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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Performance Evaluation of Reinforced concrete Bridges: A case study on the assessment of Koka Bridge

A Thesis in Structural Engineering

By

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Masters of Science

The undersigned have examined the thesis entitled **'Performance Evaluation of Reinforced concrete Bridges without Plans: A case study on the assessment of Koka Bridge'** presented by

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I certify that research work titled "**Performance Evaluation of Reinforced concrete Bridges without Plans: A case study on assessment of Koka Bridge**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ABSTRACT

In Ethiopia, reinforced concrete bridges that were constructed in the early and mid of 20th century exist in large numbers and are still in service. Due to the changing nature of vehicular load supported by these bridges and frequent permit trucks using the bridges, assessment of bridges to obtain their load carrying capacity is usually desired. The legal owner of the bridges, the Ethiopian Roads Authority, cannot provide necessary information such as as-built drawings, and properties of construction materials.

The thesis demonstrates the assessment of load carrying capacity of old and deteriorated reinforced concrete bridges without plans (as built drawings or design drawings). Assessment codes give sufficient recommendations on how to load rate bridges that does not have sufficient data to be used in load rating. The safe live load carrying capacity of a highway structure is known as its rating.

A case study has been conducted on one of the bridges along the heavily loaded route corridors bridges i.e. Koka River Bridge. Data obtained from historical records, field inspection and measurement besides design code existing by the time of construction of the bridge, 1953 G.C. has been used to prepare an effective Finite Element model on Abaqus/CAE 6.13-1. A moving tire load on the bridge deck has been used for the analysis, and the results obtained have been used to load rate the bridge for a permit load and ERA's legal loads.

The bridge was found to have sufficient capacity to withstand Legal loads and selected permit load, it has 50% and 8% reserve capacity respectively. However, it is recommended to maintain defects and distresses noticed on field inspection since they might lead to further deterioration of the bridge.

Key words: Bridges, Reinforced concrete, load rating, FEM

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Symbols

f_s = tensile unit stress in longitudinal reinforcement

f_c = compressive unit stress in extreme fiber of concrete

E_s = Modulus of elasticity of steel

E_c = modulus of elasticity of concrete

n = ratio of modulus of steel to concrete E_s / E_c

M= bending moment, or moment of resistance in general

A_s = effective cross sectional area of tension reinforcement

B = width of beam

d = effective depth, or depth from compression surface of beam to center of tension reinforcement

k = ratio of depth of neutral axis to effective depth, d

j = ratio of lever arm of resisting couple to depth, d

j_d = d-z = arm of resisting couple

p = ratio of effective area of tension reinforcement to effective area of concrete in beam = A_s/bd

z = depth from compression surface of beam to resultant of compressive stresses

W_C= Roadway width between curbs exclusive of median strip.

N = Number of design traffic lanes as shown in section 3.2.6 of Reference 27

W= Width of design traffic lane

I = impact fraction (maximum 30 per cent).

L = length in feet of the portion of the span which is loaded to produce the maximum stress in the member.

b=width of flange

b'= width of stem

t=thickness of flange

A'= area of compressive steel

p'= ratio of effective area of compression reinforcement to effective area of concrete in bema=A'/bd

f'_s = compressive unit stress in longitudinal reinforcement

C= total compressive stress in concrete

V= total shear

V' = external shear on any section after deducting that carried by the concrete,

v = shearing unit stress.

u = bond stress per unit of area of surface of bar.

o = perimeter of bar.

So = sum of perimeters of bars in one set.

a = spacing of web reinforcement bars, measured perpendicular to their direction.

s= spacing of web reinforcement bars, measured at the neutral axis and in the direction of the longitudinal axis of the beam.

$\mathbf{A_v}$ = total area of web reinforcement in tension within a distance, a, of the total area of all bars bent up in any one plane,

o = angle between web bars and longitudinal bars.

$\mathbf{f_v}$= tensile unit stress in web reinforcement

CHAPTER 1 : INTRODUCTION

1.1 Background Information

Large scale investment needs import of machineries for factories and industry parks, transformers and turbines for dams and so on. Import of such Heavy Non-divisible goods poses an interesting question on how to transport them form the dry ports that Ethiopia uses to the sought after place. This leads to another question of which route corridors of the country would be used to transport such goods. Bridges play a crucial role of linking segments of roads, therefore the Load carrying capacity of a bridge restricts the type and heaviness of goods that can be transported on the entire route corridor.

On the other hand, bridges are designed and built to meet specific load criteria. Despite safety margins in design, an overloaded transport cargo or vehicle may endanger the public and adversely affect the sustainability of the structure and its life expectancy. However, the need to transport special cargo which greatly exceeds the design criteria arises from time to time. Such journeys are sometimes made with special vehicles, such as the one in Figure 1.1. Passage of permit trucks always pose a challenge to bridge engineers, entailing design experience, field experience (especially in bridge inspection) besides a good understanding of heavy vehicles and the operation at hand.

The main problem that makes assessment of bridges very hard is unavailability of detailed data that describes structural dimensions, bridge design data, or as-built plan (structural drawings) of most of the bridges in the country. Especially for reinforced concrete bridges, since the reinforcement amount and detailing cannot be obtained by mere visual inspection and sometimes not even with availability of advanced reinforcement detecting devices. This poses a problem since assessment is typically performed by utilizing critical information available on the bridge plans. This includes information about the span lengths, the sizes and dimensions of the bridge members, the type of materials used to construct the bridge, and other relevant information. Despite the circumstances , engineering judgment, non-destructive load testing and field inspection reports has commonly been used, along with historical records and plans of similarly constructed bridges to arrive at plausible reinforcing details and asses performance of bridges; even if, these estimates tend to be conservative when the data available is not sufficient.

74.8% of bridges in Ethiopia are reinforced concrete bridges. And 30% of these bridges are constructed before 1980G.C; also they are slightly or heavily deteriorated. Besides the historical records about these bridges is limited to only year of construction. Any other record (data) available for these bridges has been obtained through field inspection and measurement done over the last seven years.

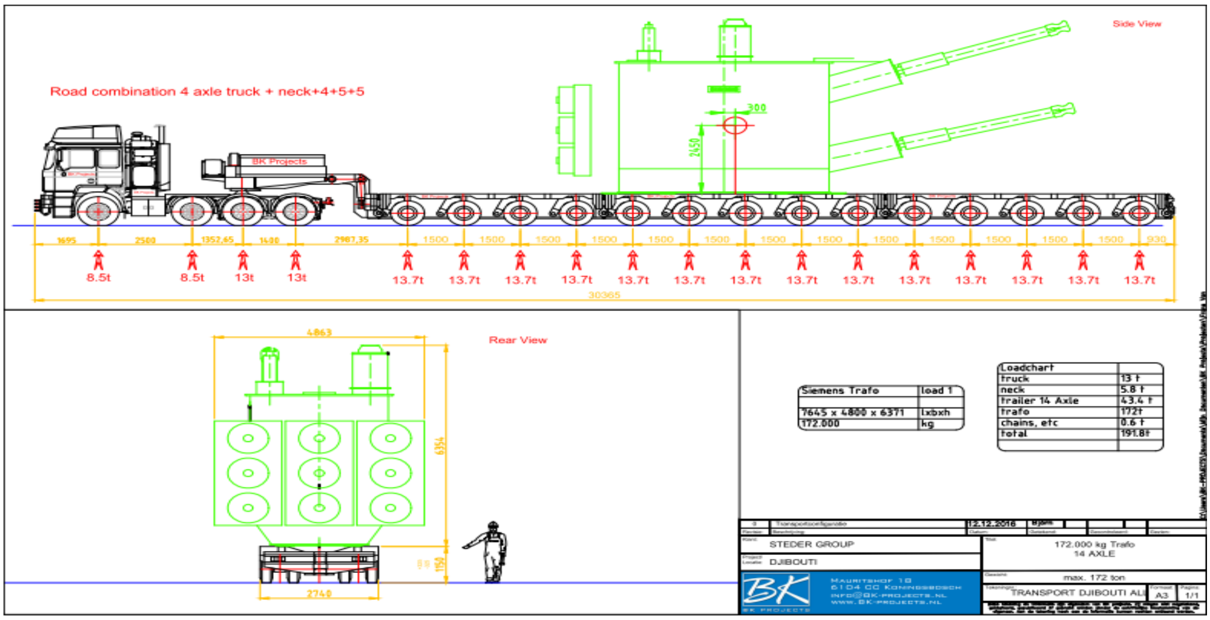


Figure 1. 1: Heavy truck load arrangement, 172 ton transformer

Another interest/concern in assessing performance of bridges is the effect of deterioration in their load carrying capacity, since deterioration of structures alters their response to loading, its effect should always be taken in to account in assessment calculations. However, quantifying the effect of deterioration is not an easy task, as it needs a detailed historical data; the most necessary one in this case is Traffic data. All of the road segments in Ethiopia has only seven years of organized traffic data, whereas bridge in Ethiopia are aged, most older than 30 years. Therefore, methods used in other countries to assess bridges without plans cannot strictly be followed.

The points raised above call for a special attention on most of bridges in Ethiopia, especially those that are aligned on the Import-Export route corridors.

1.2 Research Questions

The growth in axle loads of vehicles is apparent; there is an increase in traffic related to high rate of urbanization. Besides transport of heavy non-divisible goods (heavy permit tracks) is inevitable, for the accomplishment of mega hydropower projects, installation of factories and so on. However, load carrying capacity of the most critical members in the road network, i.e. bridges, should be checked first. This aspect should be studied thoroughly and country specific problems such as unavailability of records and testing equipment should also be taken in to account.

Therefore, the following research questions will be studied in this thesis:

- Can load carrying capacity of an old bridge without plans be identified by combining historical records, Field inspection and test data?
- Does finite element model prepared with simulation software packages reasonably represent bridge's response? And how can it be used in bridge load rating and permit issues for heavy truck loads?
- Does the bridge, in the case study of section 4; have sufficient load carrying capacity to pass the permit truck load of proposed configuration?
- What is the load rating of the bridge, in the case study of section 4, for the legal vehicular loads given in ERA design manual, section 5?

1.3 Objective

The objective of this thesis is to show a logical procedure to be used in assessing the load carrying capacity of old and deteriorated reinforced concrete bridges without plans (as built drawings or design drawings) and historical records. The performance of a Bridge along a heavily loaded route corridor of Ethiopia under selected special Permit Heavy truck Load arrangement (or "permit" loads) is taken as a case study.

Here Performance is described as load carrying capacity of the bridge and serviceability after passage of single/multiple truck loads. A permit truck load and also Legal truck load arrangements provided by ERA's bridge design manual are used in the assessment. The bridge has been load rated as per ERA's bridge design manual.

1.4 Scope and limitations

The research covers the load carrying capacity and serviceability assessment of one selected bridge, aligned on a route corridor which is expected to experience such heavy loads for a selected special Permit Heavy truck, as a case study. A simulation software package Abaqus /CAE 6.13-1 has been used to make a refined analysis on the bridge.

The literature review and the case study focus on assessment of reinforced concrete bridges, which constitute 75 % of bridges in Ethiopia. The central concept of this study is performance evaluation of reinforced concrete bridges without plans. Hence, the bridge selected for the case study, Koka River Bridge, which is constructed in 1953 G.C. The limitations on the Finite Elements Model of the bridge prepared are specified in chapter 4: Finite Element modeling of the case study bridge.

1.5 Methodology

A critical bridge along one of the heavily loaded route corridors is first selected for a case study. The selected bridge has been modeled on a simulation software package, Abaqus/CAE. The analysis results have been used to determine the performance of the bridge under proposed vehicular live loads.

The process includes:

- Selection of a reinforced concrete Bridge for a case study.
- Conducting detailed field inspection and testing on the selected bridge, besides gathering available relevant data from the concerned authority, in this case Ethiopian Roads Authority (ERA)
- Putting plausible estimate on the reinforcement amount and detailing of the bridge using engineering judgment, field inspection data and also by referring methods of design by the time the bridge is constructed.
- Modeling and Analysis of the bridge by a simulation software package, Abaqus/CAE 6/13-1 is used in this study.

- Validation of material models used in modeling the bridge in Abaqus by paralleling laboratory test results of a beam with its FE model in Abaqus.
- Verification of the bridge's FE model to identify whether it represents the response of the real bridge in acceptable manner.
- Load rating the bridge for ERA's legal loads and selected special permit load, as per ERA's design manual, Part II section 5.
- Giving recommendations and conclusions based on data gathered through field inspection and load rating calculations.

1.6 Structure of the Thesis

Chapter 1: This chapter gives a brief introduction to the aim of the thesis. Here it is tried to show the existing problem/gap and a methodology is proposed to solve it.

Chapter 2: Literatures, assessment codes, books, projects and papers done in this area have been reviewed in order to give the reader a miniature background. It has also been tried to emphasize on the prominence of the problem and also to show the reliability of the methodology employed in the thesis, by referring to trusted assessment codes.

Chapter 3: Koka River Bridge which is selected for the case study is described in a detailed manner. The existing historical records, field inspection and testing outputs, method used to estimate reinforcement detail has been described here. And also the permit load used to check the load carrying capacity of the bridge is defined.

Chapter 4: Finite element modeling and analysis of the case study bridge superstructure, validation of material model used for the bridge, verification of global bridge model are included in this chapter.

Chapter 5: Analysis and result interpretation done to load rate the bridge for permit load and legal loads proposed by ERA design manual, section 5 are illustrated in this chapter.

Chapter 6: Conclusions and recommendations are given with reference to the reviewed literation, field inspection results and outputs of analysis made on Abaqus.

CHAPTER 2 : LITERATURE REVIEW

Industrial developments often necessitate the movement of large indivisible loads that exceed legal load or dimension limits. So authorized bodies manage their bridge asset in an environment where there is a continuing increase in the axle loads, number of heavily loaded vehicles and the gross weight of heavy vehicles that access the road and bridge network. [1]

Bridge load capacity is governed by age, the design standard prevailing at the time of design, strength of materials used, quality of construction, type of environment, loading spectrum and standard of maintenance. Accordingly, the maximum permissible load on a particular road link will generally be determined by the capacity of the weakest structures. [1]

In this chapter, a literature review has been made on the basics of bridge performance evaluation of, emphasis given to the Ethiopian practice. In Ethiopia, a permit system administered by the Ethiopian Roads Authority Road Asset Management Division is authorized to manage bridge assets.

Bridge load rating is a central concept on performance evaluation on almost every assessment manual. The concept of load rating in relation of strength evaluation, loads used in load rating of bridges, and other important aspects are discussed here. Relevant and applicable aspects of the bridges in Ethiopia are also been discussed.

Since performance evaluation of a bridge is *case sensitive,* all information needed on the computations should be gathered for each and every bridge prior to the evaluation works, therefore bridge inspection becomes a crucial part here, which is briefly discussed in this Literature review. Finite element modeling of reinforced concrete for assessment has also been briefly discussed.

Bridges without plans are a central concept to this research; hence the whole hypothesis is built on the applicability of historical records, field inspection and testing in load rating bridges. Hence, it has been reasoned on the acceptability of using engineering judgment, design standards by the time of construction of a bridge without plan and field inspection of the bridge and a refined analysis method in putting an estimate on the load carrying capacity of a bridge.

2.1 Performance Evaluation of a Bridge

The evaluation of a structure is based on the simple principle that the available capacity of a structure to carry loads must exceed the required capacity to support the applied loadings. To perform an evaluation, therefore, it is necessary to know something about the available capacity, the applied loading and the response of the structure to that loading. *Knowledge and information with respect to each of these items is never complete; and therefore, evaluation can never be done precisely [7].*

The basic structural engineering equation states that the resistance of a structure must equal or exceed the demand placed on it by loads. Stated mathematically:

$$R \geq \Sigma Q_k \quad \text{(Eq. 2.1)}$$

Where: R = resistance

Q_k = effect of load k

In structural evaluation, the objective is to determine the maximum allowable live load. In the case of bridge evaluation, this usually means the maximum vehicle weight.

In determining the load rating criteria for a bridge, consideration should be given to the types of vehicles using the bridge routinely. Every effort should be made to minimize hardships related to economic hauling without jeopardizing the safety of the public. [2]

A full bridge assessment report should contain at least the following key Information [1]. All assumptions taken and the assessment criteria used should be well stated for the assessment output to be confidently used as an input in decisions of permit issues and legal load posting.

- BMS ID , Bridge Name
- Road No , Road Name, Road Section Name ,Location (GPS)
- Heavy Vehicle Route
- Loading Level
- Traffic data
- Assessment vehicles, Axle configuration
- Travel Restrictions
- Over load control
- Assessment lane width
- Bridge redundancy [high or low] ,
- Material Properties
- Analysis type for evaluation,
- Element Capacities

2.2 Bridges in Ethiopia

Ethiopian Roads Authority's Road Asset Management division is responsible for the management of approximately 1689 bridges and 2805 major culverts,. The asset management department is responsible for the periodic inspection, maintenance and documentation of the bridges all over the country. Most of the information about bridges in Ethiopia has been obtained from country wide major inspection conducted in 2013 G.C. The bridge Management division of ERA had hired consultants in to condition rate the bridges country wide, which is how most of the existing data about the bridges is obtained. Figure 2.1 and 2.2 are charts that show distribution of the bridges in Ethiopia based on construction material and age. However, an engineer who would like to conduct detailed assessment to study the load carrying capacity of the bridges should do an extensive site investigation on the bridges since most of the necessary information needed for load rating computation cannot be provided by the Authority.

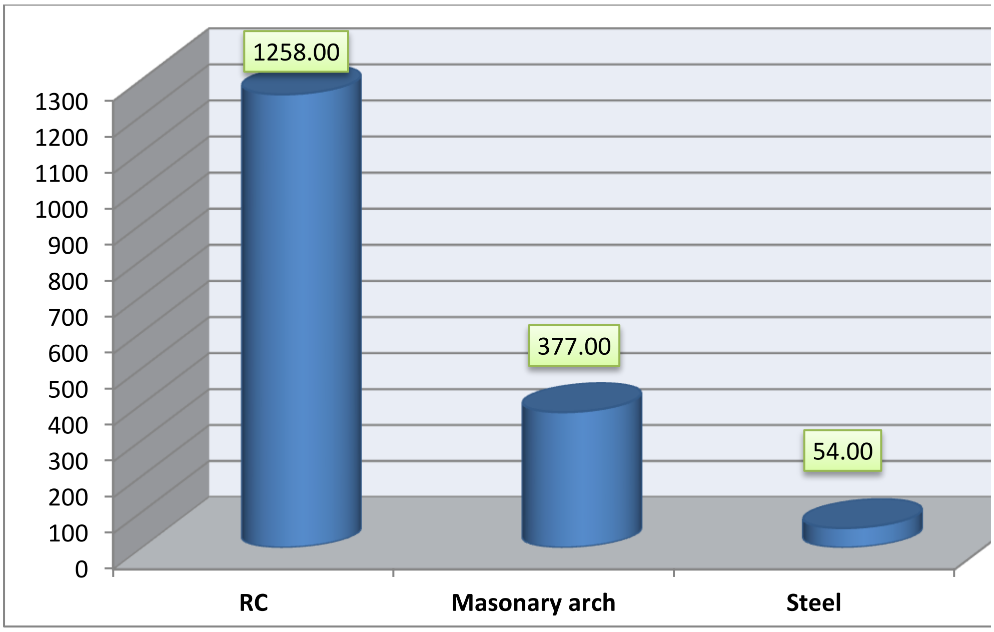


Figure 2. 1: Ethiopian bridges classified based on their construction material, (those administered by ERA)

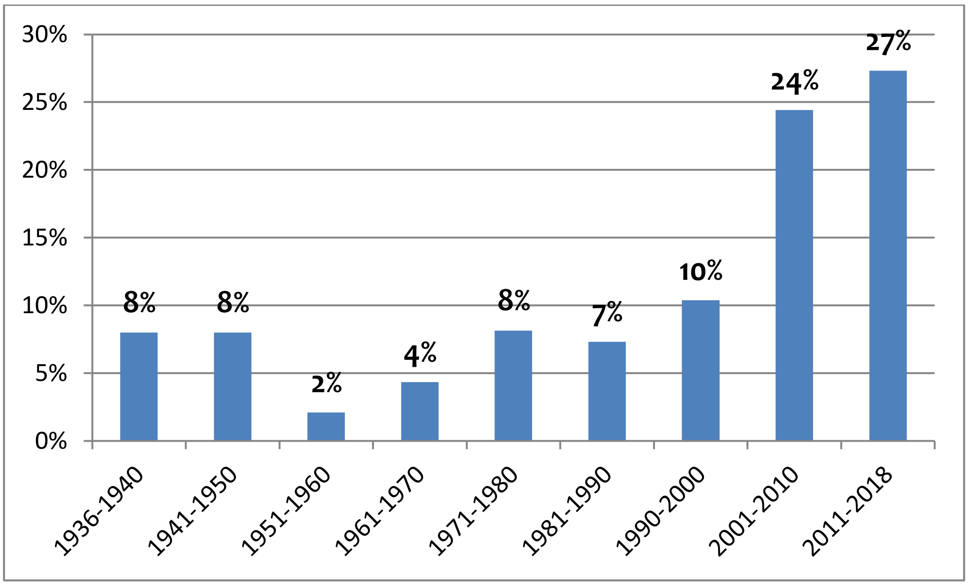


Figure 2. 2: Reinforced concrete Bridges with date of construction (those administered by ERA)

As it can be seen from Figure 2.2, 47 % of the reinforced bridges have been constructed before 2000 G. C which is 18 years ago. Also numerous bridges were constructed in the early nineties.

2.3 Bridge Load Rating

The safe live load carrying capacity of a highway structure is called its load rating. . It is generally stated as a rating factor for a particular live load model, using the general load rating equation. Load rating is specific for each case of loading, axle configuration and rating criteria. When the load rating of a bridge is greater than or equal to one, it implies that the bridge is strong enough for current traffic conditions [3].

Bridge ratings generally require the Engineer to consider a wider range of variables than is typical in bridge design. Design may adopt a conservative reliability index and impose checks to ensure serviceability and durability without incurring a major cost impact. In rating, the added cost of overly conservative evaluation standards can be prohibitive as load restrictions, rehabilitation, and replacement become increasingly necessary.
Evaluation criteria should be adjusted based on site conditions and/or structure conditions as recorded in the most recent inspection report. So, load rating of a bridge is based on existing structural conditions, material properties, loads and traffic conditions at the bridge site.

The load rating equation is generally expressed as a rating factor for a particular live load model, using the general load-rating equation provided in [Article 6A.4.2 of REF 2]

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW}) \pm (\gamma_{P})(P)}{\gamma_{LL}(LL + IM)} \quad \text{....Eq. 2.2}$$

Where:
RF =Rating factor
C = Capacity
f_R = Allowable stress specified in the LRFD code
Rn = Nominal member resistance (as inspected)
DC = Dead load effect due to structural

For the strength limit states:

$$C = \varphi_c \varphi_s \varphi R_n$$

Where the following lower limit shall apply

$$\varphi \varphi \geq 0.85$$

For the service limit states:

$$C = f_R$$

components and attachments

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

γ_{DC} = LRFD load factor for structural components and attachments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

γ_P = LRFD load factor for permanent loads other than dead loads =1.0

γ_{LL} = Evaluation live load factor

φ_c = condition factor

φ_s = system

φ = LRFD resistance factor

In load rating, ductility is considered in conjunction with redundancy and incorporated in the system factors [5].

Definition of terms in the General Load rating Equation is as follows can be found in reference [5].

Whereas, the simplified Rating equation of ERA's design manual, section 5 is as follows:

$$RF = \frac{C - (\gamma_D)(D)}{\gamma_{LL}(LL + IM)} \quad \text{……………………………………………………………………..Eq. 2.3}$$

$$C = \varphi R_n$$

Figure 2.3 shows bridge load rating process in ERA design manual, part II section 5.

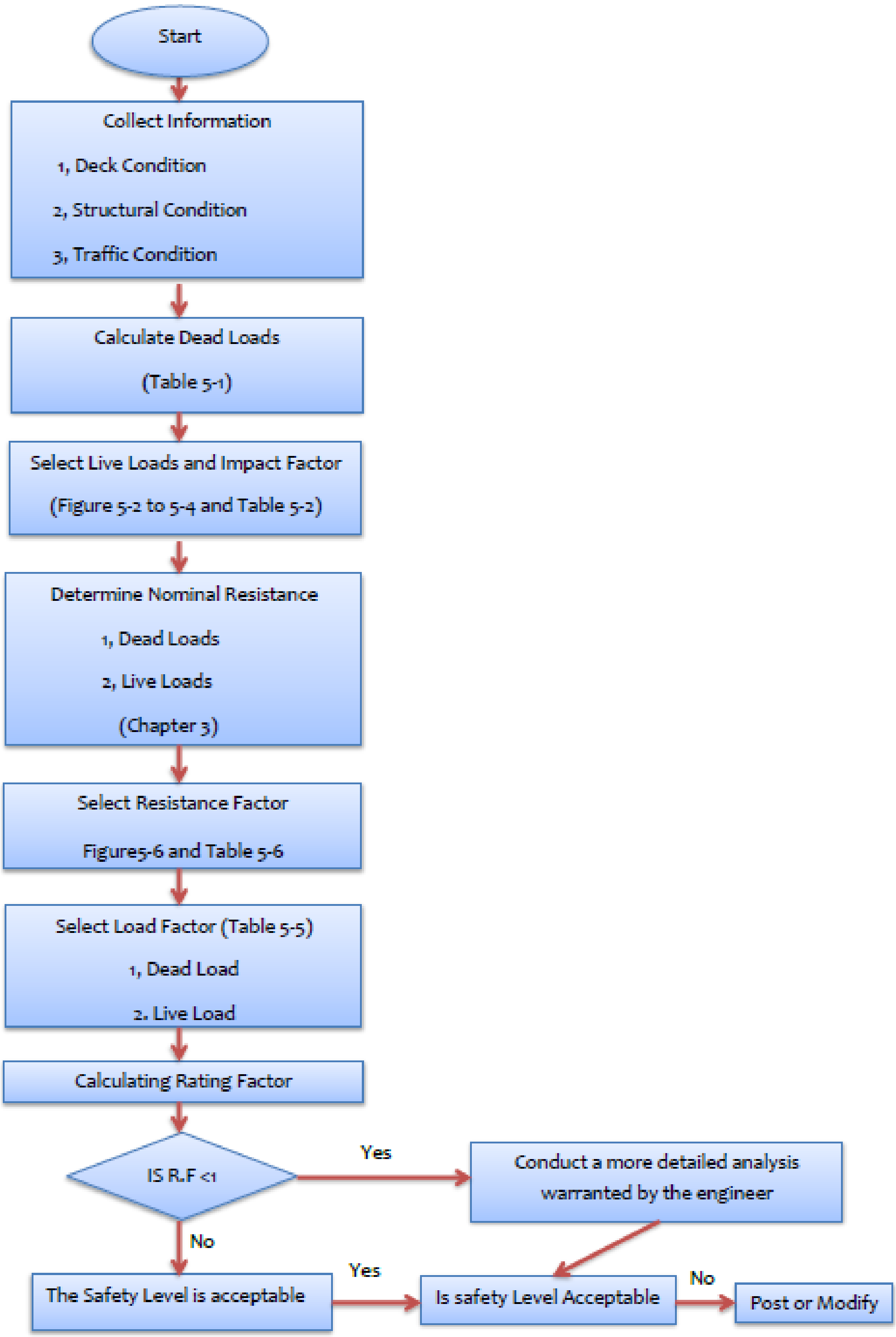


Figure 2. 3: Flow chart for Load rating procedure, ERA bridge design Manual [8]

N.B: The tables, figures stated to be referred in this flow chart are of ERA's bridge design manual

2.4 Loads for Rating Bridges

Unlike design where all applicable loads and load combinations are considered in the design process, in rating there is special interest in the live load and in the capacity of a member relative to the live load [5].

Rating Loads consist of forces that are applied directly to the bridge or result from deformations or the constraint of deformations. *Loadings other than dead load and traffic live load usually do not result in significant Bending or shear in the superstructure. Since the critical mode of failure for traffic live load almost always occurs in the superstructure, other types of loads will seldom affect the live load capacity of the bridge*[2].

Extreme events have a very low probability of occurrence but impart very high-magnitude forces on a structure, which is one of the reasons that the primary focus of most assessment manuals is the assessment of the safety of bridges for live loads (including overloads) and fatigue [2].

But, fatigue need not be investigated for concrete deck slabs in multi-girder applications. Stresses measured in concrete deck slabs of bridges in service are far below infinite fatigue life, most probably due to internal arching action; see Article C9.7.2 of [4].

Hydraulic considerations (scour/ice/debris), wind loads, temperature effects, collisions, and the effects of creep and shrinkage are generally not considered in the load rating of bridges, but should be considered under unique circumstance and structure types [2].

2.4.1 Vehicular live loads

Highway vehicles come in a wide variety of sizes and configurations. *No single vehicle or load model can accurately reflect the effects of all of these vehicles.* To minimize this difference, it is necessary to select a rating Legal Truck with axle spacing and relative axle weights similar to actual vehicles. [8] Manuals specify the Legal truck load in terms of number of axles, axle configuration, and axle load. ERA bridge design manual, section 5, strength evaluation of bridges, Figure 2.4, illustrates the axle configuration and axle load of three Legal truck loads to be used for strength evaluation.

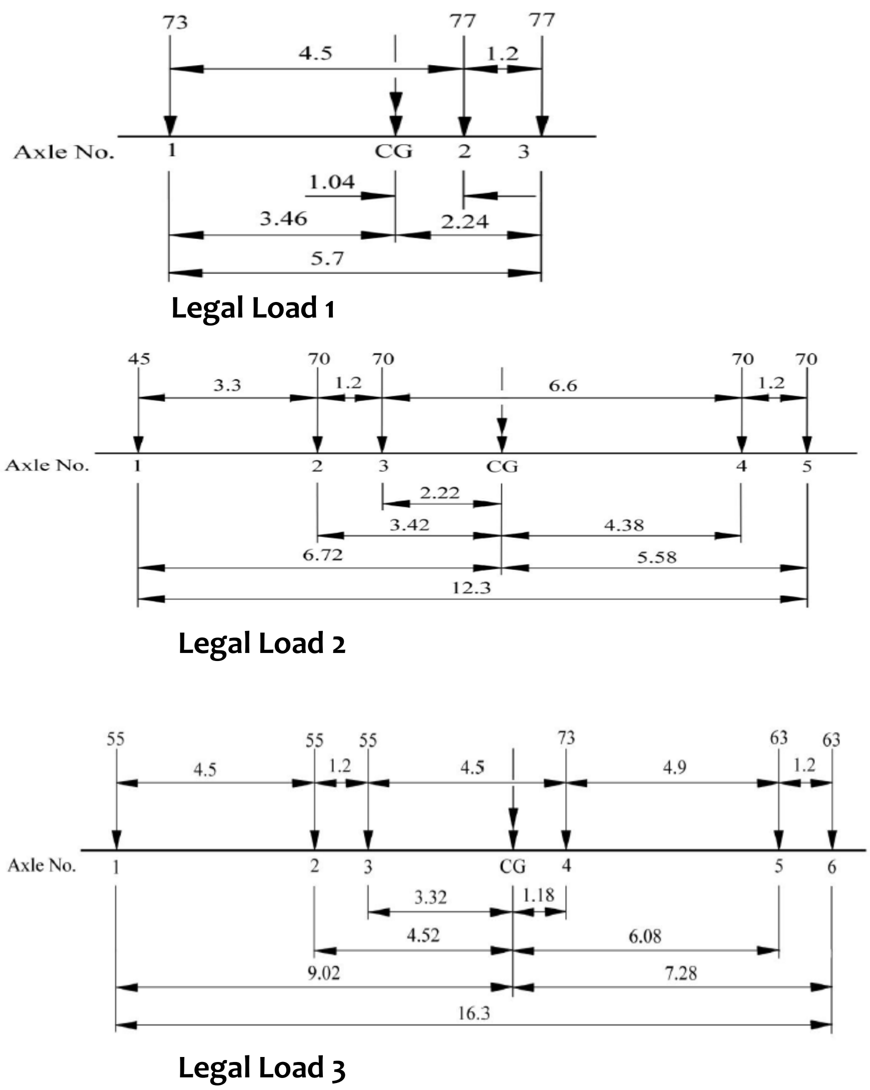


Figure 2. 4: load configuration of the three of the legal loads, ERA design manual Part II section 5

In loading bridge superstructure, bridge deck, determining the contact area between the tire and the bridge deck (Tire Contact Area) is essential. Contact patch is the portion of a vehicle's tire that is in actual contact with the road surface. [13]

The contact between the tire and the bridge deck should be approximated to rectangular patch. ERA bridge design manual gives expression to estimate tire contact area. The manual states that the tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose with width and length computed from Equation 2.2 below. The tire pressure shall be assumed to be uniformly distributed over the contact area.

Tire width = P/0.8

Tire length = 6.4γ (1 + IM/100) ………………………………………..Eq. 2.4

Where:

γ = load factor

IM = dynamic load allowance percent

P = design wheel load (kip)

2.4.2 Dynamic Load Allowance (Impact factor)

An impact allowance shall be added to the static loads used for rating as shown in the load rating equation. Impact values in Part 1Chapter 3 of ERA design manual reflect conservative conditions that may possibly prevail under certain circumstances. Under an enforced speed restriction, impacts shall be reduced.

The main parameters affecting dynamic load allowance are the bridge approach, bumps, and other pavement roughness. Field tests have shown that the most important factor affecting impact is roadway roughness and any bumps, sags, or other discontinuities which may initiate or amplify dynamic response to truck passages. Any of these surface factors should be noted during a bridge inspection [8]. Three values of impact factors are provided by correlating the roughness of the surface to the deck conditions survey values[8].

2.4.3 Permit Load

Loads beyond the range of standard legal vehicles that need to cross the bridges, are referred as "permit" loads. Most of the time, the vehicles that need special permit are those that carry the "Non-divisible loads".

"Non-divisible load" means that which cannot be reduced in size or weight, or which is impractical to divide, or which cannot be so adjusted as to be within the size and weight limitations specified within limits in codes, because to do so would:

I. Compromise the intended use of the vehicle, i.e., make it unable to perform the function for which it was intended;

II. Destroy the value of the load or vehicle, i.e., make it unusable for its intended purpose; or

III. Require more than eight work hours to dismantle using appropriate equipment. The applicant for non-divisible load permit has the burden of proof as to the number of work hours required to dismantle the load [4].

2.5 Capacity/resistance of structural Elements

The determination of structural resistance is one of the primary tasks in the evaluation process. Nominal strength calculations shall take into consideration the observable effects of deterioration, such as loss of concrete or steel cross-sectional area, loss of composite action or corrosion. Where steel is severely corroded, concrete deteriorated, make a determination of the loss in a cross-sectional area as closely as reasonably possible [2].

Different methods for considering the observable effects *of deterioration* were studied in developing the guidelines in reference [8]. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths. An alternate approach is to calculate resistance based on plan dimensions and use a smaller capacity reduction factor [8].

The ERA design manual relates condition of elements to be rated with the capacity reduction factor (resistance factors). By its essence capacity reduction factor takes into consideration the dimensional variations of the structure, differences in material properties, *current condition and future deterioration*, and the inaccuracies in the theory for calculating resistance.

However it should be recognized that, Deterioration of concrete components does not necessarily reduce their resistance. For instance, loss of cover due to spalling might not have a significant influence on the member resistance if the main load-carrying reinforcing steel remains properly anchored and confined. [2]

2.6 Finite Element Modeling and Analysis of Existing Bridges

2.6.1 Structural Analysis for Performance Evaluation

For economic reasons, it is desirable to keep evaluation effort to a minimum. If the capacity of a bridge can be shown to be sufficient by making some approximations, there is no need to resort to an expensive evaluation procedure. On the other hand, if the sufficiency of a bridge cannot be reliably established using a more approximate method, it can be resorted to a more sophisticated approach in order to demonstrate the sufficiency of the bridge [5]. One of the many cases where refined analysis methods would be considered appropriate is when bridges are low rated and when load rating for permit loads.

As Non-strength related criteria, use of refined analytical methods could significantly influence the repair/rehabilitation strategy or posting load that may be governed by service or fatigue criteria. Evaluation of the fatigue and service limit states is concerned with non-ductile failure modes and service level loads where there is little likelihood of load redistribution. Analytical procedures that underestimate the load effects in some locations and overestimate the effects in others, while acceptable at the strength limit state may result in significant inaccuracies for the fatigue and service limit states. Refined analysis procedures that can properly model the relative stiffness of all bridge components assume added significance when evaluating bridge [2].

2.6.2 Reinforced Concrete (RC) Finite Element modeling

Modeling reinforced concrete in Abaqus is not an easy task, as RC is made of concrete and steel, two materials with different physical and mechanical behavior. Concrete exhibits nonlinear behavior even under low-level loading due to nonlinear material behavior, environmental effects, cracking, biaxial stiffening and strain softening, and time-dependent effects such as creep and shrinkage [6].

Because of the difference in the short- and long-term behavior of constituent materials of RC, the popular method of representing RC consists of developing separate models for concrete and steel and combining those models either at the element level, through the addition of constitutive matrices, or at the structure level, through the use of different elements for each material [6].

For nominal design capacities, an elasto-plastic model is customarily adopted to provide a dependable estimate for design, although in assessment computations which is the exact analysis of existing RC members, a realistic stress–strain model should be applied using expected values of the control parameters [6].

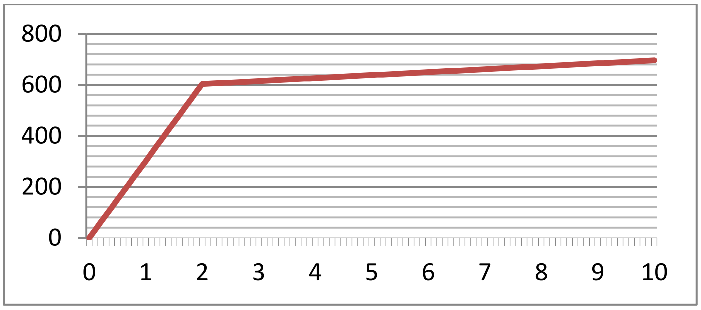


Figure 2. 5: Idealized stress-strain curve for reinforcing steel

2.6.3 Concrete Damaged Plasticity Model (CDP) in Abaqus

The non-linear behavior of reinforced concrete is very important in the analysis of structures performance, especially in predicting the ultimate load carrying capacity of structures. And with the development of computer capability, nonlinear FEA can be applied to large RC structures.

CDP assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material. The evolution of the yield (or failure) surface is controlled by two hardening variables, linked to failure mechanisms under tension and compression loading, respectively. We refer to as tensile and compressive equivalent plastic strains, respectively. The following sections discuss the main assumptions about the mechanical behavior of concrete [11].

The model assumes that the uniaxial tensile and compressive response of concrete is characterized by damaged plasticity. The isotropic damaged elasticity and the isotropic tensile and compressive plasticity were used in the concrete damaged plasticity model to study the behavior of concrete in a non-elastic manner [11].

$$\begin{aligned} \varepsilon &= \varepsilon el + \varepsilon pl \\ \sigma &= Del : (\varepsilon - \varepsilon pl) \\ \sigma' &= D_0 el ; (\varepsilon - \varepsilon pl) \\ Del &= (1 - d) D_0 el \end{aligned} \quad \text{…………………………………………………………… (Eq. 2.5)}$$

$$d_c = 1 - \frac{\sigma_c}{\sigma_{cu}} \quad \text{…………………………………………………………… (Eq. 2.6)}$$

$$d_t = 1 - \frac{\sigma_t}{\sigma_{tu}} \quad \text{…………………………………………………………… (Eq. 2.5)}$$

Where d_t and d_c were two scalar damage variables, ranging from 0 (undamaged) to 1 (fully damaged). The damage model used for concrete was based on plasticity and considered the failure process of tensile cracking and compressive crushing.

In addition to uniaxial compression and tension curves and damage parameters, the other plasticity parameters in Abaqus are *dilation angle; eccentricity, fb0/fc0, K, and viscosity parameter* are needed to fully define the concrete model.

2.7 Bridge Inspection for performance Evaluation

The quality and the availability of data will have a direct influence on the accuracy and reliability of the load rating results. So, before load rating a bridge current condition and loading data for the bridge will have to be collected. Where certain data is unavailable or unknown provides guidance on arriving at suitable estimated values [2].

There are also some cases where judgment must be exercised in making an evaluation of a structure and the condition factors and safety criteria may be adjusted based on site conditions and structure conditions, or both, as recorded in the most recent inspection report [2].

The following important items of data, table 2.1, required for load rating should be obtained from field inspection and from available bridge records. Where feasible, all important plan data used should be verified in the field at the time of inspection.

Table 2. 1: Format of Important data that must be obtained through bridge inspection [2]

Geometric Data	**Member and Condition Data**	**Loading and Traffic Data**
▪ Span length/member lengths ▪ Support conditions/ continuity /overhangs ▪ Bridge skew at each bearing ▪ Girder/truss/floor beam spacing ▪ Roadway, traffic lane, and sidewalk widths	▪ Member types and actual member sizes ▪ Material grade and specifications ▪ Reinforcing/pre-stressing/post-tensioning data ▪ Material losses due to deterioration ▪ Condition ratings/flagged conditions ▪ Presence of fatigue-sensitive details ▪ Presence of fracture-critical members and Connections	▪ Actual wearing surface thickness, if present ▪ Non-structural attachments and utilities ▪ Depth of fill, soil type, and condition (buried ▪ structures) ▪ Number and positioning of traffic lanes on the bridge ▪ Pedestrian traffic intensity ▪ ADTT or traffic volume and composition ▪ Posted load limit, if any ▪ Posted speed limit, if any ▪ Roadway surface conditions at approaches and on bridge ▪ Roadway condition/bumps at deck joints

2.8 Performance Evaluation: Serviceability criteria

Strength is the primary basis for evaluation. *The focus of serviceability checks in evaluation is to identify and control live load effects that could potentially damage the bridge structure, and impair its serviceability and service life*. Serviceability checks are necessary even though the live load may have been determined to be safe at the strength limit state.

Most maintenance problems in existing bridges are service related, for instance *cracking in concrete bridges. If ignored, serviceability problems could ultimately develop into strength and safety problems*. Consequently, serviceability considerations in evaluation are aimed at avoiding or minimizing bridge damage due to live loads by placing *limits on service load stresses under normal use and controlling permanent inelastic deformations under authorized or unauthorized overloads.*

If no past performance problems have been observed or reported, the bridge has in effect passed a proof test for serviceability. The same logic does not apply to heavy permit loads, which may introduce a load greater than that imposed in the past. Service limit states checks are therefore stressed for permit reviews. A SERVICE I limit *check is applied to permit checks and uses limiting criteria of 0.9* f_y *in the extreme tension reinforcement [5].*

During permit load rating, the stresses in the reinforcing bars nearest the extreme tension fiber of the member should not exceed 0.90 of the yield point stress for un-factored loads. This check has been added as the low live load factors possible for permit vehicles operating under controlled crossing conditions (i.e. Escorted with no other vehicles on the bridge), combined with an ultimate strength resistance check could result in the possibility of inelastic stresses in the tensile steel that would reduce the long-term serviceability and durability of a bridge without causing an immediate failure or collapse. Limiting steel stress to 0.9Fy will mean that cracks that develop during the passage of overweight vehicles will close once the vehicle is removed. *It also ensures that there is reserve ductility in the member.*

For concrete members with standard designs and closely clustered reinforcement, the engineer may as an alternate to limiting the steel stress choose to limit un-factored moments to 75 percent of nominal flexural capacity [5].

2.9 Rating Bridges without plans

In Ethiopia, there are large numbers of reinforced concrete bridges, which were constructed in the 20th and are still in service. The legal owner of the bridges, the Ethiopian Roads Authority, cannot provide necessary information of design details, and properties of materials used during the construction of most of those old reinforced concrete bridges.

This is a headache for a practicing engineer that needs to conduct performance assessment/load rating on the bridges since the load bearing capacity of structures depends upon the physical dimensions, properties of construction materials, support conditions, and also their physical condition.

Structural properties for bridges without original design plans cannot be easily be obtained. Particularly for reinforced concrete bridges, the detailing of the reinforcement is crucial in estimating their load carrying capacity. There are also numerous unknown parameters, such as clear cover, size, bar spacing and compressive strength of the concrete, which have to be determined somehow, in order to know the performance of the bridges. This is especially a problem for older concrete bridges if no data is available. As a result, the bridge capacity or rating cannot be evaluated easily using traditional methods, based on only simplified theoretical models.

In case of reinforced concrete slab bridges, it might be possible to use reinforcement detecting devices to determine the amount of reinforcement; diameter of bars, spacing and clear cover. However in case of T-girder and box girder bridges with multiple layer of reinforcement in the girders, it is impossible to know the reinforcement detailing since such devices can only detect one layer of reinforcement and it cover.

In cases like this, which is abundant in Ethiopia, AASHTO strength evaluation manual [2] advises to use Knowledge of the live load used in the original design, the current condition of the structure, and live load history to provide a basis for assigning a safe load carrying capacity. It also advises for assessment engineers to consider non-destructive proof load tests to establish a safe load capacity for such bridge [2].

And also, for bridges where necessary details, such as reinforcement in a concrete bridge, are not available from plans or field measurements, a physical inspection of the bridge by a qualified inspector and evaluation by a qualified engineer may be sufficient to establish an approximate load rating based on rational criteria [2].

Load tests are also recommended establishing the safe load capacity for such structures. It can be deduced from their construction date that most existing bridges were designed by the Allowable Stress method, where designs are carried out at the service load level, and serviceability criteria are implicitly considered [5].

According to a research made in Dakota, USA,[8] such procedure of estimation of reinforcement for a bridge based on field tests, inspections and onsite material strength measurements may be followed and relied up on given that the following conditions are met:

- The bridge has been carrying unrestricted traffic.
- There are no signs of significant distress on the bridge.
- The bridge exhibits proper span-to-depth ratio.
- The construction details should match the specifications current at the time of estimated construction date.
- The appearance of the bridge shows that construction was done by a competent builder [8]

CHAPTER 3 : PERFORMACE EVALUATION OF KOKA BRIDGE

A case study has been conducted on one of Reinforced concrete bridges, i.e. Koka River Bridge. The country has most of its bridges old and deteriorated. Among them is Koka Bridge which is along the Addis – Modjo –Zeway road segment. The bridge lies along one of the heavily overloaded road segments in the country and twelve heavily loaded permit truck loads has passed through this route in the past two years.

Koka Bridge is preferred for this case study because the bridge's deflection has been measured as one of the permit loads passes through the bridge, which can be taken as a good input in to verifying whether if the FEM model prepared in Abaqus/CAE 6.13.1 represents the real behavior of the bridge.

Field Inspection has been conducted on the bridge since assessment of existing structures requires extensive data collection besides reasonable assumptions. This is because outcomes of the assessment are highly dependent on the existing condition of the structure.

Detailed information that describes, structural dimensions, bridge design data, or as-built plan (structural drawings) of Koka Bridge is not available. Methods well accepted and usually practiced by researchers and assessment engineers in other countries to evaluate the strength of bridges without plans, as explained in section 2.11., has been used to put a logical and controlled estimate on the load carrying capacity of the bridge.

The following step, Figure 3.1, is gone through to estimate the safe load carrying capacity of Koka Bridge;

1 Selection of Reinforced Concrete Bridge

2
i. Collecting available data, historical records (Chapter3)
ii. Bridge inspection, measurement and testing (Chapter3)

3 Finite Element modeling of the bridge on a simulation software, Abaqus 6.13-1 (Chapter4)

4
i. Validation of material model (Chapter4)
ii. Verification of global bridge model (Chapter4)

5 Analysis of bridge for maximum effects of Permit and three ERA legal loads (Chapter 5)

6 Bridge load rating computation (Chapter 5)

7 Conclusion and recommendations (Chapter6)

3.1 Description of the Bridge

Koka Bridge, shown in Figure 3.1, referenced as *A7-1-001* by ERA's bridge management System, is a 56.4m Reinforced concrete deck Girder Bridge. The bridge has been constructed by a contractor known as ***Pantelli***.

Figure 3. 1: Koka river bridge

The bridge has been in service since 1953 G.C., which makes it more than 66 years of age, similar to most of the bridges in Ethiopia.

Table 3. 1: Description of Koka River Bridge (ERA code: A7-1-001)

Bridge Location	Modjo section, Modjo - Zeway Road Segment, 91.5 Km from Addis Ababa
Bridge Type	Reinforced Concrete deck girder
Year Of Construction	1953 G.C.
Carriage Way Width	7.4 m
Support Type	Multiple span (Three spans)
Span Composition	18.8+18.7+18.9 (as measured on site , in meters)
Cross Sectional Dimensions	Figure 3.1
Pier and Foundation	RC direct/MAT, RC Abutments

The bridge girders are supported on piers and reinforced concrete abutments through elastomeric bearings as shown in the Figure 3.2 below. The bridge accommodates two traffic lanes.

Figure 3. 2: Bridge support, Elastomeric bearing

3.2 Field Inspection and Testing

Detailed field inspection and testing is a major precedent to performance evaluation of a bridge, Since the rating of an old bridge should be based on a recent thorough field investigation. As Koka Bridge is an old and deteriorated one field inspection is inevitable. The inspection has been conducted with the help of ERA's bridge management department and as per ERA's inspection manual

The following has been done on the field inspection of the bridge:

- Measuring the dimensions of the bridge parts. Dimensions of girders, diaphragms, abutments, span length, road way width, overall width of the bridge and location of diaphragms, bearings, and has been measured.

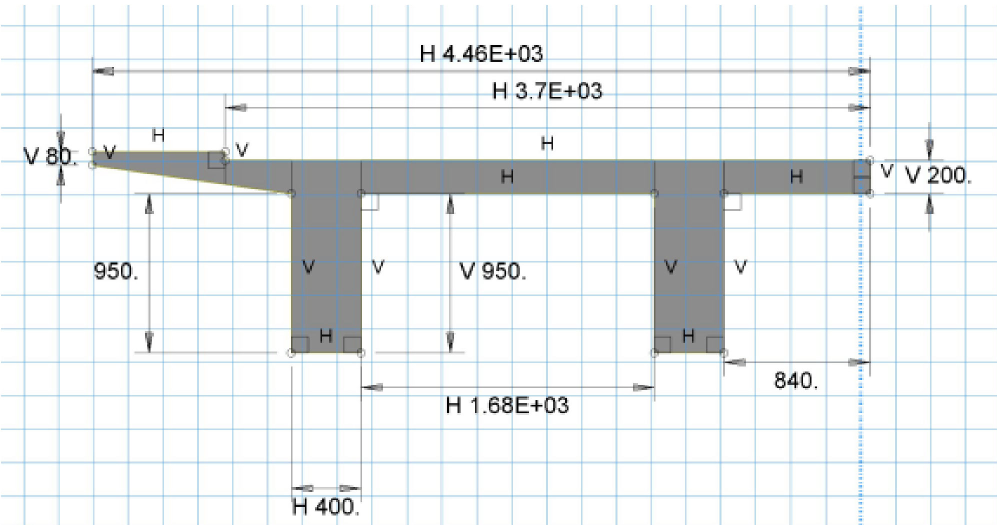


Figure 3. 3: Bridge Cross sectional dimensions; as per field inspection measurement

Figure 3.3 shows a drawing illustrating the bridge's super-structure cross-sectional dimensions.

- Conducting through inspection of the bridge to recognize its existing condition.

Of the damages/defects expected in an old reinforced concrete deck Girder Bridge;

i. Flexural cracks; shear cracks, see Figure 3.4, which principal cause is excessive loading.
ii. Honey comb, which might be due to poor construction,
iii. Abrasion and distortion of expansion joint which created unevenness of the road surface, which cause is clearly due to aging,
iv. Deformation (break failure) of the guard rail due to vehicle collision has been noticed, see Figure 3.5.

For major cracks that can easily be seen by the naked eye, the size of crack openings has been measured with a crack scale, Figure 3.4. Determining the size of the crack openings is important to condition rate the bridge using ERA's inspection manual. And length of continuous cracks has been estimated, since it helps in quantifying the existing damage which is used in maintenance schemes.

Figure 3. 4: Measuring the size of crack openings with crack scale

The bearing, piers and abutments are found in good condition. Although there is a considerable amount of siltation near the abutments, it is not considered contributory to the bridges load carrying capacity.

Figure 3. 5: Damage and Distresses on bridge super-structure.

- Conducting Schmidt hammer test to determine Concrete Compressive Strength. The test has been conducted on each of the three spans of the bridge, on three locations for all i.e. exterior girder, interior girder and deck slab. The Schmidt hammer, Figure 3.6, which is owned by ERA, has been calibrated by AAiT laboratory personnel.

Figure 3. 6: Measurement and Schmidt hammer test on Koka bridge

Fifteen (15) rebound tests are conducted for each location and the average of those values obtained is taken as a rebound value. The compressive strength values of bridge concrete obtained are as shown in Table 3.2.

Table 3. 2: Schmidt hammer test results

It. No	Location	Compressive strength of concrete in MPa		
		Addis Ababa's side	Middle span	Zeway's side
1	Exterior girder	52	59.6	52
2	Interior Girder	46.5	53.1	47.5
3	Deck Slab	50	49	51

The values in Table 3.2 have been disregarded. They are found in an unacceptable range since they are found significantly higher than the expected value. As, it is rare to achieve such high level of compressive strength by the time the bridge is constructed. In such cases, a core test would be preferable, if the sampling and testing equipment is available.

3.3 Estimation of Bridge Reinforcement

As mentioned above as-built plan (structural drawings) or design document which is needed to compute load carrying capacity of Koka Bridge is not available.

Despite the circumstances, as briefly explained in section 2.12 engineering judgment can be used, along with;

- Historical records,
- method of design used by the time of construction of the bridge,
- field inspection and measurement and
- plans of similarly constructed bridges (if available) to arrive at plausible

Reinforcing details; even if, these estimates tend to be conservative.

Since there is also no material data such as the concrete grade and steel grade values corresponding to the year of construction of the bridge, as provided in ERA design manual part II section 5, has been used in subsequent computations.

The ***STANDARD SPECIFICATIONS for HIGHWAY BRIDGES, Adopted by The American Association of State Highway Officials (AASHTO) Fifth Edition*** is referenced in the computations made to estimate amount of bridge reinforcement. The standard specification has been published in **1949 G.C**, which is before the construction date of the bridge, 1953 GC.

ERA design manual, Part II, section 5 states that in cases where reinforcement details are not known, the area of tension steel to be used in computing the ultimate flexural strength of reinforced concrete members shall not exceed that available in the section; or 75 percent of the steel reinforcement required for a balanced condition. This has been taken in to account while computing the reinforcement area for the bridge.

3.4 Reinforcement computation (Bridge Superstructure)

Standard Specifications For Highway Bridges, Adopted by The American Association of State Highway Officials (AASHTO) FIFTH EDITION, 1949 is used as a reference in the computations made in this section. The Design Philosophy of the code **is Allowable Stress (Working stress) Method.**

3.4.1 Material Properties

- **Concrete**: As per [8] the strength of sound concrete shall be assumed to be equal to either the values taken from the plans and specifications or the average of construction test values. When these values are not available, the ultimate stress of sound concrete shall be assumed to be 25 MPa. A compressive strength of f_c' equal to 25 MPa has been used.
- **Reinforcing steel:** ERA design Manual [8] recommends taking the yield strength of unknown steel prior to 1954 as 228 MPa.
- **Allowable Design Strength:** Allowable stresses are specified in section 3.4.11 and 3.4.12 of [10]

f_c (allowable) = 1/3 (fc') = 8.33 MPa, concrete

f_S (allowable) = 124 MPa, reinforcement

3.4.2 Design Loads

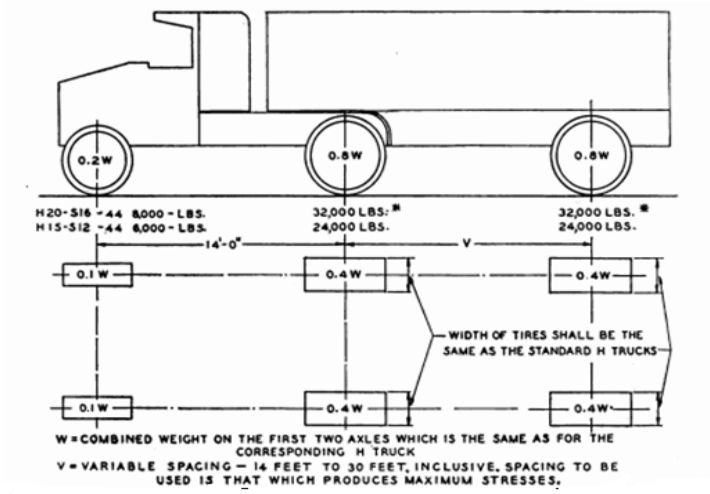
Structures are proportioned and designed for the following loads and forces; dead load, live load, Impact or dynamic effect of the live load, wind loads and other forces when they exist.

- **Dead Load**: Self-Weight of Materials as recommended in ERA's design manual part II, section 5 is shown in Table 3.3:

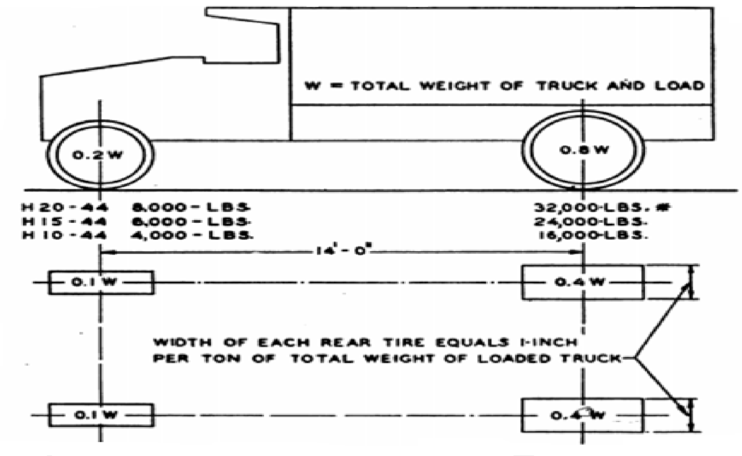
Table 3. 3: Self-Weight of Materials, ERA design manual, section 5

Material	Force Effect[KN/m^3]
Asphalt surfacing	22.5
Concrete, plain or reinforced (normal weight)	24
Steel	79

- **Live Load:** The live load consists of the weight of the applied moving load of vehicles. Standard design H and H-S vehicles configuration is as shown in Figure 3.8.



H Truck



HS Truck

Figure 3. 7: Live load configuration, H and HS trucks

- **Design Lane Width:** Where the spacing of main supporting members exceeds 10.5 feet (3.2 m) for concrete bridges, the lane loading or standard trucks shall be assumed to occupy a width of 10 feet. These loads shall be placed in design traffic lanes having a width of

$$W = \frac{Wc}{N} \quad \text{(Eq. A1)}$$

W=3700mm, is taken as the design lane width.

- **Span Length:** the effective span length that shall be used in calculating distribution and moment of slabs continuous over more than two supports, which are also monolithic with the beam is the clear span between the girders. Therefore;

S= 1.680m (for deck slab)

For simple spans the span length shall be the distance center to center of supports but not to exceed clear span plus thickness of slab. Therefore:

S= 18.8 m (for Girders)

- **Impact load allowance:** the amount of this allowance or increment is expressed as a fraction of live load stress, and shall be determined by the formula:

$$I = \frac{50}{L + 125} \quad \text{(Eq. A2)}$$

Based on this computation, I = 29.8 % is used to amplify live load action effects.

3.4.3 Deck slab reinforcement

The deck slab is designed as 1000mm strip continuous beam continuously supported on the girders. The dead load incorporates self-weight of the deck slab, 50mm of Asphalt layer and End barrier.

Table 3. 4: Self-weight (Dead Load) and Vehicular load (Live load) on the bridge

<table>
<tr><th colspan="2">Dead Load</th></tr>
<tr><td rowspan="2">Deck slab</td><td>4.8 KN/m (roadway)</td></tr>
<tr><td>4.51 KN/m (curb)</td></tr>
<tr><td>Asphalt Layer</td><td>1.125 KN/m (50mm Asphalt layer)</td></tr>
<tr><td>End Barrier</td><td>4.28 KN/m</td></tr>
<tr><th colspan="2">Live load</th></tr>
<tr><td colspan="2">It is also noted that, in the design for H-20 or H20-S16 loads, the single 24,000 pound axle governs for spans under 10.5 feet (3.2m).</td></tr>
</table>

As a result the maximum negative design bending moment at the supports of the slab (near the girders) and maximum positive design bending moment are computed as follows.

- **Dead Load Moment:**

The resulting Maximum total bending moment is the following:

M_D =7809.8KN-mm, negative moment at the support around the interior girders

M_D =2993 KN-mm, positive moment at the middle deck span

- **Live Load Moment:**

Maximum moments per foot width of the slab, due to the proposed H and HS design truck loads shall be calculated by Equation A3, as per the design code. For slabs supported transversally (if the main reinforcement is perpendicular to traffic) the following formula is applied.

For span Length of 2 to 7 ft. (0.6 to 2.1m):-

$$M = \pm 0.2 \frac{P1}{E} s$$
$$where, E = 0.6S + 2.5 \quad \text{.....................} \quad \text{(Eq.A3)}$$

E= 5807.25 mm, which is the width of slab over which a wheel load is distributed

S= 1680mm, which is effective span length

P1 =12000 lb, which is the load on one wheel of governing single axle

Based on the above criterions bending moment per meter width of the slab is calculated to be:

M= +/- 10,292.0KN-mm

- **Total Moment**

The resulting Maximum total bending moment is the following:

M_D = 13,292.1 KN-mm, positive moment at the middle deck span
M_D =22,745.4 KN-mm, negative moment at the support around the interior girders

Computation of reinforcement area is as per the stress diagram show in Figure 3.8

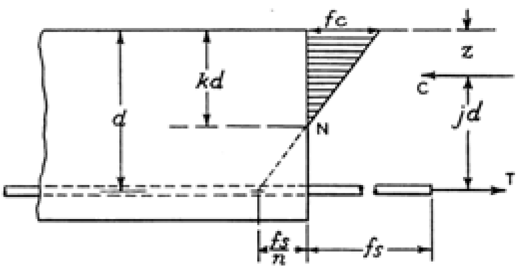


Figure 3. 8: Stress diagram, Allowable Stress (Working stress) Method

Position of neutral axis

$$k = \sqrt{2\rho n + (\rho n)2} - \rho n \text{ .... (Eq. A4)}$$

Where p, the Steel ratio for balanced reinforcement is computed by

$$\rho = \frac{1}{2}\frac{1}{\frac{fs}{fc}(\frac{fs}{nfc}+1)} \text{ .... (Eq. A5)}$$

And the arm of the resisting couple is,

$$j = 1 - \frac{k}{3} \text{ .... (Eq. A6)}$$

Then compressive unit stress in extreme fiber of concrete is calculated by this formula

$$fc = \frac{2M}{jkbd2} = \frac{2\rho\ fs}{k} \text{ .... (Eq. A7)}$$

Reinforcing steel area for tension is equal to:

$$fs = \frac{M}{Asjd} = \frac{M}{\rho\ jbd2} \text{ .... (Eq. A8)}$$

Reinforcing steel area calculated using Equations A1 to A5 is as follows:

As = 1190.377 mm^2 per meter width of the slab, bottom reinforcement

As = 1479.12 mm^2 per meter width of the slab, top reinforcement

3.4.4 Girder Reinforcement

- **Interior Girder**

Maximum reaction force due to dead load from the slab, self-weight of the girder and bridge Diaphragms are loaded on the exterior girder described in Table 3.5.

The live load arrangement that gives the maximum shear force and bending moment, H20-S16-44, as recommended by the code.

Table 3. 5: Dead and live loading for interior girders

Dead Load	
Self-weight of web	9.12 KN/m
Dead load reaction force from Deck slab	9.06 KN/m (50mm Asphalt layer)
Diaphragm (middle)	7.68 KN
Diaphragm (End)	4.8 KN
Live load	
H20-S16 loads truck load gives the maximum action effect in for both shear and bending moment as referred from Appendix II the code ; for the specific span length	

Total Moment

M_D =850,790 KN-m; maximum Dead load moment

M_{LL} =3,320.79 KN-m; maximum live load moment

M_{Total} = 4,172 KN-m; maximum Total Moment

V_D = 271.34 KN; maximum Dead load shear

V_{LL}= 178.572 KN; maximum Dead load shear

- **Exterior Girder**

Maximum reaction force due to dead load from the slab, self-weight of the girder and bridge Diaphragms are loaded on the exterior girder shown in table 3.6.

Table 3. 6: Dead and live loading for Exterior girders

Dead Load (exterior)	
Self-weight of web	9.12 KN/m
Dead load reaction force from Deck slab	9.06 KN/m (50mm Asphalt layer)
Diaphragm (middle)	3.84 KN
Diaphragm (End)	2.4 KN
Live load	
From Appendix II of the code **H20-S16** loads truck load gives the maximum action effect for both shear and bending moment, for the specific span length	

Total Moment

M_D = 1258.966 KN-m; maximum Dead load moment

M_D = 148.92 KN-m; maximum live load moment

M_{total} = 1,406.5 KN-m; maximum Moment

V_D = 271.34 KN; maximum Dead load shear

V_{LL}= 266.758 KN; maximum Dead load shear

- **Effective Flange width of T-Beams**

In beam and slab construction, effective and adequate bond and shear resistance shall be provided at the junction of the beam and slab. The slab may then be considered an integral part of the beam, but it's assumed effective width as a T-beam flange (for interior girder) shall not exceed the following:

- One-fourth of the span length of the beam

- The distance center to center of beams.
- Twelve times the least thickness of the slab plus the width of the girder stem.

Therefore, b=2080 mm (Interior Girder)

For beams having a flange on one side only, Exterior Girders, the effective overhanging flange width shall not exceed

- one-twelfth of the span length of the beam, nor
- six times the thickness of the slab, nor
- One-half the clear distance to the next beam.

Therefore, b=840 mm (Exterior Girder)

- **Distribution of load to Girders**

The code specifies that Interior stringers/girders shall be designed for a fraction of wheel load which is, $^{s}/_{5.0}$; where S is the average spacing of stringers in feet:

So, fraction of wheel load to each girder is calculated as 0.58 (58% of wheel load near the girder).

Besides, the live load supported by *outside stringers/girders* is the reaction of the truck wheels, assuming the flooring to act as a simple beam between stringers

The following formulas, based on the stress distribution shown in Figure 3.9, take into account the compression in the stem. They are recommended where the flange is small compared with the stem, which is the case in the current cross section. The Neutral axis depth is assumed to lie below the flange depth, which has been checked and found to be correct later.

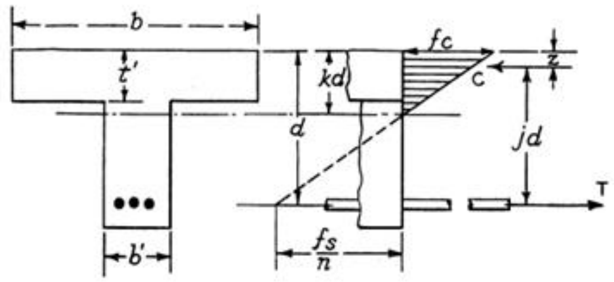


Figure 3. 9: Stress diagram, girder T-beam

Position of neutral axis,

$$kd = \sqrt{\frac{2ndAs + (b - b')t2}{b'} + (\frac{nAs + (b - b')t2}{b'})2} - \frac{nAs + (b - b')t}{b'} \quad \text{(Eq. B1)}$$

Position of resultant compression

$$z = \frac{(kdt2 - \frac{2}{3}t3)b + ((kd - t)2(t + \frac{1}{3}(kd - t)))b'}{t(2kd - t)b + (kd - t)2b'} \quad \text{(Eq. B2)}$$

Arm of resisting couple

$$jd = d - z \quad \text{(Eq. B3)}$$

Compressive unit stress in the extreme fiber of concrete

$$fc = \frac{2Mkd}{((2kd - t)bt + (kd - t)2b')jd} \quad \text{(Eq. B4)}$$

Reinforcing steel Area

$$fs = \frac{M}{Asjd} \quad \text{(Eq. B5)}$$

Using the formulas form (B1 to B5), the amount of reinforcing steel area calculated is as follows:

Reinforcing steel Area

Exterior Girder	**Interior Girder**
$As=10,566.64\ mm^2$	$As=7160.217mm^2$

- **Shear Reinforcement**

As recommended by the code, the flange shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

Shearing unit stress

$$v = \frac{V}{bjd} \quad \text{(Eq. C1)}$$

Stress in vertical web reinforcement

$$fv = \frac{V's}{A_v jd} \quad \text{(Eq. C2)}$$

When the web reinforcement consists of bars bent up in a single plane so as to reinforce all sections of the beam which require it, the bent-up bars shall be designed in accordance with the formula:

$$Av = \frac{V'}{fv \sin a} \quad \text{(Eq. C3)}$$

- **Distribution reinforcement**

Reinforcement shall be placed in the bottom of all slabs normal to the main steel to provide for lateral distribution of the loads. The amount shall be the percentage of the main reinforcing steel required for positive moment as given by the following formula:

Percentage = Max $\left(\frac{100}{\sqrt{S}}, 50\%\right)$

Where: S= effective span of the slab in feet

Therefore, percentage of distribution reinforcement is equal to 41.38 % , which **496.57 mm²** distribution reinforcement per meter width of slab has been provided in the longitudinal direction.

- **Curb Reinforcement**

Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot of curb, applied at the top of the curb, but at a point not over 10 inches (254mm) above the floor.

Dead load moment just before the exterior girder (support) is M_{DL}= 10.784 KN-m

M_{LL}= 1,853,425.67, applied at 2.54 cm from the deck slab

The reinforcement ratio, ρ=0.00095, which is less than the minimum

Therefore, ρ_{min} = 0.0021 is used in to compute the amount of reinforcement, and

$A_{s,\,curb}$ = 1558.2 mm^2 per meter length of the curb.

Although it is impossible to identify the exact arrangement of reinforcement, the area of reinforcement with in the bridge can be estimated from a design code which has been in use by the time of construction of the bridge as shown in this section.

3.5 Permit load

The construction of Wolayta - Sodo sub-station is the major ongoing project in the southern Ethiopia. Seimence international is responsible for the design and transport of fourteen (14) transformers to be installed in the project. Seven of the designed transformers weigh 172 tons and the remaining seven weigh 190tons.

Based on a the condition assessment and strength evaluation conducted for all of the bridges along the mentioned road segments, Ethiopian Roads Authority permitted the transportation of the 14 transformers along Djibouti – Adama – Sodo road segment.

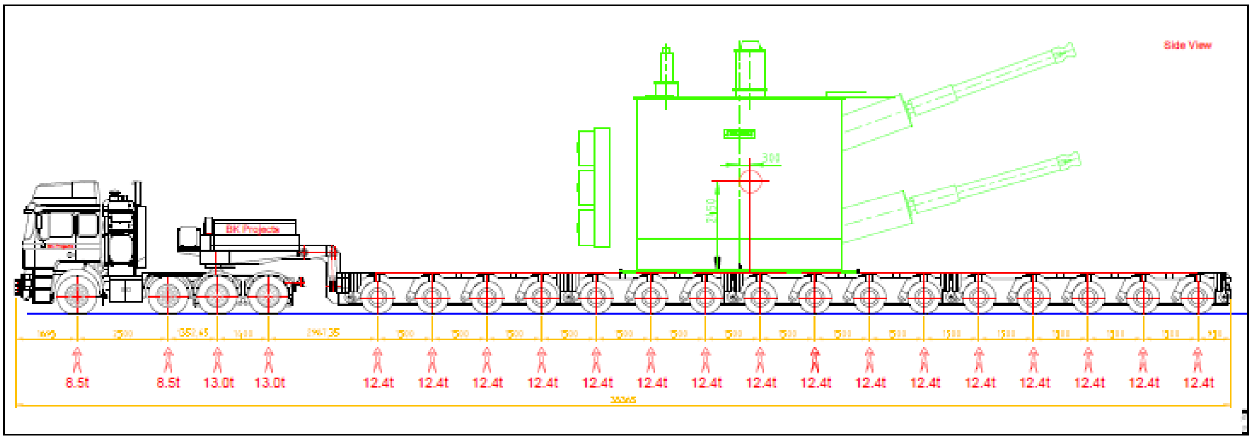


Figure 3.10: 16 axle trailer truck, for 170 ton transformers

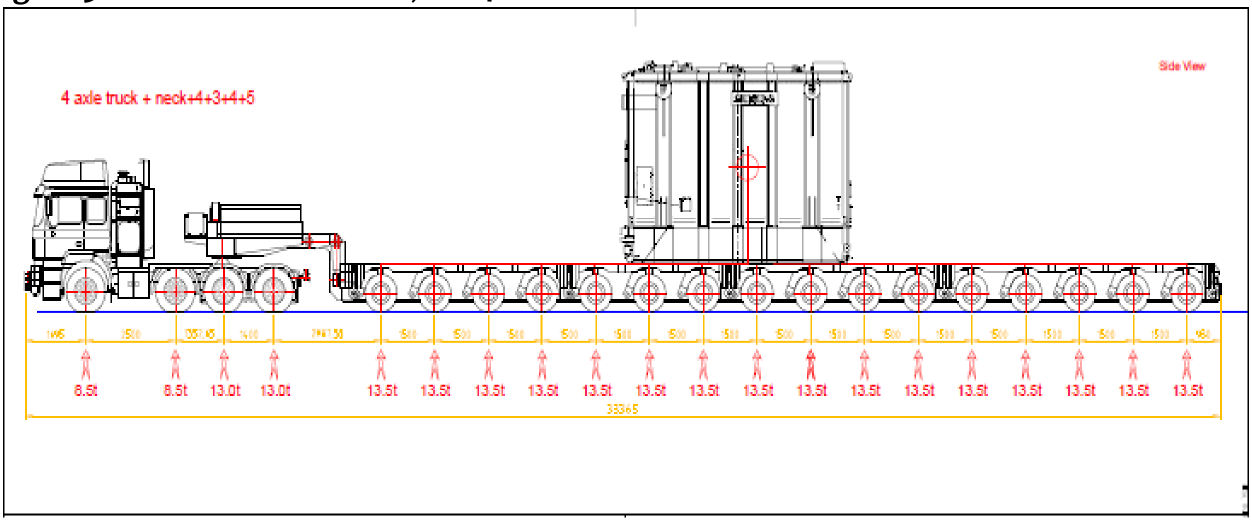


Figure 3. 11: 18 axle trailer truck, for 190 ton transformers

It was recommended to transport the 172 ton transformers with 16 axel trailer truck as shown in Figure 3.10 whereas for 190 ton transformers, 18 axel trailer trucks as shown in Figure 3.11 were recommended for the transportation.

Temporary strengthening of the bridge has been recommended based on the analysis made on SAP 2000. It was recommended to install steel plates of 10-13mm thickness at the support, to temporarily strengthen the bridge.

The strengthening was recommended because, the shear capacity of the bridge was found to be less than the effect of the permit truck, which is the bridge has load rating less than one (1) as calculated for carrying capacity in shear at the supports. Therefore, a steel plate of 12 mm thickness has been placed at the supports, location of piers and also joint of the approach road and the bridge abutment, in order to acquire better distribution of load to the girders. The strengthening plates have been included in the Abaqus FE model.

As mentioned earlier, the mid-span deflection of one of the spans of the bridge has been measured on the passage of one of the 16 axle trailer permit trucks on the bridge. A gauge has been installed at the bottom of the bridge girder as shown in Figure 3.12 to measure the deflection as the permit load passes through the bridge.
The strain gauge is calibrated to measure 1 mm of deflection as it rotates one full cycle. The deflection measured with time passage is as shown in Figure 3.12 below. The maximum deflection measured is 18 mm.

Figure 3.12: Deflection measuring gauge, installed at the mid of the first span of Koka river bridge

CHAPTER 4 FINITE ELMENT MODELING AND ANALYSIS

This Chapter attempts to develop an approach to evaluation of load carrying capacity of reinforced concrete bridges without plans. The technique used combines advantages of the field inspection and testing also three-dimensional non-linear Finite Elements Analysis using the commercial simulation software package, Abaqus/CAE 6.13-1.

4.1 Bridge Finite Element Model

In this section the steps taken to establish the three dimensional Finite Element model for the selected bridge are outlined. Field inspection and measurement data has been used to establish the model. The outputs obtained from analysis made on the FE model are later used in load rating calculations and verification of the global bridge model.

As introduced in chapter three, an existing bridge is used as a case study to introduce the procedure of implementing the Finite elements simulation in load rating reinforced concrete bridges without plans.

Since Abaqus does not have inherent system of measurement units, the numbers entered by the user need to have a consistent unit throughout the modeling of geometry, material definitions, and loading of the model. The basic measuring units used in the current model are N, mm, and Sec.

The following Steps are taken in creating Bridge FE model in Abaqus:

1. Creating constituent parts of the bridge super-structure
2. Defining Material Properties and sectional properties
3. Creating constituent parts of the bridge super-structure
4. Assigning sectional properties to constituent parts
5. Assembling different parts and merging some of the parts of the bridge
6. Adding boundary conditions
7. Defining Loads and Load Amplitudes
8. Requesting output requests and submitting the Model for analysis
9. Post processing of analysis, visualization of results

Figure 4. 1: Steps taken in developing the Finite ElementFE model

4.1.1 Model characteristics (Assumptions)

Certain assumptions were made to simplify model development with negligible loss in the accuracy of the representation.

- The deck gradient, provided for the purpose of good drainage, was neglected in the model. This is done so since the (1.4-2.5) % of deck gradient is small enough to be neglected.
- An average deck thickness of 200 mm was used, disregarding chamfers and rounds.
- The bridge railing, which is an external and non- load bearing component of the bridge has not been modeled with the deck.
- Since the model and the loading conditions are symmetrical about vertical plane through the longitudinal direction of the bridge, half of the bridge is assumed to represent the whole of the bridge. This is done so reduce the computation time for analysis.

With these assumptions in mind, the components of the bridge were represented by the finite element strategies as explained in sections that follow, which also shows steps taken to create an effective bridge model in Abaqus.

4.1.2 Modeling Bridge Geometry

The geometry of the bridge has been modeled with constituent parts with different properties, as stated in Table 4.1, positioned relative to each other in a global coordinate system and assembled at a later stage to create the desired bridge super-structure.

Table 4. 1: Constituent parts of bridge model and their property definition

It. No.	Constituent Part	Type	Section Type
1	Bridge and deck slab combination	Concrete	Solid, Homogenous
2	Diaphragm (at the middle)		
3	Diaphragm (at the support)		
4	Longitudinal reinforcement	Reinforcement	Wire, constant cross section
5	Transverse reinforcement		
6	Stirrup		
7	curb reinforcement		
8	Virtual tires	Rigid cylinder	Solid, Homogenous
9	Strengthening steel plate	Steel	

Figure 4.2 illustrates the model of the complete span and the bridge reinforcement layout.

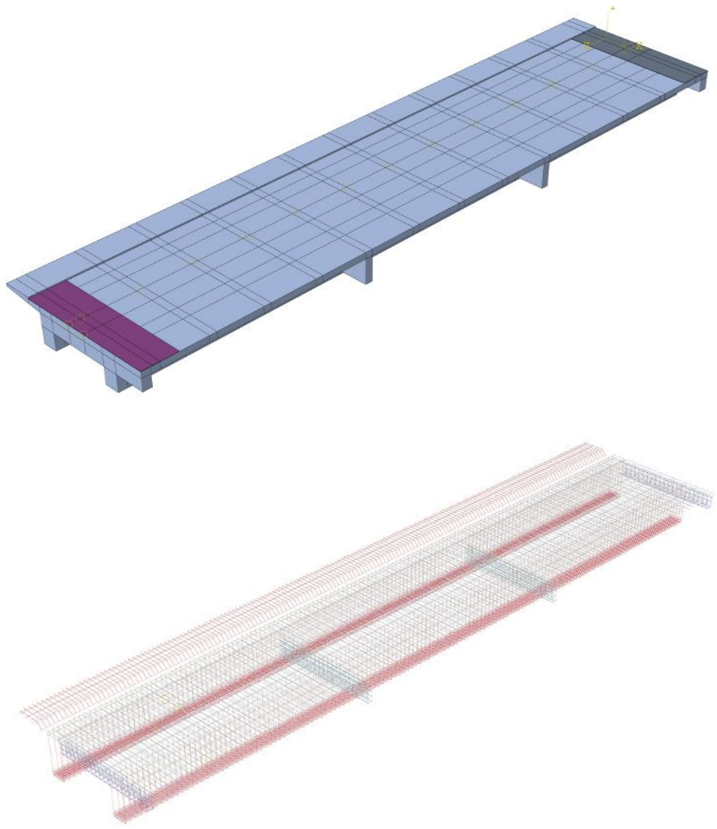


Figure 4.2: Constituent parts of bridge model and Bridge Reinforcement layout

4.1.3 Material Properties

Parts have been assigned a section property, in which the material property and cross section type (for wire elements) is decided. The properties of the materials are as shown in Tables 4.2 and 4.3

Table 4. 2: Material properties [8]

Material Property	Concrete	Steel
Elastic Modulus	28.96 GPa	200Gpa
Poisson's ratio	0.2	0.3
Density	2400 Kg/m^3	7800 Kg/m^3
Plasticity model	Concrete Damaged Plasticity	

- **Material Model for concrete**

Concrete Damaged Plasticity model, which is described in section 2.7.3, is used to model the elastic and non-linear properties of concrete; it is one of the two models Abaqus's materials library offers to model concrete.

Stress-strain relation for non-linear structural analysis has been formulated based on EBCS design Manual, section 3.1.5. The relation between σ_c and ε_c (compressive stress and shortening Strain) for short term uniaxial loading is described by Equation 4.1:

$$\frac{\sigma c}{fcm} = \frac{k\eta - \eta 2}{1 = (k - 2)\eta}$$

$where:$

$$\eta = {}^{\varepsilon c}\!/_{\varepsilon c1}$$

$\varepsilon c1$ is the strain at peak stress

$$k = \frac{1.05Ecm\left(\varepsilon c1\right)}{fcm} \quad \text{..................Eq. 4.1}$$

Tensile strength value has been used based on a formula form EBCS 2.

Table 4. 3: Material property input for Concrete CDP model in Abaqus

Material parameter	f_{cu}= 25 MPa	Plasticity Parameters	
		Dilation angle	33
		Eccentricity	0.1
Concrete Elasticity		f_{bo} / f_{co}	1.16
E(GPa)	28.96	K	0.666
ν(Poisons Ratio)	0.2	Viscosity	0
Concrete Compressive Behavior		**Concrete Compressive Damage**	
Stress (MPa)	**Inelastic Strain**	**Damage Parameter (d_c)**	**Inelastic Strain**
11.99	0.0	0.0	0.0
19.12	0.0002	0.0	0.0002
23.22	0.0004	0.0000	0.0004
24.89	0.0008	0.0000	0.0008
24.62	0.0012	0.0000	0.0012
22.75	0.0017	0.0760	0.0017
19.55	0.0023	0.2059	0.0023
15.24	0.0029	0.3809	0.0029
9.99	0.0035	0.5941	0.0035
3.94	0.0042	0.8399	0.0042
Concrete Compressive Behavior		**Concrete Compressive Damage**	
Yield Stress (MPa)	**Cracking Strain**	**Damage Parameter (d_c)**	**Inelastic Strain**
1.96	0	0	0
1.600666667	0.0001	0.183333333	0.0001
1.225	0.0003	0.375	0.0003
1.0535	0.0004	0.4625	0.0004
0.882	0.0005	0.55	0.0005
0.59045	0.0008	0.69875	0.0008
0.395266667	0.001	0.798333333	0.001

4.1.4 Meshing

For meshing to be better controlled while meshing with Abaqus's automatic mesh generator, a partitioning tool has been used to divide (partition) the model into separate solid parts.

In order to choose an appropriate mesh size, convergence studies done on similar Finite element analysis of bridge were reviewed and an appropriate mesh size of 200mm was chosen. The concrete component is modeled with C3D8 elements while all of the reinforcement is meshed with T3D2 elements.

4.1.5 Boundary Conditions and Interactions

➢ **Concrete-Reinforcement Interaction**

The embedded element technique, which is a constraint type in Abaqus, is used to specify that the elements of bridge reinforcement are embedded in "host" elements, which are the bridge concrete elements. This constraint is specially designed to model rebar reinforcement. If a node of an embedded element lies within a host element, the translational degrees of freedom at the node are eliminated and the node becomes an "embedded node."

➢ **Boundary Conditions**

A displacement/rotation boundary conditions are set, in the movement of the nodes that belong to the surfaces around bearing and bridge girder connection.

In order to simulate a simply supported condition the following BC's are applied to the degrees of freedom of the nodes in around bearing and girder connection;

- BC 1 : constrains U3 and U2 to zero, (Right end of the bridge)
- BC 2: constrains U2 to zero, (Left end of the bridge)

Since the bridge model and its loading are symmetric with respect to its longitudinal axis, half of the model was used in the analysis and symmetric boundary condition has been applied. This is to help reduce the computational time in analysis. Figure 4.3 shows the plane in which the plane symmetry boundary condition is applied.

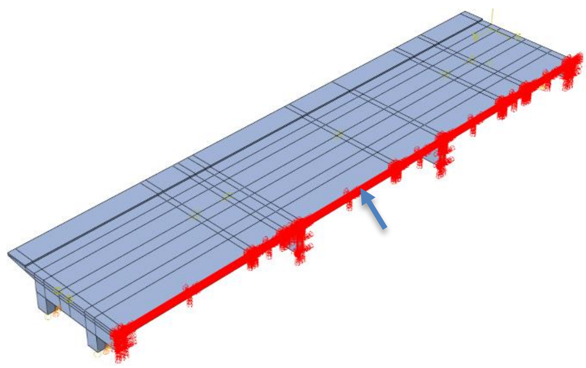


Figure 4.3: Symmetry Boundary Condition

The displacement component normal to the surface (**U_1**) and rotational degrees of freedom (**UR2 and UR3)** are set to zero, for the nodes on the symmetry plane.

4.1.6 Loading

All of the load types recommended to be considered by AASHTO bridge evaluation Manual [2], for the load rating/ strength evaluation of a bridge are considered in the simulation of the bridge. The applied loads consist of the following:

Vehicular Load

- Load transferred from virtual moving tires
- Pressure loads applied to partitioned surfaces of bridge deck, as shown in the Figure 4.4.
- Impact load, taken in to account by multiplying vehicular loads by an impact factor.

Dead Load

- Body Force; concrete, reinforcement, strengthening plate
- Asphalt Layer load
- End Barrier Load

The tire footprint dimensions in which the truck loads lie are calculated from ERA's bridge design manuals' recommendation. The calculated dimensions of each tire patch Table 4.4.

Table 4. 4: Dimensions for tire contact area used in analysis

It. No.	Load	a (mm)	b (mm)	a=width of tire(transversally) b=length of tire(longitudinally)
1	Permit load	500	170	
2	Legal Loads	370	235	

The tire footprint positions are for the permit load as shown in Figure 4.4.

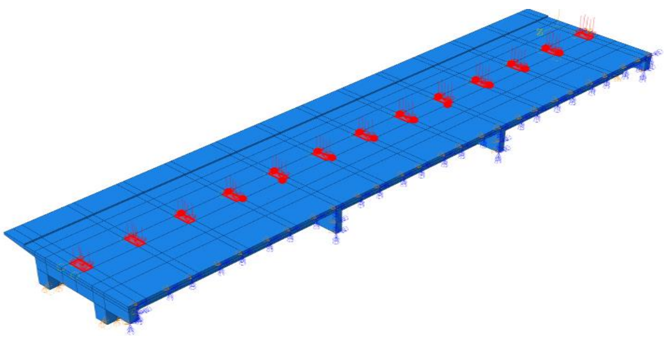


Figure 4. 2: Tire footprint positions for the permit load

4.1.7 Modeling a moving Tire load

The tires used to transfer a moving load on the bridge deck are virtual tires (i.e. do not correspond to actual vehicle tire material properties). As shown in the Figure 4.3 they have cylindrical shape of width equal to the width the actual tire load would lie on. To keep out of the extra complexity of modeling movement of a deforming body in Abaqus, the tires were made rigid enough not to have considerable deformation while moving on the bridge deck.

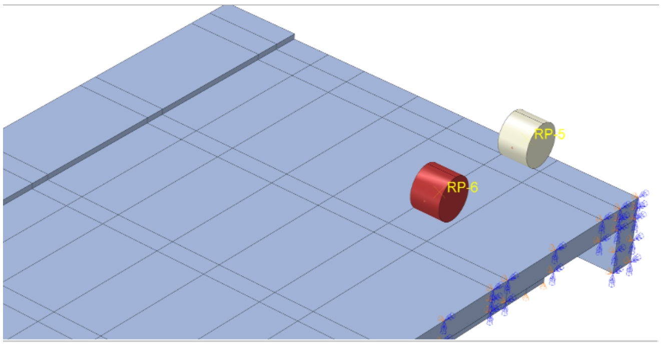


Figure 4. 3: Moving virtual tires on bridge deck

A kinematic coupling constraint is used to prescribe a translational motion of the virtual tires. Coupling constraint is of type kinematic when the group of nodes is coupled to the rigid body motion defined by the reference node. So, in order to apply a translational motion to the tires, kinematic coupling constraint is applied. Each tire's motion is constrained with its reference point (RP as shown in Figure 4.3), where loads and boundary conditions are assigned to. The reference points lie at the geometric center of the tires.

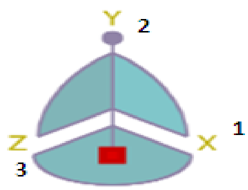
- **Bridge - Tire contact definition**

To successfully transmit a moving load to the bridge, contact/bridge-deck/ interaction has to be defined properly. A surface-to-surface contact, with hard contact in the normal direction is defined between the bridge deck and virtual tire surfaces.

- **Boundary conditions and Interactions**

To keep the virtual tires moving, while applying load on the bridge deck the following boundary conditions has been set on the reference points coupled with the virtual tires.

Table 4. 5: Boundary conditions on moving wheels

It. No	Analysis Step	Boundary condition	Load amplitude	
1	Initial step	U1=U3=0 & UR1=UR2=UR3=0	No load	
2	One	U1=U3=0 & UR1=UR2=UR3=0	Ramped	
3	Two	U1=0 & UR1= UR2 = UR3 =0 V3=50[mm/sec]	constant	

Analysis steps loading and load amplitudes are shown in Table 4.76

Table 4. 6: Analysis steps loading and amplitudes

<table>
<tr><th colspan="2">Loading</th><th colspan="2">Analysis Step</th></tr>
<tr><th rowspan="2">Loading</th><th rowspan="2">Type</th><th>One</th><th>Two</th></tr>
<tr><th colspan="2">Amplitude</th></tr>
<tr><td rowspan="3">Self-weight</td><td>Body Force, concrete, reinforcement, strengthening plates</td><td rowspan="3">Ramped</td><td rowspan="3">Inactive</td></tr>
<tr><td>Pressure Load, barrier load</td></tr>
<tr><td>Pressure Load, Asphalt Layer</td></tr>
<tr><td>Vehicular load</td><td>Permit truck load/ legal load</td><td>Inactive</td><td>Ramped</td></tr>
</table>

4.2 Validation of Material model

As specified earlier, Abaqus/CAE 6.13.1 is used for the bridge simulation and load rating. Various methods of analysis, ranging from simple formulae to detailed finite element procedures, are implemented in programs like Abaqus. As many other computer programs, Abaqus have specific engineering assumptions embedded in its code, which may or may not be applicable to each specific case. Accordingly, all output should be verified to the extent possible. And computer programs should be verified against the results of universally accepted closed-form solutions, other previously verified computer programs, or physical testing [2]. Here, results obtained from Abaqus, has been verified against the results of a Laboratory physical testing of RC beam.

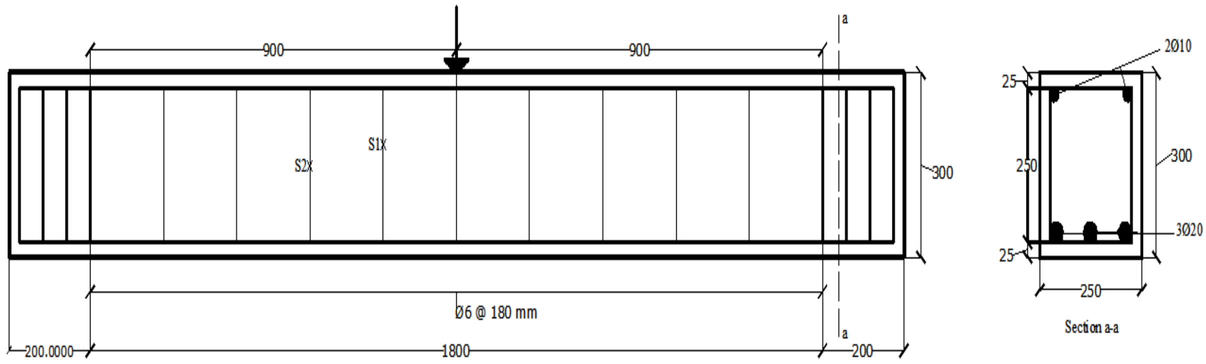


Figure 4.4: Dimensions and reinforcement details of reinforced concrete beam used for validation

4.2.1 Abaqus FEM model: beam

- **Material properties**

Table 4.7 shows the material properties of concrete and reinforcing steel used in the model. Concrete damaged plasticity model is used for the concrete. And the idealized stress strain curve for reinforcing steel was generated, as per EBCS Fig. 3.8.)

Table 4. 7: Reinforcement used for the laboratory beam

Specimen	Average diameter	Average Yield stress (MPa)	Used as
φ 6	6	574.64	Shear reinforcement
φ 10	10.39	486.40	Top reinforcement
φ 20	19.81	603.47	Bottom reinforcement

Table 4. 8: Material property , Concrete Damaged Plasticity model

Material parameter	f_{cu}= 31.98Mpa	Plasticity Parameters	
		Dilation angle	33
		Eccentricity	0.1
Concrete Elasticity		f_{bo}/f_{co}	1.16
E(GPa)	29.571	K	0.666
ν(Poisons Ratio)	0.2	Viscosity	0
Concrete Compressive Behavior		**Concrete Compressive Damage**	
Stress (MPa)	**Inelastic Strain**	**Damage Parameter (d_c)**	**Inelastic Strain**
12.58	0	0	0
22.03	0.00012233	0	0.00012233
28.16	0.000355541	0	0.000355541
31.34	0.00068757	0	0.00068757
31.88	0.001108043	0	0.001108043
30.04	0.001607988	0.0576	0.001607988
26.07	0.002179608	0.1824	0.002179608
20.15	0.002816093	0.3679	0.002816093
12.48	0.00351147	0.6087	0.00351147
3.20	0.004260476	0.8997	0.004260476
Concrete Tensile Behavior		**Concrete Tensile Damage**	
Yield Stress (MPa)	**Cracking Strain**	**Damage Parameter (d_c)**	**Inelastic Strain**
2.4	0	0	0
1.96	0.0001	0.183333	0.0001
1.5	0.0003	0.375	0.0003
1.29	0.0004	0.4625	0.0004
1.08	0.0005	0.55	0.0005
0.723	0.0008	0.69875	0.0008
0.484	0.001	0.798333	0.001

The concrete beam has been modeled as a solid part, while the reinforcements are embedded in the concrete beam as wire mesh parts. Section properties in accordance with the measured laboratory values are assigned to each of concrete and reinforcement part.

The beam is supported on two virtual supporting cylinders that would best simulate the real support condition. And the concentrated load is applied through a rigid cylinder placed at the top of the laboratory beam as shown in Figure 4.6. Kinematic coupling, as briefly explained in section 4.1.7 has been set between reference point A and the loading cylinder, to couple the vertical displacement of the beam with the loading cylinder.

Surface to surface Contact [12] has been assigned between the supporting cylinder, the loading cylinder and the reinforcing concrete beam. The final and assembled model of the laboratory beam on Abaqus /CAE interface is as shown in Figure 4.6.

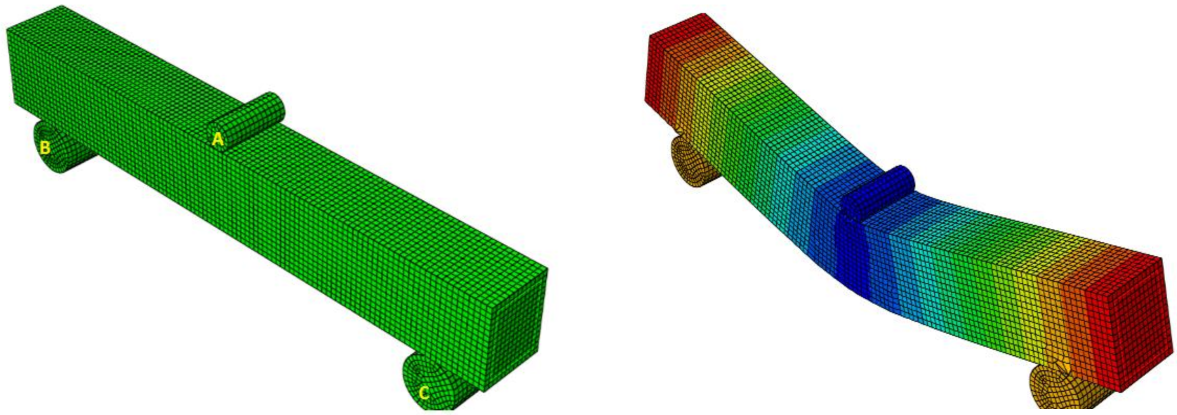


Figure 4. 5: The final and assembled model of the laboratory beam on Abaqus /CAE

Loading and Boundary conditions

Boundary conditions are set to the three points, two supports and one loading cylindars; as shown Table 4.9 and Figure 4.5.

Table 4. 9: Boundary Conditions, test beam

Point	Boundary condition	Remark
A	U1=U3=UR1=UR2=UR3, U2 is free	Loading Cylinder
B	U1=U2=U3=UR1=UR2=UR3=0	Support 1
C	U1=U2=U3=UR1=UR2=UR3=0	Support 2

A concentrated load of 277 KN, which is the peak load on the laboratory test, has been applied steadily to the beam. Dynamic implicit analysis step with its quasi-static application option has been used as an analysis option.

Figure 4.6 shows the mid-point deflection of the beam as computed in Abaqus and as measured in laboratory. There is an acceptable conformity between the laboratory results and the results obtained by Abaqus.

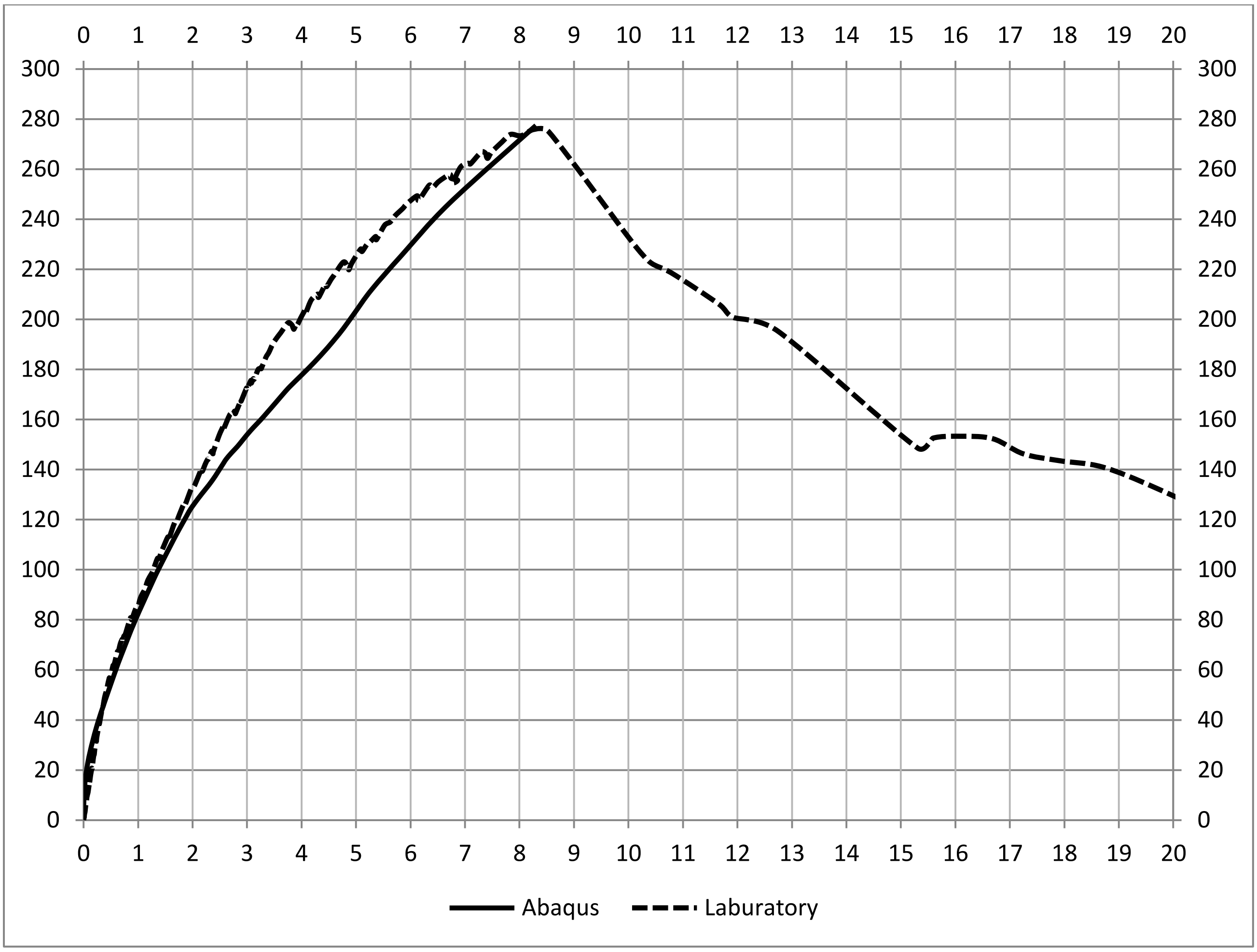


Figure 4. 6: Comparison of Mid span deflection of the laboratory tested beam with FE analysis

4.3 Verification of the global bridge model

The reliability of the FE model in simulating the response of the Reinforced Concrete T-girder Bridge under vehicular loading is verified through comparison with a data measured on site. The verification of the bridge model has been made by comparing the deflections measured by the installed deflection measuring gauge on the bridge, on the passage of the permit vehicle with the corresponding values obtained in Abaqus analysis.

The permit loading has been imitated on the FE model of the bridge, and the mid span deflection on the model has been paralleled with the on-site deflection measurement.

In the analysis, the truck load's longitudinal position which would result in the maximum deflection at mid-span of the bridge was used as a critical load position to compare analysis results of the model with the measured data. The permit load was made to pass through the center of the bridge, therefore that transverse position has been used.

The load, 6.2 ton (half of the axle load), was increased (ramped) until it reaches the actual wheel load of the permit truck. The results of the analysis are as shown in Figure 4.8.

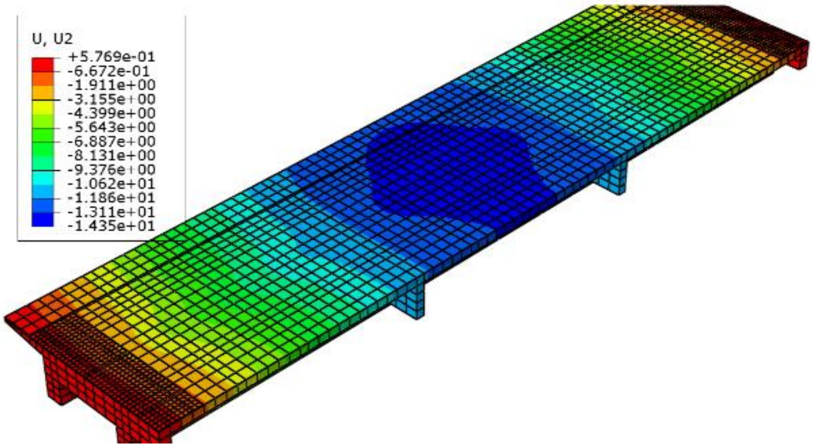


Figure 4. 7: Displacement(U2) contour of the strengthened bridge under the permit load

Figure 4.7 visualizes the Displacement (U2) contour of the strengthened bridge under the permit load. The displacement contour shown above is as the permit wheel load of gradually applied on the bridge. As it can be seen on Figure 4.7, the peak displacement at mid-span of the bridge is 14.35 mms, which is close to 18 mm deflection measured on field measurement. Although the result shows that the model represents the actual bridge to an acceptable extent, the discrepancy between the test and analysis results are due to some miss-represented model parameters.

The discrepancy between the field measured result and the results of the Abaqus model can be attributed to phenomenon not represented; such as fatigue induced cracks, rusting of reinforcement etc. Although modifications can be made, by altering some model parameters such as modulus of elasticity of concrete and diameter of reinforcing bars: quantifying the change cannot be supported by field measured data due to its unavailability. Therefore, it was resorted to using the current model, since it represents the bridge to an acceptable extent.

CHAPTER 5 RESULTS AND DISCUSSION

The analysis made with Abaqus/CAE 6.13.1 and the results obtained are presented in this chapter. Of the three load-rating procedures provided in [2] (design, legal and Permit) the results of each procedure serve a specific use and also guide the need for further evaluations to verify bridge safety or serviceability. And once confirmed that the model is reliable in predicting the real behavior of the bridge, through verification, the bridge is load rated for ERA's legal loads and the Permit load.

The transverse and longitudinal positions of the legal loads that gives out maximum response; in terms of deflection and reaction force at supports, is obtained by loading the bridge in different transverse and longitudinal positions with moving tire loads as shown in Figure 5.1.

Once the legal load and the legal load position that gives out the maximum response is identified, the loading position has been used to load rate the bridge. The wheels loads are distributed over the tire-contact area, as calculated in section 4.1.6. The virtual tires apply the legal load in a line load form because the contact between the bridge deck and the tires is a line. But a real tire applies distributed load over a small patch of area, refer section 4.1.6, on the deck slab of the bridge.

The basic need of software packages like Abaqus is as a simulation tool not as design software. Simulation software packages replicate a laboratory testing machine. A laboratory testing machine does not give out sectional outputs i.e. Bending moment and shear force, but it will show the amount of deflection/strain/applied load or displacement, etc. Hence, action effects such as, bending moment and shear force can be inferred as a secondary data from the outputs of simulation software. It is to be noted though; Abaqus gives out such results if one dimensional beam elements are used.

Since LRFD load rating equation uses sectional outputs i.e. bending moment, shear force or sectional forces, this action effects are inferred as a secondary data from the outputs of Abaqus. Reaction forces outputs are collected from the nodes on which boundary conditions are set and, bending moment and shear force in each section of the bridge are obtained from the nodal forces. The following sub-sections show the analysis results and their interpretation.

5.1 Analysis: ERA's Legal Loads

ERA design manual part I recommends four legal loads to use in load rating bridges, three of them recommended for short and medium span bridges and the last one recommended for long span bridges. Since Action effects are sensitive to the position of the Assessment loads, different positions of the legal loads (both transversally and longitudinally) are tried for maximum effect. Locations for maximum action effects are determined, in terms of deflection of the bridge, reactions on abutments and shear and bending moments.

According to [2] the general rule *for simple spans carrying moving concentrated loads states that the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support.* This rule can only be applied in simple computations that assume the bridge to have the same cross section longitudinally. However a typical T-girder bridge has different cross section throughout its length, since diaphragms are provided in certain locations, and in some cases it has varying girder depth. Besides longitudinal positions of the axle loads, their transvers location that gives the maximum action effect should also be determined.

The moving virtual tires representing the legal load axle configurations are used to determine the desired position of maximum action effects, the modeling can be seen in section 4.1.7. The moving tires are made to pass through the bridge longitudinally, in different transversal positions. Different transverse positions (I-IV) of each of the legal loads have been tried, look at Figure 5.1 and Table 5.1.

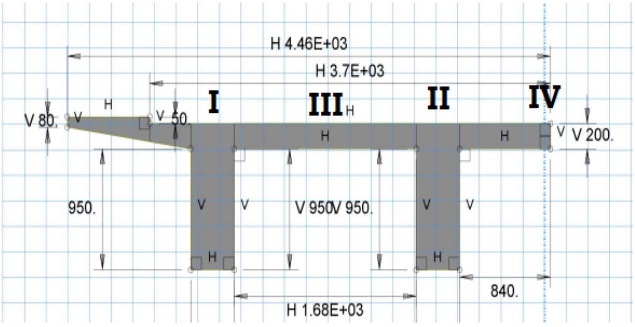


Figure 5. 1: Lateral positions (I to IV) of wheels for maximum effect; half of bridge cross-section

The legal load and position with the maximum action effect is used in load rating the bridge for legal load.

Table 5. 1: Lateral positions(I to IV) of wheels for maximum effect

It. No	Lateral Position	Distance from end of curb(mm)
1	I	200
2	II	940
3	III	2280
4	IV	3700

Modeling of the moving Tire load; Bridge-Tire contact definition, Boundary conditions and Interactions of the wheels with the bridge are explained in section 4.1.7. Dynamic implicit analysis step with its quasi-static form of application has been used for the analysis. The wheels were made to travel 50mm/sec, and output requests have been made for every second. Therefore, the effect of the moving load for each 50mm movement along the bridge can be known. Figure 5.2 shows, axle configuration of Legal load 3, at location III.

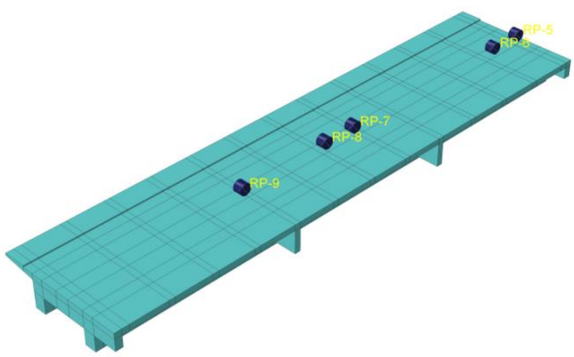


Figure 5. 2: Moving wheels with the configuration of Legal Load 2

The analysis results for some of the different transverse positions of the legal loads are presented in Figure 5.3 to 5.5. Vertical displacement of the bridge (U2) is presented here, and displacements (deflections) are presented in mm.

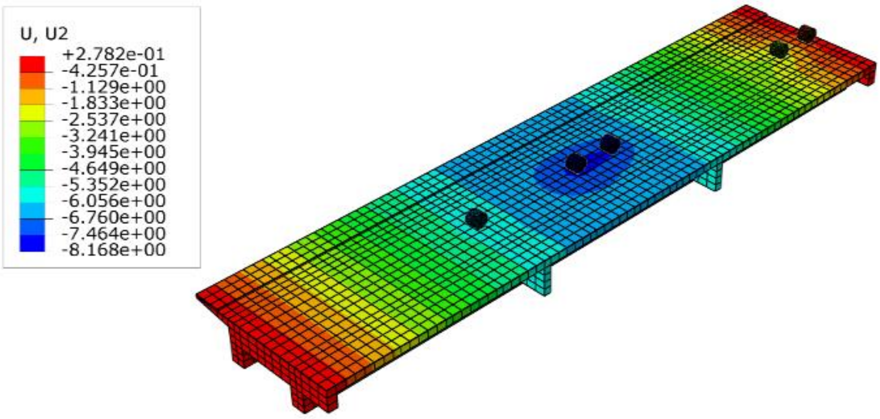


Figure 5. 3: Displacement(U2) contour for Legal load 1 at transverse position I

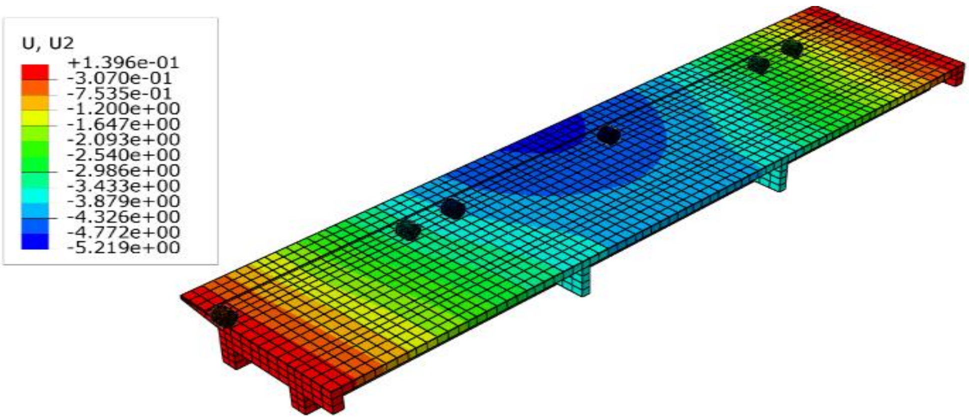


Figure 5. 4: Displacement(U2) contour for Legal load 3 at transverse position III

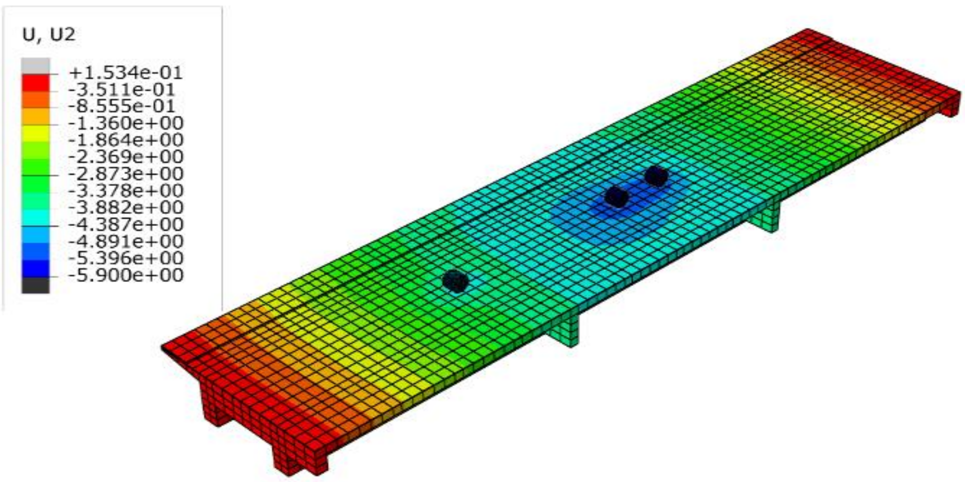


Figure 5. 5: Displacement (U2) contour for Legal load 1 at transverse position III

The highest deflection of *8.168 mm* has been obtained under Legal load 2 at position III, Figure 5.3. And the maximum reaction force and bending moment has been obtained under legal load 3 at position III, Figure 5.4.

Table 5. 2: Maximum action effect, Legal Load 3, at position 3

Sectional Output		V(shear Force), KN	M (bending moment), KN-m
Load configuration 2 : Legal Load 3		174.67	809.16
	Location	At right end support	At X= 4.57 m from the right end

The rating of the bridge for legal load is calculated as follows in section 5.3.

5.2 Analysis: Permit Load

Analysis has been made for the 172 ton vehicle load, with 16 axle configuration as shown in Figure 3.8. Longitudinal positions of tire footprints that would result in maximum action effects of the permit load are predicted by making a simple model of the bridge on SAP 2000. Axle configuration and load that are used in the analysis and rating of the bridge are shown in Figure 3.8 of section 3.5.

It should be mentioned that, the strengthening plates are not modeled for the load rating analysis, which is why the maximum deflection of the bridge increased from 14.35 to 14.85 mm. Figure 5.6 shows the deflection contour, u2, under the permit load, and the maximum action effects are specified in Table 5.3.

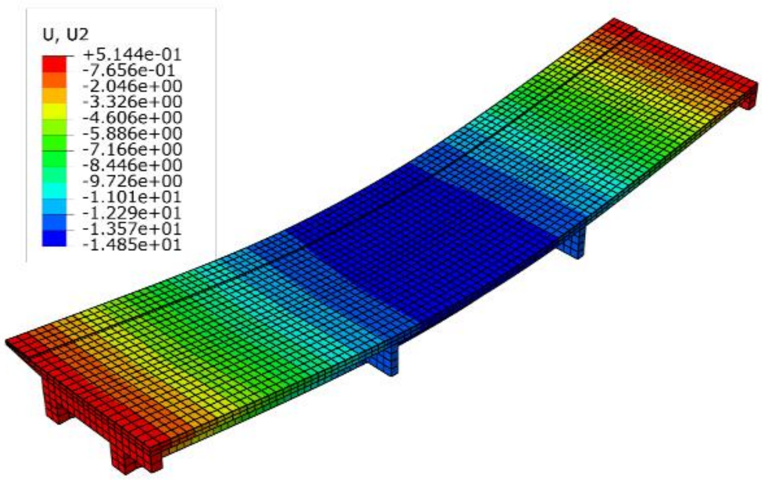


Figure 5. 6: Displacement(deflection) contour, U2, of the bridge under permit load without the strengthening plates

**** A scale of 1:70:1 (x: y: z) has been used in this figure*

Table 5. 3: Maximum action effect, permit truck load

Sectional Output		V(shear Force), KN	M (bending moment), KN-m
Load configuration : Permit truck load		398.1	1734.29
	Location	At the right end support	At X= 9.37m from right support

5.3 Load rating computation

Load rating computations, for the permit and legal load, have been conducted as per ERA's simplified load rating equation shown below;

$$RF = \frac{C - (\gamma_D)(D)}{\gamma_{LL}(LL + IM)} \qquad \text{……………………………………………………………………..Eq. 5.1}$$

$$C = \varphi Rn$$

The primary resistance of the bridge section in shear and moment are obtained by loading the bridge to failure. The load configuration of the permit load and Legal Load III has been used to load the bridge to failure to determine nominal resistance for permit and legal load rating respectively.

Shear Force and bending moments, shown in Table 5.4, are determined from the reaction forces at the failure load and used as nominal resistance values for load rating calculations. Total Load to mid span displacement curves for each case are as shown in Figure 5.7 and 5.8 below:

Table 5. 4: Action effects at maximum load, failure load

It. No.	Loading	Reaction Force(KN)	Shear force(KN)	Bending moment(KN-m)
1	Permit load	1214.8	1214.8	5482.44
2	Legal load 3	1320	1320	6200

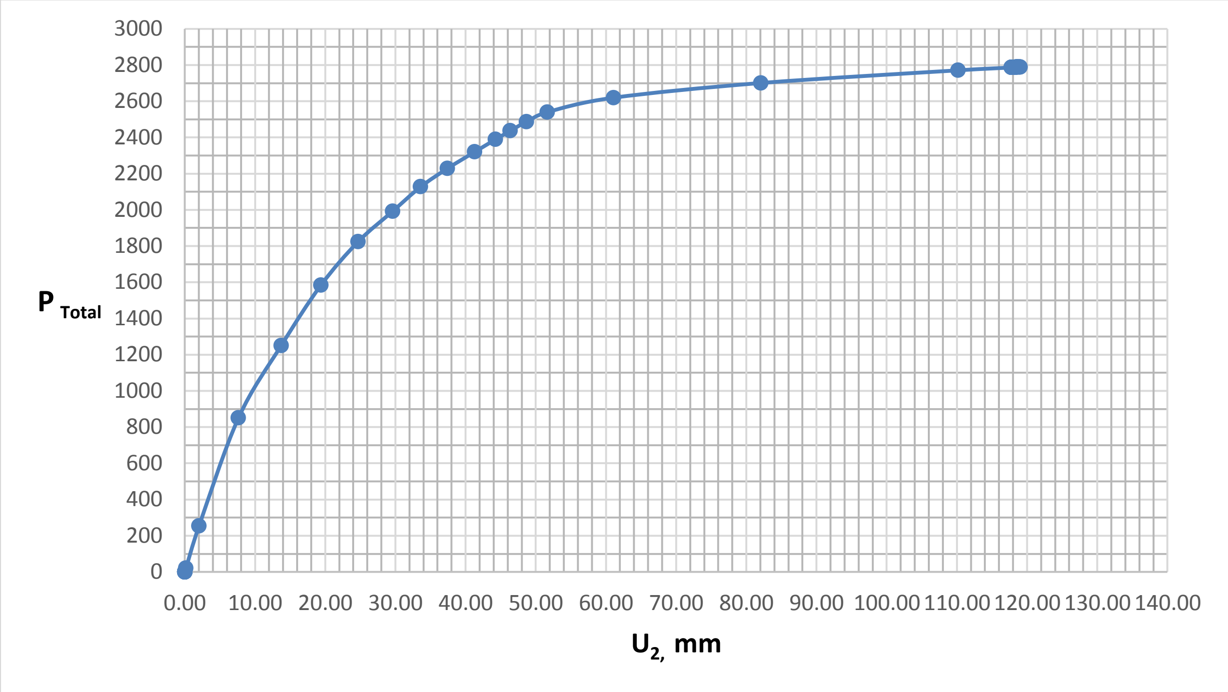


Figure 5. 7: Total Load-displacement curve; bridge loaded to failure, Legal Load 3 configuration

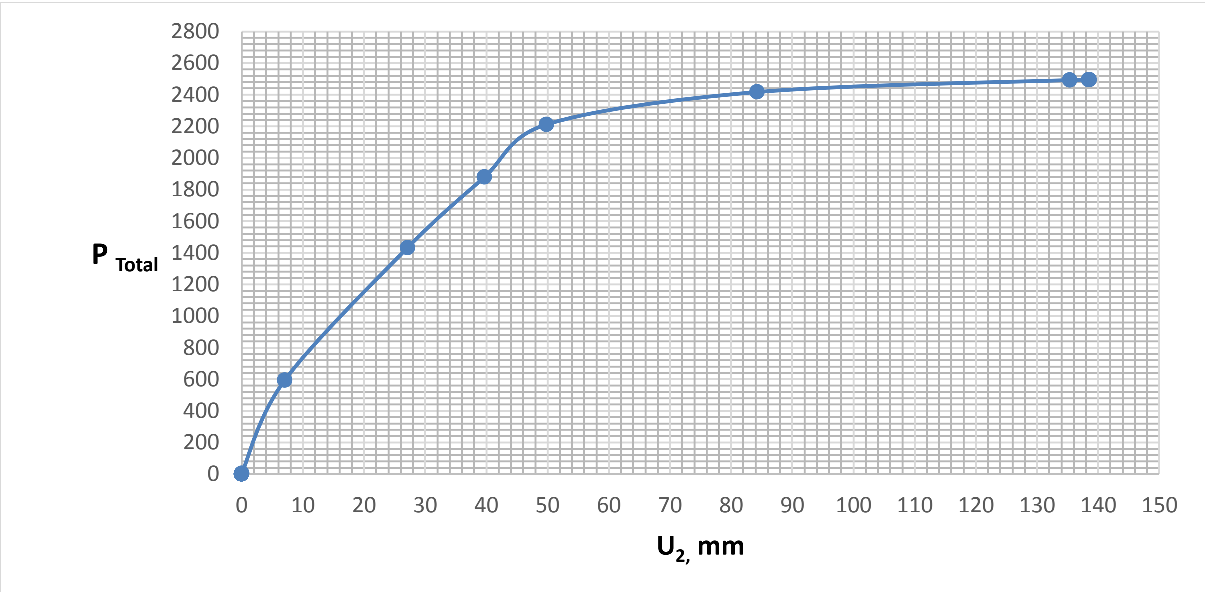


Figure 5. 8: Total Load-displacement curve; bridge loaded to failure, Permit load configuration

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects when stress is chosen as the evaluation approach and for the capacity when force or moment is chosen for use in the basic rating equation [2].

Selection of resistance factor depends on redundancy of structural system, maintenance scheme and current bridge condition. These factors are highly dependent on the bridge inspection reports.

Resistance factor equal to 0.8 has been chosen for the bridge super-structure, using data/information from the bridge inspection report. A value of 0.8 is arrived at by taking the following in to consideration, (referred from Table 5.7 of ERA bridge design manual [8])

Selection of resistance factor:

- Super-structure condition: Deteriorated, as it is shown in section 3.3 the bridge superstructure shows numerous distress; shear and flexural cracks.
- Redundancy: The bridge is simply supported, with four parallel stringers (girders) so there is redundancy
- Inspection: careful Inspection has been conducted for the bridge.
- Maintenance: based on the available information, there has not been a documented major maintenance or rehabilitation on the bridge.

The load factors are used to account for uncertainties in load effects due to uncertainties in analysis as well as load magnitudes. The dead load factor includes normal variations in material dimensions and densities [8]. Dead and live load factors have been selected according to ERA bride design manual, section 5:

Table 5. 5: Dead and live load factors; Table 5.5 of ERA design manual part I, section 5

Rating Type	Permit	Legal
Dead Load factor	1.2	1.2
Live Load factor	1	1.65

A live load factor equal to one has been used for the permit truck since the exact axle configuration, axle load and combination of axles that give out the maximum action effect is precisely known. Whereas for the legal load rating, the overloading and absence of regulation on overload, and high traffic volume on the bridge has been taken in to account by taking a higher Live load factor.

The simplified load rating equation, Eq. 5.1, in ERA design manual Part II, section 5 has been used to load rate the bridge.

$$RF = \frac{C - (\gamma_D)(D)}{\gamma_{LL}(LL + IM)} \quad \text{……………………………………………………………..Eq. 5.1}$$

$$C = \varphi Rn$$

Table 5. 6: Bridge load rating computation

Action Effect*	Resistance Factor	Nominal Resistance	Nominal DL effect	Nominal LL effect	Impact factor (1+1)	Rating factor
Rating for Permit load						
M_p	0.8	5482.436	1931.55	1734.29	1.1	1.08
V_p		1214.8	411	398.125	1.1	1.09
Legal Load Rating						
M_{LL}	0.8	6200	1931.55	809.16	1.3	1.52
V_{LL}		1320	411	174.67	1.3	1.50

* V=shear force, M=Bending moment

The rating factors obtained, for both permit load and legal load are in acceptable range, since they show availability of about 8% and 50 % reserve capacity for the permit and legal loads respectively.

CHAPTER 6 CONCLUSIONS AND RECCOMENDATIONS

6.1 Conclusions

- As it has been shown in sections three, four and five that the load carrying capacity of reinforced concrete bridges without plans can be well estimated by combining information obtained from
  - Historical records,
  - Design codes existing by the time of construction of the bridge ,
  - Field inspection, testing and measurement and
  - Refined analysis with Finite Elements simulation
- The reliability of the output FE model gives (for this case mid span deflection) has been verified against field measured deflection of Koka Bridge, and it has been found in close proximity.
- Assessment codes are found to give sufficient recommendations on how to treat bridges that does not have sufficient data to be used in load rating.
- Design codes existing by the time of construction of a bridge without design plans can be used as a good reference in estimating reinforcement amount, since they are based on the accepted design philosophy by the time of design and construction of the bridges.
- Koka River Bridge need not be load posted, since it can well withstand the effect of legal loads. Based on the calculated rating factors, in section 5.3, the bridge has around 50 % reserve capacity.
- The bridge also has a load carrying capacity to pass the proposed permit loads. It was found to have 8 % reserve capacity when load rated for the permit loads.
- The position of the legal axle configuration with maximum deflection and reaction force is slightly different from the position expected in simply supported monolithic beam. This is apparent because the bridge has diaphragms in between and at the support positions that would give it irregular cross-section longitudinally.
- The maximum live load deflection of the bridge measured on site as the permit truck passes, 18mm, is well below the allowable live load deflection of reinforced concrete

bridges [5], L/800 which is 23.5mm. According to [5], this assures closing of cracks after the passage of the permit truck. So, it is concluded that the bridge can withstand the legal load and the proposed permit load.

6.2 Recommendations

The following recommendations are given based on the study made:

- Based on the inspection done on the bridge; maintenance of the bridge is recommended in order to extend the service life of the bridge.
    - Repair of the expansion joints in between spans will decrease the impact effect of speeding vehicles on the bridge.
    - Clogged drainage system of the bridge should also be maintained since retention of water intensifies deterioration of concrete and corrosion of reinforcement bars.
    - Cracks should also be sealed, so that propagation is inhibited.
- Non-destructive load testing of bridges, especially for bridges without plans is highly recommended before load rating. Measurements of bridge deflection and strain in reinforcements have been found to give a good input in predicting response of bridges.
- Load posting and permit issues should be based on refined simulation analysis of bridges, since they can accurately show bridges' reserve capacity. If they are based on conservative structural analysis approaches, they might put unnecessary constraint in transporting heavy goods through the road network. This has negative economic implication since it can increase the cost of mega –projects since it escalates costs for logistics and transportation.
- Finite element models of bridges, prepared on simulation software packages such as Abaqus provide a good interface for assessment. If data on extent of deterioration, previous traffic data and extensive load testing data is available, finite element models can be calibrated to best simulate the response of the real bridge. Calibration can be done decreasing reinforcement amount due to corrosion and stiffness of concrete section due to cracking and so on.

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APPENDIX I

Plan and cross-sectional drawing of Koka River Bridge

(As per field inspection and Measurement)