

**PROBABILISTIC EVALUATION OF HORIZONTALLY CURVED
ALUMINIUM ALLOY BRIDGE DECKS ON STEEL GIRDERS**

BY

Abdulhakim SHITTU, B.Eng. (A.B.U) 2008

M.Sc./ENG/5326/09-10

**A THESIS SUBMITTED TO THE SCHOOL OF POSTGRADUATE
STUDIES, AHMADU BELLO UNIVERSITY, ZARIA**

**IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE
AWARD OF A MASTER DEGREE
IN STRUCTURAL ENGINEERING**

**DEPARTMENT OF CIVIL ENGINEERING,
FACULTY OF ENGINEERING
AHMADU BELLO UNIVERSITY,
ZARIA, NIGERIA**

JULY, 2015

DECLARATION

I declare that the work in this thesis entitled “Probabilistic Evaluation of Horizontally Curved Aluminium Alloy Bridge Decks on Steel Girders” has been carried out by me in the Department of civil engineering. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this thesis was previously presented for another degree at this or any other institution.

Shittu Abdulhakim

Student

Signature

Date

CERTIFICATION

This thesis entitled “PROBABILISTIC EVALUATION OF HORIZONTALLY CURVED ALUMINIUM ALLOY BRIDGE DECKS ON STEEL GIRDERS” by Abdulhakim SHITTU meets the regulations governing the award of the degree of Master of Science in Structural Engineering of the Ahmadu Bello University, and is approved for its contribution to knowledge and literary presentation.

Prof. O. S. Abejide

Chairman, Supervisory Committee

Signature

Date

Dr. J. A. Sadeeq

Member, Supervisory Committee

Signature

Date

Dr. Y. D. Amartey

Head of Department

Signature

Date

Prof. K. Bala

Dean, School of Postgraduate Studies

Signature

Date

ACKNOWLEDGEMENT

My deepest gratitude goes to the Almighty ALLAH, the most beneficent, the most merciful, for HIS guidance and protection throughout my studies in this school and also for HIS grace on me to Insha ALLAH complete thisMSc programme successfully.

To Prof. O. S. Abejide, my supervisor, I wish to express my sincere thanks for patiently guiding me towards a successful completion of this thesis. His assistance, fatherly advice and timely corrections proved to be very effective and would not be forgotten. Also to Dr. J. A. Sadeeq my Second Supervisor, I wish to express my sincere gratitude for his time, kindness and encouragement throughout this research work.

My deep heart felt appreciation goes to my parents Alh. and Haj. Shittu for their encouragement towards my studies.

ABSTRACT

The Probabilistic evaluation of horizontally curved aluminium alloy bridge decks on steel I-Girders using AASHTO (2010) specification is presented. The analysis was carried out using ABAQUS and the First Order Reliability Method (FORM).

Structural behavior of the horizontally curved aluminium alloy decks on steel girders were studied under varying loadings while varying their geometrical properties. The result of finite element analysis carried out on the horizontally curved deck shows that a minimum thickness of stiffener of 7mm is ideal for the safety of the horizontally curved deck. The Von Mises stress was within acceptable limits i.e. $< 1.7 \times 10^5 \text{ kN/m}^2$ which is the yield strength of the Aluminum deck. Hence, it is shown that the proprietary Alumadeck can be used in a region of curved alignment up to an angle of $\theta = 34.4^\circ$.

It is shown that the proprietary Alumadeck system is in conformity with the Load Resistance Factor Design (LRFD) Specification (2004) which stipulates that the target value of reliability index is 3.5 for resistance factor of 1.0 (Assuming 80% composite action compression flange). The result of the composite action between the deck and the girder for RF of 1.0 for full composite action (Considering failure of the bottom flange) shows a safety index within acceptable limit which shows that the Alumadeck can withstand the live load it is subjected to considering the HL 93 loading condition. Also the minimum composite action for safety is 40% corresponding to a safety index of 1.16. An I girder of depth 2.4m , flange thicknesses of 21mm , flange width 50cm and web thickness of 16mm is safe to carry the applied loading i.e. truck load and selfweight.

TABLE OF CONTENTS

| | |
|--|-----------|
| CERTIFICATION | iii |
| ACKNOWLEDGEMENT | iv |
| ABSTRACT | v |
| LIST OF TABLES | ix |
| LIST OF SYMBOLS | xii |
| CHAPTER ONE | 1 |
| 1.0 INTRODUCTION | 1 |
| 1.1 General | 1 |
| 1.2 Statement of Research Problem | 6 |
| 1.3 Aim and Objectives | 8 |
| 1.3.1 Aim | 8 |
| 1.3.2 Objectives | 9 |
| 1.4 Scope and Limitation of the Reseach | 9 |
| 1.4.1 scope | 9 |
| 1.4.2 Limitation | 9 |
| CHAPTER TWO | 10 |
| 2.0LITERATURE REVIEW | 10 |
| 2.1 Horizontally Curved Skewed Bridges | 10 |
| 2.1.1 Approximate Methods | 11 |
| 2.1.2 Refined Methods | 11 |
| 2.2 Aluminum Alloys in Bridges | 14 |
| 2.2.1 Bridge Dynamics | 21 |
| 2.2.2 Bridge Design | 22 |
| 2.2.3 The Need for Reliability-Based Design | 22 |
| 2.2.4 Types of Bridge Loadings | 24 |
| 2.3 Structural Reliability | 24 |
| 2.3.1 Limit State Functions | 27 |
| 2.3.2 Reliability of Bridge Structures | 29 |
| 2.4 Finite-Element Method | 30 |
| 2.5 Stochastic Finite Element Method | 33 |
| 2.6 Description of the Bridge Deck (AlumadeckTm) | 35 |
| 2.7 Loadings and Load Application | 36 |
| 2.7.1 Dead Loads | 36 |

| | | |
|----------------------|--|----|
| 2.7.2 | Live Loads | 36 |
| 2.7.3 | Lane Loads | 37 |
| 2.7.4 | Truck Loading | 38 |
| 2.7.5 | Load Application | 38 |
| 2.7.6 | Centrifugal Load | 39 |
| 2.7.7 | Distribution of Live Load | 40 |
| 2.8 | Highway Bridge Loading According to BS 5400: Part 2, 1978 | 41 |
| 2.9 | Limit States | 42 |
| 2.9.1 | Strength | 43 |
| 2.9.2 | Constructability | 43 |
| 2.9.3 | Fatigue | 43 |
| 2.10 | Load Combination | 43 |
| 2.11 | Bridge Deck Design | 45 |
| 2.11.1 | Design of Curved Decks | 46 |
| 2.12 | Curved Beam Design | 49 |
| 2.13 | The Aashto (2003) Flexural Resistance Equations | 50 |
| 2.14 | Application of Reliability/ Probabilistic Evaluation | 52 |
| CHAPTER THREE | | 54 |
| 3 | METHODOLOGY | 54 |
| 3.1 | General | 54 |
| 3.2 | Concept of Structural Reliability Analysis | 55 |
| 3.3 | Causes and Effects of Structural Failure | 56 |
| 3.4 | Uncertainties in Engineering Design | 57 |
| 3.5 | Methods of Reliability Analysis | 58 |
| 3.6 | Resistance and Load Interaction | 60 |
| 3.7 | Limit State Function | 62 |
| 3.8 | Computation of Reliability Index | 63 |
| 3.9 | First Order Reliability Method | 65 |
| 3.10 | The Abaqus | 67 |
| CHAPTER FOUR | | 68 |
| 4 | FINITE ELEMENT ANALYSIS, DESIGN AND STOCHASTIC OPTIMIZATION | 68 |
| 4.1.1 | Finite Element Analysis With Use of Abaqus | 68 |
| 4.1.2 | Radial Moment and Shear Force Acting On Deck | 78 |
| 4.1.3 | Bridge Deck Design Parameters | 79 |

| | | |
|---|---|-----|
| 4.1.4 | Design Data for I-Girder G4 | 80 |
| 4.1.5 | Design Moment | 82 |
| 4.2 | Bridge Component Design and Derivation of Limit State Functions | 83 |
| 4.2.1 | Girder Stress Check Critical Section On G4 – Section Proportioning | 84 |
| 4.2.2 | Girder Stress Check Critical Section (Mid Span) on G4 Transversely Stiffened Web – Strength – Top Flange | 85 |
| 4.2.3 | Girder Stress Check Strength Limit State - Web | 88 |
| 4.2.4 | Girder Stress Check Critical Section (Mid Span) on G4 Transversely Stiffened Web – Strength – Bottom Flange | 88 |
| 4.2.5 | Girder Stress Check Critical Section (Mid Span) on G4 Transversely Stiffened Web – Shear Strength – Web | 90 |
| CHAPTER FIVE | | 96 |
| 5 | SUMMARY, CONCLUSION AND RECOMMENDATION | 96 |
| 5.1 | Summary | 96 |
| 5.2 | Conclusion | 96 |
| 5.3 | Recommendation | 97 |
| REFERENCES | | 98 |
| APPENDIX A: RESULTS OF FINITE ELEMENT ANALYSIS | | 106 |
| APPENDIX B: RELIABILITY ANALYSIS PROGRAMS | | 111 |

LIST OF TABLES

| | |
|--|-----|
| Table 4-1: Parameters of the stochastic Model For strength Limit State | 92 |
| Table A-1: Maximum Displacements at a Thickness of stiffener = $5mm$ | 106 |
| Table A-2: Maximum Displacements at a Thickness of stiffener = $7mm$ | 106 |
| Table A-3: Maximum Displacements at a Thickness of stiffener = $9mm$ | 107 |
| Table A-4: Von Mises stresses at a Thickness of stiffener = $5mm$ | 108 |
| Table A-5: Von Mises stresses at a Thickness of stiffener = $7mm$ | 109 |
| Table A-6: Von Mises stresses at a Thickness of stiffener = $9mm$ | 109 |

LIST OF FIGURES

| | |
|--|----|
| Figure 1-1: System I Stresses – Longitudinal Bending of the Composite Deck/ Girder Section | 8 |
| Figure 1-2: System II Stresses – Transverse Bending of the Deck Between Girders | 8 |
| Figure 2-1: Nomenclature of Top and Bottom Deck surfaces | 35 |
| Figure 2-2: Nomenclature of stiffeners | 35 |
| Figure 2-3: Section A-A of the bridge illustrating the HL93 loading locations. | 39 |
| Figure 2-4: Bridge Deck geometry on plan | 49 |
| Figure 2-5: Girder effective width for composite action | 52 |
| Figure 4-1: The horizontally curved bridge deck as drawn in ABAQUS | 68 |
| Figure 4-2: The ABAQUS showing the vertical and inclined stiffeners of the horizontally curved Aluminium alloy deck | 69 |
| Figure 4-3: Loadings on the Alumadeck | 70 |
| Figure 4-4: Meshing of the deck into nodes and elements | 70 |
| Figure 4-5: Boundary condition | 71 |
| Figure 4-6: Analysis result showing the contours of Von mises stress after performing a job on ABAQUS. | 71 |
| Figure 4-7: Variation of maximum displacement (mm) against applied centrifugal stress (kN/m^2) | 72 |
| Figure 4-8: Variation of maximum Von Mises stress (kN/m^2) against applied centrifugal stress (kN/m^2) | 73 |
| Figure 4-9: Variation of maximum displacement (mm) against applied centrifugal stress (kN/m^2) | 74 |
| Figure 4-10: Variation of maximum Von Mises stress (kN/m^2) against applied centrifugal stress (kN/m^2) | 75 |
| Figure 4-11: Variation of maximum displacement (mm) against applied centrifugal stress (kN/m^2) | 76 |
| Figure 4-12: Variation of maximum Von Mises stress (kN/m^2) against applied centrifugal stress (kN/m^2) | 77 |
| Figure 4-13: I girder bridge cross section showing cross frames. Source: Kulicki et al. (2005) | 78 |
| Figure 4-14: Reliability Index at various Resistance Factors for Various Composite action between Alumadeck and girder (Considering Failure of Top flange) | 94 |

Figure 4-15: Reliability Index at various Resistance Factors for Various Composite action between Alumadeck and girder

95

LIST OF SYMBOLS

| | |
|---------------|--|
| A | Cross sectional area |
| A_s | Effective area of slab |
| A_{fc} | Effective area of compression flange of girder |
| A_{ft} | Effective area of tension flange of the girder section |
| A_w | Area of web of the girder section |
| BR | Vehicular braking force |
| C_y | Covariance matrix |
| CE | Vehicular centrifugal force |
| Cov | Coefficient of variation |
| $D(G)$ | Standard deviation of the safety margin |
| D_r | Flexural rigidity in r direction |
| D_θ | Flexural rigidity in θ direction |
| $D_{r\theta}$ | Torsional rigidity |
| E | Expected value |
| E_{al} | Modulus of elasticity for aluminium |
| f_{bu} | Flange major-axis bending stress |
| f_l | Flange lateral bending stress |
| f_{yc} | Yield stress of compression flange |
| f_{yt} | Yield stress of the tension flange |
| f_{yw} | Yield stress of the web of girder |

| | |
|--------------|--|
| $\phi_f F_n$ | Factored flexural resistance in terms of the major-axis bending stress |
| $F()$ | Cumulative probability function |
| f_s | Slab bending stress |
| $g()$ | Limit state function |
| IM | Vehicular dynamic load allowance |
| K_1 | Torsional Constant |
| LL | Vehicular live load |
| m | Multiple presence factor |
| M_r | Radial moment |
| M_θ | Angular moment |
| $\phi_f M_n$ | Factored flexural resistance in terms of member major-axis bending moment |
| P | Loading |
| P_f | Probability of failure |
| w | Clear roadway width between the curbs |
| r | Radius of curvature |
| R | Resistance |
| R_r | Radial Shear |
| s | Girder spacing (mm) |
| S | Loading |
| S_x | Elastic section modulus about the major-axis of the section to the flange under consideration taken as $\frac{M_{yf}}{F_{yf}}$ |
| t_{fc} | Thickness of compression flange |

| | |
|--------------|-----------------------------|
| t_{ft} | Thickness of tension flange |
| t_{haunch} | Thickness of haunch |
| t_s | Slab thickness |
| TU | Uniform temperature. |
| V | Highway design speed |
| μ | Poisson constant |
| θ | Subtended angle |
| η | Deflection of the section |

CHAPTER ONE

1.0 INTRODUCTION

1.1 GENERAL

The use of curved girder bridges in highway construction has grown steadily during the last 40 years. Today, roughly 25% of newly constructed bridges have a curved alignment (Lydzinski et al., 2008). The modern transportation industry encounters an increasing use of curved and skewed I-girder/ beam bridges for a number of reasons. These types of bridges are becoming more common as highway infrastructure is increasingly rebuilt atop existing structures to handle increasing traffic volumes or new interchange geometries within the context of urban settings. In particular, curved I-girder/ beam bridges have the ability to change direction within each span and thus are ideal structures for applications such as highway interchanges or to connect existing roadways where abutments cannot be relocated for physical or economic reasons (Ozgur, 2007; Linzell et al., 2010). These bridges are particularly advantageous for the construction of roadways in areas that have serious geographical or manmade impediments. In addition, they allow for more economical construction relative to straight girder chords that are aligned in a curved fashion, since they often allow the use of fewer piers and longer spans. Their utilization is especially common on major highway overpasses and on highway on-ramps and off-ramps (Lydzinski,et al., 2008).

According to White et al. (2001), due to the need to augment the traffic capacity of urbanhighways, restrictions on existing land use, and consideration of aesthetics, there has been a steady growth in the use of curved steel bridges in the past twenty-five years.

Additionally, specification of curved structure, while generating more superstructure costs in terms of materials and engineering, actually reduces the structures cost through the elimination of interior supports, significant deck overhangs, and expensive right-of-way acquisitions. Similarly, skewed I girder/ beam bridges are also useful when roadway alignment changes are not feasible or economical because of the topography of the site and also at particular areas where environmental impact is an issue (Linzell et al., 2010). Increasingly complex interchanges and the desire to conform to existing terrain have made curved girder bridges more attractive (Brett and Laman, 2000).

Majority of bridge decks today are in deplorable conditions (Hadipriono, 1985). The conventional highway bridge deck system deteriorates over time as a result of changes in microclimate such as changes in temperature, moisture content fluctuation and freeze-thaw cycle. Corrosion of the reinforcing steel has been identified as the major cause of the deterioration in concrete therefore it is the most important factor responsible for the large majority of structurally deficient bridges (Siwowski, 2006).

Also, ageing, inadequate maintenance, increase in load spectral and environmental contamination have contributed to this present problem of deterioration (Agboola et al., 2010).

Bridge decks require major repair or replacement every 15-20 years while the substructure and super structure tend to last 40 years or more (Wesley and John, 2006). There has been limited success in the past in offering effective solutions to these problems of deterioration of bridge decks. The application of coatings and other protective systems to cover shrinkage cracks in concrete bridge decks (which commonly provides a passage for water leakage, and reinforcement corrosion) have yielded less than desired results (Robert, 2002). Repairing a

deteriorated concrete deck will only serve to partially remedy the situation. Other areas of the deck will be as likely to deteriorate and will eventually require repair. It is illogical or sometimes impossible to use reinforced concrete decks a second time if we are perhaps to replace the entire deck (Robert, 2002). To reduce highway closure time and make retrofitting that meets current design specifications possible, has spirited the search for alternative decks that can resist environmental factors without protective coating. It would require considerable time for formwork, concrete placement, and concrete curing and as such resulting in extended highway closure time (Agboola et al., 2010). Other reasons why we perhaps replace bridge decks includes structural damage from collision, alignment and geometric inadequacies, etc. (Mark, 2009).

Improving the life of bridges can be achieved using alternative materials, such as fibre reinforced composites or aluminum. Since composites and aluminum are: lighter than steel and concrete, do not rust nor need painting or protective coatings, and require lower fabrication and erection time, hence the lower cost, they have distinct advantage over other construction materials (Siwowski, 2006).

The suggestion made by Reynolds Aluminium Company (manufacturers of the AlumadeckTM) has given way to a feasible alternative to conventional reinforced concrete decks (Dobmeier et al., 2001). This deck is new and requires a close study of its characteristic structural behavior and compatibility to the current design codes of practice for the design of bridges. Aluminium is not widely used by bridge Engineers because its properties have not been fully explored (Agboola, 2010). There exists a highway bridge 153m long, with a main span arch of 91.5 m, which was built of aluminium in 1950 and in 2008, it is still in service. Aluminium is a material that has distinctive properties which need to be worked with. When used correctly, the results are light, durable structures that are economical (Tindall, 2008).

Savings as high as 40% on steel material could be realized utilizing an aluminum deck and steel girder system. The savings were attributed to the decrease in dead load that aluminum provides, and the fact that the steel girders do not have to support large construction loads that are required during 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 extremely useful for handling during fabrication and construction, as well as in the final design. The durability of aluminium alloys is very good and 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. With the right approach, an aluminium structure will compete with any other material on cost, and will outperform most in service. Price per tonne for the basic material is high compared with steel, but when fabrication, erection and treatment costs are taken into account there is little difference for the completed structure. Aluminium will often be cheaper than concrete when whole-life costs are calculated (Tindall, 2008; Dwight, 1999).

Horizontally curved bridges continue to occupy a growing share of the bridge market. Horizontally curved bridges also offer aesthetic and cost benefits over more traditional chorded structures that make their selection attractive even when site restrictions are not an issue. They are invariably shallower than curved concrete bridges, which can often result in shorter approaches or fewer retaining walls when compared to structures containing deeper girders.

Horizontally curved bridges can also be skewed to meet site demands. Given the benefits realized with the design and construction of horizontally curved steel bridges (e.g., reduced number of substructure units and length of deck overhangs; increased spans and traffic sight distances) and the continued decrease in available land space for new and replacement structures, the use of these bridge types is certain to increase (Linzell et al, 2004).

However, there is considerable additional complexity associated with their analysis, design and construction compared to that for typical straight bridges. There are two general types of horizontally curved steel bridges. The box or tub girder is able to resist significant torsion if its shape is maintained with adequate internal bracing. This type of bridge was initially popular for that reason. The earliest box girder bridges had closed box sections (i.e., steel on four sides with a composite or noncomposite concrete deck). It is impractical to fabricate closed boxes therefore, tub girders with top lateral bracing have replaced closed box girders. Curved I girders are more commonly used for horizontally curved bridges. These members have very little torsional stiffness and are stable only when connected to other girders using cross frames or diaphragms (Linzell et al., 2010).

There is a need in view of the anticipated uncertainties to use a probabilistic approach (Ellingwood et al, 1980) to evaluate the performance of the suggested aluminum alloy bridge deck in terms of strength characteristics and durability; while the result will be compared with recommended design specification approach as in AASHTO LRFD (2004).

1.2 STATEMENT OF RESEARCH PROBLEM

Many sources of uncertainties are inherent in structural designs. Despite what is often thought, the parameters of the loading and load carrying capacities of structural members are not deterministic quantities, i.e., quantities which are perfectly known. They are random variables and thus, absolute safety (or zero probability of failure) cannot be achieved. Consequently, structures must be designed to serve their functions with a finite probability of failure (Nowak and Collins, 2000).

Accumulation of research in the field of bridge evaluation has indicated the justification of using reliability indices for the measure of safety (Ferhat and Dan, 2004). Reliability methods can be used as an important tool in the evaluation of existing or proposed structures (Melchers, 1999) as the traditional component-based approach often does not allow revealing the actual load capacity (Nowak, 2004).

The loads on a bridge at any time depend on many factors, such as the number of vehicles on the bridge, weight of the vehicle and the approach speed of the vehicles. The fact that we cannot ascertain the details about each vehicle passing over the bridge or the number of vehicles on the bridge at any time space means that there are some uncertainties about the loads, the total load on bridge and the bridge resistances (Nowak and Collins, 2000).

Ditlevsen and Madsen, (2005) envisaged engineering judgment as the art of being able to decide whether results obtained from a structural analysis or design model are sufficiently realistic that the engineer dares to base his or her practical decisions on these results. They added that the formulation of a mathematical model is guided by the wish of getting a realistic description, but

it is also necessary that the model becomes operational in the sense that it is suited for solution, that is, that it can deliver answers to posed questions, and that important aspects of the model formulation process is therefore the art of balancing realism against operability. Hence, probabilistic structural analysis is the art of formulating a mathematical model within which one can ask and get answers to questions such as:

1. What is the probability that a structure behaves in a specified way when given that one or more of its material properties or geometric dimensions and properties are of a random or incompletely known nature, and/or that the actions on the structure in some respects have random or incompletely known properties?
2. How is a structure behaving when its material properties, geometric properties and actions all are uniquely given?
3. What dimensions should be assigned to the structure in order that it has optimal properties in a given well-defined sense within the possibilities of the probabilistic model?
4. What value should the probability of occurrence of a specified behavior be in order that the structure in a well-defined sense is optimally designed with respect to this behavior?
5. How much is presently known about the actual safety levels of decks?
6. If anything is known about current safety levels, are these safety levels acceptable?

These are the questions posed on the design of aluminum decks for bridges. The correct, appropriate and distinct answers to these questions will undoubtedly justify the use of aluminum decks as formulated herein for bridges.

1.3 AIM AND OBJECTIVES

1.3.1 AIM

The aim of this study is to investigate using Finite Element Analysis software to ascertain the durability and use First Order Reliability Method (FORM) to obtain the value of safety and suggest improvement of the safety levels of the proposed curved aluminium deck and the corresponding supporting curved steel I-girder under wheel loads.

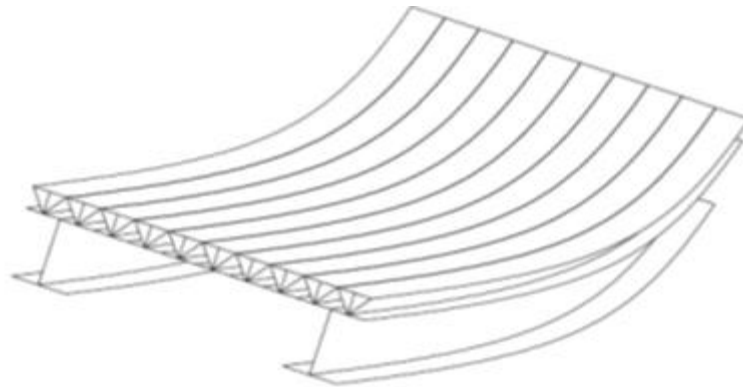


Figure 0-1: System I Stresses – Longitudinal Bending of the Composite Deck/ Girder Section

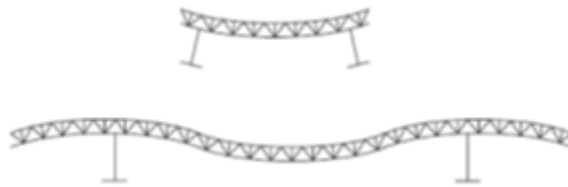


Figure 0-2: System II Stresses – Transverse Bending of the Deck between Girders

1.3.2 OBJECTIVES

The objectives of the study are:

- (i) To analyze the horizontally curved bridge deck using finite element method for system II Stress.
- (ii) To obtain a suitable single equation based on the requirement of AASHTO (2003) and AASHTO (2010) that will serve as the limit state function in the reliability analysis.
- (iii) Perform a reliability evaluation of a horizontally curved bridge deck for system I stress (i.e. representing the longitudinal bending of the composite aluminium deck with the I-girder) using the First Order Reliability Method (FORM) to determine the safety indices.
- (iv) To propose geometric properties at given reliability levels.

1.4 SCOPE AND LIMITATION OF THE RESEARCH

1.4.1 SCOPE

This study will estimate the load carrying capacity of the bridge using the model without springs and with composite action as suggested by Eamon et al, (2000). The effect of deck splice shall be ignored.

1.4.2 LIMITATION

This study is limited to the reliability of system I and II only of the deck-girder and not the reliability of the whole system.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 HORIZONTALLY CURVED SKEWED BRIDGES

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.

A number of modeling strategies for the design and construction analysis of curved I-girder bridge systems ranging from modified line girder analyses to finite element approaches have been recently published (Ozgor, 2007; Linzell et al., 2010). The analysis methods found in the literature can be classified into two major categories: approximate and refined methods. Approximate methods require a minimal modeling effort on the part of the designer, and hence, are adequate for preliminary analysis and design purposes. The following approximate methods are most frequently used in the analysis of curved girders (Zureick and Naqib, 1999): (1) The plane-grid method (2) The space-frame method (3) The V-load method

Refined methods, on the other hand, are somewhat more elaborate, computationally intensive, and time consuming in terms of modeling. Therefore, the methods that fall in this class should be used for a final or detailed analysis. The following are classified as refined methods:

(1) The finite-element method (2) The finite-strip method (3) The finite-difference method (4) Analytical solution to differential equations (5) The slope deflection method

2.1.1 APPROXIMATE METHODS

(i) Plane-Grid Method

The structure in this method is modelled as an assemblage of two-dimensional grid members with one translational and two rotational degrees of freedom. This method does not account for warping. The method was first introduced by Lavelle and Boick (1965) and developed further at later dates by Lavelle et al. (1971) and Lavelle and Laska (1975a).

(ii) Space-Frame Method

This method was first introduced by Brennan and Mandel (1973) for the analysis of open and closed curved members. The curved members are idealized as three-dimensional straight members, while the diaphragms and lateral bracing are assumed as truss-like members that can carry only axial loads. The effect of warping is not usually included in this analysis.

(iii) V-Load Method

This method uses equivalent straight girders with span lengths equal to the arc lengths instead of the individual curved girders by adding self-equilibrating vertical shear forces (acting on diaphragm locations) that take the curvature into account. These loads are dependent on the radius of curvature, the bridge width, and the diaphragm spacing (Fiechtl et al, 1987; Poellot, 1987).

2.1.2 REFINED METHODS

(i) Finite-Element Method

This approach discretizes the structure into small divisions (elements) where each element is defined by a specified number of nodes. The behavior of each element (and ultimately the structure) is assumed to be a function of its nodal quantities (displacements and/or stresses), that serve as the primary unknowns in this formulation. This is one of the most general and accurate

methods to use, because it does not put any limitation on the geometry, loads, or boundary conditions, and can be applied to open/closed girders and static/dynamic analysis. Additionally, the structure's response can always be improved by refining the mesh and increasing the number of nodes (or degrees of freedom) for each element. However, the rather involved modeling and analysis efforts required by this method may in some cases make it impractical for preliminary analysis (Benedetti and Tralli, 1989).

(ii) Finite-Strip Method

In this numerical method, the curved bridge is divided into narrow strips in the circumferential direction that are supported in their radial direction. The analysis includes bending and membrane actions as well as warping and distortional effects (Hsu, 1989). Although this method provides some simplicity over the finite-element method because of the smaller number of unknowns required, it does not offer the flexibility and versatility of the latter method.

(iii) Finite-Difference Method

In this method, a grid is superimposed on the structure and the governing differential equations are replaced by algebraic difference equations that are solved for each grid point.

(iv) Solution to Governing Differential Equations (GDE)

In this class, an analytical solution to the GDE is obtained. The solution is usually a closed form or a convergent series solution, such as a Fourier series.

(v) Slope Deflection Method

In this approach, the partial differential equations are established in terms of slope-deflection equations, and the solution is assumed to be a Fourier series. The analysis includes the effects of curvature, non-uniform torsion and diaphragms (Zureick, 1999).

Cheung (1969) performed a study of horizontally curved bridge decks where the slab was divided into concentric simply supported strips and assumed the deflection to be a Fourier series in the longitudinal direction and a beam function in the transverse direction. Juhl (1970) derived the equations for a statically determinate lateral support system to minimize high internal stresses due to temperature change of the superstructure and to predict the boundary displacements of a skew and curved structure. Abdelraouf and Matlock (1972) introduced refined triangular planar and nonplanar elements and compared them with less refined triangular and quadrilateral elements in the analysis of bridge decks. Unlike the less refined elements, the proposed elements in Abdelraouf and Matlock (1972) proved suitable for coarse meshes. Cheung (1978) studied analytically and experimentally the behavior of simply supported curved bridge decks with intermediate column supports. The analytical study was based on the finite-strip method, the results of which compared favorably with experimental values obtained from testing thirty 1:60 scale asbestos cement curved slab decks (Zureick and Naqib, 1999).

In applying the First-Order Reliability Method to the evaluation of BS 449 (1969) design criteria for plate girders, Afolayan (1997) was able to accomplish the task of choosing structural dimensions and appropriate loadings that are commensurate with known levels of failure probability and consequent potential loss with known levels of performance.

Tete (2003), conducted safety checks on a horizontally curved single span concrete bridge deck slab and deduced that the variability of the dead load of the deck system has a great effect on its reliability and provided plots which can be used for a safe economical bridge deck system.

The type and arrangement of bearings for a bridge superstructure are important considerations in bridge design. For a curved continuous girder bridge, the support conditions for the bridge superstructure may significantly increase the distribution of maximum stresses, reactions, and shear forces, as well as the bridges natural frequencies and mode shapes (Linzell et al, 2010). Samaan et al (2002) described an extensive study carried out using an experimentally calibrated finite element model in which curved, continuous prototype bridges were analyzed to determine their structural response. Six different types and arrangements of support bearings were studied to determine their effects on the maximum stress and reaction distributions, as well as on the natural frequencies of such bridges.

2.2 ALUMINUM ALLOYS IN BRIDGES

There are lots of different aluminium alloys available, and each of these in diverse tempers or heat treatments, such that their groupings run into hundreds. However, there are only three families of alloys that need to be considered, and a relatively small number of alloys and tempers within each family. These are all alloys that are readily obtainable, have good corrosion and strength characteristics and are easily fabricated. Alternative alloys are available and are used in other industries, some offering much higher strengths at the expense of durability or workability, others offering better formability at the expense of strength. Aluminium alloys are categorized by the main alloying element and use an internationally recognized four-digit reference (Mazzolani, 2006).

The alloys that are of interest to the bridge engineer include; 5xxx series alloys having magnesium as the main alloying element. These alloys have the best corrosion resistance but are generally only available in sheet or plate form. The alloys have increased strength from work hardening during the rolling process and are available in several different degrees of hardness (O denotes the base condition; the letter H followed by two numbers denotes work-hardened material). A common alloy is 5083 H12. Proof strength and ultimate strength increase with the work hardening, but formability decreases. The alloys are readily available and are a good selection if forming a structure from plate materials. 6xxx series alloys have magnesium and silicon added as the main alloying elements. The 6xxx series alloys are readily extrudable as well as being available in sheet and plate form. These alloys are the most commonly used in structural and architectural applications, principally on account of the forms and shapes that can be created by extrusion. The alloys have their strength increased by heat treatment processes and are available in a range of tempers (indicated by the letter T followed by a number, e.g. 6082 T6).

They are readily weldable and give good all-round performance. 7xxx series alloys have zinc and magnesium as their main alloying elements. The 7xxx alloys are stronger than the 5xxx and 6xxx alloys, and have their strength increased by heat treatment. They are difficult to form and are more expensive than other common alloys. They are readily weldable and have good post-weld strength. Aluminium castings are also available. They are generally cast from different alloys to those used for plates and extrusions. The design of castings is highly specialised and is not covered in detail in the current design codes. They can be used if a suitable and rigorous testing regime is established. The Aluminium bridge deck used in this study is fabricated from the 6063-T6 aluminum.

Aluminium structures that are subjected to fluctuating service loads may be liable to fail by fatigue in a similar manner to steel structures. Fatigue failure usually initiates at a point of high stress concentration associated with abrupt changes in geometry or at welds. Careful attention to detail can therefore make significant differences to fatigue life. For bridge work, it is recommended that the structures are designed on a 'safe life' basis, i.e. the members are proportioned such that the predicted cyclic stress levels do not result in any fatigue cracks. In general, the 'allowable' fatigue stress is independent of the alloy being used (Tindall, 2008).

The theory of the fire safety of structures in aluminium alloys is governed by the same principles and methods as those used for steel structures. However, most of the aluminium alloys start to lose some strength when held at temperatures above 100 °C, and have lost a significant proportion by 350 °C. Contrary to popular opinion, aluminium is classified as non-combustible, and does not burn. Its high coefficient of thermal conductivity is advantageous in situations with local fires, in that the heat is dissipated rather than rising quickly at the fire location. Applications that need a specific fire resistance should have insulation applied in a similar manner to steel structures. Eurocode 9 Part 1-2 gives comprehensive rules for determining the fire resistance of aluminium structures in such cases (Tindall, 2008).

The first recorded use of aluminium for bridgework was for the reconstruction of the bridge deck of Smithfield Street Bridge in Pittsburgh in 1933. The saving in weight allowed a significant increase in bridge capacity and was a good example of the weight advantage of the material. The bridge continued in service with an aluminium deck for over 60 years. The Massena Bridge built in New York in 1946 incorporated a 30m span formed of two aluminium alloy riveted plate girders. Four years later, in 1950, the construction of the Arvida Bridge in Canada was

completed. With an arch span of 91.5m and total length of 153m this was the first example of an all-aluminium civilian bridge and is still one of the largest. They were fabricated from a high-strength alloy containing copper, which would not be recommended for structural use today, as the corrosion characteristics are significantly inferior to other alloys. Despite that, in 2008 the Arvida Bridge is still in service and maintenance work has been relatively low. Two opening bascule bridges were built of aluminium in the UK during the same period, the first being the Hendon Dock Bridge in Sunderland and the second in Aberdeen. These used a similar copper-based alloy for the deck plate, but used a 6000 series alloy for the riveted truss girders. The weight saving leads to economy in the bearings, machinery and counterweights, and it is surprising that aluminium has not been used more widely in opening bridges (Siwowski, 2006; Tindall, 2008; Subodh and Gilbert, 2007).

Several manufacturers have developed large multi-voided hollow extrusions specifically for forming orthotropic bridge decks. These have been used extensively in Sweden for bridge rehabilitation and over 70 bridges have had deteriorated concrete decks replaced by the SAPA bridge deck system. In the mid 1990's, Reynolds Metal company had developed several aluminum, made into various shapes and sizes. The recent suggestions of aluminum as bridge decks made use of the 6xxx-series, multiple hollow extrusions for decks and 5xxx-series for plate because of their superior combination of strength, corrosion resistance and overall ease of fabrication (Agboola et al, 2010). The first application was developed specifically for the 97.54m long wide historical corbin suspended bridge in Huntingdon county, Pennsylvania, a 60 year old suspended bridge was renovated in 1996 with a prefabricated orthotropic aluminium deck, thereby reducing the weight of the deck. The second use was for the US route 58 bridge over the Little Buffalo Creek, Clarksville, VA. The bridge was 16.25m long and 9.75m wide. The deck

was made of 6063-T6 extrusions welded on both the top flange and the bottom flange from one side with a removable backing (Robert, 2002). The extrusions were oriented parallel to traffic and attached to the four longitudinal beams to act as a composite.

Similar systems developed in other countries including Germany and Norway have also been used for the decks of new or replacement bridges. The extrusions are typically formed from alloy 6063 in the T6 condition. The weight of these deck systems is between 50 and 70 kg/m², which is only about one-tenth that of a typical concrete deck. The aluminium decks have good corrosion resistance which, allied to the low weight, makes them ideal for bridge upgrades, opening bridges and long-span bridges. Many of the Swedish decks have been surfaced with a 6mm thick acrylic surfacing system to minimize total weight. Others have a 40mm thick layer of a mastic asphalt type of surfacing. The close tolerances inherent in the extruded decks allows for good ride quality even with such thin surfacing systems. Aluminium bridge parapets have been used for many years, generally with no paint system or other surface treatment. The good corrosion resistance is highlighted by their excellent performance despite being subject to road pollutants, salt spray, etc. (Tindall, 2006).

The aluminium alloys in general structural use have excellent corrosion resistance, which is attributable to the protective oxide film which forms naturally on exposure to air. The film is usually invisible, relatively inert and adheres strongly to the metal surface. Once formed, it prevents further oxidation and reforms naturally if damaged. It is thus self-healing. The 5xxx and 6xxx series alloys will develop small pits when exposed to industrial pollution. A layer of corrosion product which inhibits further corrosion, seals the pits and results in a dull grey weathered appearance for untreated aluminium. The 7xxx series alloys exhibit a layered form of

corrosion rather than pitting, and have slightly inferior durability. The oxide film on the aluminium alloy acts as a natural barrier and until this breaks down from acidic or alkaline moisture, or through abrasion, that the bi-metallic cell will activate. Thus corrosion between stainless steel and aluminium, which have a large potential difference, is non-existent in most environments as the contact is between two inert oxide layers. Connections between dissimilar metals that are subject to fretting or relative movement can sometimes corrode rapidly, as the protective oxide film is repeatedly worn away. Insulating the dissimilar metals with an inert non-absorbent barrier can prevent reaction (Dwight, 1999).

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 are such that resource depletion will not be an issue. It is extracted by electrolysis of alumina from bauxite fused with cryolite. Extraction rates are good, with 4t of bauxite producing 1t of aluminium. Various studies, (Adresen, et al., 2001; O'Connor, 1995) have been carried out to consider ecological impacts by applying life-cycle analysis techniques, and comparing aluminium with other materials such as steel for structural components in buildings. Using the cumulative energy demand as a comparator, aluminium structures with an anodised or painted surface have slightly less of an impact than painted or galvanised steel, once recycling is taken into account. If the aluminium is left with no surface treatment, the difference becomes greater. The studies have generally assumed that the structural elements of the structure will not be repainted in their lifetime. Similar studies for bridgework should therefore show an even greater benefit from using aluminium. The electrolytic process used in a primary smelter is energy intensive, and it is not surprising that the majority of primary aluminium is produced using hydroelectric power. Current figures show that about 95% of aluminium used in the construction industry is recycled.

Energy consumption in the recycling process is only about 5% of that used in producing new aluminium. Other advantages accrue from savings in fabrication, transport and handling due to the low self-weight and from labour-saving features that can be incorporated in extrusion design.

Future trends; Research into structural application of aluminium alloys in civil engineering structures is disappointingly sparse, and most future developments are likely to arise from advances in other applications, such as the aerospace and transport industries that have different criteria to address. Traditionally, the use of castings have been discouraged for structural applications, as the quality and brittle characteristics of many castings 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 the development and tooling costs for any specific project or proprietary system (Tindall, 2008).

Development of material technology continues. New processes using fibre-reinforced aluminium and powder metallurgy promise materials that are stronger and stiffer 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. Developments in friction stir welding and adhesive bonding over the past ten years have yet to become firmly established in the civil engineering industry, and are not covered in detail by the current design codes. They are, however, well established in the manufacture of railway carriages and in military bridging (Siwowski, 2006; Tindall, 2008).

2.2.1 BRIDGE DYNAMICS

The response of bridges to dynamic vehicular loading is important to bridge evaluation. A bridge's strength, rating, and serviceability depends on the manner upon which the bridge behaves under dynamic loads. The actual behavior of a bridge may be different from the manner that it was designed. The actual load distribution characteristics and dynamic amplification of responses in a bridge have been studied using reliability analysis (Agboola et al., 2010).

Typically, analysis for vehicle- and wind-induced vibration is not to be considered in the bridge design. Although a vehicle crossing a bridge is not a static situation, the bridge is analyzed by statically placing the vehicle at various locations along the bridge and applying a dynamic load allowance as stated in AASHTO Specifications.

Curved and skewed I-girder bridges tend to significantly deflect and rotate out of plane under the action of gravity. Its construction must therefore control not only the vertical displacement of the I-girders but also the out-of-plane displacements such that structural components, including cross frames, can be erected with limited difficulty. To achieve better understanding of the effects of design on the geometry and load distribution in a curved/ skewed I-girder bridge system, further study on such bridges is required (Linzell et al., 2010); thus a probabilistic evaluation of an horizontally curved aluminum bridge deck is carried out herein.

Because of coupling action of the vertical bending and torsion, curved and skewed girders are subject to significant rotations. During construction, the non-composite 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 curved girder bridges during placement of concrete decks that make it difficult to maintain the specified final cambers and super-elevations

and to form and to key in the construction joints (Zureick and Naqib, 1999). The Aluminium alloy deck is shop fabricated and therefore will not provide the same issues as concrete decks during construction.

Recent studies carried out by the Virginia Transportation Research Council to evaluate two Reynolds aluminum deck systems gave results which demonstrated that aluminium bridge decks are a feasible alternative to reinforced concrete decks from the standpoint of strength and serviceability. Safety level studies carried out on straight girders by Agboola et al, (2010), also demonstrated the superiority of aluminium decks over conventional concrete decks on steel girders.

2.2.2 BRIDGE DESIGN

Parameters such as the Dynamic Load Allowance, DLA and the Girder Distribution Factors, GDFs significantly influence the design of a bridge. In a typical highway bridge, the load distribution behavior involves the distribution of load amongst the longitudinal supporting members of the bridge system. An accurate estimate of the distribution of load in a bridge allows for the development of more efficient and reliable bridge design codes. In bridge design, the maximum moments in the girders are necessary in the determination of the bridge section. The problem is three dimensional and involves complex behavior of load transfer from deck to steel girder (NSBA, 2005).

2.2.3 THE NEED FOR RELIABILITY-BASED DESIGN

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 (Abubakar, 2006).

Engineering design is usually a trade-off between maximizing safety levels and minimizing cost. However, deterministic safety factors do not provide adequate information to achieve optimal use of the available resources to maximize safety, while the probabilistic analysis does (Ellingwood, et al., 1980; Augusti et al, 2006; Melchers, 1999). The use of probabilistic analysis is expected to provide more information about system behavior, the influence of different uncertain variables on system performance, and the interaction between different system components (Ditlevsen and Madsen, 2005). For many years it has been assumed in design of structural systems that all loads and strengths are deterministic (Kirsch, 1981). The strength of an element was determined in such a way that it exceeded the load with a certain margin. The ratio between the strength and the load was denoted as the safety factor. This number was considered as a measure of the reliability of the structure. In codes of practices for structural systems values for loads, strengths and safety factors are prescribed. These values are traditionally determined on the basis of experience and engineering judgment (Augusti, et al., 2006).

However, in new codes, partial safety factors are used. Characteristic values of the uncertain loads and resistances are specified and partial safety factors are applied to the loads and strengths in order to ensure that the structure is safe enough. The partial safety factors are usually based on experience or calibrated to existing codes or to measures of the reliability obtained by probabilistic techniques (Melchers, 1999; Nowak and Collins, 2000). As described above structural analysis and design have traditionally been based on deterministic methods. However, uncertainties in the loads, strengths and in the modeling of the systems require that methods

based on probabilistic techniques in a number of situations have to be used (Ellingwood, et al., 1980; Ditlevsen and Madsen, 2005).

2.2.4 TYPES OF BRIDGE LOADINGS

According to AASHTO LRFD (2004), Road Research (1979) and the BS5400-2 (1978), have shown the types of loadings that act on bridges. According to Road Research (1979), 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 as well as loads due to creep and shrinkage. Dead load carried by a bridge member consists of its own weight and the portions of the weight of the superstructure 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 loadings used in this analysis are classified into dead loads and imposed loads. The imposed/ live load used is the HL-93 loading (AASHTO, 2010).

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

2.3 STRUCTURAL RELIABILITY

Sorensen (2003) defined reliability of structural systems as the probability that the structure under consideration has a proper performance throughout its lifetime. The reliability of a

structure could be defined as the ability that the structure fulfills its design purpose for some specified reference period (Togan & Daloglu, 2005).

Computer techniques of structural analysis have improved the accuracy of representing the actual behavior of bridge components. Advanced programs (e.g. ABAQUS, NASTRAN, ANSYS) are available for linear and nonlinear analysis of complex structural systems. A dense element mesh allows for an accurate determination of strain/stress at almost any point in the structure. One major problem that remains is how to represent boundary conditions and material properties. For example, the actual support insitu is often different than an idealized type, and strength of material and modulus of elasticity can be different than what is assumed in design. The deterministic analysis is a useful tool, but there is need to include the randomness of parameters such as these (Nowak, 2004).

The study of structural reliability is concerned with the calculation and prediction of the probability of a limit state violation for engineered structures at any stage during their life. In particular, the study of structural safety is concerned with the violation of the ultimate or safety limit state for the structure. The “violation” of a limit state is the attainment of an undesirable condition for the structure, i.e. damage to a part of the structure or total collapse of the structure which could lead to loss of human lives (Melcher’s, 1999).

Society expects buildings and bridges to be designed with a reasonable safety level. In practice, these expectations are achieved by following code requirements specifying design values for minimum strength, maximum allowable deflection and so on. Code requirements have evolved to include design criteria that take into account some of the sources of uncertainties in design. Such criteria are often referred to as reliability-based design criteria.

The reliability of a structure is its ability to fulfill its design purpose for some specified design lifetime. Reliability is often understood to equal the probability that a structure will not fail to perform its intended function. The term failure does not mean catastrophic failure but is used to indicate that the structure does not perform as desired. The reliability of a structure can be considered as a rational evaluation criterion. It provides a good basis for decision about repair, rehabilitation, or replacement (O'Connor, 1990). A structure is said to have failed when the nominal value of load exceeds the nominal value load carrying capacity. But in most cases, a structure is a system of components, and failure of one component does not necessarily mean failure of the structural system. When a component reaches its ultimate capacity, it may continue to resist the load while loads are redistributed to other components. System reliability provides a methodology to establish the relationship between the reliability of an element and the reliability of the system. Reliability based design, which accounts for uncertainties in design variables can be used as an essential means for checking the consistency of existing codes, with the view of optimizing the code so as to produce more consistent and economic codes for a better design (Nowak and Collins, 2000; Augusti et al., 2006; Melchers, 1999; Ellingwood, et al., 1980).

The probability of occurrence of an event such as limit state violation is a numerical measure of the chance of its occurrence. This measure either may be obtained from the measurement of the long-term frequency of occurrence of the event for generally similar structures, or may be simply subjective estimation of the numerical value. In practice, it is not usually possible to observe for a sufficiently long period of time, a combination of subjective estimation and frequency observations for structural components and properties used to predict the probability of limit state violation for the structure as a whole.

Structural reliability with respect to some limit states is quantitatively expressed by a number on a suitable scale being related to a given time period (Augusti et al, 2006). Taking for granted that the reliability evaluation is based on probabilistic modeling, the reliability measure scale is in one-to-one correspondence with the probability that the adverse event occurs within the considered time period (Ditlevsen and Madsen, 2005).

Lately, the use of reliability in structural assessments improved the quality of bridges concerning safety, serviceability, and durability (Nowak and Collins, 2000). Partial safety factors based on load and resistance uncertainties were proposed in the United States for building design codes not before the late 1970's (Galambos and Ravindra, 1978; Ellingwood, et al., 1980). 'New bridge design codes are based on probabilistic methods' (Nowak and Collins, 1999) and load and resistance factor parameters are considered as random variables. The reliability index, β , is used to measure the reliability of structural member as well as structural systems. This index accounts for the margin of safety implied by the design procedure as well as the uncertainties in estimating the number strengths and the applied loads. It is related to the probability of failure (Nowak and Collins, 2000).

2.3.1 LIMIT STATE FUNCTIONS

When applying probabilistic evaluation methods in Structural Engineering, it is essential to have a means of defining performance criteria. This is done by the limit state functions. In structural reliability analysis, the term 'failure' is used to denote the event of not meeting performance criteria. Limit states are the boundaries between safety and failure. In bridge structures failure can be defined as inability to carry traffic.

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. Members can fail in a ductile or brittle manner. In the traditional approach, each mode of failure is considered separately. There are three types of limit states. 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 example, in prestressed concrete girders, a crack opening under heavy live load is not a problem in itself. However, a repeated crack opening may allow penetration of moisture and corrosion of the prestressing steel. The serviceability limit states such as cracking, deflection or vibration, often govern the bridge design. The critical factors are both magnitude and frequency of load. Other serviceability limit states, vibrations or deflections, are related to bridge user's comfort rather than structural integrity. The third type of limit state is fatigue. The main concern is accumulation of damage caused by repeated applications of load (trucks). Therefore, the model must also include the load magnitude and frequency of occurrence, rather than just load magnitude as is the case in the ultimate limit states (Nowak, 2004).

Limit state functions can be defined in terms of engineering demand parameters, damage measures or decision variables. Estimation of the probability of the defined failure event poses a 'component' reliability problem. However, failure according to one performance criterion may not constitute failure of the entire system. Limit state functions for structural components are commonly denoted $g(x)$, where x is the vector of basic random variables. The dependence on x can be implicit and through the structural response quantities. However, $g(x)$ must be a continuous and differentiable function of x , at least in the realizable domain of x . The numerical

value of limit state function distinguishes the failure state from the safe state. Hence, $g > 0$ implies Structural element is safe, $g = 0$ limit state, $g < 0$ implies failure.

2.3.2 RELIABILITY OF BRIDGE STRUCTURES

The current increase in traffic will sooner or later pose threat on our existing bridges. Hence an urgent need for better monitoring of our existing bridges. The present methods used for bridge evaluation are expensive, defective and have many limitations (Kwasnieski et al, 2000). Various researches which deal with reliability of existing bridges had been carried out (Tedesco et al, 1995). Based on recent research the older deteriorated bridge decks can have their modulus of elasticity calibrated using the FE model (Tedesco et al, 1995). Kwasnieski et al (2000) investigated the sensitivity analysis for slab-on-girder bridges using the finite element method and concluded thus;

- (a) Boundary conditions and modulus of concrete are the most important parameters in the modeling of the actual behavior of bridges under service conditions.
- (b) Model without springs and with non-composite should be used in estimating load carrying capacities of bridge decks, and
- (c) The calculated maximum strains in the girders are smaller than the plastic strains.

Abenroth et al (1997) carried out a study on four span continuous aluminum I-girders which have been in service for about 35 years. Before it was demolished due to a need to construct an intersection at the location, load/ fatigue tests were conducted on the aluminum girders to determine its behavior and the results are outlined thus;

- (a) The nominal S-N curves obtained matched that obtained for category E of aluminum weldments given in the AASHTO LRFD (2004) specifications.

- (b) The category E fatigue fractures that developed in the I-girder test specimen satisfied the allowable S-N relationship as specified by the Aluminum Association.
- (c) The lower bound strength line set at two standard deviations below the least square regression line through the fatigue data agrees with the aluminum association S-N curve.

Thence, Abenroth et al (1997) have provided extra information regarding the typical behavior of aluminum members and confirmed that aluminum has the essential strength properties required for highway bridge girders. Also, the conserved tensile and fatigue properties of aluminum alloy members are consistent with what would be expected in a new structure today.

2.4 FINITE-ELEMENT METHOD

Many challenges face designers of modern engineering structures. Among them include the development of a reliable model capable of predicting the behavior of engineering designs in cost effective time scale. The Finite Element Method is a very powerful one, and it lends itself effectively to solution via computer. The method is easily applied to any structure with complex geometry made up of one or more materials and having a mixed set of boundary conditions (Zienkiewicz, 1971).

This approach discretizes the structure into small divisions (elements) where each element is defined by a specified number of nodes. The behavior of each element (and ultimately the structure) is assumed to be a function of its nodal quantities (displacements and/or stresses), that serve as the primary unknowns in this formulation. This is one of the most general and accurate methods to use, because it does not put any limitation on the geometry, loads, or boundary conditions, and can be applied to open/closed girders and static/dynamic analysis. Additionally, the structure's response can always be improved by refining the mesh and increasing the number

of nodes (or degrees of freedom) for each element. However, the rather involved modeling and analysis efforts required by this method may in some cases make it impractical for preliminary analysis (Benedetti and Tralli, 1989).

The finite method of analysis is a widely accepted numerical technique for the solution of a wide variety of problems found in engineering. The method is now by far the most effective tool available to analysts interested in computer assisted solutions of complex engineering problems. The method has the virtue of simplicity in concept, precision in development and potency in application. As a result of this, there exist at present numerous computer programs that can handle, at reasonable cost, very large finite element systems of great engineering significance. Accuracy and efficiency are the two major concerns in any finite element analysis that are forcing engineers and design analysts to seek reliable yet economical methods to determining the responses of structural components (Shim, et al,2002; Lydzinski, et al, 2008)

Finite Element Method is a generalization of standard structural analysis procedures, which permits the calculation of stresses in two or three dimensional structures. It approximates the governing differential equations for a given system with a set of algebraic equations relating to a finite number of variables to specific points called nodes. This method can be used for solving structural frameworks and other elastic continua utilizing discrete elements. The solution obtained in this case for joints displacements and member forces are identical to solutions obtained using other structural analysis methods (Cook, et al, 1989).

The Finite Element Method of analysis consists of a computer model of a material or design that is stressed and analyzed for specific results. The Finite Element Method originated as a method of stress analysis. Today, finite elements are also used to analyze problems of heat transfer, fluid

flow, lubrication, electric and magnetic fields and many others (Monaghan, 2001). Problems that previously were utterly intractable are now solved routinely. Results are rarely exact. However, errors are reduced by processing more equations and results accurate enough for engineering purposes are obtainable at reasonable cost. In general, Finite Element Method models a structure as an assemblage of small parts (elements). Each element is of simple geometry and therefore is much easier to analyze than the actual structure. In essence, we approximate a complicated solution by a model that consists of piece wise continuous simple solutions. Elements are called 'finite' to distinguish them from differential elements used in calculus (Cook, et al, 1989).

Alexander and Gunasekara (1991) stated that finite element analysis can be divided into three specific methods:

- (1) The direct stiffness or displacement method where the nodal displacement are the basic unknowns in the set of algebraic equations.
- (2) The force method in which the internal nodal forces are unknown.
- (3) The mixed method where the equations may be expressed in terms of nodal displacement and internal forces.

It was found (Clough, 1965) that for highly complex structures of arbitrary form, the displacement method provides the simpler formulation and computer programming task. Therefore, it is the most widely used. Mathematically, the structure to be analysed is subdivided into a mesh of finite elements of simple shape. Within each element, the variation of displacement is assumed to be determined by simple polynomial shape functions and nodal displacements. From these the equations of equilibrium are assembled in a matrix form which can be easily programmed and solved on a computer. After applying the appropriate boundary

conditions, the nodal displacements are found by solving the matrix stiffness equation. Once the nodal displacements are known, element stresses and strains can be calculated.

2.5 STOCHASTIC FINITE ELEMENT METHOD

Stochastic finite element methods 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. The finite element analysis and the stochastic (probabilistic) analysis. The 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 (Klieber and Hein, 1992). It is used to describe the merging of advanced reliability methods with the finite element method to obtain probability estimates for predefined performance criteria. These methods are generally applicable to both linear and non-linear problems. Stochastic finite element method is the generalization of the deterministic finite element method to incorporate the random fluctuations in material and geometric properties of the model and the random forces that may act on it. The aim of stochastic finite element is two folds, first, to compute the variability of the response of a random system and subsequently, using these results to estimate the structural reliability. The key steps involved in the analysis of the system using stochastic finite element method are:

- (1) Selection of appropriate probability models to describe the uncertainties in the system parameters and the boundary conditions.
- (2) Discretization of the random fields, i.e., the replacement of these random fields by an equivalent set of random variables.

- (3) Formulation of finite element equations, using standard methods like energy method, virtual work method or weighted residual method and its solution.
- (4) Estimation of system response statistic, which involves the inversion of random matrix operators.
- (5) Using these results to make a safety assessment of the structure.

Stochastic finite element method combines areas of finite element analysis and that of reliability. The distinguishing feature of stochastic finite element method, which is based on the perturbation approach, is treating probabilistic non-linear problem with deterministic computational techniques of the mathematical properties of linear operators (Raji, 2003). SFEM is an extension of the classical deterministic FE approach to the stochastic framework i.e. to the solution of stochastic (static and dynamic) problems involving finite elements whose properties are random. From a mathematical point of view, SFEM can be seen as a powerful tool for the solution of stochastic partial differential equations (PDEs) and it is treated as such in numerous studies where convergence and error estimation issues are examined in detail.

In order to apply the finite element technique, the region of interest is discretised by a finite element mesh. The basic idea of the mean-based, second-moment analysis as used in stochastic finite element analysis is to expand, via Taylor series, all the vector and the matrix stochastic field variables typical of deterministic finite element method 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 (Kleiber and Hein, 1992; Raji, 2003; Musa, 2003).

2.6 DESCRIPTION OF THE BRIDGE DECK (ALUMADECK™)

Figure 2-1 refers to the two-dimensional view of an aluminium deck which is proposed for evaluation in this work.

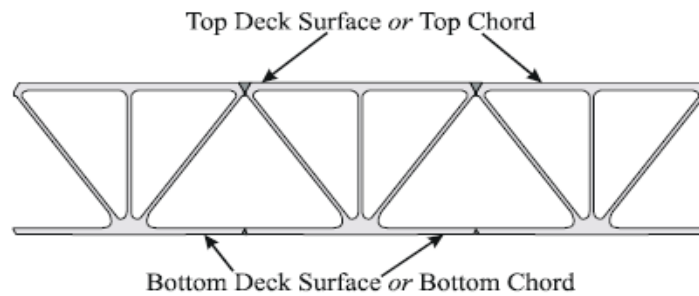


Figure 0-1: Nomenclature of Top and Bottom Deck surfaces

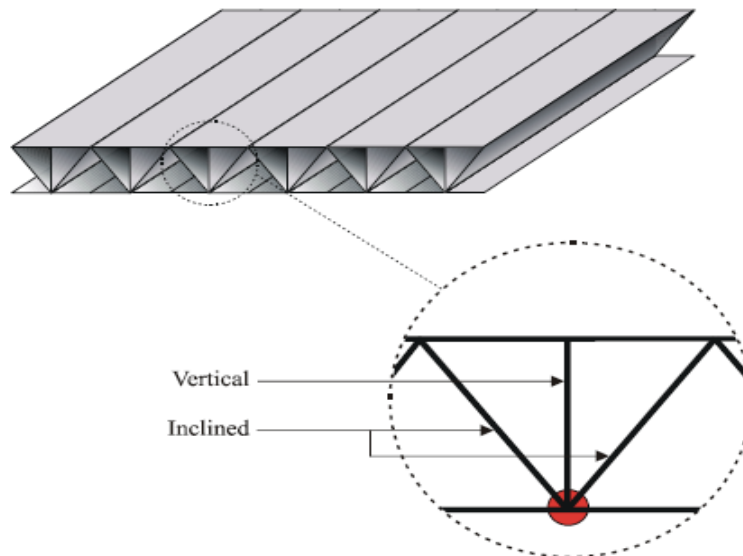


Figure 0-2: Nomenclature of stiffeners

As shown in Figures 2.1 and 2.2 the Alumadeck comprises of the top and bottom surfaces and vertical and inclined stiffeners.

2.7 LOADINGS AND LOAD APPLICATION

2.7.1 DEAD LOADS

According to AASHTO, (2010), the dead load is to include weight of all components of the structure, appurtenances and utility, earth cover, future overlays, and planned widening. Dead loads are sub-divided into three categories:

Dead load of structural component (DC) which is divided into weight of aluminium deck, DC 1 and weight from barriers, medians, and side walk/ weight of parapets acting on the composite structure, DC 2. Future wearing surface (FWS) loads are considered DW loads.

2.7.2 LIVE LOADS

The live loads of interest in this research are the AASHTO design vehicular live loads with the appropriate dynamic load allowance, multiple presence factors and centrifugal forces evaluated in this work. Braking forces and vehicular collision forces are neglected due to their anticipated small effect on the bridge responses. Additionally, according to the provisions of AASHTO (2003), the effect of superelevation can be neglected for superelevation angles between 0 and 10 percent. Therefore, the effect of superelevation is neglected. Since the bridge is horizontally curved, the effects of centrifugal force must be considered. Centrifugal force is a radial force, applied above the deck, which is transferred through the wheels of the vehicle to deck. Since centrifugal force is applied above the deck, it creates an overturning moment. As a result, the overturning moment tends to increase the vertical wheel forces towards the outside of the bridge and decrease them towards the inside of the bridge.

AASHTO (2010), specifies that the loading, designated as HL-93, comprises a combination of:

1. Design truck or tandem with dynamic allowance.

The design truck is the same as the HS20 design truck specified by the Standard Specifications.

The design tandem consists of a pair of 144 kN axles spaced at 1.22 m apart.

2. Design lane load of 9.36 kN/m without dynamic allowance.

$$IM = 33\%$$

Where IM = Dynamic load allowance, applied to design truck or design tandem only

The number of design lanes = the integer part of the ratio of $w/3.65$, where w is the clear roadway width between the curbs:

$$w = 14.59 \text{ m}$$

Number of design lanes = the integer part of the ratio of $(48/3.65) = 4 \text{ lanes}$

Multiple presence factor, m : for four lanes, $m = 0.65$. Stresses from truck and lane loads obtained from refined analysis will be multiplied by 0.65.

2.7.3 LANE LOADS

The lane load is uniformly distributed over a 3.05m width within the 3.65m design lane. To maximize the effect of the live load, the 3.05m loaded width is shifted to the left within each design lane. This causes the lane load to have an eccentricity of 34.8mm relative to the lane centerline, and the four lane loads have an eccentricity of 34.8mm relative to the bridge centerline. The average arc length increases by the ratio of 183.19m radius/ 182.9m radius, to 36.64m.

The total lane loading for the four design lanes is $4(36.64 \text{ m}) \left(9.36 \frac{\text{kN}}{\text{m}}\right) (0.65) = 891.7 \text{ kN}$.

The 0.65 factor is the factor, m .

2.7.4 TRUCK LOADING

The total weight of the design truck is $35.58 + 142.34 + 142.34 = 320.25 \text{ kN}$. Including 33% impact, $1.33 \times 320.25 = 425.94 \text{ kN}$.

For 4 trucks including the multiple presence factor, m :

$$4(425.94)(0.65) = 1,107.4 \text{ kN} \quad (2.1)$$

$$\text{Total live Load} = 891.7 + 1107.4 = 1999.1 \text{ kN} \quad (2.2)$$

2.7.5 LOAD APPLICATION

The rules for the application of the AASHTO (2004) loadings are given as follows:

- (i) The loading or standard truck loading shall be assumed to occupy a width of 3.0 m . These loads shall be placed in 3.3 m wide design traffic lanes spaced across the entire bridge roadway width in numbers and positions required to produce maximum stress
- (ii) Each 3.0 m lane loading or single standard truck shall be considered as a unit and fractional values for lane loading or trucks shall not be used.
- (iii) AASHTO allows for reduction in maximum stress that are produced in any member by loading any number of traffic lanes simultaneously. For one or two lanes (100%); Threelanes (90%); Four or more lanes (70%).

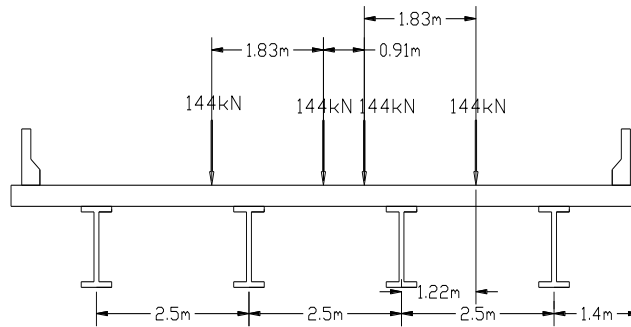


Figure 0-3: Section A-A of the bridge illustrating the HL93 loading locations.

2.7.6 CENTRIFUGAL LOAD

The design speed is 40 mph (64.4 km/hr). the centrifugal force coefficient is given by:

$$C = \left(\frac{4}{3}\right) \frac{V^2}{gR} \quad (2.3)$$

Where

C = coefficient to compute centrifugal force

V = highway design speed, m/s

g = gravitational acceleration, 9.81 m/sec²

R = radius of curvature of the traffic lane, m

The design speed in m/s = 40 mph/0.628 = 18 m/sec

$$C = \left(\frac{4}{3}\right) \frac{(18)^2}{(9.81)(182.88)} = 0.24$$

This is applied to the truck axle loads only, without the dynamic load allowance, and with the factor, m . The centrifugal force for four trucks is $4(320.26 \text{ kN})(0.24)(0.65) = 199 \text{ kN}$.

For the design of bridges, both the truck and lane loading are to be considered and the one which yields the worst effect should be adopted. In case of truck loading, only one truck is considered for each traffic lane for the whole of its length. There is no reduction in load intensity for up to two lanes of traffic loaded lanes.

2.7.7 DISTRIBUTION OF LIVE LOAD

When a concentrated load is placed on a bridge deck, the load is distributed over an area larger than the contact area on account of the stiffness of the slab. For wheel load distribution factor, AASHTO (1993) standard equation recommends the S-over equation for concrete slab on steel girder bridges with two or more lanes loaded as given in the Equation (2.1)

$$S/1676 \tag{2.4}$$

Where S is the girder spacing in (mm). A more accurate LDF equation is introduced in the AASHTO LRFD, (2004) which is based on finite element analysis and statistics. The wheel load distribution factor equation from AASHTO LRFD for concrete slab on steel girders is

$$DF = 0.06 + \left(\frac{S}{4272}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{Lt_s^3}\right)^{0.1} \text{ for one lane loaded} \tag{2.5}$$

$$DF = 0.075 + \left(\frac{S}{2896}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{Lt_s^3}\right)^{0.1} \text{ for two or more lane loaded} \tag{2.6}$$

Where S is the girder spacing in (mm), $K_g = n(I - Ae^2)$ is the longitudinal stiffness (mm^4), t_s is the slab thickness in (mm), and n the modular ratio between steel and concrete, I is the moment

of inertia of the steel girder (mm^4), A is the girder area (mm^2), and e is the eccentricity between the centroids of the girder and the slab.

Other loads to be considered in the design of decks are thermal force, braking force, vehicle collision, seismic loads, wind loads, ice loads, water loads, construction loads, transportation loads whilst limit states considerations include strength, constructability, serviceability, fatigue and fracture, and extreme event.

2.8 HIGHWAY BRIDGE LOADING ACCORDING TO BS 5400: PART 2, 1978

Clause 4.4.1 of BS 5400 states the combination 1 which includes the permanent loads together with the appropriate primary live loads as used in this study. For the load component, the dead load includes self-weight of all permanent structural members of the bridge system. The superimposed dead load includes weight of filling imposed upon the deck of the bridge such as wearing surface made of asphalt. Clause 5.2.2 of BS 5400: part 2(1978) specified the factor to be applied to all parts of superimposed dead load as 1.75 for ultimate limit state.

Standard highway live load consists of HA and HB loading. HA loading is a formula loading representing normal traffic in Great Britain. HB loading is an abnormal vehicle unit loading. The BS 5400 considered loading of a closely spaced vehicle of 24t laden weight in each of the two traffic lane for load length up to 30m.

The primary live load is the static live load due to the mass of traffic on the highway while secondary live loads are due to change in speed or direction of vehicle traffic, e.g. lurching, nosing, centrifugal, longitudinal, skidding and collision loads.

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

Type HA loading consists of a uniformly distributed load and a knife edge load combined or of single wheel load. The nominal uniformly distributed load for the type HA loading is taken as $30kN$ per linear meter of notional lane for loaded length up to $30m$. For load length in excess of $30m$, the equation below is used to derive the required load on the bridge (BS 5400, 1978)

$$W = 151 \left(\frac{1}{L}\right)^{0.475} \text{ but not } > 9 \quad (2.7)$$

Where:

L = Loaded length (m)

W = Load per meter of lane (kN)

The Nominal knife edge load (KEL) per notional lane shall be taken as $120kN$. According to BS5400, The UDL and KEL shall be taken to occupy one notional lane, uniformly distributed over the full width of the lane.

2.9 LIMIT STATES

There are several limit states considered during design some of which include the constructability, fatigue, strength, live load deflection and serviceability limit states. The strength limit state is the most crucial and will be considered in this study.

2.9.1 STRENGTH

Live load responses for HL-93 plus the dynamic load allowance are to be generated for the strength limit state during design. One, two and three traffic lanes are to be considered. Multiple presence reduction factors are to be applied. The centrifugal force effects are to be included.

2.9.2 CONSTRUCTABILITY

The erection sequence is to be investigated to check both deflections and stress according to article 6.10.3. The effects of forces from deck overhang brackets acting on the fascia girders is to be considered.

2.9.3 FATIGUE

The range of stress for fatigue is determined by computing the maximum and minimum stress due to one fatigue truck traversing the length of the bridge in the critical transverse position on the deck for each response. The transverse position of the truck may be different for each response and for positive and negative values of the same response. The fatigue truck is assumed to travel in either direction, or in opposite directions, to produce the maximum stress range.

2.10 LOAD COMBINATION

There are two Load combinations that are considered in this study:

(a) The strength I = $1.25 DC1 + 1.25 DC2 + 1.5DW + 1.75(LL + IM + CE + BR) + 1.2(TU)$ (2.8)

(b) The service II = $1.0DC1 + 1.0DC2 + 1.0DW + 1.33(LL + IM + CE + BR) + 1.2 (TU)$ (2.9)

LL is the vehicular live load which included/ covered for IM the vehicular dynamic load allowance, CE the vehicular centrifugal force, BR the vehicular braking force and TU Uniform temperature.

2.11 BRIDGE DECK DESIGN

According to AASHTO LRFD (2010), the primary direction for analysis and design of the deck slab is perpendicular to the center line of traffic. The deck slab is considered to be a continuous and a simply supported beam in the transverse direction. The LRFD Specifications allow the use of either a refined or an approximate method of analysis. An approximate method of analysis can be utilized to determine the lateral live-load distribution to individual girders for typical highway bridges. Lateral live-load distribution factors are dependent on multiple characteristics of each bridge. There are specific ranges of applicability for the use of approximate methods of analysis. Extending the application of such approximate methods beyond the limits requires sound and reasonable judgement. Otherwise, a refined analytical method should be used.

Chu and Krishnamoorthy, (1962), showed that the reactions on a plate are influenced by flexural and torsional rigidities of slab loadings. According to Clifton, et al, (1963), the slab and the girders of a bridge deck structure should be treated as a single plate system restrained by girder and end plate moment reactions. The material to be used for the bridge deck should be elastic in behavior and can be described as an orthotropic material (Troitsky, 1976). Curved plate equations have been derived and used for stiffed behavior, deflections and end reactions in previous studies.

Daiguji (1972), carried out an analytical study of curved orthotropic steel bridge decks using the Finite Strip Method to determine the moment and deflection, and then concluded that the difference between the finite strip solution and the examples in ASCE manual is less than 5%. Also, Daiguji deduced that for larger radius, horizontally curved steel decks almost gave the same results as the right bridge deck analyses; at radius less than 60.96m (200ft), the right orthotropic theory is no longer applicable to the analysis of curved decks.

For analysis of orthotropic plates, which are in a state of elastic deformation due to flexural loads it is necessary to determine the elasticity and rigidity of the materials. The mathematical methods of solution to orthotropic plates to determine deflection and force reactions include series analysis and finite difference method, slope deflection method and finite element theory.

The basic partial differential equations governing the behavior of a curved plate are derived on the assumptions that external loads are normal to the surface of the deck plate, that the plate acts as a monolithic unit, that the deck plate is homogenous, isotropic and elastic, that the plane surface initially perpendicular to the middle surface of the deck plate remains plane and perpendicular to the middle surface during bending, and that the deflections are small in relation to the thickness of the orthotropic plate (Clifton, et al, 1963).

Heins and Hails (1969), showed that the reactions and deflections were dependent on the flexural and torsional properties of the plates. Analysis of a curved plate or girder can be done by the solution to the partial differential equation using finite difference, finite element and slope deflection method.

2.11.1 DESIGN OF CURVED DECKS

According to Timoshenko and Woinowsky, (1982) and Heins and Hails (1969), the differential equation for flexure of a curved plate is given as;

$$D_r \frac{\partial^4 \eta}{\partial r^4} + 2H \frac{\partial^4 \eta}{r^2 \partial r^2 \partial \theta^2} + D_\theta \frac{\partial^4 \eta}{r^4 \partial \theta^4} + 2D_r \frac{\partial^3 \eta}{r \partial r^3} - 2H \frac{\partial^3 \eta}{r^3 \partial r \partial \theta^2} - D_\theta \frac{\partial^2 \eta}{r \partial r^2} + 2(D_\theta + H) \frac{\partial^2 \eta}{r^4 \partial \theta^4} + D_\theta \frac{\partial \eta}{\partial r^3 \partial r} = q \quad (2.10)$$

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

$$D_r = \frac{Et^3 r}{[12(1-\mu^2)]} \quad (2.11)$$

$$D_{\theta} = \frac{Et^3r}{[12(1-\mu^2)]} \quad (2.12)$$

$$H = GK_1 \quad (2.13)$$

$$D_{r\theta} = H/2$$

where P = Loading, r = Radius of bridge, E = Modulus of elasticity, K₁ = Torsional Constant, μ = Poison Constant, t = Thickness of Deck, r = Unit Radius of Deck, G = Modulus of Rigidity.

r = Radius; θ = Angle subtended by section; η = Deflection of the section; q = Uniformly Distributed load; D_{rθ} = Torsional rigidity; D_r = Flexural rigidity in the r-direction; D_θ = Flexural rigidity in the θ direction

The general solution to equation (2.4) determined by Heins and Hails (1969) is

$$\eta = \sum[AX^{m_1} + BX^{m_2} + CX^{m_3} + DX^{m_4}] \sin \lambda\theta + \frac{4pr^4 \sin \lambda\theta}{D_r n\pi[72-18\beta\lambda^2-\alpha(8+2\lambda^2-\lambda^4)]} \quad (2.14)$$

$$\alpha = \frac{D_{\theta}}{D_r} = 1 \quad (2.15)$$

$$\beta = \frac{H}{D_r} = 0.98 \quad (2.16)$$

$$\lambda = \frac{n\pi}{\theta} = 0.37 \quad (2.17)$$

$$X = \frac{r_i}{r} \quad (2.18)$$

In which the four roots of the equations m₁, m₂, m₃, and m₄ can be expressed as;

$$m_1, m_2, m_3, m_4 = \pm \left\{ (\alpha + 2\beta\lambda^2 + 1)/2 \pm [0.25(1 + \alpha^2\beta^2)^2 - (\lambda^2 - 1)^2\alpha]^{1/2} \right\}^{1/2} + 1 \quad (2.19)$$

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 as follows;

$$M_r = -D_r \frac{\partial^2 \eta}{\partial r^2} \text{ (Radial Moment)} \quad (2.20)$$

$$M_\theta = -D_r \frac{\partial \eta}{\partial r} + \frac{\partial^2 \eta}{r^2 \partial \theta^2} \text{ (Angular Moment)} \quad (2.21)$$

$$R_r = \left[\left(\frac{\partial^3 \eta}{\partial r^3} + \frac{\partial^2 \eta}{\partial r^2} \right) + 2 \frac{H}{D_r} \left(\frac{\partial^3 \eta}{r^3 \partial r \partial \theta^2} - \frac{\partial^2 \eta}{r^3 \partial \theta^2} \right) - \frac{D_\theta}{D_r} \left(\frac{\partial \eta}{r^3 \partial r} + \frac{\partial^2 \eta}{r^3 \partial \theta^2} \right) \right] \text{ (Radial Shear)} \quad (2.22)$$

$$R_\theta = -D_\theta \left[\left(\frac{\partial^2 \eta}{r^2 \partial r \partial \eta} + \frac{\partial^3 \eta}{r^3 \partial r \partial \theta} \right) + 2 \frac{H}{D_\theta} \left(\frac{\partial^2 \eta}{r^2 \partial r \partial \theta} + \frac{\partial^2 \eta}{r^2 \partial r \partial \theta} + \frac{\partial \eta}{r^3 \partial r} \right) \right] \text{ (Angular shear)} \quad (2.23)$$

Equations (2.9) and (2.10) are expressed in terms of the general solution of Equation (2.8), 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 will be neglected. Thus,

$$-M_r \frac{r^2}{D_r} = \left[[(Am_1(m_1 - 1))X^{m_1} - Bm_2(m_2 - 1)X^{m_2} + Cm_3(m_3 - 1)X^{m_3} + Dm_4(t)X^{m_4}] \right] \sin \lambda \theta \quad (2.24)$$

And

$$\begin{aligned}
-R_r \frac{r^3}{D_r} = & [Am_1(m_1 - 1)^2 - 2\beta\lambda^2(m_1 - 1) - \alpha(m_1 - \lambda^2)]X^{m_1} + [Bm_2(m_2 - 1)^2 - \\
& 2\beta\lambda^2(m_2 - 1) - \alpha(m_2 - \lambda^2)]X^{m_2} + [Cm_3(m_3 - 1)^2 - 2\beta\lambda^2(m_3 - 1) - \alpha(m_3 - \lambda^2)]X^{m_3} + \\
& [Dm_4(m_4 - 1)^2 - 2\beta\lambda^2(m_4 - 1) - \alpha(m_4 - \lambda^2)]X^{m_4} \sin \lambda\theta
\end{aligned} \tag{2.25}$$

Figure 2-4 shows the bridge deck geometry on plan.

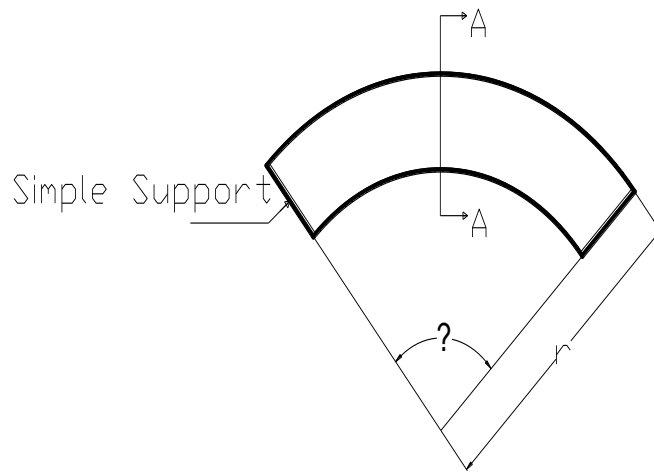


Figure 0-4: Bridge Deck geometry on plan

2.12 CURVED BEAM DESIGN

Heins (1975) derived charts for curved beams which can be used to determine the forces. The equations used from the charts can be expressed as

$$M_{radial} = M_{straight}K_1K_2K_3 \tag{2.26}$$

$$F_{radial} = F_{straight}K_1K_2K_3 \tag{2.27}$$

where K_1 is the Amplification factor, K_2 is the Distribution factor and K_3 is the Reduction factor.

$$\text{For moments, } K_1 = \frac{0.15}{n} \times \frac{L}{R} = 1.0 \tag{2.28}$$

$$K_2 = \frac{0.4L(n+3)}{R} + 0.6 = 1.0 \quad (2.30)$$

$K_3 = 1.0$ Since it is single span

$$n = \frac{R}{100}$$

For Shear, $K_1 = 1.0$

$$K_2 = \frac{0.4L(n+3)}{R} + 0.6 = 1.0 \quad (2.31)$$

$K_3 = 1.0$ Since it is single span

Again $K_1 K_2 K_3 = K$ which can be expressed as $K = 0.078\theta + 0.669$ where θ is the angle subtended by the girders in radians (Tete, 2003). The greater of the K values obtained is used in the design.

2.13 THE AASHTO (2003) FLEXURAL RESISTANCE EQUATIONS

The AASHTO (2003) flexural resistance equations that address the combined effects of major-axis bending and flange lateral bending are

$$f_{bu} + \frac{1}{3}f_l \leq \phi_f F_n \quad (2.32)$$

for members in which the flexural resistance is expressed in terms of stresses and

$$M_u + \frac{1}{3}f_l S_x \leq \phi_f M_n \quad (2.33)$$

for members in which the flexural resistance is expressed in terms of moments, 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

$$M_p = \frac{P_w}{2D} [\bar{y}^2 + (D - \bar{y})^2] + P_s d_s + P_c d_c + P_t d_t \quad (2.35)$$

$$P_s = f_s A_s \quad (2.36)$$

$$P_c = f_{yc} A_{fc} \quad (2.37)$$

$$P_t = f_{yt} A_{ft} \quad (2.38)$$

$$P_w = f_{yw} A_w \quad (2.39)$$

$$Y_{bar} = \frac{D}{2} \left(\frac{P_t - P_c - P_s}{P_w} + 1 \right) \quad (2.40)$$

$$d_s = \frac{t_{slab}}{2} + t_{haunch} + Y_{bar} \quad (2.41)$$

$$d_c = Y_{bar} - \frac{t_{fc}}{2} \quad (2.42)$$

$$d_t = D - \bar{y} + \frac{t_{ft}}{2} \quad (2.43)$$

$$D_p = Y_{bar} + t_{slab} + t_{haunch} \quad (2.44)$$

$$D_t = D + t_{haunch} + t_{slab} + t_{ft} \quad (2.45)$$

$$M_n = M_p \text{ if } D_p \leq 0.1D_t; \text{ otherwise, } M_n = M_p \left(1.07 - \frac{0.7D_p}{D_t} \right) \quad (2.46)$$

where f_s = Slab bending stress, A_s = effective area of slab, f_{yc} = yield stress of the compression flange, A_{fc} = Effective area of compression flange of 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 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 alumadeck), t_{slab} = thickness of the alumadeck slab, t_{fc} = thickness of compression flange, t_{ft} =thickness of tension flange and t_{haunch} = thickness of haunch.

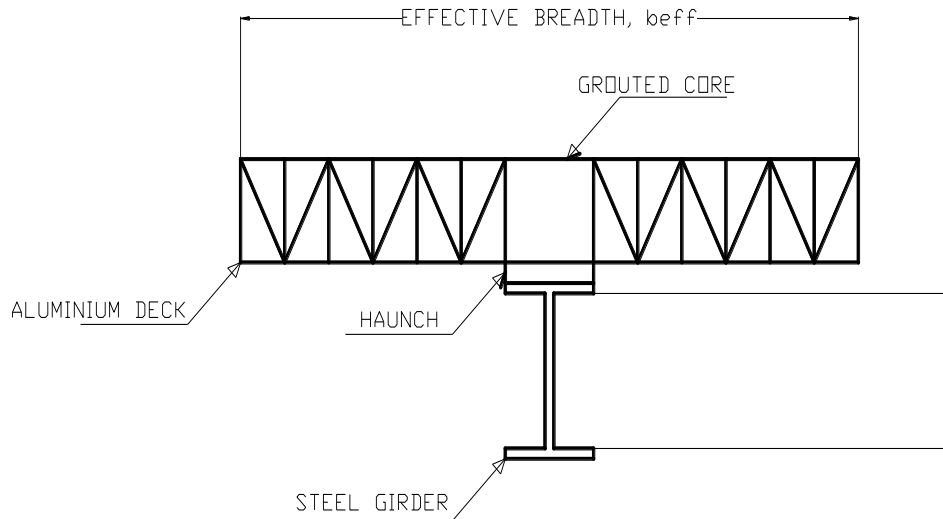


Figure 0-5: Girder effective width for composite action

2.14 APPLICATION OF RELIABILITY/ PROBABILISTIC EVALUATION

Structural design has been moving towards a more rational and probability based design procedure referred to as Limit State Design (Melchers, 1999). Such design procedure takes into account more information than deterministic method in the design of structural components. This information includes uncertainties in the strength of various structural elements, in loads, and modeling error in the analysis procedure. The non deterministic nature of some parameters (e.g., loads, material properties, dimension, etc.), which govern the behaviour of certain structural components, has currently generated considerable interest in the study of structural reliability (Melchers, 1999).

It is generally accepted that reliability methods can be characterized into one of three levels;

Level 1 uses partial safety factors to imply reliability and is used in simple codes.

Level 2 is known as second moments of FORM. The random variables are defined in terms of means and variance and considered to be normally distributed. The measure of reliability is

based on the reliability index. In advanced level 2 methods, the design variables can have any form of probability distribution.

Level 3 has multi-dimensional joint probability distributions. System effects and time variance may be incorporated. These include numerical integration and simulation techniques.

A fourth level “Level 4,” as defined by merging any of the above together with economic data from prediction of maximum benefits or minimum costs (Webster and Bannister, 2001).

All the methods are approximation methods and the problems become more difficult as the number of random variables and the complexity of the limit state function increase, and when statistical dependence between random variables is present.

Reliability analysis consists of the calculation that resistance will effectively exceed the applied load of a structure during its designed life time, with this reason; safety becomes a measure of the probability of survival. A limit state function $g(x)$, thus defines the resistance and load effects and various regions can be defined as failure or safety depending on whether the $g(x) \leq 0$ or not.

The proposed aluminium deck on steel girders is evaluated using FEM and reliability analysis and the formulations of AASHTO (2003) and AASHTO LRFD (2010) with parameters such as stiffener thickness, percentage composite action, etc, in order to determine the possibility of use in horizontal curves and sometimes in skewed situations. The safety of doing this is also evaluated. The methodology employed is discussed in the next chapter.

CHAPTER THREE

3 METHODOLOGY

3.1 GENERAL

We do not expect a bridge to collapse when we drive on it, so structures such as bridges are required to be safe. But, structures cannot be safe totally from collapse, hence there must be a risk of failure. Building a totally safe structure would mean using an infinite amount of resources in order to withstand an infinitely large load, this would be undesirable. Loads on bridges fluctuate with time and are difficult to predict, but can be described by stochastic processes. The geometry and material strength of a structure can therefore be described as stochastic variables.

The use of structural assessment can improve the quality of bridges in terms of safety, serviceability, and durability. The AASHTO (2010) bridge code is based on probabilistic methods where the load and the resistance parameters are considered as random variables. Safety and reliability theory is a method based on mathematical statistics whereby the material strengths, the loading process with time and model uncertainties are described by stochastic processes and variables and statements can be made about safety and the mean time to failure of a structure. The safety index, β , is used as a measure of safety margin between the design procedure and the uncertainties involved in estimating the member strengths and applied loads. The system reliability assessment is much more difficult than the reliability of individual components making up the system (Nowak and Collins, 2000).

Reliability-based design is found on the concept that one can estimate the probability of an undesirable event such as a fracture, occurring over the life time of a structure, despite the

uncertainties involved. It is a design method that provides an assumed level of safety by reducing the probability of such an occurrence below the target value (Melchers, 1999)

The quality of design is judged by comparing the failure probability with the target failure probability, and this use of the failure probability makes it possible to evaluate the safety of a structure quantitatively. Above all, the design method enables one to gain a firm grasp of the safety level desired and to design a structure so as to attain the prescribed reliability independently of other design requirements.

3.2 CONCEPT OF STRUCTURAL RELIABILITY ANALYSIS

The fundamental concept of reliability analysis is that resistance and load factors are statistical quantities with a central tendency (mean), dispersion about the mean (variance) and some form of distribution (probability density function e.g. normal, lognormal, gumbel). When combined together via an expression to describe the limit state (such as failure or collapse) there will be a finite Probability that the load will exceed the resistance; this defines the probability of failure (P_f) and since reliability is equal to $1-P_f$ the inherent reliability of the component against a particular failure mode and with given resistance properties is defined.

In determining the overall reliability of a structural system, account must be taken of the possible failure modes of the structure or structural elements, making due allowance for the correlation arising from common sources of loading and common material properties. However, from the definition of reliability as “probability that a structure will not attain each limit state (ultimate or serviceability) during a specified reference period”, there is thus a need for defining reference period since the majority of structural load vary with time, in a certain manner. The probability that any specified load intensity will be exceeded in a fixed interval of time is a function of the length of the interval. Hence, structural reliability is dependent on the time of exposure of the

loading. It is also affected by material property change with time. Except for rare cases where loads and strengths are constants, then reference period can be ignored.

In reliability assessments, it is necessary to determine the probability that the structure will not attain any of the limit state likely to be violated within a reference period, in presence of uncertainties such as conditions of services, human errors, change in load and material properties etc; the probability of a survival of a structure can be mathematically represented thus;

$$R = 1 - P_f \quad (3.1)$$

where R is survival probability and P_f is probability of failure.

This shows that the probability of survival of a structure is the complement of the probability that the structure will fail.

3.3 CAUSES AND EFFECTS OF STRUCTURAL FAILURE

Steel structural members which have attained limit state failure in structural design will not be able to perform their intended purpose before the expiration of the intended life span. Failures occur in structural members when they encounter extreme load cases or a combination of loads causing load effects of sufficient magnitude for the structure to attain failure state.

For a safe design, the engineer should be able to predict the magnitude of this extreme load and the strength of load-deflection of each structural component from the information available at the design stage so that probabilistic models for the design can be synthesized.

Structural failure occurs due to uncertainties such as unpredictability of future loading conditions, inability to obtain and express the material properties accurately, the use of simplified assumptions in predicting the behavior of the structure due to the loading under

consideration, the limitation in the numerical methods used and human factors. The risk of unacceptable consequences can be limited to an acceptable level in terms of probability of failure, P_f . Factors affecting failure in steel structures are:

- Incorrect selection of materials.
- Errors in design calculations.
- Poor construction methods and inadequate quality control and supervision.
- External physical and mechanical factors including alterations made to the structures.

In the event of failure, one consequence such as loss of human lives or injury, structural and environmental damage could be represented in economic terms.

3.4 UNCERTAINTIES IN ENGINEERING DESIGN

All design parameters have a certain degree of uncertainty attached to them. This is shown by the fact that designs using factors of safety fall short of satisfaction in all circumstances, a more appropriate factor of safety can only be determined if all these uncertainties can be measured and their interaction with the structure considered.

Three basic types of uncertainties can hereby be considered (Madsen et al., 1986):

(i) Physical Uncertainty

Physical parameters of design such as load, material strength are actual determinants of failure of a structure, and their actual variability is a necessary consideration.

(ii) Statistical Uncertainty

In modeling, the selection of an appropriate distribution types is necessary. Common probability distributions have between 1 and 4 parameters which immediately place lower bound and sample size required. But in practice, very large sample data are required to

establish reliable estimate of the numerical values of parameters. Therefore some level of uncertainty is created.

(iii) Model Uncertainty

Mathematical methods relate the response of structural elements to the inputs design parameters (e.g. material strength, physical dimension etc.) Models are generally deterministic in form and can also be probabilistic.

Accurate predictions of the magnitude of this response are not certain even when exact values of design parameters are known. This uncertainty is termed as model uncertainty. This results because of simplified assumptions, imprecise boundary conditions and unpredictable effects of the interaction of other design variables. The reliability is calculated from observable failure frequency limit of a design as distinguished from theoretical reliability, which are estimations from reliability models. The adjusted probability of failure is the difference between theoretical reliability and overall reliability. It is associated with gross errors (blunders such as result miscalculation or omissions by engineering designers etc.)(Madsen et al., 1986). The primary errors are systematic and random errors. Systematic errors are related to ignorance of real physical phenomena and random errors involve statistical variations.

3.5 METHODS OF RELIABILITY ANALYSIS

All methods are approximations and the problems become more difficult as the number of random variables and the complexity of the limit state function increase and when statistical dependence between random variables is present.

Madsen et al.(1986) identified methods to measure the reliability of a structure into four groups as follows:

- I. Level I methods: The uncertain parameters are modelled by one characteristic value, as for example in codes based on the partial safety factor concept.
- II. Level II methods: The uncertain parameters are modelled by the mean values and the standard deviations, and by the correlation coefficients between the stochastic variables. The stochastic variables are implicitly assumed to be normally distributed. The reliability index method is an example of a level II method.
- III. Level III methods: The uncertain quantities are modeled by their joint distribution functions. The probability of failure is estimated as a measure of the reliability.
- IV. Level IV methods: In these methods the consequences (cost) of failure are also taken into account and the risk (consequence multiplied by the probability of failure) is used as a measure of the reliability. In this way different designs can be compared on an economic basis taking into account uncertainty, costs and benefits.

Level I methods can be calibrated using level II methods, level II methods can be calibrated using level III methods, etc (Sorensen, 2003).

In First Order Reliability Methods the limit state function (failure function) is linearized and the reliability is estimated using level II or III methods.

Level II is known as Second Moment, First Order Reliability Method (FORM). The random variables are defined in terms of means and variances and are considered to be normally distributed. The measure of reliability is based on the reliability index. It involves use of certain iterative correlation procedure to obtain an approximation to the probability of failure of a structure or structural system. This generally requires an idealization of failure domain and it is often associated with a simplified representation of the joint probability distribution of a variable.

The necessity to have a method of reliability analysis which is computationally fast and efficient and which produces results with desired degree of accuracy prompted the use of this level 2 method.

Vrijing (2000) added that probabilistic techniques are further developed and refined. A lot of research is implemented, and in practice, the design is mainly made following semi-probabilistic or even fully deterministic methods.

3.6 RESISTANCE AND LOAD INTERACTION

In basic reliability problems, consideration is given to the effect of a load S and the resistance R offered by the structure. Both the load and resistance, S and R can be described by a known probability density function $F_s ()$ and $F_r ()$ respectively. S can be obtained from the applied loading through a structural analysis making sure that R and S are expressed in the same unit.

Considering only the safety of a structural element, it would be said that a structural element has failed if its resistance R , is less than the stress resultant S acting on it. The probability of failure P_f of the structural element can be expressed in any of the following ways

$$P_f = P(R - S < 0) \quad (3.2)$$

where R = strength (resistance) and S = loading in the structure. The failure in this case is defined in the region where $R-S$ is less than zero, or R is less than S , that is:

$$P_f = P(R - S \leq 0) \quad (3.3)$$

As an alternative approach to equation (3.2), the probability of failure can also be given as

$$P_f = P\left(\frac{R}{S} \leq 1\right) \quad (3.4)$$

where in this case, the failure is defined in the region where P_f is less than one, or R is less than S , which is

$$P_f \leq 1 \text{ or } R < S$$

It could also be expressed as

$$P_f = P(\ln R - \ln S \leq 1) \quad (3.5)$$

Or in general,

$$P_f = P[G(R, S) \leq 0] \quad (3.6)$$

Where $G(x)$ is the “limit state function” and the probability of failure is identical with probability of the limit state violation.

For any random variable X , the cumulative distribution function $F_X(x)$ is given by

$$F_X(x) = P(X \leq x) = \int_{-\infty}^x f_x(y) dy \quad (3.7)$$

Provided that $x \geq y$

It follows for the common, but special case where R and S are independent, the expression for the probability of failure is

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} F_R(x) f_s(x) dx \quad (3.8)$$

Expression (3.8) is known as the “convolution integral” and $F_R(x)$ is the probability that $R \leq x$, or the probability that the actual resistance R of the member is less than some value x . $f_s(x)$ represents probability that the load effect S acting in the member has a value below x and $x + \Delta x$ in the limit as $\Delta x \rightarrow 0$.

Considering all possible values of x , total failure probability is obtained as follows:

$$P_f = \int_{-\infty}^{\infty} [1 - F_s(x)] \bar{F}_R(x) dx \quad (3.9)$$

i.e., sum of all the cases of resistance for which the load exceeds the resistance.

3.7 LIMIT STATE FUNCTION

The performance function $g(x)$ is sometimes called the limit state function. It relates the random variables for the limit state of interest. The limit state function, gives a discretised assessment of the state of a structural element as being either failed or safe. It is obtained from traditional deterministic analysis, but uncertain input parameters are identified and quantified. Interpretation of what is considered to be an acceptable failure probability is made with consideration of the consequences of failure, which is predetermined.

The limit state “ $g(x) = R - S$ ” is a function of material properties, loads and dimensions. The state of the performance function $g(x)$ of a structure or its components at a given limit state is usually modeled in terms of finite uncertain basic random variables $x = (x_1, x_2, \dots, x_n)$ with joint distribution function given as:

$$F_x(x) = P \left\{ \prod_{i=1}^n (x_i \leq x_i) \right\} \quad (3.10)$$

and

$$P_f = P \{ g(x) \leq 0 \} = \int_{g(x) \leq 0} f_x(x) dx \quad (3.11)$$

where $f_x(x)$ is the joint probability distribution function of x . The region of integration of the function $g(x)$ is as follows;

$g(x) > 0$: represents safety,

$g(x) = 0$: represents attainment of the limit state

$g(x) < 0$: represents failure.

The probability of failure is given by $P(g(x)<0)$ and therefore the reliability index β can be related to probability of failure by the following equation :

$$P_f = 1 - \Phi(\beta) \quad (3.12)$$

3.8 COMPUTATION OF RELIABILITY INDEX

The basic approach to develop a structural reliability-based strength standard is to determine the relative reliability of the design. In order to do this, reliability assessment of existing structural components is needed to estimate a representative value of the reliability index, β . The First Order Reliability Method is very well suited to perform such a reliability assessment. For cases of a linear safety function, $G(x)$, and normal basic variable, X_i , the reliability index, β , can be obtained using the following computational steps, as outlined by Ayyub and McCuen (1997), using the FORM method:

(i) Assuming a design point x^{i*} and obtained $x^{i'*$ using the following equation;

$$x^{i*} = \frac{x^{i'} - \mu_{x_i}}{\sigma_{x_i}} \quad (3.21)$$

Where $x^{i*} = -\alpha_i^* \beta$, μ_{x_i} = mean value of the basic random variable and σ_{x_i} = standard deviation of the basic random variable. The mean values of the basic random variables can be used as initial values for the design points. The notation x^* and x'^* are used respectively for the design point in the regular coordinates and in the reduced coordinates.

(ii) Evaluate the equivalent normal distributions for the non-normal basic variables at the design point using the following equation:

$$\mu_x^N = x^* - \phi^{-1}(F_x(x))\sigma_x^N \quad (3.22)$$

and in general, limit state function is given by

$$G(X) = G(x_1^*, x_2^*, \dots, x_n^*) \quad (3.23)$$

where μ_x^N = mean of the equivalent distribution, σ_x^N = Standard deviation of the equivalent normal distribution. $F_x(x^*)$ is the original Cumulative Distribution Function (CDF) of X_i evaluated at the design point, $f_x(x)$ = original Probability Density Function (PDF) of X_i evaluated at the design point, $\Phi(\cdot)$ = CDF the standard normal deviation, and $\phi(\cdot)$ = PDF of the standard normal distribution.

(iii) Compute the directional cosines ($\alpha_i^*, i = 1, 2, \dots, n$) Using the following equations:

$$\alpha_i^* = \frac{\left(\frac{\partial g}{\partial x_i}\right)}{\sqrt{\sum_{i=1}^n \left(\frac{\partial g}{\partial x_i}\right)^2}} ; \text{ for } i=1, 2, \dots, n \quad (3.24)$$

(iv) where

$$\left(\frac{\partial g}{\partial x_i}\right)_* = \left(\frac{\partial g}{\partial x_i}\right) \alpha_i^N \quad (3.25)$$

(v) with α_i^* , $\mu_{x_i}^N$ and $\sigma_{x_i}^N$ now known, the following equation can be solved for the root of β

$$G \left[\mu_{x_1}^N - \alpha_{x_1}^* \sigma_{x_1}^N \beta, \dots, \mu_{x_n}^N - \alpha_{x_n}^* \sigma_{x_n}^N \beta \right] = 0 \quad (3.26)$$

(vi) using the β obtained from step 4, a new design point can be obtained from the following equation:

$$\alpha_1^* = \mu_{x_1}^N - \alpha_{x_1}^* \sigma_{x_1}^N \beta \quad (3.27)$$

Repeat step 1 to 5 until a convergence of β is achieved. The reliability index is the shortest distance to the failure surface from the origin in reduced coordinates.

3.9 FIRST ORDER RELIABILITY METHOD

The FORM is a convenient tool to assess the reliability of structural elements. It also provides a means for calculating the partial safety factors. FORM uses a combination of analytical and approximation methods and comprises three stages. Firstly, independent of whether each parameter has been defined as Normal, Log-Normal or Gumbell distribution, all variables are first transformed into equivalent normal space with zero mean and unit variance. The original limitstate surface is then mapped onto the new limit state surface. Secondly, the shortest distance between the original and the limit state surface, termed the reliability index β is evaluated; this is termed the design point, or point of maximum likelihood, and gives the most likely combination of basic variables to cause failure. Finally, the failure probability associated with this point is then calculated. FORM can be easily extended to non-linear limit states and has a reasonable balance between ease of use and accuracy.

First Order Reliability Method (FORM) and Monte-Carlo Simulation (MCS) are the most common basic techniques and are applicable to all probabilistic problems. Of these, FORM is usually the preferred method, as it does not depend on the number of simulations carried out. For

complex limit states, FORM may not converge and an answer not obtained. In this case MCS is used but a large number of simulationruns must be made when failure probability is low, thus requiring extended computing time (Webster and Bannister, 2001).

3.10 THE ABAQUS

ABAQUS is a suite of powerful engineering simulation programs, based on the finite element method that can solve problems ranging from relatively simple linear analyses to the most challenging nonlinear simulations. ABAQUS contains an extensive library of elements that can model virtually any geometry. It has an equally extensive list of material models that can simulate the behavior of most typical engineering materials including metals, rubber, polymers, composites, reinforced concrete, crushable and resilient foams, and geotechnical materials such as soils and rock.

ABAQUS offers a wide range of capabilities for simulation of linear and nonlinear applications. Problems with multiple components are modeled by associating the geometry defining each component with the appropriate material models and specifying component interactions. The ABAQUS Version 6.10 is employed for the analysis in this work. It will be used to study the variation of load with the nodal displacement. With this result we can find out how durable the proposed horizontally curved Aluminium alloy bridge deck is under applied stress representing the wheel stresses and centrifugal stresses.

CHAPTER FOUR

4 FINITE ELEMENT ANALYSIS, DESIGN AND STOCHASTIC OPTIMIZATION

In this chapter a finite element analysis to study the behavior of the deck under varying loads is conducted for the system II i.e. the bending of the deck between girders. The system II stress is chosen because it will give us a good idea about the local deck behavior.

4.1.1 FINITE ELEMENT ANALYSIS WITH USE OF ABAQUS

The deck was analyzed using Finite Element software ABAQUS. The Figure 4-1 illustrates the part; the case study considered - a horizontally curved aluminium alloy deck - Alumadeck™ – radius = 100m, Subtended angle = 34.4°.

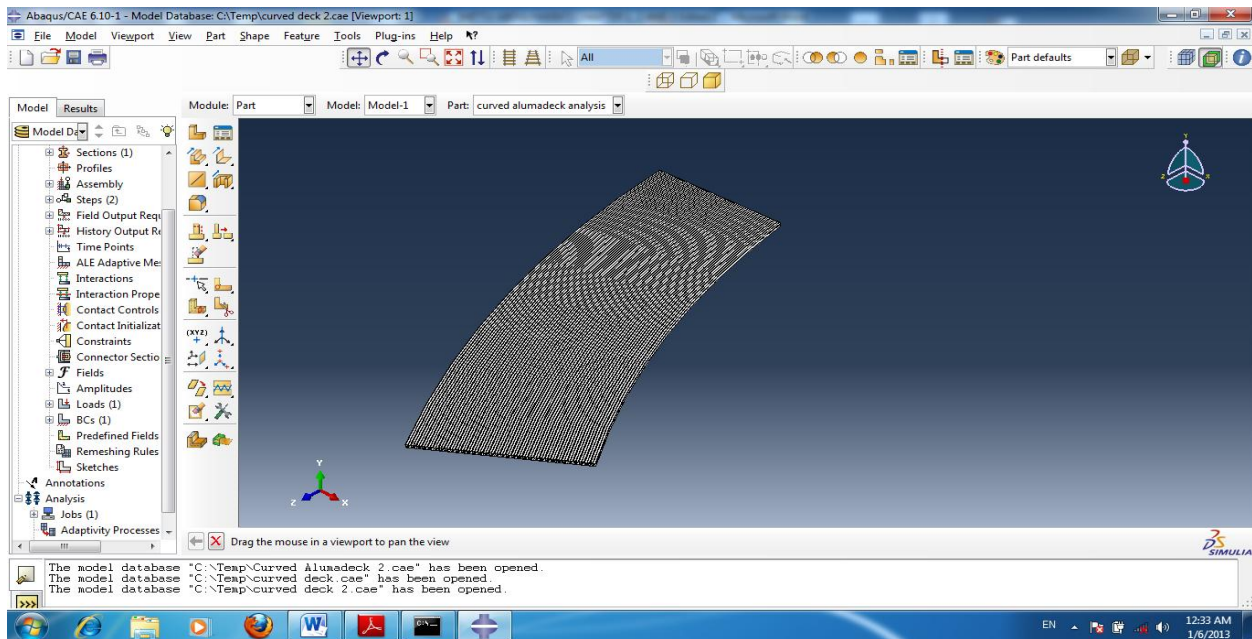


Figure 4-1: The horizontally curved bridge deck as drawn in ABAQUS

The Figure 4-2 shows the Alumadeck section used (i.e. vertical and inclined stiffeners). The solid element feature in abaqus was used.

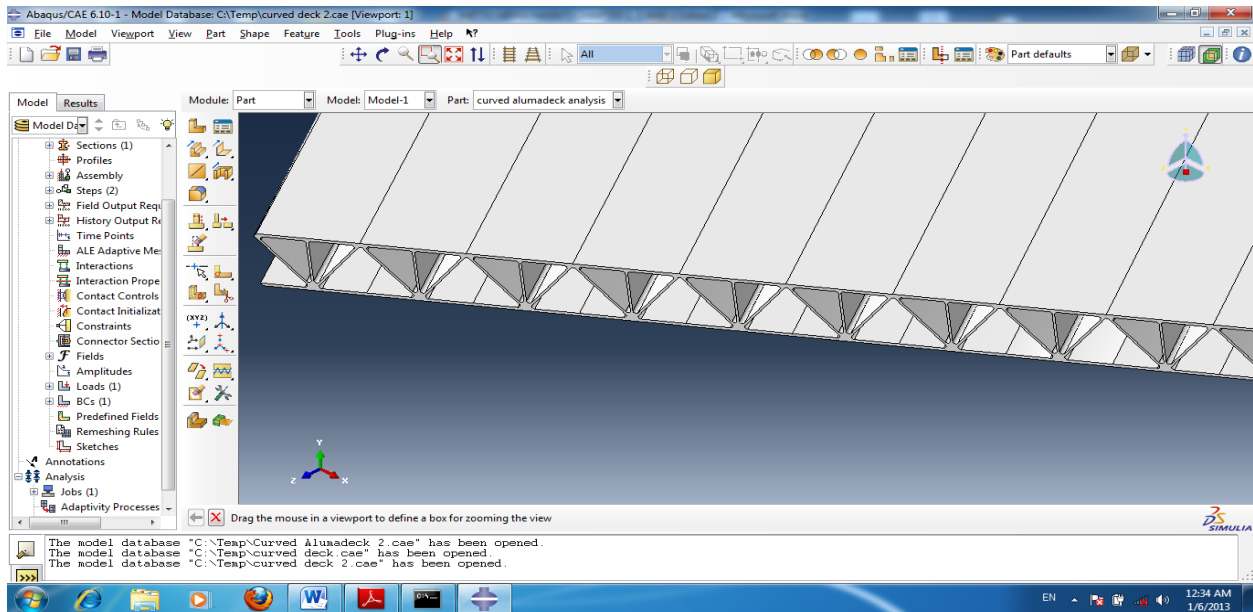


Figure 4-2: The ABAQUS showing the vertical and inclined stiffeners of the horizontally curved Aluminium alloy deck

The material properties of the Alumadeck were assigned in the software. The Young's Modulus assigned was $68.9GPa$ and the Poisson's ratio was 0.33. The density and the yield stress were also assigned.

The loadings considered were applied as uniformly distributed over the deck and the centrifugal load was applied as shown in Figure 4-3.

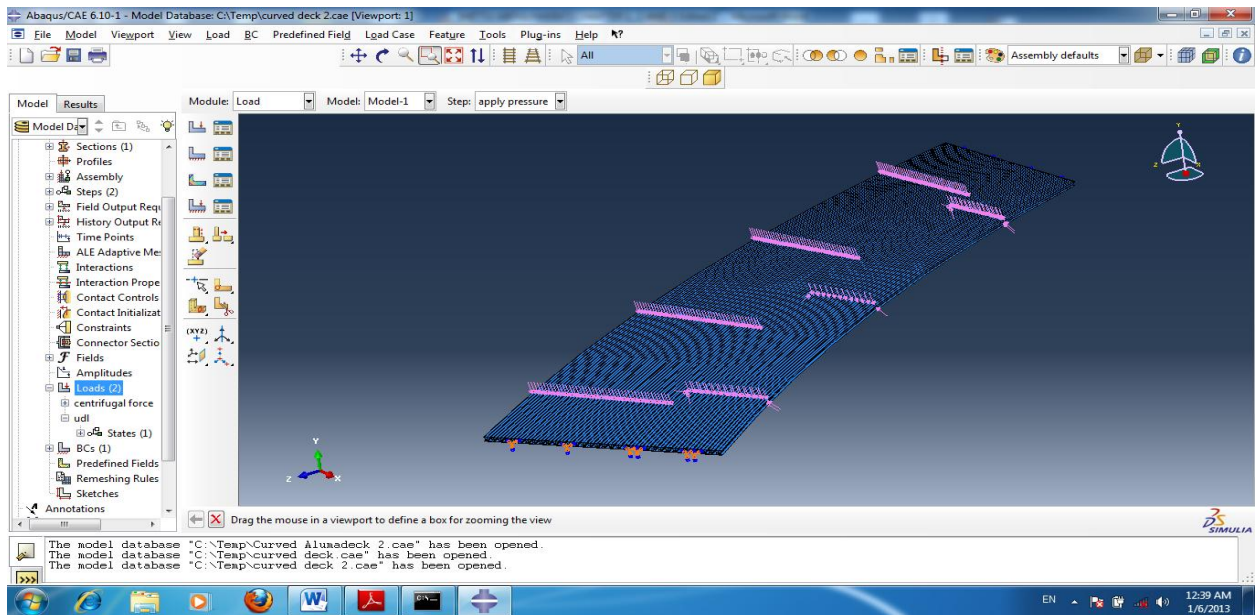


Figure 4-3: Loadings on the Alumadeck

The deck was meshed into nodes and elements as shown in the Figure 4-4.

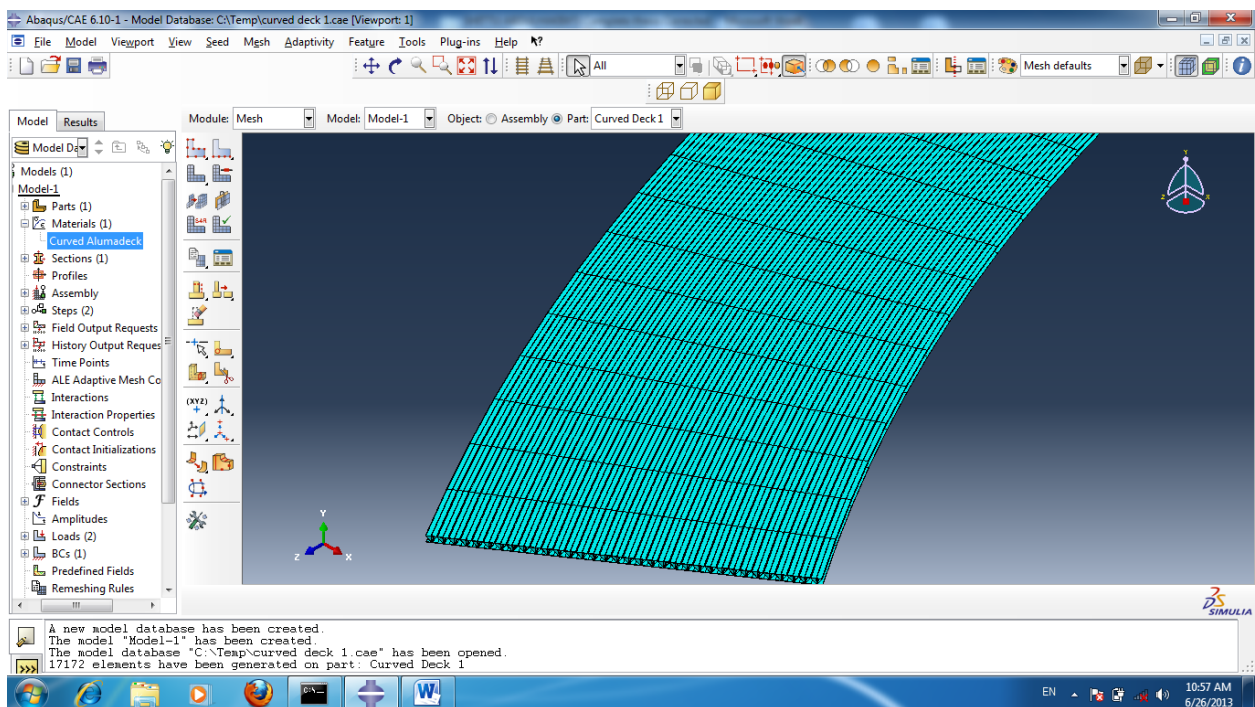


Figure 4-4: Meshing of the deck into nodes and elements

The pinned support condition was considered as shown in Figure 4-5.

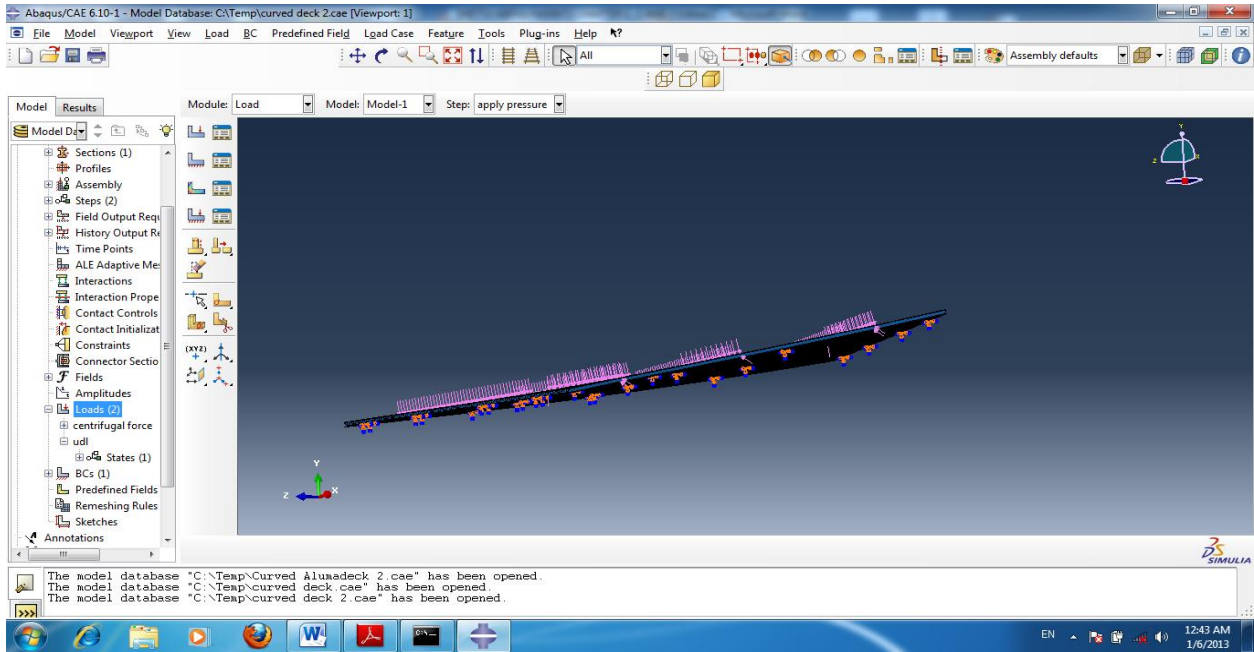


Figure 4-5: Boundary condition

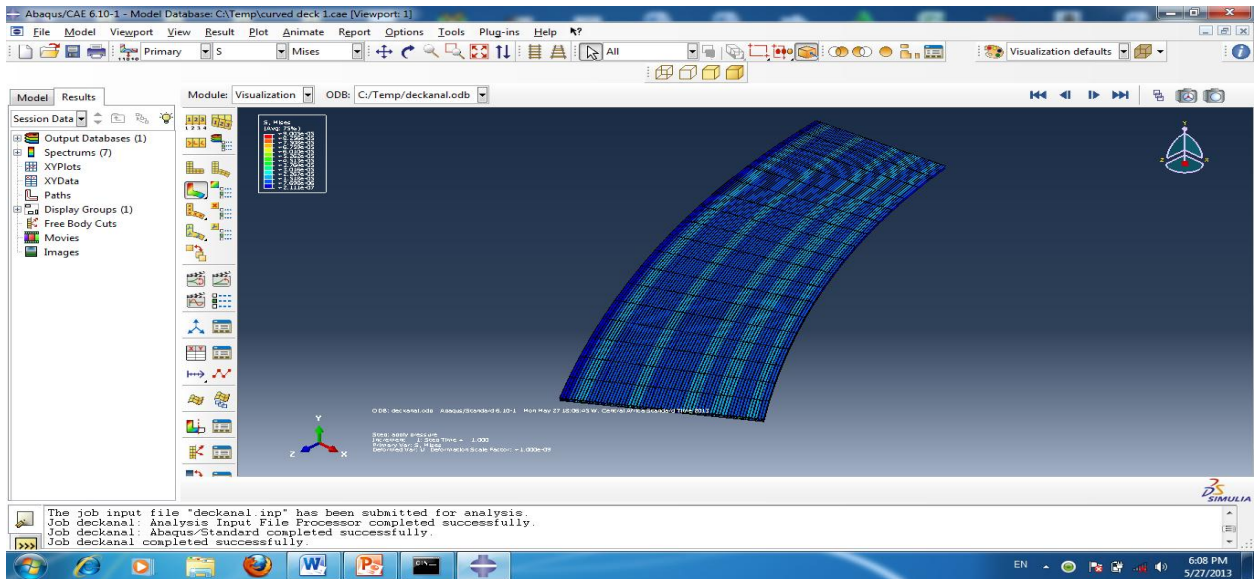


Figure 4-6: Analysis result showing the contours of Von mises stress after performing a job on

ABAQUS.

The maximum displacement, U_{max} (mm) and maximum Von Mises stresses kN/m^2 is plotted against applied centrifugal stresses kN/m^2 at various applied stresses kN/m^2 for stiffener thicknesses of 5mm, 7mm and 9mm as shown in Figures 4-6 to 4-12 below.

For thickness of stiffener = 5mm;

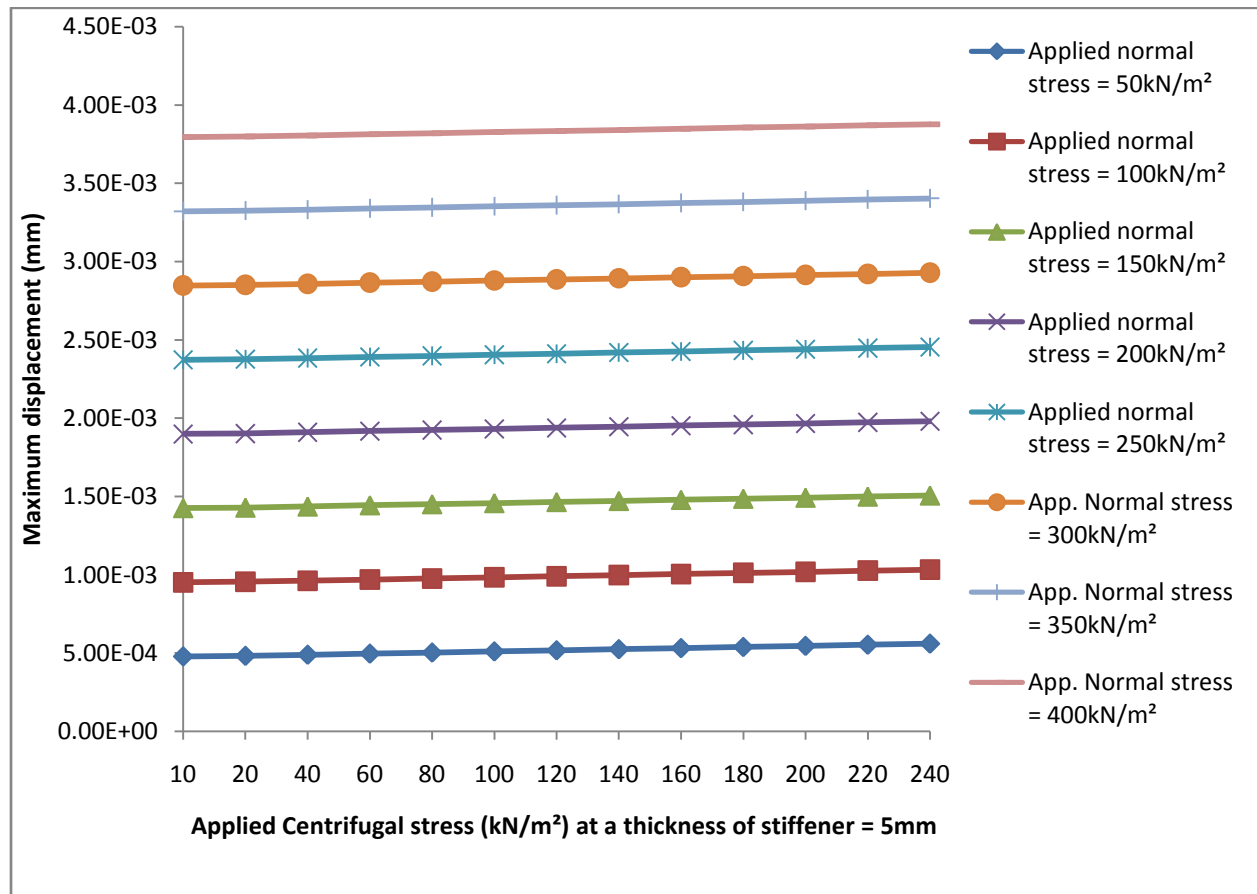


Figure 4-7: Variation of maximum displacement (mm) against applied centrifugal stress (kN/m^2)

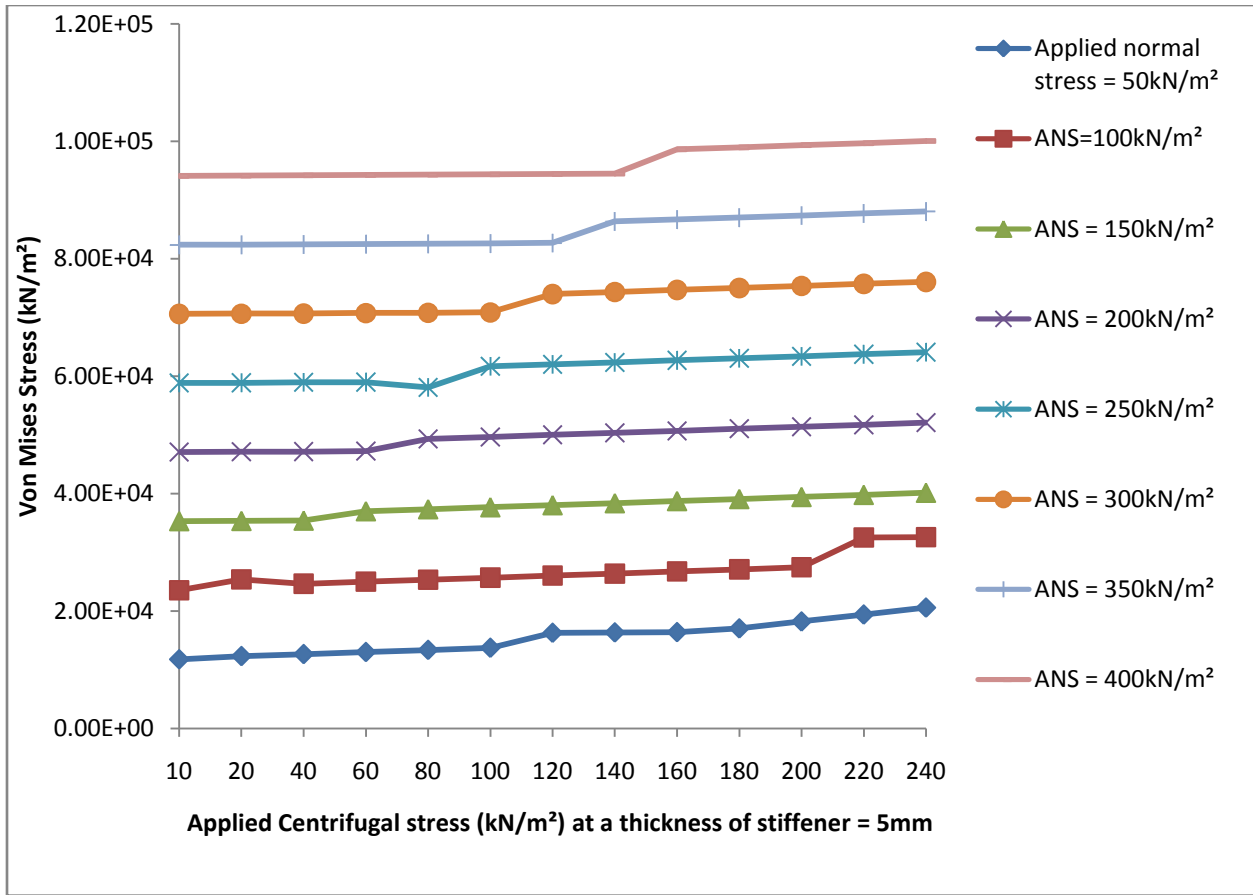


Figure 4-8: Variation of maximum Von Mises stress (kN/m^2) against applied centrifugal stress (kN/m^2)

For thickness of stiffener = 7mm;

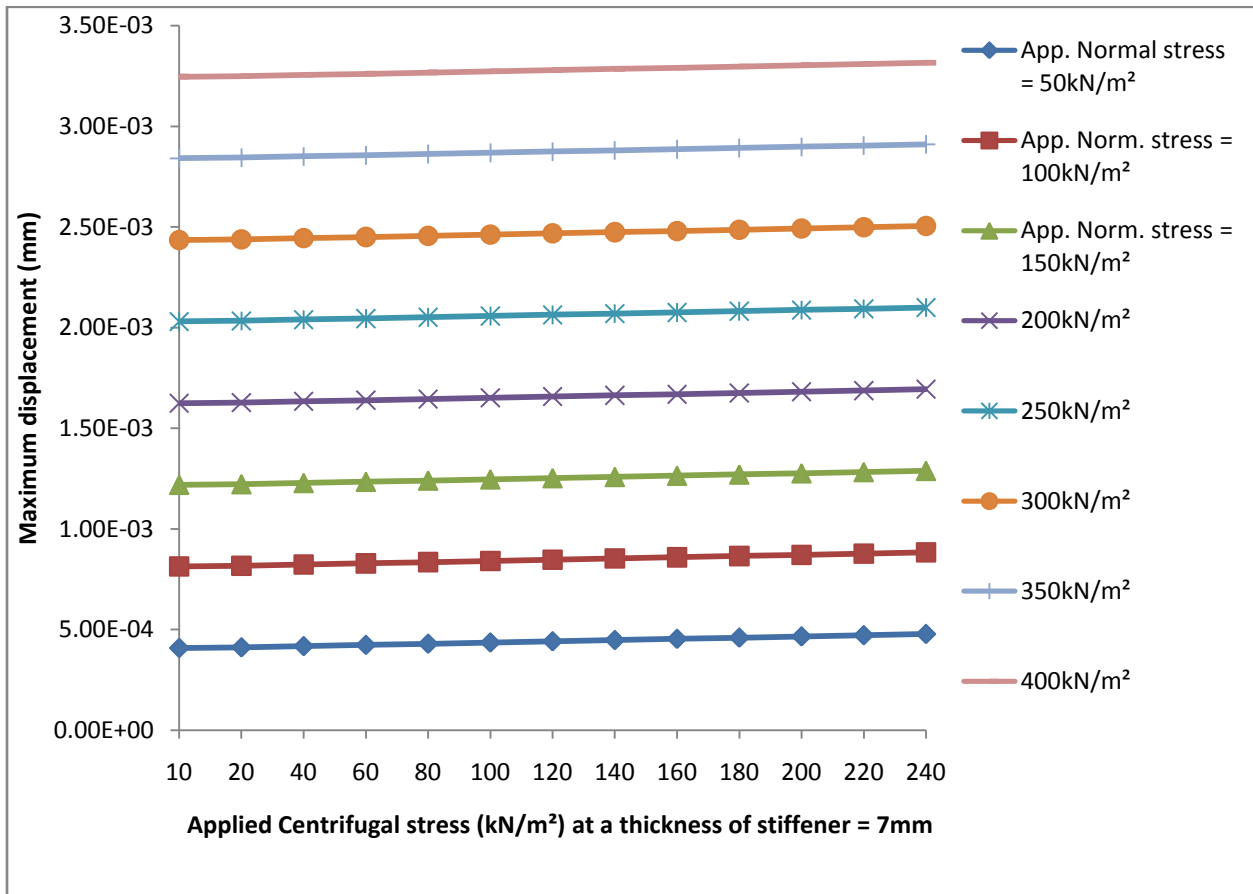


Figure 4-9: Variation of maximum displacement (mm) against applied centrifugal stress (kN/m²)

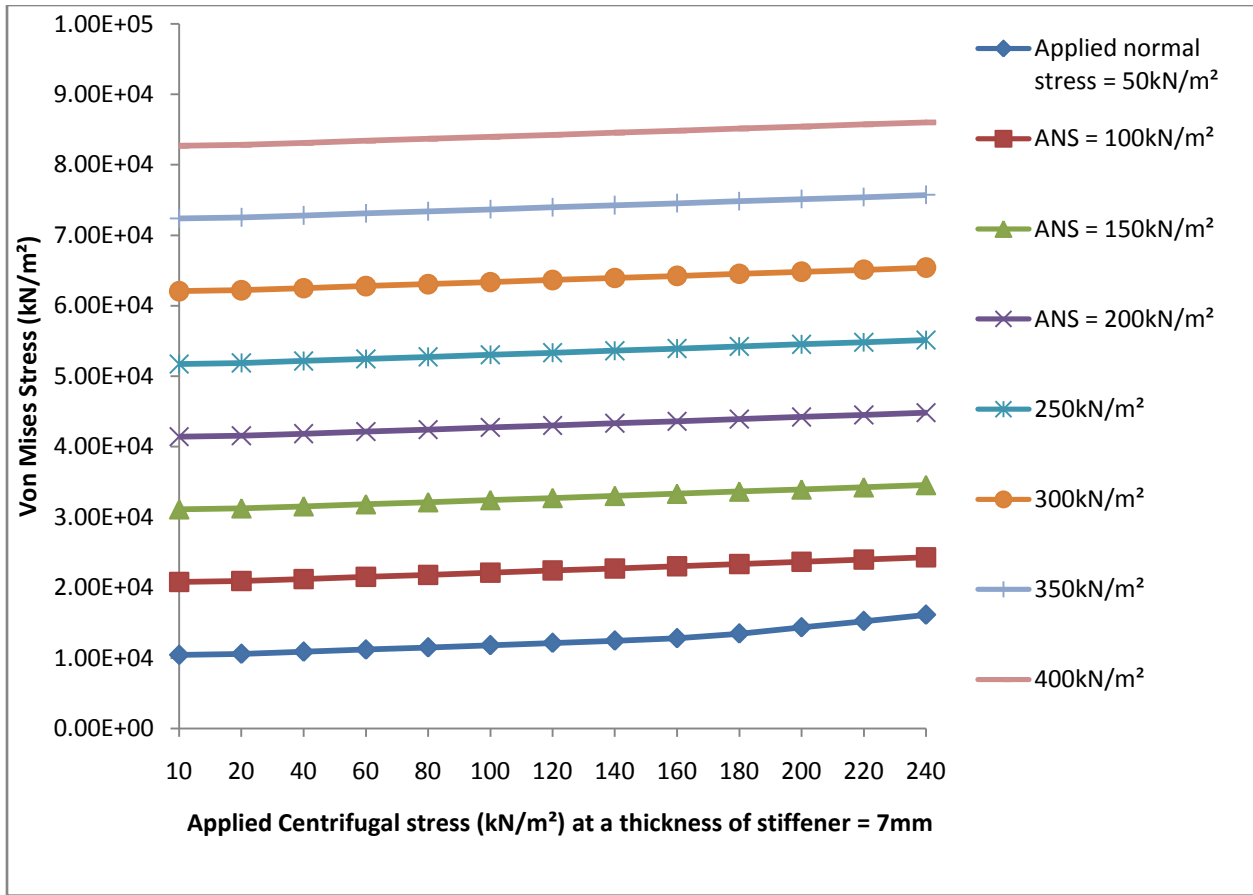


Figure 4-10: Variation of maximum Von Mises stress (kN/m^2) against applied centrifugal stress (kN/m^2)

For the 9mm;

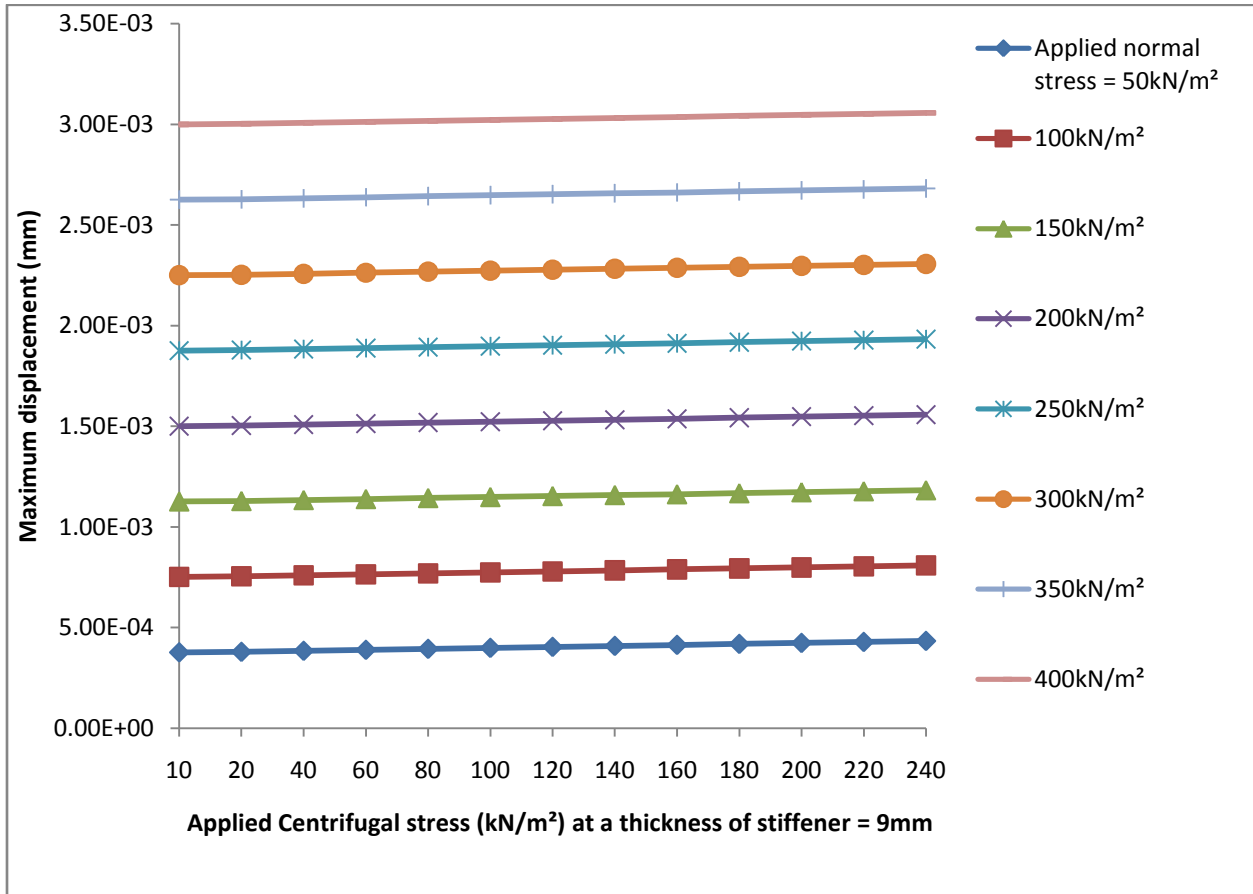


Figure 4-11: Variation of maximum displacement (mm) against applied centrifugal stress (kN/m^2)

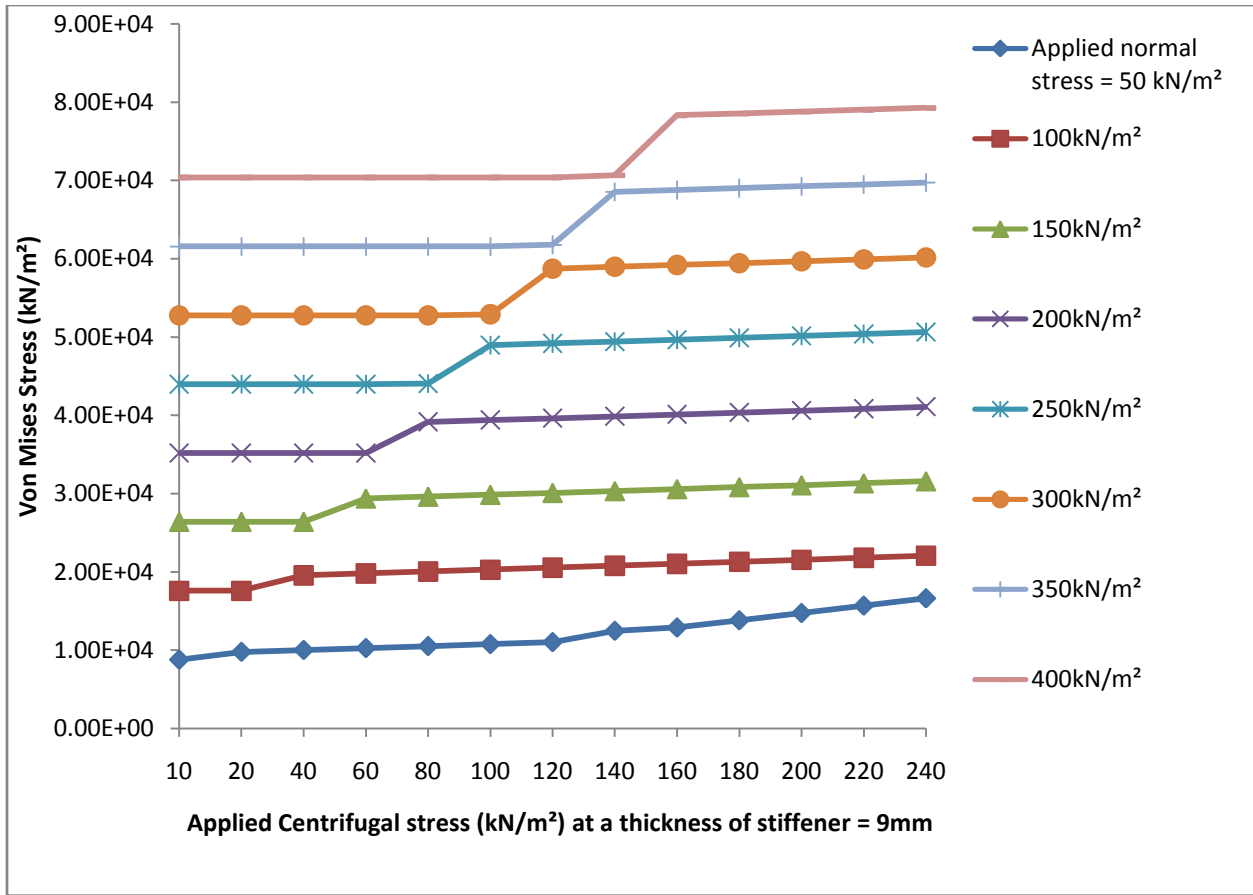


Figure 4-12: Variation of maximum Von Mises stress (kN/m^2) against applied centrifugal stress (kN/m^2)

The result of the finite element analysis shows that the maximum displacement of the Alumadeck increases linearly with increase in the applied centrifugal stresses. The maximum Von Mises stresses are acceptable since they are less than the yield stress of the deck and this is an indication of the durability of the Alumadeck. This result also shows that the thickness of stiffeners should be a minimum of $7mm$ to have a safe structure since some of the maximum displacements for $5mm$ were not satisfactory. The maximum displacements obtained for the $5mm$, $7mm$ and $9mm$ stiffener thicknesses are $0.004mm$, $0.0033mm$ and $0.003mm$ respectively. The allowable displacement according to the AASHTO (2007) is a maximum of

0.0038mm for all plates. See Appendix A for the Table of Results of the Finite Element Analysis.

A reliability of the aluminium alloy deck system is evaluated for system I i.e. the longitudinal bending of the composite alumadeck – steel girder section. The composite is designed in accordance to the AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges with Design Examples for I-Girder and Box-Girder Bridges (AASHTO, 2003). The girder is assumed to be simply supported. The horizontally curved steel I-girder bridge considered is a single span with four girders in the cross section. Figure 4.30 shows configuration of the girders stiffened and the cross frames.

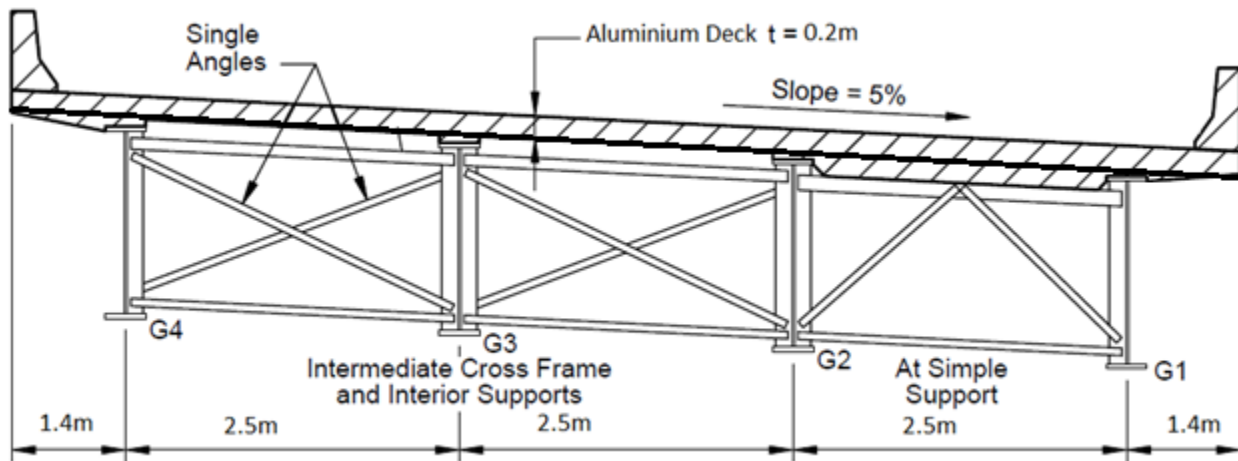


Figure 4-13: I girder bridge cross section showing cross frames. Source: Kulicki et al. (2005)

4.1.2 RADIAL MOMENT AND SHEAR FORCE ACTING ON DECK

Using the curved deck with reference to Figure 2.4 equations (2.8), (2.13) and (2.14) are solved simultaneously to obtain the radial moment and shear force by the application of boundary conditions. It is assumed that the panel span considered is the spacing between two girders.

- $\eta = 0$, at $r = r_0 = 93.6m$ thus $X = \frac{r_0}{r} = 0.936$
- $\eta = 0$, at $r = r_i = 91.1m$ thus $X = \frac{r_i}{r} = 0.911$
- $M_r = 0$, at $r = r_i = 92.35m$, thus $X = \frac{r_i}{r} = 0.9235$
- $M_r = 0$, at $r = r_0 = 89.7m$, thus $X = \frac{r_0}{r} = 0.897$

The unknowns A, B, C and D can be determined after which M_r and R_r are obtained for the HL93 loading condition.

4.1.3 BRIDGE DECK DESIGN PARAMETERS

Heins (1975) stated that the minimal radius to be used for the design of a horizontally curved bridge will be $100m$. The following parametric values were assumed for the curved deck with reference to Figure 2.4 and used in the design.

| | |
|--|---------------------------|
| 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 ultimate limit state | 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 |

4.1.4 DESIGN DATA FOR I-GIRDER G4

Other data relevant to the study include the following:

| | |
|---|---|
| Structural steel | ASTM A709 grade 50W (Yield strength, $F_y = 350MPa$) |
| Live load | HL93 (AASHTO 2007) |
| 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 |

The composite (i.e. the deck and I-girder) is designed against the strength limit state. Also the outermost girder (G4) which is the most critical is analyzed. The two void aluminium extrusions are assumed to behave in a similar way as reinforced concrete as demonstrated by Matteo, et al (1997) in experimental evaluation of an aluminium deck bridge. The Eurocode 9 (1999) stipulates that the tensile yield strengths should be multiplied by 0.8.

The dead load according to AASHTO (2010), is to include weight of all components of the structure, appurtenances and utility, earth cover, future overlay and planned widening. The dead load is sub-divided into three categories:

- Dead load of 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

$$DC1 = 26.9 \times 0.2 = 5.38 \text{ kN/m}$$

For the steel girder

$$DC1 = 78.5 \times 0.021 = 1.65 \text{ kN/m}$$

- Dead load wearing course (DW): This accounts for the future wearing course.

$$DW = 6.73 \times 0.0095 = 0.064 \text{ kN/m}$$

The AASHTO vehicular live load is designated as HL-93 loading and is a combination of design truck or a tandem plus the design lane load. The design truck specified is composed of a 36kN lead axle spaced 4.287m from the closer of the two 144kN rear axle, which has variable spacing from 4.287 to 9.144m. The transverse spacing is 1.83m. The design truck occupies 3.045mlane

width, is positioned within the design lane to produce maximum force effect, and must not be closer than 0.61m from the face of the edge of the design lane.

4.1.5 DESIGN MOMENT

In this study the position which yields the maximum stress in the deck for both single lane and two lane loaded bridges is adopted (HL 93 loading). See Figure 2.3.

For a straight girder, the maximum moment can be expressed as $M = \frac{wl^2}{8}$ and $S = \frac{Ml}{l}$

Where M = Maximum moment in the girder, w = uniformly distributed load along the length of the girder, l = the girder span length.

For the horizontally curved aluminium alloy deck and Steel girder;

$M_{radial\ DC1} = 140kNm$ For Aluminium

$M_{radial\ DC1} = KM_{DC1} = 1.0 \times 1.65 \times \frac{20^2}{8} = 82.5kNm$ For the Steel girder (See equation 2.15)

For the concrete parapet

$$M_{radial\ DC2} = KM_{DC2} = 1.0 \times 6.32 \times 0.5 \times \frac{20^2}{8} = 158kNm$$

For the polymer surface

$$M_{radial\ DW} = \frac{KM_{DW}}{4} = \frac{1.0}{4} \times 0.064 \times 2.18 \times \frac{20^2}{8} = 1.744\ kNm$$

Live Load

The live load component for the reliability analysis was determined using design truck occurring simultaneously with a uniformly distributed lane load. The maximum moment occurs when the first rear axle is $0.055m$ from the center of the girder span as the truck moves eastwards on the bridge. Live load is due to three lanes of LRFD HL-93 plus appropriate centrifugal force effects. The dynamic load allowance has been applied to live load. The overturning effect of the centrifugal force has been considered by increasing the exterior wheel load and decreasing the interior wheel load in each lane. The live load was also multiplied by 0.85 in the analysis to account for the probability of multiple presence.

$$R_1 + R_2 = 324kN$$

$$-20R_1 + 144(12.57) + 144(9.945) + 36 \times 6.825 = 0$$

$$R_1 = \frac{1}{20}(3487.86) = 174.39kN$$

$$R_2 = 149.607kN$$

$$M_{max} = 10R_1 - 144(3.061) = 1,302.57kNm$$

$$M_{HL93 LL + IM} = 1472.56kNm$$

4.2 BRIDGE COMPONENT DESIGN AND DERIVATION OF LIMIT STATE FUNCTIONS

Presented below is the detailed calculation for girder flexural design based on the AASHTO (2003). The design is carried out for the critical section for girder G4 for the strength limit state under strength I load combination on the completed structure.

4.2.1 GIRDER STRESS CHECK CRITICAL SECTION ON G4 – SECTION PROPORTIONING

For a web without longitudinal stiffeners, the web is proportioned such that:

$$\frac{D}{t_w} \leq 150$$

$$\frac{2.358}{0.016} = 147.38 < 150 \text{ OK}$$

Compression and tension flange are proportioned such that

$$\frac{b_f}{2t_f} \leq 12$$

$$\frac{50}{2(2.1)} = 11.9 \text{ Both Flanges OK}$$

$$b_f \geq \frac{D}{6}$$

$$\frac{2.358}{6} = 0.393 \text{ m} = 39.3 \text{ cm} \text{ Both Flanges OK}$$

$$t_f \geq 1.1t_w$$

$$1.1(1.6) = 1.76 \text{ cm} \text{ Both Flanges OK}$$

$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$$

$$I_{yc} = I_{yt} = 2.1 \times \frac{(50)^3}{12} = 21875 \text{ cm}^4$$

$$\frac{I_{yc}}{I_{yt}} = 1 \quad \text{OK}$$

Therefore, all section proportions for this location are satisfied.

4.2.2 GIRDER STRESS CHECK CRITICAL SECTION (MID SPAN) ON G4 TRANSVERSELY STIFFENED WEB – STRENGTH – TOP FLANGE

The factored vertical bending stress in the top flange due to dead and live load, f_{bu} is given as (Kulicki et al, 2005):

$$f_{bu,c} = \left[\frac{1.25M_{DC1}C_{nc}}{I_{nc}} + \frac{(1.25M_{DC2}+1.5M_{DW})C_{3n}}{I_{3n}} + \frac{1.75M_{LL}C_n}{I_n} \right] 12\eta \quad (4.1)$$

$$\eta = 1$$

Where C_{3n} is the centroid considering long term composite properties, C_n is the centroid considering short term, C_{nc} is the centroid considering non-composite, I_{nc} is the moment of inertia for the non-composite section.

For long term composite section properties (3n), Effective width of slab,

$$b_{s_{3n}} = \frac{b_{slab}}{3n} \quad (4.2)$$

While for the short term (n),

$$b_{s_n} = \frac{b_{slab}}{n} \quad (4.3)$$

The neutral axis for the non-composite action, C_{nc}

$$D_c = \frac{A_{fc}\left(\frac{t_{fc}}{2}\right) + A_{ft}\left(D - \frac{t_{ft}}{2}\right)A_{web}\left(\frac{D}{2} + t_{fc}\right)}{A_{girder}} - t_{fc} \quad (4.4)$$

The neutral axis for the composite action, $C_{n/3n}$

$$NA = \frac{A_{fc}\left(t_s+t_{haunch}-\frac{t_{fc}}{2}\right)+A_{ft}\left(D+t_s+t_{haunch}+\frac{t_{ft}}{2}\right)+A_{web}\left(\frac{D}{2}+t_s+t_{haunch}\right)+A_s\frac{t_s}{2}}{A_{Total}} \quad (4.5)$$

A_{fc} is the Area of compression flange, A_{ft} is the area of the tension flange, t_s is the thickness of 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 web, t_w is the thickness of web and b_{fc} is the width of the compression flange.

The neutral axis is measured from the top of the deck.

The moment of inertia considering non-composite action,

$$I_{nc} = \frac{b_{fc}t_{fc}^3}{12} + \left[A_{fc} \left(D_c + t_{fc} - \frac{t_{fc}}{2} \right)^2 \right] + \frac{b_{ft}t_{ft}^3}{12} + \left[A_{ft} \left(d - (D_c + t_{fc}) - \frac{t_{ft}}{2} \right)^2 \right] + \frac{t_w D^3}{12} + A_w \left(D_c + t_{fc} - t_{fc} - \frac{D}{2} \right)^2 \quad (4.6)$$

The moment of inertia considering composite action, (Short/ Long term)

$$I_{n/3n} = \frac{b_{fc}t_{fc}^3}{12} + \left[A_{fc} \left(NA - t_s - t_{haunch} + \frac{t_{fc}}{2} \right)^2 \right] + \frac{b_{ft}t_{ft}^3}{12} + \left[A_{ft} \left(t_s + t_{haunch} + D + t_{ft} - NA - \frac{t_{ft}}{2} \right)^2 \right] + \frac{t_w D^3}{12} + A_w \left(NA - t_s - t_{haunch} - \frac{D}{2} \right)^2 + \frac{b_s t_s^3}{12} + A_s \left(NA - \frac{t_s}{2} \right)^2 \quad (4.7)$$

The compression flange must satisfy the following relation:

$$f_{bu} \leq \phi_f F_{nc} \quad (4.8)$$

ϕ_f is the resistance factor,

where:

$$F_{nc} = R_b R_h F_{yc} \quad (4.9)$$

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} is the yield strength for the compression flange.

The limit state function for the top flange of a transversely stiffened web (Strength limit state) can be expressed as:

$$G(x) = \phi_f F_{yc} - f_{bu} = \phi_f F_{yc} - \left[\frac{1.25 M_{DC1} C_{nc}}{I_{nc}} + \frac{(1.25 M_{DC2} + 1.5 M_{DW}) C_{3n}}{I_{3n}} + \frac{1.75 M_{LL} C_n}{I_n} \right] 12\eta \quad (4.10)$$

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

$$D_p \leq 0.42 D_t \quad (4.11)$$

Where D_t is the total depth of the composite section,

$$D_t = D + t_{haunch} + t_{slab} + t_{ft} \quad (4.12)$$

D_p is the distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment.

$$D_p = Y_{bar} + t_{slab} + t_{haunch} \quad (4.13)$$

$$P_s = f_s A_s = 205 \times 60090.202 \times 10^{-3} = 12318.5kN \quad \text{Force in the slab}$$

$$P_c = f_{yc} A_{fc} = 350 \times 500 \times 21 \times 10^{-3} = 3675kN \quad \text{Force in the compression flange}$$

$$P_t = f_{yt} A_{ft} = 3675kN \quad \text{Force in the tension flange}$$

$$P_w = f_{yw} A_w = 350 \times 16 \times 2358 \times 10^{-3} = 13204.8kN \quad \text{Force in the web}$$

If $P_t + P_w \geq P_c + P_s$, then the plastic neutral axis is in the web

$$16879.8 > 15993.5$$

Therefore the PNA is in the web and \bar{Y} or

$$Y_{bar} = \frac{D}{2} \left(\frac{P_t - P_c - P_s}{P_w} + 1 \right) \quad (4.14)$$

4.2.3 GIRDER STRESS CHECK STRENGTH LIMIT STATE - WEB

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

4.2.4 GIRDER STRESS CHECK CRITICAL SECTION (MID SPAN) ON G4 TRANSVERSELY STIFFENED WEB – STRENGTH – BOTTOM FLANGE

The tension flange must satisfy the following relation:

$$f_{bu} + \frac{1}{3}f_l \leq \phi_f F_{nt} \quad (4.15)$$

The factored bottom flange vertical bending stress due to dead and live load,

$$f_{bu,T} = \left[\frac{1.25M_{DC1}C_{nc}}{I_{nc}} + \frac{(1.25M_{DC2} + 1.5M_{DW})C_{3n}}{I_{3n}} + \frac{1.75M_{LL}C_n}{I_n} \right] 12\eta \quad (4.16)$$

The lateral bending stress at the cross-frame due to curvature, f_l is given as (Eq. C4.6.1.2.4b-1 AASHTO, 2007):

$$M_{lat_NC} = \lambda M_{DC1} \quad f_{l_NC} = \frac{M_{lat_NC}}{S_{bot_fl}} \quad (4.17)$$

$$M_{lat_C1} = \lambda M_{DC2} \quad f_{l_C1} = \frac{M_{lat_C1}}{S_{bot_fl}} \quad (4.18)$$

$$M_{lat_C2} = \lambda M_{DW} \quad f_{l_C2} = \frac{M_{lat_C2}}{S_{bot_fl}} \quad (4.19)$$

$$M_{lat_LL} = \lambda M_{LL} \quad f_{l_LL} = \frac{M_{lat_LL}}{S_{bot_fl}} \quad (4.20)$$

λ is as specified in AASHTO (Kulicki et al., 2005)

Total factored lateral bending stress

$$f_l = 1.25(f_{l_NC} + f_{l_C1}) + 1.5(f_{l_C2}) + 1.75(f_{l_LL}) \quad (4.21)$$

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

$$M_n = M_p \left(1.07 - \frac{0.7D_p}{D_t} \right). \quad (4.22)$$

Yield Moment associated with strength 1 limit state (AASHTO, 2007)

$$M_{D1t} = f_{D1t}S_t, M_{D1c} = f_{D1c}S_c, M_{D2} = M_{DC2} + M_{DW} \quad (4.23)$$

Tension Flange

$$M_{ADt} = \left(f_{yt} - \frac{M_{D1t}}{S_t} - \frac{M_{D2}}{S_{t3n}} \right) S_{tn} \quad (4.24)$$

$$M_{yt} = M_{D1t} + M_{D2} + M_{ADt} \quad (4.25)$$

Compression Flange

$$M_{ADc} = \left(f_{yc} - \frac{M_{D1c}}{S_c} - \frac{M_{D2}}{S_{c3n}} \right) S_{cn} \quad (4.26)$$

$$M_{yc} = M_{D1c} + M_{D2} + M_{ADc} \quad (4.27)$$

Section Modulus,

$$S_{xt} = \frac{M_{yt}}{f_{yt}} \quad (4.28)$$

Strength limit check is OK if

$$M_u + \frac{f_l S_{xt}}{3} \leq \phi_f M_n \quad (4.29)$$

$$M_u = 1.25M_{uDC2} + 1.5M_{uDw} + 1.75M_{uLL} \quad (4.30)$$

The limit state function for strength limit can be expressed as

$$G(x) = \phi_f M_n - M_u - \frac{f_l S_{xt}}{3} \quad (4.31)$$

or

$$G(x) = \phi_f f_{nt} - f_{bu} - \frac{f_l}{3} \quad (4.32)$$

4.2.5 GIRDER STRESS CHECK CRITICAL SECTION (MID SPAN) ON G4 TRANSVERSELY STIFFENED WEB – SHEAR STRENGTH – WEB

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

$$V_u \leq \phi_v V_n \quad (4.33)$$

where:

V_u =shear in the web at the section under consideration due to factored loads

V_n =nominal shear resistance

ϕ_v = resistance factor for shear taken as 1.0.

$$V_n = CV_p \quad (4.34)$$

$$V_p = 0.58F_y D t_w$$

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

$$\frac{D}{t_w} > 1.40 \sqrt{\frac{Ek}{F_{yw}}} \quad (4.35)$$

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

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

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

$$G(x) = \phi_v V_n - V_u \tag{4.36}$$

Table 4-1: Parameters of the stochastic model for strength Limit State

| Var. No. | Variab le | Meaning | Distributio n Type | Expected Value E_x | Coeffic ient of Variati on | Standard Deviation |
|----------|-----------|--|--------------------|----------------------|----------------------------|-----------------------|
| X1 | M_{DC1} | Moment due to Curved Aluminium deck and steel girder | Log Normal | 222.5kNm | 0.12 | 26.7kNm |
| X2 | M_{DC2} | Moment due to Concrete Parapet | Log Normal | 158kNm | 0.12 | 18.96kNm |
| X3 | M_{DW} | Moment due to polymer surface | Log Normal | 1.7 44kNm | 0.12 | 0.209kNm |
| X4 | M_{LL} | Moment due to live load | Log Normal | 1473kNm | 0.12 | 176.76kNm |
| X5 | t_f | Thickness of Flange | Normal | 0.021m | 0.03 | $6.3 \times 10^{-4}m$ |
| X6 | b_f | Flange width | Normal | 0.5m | 0.03 | 0.015m |

| | | | | | | |
|-----|-----------------|--|--------|-----------|------|-------------------------|
| X7 | d | Depth of section | Normal | $2.4m$ | 0.03 | $0.072m$ |
| X8 | t_w | Thickness of web | Normal | $0.016m$ | 0.03 | $4.8 \times 10^{-4}m$ |
| X9 | D | Distance from bottom of top flange to top of bottom flange | Normal | $2.358m$ | 0.03 | $0.07074m$ |
| X10 | t_{slab} | Slab Thickness | Normal | $0.2m$ | 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 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 | Normal | 1 | 0.05 | 0.05 |
| X15 | | Percentage composite action | Normal | 100% | 0.02 | 2% |

| | | | | | | |
|-----|-------|--------------|---------------|-----------------------------------|------|---------------------------|
| X16 | f_y | Yield Stress | Log Normal | 3.5 $\times 10^5 kN$ $/m^2$ | 0.02 | 7 $\times 10^3 kN/m^2$ |
|-----|-------|--------------|---------------|-----------------------------------|------|---------------------------|

The results of the reliability study considering the strength limit state is shown as follows;

For the top flange;

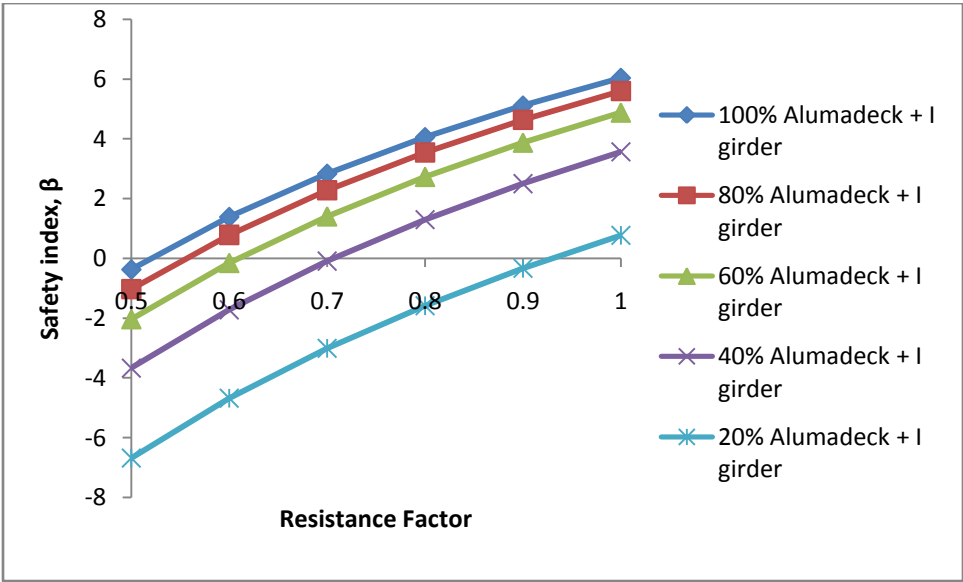


Figure 4-14: Reliability Index at various Resistance Factors for Various Composite action between Alumadeck and girder (Considering Failure of Top Flange)

For the bottom flange;

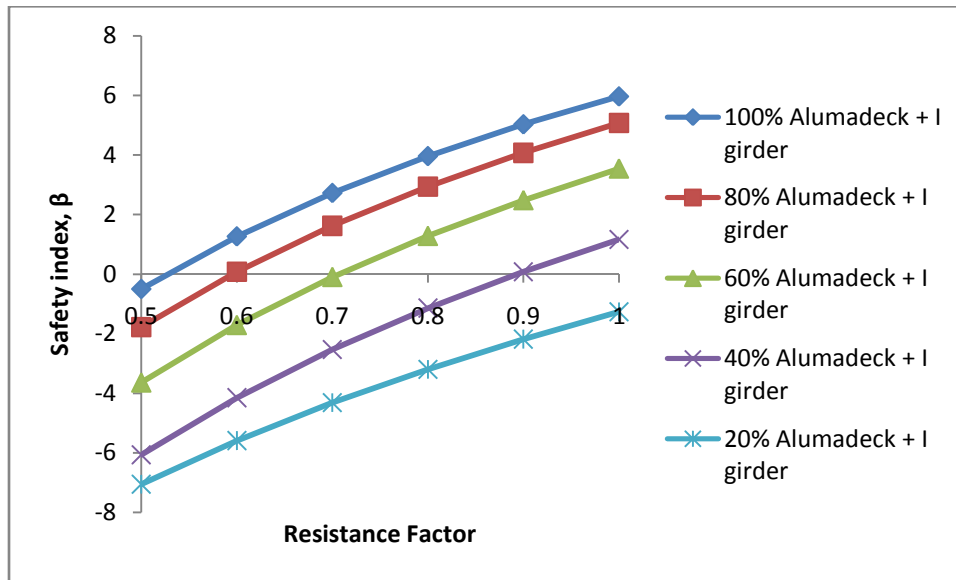


Figure 4-15: Reliability Index at various Resistance Factors for Various Composite action between Alumadeck and girder

The result shows that the design is not acceptable for a resistance factor of 0.5 at 100% and 80% composite action in both compression and tension flanges. The design is also unacceptable for a resistance factor 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. 80% composite action at a resistance factor of 0.8 the safety index is 3.55 considering failure of compression flange while that for the tension flange is 3.0 which are ideal to guarantee safety of the structure. This shows that an I Girder of a depth of 2.4m, flange thicknesses of 21mm, flange width of 500mm and Thickness of web of 16mm is safe to carry the truck loads. See Appendix B for the Reliability Analysis Programs.

CHAPTER FIVE

5 SUMMARY, CONCLUSION AND RECOMMENDATION

5.1 SUMMARY

In this Thesis, the Finite Element and Reliability/ Stochastic Analysis of a Horizontally Curved Aluminium Alloy Bridge Deck on Steel I-Girders is presented. An extensive review of the importance of horizontally curved bridge was carried out. Also an extensive review of the use of the AlumadeckTM was conducted. In the FEA the applied centrifugal as well as the vertical applied loadings were varied at various geometric dimensions. The FEA was performed in the ABAQUS environment. Reliability Analysis was conducted by the use of First Order Reliability Method Coded in FORTRAN to produce Reliability indices at various percentages of composite actions while varying Geometric Dimensions.

5.2 CONCLUSION

From the Finite Element Analysis (FEA) and reliability analysis results, the following conclusions can be inferred:

- (1) It is shown that the propriety Alumadeck system is in conformity with the Load Resistance Factor Design (LRFD) Specification (2004) which stipulates that the target value of reliability index as 3.5 for resistance factor of 1.0 (Assuming 80% composite action compression flange).
- (2) The result of the composite action between the deck and the girder for RF of 1.0 for full composite action (Considering failure of the bottom flange) shows a safety index within acceptable limit which shows that the Alumadeck can withstand the live load it is subjected to considering the HL 93 loading condition.

(3) These studies have proven that the Alumadeck can be used in an area of curved alignment. The alumadeck stiffener should have a minimum thickness of $7mm$ to effectively withstand applied stresses under working conditions. The maximum Von Mises stress was within acceptable limits i.e. $< 1.7 \times 10^5 kN/m^2$ of the yield strength of the aluminum alloy deck. An I girder of depth $2.4m$, flange thicknesses of $21mm$, flange width $50cm$ and web thickness of $16mm$ is safe to carry the applied loading i.e. truck load and selfweight.

5.3 RECOMMENDATION

It is recommended that the results of the ABAQUS analysis can be used as basis for further analysis to have a better interpretation and to be able to further understand the behavior of the deck. It is also recommended that the FORTRAN program formulated in this work be used to design horizontally curved bridge decks at target reliability levels.

REFERENCES

- AASHTO, (2004):**“LRFD Bridge Design Specifications”, American Association of State Highway and Transportation Officials, Washington, D. C.
- AASHTO, (2007):**“LRFD Bridge Design Specifications”, American Association of State Highway and Transportation Officials, Washington, D. C.
- AASHTO, (2010):**“LRFD Bridge Design Specifications”, American Association of State Highway and Transportation Officials, Washington, D. C.
- AASHTO, (1993):**“Guide specifications for horizontally curved highway bridges”, American Association of State Highway and Transportation Officials Washington, D.C.
- AASHTO (2003):**“Guide Specifications for Horizontally Curved Steel Girder HighwayBridges with Design Examples for I-Girder and Box-Girder Bridges”, AmericanAssociation of State and Highway Transportation Officials, Washington D.C.
- AASHTO (2007):**“Guide Specifications for Horizontally Curved Steel Girder HighwayBridges with Design Examples for I-Girder and Box-Girder Bridges”, AmericanAssociation of State and Highway Transportation Officials, Washington D.C.
- Abdelraough, M. R., and Matlock, H. (1972):** “Finite element analysis of bridge decks”. Ctr. for Highway Res., University of Texas at Austin, Austin, Tex.
- Abendroth, R. E., Sanders, W. W., and Hansz, S. (1997):** “Fatigue Tests of Aluminium Girders”, Center For Transportation Research and Education, Iowa State University, U. S. A.
- Abubakar, I., (2006):** “Reliability Analysis of Structural Design Parameters of Strip Footings”, Journal of Applied Science and Research, 1(4): pp 397-401.
- Afolayan, J. O., (1997):** “Evaluation of the Economic Efficiency of the BS 449 Design Criteria for Plate Girders”, Journal of Eng Appl. Sci., Vol 16, No. 1 pp 49 – 54.
- Afolayan, J. O., (2005):** “Probability-Based Design of Glued Thin-Webbed Timber Beams”, Asian Journal of Civil Engineering (Building and Housing) vol. 6, nos. 1-2 (2005) Pages 75-84.
- Agboola, A. O., Abejide, O. S. & Olowosulu, A. T., (2010):** “Probabilistic Evaluation of Aluminum Bridge Decks Design Criteria on Steel Girders”, Journal of Engineering and Applied Sciences 5 (1): 4-13, 2010.
- Alexander, J. and Gunasekara, J.S. (1991):** “Strength of Materials, Volume 2: Advanced Theory and Applications”, Ellis Horwood Limited, New York.

- Aluminium Design Manual (2005):** “The Aluminium Association”, inc. Washington DC, U.S.A.
- Andresen, I., Geissler, M. S., Barbara, R. and Thyholt, M., (2001):**“Sustainable Use of Aluminium in Buildings”, Sintef Civil and Environmental Engineering, Norway.
- Ang A H-S, (1973):** ‘Structural Risk Analysis and Reliability Based Design’, ASCE Structural Division, Vol. 99, No. ST9, pp 1891-1910.
- Augusti, G., Schueller, G. I., Ciampoli, M., (2006):**“Safety and Reliability of Engineering Systems and Structures”, Proceedings of The 9th International Conference on Structural Safety and Reliability ICOSSAR’05/Rome/Italy/19-23 June, Millpress Rotterdam, Netherlands.
- Ayyub, B., and McCuen, R. H. (1997):** “Probability, Statistics, and Reliability for Engineers”, CRC Press LLC, New York, N.Y.
- Benedetti, A., and Tralli, A. (1989):** “New hybrid FE model for arbitrarily curved beam. I: Linear analysis.” *Comp. and Struct.*, 33(6), 1437– 1449.
- Brennan, P. J., and Mandel, J. A. (1973):** “Users manual—program for three dimensional analysis of horizontally curved bridges.” Rep., Res.
- Brett, M. A. and Laman, J. A., (2000):**“Experimental Verification of Horizontally Curved I-Girder Bridge Behaviour”, ASCE Journal of Bridge Engineering, pp 284-292.
- BS5400-2, (1978):** “Steel, Concrete and Composite Bridges”, British Standard Institution, London.
- BS449-2, (1969):** “The Use of Structural Steel in Buildings”, British Standard Institution, London.
- Calderbank, V. J. (1989):**“Programming in Fortran”, Chapman and Hall London.
- Cheung, J. T. C. (1978):** “Experimental and analytical studies of pseudoslab bridge decks”. Dundee University, Dundee, U.K.
- Cheung, Y. K. (1969):** “Analysis of cylindrical orthotropic curved bridge decks.” Proc., IABSE, International Association for Bridge and Structural Engineering, 29(2), 41–52.
- Chu, K. H. and Krishnamoorthy, G. (1962):** ‘Use of Orthotropic Plate Theory in Bridge Design’ ASCE Structural division Vol. 88, No. ST3, pp 35
- Clifton, J. R., Chang C. L., and Au, T. (1963):** ‘Analysis of Orthotropic Plate Bridges, ASCE Structural Division, Vol 89, No. ST5, pp 133-139

- Clough, R. W. (1965):** “The Finite Element Method in Structural Mechanics”, Chapter 7, in “Stress Analysis”, ed. O.C Zienkiewicz and G.S. Hollister, John Wiley and Sons Ltd London.
- Cook, R. D., Malkus, D.S. and Plera, M. E. (1989):** “Concept and Application of Finite Element Analysis”, Third Edition, John Wiley and Sons, New York.
- Daiguji, H., (1972):** “Analytical Study of Orthotropic Steel Bridge Decks” Texas Tech University, Texas.
- Davis, J. R., (1993):** “ASM Speciality Handbook Aluminium and Aluminium Alloys”, ASM International, Materials Park, OH, USA., ISBN-10: 87170-469-X, pp: 605.
- Ditlevsen, O., and Madsen, H. O.,(2005):** “Structural Reliability Methods”, Coastal, Maritime and Structural Engineering, Department of Mechanical Engineering, Technical University of Denmark.
- Dobmeier, J. M., (1998):**“Analytical Evaluation of Bridge Deck Panel”, M.Sc. Thesis of University of Virginia, Charlottesville, USA.
- Dobmeier, J.M., F. W. Barton, J. P. Gomer, P.J. Massarelli and W.T. Jr. McKeel, (2001):** “Analytical evaluation of bridge deck”, J. Performance Constructed Facilities, 15: 68-75.
- Dwight, J., (1999):** “Aluminium Design and Construction”, E and FN Spon, an imprint of Routledge 11, New Fetter Lane, London.
- Eamon, C., Nowak, A. S., Ritter M. A. and Murphy, J. (2000):** “ Reliability Based Criteria for Load and Resistance Factor Design Code For Wood Bridges”, Transportation Research Board, No. 1696 Volume 1, paper No. 5B0016 pp. 1.
- Ellingwood, B., Galambos, J. V., Mac Gregor, J.G. and Cornell, C. A., (1980):**“Development of a Probability Based Load Criteria for American National Standard”, MBS special Publication 577, Washington, D. C., National Bureau of Standards.
- Eurocode 9 (1999):** “ Design of Aluminium Structures”, Centroum Hout, The Netherlands.
- Ferhat, A. and M. F. Dan, (2004):** “Reliability Correction: Comprehensive study for different bridge types”, Journal of Structural Engineering, 130: 1063-1074.
- Fiechtl, A. L., Fenves, G. L., and Frank, K. H. (1987):** “Approximate analysis of horizontally curved girder bridges.” Final Rep. No. FHWATX- 91-360-2F, Ctr. for Transp. Res., University of Texas at Austin, Austin, Tex.
- Galambos, T. V., and Ravindra, M. K. (1978):** “Load and Resistance Factor Design”, Journal of Structural Division, ASCE, ST9, proc. 14008.

- Gollwitzer, S., T. Abdo and R. Rackwitz, (1988):** “First Order Reliability Method: FORM manual. RCP-GMBH, Germany, pp: 134.
- Hadipriono, F. C., (1985):** “Analysis of events in recent structural failures”. J. Struct. Eng., 111: pp:1468-1481.
- Heins, C.P., (1975):** Bending and Torsional Design in Structural Members, Lenington Books, DC health company, London.
- Heins, C. P. and Hails. R. L., (1969):** Behaviour of Stiffened Curved Plate Model’, ASCE Structural Division. Vol. 95 ST11, pp 2353-2369.
- Heins, C. P. and Lee, W. L. H., (1971):** ‘Large Deflection of Curved Plates’, ASCE Structural Division Vol. 97, No. ST 4, pp 8045-8055.
- Heins, C. P. and O’Connor C, (1971):** ‘Design of bridge Superstructures’, 1st Ed., John Willy and sons Inc. New York.
- Heins, C. P. and Spates, K. R. (1970):** ‘Behaviour of Single Horizontally Curved Girder’ ASCE Structural Division, Vol. 96, No. ST7, pp 1511-1525
- Howell, T. D., and Earls, C. J., (2007):** “Curved Steel I-girder Bridge Response During Construction Loading: Effects of Web Plumbness”, Journal of Bridge Engineering, 12(4), 485-493.
- Hsu Y.T. (1989):** “The Development and Behavior of Vlasov Elements for the Modelling of Horizontally Curved Composite Box Girder Bridge Superstructures”. PhD Thesis, Univ. of Maryland, CA.
- Jack R. V., (1974):** ‘Structural Mechanics: The behavior of Plates and Shells’, John Wiley and Sons, New York.
- Juhl, H. G. (1970):** “Horizontal thermal movements of curved bridge decks.” J. Struct. Div., ASCE, 96(10), 2037–2044.
- Kirsch, U. (1981):** “Optimum Structural Design: Concepts, Methods and Applications”, McGraw-Hill, England.
- Kleiber M. and Hein T. D. (1992):** “The Stochastic Finite Element Method”.John Wiley and Sons, Chichester, England.
- Kulicki J. M., Wassef W. G., Smith C. and Johns K. (2005):** “AASHTO-LRFD Design Example Horizontally Curved Steel I-Girder Bridge, Final Report Prepared for National Cooperative Highway Research Program, Transportation Research Board, National Research Council.

- Kwasneiski, L., Szreszen, M. M., and Nowak A. S., (2000):** “Sensitivity Analysis for Slab on Girder Bridges. Paper Presented at the Eighth ASCE specialty conference on Probabilistic mechanics and structural Reliability, pp. 3-4.
- Lavelle, F. H., and Boick, J. S. (1965):** “A program to analyze curved girder bridges.” Engrg. Bulletin–8, University of Rhode Island, Kingston, R.I.
- Lavelle, F. H., and Laska, R. J. (1975a):** “The CUGAR2 algorithm.” CURT Tech. Rep. No. 4(L), Res. Project HPR-2(111), University of Rhode Island, Kingston, R.I.
- Lavelle, F. H., and Laska, R. J. (1975b):** “CUGAR2 and CUGAR3 user’s manuals.” CURT Final Rep., Res. Project HPR-2(111), University of Rhode Island, Kingston, R.I.
- Lavelle, F. H., Greig, R. A., and Wemmer, H. R. (1971):** CUGAR1—a program to analyze curved girder bridges. University of Rhode Island, Kingston, R.I.
- Linzell, D., Chen, A., Sharafbayani, M., Seo, J., Nevling, D. and Ashour, O., (2010):** “Guidelines for Analyzing Curved and Skewed Bridges and Designing Them for Construction”, Pennsylvania Department of Transportation, USA.
- Linzell, D., Hall, D. and White, D., (2004):**“Historical Perspective on Horizontally Curved I-Girder Bridge Design in the United State”, ASCE Journal of Bridge Engineering, pp 218-229.
- Linzell, D., Leon, R. T. and Zureick, A. H. (2004):** “Experimental and Analytical Studies of a Horizontally Curved Steel I-Girder Bridge during Erection”, Journal of Bridge Engineering, ASCE, pp 521 – 530.
- Lydzinski, J. C., Baber and Tralli, (2008):**“Finite Element Analysis of Wolf Creek Multispan Curved Girder Bridge”, Department of Civil and Environmental Engineering, University of Virginia, USA.
- Madsen, H. O., Krenk, S. and Lind, N. C., (1986):** “Methods of Structural Safety”, Prentice-Hall, Englewood Cliffs, N.J.
- Mark, A. M., (2009):** “Hydraulic Design Manual”, Texas Department of Transportation, Design Division (DES), USA.
- Matteo, A.D., (1997):** “Aluminium Bridge Deck System for Highway Bridges. Building to Last” – Proceeding of Structures Congress XV.L. Kempner, Jr. and Brown C. B. ASCE, New York. Vol.2, pp. 841-845
- Matteo, A. D., Massarelli, P. J., Gomez, J. P., Wright, W., and Cooper, J. (1997):**“Preliminary Evaluation of an Aluminium Bridge Deck Design for Highway Bridges”. Seventh Conference on Structural Fault and Repair. Edinburgh, Scotland.

- Mazzolani, F. M., (2006):** “Structural Application of Aluminium in Civil Engineering”, Structural Engineering International Reports.
- Melchers, R. E. (1999):** “Structural Reliability Analysis and Prediction”, Ellis Horwood Limited, Great Britain.
- Monaghan, D. (2001):** “Finite Element Introduction”, Finite Element Research Group, the Queen’s University of Belfast, www.gub.ac.uk/dermot/dmonaghan.html.
- Musa, M. N., (2003):** “Probabilistic Finite Element Analysis of Horizontally Curved Reinforced Concrete Waffle Slab”, A Thesis Submitted to the Postgraduate School, Ahmadu Bello University Zaria, for the Award of Master of Science Degree, Department of Civil Engineering, Ahmadu Bello University Zaria.
- Nowak, A. S. and Collins, K. R., (2000):** “Reliability of Structures”, 1st Edition, McGraw Hill Co. Inc., USA.
- Nowak, A. S. (2004):** “ System Reliability Models for Bridge Structures”, Bulletin of the Polish Academy of Sciences, Vol. 52, No. 4, pp. 321.
- O’ Connor, D. S. (1995):** “Review of Life Cycle Cost Issue Relating to Prefabricated Aluminium Bridge Decks”, Paper for Reynolds Metal Company.
- Ozgun, C., (2007):** “Behaviour and Analysis of a Horizontally Curved and Skewed I-Girder Bridge”, A Thesis Presented to the Academic Faculty, for the Award of Master of Science Degree, Department of Civil and Environmental Engineering, Georgia Institute of Technology, USA.
- Poellot, W. N. (1987):** “Computer-aided design of horizontally curved girders by the V-load method.” AISC Engrg. J., 24(1), 42–50.
- Project HPR-2(111), Syracuse University, Syracuse, N.Y. Grubb, M. A. (1984):** “Horizontally curved I-girder bridge analysis: Vload method.” Transp. Res. Rec. 982, 26–36.
- Raji, H. O., (2003):** “Reliability Assessment of Timber Shell Roof Forms”, A Thesis Submitted to the Postgraduate School, Ahmadu Bello University Zaria, for the Award of Master of Science Degree, Department of Civil Engineering, Ahmadu Bello University Zaria.
- Robert, T. P., (2002):** “Evaluation of Field Tests Performed on Aluminium Bridge”, An M.Sc Thesis of Virginia Polytechnic Institute and State University, Blacksburg.
- Road Research, (1979):** ‘Evaluation of Load Carrying Capacity of Bridges’, Organisation of Economic Cooperation and Development, Paris.

- Rosowsky, D. V., Steward M. G. and Eparachchi, (2002):** “Structural Reliability of Multi Storey Building During Construction”, Journal of Structural Engineering, 128: 205-213.
- Samaan, M., Sennah, K., and Kennedy, J. B., (2002):** “Positioning of Bearings for Curved Continuous spread-box girder Bridges,” Canadian Journal of Civil Engineering, 29:641-652.
- Shim K. W., Monaghan D. J. and Armstrong C. G. (2002):** “Mixed Dimensional Coupling in Finite Element Stress Analysis”, School of Mechanical and Manufacturing Engineering, the Queens University of Belfast, Northern Ireland.
- ABAQUS6.10 (2010):Users Documentation,** Dassault Systems, U. S. A.
- Sharp, M. L. (1993):** “Behaviour and Design of Aluminium Structures”, McGraw-Hill, Inc., New York.
- Siwowski, T., (2006):** “Aluminium Bridges – Past, Present and Future”, Structural Engineering International Reports, pp 286-293.
- Sorensen, J. D., (2003):** “Note 1&2: Structural Reliability”, Institute of Building Technology and Structural Engineering, Aalborg University, Sohngaardsholmsvej 57, DK-9000 Aalborg, Denmark.
- NSBA (2005):Steel Bridge Design Handbook,** National Steel Bridge Alliance, USA.
- Subodh, K. D.and Gilbert, J. K., (2007):** “Aluminium Alloys for Bridges and Decks”, News Letter TMS (The Minerals, Metals and Materials Society), 2007 pp. 61.
- Surahman, A. and K. B. Rojiani, (1983):** “Reliability Based Optimum Design of Concrete Frames”, Journal of Structural Engineering, 128: 205-213.
- Tedesco, J. W., Stalling, J. M., and Tow, D. R., (1995):** Finite Analysis of Bridge Girder Diaphragm Interaction. Vol. 56, No. 2/3, pp. 461-473.
- Tete, M. T. (2003):** “Computer simulated safety test of a horizontally curved reinforced concrete single span bridge”, A Thesis Submitted to the Postgraduate School, Ahmadu Bello University Zaria, for the Award of Master of Science Degree, Department of Civil Engineering, Ahmadu Bello University Zaria.
- Timoshenko, S. P. and Woinowsky-Krieger S., (1982):** Theory of Plates and Shells, McGraw Hill.
- Troitsky, M. S., (1976):**“Stiffened Plates, Bending, Stability and Vibrations”, Elsevier Scientific Publishing Company, New York.

- Tindall, P., (2008):**“Aluminium in Bridges”, ICE manual of Bridge Engineering, 2nd Edition, Thomas Telford Ltd., London.
- Togan, V. and Daloglu, A., (2005):** “Reliability and Reliability-Based Design Optimization”, Turkish Journal Eng. Env. Sci. 30 (2006), pp 237–249.
- Vrijling, J. K., (2000):** “Lecture Notes on Probabilistic Design”, Technische Universiteit, Delft.
- Webster, S.E. and Bannister, A.C. (2001):** Methods, Applications and Software for Structural Reliability Assessment. Corus UK limited.
- Wesley, J. M. and John J. M., (2006):** “Load Testing and Load Distribution Response of Missouri Retrofitted with various FRP Systems Using a non-Contact Optical Measurement System”, Transportation Research Board 85th Annual Meeting, January 22nd-26th Washington, D. C. No 06-1679, pp2.
- White, D. W., Zureick, A. H., Phoawanich, N., and Jung, S. K. (2001):** “Development of unified equations for design of curved and straight steel bridge I girders.” Final Rep., Prepared for American Iron and Steel Institute Transportation and Infrastructure Committee, Professional Service Industries, Inc., and Federal Highway Administration, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta.
- Zeinkiewicz O. C., (1971):** “The Finite Element Method in Engineering Science”, McGraw-Hill, London.
- Zureick, A. and Naqib, R., (1999):** “Horizontally Curved Steel I-Girders State of the Art Analysis Methods”, ASCE Journal of Bridge Engineering, pp 38-47.

APPENDIX A: RESULTS OF FINITE ELEMENT ANALYSIS

Table A-1: Maximum Displacements at a Thickness of stiffener = 5mm

| | Maximum Displacement, U _{max} (mm) at various applied stresses (KN per sq.m) at a Thickness of stiffner = 5mm | | | | | | | |
|--|--|----------|----------|----------|----------|----------|----------|----------|
| Applied Centrifugal Stresses (KN per sq.m) | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| 10 | 4.78E-04 | 9.52E-04 | 1.43E-03 | 1.90E-03 | 2.37E-03 | 2.85E-03 | 3.32E-03 | 3.80E-03 |
| 20 | 4.81E-04 | 9.55E-04 | 1.43E-03 | 1.90E-03 | 2.38E-03 | 2.85E-03 | 3.33E-03 | 3.80E-03 |
| 40 | 4.88E-04 | 9.62E-04 | 1.44E-03 | 1.91E-03 | 2.39E-03 | 2.86E-03 | 3.33E-03 | 3.81E-03 |
| 60 | 4.95E-04 | 9.69E-04 | 1.44E-03 | 1.92E-03 | 2.39E-03 | 2.87E-03 | 3.34E-03 | 3.81E-03 |
| 80 | 5.02E-04 | 9.77E-04 | 1.45E-03 | 1.93E-03 | 2.40E-03 | 2.87E-03 | 3.35E-03 | 3.82E-03 |
| 100 | 5.10E-04 | 9.84E-04 | 1.46E-03 | 1.93E-03 | 2.41E-03 | 2.88E-03 | 3.35E-03 | 3.83E-03 |
| 120 | 5.17E-04 | 9.91E-04 | 1.47E-03 | 1.94E-03 | 2.41E-03 | 2.89E-03 | 3.36E-03 | 3.84E-03 |
| 140 | 5.24E-04 | 9.98E-04 | 1.47E-03 | 1.95E-03 | 2.42E-03 | 2.89E-03 | 3.37E-03 | 3.84E-03 |
| 160 | 5.31E-04 | 1.01E-03 | 1.48E-03 | 1.95E-03 | 2.43E-03 | 2.90E-03 | 3.38E-03 | 3.85E-03 |
| 180 | 5.38E-04 | 1.01E-03 | 1.49E-03 | 1.96E-03 | 2.43E-03 | 2.91E-03 | 3.38E-03 | 3.86E-03 |
| 200 | 5.45E-04 | 1.02E-03 | 1.49E-03 | 1.97E-03 | 2.44E-03 | 2.92E-03 | 3.39E-03 | 3.86E-03 |
| 220 | 5.52E-04 | 1.03E-03 | 1.50E-03 | 1.97E-03 | 2.45E-03 | 2.92E-03 | 3.40E-03 | 3.87E-03 |
| 240 | 5.59E-04 | 1.03E-03 | 1.51E-03 | 1.98E-03 | 2.46E-03 | 2.93E-03 | 3.40E-03 | 3.88E-03 |

Table A-2: Maximum Displacements at a Thickness of stiffener = 7mm

| | Maximum Displacement, U _{max} (mm) at various applied stresses (KN per sq.m) at a Stiffner Thickness = 7mm | | | | | | | |
|--|---|----------|----------|----------|----------|----------|----------|----------|
| Applied Centrifugal Stresses (KN per sq.m) | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| 10 | 4.09E-04 | 8.14E-04 | 1.22E-03 | 1.63E-03 | 2.03E-03 | 2.44E-03 | 2.84E-03 | 3.25E-03 |
| 20 | 4.12E-04 | 8.17E-04 | 1.22E-03 | 1.63E-03 | 2.03E-03 | 2.44E-03 | 2.85E-03 | 3.25E-03 |

| | | | | | | | | |
|-----|----------|----------|----------|----------|----------|----------|----------|----------|
| 40 | 4.18E-04 | 8.23E-04 | 1.23E-03 | 1.63E-03 | 2.04E-03 | 2.45E-03 | 2.85E-03 | 3.26E-03 |
| 60 | 4.24E-04 | 8.29E-04 | 1.24E-03 | 1.64E-03 | 2.05E-03 | 2.45E-03 | 2.86E-03 | 3.26E-03 |
| 80 | 4.30E-04 | 8.35E-04 | 1.24E-03 | 1.65E-03 | 2.05E-03 | 2.46E-03 | 2.86E-03 | 3.27E-03 |
| 100 | 4.36E-04 | 8.41E-04 | 1.25E-03 | 1.65E-03 | 2.06E-03 | 2.46E-03 | 2.87E-03 | 3.27E-03 |
| 120 | 4.42E-04 | 8.47E-04 | 1.25E-03 | 1.66E-03 | 2.06E-03 | 2.47E-03 | 2.88E-03 | 3.28E-03 |
| 140 | 4.48E-04 | 8.53E-04 | 1.26E-03 | 1.66E-03 | 2.07E-03 | 2.48E-03 | 2.88E-03 | 3.29E-03 |
| 160 | 4.54E-04 | 8.59E-04 | 1.27E-03 | 1.67E-03 | 2.08E-03 | 2.48E-03 | 2.89E-03 | 3.29E-03 |
| 180 | 4.60E-04 | 8.65E-04 | 1.27E-03 | 1.68E-03 | 2.08E-03 | 2.49E-03 | 2.89E-03 | 3.30E-03 |
| 200 | 4.66E-04 | 8.71E-04 | 1.28E-03 | 1.68E-03 | 2.09E-03 | 2.49E-03 | 2.90E-03 | 3.31E-03 |
| 220 | 4.72E-04 | 8.78E-04 | 1.28E-03 | 1.69E-03 | 2.09E-03 | 2.50E-03 | 2.91E-03 | 3.31E-03 |
| 240 | 4.78E-04 | 8.84E-04 | 1.29E-03 | 1.70E-03 | 2.10E-03 | 2.51E-03 | 2.91E-03 | 3.32E-03 |

Table A-3: Maximum Displacements at a Thickness of stiffener = 9mm

| Applied Centrifugal Stresses (KN per sq.m) | Maximum Displacement, U _{max} (mm) at various applied stresses (KN per sq.m) at a Stiffner Thickness = 9mm | | | | | | | |
|--|---|----------|----------|----------|----------|----------|----------|----------|
| | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| 10 | 3.77E-04 | 7.52E-04 | 1.13E-03 | 1.50E-03 | 1.88E-03 | 2.25E-03 | 2.63E-03 | 3.00E-03 |
| 20 | 3.80E-04 | 7.54E-04 | 1.13E-03 | 1.50E-03 | 1.88E-03 | 2.25E-03 | 2.63E-03 | 3.00E-03 |
| 40 | 3.85E-04 | 7.59E-04 | 1.13E-03 | 1.51E-03 | 1.88E-03 | 2.26E-03 | 2.63E-03 | 3.01E-03 |
| 60 | 3.90E-04 | 7.64E-04 | 1.14E-03 | 1.51E-03 | 1.89E-03 | 2.26E-03 | 2.64E-03 | 3.01E-03 |
| 80 | 3.94E-04 | 7.69E-04 | 1.14E-03 | 1.52E-03 | 1.89E-03 | 2.27E-03 | 2.64E-03 | 3.02E-03 |
| 100 | 3.99E-04 | 7.74E-04 | 1.15E-03 | 1.52E-03 | 1.90E-03 | 2.27E-03 | 2.65E-03 | 3.02E-03 |
| 120 | 4.04E-04 | 7.79E-04 | 1.15E-03 | 1.53E-03 | 1.90E-03 | 2.28E-03 | 2.65E-03 | 3.03E-03 |
| 140 | 4.09E-04 | 7.84E-04 | 1.16E-03 | 1.53E-03 | 1.91E-03 | 2.28E-03 | 2.66E-03 | 3.03E-03 |

| | | | | | | | | |
|-----|----------|----------|----------|----------|----------|----------|----------|----------|
| | | | | | | | 03 | 03 |
| 160 | 4.14E-04 | 7.89E-04 | 1.16E-03 | 1.54E-03 | 1.91E-03 | 2.29E-03 | 2.66E-03 | 3.04E-03 |
| 180 | 4.19E-04 | 7.94E-04 | 1.17E-03 | 1.54E-03 | 1.92E-03 | 2.29E-03 | 2.67E-03 | 3.04E-03 |
| 200 | 4.24E-04 | 7.99E-04 | 1.17E-03 | 1.55E-03 | 1.92E-03 | 2.30E-03 | 2.67E-03 | 3.05E-03 |
| 220 | 4.29E-04 | 8.04E-04 | 1.18E-03 | 1.55E-03 | 1.93E-03 | 2.30E-03 | 2.68E-03 | 3.05E-03 |
| 240 | 4.34E-04 | 8.08E-04 | 1.18E-03 | 1.56E-03 | 1.93E-03 | 2.31E-03 | 2.68E-03 | 3.06E-03 |

Table A-4: Von Mises stresses at a Thickness of stiffener = 5mm

| Von Misses stress (kN per sq. m) at various applied stresses (KN per sq.m) at a Thickness of stiffner = 5mm | | | | | | | | |
|---|----------|----------|----------|----------|----------|----------|----------|----------|
| Applied Centrifugal Stresses (KN per sq.m) | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| 10 | 1.18E+04 | 2.36E+04 | 3.53E+04 | 4.71E+04 | 5.89E+04 | 7.06E+04 | 8.24E+04 | 9.42E+04 |
| 20 | 1.23E+04 | 2.54E+04 | 3.54E+04 | 4.71E+04 | 5.89E+04 | 7.07E+04 | 8.24E+04 | 9.42E+04 |
| 40 | 1.27E+04 | 2.47E+04 | 3.54E+04 | 4.72E+04 | 5.89E+04 | 7.07E+04 | 8.25E+04 | 9.42E+04 |
| 60 | 1.30E+04 | 2.50E+04 | 3.70E+04 | 4.72E+04 | 5.90E+04 | 7.08E+04 | 8.25E+04 | 9.43E+04 |
| 80 | 1.34E+04 | 2.54E+04 | 3.74E+04 | 4.93E+04 | 5.81E+04 | 7.08E+04 | 8.26E+04 | 9.44E+04 |
| 100 | 1.38E+04 | 2.57E+04 | 3.77E+04 | 4.97E+04 | 6.17E+04 | 7.09E+04 | 8.26E+04 | 9.44E+04 |
| 120 | 1.63E+04 | 2.61E+04 | 3.80E+04 | 5.00E+04 | 6.20E+04 | 7.40E+04 | 8.27E+04 | 9.45E+04 |
| 140 | 1.64E+04 | 2.64E+04 | 3.84E+04 | 5.04E+04 | 6.24E+04 | 7.44E+04 | 8.64E+04 | 9.45E+04 |
| 160 | 1.64E+04 | 2.68E+04 | 3.87E+04 | 5.07E+04 | 6.27E+04 | 7.47E+04 | 8.67E+04 | 9.87E+04 |
| 180 | 1.70E+04 | 2.71E+04 | 3.91E+04 | 5.11E+04 | 6.31E+04 | 7.50E+04 | 8.70E+04 | 9.90E+04 |
| 200 | 1.82E+04 | 2.75E+04 | 3.94E+04 | 5.14E+04 | 6.34E+04 | 7.54E+04 | 8.74E+04 | 9.94E+04 |
| 220 | 1.94E+04 | 3.26E+04 | 3.98E+04 | 5.18E+04 | 6.37E+04 | 7.57E+04 | 8.77E+04 | 9.97E+04 |
| 240 | 2.06E+04 | 3.26E+04 | 4.02E+04 | 5.21E+04 | 6.41E+04 | 7.61E+04 | 8.81E+04 | 1.00E+05 |

Table A-5: Von Mises stresses at a Thickness of stiffener = 7mm

| | Von Misses stress (kN per sq. m) at various applied stresses (KN per sq.m) at a Thickness of stiffner = 7mm | | | | | | | |
|--|---|----------|----------|----------|----------|----------|----------|----------|
| Applied Centrifugal Stresses (KN per sq.m) | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| 10 | 1.05E+04 | 2.08E+04 | 3.11E+04 | 4.14E+04 | 5.18E+04 | 6.21E+04 | 7.24E+04 | 8.27E+04 |
| 20 | 1.06E+04 | 2.09E+04 | 3.13E+04 | 4.16E+04 | 5.19E+04 | 6.22E+04 | 7.25E+04 | 8.29E+04 |
| 40 | 1.09E+04 | 2.12E+04 | 3.15E+04 | 4.19E+04 | 5.22E+04 | 6.25E+04 | 7.28E+04 | 8.31E+04 |
| 60 | 1.12E+04 | 2.15E+04 | 3.18E+04 | 4.22E+04 | 5.25E+04 | 6.28E+04 | 7.31E+04 | 8.34E+04 |
| 80 | 1.15E+04 | 2.18E+04 | 3.21E+04 | 4.24E+04 | 5.28E+04 | 6.31E+04 | 7.34E+04 | 8.37E+04 |
| 100 | 1.18E+04 | 2.21E+04 | 3.24E+04 | 4.27E+04 | 5.31E+04 | 6.34E+04 | 7.37E+04 | 8.40E+04 |
| 120 | 1.22E+04 | 2.24E+04 | 3.27E+04 | 4.30E+04 | 5.33E+04 | 6.37E+04 | 7.40E+04 | 8.43E+04 |
| 140 | 1.25E+04 | 2.27E+04 | 3.30E+04 | 4.33E+04 | 5.36E+04 | 6.40E+04 | 7.43E+04 | 8.46E+04 |
| 160 | 1.28E+04 | 2.30E+04 | 3.33E+04 | 4.36E+04 | 5.39E+04 | 6.42E+04 | 7.46E+04 | 8.49E+04 |
| 180 | 1.35E+04 | 2.34E+04 | 3.36E+04 | 4.39E+04 | 5.42E+04 | 6.45E+04 | 7.49E+04 | 8.52E+04 |
| 200 | 1.44E+04 | 2.37E+04 | 3.39E+04 | 4.42E+04 | 5.45E+04 | 6.48E+04 | 7.52E+04 | 8.55E+04 |
| 220 | 1.53E+04 | 2.40E+04 | 3.42E+04 | 4.45E+04 | 5.48E+04 | 6.51E+04 | 7.54E+04 | 8.58E+04 |
| 240 | 1.61E+04 | 2.43E+04 | 3.46E+04 | 4.48E+04 | 5.51E+04 | 6.54E+04 | 7.57E+04 | 8.61E+04 |

Table A-6: Von Mises stresses at a Thickness of stiffener = 9mm

| | Von Misses stress (kN per sq. m) at various applied stresses (KN per sq.m) at a Thickness of stiffner = 9mm | | | | | | | |
|--|---|----------|----------|----------|----------|----------|----------|----------|
| Applied Centrifugal Stresses (KN per sq.m) | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| 10 | 8.80E+03 | 1.76E+04 | 2.64E+04 | 3.52E+04 | 4.40E+04 | 5.28E+04 | 6.16E+04 | 7.04E+04 |

| | | | | | | | | |
|-----|----------|----------|----------|----------|----------|----------|----------|----------|
| | | | | | | 4 | | 04 |
| 20 | 9.79E+03 | 1.76E+04 | 2.64E+04 | 3.52E+04 | 4.40E+04 | 5.28E+04 | 6.16E+04 | 7.04E+04 |
| 40 | 1.00E+04 | 1.96E+04 | 2.64E+04 | 3.52E+04 | 4.40E+04 | 5.28E+04 | 6.16E+04 | 7.04E+04 |
| 60 | 1.03E+04 | 1.98E+04 | 2.94E+04 | 3.52E+04 | 4.40E+04 | 5.28E+04 | 6.16E+04 | 7.04E+04 |
| 80 | 1.05E+04 | 2.01E+04 | 2.96E+04 | 3.92E+04 | 4.41E+04 | 5.28E+04 | 6.16E+04 | 7.04E+04 |
| 100 | 1.08E+04 | 2.03E+04 | 2.99E+04 | 3.94E+04 | 4.90E+04 | 5.29E+04 | 6.16E+04 | 7.04E+04 |
| 120 | 1.10E+04 | 2.06E+04 | 3.01E+04 | 3.96E+04 | 4.92E+04 | 5.88E+04 | 6.18E+04 | 7.04E+04 |
| 140 | 1.25E+04 | 2.08E+04 | 3.03E+04 | 3.99E+04 | 4.94E+04 | 5.90E+04 | 6.86E+04 | 7.07E+04 |
| 160 | 1.29E+04 | 2.11E+04 | 3.06E+04 | 4.01E+04 | 4.97E+04 | 5.92E+04 | 6.88E+04 | 7.83E+04 |
| 180 | 1.38E+04 | 2.13E+04 | 3.08E+04 | 4.04E+04 | 4.99E+04 | 5.95E+04 | 6.90E+04 | 7.86E+04 |
| 200 | 1.48E+04 | 2.16E+04 | 3.11E+04 | 4.06E+04 | 5.02E+04 | 5.97E+04 | 6.93E+04 | 7.88E+04 |
| 220 | 1.57E+04 | 2.18E+04 | 3.13E+04 | 4.09E+04 | 5.04E+04 | 5.99E+04 | 6.95E+04 | 7.91E+04 |
| 240 | 1.66E+04 | 2.21E+04 | 3.16E+04 | 4.11E+04 | 5.06E+04 | 6.02E+04 | 6.97E+04 | 7.93E+04 |

APPENDIX B: RELIABILITY ANALYSIS PROGRAMS

```
PROGRAM STRENGTH
C   PROBABILISTIC EVALUATION OF AN HORIZONTALLY CURVED
C   ALUMINIUM ALLOY BRIDGE DECK ON STEEL I GIRDERS
C   considering strength limit state of girder G4
C   Top flange at midspan
c   x(1)=Moment due to self weight Alumadeck and I-girder
c   x(2)=Moment due to concrete parapet
c   x(3)=Moment due to polymer/ wearing surface
c   x(4)=Live load moment
c   x(5)=Thickness of flanges
c   x(6)=Width of flanges
c   x(7)=Depth of section
c   x(8)=Thickness of web
c   x(9)=Depth of web
c   x(10)=Thickness of slab
c   x(11)=Haunch thickness
c   x(12)=Area of slab long term
c   x(13)=Area of slab short term
c   x(14)=Load resistance factor
c   x(15)=Percentage composite action
c   x(16)=Yield Stress of Steel
C   IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C   THE SUBROUTINE WITH THE LIMIT STATE FUNCTION IS
C   DECLARED EXTERNAL
      EXTERNAL GSHEAR
      DIMENSION X(16),EX(16),SX(16),VP(10,16),COV(16,16),ZES(3),
+         UU(16),EIVEC(16,16),IV(2,16)
      CHARACTER*10 PRT
      common/cshear/load
      DATA EX/2.7D2,2.2D2,2.5D1,7.0D2,2.1D-2,5.0D-1,2.4D0,1.6D-2,
+         2.4D0,2.0D-1,9.0D-2,6.0D-2,1.8D-1,1.0D0,1.0D2,3.5D5/,
+         SX/3.2D1,2.7D1,2.9D0,8.3D1,6.3D-4,1.5D-2,7.2D-2,4.8D-4,
+         7.0D-2,6.0D-3,2.7D-3,1.8D-3,5.4D-3,5.0D-2,2.0D0,7.0D3/,
+         N/16/,NC/16/,NE/16/,IRHO/1/
      WRITE(*,*)'ENTER RF, Percentage Alumadeck (%)....>'
      READ(*,*)EX(14),EX(15)
      SX(14) = 0.05*EX(14)
      SX(15) = 0.02*EX(15)
C   THE RESULTS ARE WRITTEN TO NAUS 7
      NAUS=7
C   PRINT TO SCREEN
      ICRT=0
      OPEN(7,FILE='SHITTU1.RES',STATUS='OLD',ERR=10)
      GOTO 20
10     OPEN(7,FILE='SHITTU1.RES',STATUS='NEW')
C   PRESETTING VARIABLES VP,COV,AND IV IS DONE USING YINT
20     CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,2)=3
      IV(1,3)=3
      IV(1,4)=3
      IV(1,16)=3
```

```

DO 100 I=1,N
X(I) = EX(I)
100 CONTINUE
V1=1.0
BETA=1.D0
C INITIAL SOLUTION ESTIMATE
WRITE (NAUS,5000)
5000 FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
+ 'CMPSTE ACTN BTWN ALUMADECK AND STEEL I GIRDER, 16 VARIABLES:')
CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
WRITE (ICRT,*) ' START OF FORM5'
WRITE (ICRT,*) ' STOCHASTIC MODEL :'
CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
PRT=' COV '
CALL YMAUS (NAUS,NC,N,COV,PRT)
CALL FORM5 (N, IV, EX, SX, VP, GSHEAR, IRHO, COV, NC,
+ EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
C THE CORDINATE OF THE BETA POINT ARE PRINTED WITH THE
C THE TITLE VECTOR UU
PRT='UU'
CALL YFAUS (NAUS,N,UU,PRT)
C THE VECTOR ZES IS PRINTED WITH THE TITLE VECTOR ZES
PRT='ZES'
CALL YFAUS (NAUS,3,ZES,PRT)
WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
WRITE (ICRT,*) ' RESULTS SEE FILE SHITTU1.RES'
STOP
END

```

```

SUBROUTINE GSHEAR (N, X, FX, IER)
IMPLICIT DOUBLE PRECISION (A-H,O-Z)
DIMENSION X(N)
common/cshear/Load
Af= X(5)*X(6)
Aw= X(8)*X(9)
Aift= X(6)*(X(5)**3)/12
AIw=X(8)*(X(9)**3)/12
Cnc=X(7)/2+0.29
Aq=Af*(Cnc-(X(5)/2))**2
Az=Aw*(Cnc-(X(9)/2)-X(5))**2
AInc=2*Aift+Aq+Af*(X(7)-Cnc-(X(5)/2))**2+AIw+Az
Ba=Af*(X(10)+X(11)-(X(5)/2))
Bb=Af*(X(9)+X(10)+X(11)+(X(5)/2))
Ar=Ba+Bb
Bc=Aw*((X(9)/2)+X(10)+X(11))
Bd=X(12)*0.01*X(15)*X(10)*0.5
At=Bc+Bd
Ctn=(Ar+At)/(Af+Aw+X(12)*0.01*X(15))
Ay=(Af*(Ctn-X(10)-X(11)+(X(5)/2))**2)
Au=(Af*(X(10)+X(11)+X(9)+X(5)-Ctn-(X(5)/2))**2)
Ai=Aw*(Ctn-X(10)-X(11)-(X(9)/2))**2
Ao=(X(12)*0.01*X(15)*X(10)**2/12)
Ap=X(12)*0.01*X(15)*(Ctn-(0.5*X(10)))**2
AItn=2*Aift+Ay+Au+AIw+Ai+Ao+Ap

```

```

Be=Af*(X(10)+X(11)-(X(5)/2))
Bf=Af*(X(9)+X(10)+X(11)+(X(5)/2))
As=Be+Bf
Bg=Aw*((X(9)/2)+X(10)+X(11))
Bh=X(13)*0.01*X(15)*X(10)*0.5
Ad=Bg+Bh
Cn=(As+Ad)/(Af+Aw+X(13)*0.01*X(15))
Ah=Af*(Ctn-X(10)-X(11)+(X(5)/2))**2
Aj=Af*(X(10)+X(11)+X(9)+X(5)-Ctn-(X(5)/2))**2
Ak=Aw*(Ctn-X(10)-X(11)-(X(9)/2))**2
Al=(X(13)*0.01*X(15)*X(10)**2/12)
Ax=X(13)*0.01*X(15)*(Ctn-(0.5*X(10)))**2
AIn=2*Aift+Ah+Aj+AIw+Ak+Al+Ax
AInc=2*Aift+Aq+Af*(X(7)-Cnc-(X(5)/2))**2+AIw+Az
Bi=(1.25*X(1)*Cnc)/AInc
Bj=((1.25*X(2)+1.5*X(3))*Ctn)/AIn
Av=Bi+Bj
fbu=12*(Av+(1.75*X(4)*Cn/AIn))
fyc=X(14)*X(16)
C CHECK FOR ERRORS
IF(fbu.GT.0)THEN
FX=fyc-fbu
IER=0
ELSE
FX=1.D+20
IER=1
ENDIF
RETURN
END

```

```

PROGRAM STRENGTH
C PROBABILISTIC EVALUATION OF AN HORIZONTALLY CURVED
C ALUMINIUM ALLOY BRIDGE DECK ON STEEL I GIRDERS
C considering strength limit state of girder G4
C Bottom flange at midspan
c x(1)=Moment due to self weight Alumadeck and I-girder
c x(2)=Moment due to concrete parapet
c x(3)=Moment due to polymer/ wearing surface
c x(4)=Live load moment
c x(5)=Thickness of flanges
c x(6)=Width of flanges
c x(7)=Depth of section
c x(8)=Thickness of web
c x(9)=Depth of web
c x(10)=Thickness of slab
c x(11)=Haunch thickness
c x(12)=Area of slab long term
c x(13)=Area of slab short term
c x(14)=Load resistance factor
c x(15)=Percentage composite action
c x(16)=Yield Stress of Steel
IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C THE SUBROUTINE WITH THE LIMIT STATE FUNCTION IS
C DECLARED EXTERNAL

```

```

EXTERNAL GSHEAR
DIMENSION X(16),EX(16),SX(16),VP(10,16),COV(16,16),ZES(3),
+      UU(16),EIVEC(16,16),IV(2,16)
CHARACTER*10 PRT
common/cshear/load
DATA EX/2.7D2,2.2D2,2.5D1,7.0D2,2.1D-2,5.0D-1,2.4D0,1.6D-2,
+      2.4D0,2.0D-1,9.0D-2,6.0D-2,1.8D-1,1.0D0,1.0D2,3.5D5/,
+      SX/3.2D1,2.7D1,2.9D0,8.3D1,6.3D-4,1.5D-2,7.2D-2,4.8D-4,
+      7.0D-2,6.0D-3,2.7D-3,1.8D-3,5.4D-3,5.0D-2,2.0D0,7.0D3/,
+      N/16/,NC/16/,NE/16/,IRHO/1/
WRITE(*,*)'ENTER RF, Percentage composite action (%)...>'
READ(*,*)EX(14),EX(15)
SX(14) = 0.05*EX(14)
SX(15) = 0.02*EX(15)
C      THE RESULTS ARE WRITTEN TO NAUS 7
      NAUS=7
C      PRINT TO SCREEN
      ICRT=0
      OPEN(7,FILE='SHITTU2.RES',STATUS='OLD',ERR=10)
      GOTO 20
10     OPEN(7,FILE='SHITTU2.RES',STATUS='NEW')
C      PRESETTING VARIABLES VP,COV,AND IV IS DONE USING YINT
20     CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,2)=3
      IV(1,3)=3
      IV(1,4)=3
      IV(1,16)=3
      DO 100 I=1,N
      X(I) = EX(I)
100    CONTINUE
      V1=1.0
      BETA=1.D0
C      INITIAL SOLUTION ESTIMATE
      WRITE (NAUS,5000)
5000   FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
+ 'CMPSTE ACTN BTWN ALUMADECK AND STEEL I GIRDER, 16 VARIABLES:')
      CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
      WRITE (ICRT,*) ' START OF FORM5'
      WRITE (ICRT,*) ' STOCHASTIC MODEL :'
      CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
      PRT=' COV '
      CALL YMAUS (NAUS,NC,N,COV,PRT)
      CALL FORM5 (N, IV, EX, SX, VP, GSHEAR, IRHO, COV, NC,
+      EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
C      THE CORDINATE OF THE BETA POINT ARE PRINTED WITH THE
C      THE TITLE VECTOR UU
      PRT='UU'
      CALL YFAUS (NAUS,N,UU,PRT)
C      THE VECTOR ZES IS PRINTED WITH THE TITLE VECTOR ZES
      PRT='ZES'
      CALL YFAUS (NAUS,3,ZES,PRT)
      WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
      WRITE (ICRT,*) ' RESULTS SEE FILE SHITTU2.RES'

```

```

STOP
END

SUBROUTINE GSHEAR (N, X, FX, IER)
IMPLICIT DOUBLE PRECISION (A-H,O-Z)
DIMENSION X(N)
common/cshear/Load
Af= X(5)*X(6)
Aw= X(8)*X(9)
AIft= X(6)*(X(5)**3)/12
AIw=X(8)*(X(9)**3)/12
Cnc=2.69-(X(7)/2+0.29)
Aq=Af*(Cnc-(X(5)/2))**2
Az=Aw*(Cnc-(X(9)/2)-X(5))**2
AInc=2*AIft+Aq+Af*(X(7)-Cnc-(X(5)/2))**2+AIw+Az
Ba=Af*(X(10)+X(11)-(X(5)/2))
Bb=Af*(X(9)+X(10)+X(11)+(X(5)/2))
Ar=Ba+Bb
Bc=Aw*((X(9)/2)+X(10)+X(11))
Bd=X(12)*0.01*X(15)*X(10)*0.5
At=Bc+Bd
Ctn=2.69-((Ar+At)/(Af+Aw+X(12)*0.01*X(15)))
Ay=(Af*(Ctn-X(10)-X(11)+(X(5)/2))**2)
Au=(Af*(X(10)+X(11)+X(9)+X(5)-Ctn-(X(5)/2))**2)
Ai=Aw*(Ctn-X(10)-X(11)-(X(9)/2))**2
Ao=(X(12)*0.01*X(15)*X(10)**2/12)
Ap=X(12)*0.01*X(15)*(Ctn-(0.5*X(10)))**2
AItn=2*AIft+Ay+Au+AIw+Ai+Ao+Ap
Be=Af*(X(10)+X(11)-(X(5)/2))
Bf=Af*(X(9)+X(10)+X(11)+(X(5)/2))
As=Be+Bf
Bg=Aw*((X(9)/2)+X(10)+X(11))
Bh=X(13)*0.01*X(15)*X(10)*0.5
Ad=Bg+Bh
Cn=2.69-((As+Ad)/(Af+Aw+X(13)*0.01*X(15)))
Ah=Af*(Ctn-X(10)-X(11)+(X(5)/2))**2
Aj=Af*(X(10)+X(11)+X(9)+X(5)-Ctn-(X(5)/2))**2
Ak=Aw*(Ctn-X(10)-X(11)-(X(9)/2))**2
Al=(X(13)*0.01*X(15)*X(10)**2/12)
Ax=X(13)*0.01*X(15)*(Ctn-(0.5*X(10)))**2
AIn=2*AIft+Ah+Aj+AIw+Ak+Al+Ax
AInc=2*AIft+Aq+Af*(X(7)-Cnc-(X(5)/2))**2+AIw+Az
Bi=(1.25*X(1)*Cnc)/AInc
Bj=((1.25*X(2)+1.5*X(3))*Ctn)/AItn
Av=Bi+Bj
fbu=12*(Av+(1.75*X(4)*Cn/AIn))
Ca=0.00017286
fl=1.25*(Ca*(X(1)+X(2)))+1.5*Ca*X(3)+1.75*Ca*X(4)
fr=fbu+((1/3)*fl)
fyc=X(14)*X(16)
CHECK FOR ERRORS
IF(fr.GT.0)THEN
FX=fyc-fr
IER=0

```

C

```
ELSE  
FX=1.D+20  
IER=1  
ENDIF  
RETURN  
END
```



```

*surface, type=ELEMENT, name=ASSEMBLY__PICKEDSURF5
*surface, type=ELEMENT, name=ASSEMBLY__PICKEDSURF4
*surface, type=ELEMENT, name=ASSEMBLY__PICKEDSURF5
*material, name="CURVED ALUMADECK"
*density
*elastic
*plastic
*solidsection, elset="ASSEMBLY_CURVED DECK 1-1__PICKEDSET2",
material="CURVED ALUMADECK"
*boundary
*solidsection, elset="ASSEMBLY_CURVED DECK 1-1__PICKEDSET2",
material="CURVED ALUMADECK"
*surface, type=ELEMENT, name=ASSEMBLY__PICKEDSURF4
*surface, type=ELEMENT, name=ASSEMBLY__PICKEDSURF5
*output, field, variable=PRESELECT
*output, history, variable=PRESELECT
*output, field, variable=PRESELECT
*output, history, variable=PRESELECT
*output, field, variable=PRESELECT
*output, history, variable=PRESELECT
*output, field, variable=PRESELECT
*output, history, variable=PRESELECT
*Step, name="apply pressure"
*output, field, variable=PRESELECT
*output, history, variable=PRESELECT
*Step, name="apply pressure"
*Step, name="apply pressure"
*static
*boundary
*dsload
*dsload
*output, field, variable=PRESELECT
*output, history, variable=PRESELECT
*endstep
*Step, name="apply pressure"
*static
*boundary

***WARNING: DEGREE OF FREEDOM 4 IS NOT ACTIVE IN THIS MODEL AND CAN NOT
BE
          RESTRAINED

***WARNING: DEGREE OF FREEDOM 5 IS NOT ACTIVE IN THIS MODEL AND CAN NOT
BE
          RESTRAINED

***WARNING: DEGREE OF FREEDOM 6 IS NOT ACTIVE IN THIS MODEL AND CAN NOT
BE
          RESTRAINED
*output, field, variable=PRESELECT
*output, history, variable=PRESELECT
*endstep

- (RAMP) OR (STEP) - INDICATE USE OF DEFAULT AMPLITUDES ASSOCIATED
WITH THE STEP

```

- (RAMP) OR (STEP) - INDICATE USE OF DEFAULT AMPLITUDES ASSOCIATED WITH THE STEP

P R O B L E M S I Z E

NUMBER OF ELEMENTS IS 17172
NUMBER OF NODES IS 14326
NUMBER OF NODES DEFINED BY THE USER 14326
TOTAL NUMBER OF VARIABLES IN THE MODEL 42978
(DEGREES OF FREEDOM PLUS MAX NO. OF ANY LAGRANGE MULTIPLIER VARIABLES. INCLUDE *PRINT,SOLVE=YES TO GET THE ACTUAL NUMBER.)

END OF USER INPUT PROCESSING

JOB TIME SUMMARY

USER TIME (SEC) = 1.3000
SYSTEM TIME (SEC) = 0.20000
TOTAL CPU TIME (SEC) = 1.5000
WALLCLOCK TIME (SEC) = 2

1

Abaqus 6.10-1 Date 23-Oct-2013 Time
17:12:58

For use by TEAM TBE under license from Dassault Systemes or its subsidiary.

STEP 1 INCREMENT 1

TIME COMPLETED IN THIS STEP 0.00

S T E P 1 S T A T I C A N A L Y S I S

AUTOMATIC TIME CONTROL WITH -

A SUGGESTED INITIAL TIME INCREMENT OF 1.00
AND A TOTAL TIME PERIOD OF 1.00
THE MINIMUM TIME INCREMENT ALLOWED IS 1.000E-05
THE MAXIMUM TIME INCREMENT ALLOWED IS 1.00

LINEAR EQUATION SOLVER TYPE DIRECT SPARSE

M E M O R Y E S T I M A T E

| PROCESS | FLOATING PT OPERATIONS PER ITERATION | MINIMUM MEMORY REQUIRED (MBYTES) | MEMORY TO MINIMIZE I/O (MBYTES) |
|---------|--|--|---------------------------------------|
| 1 | 3.14E+009 | 42 | 160 |

NOTE:

(1) SINCE ABAQUS DOES NOT PRE-ALLOCATE MEMORY AND ONLY ALLOCATES MEMORY AS NEEDED DURING THE ANALYSIS, THE MEMORY REQUIREMENT PRINTED HERE CAN ONLY BE VIEWED AS A GENERAL GUIDELINE BASED ON THE BEST KNOWLEDGE AVAILABLE AT THE BEGINNING OF A STEP BEFORE THE SOLUTION PROCESS HAS BEGUN.

(2) THE ESTIMATE IS NORMALLY UPDATED AT THE BEGINNING OF EVERY STEP. IT IS THE MAXIMUM VALUE OF THE ESTIMATE FROM THE CURRENT STEP TO THE LAST STEP OF THE ANALYSIS, WITH UNSYMMETRIC SOLUTION TAKEN INTO ACCOUNT IF APPLICABLE.

(3) SINCE THE ESTIMATE IS BASED ON THE ACTIVE DEGREES OF FREEDOM IN THE FIRST ITERATION OF THE CURRENT STEP, THE MEMORY ESTIMATE MIGHT BE SIGNIFICANTLY DIFFERENT THAN ACTUAL USAGE FOR PROBLEMS WITH SUBSTANTIAL CHANGES IN ACTIVE DEGREES OF FREEDOM BETWEEN STEPS (OR EVEN WITHIN THE SAME STEP). EXAMPLES ARE: PROBLEMS WITH SIGNIFICANT CONTACT CHANGES, PROBLEMS WITH MODEL CHANGE, PROBLEMS WITH BOTH STATIC STEP AND STEADY STATE DYNAMIC PROCEDURES WHERE ACOUSTIC ELEMENTS WILL ONLY BE ACTIVATED IN THE STEADY STATE DYNAMIC STEPS.

(4) FOR MULTI-PROCESS EXECUTION, THE ESTIMATED VALUE OF FLOATING POINT OPERATIONS FOR EACH PROCESS IS BASED ON AN INITIAL SCHEDULING OF OPERATIONS AND MIGHT NOT REFLECT THE ACTUAL FLOATING POINT OPERATIONS COMPLETED ON EACH PROCESS. OPERATIONS ARE DYNAMICALLY BALANCED DURING EXECUTION, SO THE ACTUAL BALANCE OF OPERATIONS BETWEEN PROCESSES IS EXPECTED TO BE BETTER THAN THE ESTIMATE PRINTED HERE.

(5) THE UPPER LIMIT OF MEMORY THAT CAN BE ALLOCATED BY ABAQUS WILL IN GENERAL DEPEND ON THE VALUE OF THE "MEMORY" PARAMETER AND THE AMOUNT OF PHYSICAL MEMORY AVAILABLE ON THE MACHINE. PLEASE SEE THE "ABAQUS ANALYSIS USER'S MANUAL" FOR MORE DETAILS. THE ACTUAL USAGE OF MEMORY AND OF DISK SPACE FOR SCRATCH DATA WILL DEPEND ON THIS UPPER LIMIT AS WELL AS THE MEMORY REQUIRED TO MINIMIZE I/O. IF THE MEMORY UPPER LIMIT IS GREATER THAN THE MEMORY REQUIRED TO MINIMIZE I/O, THEN THE ACTUAL MEMORY USAGE WILL BE CLOSE TO THE ESTIMATED "MEMORY TO MINIMIZE I/O" VALUE, AND THE SCRATCH DISK USAGE WILL BE CLOSE-TO-ZERO; OTHERWISE, THE ACTUAL MEMORY USED WILL BE CLOSE TO THE PREVIOUSLY

MENTIONED MEMORY LIMIT, AND THE SCRATCH DISK USAGE WILL BE
ROUGHLY PROPORTIONAL TO THE DIFFERENCE
BETWEEN THE ESTIMATED "MEMORY TO MINIMIZE I/O" AND THE MEMORY
UPPER LIMIT. HOWEVER ACCURATE
ESTIMATE OF THE SCRATCH DISK SPACE IS NOT POSSIBLE.
(6) USING "*RESTART, WRITE" CAN GENERATE A LARGE AMOUNT OF DATA
WRITTEN IN THE WORK DIRECTORY.

THE ANALYSIS HAS BEEN COMPLETED

ANALYSIS COMPLETE
WITH 3 WARNING MESSAGES ON THE DAT FILE
AND 1 WARNING MESSAGES ON THE MSG FILE

JOB TIME SUMMARY
USER TIME (SEC) = 3.9000
SYSTEM TIME (SEC) = 0.10000
TOTAL CPU TIME (SEC) = 4.0000
WALLCLOCK TIME (SEC) = 6