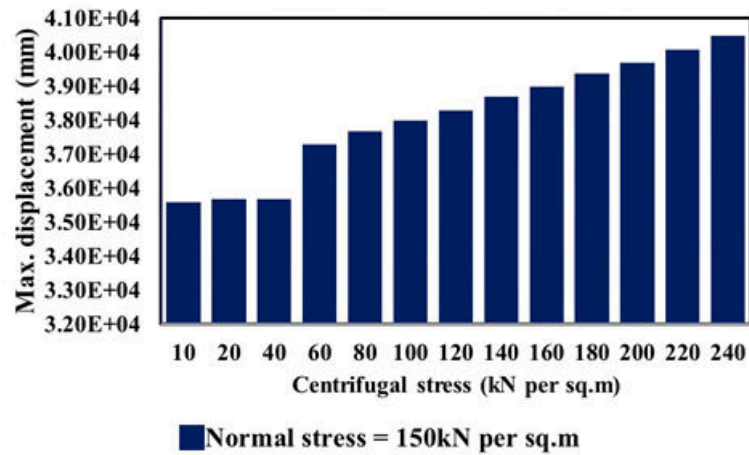
Figure 13. Variation of maximum displacement (mm) against centrifugal stress (kN/m^2) - $t = 7mm$



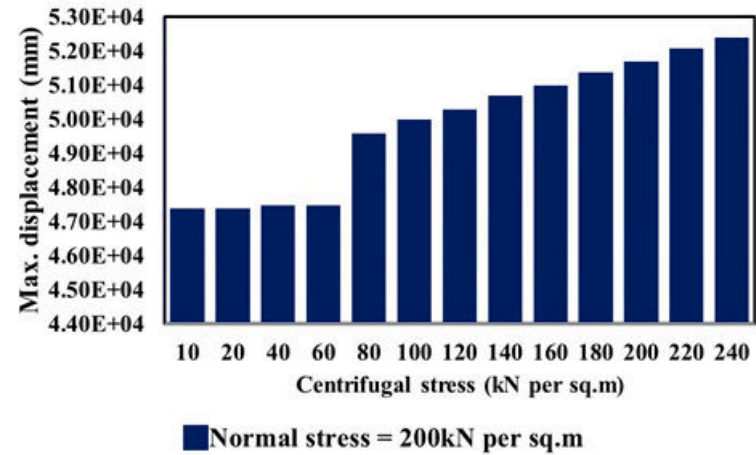
(a)



(b)

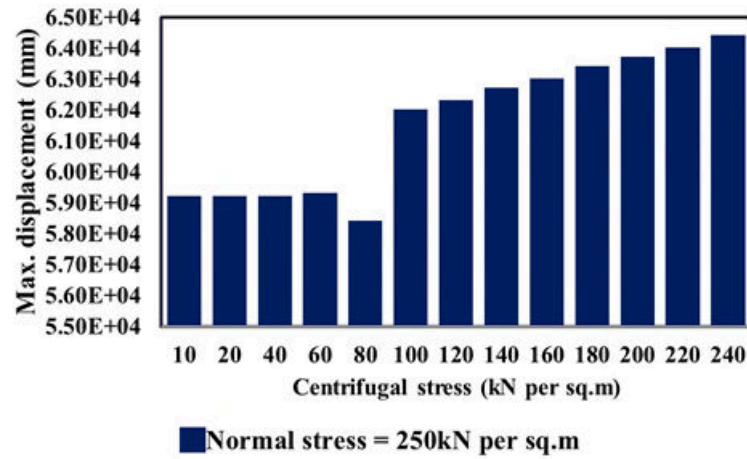


(c)

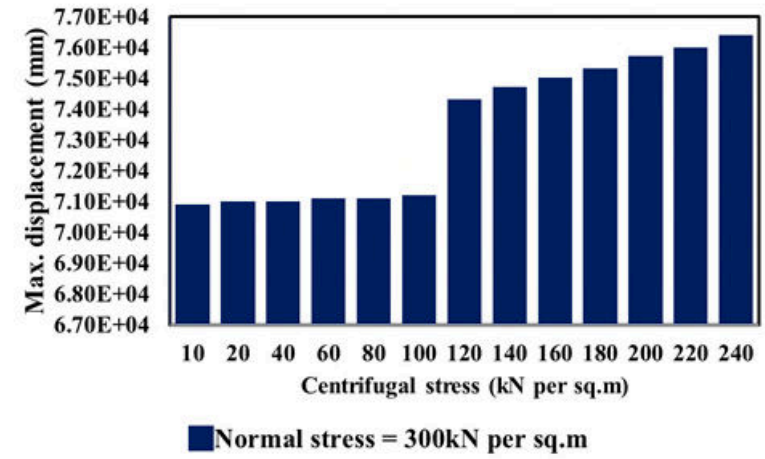


(d)

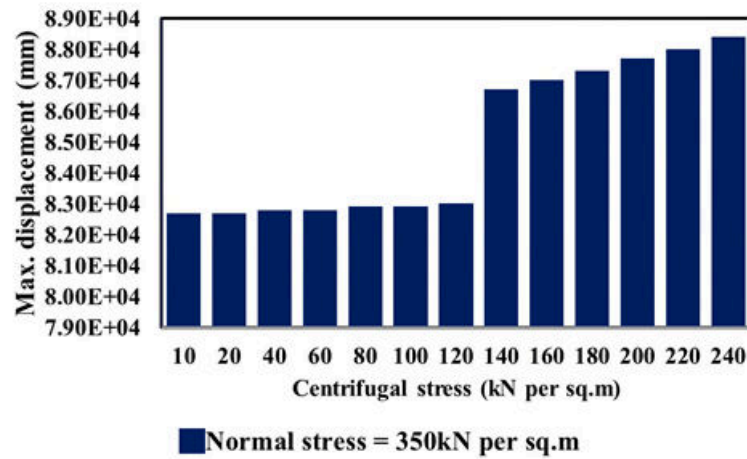
Figure 14. Variation of maximum Von Mises stress (kN/m^2) against centrifugal stress (kN/m^2) - $t = 7mm$



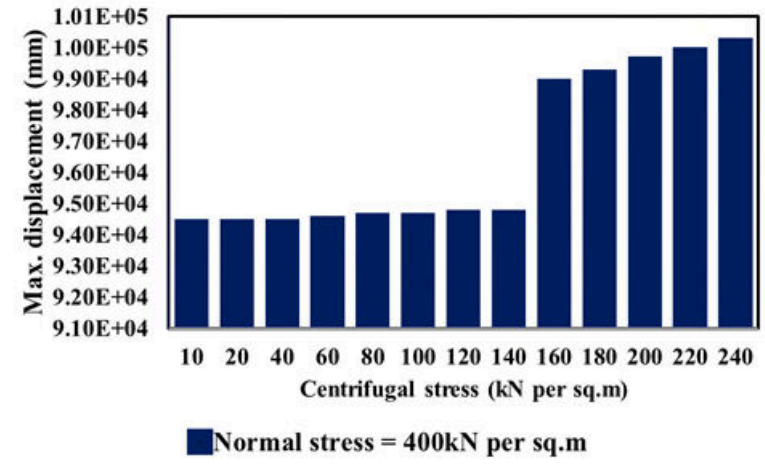
(a)



(b)

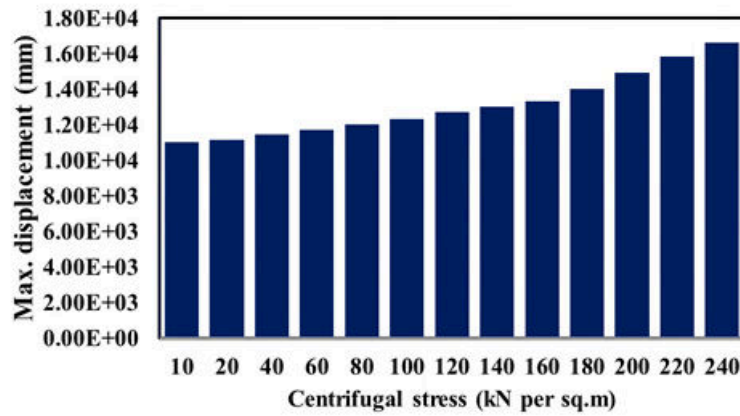


(c)



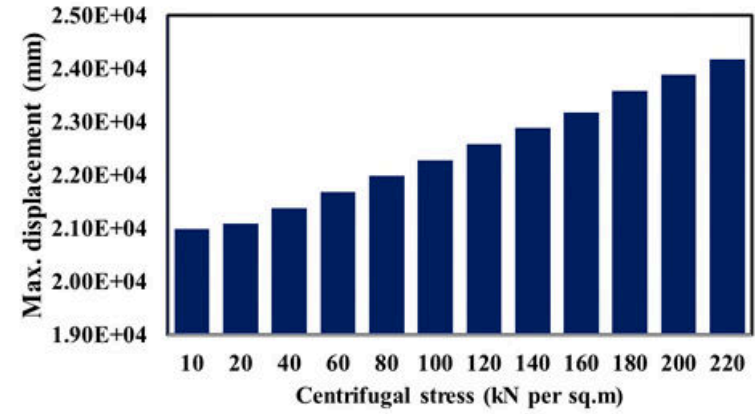
(d)

Figure 15. Variation of maximum Von Mises stress (kN/m^2) against centrifugal stress (kN/m^2) - $t = 7mm$



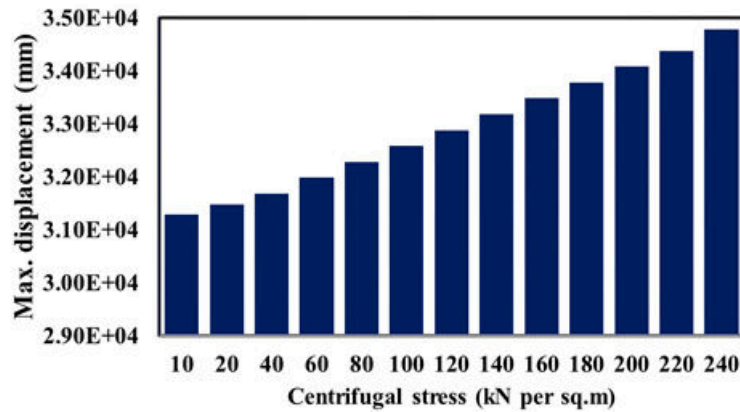
■ Normal stress = 50kN per sq.m

(a)



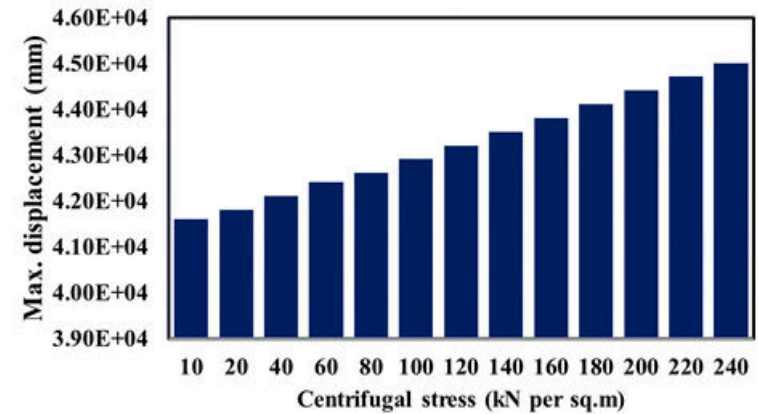
■ Normal stress = 100kN per sq.m

(b)



■ Normal stress = 150kN per sq.m

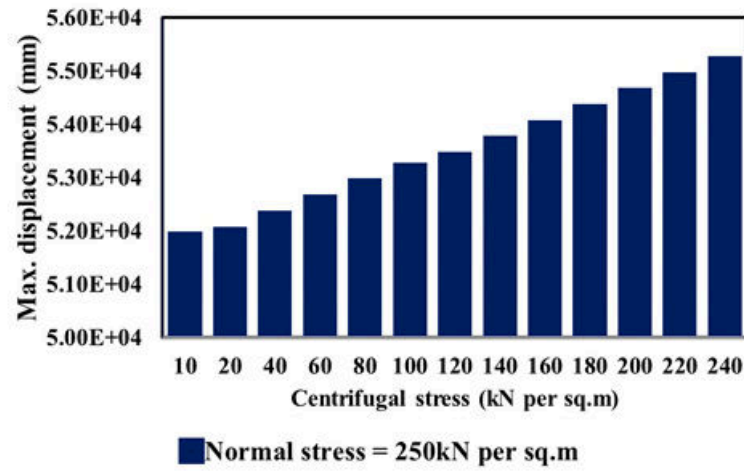
(c)



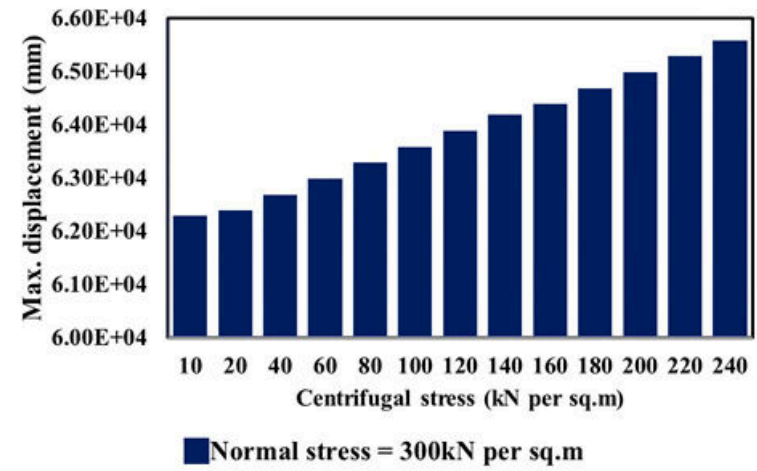
■ Normal stress = 200kN per sq.m

(d)

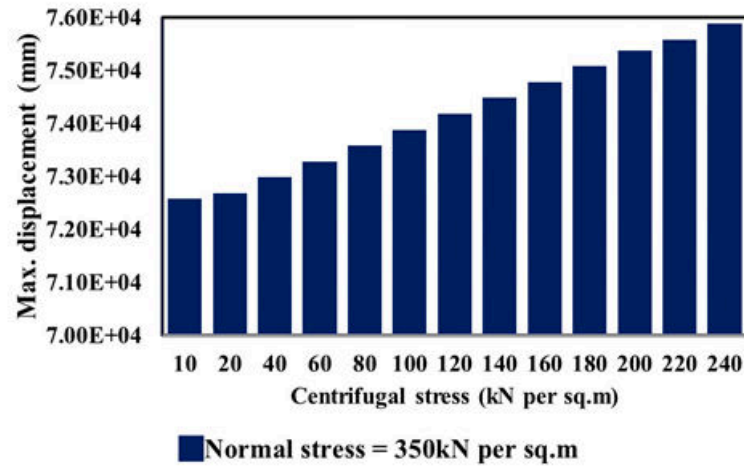
Figure 16. Variation of maximum displacement (mm) against centrifugal stress (kN/m^2) - $t = 9mm$



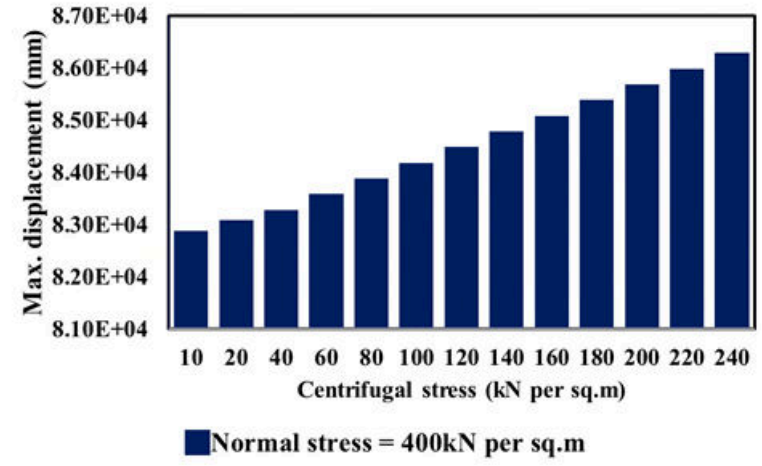
(a)



(b)



(c)



(d)

Figure 17. Variation of maximum displacement (mm) against centrifugal stress (kN/m^2) - $t = 9mm$

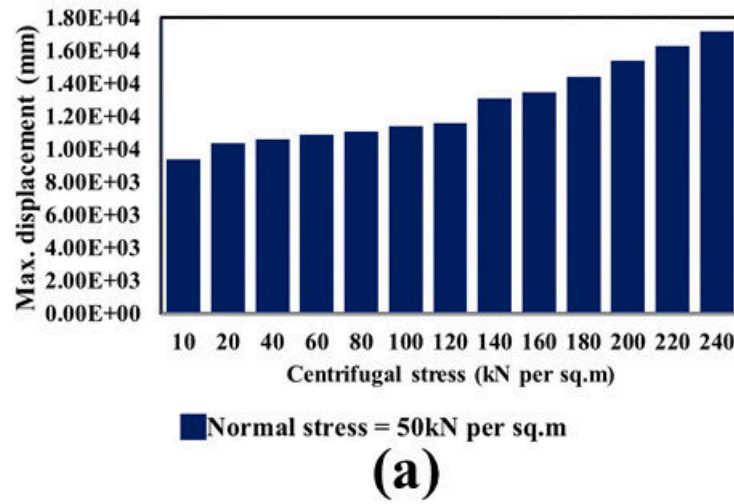
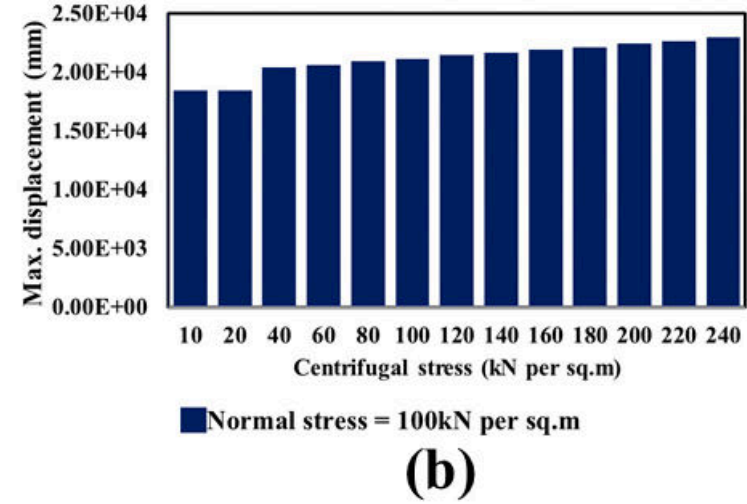
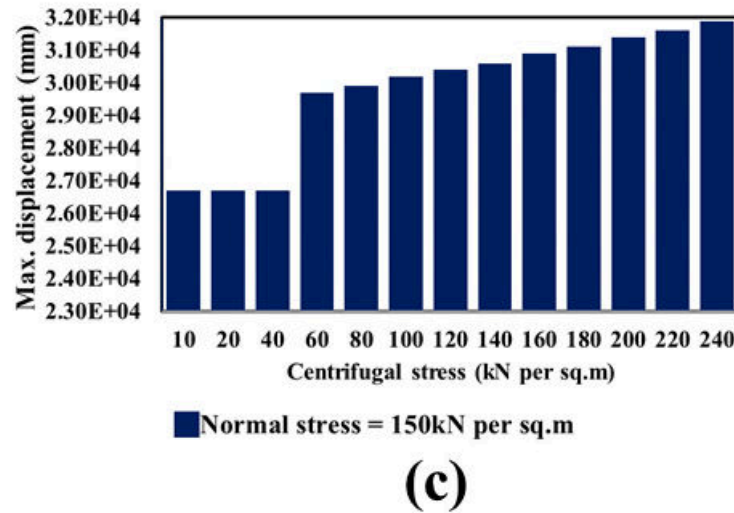
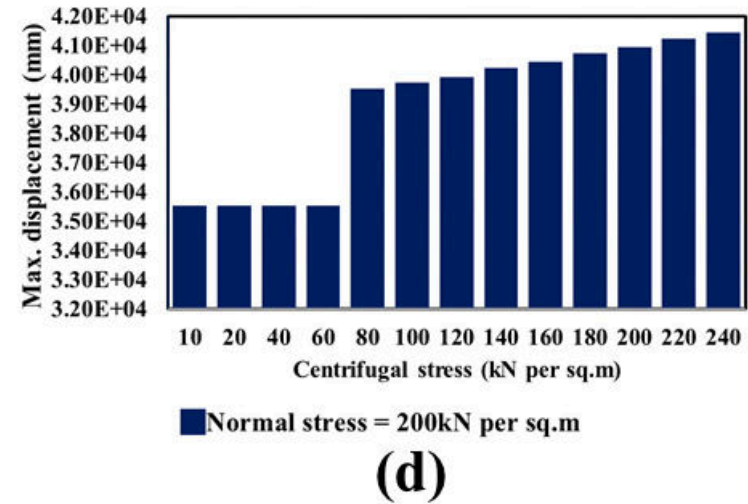
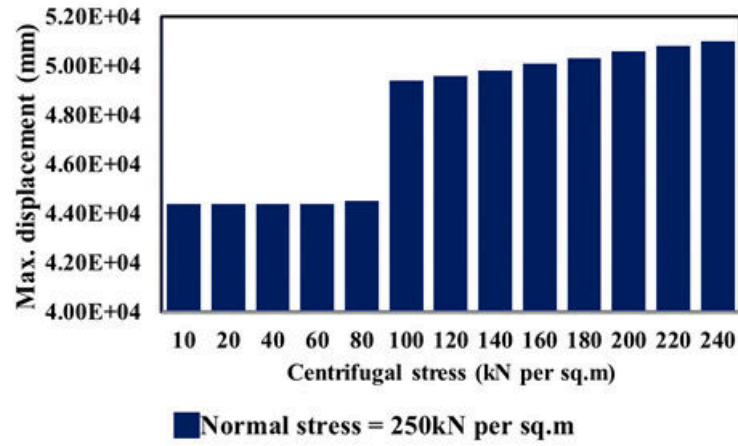
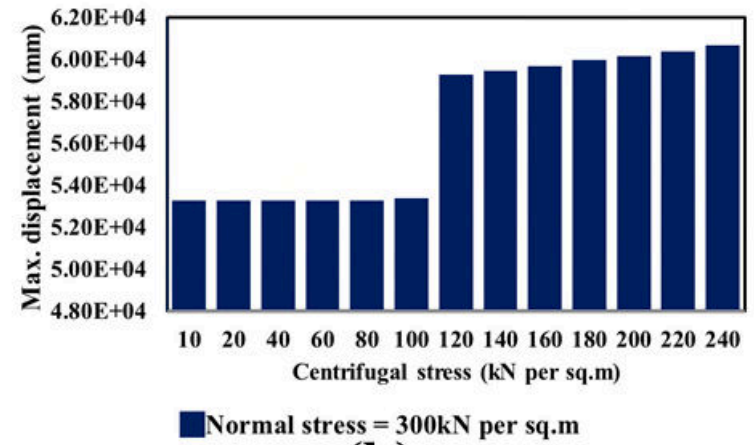


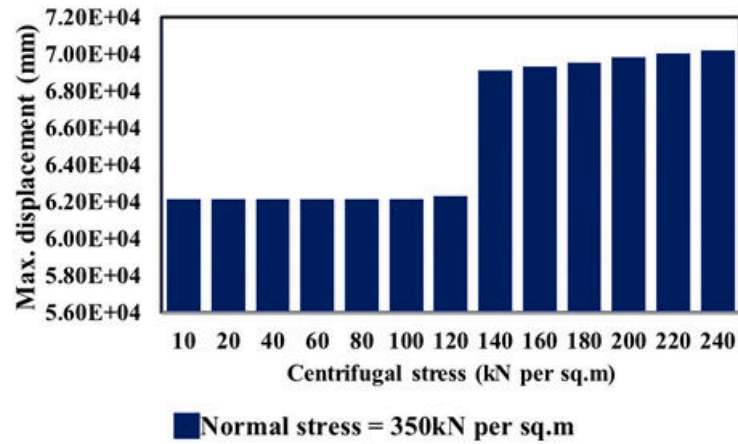
Figure 18. Variation of maximum Von Mises stress (kN/m^2) against centrifugal stress (kN/m^2) - $t = 9mm$



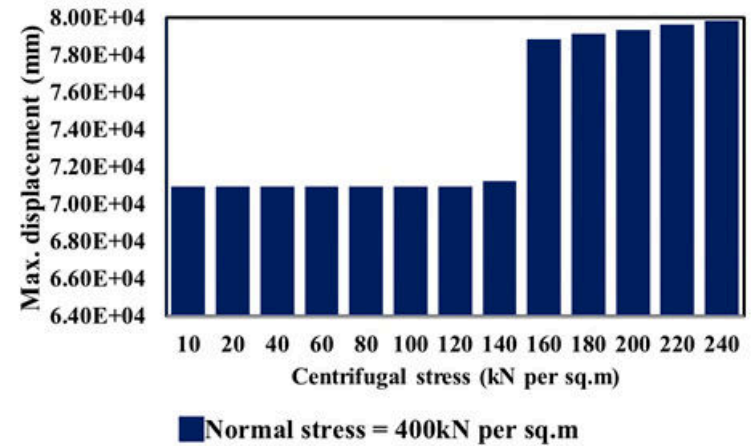
(a)



(b)



(c)



(d)

Figure 19. Variation of maximum Von Mises stress (kN/m^2) against centrifugal stress (kN/m^2) - $t = 9mm$

In an FEA case study, as depicted in Figure 7-Figure 19, results show that the maximum displacement of the Alumadeck™ increases linearly with an increase in the centrifugal stresses signifying a positive correlation. A majority of the maximum Von Mises stresses calculated were within acceptable limits since these are less than the factored yield stress of the deck (considering the Von Mises criterion, with reference to Subsection 3.1.1), and this is a measure of the durability of the Alumadeck™. The maximum displacements obtained for the 5mm, 7mm and 9mm stiffner thicknesses are 0.004mm, 0.0033mm and 0.003mm, respectively. Based on these results, comparing with the allowable displacement prescribed in design standards [16,17,41] for all plates, it is recommended that the thickness of stiffners should be a minimum of $t_{stiffner} = 7mm$ to have a durable structure since some of the maximum displacements for lower dimensions, such as for $t_{stiffner} = 5mm$ was not satisfactory. Hence, the displacements are acceptable for the selected geometric dimensions employed for the SRA later in the upcoming Subsection 4.2, indicating the durability of the Alumadeck™ adopting these properties.

Furthermore, this case study revealed that the measured stiffner thickness assumed to suffice to satisfy the durability criteria can optimise the deck structure in terms of reducing the structure's self-weight, saving manufacturing costs of more than 25% when compared with values suggested in [3,4,23].

4.1.2 Validation

In order to validate the structural FEA model developed, the results of the deformation analysis at salient points on the structure were compared against results from structural mechanics equations (i.e. Appendix (A1)-(A14**Error! Reference source not found.**) summarised in Subsection 3.3 as coded in MATLAB) for the same test structural configuration set-up. Results of the assessment show that there is good agreement in the deflection calculated using the analytical method compared to the developed FEA model approximately predicted (since the percentage difference calculated is less than ten (10%) with reference to Table 1), which demonstrates the scientific soundness of the FEA model adopted herein. Response surface modelling (RSM) methodologies are now extensively used within the field of SRA, as noted by several studies [7–12,42,43], owing to their capability of enhanced LSF

approximations with high fidelity. LSF concepts as derived from these sources are adapted in the present study as presented in Subsections 4.2 and 4.3.

Table 1. Static deformation of the curved bridge deck at midspan

Analytical model	Numerical model	% Difference
1.25E-03m	1.15E-03m	9%

4.2 Stochastic assessment

The SRA of the aluminium alloy deck system is evaluated for the AlumadeckTM–steel girder section considering the system I and II, applying the validated model in Subsection 4.1.2. The composite is designed per the AASHTO guidelines [16]. Figure 20 illustrates a schematic depicting the configuration of the girders stiffened and the cross frames.

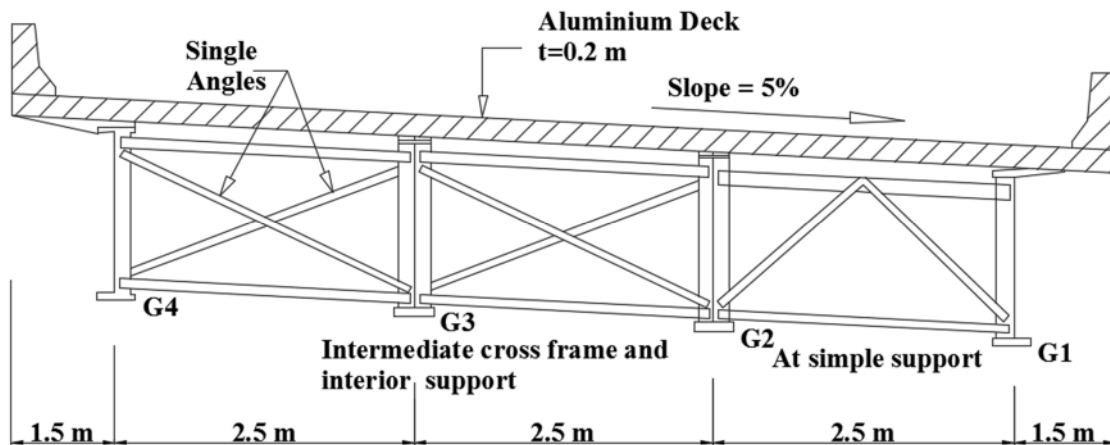


Figure 20. I-girder bridge cross-section showing the cross frames.

4.2.1 Bridge deck design parameters and design data for I-girder G4

As derived from Heins [44], the minimum radius for the design of a horizontally curved bridge is 100m. The following parametric values defined in Appendix (B1) were assumed for the curved deck with reference to Figure 2 and used in the assessment. The composite (i.e. the deck and I-girder) is designed against the strength LS. Also, the outermost girder (G4), which is the most critical, is analysed. Eurocode 9 [41] stipulates that the tensile yield strengths should be multiplied by 0.8. According to AASHTO [17], the dead load is subdivided into three categories: (1) Dead load of a structural

component (DC): This is divided into DC1 and DC2. DC1 accounts for the weight of the deck and the steel, while DC2 accounts for the dead weight of the parapets. For the aluminium alloy deck: $DC1 = 26.9 \times 0.2 = 5.38 \text{ kN/m}$, while for the steel girder: $DC1 = 78.5 \times 0.021 = 1.65 \text{ kN/m}$. (2) Dead load wearing course (DW): This accounts for the future wearing course. Thus, $DW = 6.73 \times 0.0095 = 0.064 \text{ kN/m}$.

The AASHTO vehicular live load is designated as HL-93 loading and is a combination of a design truck or a tandem plus the design lane load. The design truck specified comprises a 36 kN lead axle spaced 4.287 m from the closer of the two 144 kN rear axles, which has variable spacing from 4.287 to 9.144 m . The transverse spacing is 1.83 m . The design truck occupies 3.045 m lane width, is positioned within the design lane to produce maximum force effect, and must not be closer than 0.61 m from the face of the edge of the design lane.

4.3 Bridge component design and derivation of LSFs

This Subsection conducts detailed calculations for girder flexural design based on the AASHTO [16]. In addition, the design is carried out for the critical section for girder G4 for the strength LS under the strength-I and II load combination on the completed structure. Further advanced design and stochastic optimisation calculations concerning this, as carried out by the same author, can be found in Wang et al. [22].

4.3.1 Girder stress check the critical section on G4 – section proportioning

For a web without longitudinal stiffeners, the web is proportioned such that: $\frac{D}{t_w} \leq 150$; $\frac{2.358}{0.016} = 147.38 < 150$ **OK**. Compression and tension flange are proportioned such that: $\frac{b_f}{2t_f} \leq 12$; $\frac{50}{2(2.1)} = 11.9$ – thus, both flanges **OK**. $b_f \geq \frac{D}{6}$; $\frac{2.358}{6} = 0.393 \text{ m} = 39.3 \text{ cm}$ – thus, both flanges **OK** $t_f \geq 1.1t_w$; $1.1(1.6) = 1.76 \text{ cm}$, both flanges **OK**. $0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$; $I_{yc} = I_{yt} = 2.1 \times \frac{(50)^3}{12} = 21875 \text{ cm}^4$; $\frac{I_{yc}}{I_{yt}} = 1$ – **OK**.

Therefore, from the foregoing, it can be inferred that all section proportions for this location are satisfied.

Other detailed calculations for the girder stress checks can be found in Appendix A.1.1-A.1.4. The LSF for the top flange of a transversely stiffened web (i.e. considering the strength LSS) can be expressed as Eq (15):

$$\begin{aligned}
 G(x) &= \phi_f F_{yc} - f_{bu} \\
 &= \phi_f F_{yc} \\
 &\quad - \left[\frac{1.25 M_{DC1} C_{nc}}{I_{nc}} + \frac{(1.25 M_{DC2} + 1.5 M_{DW}) C_{3n}}{I_{3n}} \right. \\
 &\quad \left. + \frac{1.75 M_{LL} C_n}{I_n} \right] 12\eta.
 \end{aligned} \tag{15}$$

The LSF for strength criteria considering transversely stiffened web – strength LS – bottom flange can be expressed as Eq (16)

$$G(x) = \phi_f M_n - M_u - \frac{f_l S_{xt}}{3} \tag{16}$$

or,

$$G(x) = \phi_f f_{nt} - f_{bu} - \frac{f_l}{3}. \tag{17}$$

The LSF considering transversely stiffened web – shear strength – web is expressed as Eq. (18)

$$G(x) = \phi_v V_n - V_u. \tag{18}$$

Having completed the critical design exercise as elucidated in Appendix A.1.1-A.1.4, and Subsection 4.3, we now proceed into the stochastic optimisation by deriving the LSF – Eq. (15)-(18) and then performing the FORM simulation executed in MATLAB applying the stochastic variables defined in Appendix B2ERROR! REFERENCE SOURCE NOT FOUND. to calculate the reliability indices. The HL-RF recursive FORM algorithm developed by the same authors in [8–12] is adapted for the present study. The iterative FORM algorithm has been validated against the MCS in previous studies [9–11]. The well-established FORM is used as it offers a good balance between efficiency and accuracy for realistic problems. One limitation of the MCS method is that many simulations are required (i.e. requiring an excessive

computational effort) to calculate the very low values of failure probability, which is a characteristic of complex structures [8–12]. The FEA and SRA results of this study considering the strength LS are depicted in see Figure 7-Figure 21::

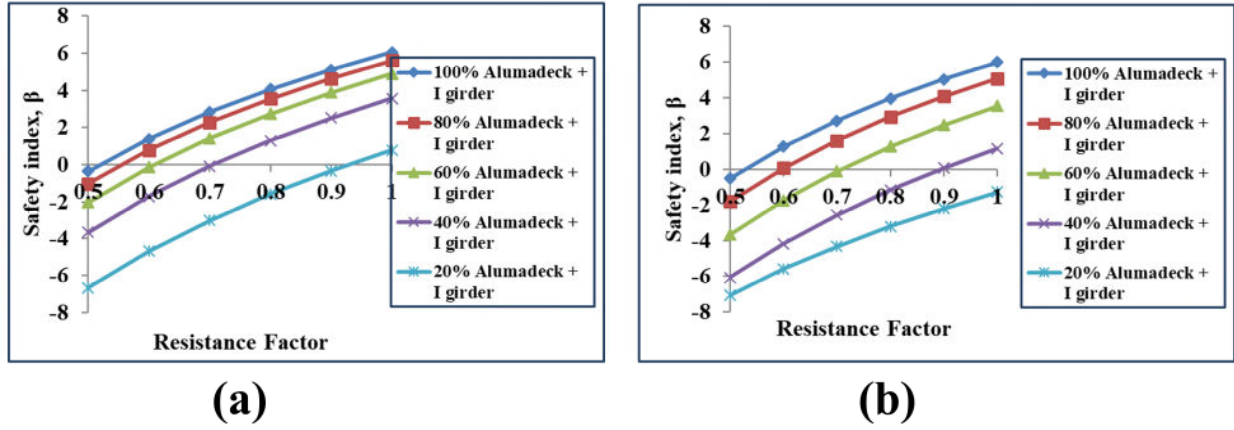


Figure 21. RI at various RFs at varying AlumadeckTM-I-girder composite action: (a) Considering the failure of the top flange, (b) Bottom flange.

The compound action is guaranteed by means of a haunch, grouted core and stud shear connectors.

Eurocode-4 stipulates that the design strength of stud connectors are as follows:

$$P_{RD} = 0.8f_u \left(\frac{\pi d^2}{4} \right) / \gamma_v \quad (19)$$

$$P_{RD} = 0.29\alpha d^2 \sqrt{f_{ck} E_c} / \gamma_v \quad (20)$$

The smaller of the two Equations should be chosen. According to the ASHTO LRFD another way to calculate the ultimate shear resistance of shear connectors is as follows:

$$Q_r = 0.5A_s \sqrt{f_c' E_c} \leq A_s F_u \quad (21)$$

Where f_u is the tensile strength of the stud material; d is the diameter of the shank of studs; f_c' , f_{ck} is the characteristic compressive strength; E_c is the elastic modulus; A_s is the area of the shank section of studs; $\alpha = 1.0$; $\gamma_v = 1.0$ is the partial factor.

4.3.2 Stochastic optimisation

This Subsection presents the results of the optimisation exercise, Figure 7- Figure 21: On completion of the probabilistic assessment, the result recorded that the RI is sensitive to the RF, and the RI also increases linearly with an increase in RF, signifying a positive correlation. Also, the results reveal that the design is not acceptable for an RF of 0.5 at 100% and 80% composite action in both compression and tension flanges. The design is also unacceptable for an RF of 0.6 and below at 60% composite action for both flanges; 0.7 and below at 40% and completely unacceptable at 20% composite action. For 80% composite action at an RF of 0.8, the safety index is 3.55, considering the failure of the compression flange. The tension flange is 3.0, which is ideal to guarantee the structure's safety. This shows that an I Girder of a depth of 2.4m, flange thicknesses of 21mm, flange width of 500mm and web thickness of 16mm is safe to carry the truck loads. The SRA framework developed herein will be an invaluable tool for designers with respect to reliability-based design optimisation. Bridge industry practitioners can harness the framework's capability to address the problem of optimising the manufacturing expenditure, which is an issue of priority and hence allows cost reduction activities needed to ensure structural safety and efficient performance of bridge structures.

Research into the structural application of aluminium alloys in civil engineering structures is disappointingly sparse. Most future developments are likely to arise from advances in other applications, such as the aerospace and transport industries, with different criteria to address. Traditionally, castings have been discouraged for structural applications, as many castings' quality and brittle characteristics have been unsuitable. The automotive and aerospace industries have developed the use of high-precision castings that have good ductility and are increasingly using these in combination with purpose-designed extrusions. Such methods are particularly appropriate for volume production. The technology is likely to be useful in civil engineering if sufficient quantities can justify developing and tooling costs for any specific project or proprietary system. Development of material technology continues. New processes using fibre-reinforced aluminium and powder metallurgy promise stronger and stiffer alloys than the current alloys in structural use. The increase in stiffness will be particularly relevant. Technology for joining materials is an area of research and development that

transcends different industries quite well. Results in friction stir welding and adhesive bonding over the past ten years have yet to become firmly established in the civil engineering industry. They are not covered in detail by the current design codes. They are, however, well established in the manufacture of railway carriages and military bridging.

5 Conclusion

In this study, the FE and SRA of a horizontally curved aluminium alloy bridge deck on steel I-girders is presented. To calculate the critical response of the structural assembly, an FEA model was developed in the ABAQUS© environment (a well-established FEA modelling software, thanks to its high fidelity and precision). A probabilistic analysis was conducted using the FORM coded in MATLAB to predict the RI (β) at various percentages of composite actions while varying geometric dimensions. From the results revealed herein, the following conclusions could be inferred:

- A novel FEA model of a horizontally curved AlumadeckTM was developed to calculate the critical responses of the structure in the presence of truck HL-93 loading configuration.
- In a case study, it was revealed from FEA response results that the thickness of stiffeners, $t_{stiffner}$ should be a minimum of 7mm to have a safer structure since some of the maximum displacements for the 5mm were not satisfactory, which could result in savings of more than 25% in production costs compared with what is obtained in the existing design
- These critical responses predicted using the FEA is entered into the LSF subroutine (derived via AASHTO LRFD flexural resistance formulation) of the stochastic model in an HL-RF iterative FORM algorithm executed in the MATLAB environment
- It is shown that the propriety AlumadeckTM system conforms with the load resistance factor design (LRFD) specification, which stipulates that the target value of reliability index is 3.5 for a resistance factor (RF) of 1.0 (assuming 80% composite action compression flange in this study).
- The result of the composite action between the deck and the girder for RF of 1.0 for full composite action (i.e. considering the failure of the bottom flange) shows a safety index within the acceptable limit which suggests that the AlumadeckTM can withstand the live load it is subjected to considering

the HL-93 loading condition. Also, it can be established from this study that the RI is sensitive to the RF.

Data availability

The data presented in this study are available on request from the corresponding author. The data and codes are not publicly available because it also forms part of an ongoing study.

CRedit authorship contribution statement

Abdulkhaleem Adeoye Shittu: Conceptualisation; Data curation; Formal analysis; Funding acquisition; Investigation; Methodology; Validation; Visualisation; Writing - original draft; Writing - review & editing. **Samuel Abejide:** Investigation; Methodology; Writing - review & editing. **Samuel C. Olisa:** Investigation; Methodology; Writing - review & editing. **Ali Mehmanparast:** Investigation; Methodology; Project administration; Supervision; Writing - review & editing. **Athanasios Kolios:** Investigation; Methodology; Project administration; Supervision; Writing - review & editing.

Declaration of competing interest

The authors declare no conflict of interest

Acknowledgement

Authors Abdulkhaleem A. Shittu and Samuel Olisa would like to acknowledge the Petroleum Technology Development Fund (PTDF), Nigeria, for doctoral study scholarship, award number: PTDF/ED/PHD/SAA/1142/17 and PTDF/ED/OSS/PHD/SCO/1520/19 respectively.

Appendix A: Equations and derivations

$$M_p = \frac{P_w}{2D} [\bar{y}^2 + (D - \bar{y})^2] + P_s d_s + P_c d_c + P_t d_t \quad (A1)$$

where, $P_s = f_s A_s$; $P_c = f_{yc} A_{fc}$; $P_t = f_{yt} A_{ft}$; $P_w = f_{yw} A_w$; $Y_{bar} = \frac{D}{2} \left(\frac{P_t - P_c - P_s}{P_w} + 1 \right)$; $d_s = \frac{t_{slab}}{2} + t_{haunch} + Y_{bar}$; and $d_c = Y_{bar} - \frac{t_{fc}}{2}$; $d_t = D - \bar{y} + \frac{t_{ft}}{2}$. Again, $D_p = Y_{bar} + t_{slab} + t_{haunch}$; $D_t = D + t_{haunch} + t_{slab} + t_{ft}$. In addition, $M_n = M_p$ if $D_p \leq 0.1 D_t$; otherwise, $M_n = M_p \left(1.07 - \frac{0.7 D_p}{D_t} \right)$. where f_s = slab bending stress, A_s = effective area of the slab, f_{yc} = yield stress of the compression flange, A_{fc} = effective area of compression flange of the girder, f_{yt} =yield stress of the tension flange, A_{ft} = effective area of tension flange of the girder section, f_{yw} =yield stress of the web of the girder, A_w = area of web of the girder section, P_n = respective force, $\bar{y}$ = centroid of the composite section (i.e. girder plus haunch plus AlumadeckTM), t_{slab} = thickness of the AlumadeckTM slab, t_{fc} = thickness of compression flange, t_{ft} =thickness of tension flange and t_{haunch} = thickness of haunch.

$$D_r \frac{\partial^4 \eta}{\partial r^4} + 2H \frac{\partial^4 \eta}{r^2 \partial r^2 \partial \theta^2} + D_\theta \frac{\partial^4 \eta}{r^4 \partial \theta^4} + 2D_r \frac{\partial^3 \eta}{r \partial r^3} - 2H \frac{\partial^3 \eta}{r^3 \partial r \partial \theta^2} - D_\theta \frac{\partial^2 \eta}{r \partial r^2} + 2(D_\theta + H) \frac{\partial^2 \eta}{r^4 \partial \theta^4} + D_\theta \frac{\partial \eta}{\partial r^3 \partial r} = q \quad (A2)$$

where the parameters D_r , D_θ and H is the stiffness parameters expressed as

$$D_r = \frac{Et^3 r}{[12(1 - \mu^2)]} \quad (A3)$$

$$D_\theta = \frac{Et^3 r}{[12(1 - \mu^2)]} \quad (A4)$$

$$H = GK_1 \quad (A5)$$

$$D_{r\theta} = H/2 \quad (A6)$$

$$\eta = \sum [AX^{m_1} + BX^{m_2} + CX^{m_3} + DX^{m_4}] \sin \lambda \theta + \frac{4pr^4 \sin \lambda \theta}{D_r n \pi [72 - 18\beta \lambda^2 - \alpha(8 + 2\lambda^2 - \lambda^4)]} \quad (A7)$$

where, p =Loading, $\alpha = \frac{D_\theta}{D_r} = 1$, $\beta = \frac{H}{D_r} = 0.98$, $\lambda = \frac{n\pi}{\theta} = 0.37$, $X = \frac{r_i}{r}$ in which the four roots of

the equations m_1 , m_2 , m_3 , and m_4 can be expressed as Eq. (14):

$$m_1, m_2, m_3, m_4 = \pm \left\{ (\alpha + 2\beta\lambda^2 + 1)/2 \right. \quad (\text{A8})$$

$$\left. \pm [0.25(1 + \alpha^2\beta^2)^2 - (\lambda^2 - 1)^2\alpha]^{1/2} \right\}^{1/2} + 1$$

where for the above Equation, $X = \frac{r}{r_\theta}$, r = radius at the point of consideration, r_θ = radius of the

bridge deck system. The curved plate moment DEs are expressed as Eq. (15)-(18):

$$M_r = -D_r \frac{\partial^2 \eta}{\partial r^2} \text{ (Radial Moment)} \quad (\text{A9})$$

$$M_\theta = -D_r \frac{\partial \eta}{\partial r} + \frac{\partial^2 \eta}{r^2 \partial \theta^2} \text{ (Angular Moment)} \quad (\text{A10})$$

$$R_r = \left[\left(\frac{\partial^3 \eta}{\partial r^3} + \frac{\partial^2 \eta}{\partial r^2} \right) + 2 \frac{H}{D_r} \left(\frac{\partial^3 \eta}{r^3 \partial r \partial \theta^2} - \frac{\partial^2 \eta}{r^3 \partial \theta^2} \right) - \frac{D_\theta}{D_r} \left(\frac{\partial \eta}{r^3 \partial r} + \frac{\partial^2 \eta}{r^3 \partial \theta^2} \right) \right] \quad (\text{A11})$$

(Radial Shear)

$$R_\theta = -D_\theta \left[\left(\frac{\partial^2 \eta}{r^2 \partial r \partial \eta} + \frac{\partial^3 \eta}{r^3 \partial r \partial \theta} \right) + 2 \frac{H}{D_\theta} \left(\frac{\partial^2 \eta}{r^2 \partial r \partial \theta} + \frac{\partial^2 \eta}{r^2 \partial r \partial \theta} + \frac{\partial \eta}{r^3 \partial r} \right) \right] \quad (\text{A12})$$

(Angular shear)

$$\begin{aligned} -M_r \frac{r^2}{D_r} = & \left[(Am_1(m_1 - 1))X^{m_1} - Bm_2(m_2 - 1)X^{m_2} \right. \\ & \left. + Cm_3(m_3 - 1)X^{m_3} + Dm_4(t)X^{m_4} \right] \sin \lambda \theta, \end{aligned} \quad (\text{A13})$$

and

$$\begin{aligned} -R_r \frac{r^3}{D_r} = & \left[[Am_1(m_1 - 1)^2 - 2\beta\lambda^2(m_1 - 1) - \alpha(m_1 - \lambda^2)]X^{m_1} \right. \\ & + [Bm_2(m_2 - 1)^2 - 2\beta\lambda^2(m_2 - 1) - \alpha(m_2 \\ & - \lambda^2)]X^{m_2} \\ & + [Cm_3(m_3 - 1)^2 - 2\beta\lambda^2(m_3 - 1) - \alpha(m_3 \\ & - \lambda^2)]X^{m_3} \\ & \left. + [Dm_4(m_4 - 1)^2 - 2\beta\lambda^2(m_4 - 1) - \alpha(m_4 \right. \\ & \left. - \lambda^2)]X^{m_4} \right] \sin \lambda \theta \end{aligned} \quad (\text{A14})$$

A.1.1 Girder stress check critical section (mid-span) on G4 transversely stiffened web – strength LS – top flange

The factored vertical bending stress in the top flange due to dead and live load, f_{bu} is given as Eq (A15) [45]:

$$f_{bu,c} = \left[\frac{1.25M_{DC1}C_{nc}}{I_{nc}} + \frac{(1.25M_{DC2} + 1.5M_{DW})C_{3n}}{I_{3n}} + \frac{1.75M_{LL}C_n}{I_n} \right] 12\eta \quad (A15)$$

But, $\eta = 1$, where C_{3n} is the centroid considering long term composite properties, C_n is the centroid considering the short term, C_{nc} is the centroid considering non-composite, I_{nc} is the moment of inertia for the non-composite section. For the long term composite section properties (3n), the effective width of the slab can be expressed as Eq. (A16),

$$b_{s_{3n}} = \frac{b_{slab}}{3n} \quad (A16)$$

while for the short term (n),

$$b_{s_n} = \frac{b_{slab}}{n} \quad (A17)$$

whereas the NA for the non-composite action, C_{nc} can be expressed as

$$D_c = \frac{A_{fc} \left(\frac{t_{fc}}{2} \right) + A_{ft} \left(D - \frac{t_{ft}}{2} \right) A_{web} \left(\frac{D}{2} + t_{fc} \right)}{A_{girder}} - t_{fc} \quad (A18)$$

while the NA for the composite action, $C_{n/3n}$ can be expressed as Eq. (A19),

$$NA = \frac{A_{fc} \left(t_s + t_{haunch} - \frac{t_{fc}}{2} \right) + A_{ft} \left(D + t_s + t_{haunch} + \frac{t_{ft}}{2} \right) + A_{web} \left(\frac{D}{2} + t_s + t_{haunch} \right) + A_s \frac{t_s}{2}}{A_{Total}} \quad (A19)$$

A_{fc} is the area of compression flange, A_{ft} is the area of the tension flange, t_s is the thickness of the slab, t_{haunch} is the thickness of the haunch, D is the depth of the web, t_{fc} is the thickness of compression flange, A_w is the area of the web, t_w is the thickness of web and b_{fc} is the width of the compression

flange. The NA is measured from the top of the deck. The moment of inertia considering non-composite action is given by Eq. (A20):

$$I_{nc} = \frac{b_{fc}t_{fc}^3}{12} + \left[A_{fc} \left(D_c + t_{fc} - \frac{t_{fc}}{2} \right)^2 \right] + \frac{b_{ft}t_{ft}^3}{12} \quad (A20)$$

$$+ \left[A_{ft} \left(d - (D_c + t_{fc}) - \frac{t_{ft}}{2} \right)^2 \right] + \frac{t_w D^3}{12}$$

$$+ A_w \left(D_c + t_{fc} - t_{fc} - \frac{D}{2} \right)^2.$$

while the moment of inertia considering composite action, (i.e. for the short/long term) is expressed as Eq. (A21):

$$I_{n/3n} = \frac{b_{fc}t_{fc}^3}{12} + \left[A_{fc} \left(NA - t_s - t_{haunch} + \frac{t_{fc}}{2} \right)^2 \right] + \frac{b_{ft}t_{ft}^3}{12} \quad (A21)$$

$$+ \left[A_{ft} \left(t_s + t_{haunch} + D + t_{ft} - NA - \frac{t_{ft}}{2} \right)^2 \right]$$

$$+ \frac{t_w D^3}{12} + A_w \left(NA - t_s - t_{haunch} - \frac{D}{2} \right)^2 + \frac{b_s t_s^3}{12}$$

$$+ A_s \left(NA - \frac{t_s}{2} \right)^2.$$

whereas the compression flange must satisfy the relation in Eq. (A22):

$$f_{bu} \leq \phi_f F_{nc} \quad (A22)$$

ϕ_f is the resistance factor (RF), where:

$$F_{nc} = R_b R_h F_{yc} \quad (A23)$$

R_b is the web load shedding factor = 1, for $\frac{D}{t_w} < 150$, R_h is the hybrid factor = 1 and F_{yc} represents the yield strength for the compression flange. The LSF for the top flange of a transversely stiffened web (i.e. considering the strength LSS) can be expressed as:

$$G(x) = \phi_f F_{yc} - f_{bu} \quad (A24)$$

$$= \phi_f F_{yc} - \left[\frac{1.25 M_{DC1} C_{nc}}{I_{nc}} + \frac{(1.25 M_{DC2} + 1.5 M_{DW}) C_{3n}}{I_{3n}} + \frac{1.75 M_{LL} C_n}{I_n} \right] 12\eta.$$

The ductility requirements have to be checked to prevent the aluminium deck from premature crushing.

$$D_p \leq 0.42 D_t \quad (A25)$$

where D_t is the total depth of the composite section expressed as:

$$D_t = D + t_{haunch} + t_{slab} + t_{ft} \quad (A26)$$

D_p is the distance from the top of the deck to the NA of the composite section at the plastic moment:

$$D_p = Y_{bar} + t_{slab} + t_{haunch} \quad (27)$$

$P_s = f_s A_s = 205 \times 60090.202 \times 10^{-3} = 12318.5 kN$ - force in the slab; $P_c = f_{yc} A_{fc} = 350 \times 500 \times 21 \times 10^{-3} = 3675 kN$ - force in the compression flange; $P_t = f_{yt} A_{ft} = 3675 kN$ - force in the tension flange; $P_w = f_{yw} A_w = 350 \times 16 \times 2358 \times 10^{-3} = 13204.8 kN$ - force in the web. If $P_t + P_w \geq P_c + P_s$, then the plastic NA is on the web: $16879.8 > 15993.5$. Therefore, the PNA is on the web and $\bar{Y}$ or

$$Y_{bar} = \frac{D}{2} \left(\frac{P_t - P_c - P_s}{P_w} + 1 \right) \quad (A28)$$

A.1.2 Girder stress check strength limit state - web

According to the AASHTO specifications, composite sections subjected to positive flexure, as in this case study, need not be checked for web bend-buckling in its final composite condition when the web does not require longitudinal stiffeners.

A.1.3 Girder stress check critical section (at mid-span) on G4 transversely stiffened web – strength LS – bottom flange

The tension flange must satisfy the following relation:

$$f_{bu} + \frac{1}{3}f_l \leq \phi_f F_{nt}. \quad (A29)$$

The factored bottom flange vertical bending stress due to dead and live load is expressed as:

$$f_{bu,T} = \left[\frac{1.25M_{DC1}C_{nc}}{I_{nc}} + \frac{(1.25M_{DC2} + 1.5M_{DW})C_{3n}}{I_{3n}} + \frac{1.75M_{LL}C_n}{I_n} \right] 12\eta. \quad (A30)$$

The lateral bending stress at the cross-frame due to curvature, f_l is given as [16]:

$$M_{lat_NC} = \lambda M_{DC1} f_{l_NC} = \frac{M_{lat_NC}}{S_{bot_fl}} \quad (A31)$$

$$M_{lat_C1} = \lambda M_{DC2} \quad f_{l_C1} = \frac{M_{lat_C1}}{S_{bot_fl}} \quad (A32)$$

$$M_{lat_C2} = \lambda M_{DW} \quad f_{l_C2} = \frac{M_{lat_C2}}{S_{bot_fl}} \quad (A33)$$

$$M_{lat_LL} = \lambda M_{LL} f_{l_LL} = \frac{M_{lat_LL}}{S_{bot_fl}}, \quad (A34)$$

λ is as specified in Kulicki et al. [45], the total factored lateral bending stress:

$$f_l = 1.25(f_{l_NC} + f_{l_C1}) + 1.5(f_{l_C2}) + 1.75(f_{l_LL}). \quad (A35)$$

The nominal flexural resistance, $M_n = M_p$ if $D_p \leq 0.1D_t$; otherwise,

$$M_n = M_p \left(1.07 - \frac{0.7D_p}{D_t} \right). \quad (A36)$$

Yield moment associated with the strength-I LS [16]

$$M_{D1t} = f_{D1t}S_t, M_{D1c} = f_{D1c}S_c, M_{D2} = M_{DC2} + M_{DW}. \quad (A37)$$

Tension flange

$$M_{ADt} = \left(f_{yt} - \frac{M_{D1t}}{S_t} - \frac{M_{D2}}{S_{t3n}} \right) S_{tn} \quad (A38)$$

$$M_{yt} = M_{D1t} + M_{D2} + M_{ADt}. \quad (A39)$$

Compression flange

$$M_{ADc} = \left(f_{yc} - \frac{M_{D1c}}{S_c} - \frac{M_{D2}}{S_{c3n}} \right) S_{cn} \quad (A40)$$

$$M_{yc} = M_{D1c} + M_{D2} + M_{ADc}. \quad (A41)$$

Section modulus,

$$S_{xt} = \frac{M_{yt}}{f_{yt}}. \quad (A42)$$

Thus, strength limit check is **OK** if

$$M_u + \frac{f_l S_{xt}}{3} \leq \phi_f M_n \quad (A43)$$

$$M_u = 1.25M_{uDC2} + 1.5M_{uDw} + 1.75M_{uLL}. \quad (A44)$$

A.1.4 Girder stress check critical section (at mid-span) on G4: transversely stiffened web – shear strength – web

The shear strength of the mid-span in girder G4 is determined as:

$$V_u \leq \phi_v V_n \quad (A45)$$

where: V_u =shear in the web at the section under consideration due to factored loads; V_n =nominal shear resistance; ϕ_v = RF for shear taken as 1.0.

$$V_n = CV_p \quad (A46)$$

$$V_p = 0.58F_y D t_w \quad (A47)$$

where C is the ratio of the shear-buckling resistance to the shear yield strength

$$\frac{D}{t_w} > 1.40 \sqrt{\frac{Ek}{F_{yw}}} \quad (A48)$$

where k is the shear buckling coefficient $= 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} = 10$

$$147.4 > 1.4 \sqrt{\frac{200000(10)}{350}} = 75.6. \quad (A49)$$

Therefore $C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \left(\frac{Ek}{F_{yw}}\right)$

$$G(x) = \phi_v V_n - V_u. \quad (A50)$$

Appendix B: Tables of parameters

Table B1: Parameters of interest in the design model

Parameter	Value
Radius	100m
Bridge span	20m
Deck thickness, h	200mm
D_r	$5.2071 \times 10^9 Nm^2$
D_θ	$5.2071 \times 10^9 Nm^2$
H	$5.1029 \times 10^9 Nm^2$
m_1	1.633687
m_2	0.366313
m_3	2.36496
m_4	0.36496
$M_{radial}(kNm)$ for ULS	140
$R_{radial}(kN)$	0.142
Aluminium alloy deck compressive strength, F_c	170MPa
Yield strength, F_y	170MPa
Ultimate strength, F_u	205MPa
Effective area of the aluminium alloy deck/ slab (long term), A_{s_3n}	60,090.202 mm ²
Short term effective area of deck, A_{s_n}	180,270.606 mm ²
Structural steel	ASTM A709 grade 50W (yield strength, $F_y = 350MPa$)
Live load	HL93 [16]
Unit weight of deck	26.9 kN/m ³

Unit weight of steel	78.5 kN/m^3
Future wearing surface	6.73 kN/m^3
Elastic modulus of aluminium, E_{AL}	69.6 GPa
Elastic modulus of steel, E_{ST}	200 GPa
Girder depth, d	2.4 m
Web thickness, t_w	1.6 cm
Flange thickness; top (compression flange) and bottom (tension flange), t_{fc} and t_{ft}	2.1 cm
Flange width, b_f	50 cm

Table B2: Parameters of the stochastic model for the strength limit state [10,20–22,24,27–29,46]

Var. no.	Variable	Meaning	Distribution type	Expected value E_x	Coefficient of variation	Standard deviation
X1	M_{DC1}	Moment due to the curved aluminium deck and steel girder	Log-Normal	222.5 kNm	0.12	26.7 kNm
X2	M_{DC2}	Moment due to the parapet	Log-Normal	158 kNm	0.12	18.96 kNm
X3	M_{DW}	Moment due to the polymer surface	Log-Normal	1.744 kNm	0.12	0.209 kNm
X4	M_{LL}	Moment due to live load	Log-Normal	1473 kNm	0.12	176.76 kNm
X5	t_f	Thickness of flange	Normal	0.021 m	0.03	$6.3 \times 10^{-4} \text{ m}$
X6	b_f	Flange width	Normal	0.5 m	0.03	0.015 m
X7	d	Depth of section	Normal	2.4 m	0.03	0.072 m
X8	t_w	Thickness of web	Normal	0.016 m	0.03	$4.8 \times 10^{-4} \text{ m}$

X9	D	Distance from bottom of top flange to top of the bottom flange	Normal	2.358m	0.03	0.07074m
X10	t_{slab}	Slab thickness	Normal	0.20m	0.03	$6 \times 10^{-3}m$
X11	t_{haunch}	Haunch thickness	Normal	0.09m	0.03	$2.7 \times 10^{-3}m$
X12	$A_{slab_{3n}}$	Long term area of the slab	Normal	$0.06m^2$	0.03	$1.8 \times 10^{-3}m^2$
X13	A_{slab_n}	Area of slab short term	Normal	$0.18m^2$	0.03	$5.4 \times 10^{-3}m^2$
X14	ϕ_f	Resistance factor (RF)	Normal	1.0	0.05	0.05
X15		Percentage composite action	Normal	100%	0.02	2.0%
X16	f_y	Yield stress	Log-Normal	3.5 $\times 10^5 kN$ $/m^2$	0.02	$7 \times 10^3 kN/m^2$

6 References

- [1] Ozgur C. Behaviour and Analysis of a Horizontally Curved and Skewed I-Girder Bridge. Georgia Institute of Technology, 2007.
- [2] Linzell D, Chen A, Sharafbayani M, Seo J, Nevling D, Ashour O. Guidelines for Analysing Curved and Skewed Bridges and Designing them for Construction. USA: 2010.
- [3] Siwowski TW. Structural behaviour of aluminium bridge deck panels. Eng Struct 2009;31:1349–53. <https://doi.org/10.1016/j.engstruct.2009.02.002>.
- [4] Mazzolani FM. Structural applications of aluminium in civil engineering. Struct Eng Int J Int Assoc Bridge Struct Eng 2006;16:280–5. <https://doi.org/10.2749/101686606778995128>.
- [5] Tindall P. Aluminium in Bridges. ICE Man. Bridge. Eng. Second, London, UK: Thomas Telford Ltd.; 2008.
- [6] Gentils T, Wang L, Kolios A. Integrated structural optimisation of offshore wind turbine support

- structures based on finite element analysis and genetic algorithm. *Appl Energy* 2017;199:187–204. <https://doi.org/10.1016/j.apenergy.2017.05.009>.
- [7] Kolios A, Di Maio LF, Wang L, Cui L, Sheng Q. Reliability assessment of point-absorber wave energy converters. *Ocean Eng* 2018;163:40–50. <https://doi.org/10.1016/j.oceaneng.2018.05.048>.
- [8] Shittu AA, Kolios A, Mehmanparast A. A Systematic Review of Structural Reliability Methods for Deformation and Fatigue Analysis of Offshore Jacket Structures. *Met* 2021, Vol 11, Page 50 2020;11:50. <https://doi.org/10.3390/MET11010050>.
- [9] Shittu AA, Mehmanparast A, Wang L, Saloniitis K, Kolios A. Comparative Study of Structural Reliability Assessment Methods for Offshore Wind Turbine Jacket Support Structures. *Appl Sci* 2020;10. <https://doi.org/10.3390/app10030860>.
- [10] Shittu AA. Structural reliability assessment of complex offshore structures based on non-intrusive stochastic methods. Cranfield University, Bedford, UK, 2020.
- [11] Shittu AA, Mehmanparast A, Shafiee M, Kolios A, Hart P, Pilario KE. Structural Reliability Assessment of Offshore Wind Turbine Support Structures Subjected to Pitting Corrosion-Fatigue: A Damage Tolerance Modelling Approach. *Wind Energy* 2020;23:2004–26. <https://doi.org/10.1002/we.2542>.
- [12] Shittu AA, Mehmanparast A, Hart P, Kolios A. Comparative study between S-N and fracture mechanics approach on reliability assessment of offshore wind turbine jacket foundations. *Reliab Eng Syst Saf* 2021;215:107838. <https://doi.org/10.1016/J.RESS.2021.107838>.
- [13] Breitung K. 40 years FORM: Some new aspects? *Probabilistic Eng Mech* 2015;42:71–7. <https://doi.org/10.1016/j.probengmech.2015.09.012>.
- [14] Papadopoulos V, Papadrakakis M, Deodatis G. Analysis of mean and mean square response of general linear stochastic finite element systems. *Comput Methods Appl Mech Eng* 2006;195:5454–71. <https://doi.org/10.1016/j.cma.2005.11.008>.
- [15] Beck AT, Gomes WJDS. A comparison of deterministic, reliability-based and risk-based structural optimisation under uncertainty. *Probabilistic Eng. Mech.*, vol. 28, Elsevier; 2012, p. 18–29. <https://doi.org/10.1016/j.probengmech.2011.08.007>.
- [16] AASHTO. Guide Specifications for Horizontally Curved Steel Girder Highway Bridges with Design

- Examples for I-Girder and Box-Girder Bridges”, , 2007.
- [17] AASHTO. LRFD Bridge Design Specifications 2010.
 - [18] Siwowski T. FEA Modelling and Analysis of a Certain Aluminium Bridge Deck Panel. Arch Civ Eng 2009;3:347–65.
 - [19] Dobmeier JM, Barton FW, Gomez JP, Massarelli PJ, Jr WTM. Analytical and experimental evaluation of an aluminum bridge deck panel, Part I: Service load performance. 1999.
 - [20] A.O. A, O.S. A, A.T. O. Probabilistic Evaluation of Aluminum Bridge Decks Design Criteria on Steel Girders. J Eng Appl Sci 2010;5:4–13.
 - [21] Wang N, Ellingwood BR, Zureick AH. Bridge rating using system reliability assessment. II: Improvements to bridge rating practices. J Bridg Eng 2011;16:863–71. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000171](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000171).
 - [22] Shittu A. Probabilistic Evaluation of Horizontally Curved Aluminium Alloy Bridge Decks on Steel Girders. Ahmadu Bello University, 2015.
 - [23] Siwowski T. Aluminium bridges - Past, present and future. Struct Eng Int J Int Assoc Bridg Struct Eng 2006;16:286–93. <https://doi.org/10.2749/101686606778995137>.
 - [24] Yanweerasak T, Pansuk W, Akiyama M, Frangopol DM. Life-cycle reliability assessment of reinforced concrete bridges under multiple hazards. Struct Infrastruct Eng 2018;14:1011–24. <https://doi.org/10.1080/15732479.2018.1437640>.
 - [25] Thanapol Y, Akiyama M, Frangopol DM. Updating the Seismic Reliability of Existing RC Structures in a Marine Environment by Incorporating the Spatial Steel Corrosion Distribution: Application to Bridge Piers. J Bridg Eng 2016;21:1–17. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000889](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000889).
 - [26] Nie J, Ellingwood BR. Finite element-based structural reliability assessment using efficient directional simulation. J Eng Mech 2005;131:259–67. [https://doi.org/10.1061/\(ASCE\)0733-9399\(2005\)131:3\(259\)](https://doi.org/10.1061/(ASCE)0733-9399(2005)131:3(259)).
 - [27] Ren J, Song J, Ellingwood BR. Reliability assessment framework of deteriorating reinforced concrete bridges subjected to earthquake and pier scour. Eng Struct 2021;239:112363. <https://doi.org/10.1016/j.engstruct.2021.112363>.

- [28] Ellingwood BR, Vrouwenvelder T, Gulvanessian H. Eurocodes and their implications for bridge design: Background, implementation, and comparison to North American practice. *J Bridg Eng* 2014;19:3–4. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000567](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000567).
- [29] Wang N, O'Malley C, Ellingwood BR, Zureick AH. Bridge rating using system reliability assessment. I: Assessment and verification by load testing. *J Bridg Eng* 2011;16:854–62. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000172](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000172).
- [30] Zureick A, Naqib R. Horizontally Curved Steel I-Girders State-of-the-Art Analysis Methods. *J Bridg Eng* 1999;4:38–47. [https://doi.org/10.1061/\(asce\)1084-0702\(1999\)4:1\(38\)](https://doi.org/10.1061/(asce)1084-0702(1999)4:1(38)).
- [31] Road Research. Evaluation of Load Carrying Capacity of Bridges. Paris: 1979.
- [32] BS5400-2. BSI Steel, Concrete and Composite Bridges 1978.
- [33] Heins CP, Hails RL. Behaviour of Stiffened Curved Plate Model. *ASCE Struct Div T11* 1969;95:2353–69.
- [34] Sorensen JD. Note 1&2: Structural Reliability. Aalborg, Denmark: 2003.
- [35] Choi S-K, Grandhi R, Canfield R. Reliability-based Structural Design. USA: Springer; 2006.
- [36] Melchers R, Andre T. Structural reliability analysis and prediction. 3rd ed. Hoboken, USA: Wiley; 2018.
- [37] Nowak AS. System reliability models for bridge structures. *Bull Polish Acad Sci* 2004;52:321.
- [38] Shittu AA, Mehmanparast A, Amirafshari P, Hart P, Kolios A. Sensitivity analysis of design parameters for reliability assessment of offshore wind turbine jacket support structures. Under Rev 2020.
- [39] Webster SE, Bannister AC. Methods, Applications and Software for Structural Reliability Assessment. Corus UK limited; 2001.
- [40] Timoshenko SP, Woinowsky-Krieger S. Theory of Plates and Shells. First. London, UK: McGraw Hill; 1982.
- [41] Eurocode 9. Design of Aluminium Structures 1999.
- [42] Al-Sanad S, Wang L, Parol J, Kolios A. Reliability-based design optimisation framework for wind turbine towers. *Renew Energy* 2021;167:942–53. <https://doi.org/10.1016/j.renene.2020.12.022>.

- [43] Ivanhoe RO, Wang L, Kolios A. Generic framework for reliability assessment of offshore wind turbine jacket support structures under stochastic and time dependent variables. *Ocean Eng* 2020;216:107691. <https://doi.org/10.1016/j.oceaneng.2020.107691>.
- [44] Heins CP. *Bending and Torsional Design in Structural Members*. London, UK: Lenington Books, DC Health Company; 1975.
- [45] J.M. K, W.G. W, C. S, K. J. *AASHTO-LRFD Design Example Horizontally Curved Steel I-Girder Bridge*, . USA: 2005.
- [46] Wang C, Li Q, Ellingwood BR. Time-dependent reliability of ageing structures: an approximate approach. *Struct Infrastruct Eng* 2016;12:1566–72. <https://doi.org/10.1080/15732479.2016.1151447>.



DSSPACE

<https://dspace.org/>

A stochastic framework for the assessment of horizontally curved aluminium bridge decks on steel girders

Shittu, Abdulhakim Adeoye; Abejide, Samuel; Olisa, Samuel Chukwuemeka; Mehmanparast, Ali; Kolios, Athanasios

2022-06-11

Elsevier

<https://doi.org/10.1016/j.istruc.2022.06.007>

Shittu AA, Abejide S, Olisa SC, et al., (2022) A stochastic framework for the assessment of horizontally curved aluminium bridge decks on steel girders. Structures, Volume 42, August 2022, pp. 125-143