Jimma University

School of Graduate Studies

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Rating and Lifetime Evaluation of Existing Reinforced Concrete Girder Bridges: A Case Study on Oda Bridge

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

By

Kassahun Kebede

September, 2016

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SCHOOL OF POST GRADUATE STUDIES

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As members of the examining board of the final MSc open defense, we certify that we have read and evaluated the thesis prepared by Kassahun Kebede entitled: "*Rating and Lifetime Evaluation of Existing Reinforced Concrete Girder Bridges: A Case Study on Oda Bridge*" and recommended that it would be accepted as fulfilling the thesis requirement for the degree of Master of Science in Structural Engineering.

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DEDICATION

This Thesis Is Dedicated To My Advisor and Family

"There are always many possible solutions, the research is for the best--but there is no best--*just more or less good"………Ove Arup.*

ABSTRACT

Existing bridges are being subjected to an ever increasing volume of heavy truck traffic, and a growing number of exceptional vehicles. This may reduce their life and strength they were designed for. Due to this performance declining, there is a need to determine the available strength and remaining life in existing bridges. Bridge evaluation is, therefore, required for safety assurance as it provides evidence if the bridge function safely over a specified design life.

Load rating is the most common method employed in safety evaluation of existing bridges as it measures a bridge's load-carrying capacity for specific vehicles. The theoretical analysis done to Oda Bridge under the specific trucks confirmed that strength is not the problem. But the effect of stress accumulation called fatigue was not accounted. Fatigue is another form of performance problem caused by accumulation of stresses from trucks traveling across the bridge. This makes it suspicious for fatigue strength. Analysis for fatigue strength is important, in addition to load rating, to determine performance and overall safety of existing concrete bridges.

The objective of this study is to assess a performance of Oda Bridge using LRFR, fracture mechanics and Stress-life approaches. Load rating was executed under ERA type-3 legal truck to determine its available capacity and fatigue analysis was performed under AASHTO 1990 fatigue truck model to determine its available lifetime in years. The girder line analysis method is utilized for both load rating and fatigue analysis.

The rating values determined are greater than the rating truck load weight both for flexure and shear. The available life of the bridge is also predicted. Accordingly, it is concluded that the bridge is safe in carrying the current live loads but fatigue failure is likely to happen in its lifetime.

The substructure part of the bridge is deteriorated and eroded. Assumptions are also used in the one dimensional analysis. Hence, it is highly recommended that a three dimensional analysis as well as evaluation for substructure of the bridge is important.

Key words: *existing concrete bridges, fatigue life, fatigue truck load, load rating, Non-destructive test, stress-life, and LEFM approach,.*

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ACRONYMS

The following lists of symbols are used in this Thesis;

a	Crack/flaw length, LEFM
a	moment distribution factor
a_{cr}	Critical crack length, LEFM
a_i	Initial crack length, LEFM
AASHTO	American Association of State highway and Transportation Officials
ADTT	Average Daily Truck Traffic
ASR	Allowable Stress Rating
As	area of flexural steel
C	average crack growth metal constant, LEFM
da/dN	crack growth rate per cycle of loading, LEFM
DF	load distribution factor
GF1	past growth factor
GF2	future growth factor
DL	dead load
ECCS	European convention for constructional steelwork
EC 2	European Code two
ERA	Ethiopian Roads Authority
fc'	Concrete cylindrical compressive strength
FE	Finite Element
GDF	girder distribution factor
gm_1	moment distribution factor for interior girder
gm_2	moment distribution factor for exterior girder
gv_1	shear distribution factor for interior girder
gv_2	shear distribution factor for exterior girder
GVW	gross vehicle weight
I	moment of inertia of beam
ΔK	stress intensity factor range, LEFM
KIC	the fracture toughness of steel metal
ΔKth	fatigue threshold, LEFM
L	center to center span length
LRFD	Load and Resistance Factor Design

LFR	Load Factor Rating
LRFR	Load and Resistance Factor Rating
LEFMA	Linear Elastic Fracture Mechanics Approach
m	slope S-N curve of steel bars
M	accumulated number of cycles up to recent years
MDF	moment distribution factor
M_{DL}	total dead load moment
M_{LLIM}	total moment due to live load
M_n	nominal moment capacity of section
MBE	Manual for Bridge Evaluation
n_f	future year for which ADTT is estimated
n_1	previous year which ADTT is known
n_2	recent year which ADTT is known
n_3	difference between n_1 and
n_4	difference between n_f and n_2,
N_f	total fatigue life in cycles, LEFM
N	accumulated number of load cycles
NCHRP	National Cooperative Highway Research Program
NDT	Non-Destructive Testing
$\Delta\sigma$	live load stress range, LEFM
P	axle load
RF	Rating Factor
RT	rating in tons for truck used in computing live load effect
SB	Sustainable Bridge
S	stress
SDF	shear distribution factor
S_v	shear reinforcement spacing
T1	front type 3 truck axle load
T2	middle type 3 truck axle load
T3	rear type 3 truck axle load
Vc	concrete section shear capacity
V_{DL}	dead load shear in the girders

V_{LLIM}	total shear force from live load
Vn	nominal shear capacity
W	weight in tons of the rating truck
w	uniform dead load weight on girders
WIM	Weigh-in-motion
W_T	total dead load weight
Y	crack shape factor

CHAPTER ONE

1. INTRODUCTION

1.1 Background

Highway bridges are not simply structures made out of materials, they are part of life. In many places across the world, life would be seriously disrupted, traffic would be hindered and business would be terribly affected if the bridges fail functioning [1].In-service bridges are being subjected to an ever increasing volume of heavy truck traffic, and a growing number of exceptional live loads, during their design lifespan. This effect will reduce their performance and become safety problems. Performance assessment is, therefore, required for older bridges to estimate their available capacity and life. The assessment of existing bridges provides evidence if the bridge function safely over a specified design life. Due to this, the evaluation of bridges for performance is a vital task in efficient bridge management [2].

Load ratings and fatigue evaluations are most common types of performance assessment methods in existing bridges. Rating is performed to determine the available capacity while fatigue estimates the remaining age of the bridge considering accumulated stresses.

Bridge load rating is a process of available load bearing capacity determination using recent inspection results. In Ethiopia, bridges are load rated under the legal trucks to confirm their carrying capacity according to ERA-2002Bridge Design Manual. Depending on the values obtained from the analysis, bridges are closed or left open for traffic.

Fatigue is another deteriorating agent need to be determined. It is a phenomenon of weakening of a material as a result of accumulation of damages from repeated loads. Fatigue cracks initiate and then propagate under a repeated loads resulting in damage and complete fracture of a member that finally collapse the structure at all. Hence, more attention should be given to fatigue damages of bridges in addition to rating for heavy trucks [3].

The road Addis Ababa to Nekemte is experiencing heavy vehicle traffic volume and long vehicles in recent days than before. Vehicles transporting machines for Hidase dam, Dhidhessa sugar factory construction and Gimbi Marble stone extraction and cartage have been using the road. There is evidence from designers of the road that most of heavy vehicles using this road section are increased after rehabilitation. As a result, existing bridges are being subjected to heavy and large number of trucks most frequently than usual. This made safety assessment of bridges along this road segment a very important task.

In this thesis, analytical means of assessment was employed to rate and predict fatigue life of a concrete girder bridge called Oda Bridge. Load rating was performed using LRFR method. Fatigue evaluation is also executed using linear elastic fracture mechanics and stress-life approaches. A residual capacity and a remaining fatigue life of the bridge are determined utilizing a girder line analysis method in combination with inspections and field tests (hammer testing).

1.2 Problem Statement

An increase in traffic volume over time causes bridges to deteriorate and become weaker and weaker as its age increases. Not only traffic increment but also damage accumulation is another problem to cause life reduction in existing bridges. Hence, evaluation is required to know the available strength of the bridge and its remaining life which in turn used to determine public safety.

Oda Bridge is an old bridge. Rating need to be done because of that the vehicle for which it was designed is not known and also, a traffic volume using the bridge today is increased. From the theoretical rating analysis done to the bridge, strength is not an issue for vehicles currently using the bridge. That is why no considerable maintenance has done for this bridge at all. However, questions arise as to whether or not the assessment performed was made for damage accumulation (fatigue) and if there is sufficient fatigue reserve to reach its design lifespan under current loads and the increased traffic volume.

Fatigue is another form of deterioration problem induced by the trucks crossing bridges. To tackle this problem, ERA limits stress range in the steel reinforcement, which is concerned with preventing crack initiation. It does not consider fatigue damage accumulation but for new design. Moreover, the bridge was designed before the fatigue design specifications were modified which makes it more susceptible to fatigue. Fatigue evaluation, in addition to rating is, therefore, very useful to determine a performance and safety of in-service bridges.

1.3 Objectives

1.3.1 General objective

The primary objective of this study is to assess performance of existing reinforced concrete girder bridges using LRFR, fracture mechanics and Stress-life approaches.

1.3.2 Specific objectives

The following specific objectives will be attained at the end.

- To determine available capacity of the bridge.
- To estimate the remaining life time of the bridge.

1.4 Significance of the study

On its completion, this thesis may give some awareness to client about safety of the bridge. In addition, it provides fatigue effects that may happen when the stresses have been repeated a very large number of times. Since this is a case in bridges, consideration should be required when evaluating bridges, especially older concrete bridges, in addition to load rating checkup for fatigue failure is necessary. Due to this, the thesis may be used as a benchmark to introduce the requirement and importance of fatigue evaluation for concrete bridges because not well known yet. Country's engineers extend the evaluation methods employed in this research to use for other bridges evaluation, especially for evaluation of railway bridges, which the country is constructing to increase railway transportation these days. Because railway bridges suffer with heavy dynamic cyclic stress generated from trains.

1.5 Thesis Organization

The thesis is composed of five chapters that are organized as follows. Chapter 1 introduces the background for the research, details the objectives of the study, statement of problem and summarizes the organization of the thesis. It also explains significant of the thesis. Chapter 2 of this thesis provides a literature review of load rating and fatigue of concrete bridges. Types and methods of ratings are presented in detail. In addition, fatigue evaluation methods, equations, as well as fatigue of concrete and rebar represented in detail. Chapter 3 states the methodologies used in the thesis. Analysis methods and NDT and visual inspection assessments are presented. Chapter 4 provides analysis of a case study bridge in accordance with the available methods and procedures reviewed in chapter two. Finally, it discusses the available capacity and remaining life of Oda Bridge and provides the rating values obtained under legal trucks. It also presents the fatigue evaluation procedures used to determine fatigue life and results obtained from analysis.

Chapter 5 summarizes the analysis outcomes from chapter four. It also provides the conclusions and recommendations of the thesis.

1.6 Scope and Limitations of the thesis

Some of the scope and limitations of the thesis include the following:

1.6.1 Scope

In this thesis, only superstructure part of the concrete girder bridge is evaluated.

1.6.2 Limitations

- ✓ The scarcity of appropriately documented plan and detail of the bridge,
- ✓ Tensile strength (tension stiffening effects) of concrete is not considered
- ✓ Influencing factors like creep and shrinkage of concrete were ignored.
- ✓ Vehicle passage, especially truck passage, on the bridges is assumed to be the only source of fatigue loading.
- ✓ Effect of intermediate diaphragm is not considered.

CHAPTER TWO

2. REVIEW OF RELATED LITRATURES

In this chapter, review of literatures concerning load rating and fatigue evaluation has been made. The rating methods, rating types, loads for ratings and rating procedures as well as analysis methods used in ratings were briefly discussed. And also fatigue behavior of concrete structures (that of steel bars and concrete separately) and methods and fatigue truck used in the analysis of fatigue life, are reviewed.

2.1 Bridge Capacity Evaluation

Bridge evaluation is performed to determine the load-carrying capacity of all critical elements of the bridge, and the bridge as a whole. Using the information obtained from the field inspection, dimension and geometry evaluation, and material evaluations, the load-carrying capacity of the bridge or portion of the bridge undergoing evaluation should be determined. The capacity evaluation process is defined as load rating [4].

2.1.1 Bridge Load Rating

Load rating is defined as the determination of the live load carrying capacity of a bridge using as-built bridge plans and supplemented by information gathered from the latest field inspection. Load ratings are expressed as a rating factor (RF) or as a tonnage for a particular vehicle. Emphasis in load rating is on the live-load capacity and dictates the approach of determining rating factors instead of the design approach of satisfying limit states.

In general, bridge load ratings are used to determine carrying capacity of existing bridges, which in turn used to allocate funding for repair and rehabilitation [5].This rating is crucial to verify the safety of public traffic and economy.

There are circumstances under which bridge load rating is required as described by [1]:

- The design live load is less than that of the heaviest statutory commercial vehicle plying or likely to ply on the bridge;
- Exact live load for which the bridges are designed is unknown;
- Tackling today's increased traffic demand;
- Regulating with adverse environmental effects;
- Indication of distress leading to doubt about structural and functional adequacy.

2.1.2 Review of global practice

Most of the countries in the world do not have any comprehensive bridge rating system. In many countries, load rating of bridges is carried out in connection with passage of exceptional loads only. The national loading standards, bridge codes and standards vary to a large extent for all these countries and so also the systems followed in evaluation [1].In general, load rating includes the concept of "inventory" and "operational" ratings.

2.1.3 Bridge load rating categories

Bridge load rating provides a basis for determining a bridge's safe load carrying capacity. There are three different load rating categories depending on three different load intensities. These are inventory, operating and posting ratings. Bridges are rated according to the weight of standard trucks. Each standard truck has a different axle loading and configuration. The ratings will vary for each standard truck since each truck loads the bridge differently.

Any bridge component can be load rated. However, it is generally assumed that the bending moment and shear in the girders will give the critical rating values [5].

2.1.3.1 Inventory rating

Inventory rating captures the lower range of bridge performance. It is the capacity rating for the vehicle type used in the rating that will result in a load level which can safely utilize an existing structure for an indefinite period of time. Inventory load level approximates the design load level for normal service conditions. In simple terms, this rating indicates the bridge's performance under the loading of everyday traffic [5].

2.1.3.2 Operating rating

Operating rating captures the upper range of bridge performance. Sometimes a bridge may need to handle an abnormally large live load [5]. The bridge's life span would be shortened if that loads repeatedly pass over the bridge. Therefore, the operating rating represents the absolute maximum permissible load level to which the structure may be subjected for the vehicle type used in the rating. This rating determines the capacity of the bridge for occasional use. This value is typically used when overweight permit vehicle moves.

2.1.3.3 The posting rating

The posting rating is used for legal purposes in each state. The posting rating is the capacity rating for the vehicle type used in the rating that will result in a load level which may safely utilize an existing structure on a routine basis for a limited period of time. The posting rating for a bridge is based on inventory level plus a fraction of the difference between inventory and operating [1].

2.1.3 Bridge Load Rating Methods

There are three methods of bridge ratings. These are: the Allowable Stress Rating (ASR), the Load Factor Rating (LFR) and the Load and Resistance Factor Rating (LRFR) methods [5].

The allowable or working stress rating method constitutes a traditional specification to provide structural safety. The actual loadings are combined to produce a maximum stress in a member which is not to exceed the allowable or working stress. The working stress is found by taking the limiting stress of the material and applying an appropriate factor of safety.

The load factor rating method is based on analyzing a structure subject to multiples of the actual loads. Different factors are applied to each type of load which reflects the uncertainty likely to occur in the load calculations. The rating is determined such that the effect of the factored loads does not exceed the strength of the member.

The load and resistance factor rating method is the newer method and was created in conjunction with the LRFD philosophy. In this method, different factors are applied to both loads and materials to overcome the uncertainties inherent in load and resistance calculations during rating. The United States Federal Highway Administration (FHWA, 2012) considers LRFR as the preferred load-rating methodology for all existing bridges.

The LRFR methodology consists of three distinct levels of evaluation. These are: design load rating (first level evaluation); legal load rating (second level evaluation); permit load rating (third level evaluation).

2.1.3.1 Design-load rating

Design load rating is a first-level assessment of bridges based on the design vehicle (HL-93 loading) and LRFD design standards. It is a measure of the performance of existing bridges to current LRFD bridge design standards. Under this check, bridges are screened for the strength limit state at the LRFD design level of reliability (Inventory level), or at a second lower evaluation level of reliability (Operating level). Design load rating can serve as a screening process to identify bridges that should be load rated for legal loads per the following criteria [6]:

- Bridges that pass HL-93 screening at the Inventory level will have adequate capacity for all AASHTO legal loads and State legal loads that fall within the exclusion limits described in the AASHTO LRFD Bridge Design Specifications.
- Bridges that pass HL-93 screening only at the Operating level will have adequate capacity for AASHTO legal loads, but may not rate (RF < 1) for all State legal loads, specifically those vehicles significantly heavier than the AASHTO trucks.

A dynamic load allowance of 33% LRFD Design was applied to the truck/tandem load only, while a multiple presence factor according to LRFD Design was applied to both truck/tandem and lane loads.

Table 2.1 MBE 6A.4.3.2.2-1 Load Factors for Design Load: γL[6]

Evaluation Level	Load Factor
Inventory	1.75
Operating	1.35

2.1.3.2 Legal-load rating

Bridges that do not have sufficient capacity under the design-load rating shall be load rated for legal loads to establish the need for load posting or strengthening. This second level rating provides the safe load capacity of a bridge for the AASHTO family of legal loads or State legal loads, whichever is greater. The figures that follow present Nebraska legal loads (Type 3, Type 3S2, and Type 3-3), which are heavier than AASHTO legal loads, in addition to the lane-type loading for spans greater than 200 ft [6].

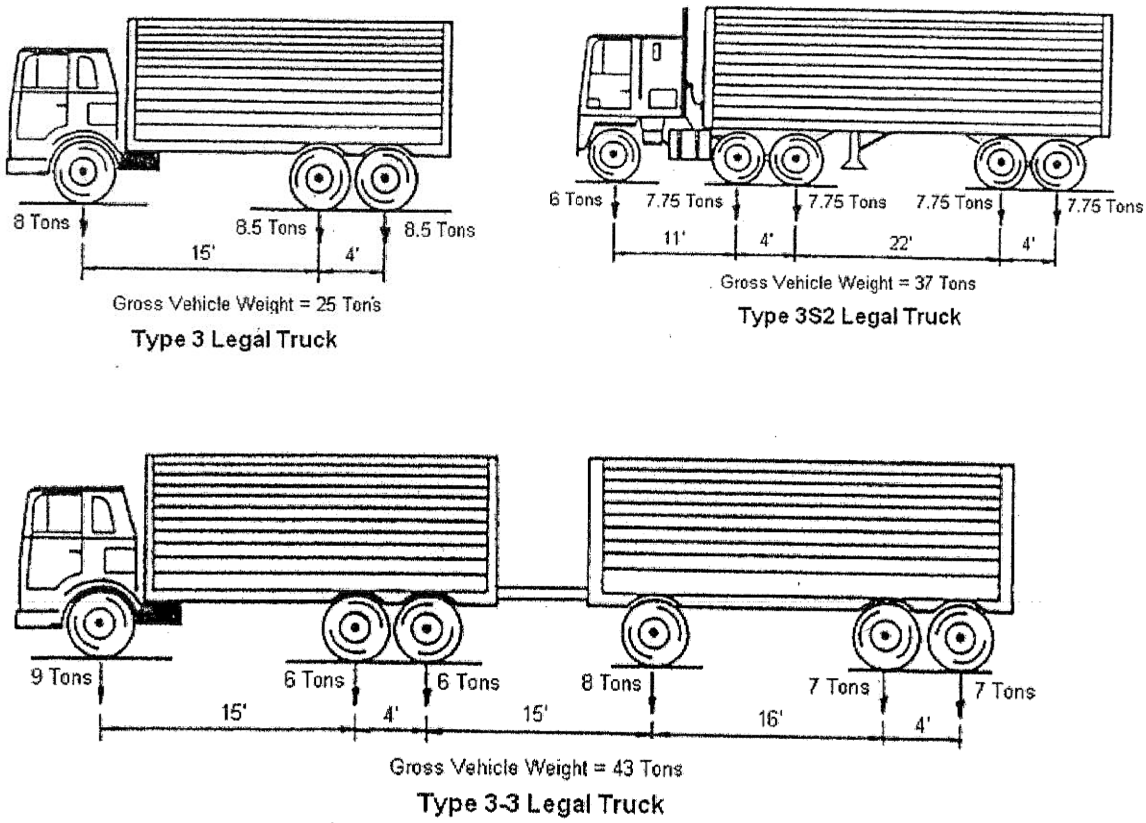

Figure 2.1 AASHTO legal trucks [6]

Strength is the primary limit state for legal load rating. Live load factors were selected based on the ADTT at the bridge as shown in table 2.2. The three AASHTO families of legal loads (Type 3, Type 3S2 and Type 3-3) are used in load rating for routine commercial traffic.

Table 2.2 Live-Load Factors, γL for AASHTO Legal Loads for Strength-I [6]

Traffic Volume (One direction)	Load Factor forType3, Type3S2,Type3-3 and lane loads
Unknown	1.80
ADTT≥ 5000	1.80
ADTT=1000	1.65
ADTT ≤ 100	1.40

Linear interpolation is permitted for other ADTT

2.1.3.3 Permit-load rating

Permit load rating checks the safety of bridges in the review of permit applications for the passage of vehicles above the legally established weight limitations. This is a third level rating that should be applied only to bridges having sufficient capacity for legal loads. Load factors by permit type and traffic conditions on the bridge are specified for reviewing the safety inherent with the passage of the overweight truck [6].

2.1.4 General Rating Equations

The load rating is generally expressed as a rating factor for a particular live load model, using the general load-rating equation:

$$\mathrm{RF} = \frac{\mathrm{C} - (\gamma_{\mathrm{DC}}) * \mathrm{DC} - (\gamma_{\mathrm{DW}}) * \mathrm{DW} + (\gamma_{\mathrm{P}}) * \mathrm{P}}{\gamma_{\mathrm{LL}} * (\mathrm{LL} + \mathrm{IM})} \tag{2.1}$$

Where

RF = rating factor

DC = dead load effect due to structural components and attachments

DW = dead load effects due to wearing surface and utilities

P = permanent loads other than dead loads

LL = live load effect

IM= dynamic load allowance equal to 33%

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

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

γ_{LL}= live load factor

γ_{P}= LRFD load factor for permanent loads other than dead loads=1

C= member Capacity

For strength limit states the capacity can be determined by Equation 2.

$$\mathrm{C} = \emptyset\mathrm{c} * \emptyset\mathrm{s} * \emptyset * \mathrm{R_n} \tag{2.2}$$

Where the following lower limit shall apply: $\emptyset c \emptyset s \geq 0.85$

Φc=condition factor

Φs=system factor

Φ = LRFD resistance factor

Rn= nominal member resistance (as inspected)

The rating factor (RF) obtained is used to determine the safe load capacity of the bridge in tons. This can be expressed as:

$$\mathrm{RT} = \mathrm{RF} * \mathrm{W} \tag{2.3}$$

Where RT=rating in tons for truck used in computing live load

W=weight in tons of truck used in computing live load effect.

Depending on the obtained values of RF, one could give two recommendations:

- When the RF is greater or equal to unity, the bridge is capable of carrying the rating vehicle.
- On the contrary, when the RF is less than unity, the bridge may be overstressed while carrying the rating vehicle [6].

1. **Load Factors-**The load factors are used to account for uncertainties in load effects due to the method of analysis as well as load magnitudes. The dead load factor includes normal variations in material dimensions and densities. The live load factor accounts for uncertainties in expected maximum vehicle loading effect, impact, and distribution of loads during a time period between inspections [6].

The evaluation live-load factors for legal-load rating at the Strength I limit state load combination are a function of the average daily truck traffic (ADTT).

Strength is the primary limit state for load rating. Service and fatigue limit states are selectively applied in accordance with the provisions of this manual. Applicable limit states and the corresponding load factors are summarized in table 6A.4.2.2-1 [6].

Table 2.3 Limit States and Load Factors for Load Rating [6]

Bridge Type	Limit State*	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load Inventory γ_{LL}	Design Load Operating γ_{LL}	Legal Load γ_{LL}	Permit Load γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.75	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	0.80	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1

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2. **Condition Factors**-the condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

Table 2.4 AASHTO LRFR Condition Factor, φc [7]

Structural Condition of Member	φc
Good or satisfactory	1.00
Fair	0.95
Poor	0.85

3. System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings.

Table 2.5 AASHTO LRFR System Factor, φs, for Flexural and Axial Effects [7]

Superstructure Type	φs
Welded members in two-girder/truss/arch bridges	0.85
Riveted members in two-girder/truss/arch bridges	0.90
Multiple eye bar members in truss bridges	0.90
Three-girder bridges with girder spacing of 6 ft.	0.85
Four-girder bridges with girder spacing≤4ft.	0.95
All other girder bridges and slab bridges	1.00
Floor beams with spacing >12ft.and non-continuous stringers	0.85
Redundant stringer subsystems between floor beams	1.00

2.1.5 Ethiopian Practice

The existing bridge evaluation methodology in Ethiopia is based on the Load and Resistance Factor Design (LRFD) philosophy. The methodology can be used to evaluate almost all existing bridge types in Ethiopia.

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.

The rating check is done by comparing the factored load effects (both dead and live) with the factored resistance at all critical sections. The output is a rating factor, which determines the suitability of the given bridge for the loads under considerations [8].

2.1.5.1 The Rating Equation

The evaluation is carried out with a comparison of the factored live load effects and the factored strength or resistance. The rating procedure (equation 2.4) is carried out for all strength checks (moment, shear and reaction) at all potentially critical sections with the lowest value determining the rating factor for the entire span.

The rating factor is the ratio of the safe level of loading to the load produced by the nominal or standard vehicle. It shall be used in the consideration of posting levels and/or the consideration and justifications for future repairs or replacement [8].

2.1.6 Capacity Evaluation Steps

The procedure for rating existing bridges requires knowledge of the physical conditions of the bridge and the applied loading. In determining load and resistance factors for the rating equation, the following steps shall be carried out in evaluating a bridge span [8]:

1) Collection of information
2) Selection of nominal loadings and resistances
3) Distribution of loads
4) Selection of load and resistance factors
5) Calculation of rating factors

Since neither resistance nor the load effect can be established with certainty, safety factors must be introduced that give adequate assurance that the limit states are not exceeded. This shall be done by stating the equation in a load and resistance factor (LRFD) format.

The basic rating equation used in the guidelines is simply a special form of the basic structural engineering equation with load and resistance factors introduced to account for uncertainties that apply to the bridge evaluation problem. It is written as follows [8]:

$$\mathrm{RF} = \frac{\emptyset R_n - \sum_{i=1}^{m} {\gamma_i}^{D} * D_i - \sum_{j=1}^{n} {\gamma_j}^{L} * L_j (1+I)}{\gamma^{LR} L_R (1+I)} \qquad (2.4)$$

Where RF= rating factor (the portion of legal truck allowed on the bridge)

$\emptyset$=resistance factor

m= number of elements included in the dead loads

R_n= nominal resistance

n=number of live loads other than the rating vehicles

${\gamma_i}^D$=dead load factor for element "i"

D_i=nominal dead load of element "i"

${\gamma_j}^L$=live load factor for live load "j" other than rating vehicle(s)

L_j=nominal traffic live load effects for load "j" other than rating vehicle(s)

γ^{RL}=live load factor for rating legal truck

L_R=nominal live load effect for the rating legal truck

I=live load impact factor

2.1.6.1 Collection of Information

Before the load rating of a specific bridge can be conducted, a certain amount of information has to be gathered [8].

The following items can have an influence on the selection of load and resistance factors [8].

1. Deck condition – Field tests have shown that the single 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.
2. Structural Condition - Signs of recent deterioration in structural members, which may go unchecked and increase the likelihood of further section capacity loss before the next cycle of inspection and rating should be noted.
3. Traffic Condition - The expected loading during the inspection interval is affected by the truck traffic at the site. Advice should be sought from the traffic division regarding truck traffic volume, composition, permit activities, overload sources, and degree of enforcement.

2.1.6.2 Selection of Nominal Loading and Resistances

For bridge evaluations, the most important loads are dead load and vehicular live load plus its accompanying dynamic effects, since each of these loadings induce high superstructure stresses.

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. When other combinations of loads can affect the capacity of the bridge such as when substructure components can fail due to traffic live loading, proper investigation should be carried out to check its safety [8].

1. Dead Loads

The dead load shall be estimated from data available from the inspection at the time of analysis. The dead load factor accounts for normal variations of material densities and dimensions. Nominal dimensions and densities shall be used for calculating dead load effects. For overlays, either cores shall be used to establish the true thickness or an additional allowance of 20% should be placed on the nominal overlay thickness indicated at the time of analysis. The recommended unit weights of materials used in computing the dead load shall be as provided in Table 2.5 [8].

Table 2.6Unit Weights of Materials [8]

MATERIAL	FORCE EFFECT [kN/m^3]
Asphalt surfacing	22.5
Concrete, plain or reinforced (normal weight)	24.0
Steel	79.0
Cast iron	72.0
Timber (treated or untreated)	8.0
Earth (compacted), sand gravel or ballast	18.0

The dead load of the structure is computed in accordance with the conditions existing at the time of the analysis. Dead load can usually be determined more accurately than any other type of loading.

Items that can affect the calculation of dead load are dimensional variations in the concrete section and variations in the unit weight of material. This is because the material unit weight may not be constant along the element and/or its dimension may vary because of construction defects.

The prescribed dead load factor recognizes the uncertainties in the nominal dimensions and analysis of dead load effects. Overlay thicknesses are a source of greater uncertainty in the dead load so they are assigned a 20% higher load factor unless cores or more detailed measurements are made [8].

2. Live Loads

The guidelines specify the number of vehicles to be considered on the bridge at any one time. These numbers are based on an estimate of the maximum likely number of vehicles under typical traffic situations.

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. The variation will usually be greater than the variation in dead load effect. To minimize this difference, it is necessary to select a rating Legal Truck with axle spacing and relative axle weights similar to actual vehicles. Three Legal Trucks shown in Figures 2.2 to 2.4 are recommended as evaluation vehicles. These vehicles, together with the prescribed live load factors, give a realistic estimate of the maximum live load effects of a variety of heavy trucks in actual traffic. For longer spans, a lane loading is specified in the evaluation, shown in Figure 2-5.Reduction factors for live loading of more than two traffic lanes are provided. These rationally account for the lower possibility of such occurrences [8].

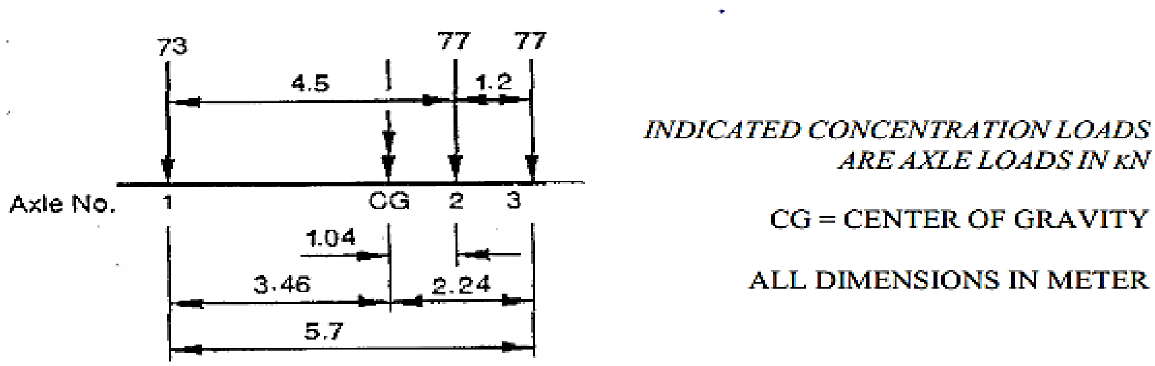

Figure 2.2 Truck Type 3-1 Unit Weight = 227 kN

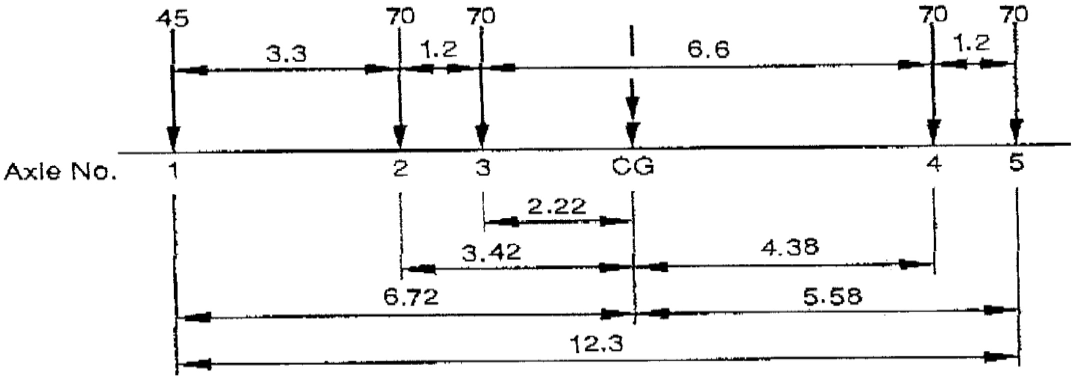

Figure 2.3 Truck Type 3-2 Unit Weight = 325 kN

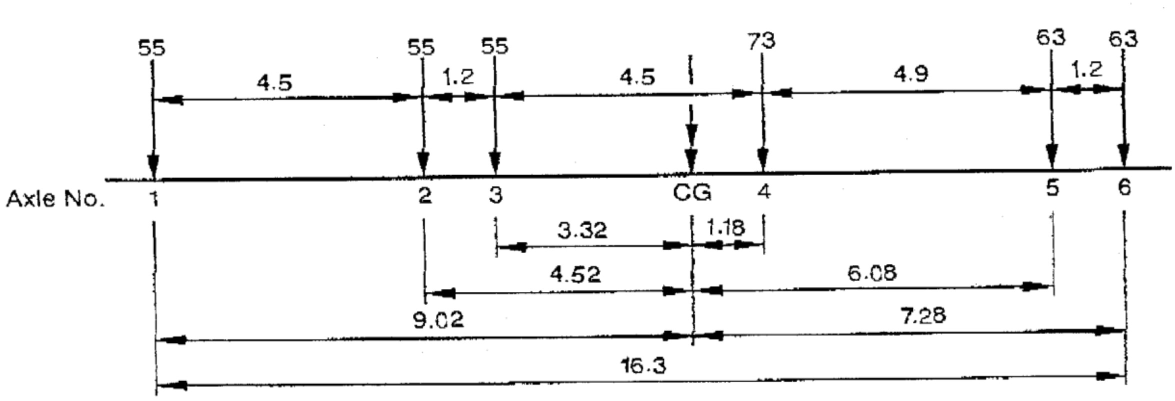

Figure 2.4 Truck Type 3-3 Unit Weight = 364 kN

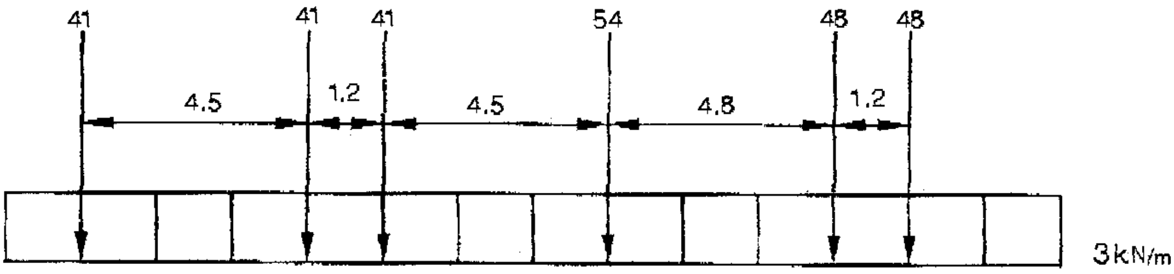

Figure 2.5 A Legal Lane Loading (mainly for large spans)

For longer spans, the Legal Lane Loading given in Figure 2-5 will govern the evaluation (upto90m).This is a combination of a vehicle load and a uniformly distributed load. For all span lengths where the rating factor is less than one, it shall be necessary to place more than one, it shall be necessary to place more than one vehicle in each lane. In lieu of this, the evaluator should check the lane loading for all span lengths together with the rating Legal Truck as shown in Fig.2.5 [8].

3. Impact

An impact allowance shall be added to the static loads used for rating. Impact values are used for design of new bridges to reflect conservative conditions that may possibly prevail under certain circumstances. Under an enforced speed restriction, impacts shall be reduced.

Impact loads are taken to be primarily due to the roughness or unevenness of the road surface, especially the approach spans. Three values of impact factors are provided by correlating the roughness of the surface to the deck conditions survey values. This information is more likely known during evaluation than in the original design [8].

For smooth approach and deck conditions, the impact shall be taken as 0.10. For a rough surface with bumps, a value of 0.20 should be used. Under extreme adverse conditions of high speed, spans less than 12m and highly distressed pavement and approach conditions, a value of 0.30 should be taken [8].

If such a judgment cannot be made, refer to the bridge inspection report and relate impact to the condition of the wearing surface.

Table 2.7Condition of Wearing Surface and Impact Value [8]

WEARING SURFACE		IMPACT
1 -Good condition	No repair required	0.1
2 – Fair condition	Minor deficiency, item still functioning as designed	0.1
3 -Poor condition	Major deficiency, item in need of repair to	0.2
4 -Critical condition	Item no longer functioning as designed	0.3

4. Resistances

The shear force and moment resisting strength of members are calculated based on current condition of the bridge. Standard formulas using Load and Resistance Factor Method are used to calculate these values. ERA Bridge design manual specifies that "Unless otherwise stated, the Ethiopian Building Code Standard, Vol. 2 Structural Use of Concrete, 1995, shall be used." [8]. 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.

4.1 Shear Strength

The shear resistance consists of a component which depends on the concrete and a component which relies on tensile stresses in the transverse reinforcement. The nominal shear resistance is determined as the lesser of:

$$V_n = V_c + V_s \tag{2.6}$$

Or

$$V_n = 0.25f'_c b_v \tag{2.7}$$

For which Vs and Vc were given below

$$\mathrm{V_c} = 0.083\beta\sqrt{\mathrm{f'_c}}\mathrm{b_v d_v} \tag{2.8}$$

$$\mathrm{V_s} = \frac{\mathrm{A_v f_y d_v}(\cot\theta + \cot\alpha)\sin\alpha}{\mathrm{S}} \tag{2.9}$$

Where: b_v = effective web width taken as the minimum web width within the depth dv (mm)

d_v = effective shear depth (mm)

S = spacing of stirrups (mm)

β = factor indicating ability of diagonally cracked concrete to transmit tension

θ = angle of inclination of diagonal compressive stresses

α = angle of inclination of transverse reinforcement to longitudinal axis

A_v = area of shear reinforcement within a distance s (mm^2)

For non-prestressed concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement specified or having an overall depth of < 400 mm, the following values shall be used.

When the values of β =2.0 and $\theta = 45^0$ taken, the shear strength can be expressed as:

$$\mathrm{V_C} = 0.166\mathrm{b_v d_v}\sqrt{\mathrm{f'_C}} \tag{2.10}$$

4.2 Flexural Strength

Flexural strength is calculated by taking the rectangular stress block as shown in Figure 2.6.

$$\mathrm{M_n} = \mathrm{A_s f_y}(\mathrm{d} - \frac{\mathrm{a}}{2}) \tag{2.11}$$

Where a= cβ1 is center of stress block, A_s=area of tensile bars, fy=rebar yield strength, d=depth from top extreme fiber to center of tensile rebar M_n=nominal flexural resistance of the section.

Moreover the area of tension steel to be used in computing the ultimate flexural strength of reinforced concrete members is that available in the section or 75 percent of the steel reinforcement required for a balanced condition, assuming a rectangular stress block.

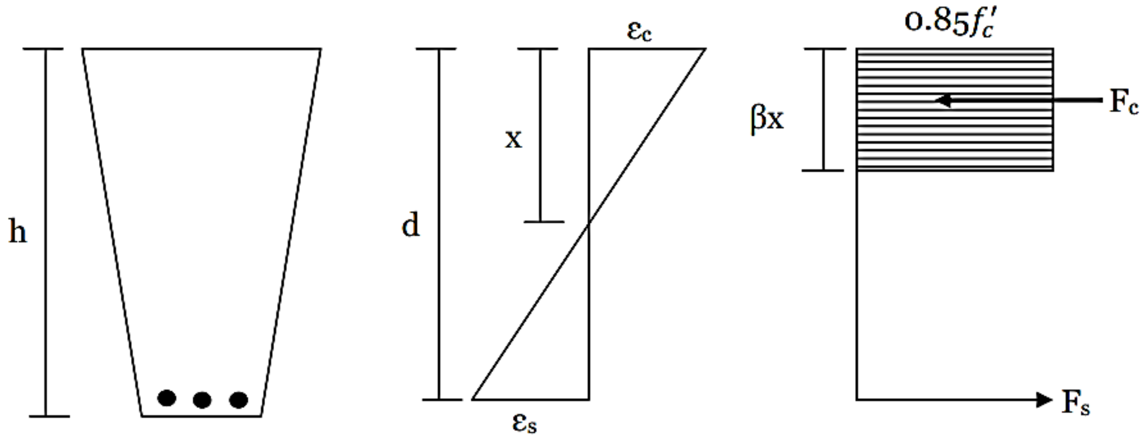

Figure 2.6 Reinforced concrete beams stress diagram [8]

The depth of the neutral axis can be solved as:

$$\beta_1 = \begin{cases} 0.85 \quad for\ f'_c \leq 28\ MPa \\ 0.85 - 0.05\left(\dfrac{f'_c - 28}{7}\right)\ for\ 28\ MPa\ \leq f'_c \leq 56\ MPa \\ 0.65\ for\ f'_c \geq 56\ MPa \end{cases} \qquad (2.12)$$

The material strengths to be used in the resistance calculation of the existing concrete and reinforcing steels were specified by ERA design manual as below.

Concrete: 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 reduced ultimate strength shall be assumed (no less than 15 MPa, however) for unsound or deteriorated concrete unless evidence to the contrary is gained by field-testing [8].

Reinforcing steel: The area of tension steel to be used in computing the ultimate flexural strength of reinforced concrete members shall not exceed that available value in the section or 75 percent of the steel reinforcement required for a balanced condition. The steel yield stresses to be used for various types of reinforcing steel are given below.

Table 2.8 Reinforcing Steel Yield Stresses [8]

Reinforcing Steel	**Yield Stress F_y(MPa)**
Unknown steel (prior to1954)	228
Structural Grade	248
Intermediate Grade 300 and unknown after 1954(former Grade40)	276
Hard Grade (former Grade 50)	314
Grade 420 (former Grade 60)	614
Grade 520 (former Grade 75)	517

The determination of structural resistance is one of the primary tasks in the evaluation process. In a load and resistance design (LRFD - also known as limit state) approach it is necessary to define the condition at which resistance will be determined. These should provide for similar structural performance regardless of the material or structure type.

2.1.6.3 Distribution of Loads

The fraction of vehicle load effect transferred to a single member is selected in accordance with the specification given in ERA Bridge Design Manual. These values represent a possible combination of adverse circumstances. The option exists to substitute field measured values, analytically calculated values or those determined from advanced structural analysis methods utilizing the properties of the existing span(s) [8].

The lateral load distribution for interior and exterior girder is calculated by using the approximate method given in ERA Bridge Design Manual.

Distribution of live load per lane for moment in INTERIOR longitudinal girder:

• When one design lane loaded:

$$\text{gm}_1 = 0.06 + \left(\frac{S}{4300}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{Lt_s^{\,3}}\right)^{0.1} \qquad (2.13)$$

• When two or more design lanes loaded**:**

$$\text{gm}_2 = 0.075 + \left(\frac{S}{4300}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{Lt_s^{\,3}}\right)^{0.1} \qquad (2.14)$$

Where $K_g = n(I + A.e_g^2)$ and $n = \frac{I_B}{I_D}$

Distribution of live load per lane for moment in EXTERIOR longitudinal girders

- When one design lane loaded: lever rule is used
- When two or more design lanes loaded:

$$g = e * g_{interior} \tag{2.15}$$

$$e = 0.77 + \frac{de}{2800} \tag{2.16}$$

Distribution of live load per lane for shear in INTERIOR longitudinal girders

- When one design lane loaded

$$gv1 = 0.36 + \frac{S}{7600} \tag{2.17}$$

- When two or more design lanes loaded

$$gv2 = 0.2 + 0.36 + \frac{S}{3600} - (\frac{S}{10700})^2 \tag{2.18}$$

Distribution of live load per lane for shear in EXTERIOR longitudinal girders

- When one design lane loaded: lever rule is used
- When two or more design lane loaded

$$g = e \text{ x } gv_{interior} \tag{2.19}$$

$$e = 0.6 + \frac{de}{3000} \tag{2.20}$$

Where***:***

L = span of beam (mm)

S = spacing of supporting components (mm)

ts = deck slab thickness (mm)

Kg = longitudinal stiffness parameter (mm^4)

e = correction factor for distribution; eccentricity of a lane from the center of gravity of the pattern of girders (mm)

g = distribution factor

d_e = distance from the exterior web of exterior beam to the interior edge of curb or traffic barrier (mm)

n = modular ratio between beam and deck

E_B =modulus of elasticity of beam material (MPa)

E_D =modulus of elasticity of deck material (MPa)

I = moment of inertia of beam (mm^4)

e_g = distance between the centers of gravity of the basic beam and deck (mm)

A = Area of concrete (mm^2)

Table 2.9 Correction Factor for Analysis* [8]

		Correction Factor*		
		Steel	Prestressed	Concrete
1	AASHTO Distribution, Chapter 13	1.00	1.00	1.00
2	Tabulated analysis with simplified assumptions	1.10	1.05	0.95
3	Refined analysis: finite elements, orthotropic plate, grillage analogy	1.07	1.03	0.90
4	Field measurements	1.03	1.01	0.90
Actual girder distribution shall be multiplied by the appropriate correction factors to obtain the girder distribution for rating.				

*Correction factors are applied if average or expected values are used for R.F. from analysis or measurements.

The correction factor shall be used to increase the load factor taken from Table 2-8.

** These correction factors reflect the bias in present Volume I distribution factors for each material type.

2.1.6.4 Selection of Load and Resistance Factors

The statistics of the dead load, live load and resistances have been determined from existing data. Based on this data, the safety implicit in current designs has been determined. The load and resistance factors provided ensure that an acceptable level of safety is achieved or exceeded [8].

1. **Load Factors:** The load factors used for rating of bridges are those shown in Table below. These load factors are intended to represent actual traffic conditions. They are based on field data obtained from a variety of locations using weight-in-motion and other data gathering methods.

Table 2.10 Load Factors [8]

Loading		Load Factor
Dead Load		γ_D = 1.2
Allow an additional allowance of 20% on overlay thickness if nominal thicknesses are used. No allowance is needed when measurements are made for thickness.		
1	Low volume roadways (ADTT less than 1000), reasonable enforcement and apparent control of overloads	γ_D = 1.30
2	Heavy volume roadways (ADTT greater than 1000), reasonable enforcement and apparent control of overloads (not common in Ethiopia)	γ_L = 1.45
3	Low volume roadways (ADTT less than 1000), significant sources of overloads without effective enforcement (common in Ethiopia)	γ_L = 1.65
4	Heavy volume roadways (ADTT greater than 1000), significant sources of overloads without effective enforcement	γ_L = 1.80

If unavailable from traffic data, estimates for ADTT shall be made from ADT as follows: urban areas, ADTT =25% of ADT; rural areas, ADTT - 50% of ADT. In the absence of accurate data on overloads, it shall be assumed that 30% of the trucks in Ethiopia exceed the local legal gross weight limits.

Dead load factor reflects variations in dimensions, unit weights and methods of calculating dead load effect. The variation in the dead load of different components will depend on the accuracy with which the components can be manufactured and/or measured. The higher dead load factor for asphalt recognizes the greater uncertainty in overlay thickness.
Live load factors have been provided to account for the large uncertainty of the maximum live load effects on a structure over a period of time [8].

2. **Resistance Factors**

A capacity reduction factor (φ) is included in the basic rating equation to account for variation in the calculated resistance. It 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.

Resistance (capacity reduction) factors are to be applied to the following for the case where members are in good condition.

• Redundant Steel Members: Φ = 0.95; Non-redundant Steel Members: Φ = 0.80; Pre-stressed concrete beams: Φ = 0.95; Reinforced concrete beams: Φ = 0.90.

2.1.6.5 Calculation of Rating Factors (RF)

The rating factor is calculated from Equation 2.1. If it exceeds 1.0, the span is satisfactory for the legal loads in Ethiopia. The rating factors obtained herein may also safely be applied to permit loadings. In some instances where a permit might otherwise be rejected, the live load factors contained herein shall be reduced to reflect known weight conditions associated with the permit vehicle. This reduction in load factor may depend on the degree of control of the permit and the number of permits that shall be issued [8].

2.2 Bridge Fatigue Evaluation

2.2.1 Introduction

Fatigue is defined as a phenomenon of weakening of strength of materials subjected to cyclic loads. It occurs in all materials exposed to cyclic stresses of variable magnitudes. Although fatigue failure has been seldom reported to date, considerable interest has developed in the fatigue behavior of reinforced concrete members recently. There are some reasons for this interest. The widespread adoption of ultimate strength design procedures and the use of higher strength and more durable materials require that structural concrete members perform satisfactorily under high stress levels for a longer period of time. There is also a new recognition of the effects of repeated loading on a member. Repeated loading may lead to internal cracking of a member that alters its stiffness and load-carrying characteristics [9]. As time goes, the cracks sizes will increase by the cyclic loadings. This creates a stress in reinforcement bar imbedded in the RC members. Hence, the fatigue of reinforcement bars is sufficiently considered as controlling the fatigue performances of concrete members [10]. Fatigue life of steel reinforcement can be divided into different failure phases as described next.

2.2.2 Fatigue Failure Stages

Fatigue failure in reinforcing bars is due to crack formation and propagation. This failure has three distinct stages: crack initiation, crack propagation, and sudden fracture.

For ductile metals, like those composing steel reinforcing bars, the crystal grains are oriented in a fashion in which slip bands easily occur at the grain boundaries due to the applied stress. As the applied stress is cycled, these slip bands extend leading to initiation of a crack. Crack initiate at discontinuities or notches [11].

Once a crack is formed, it propagates perpendicular to the applied stress; in the case of tensile rebar in flexural members, the cracks propagate transversely due to the tensile forces developed. The crack will continue to propagate as long as the stress intensity factor range is above the threshold value.

After the crack propagates to a sufficient degree, the cross-section of the component is effectively decreased to the point where the applied load induces a stress no longer below the ultimate strength of the material, and fracture occurs. The fracture stage of fatigue failure frequently occurs with no warning [11].

2.2.3 Fatigue of Reinforced Concrete Structures

Concrete bridges are widely used in almost all countries, accounting for larger percent of all highway bridges. Fatigue is not a major issue in the design of most concrete structures. Yet, there have been no fatigue failures reported in concrete structures under normal service loading. Fatigue is generally relevant for steel bridges for very long period. However, research by [12] stated that with the rapid development of highway transportation, traffic volume and vehicle weight, it is impossible to neglect fatigue damages in concrete bridges.

The stresses due to these loads may cause fatigue in the structures and result in premature failure [3]. As a result, fatigue analysis need to be taken in to account for older bridge evaluation because they are subjected to time dependent variable loads from vehicles.

Generally, fatigue damage in concrete structures is a complex area and not as well researched as in steel. The fatigue performance of a reinforced concrete member depends on the composite action between steel and concrete. Whereas an under reinforced member has its flexural fatigue performance dominated by the steel bar, a heavily reinforced member may fail in flexure or shear depending on whether the concrete or steel strength is critical. Fatigue of concrete and reinforcement bars is discussed separately [12].

2.2.3.1 Fatigue of Steel Reinforcement

The fatigue strength of reinforcing steel is a vital parameter on the resistance side of reinforced concrete members subjected to cyclic loading [1]. The fatigue behavior of reinforcement bar and pre-stressing steel reinforcement is similar to that of steel structures [13]. For steel reinforcement, the fatigue relevant parameters are:

(1) **The stress ranges $\Delta\sigma$**. Due to stress concentrations that always are present; the maximum stress level will mostly be the yield stress. The stress range will thus always have its maximum value at the yield stress and any calculated mean stress has little or no influence.

(2) **The number of load cycles, N** and (3) **discontinuities** both in the cross section and layout of the steel reinforcement resulting in stress concentration at possible fatigue damage location.

The fatigue behavior of the reinforcement can be represented by means of the S-N-diagram (Wöhler line) in a double-logarithmic representation as in Figure 2.7. Test results are plotted using double-logarithmic scale and, usually, a 5% fractile-criterion is used to determine the slope of S-N curve and the detail category defined as the fatigue strength at 2 million load cycles [1].

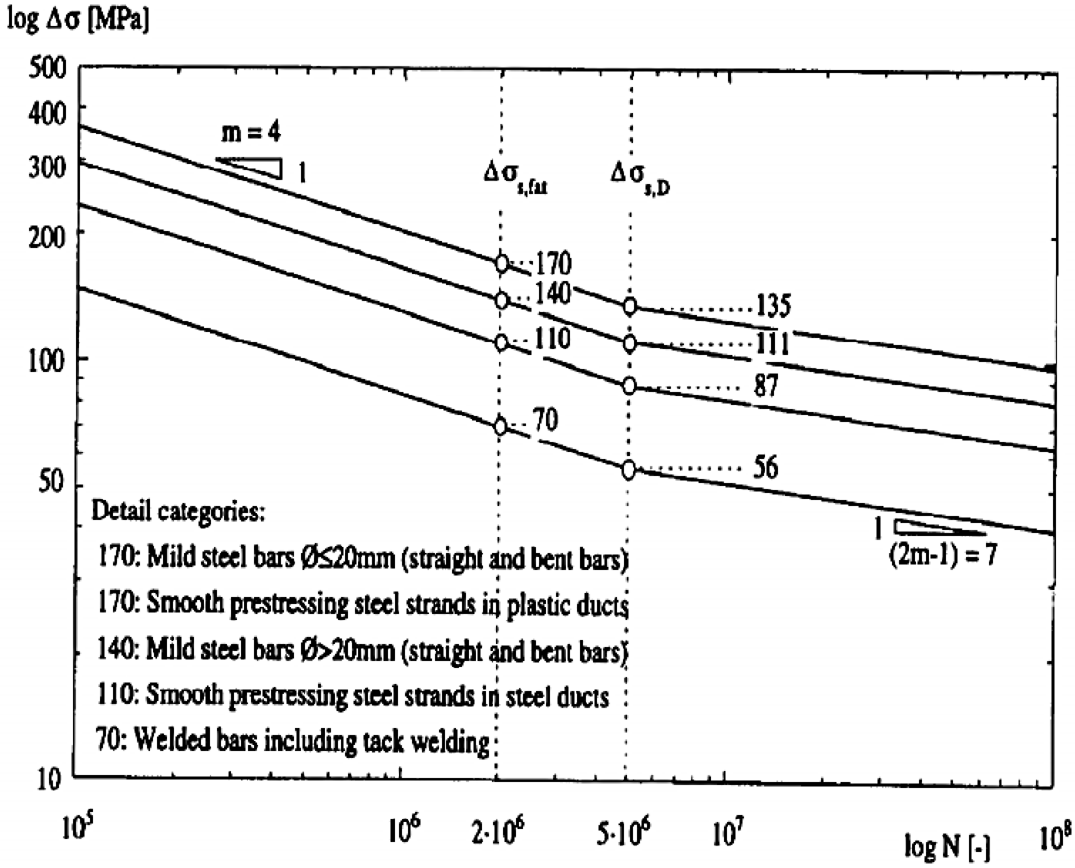

Figure 2.7 Fatigue strength of steel reinforcement [1]

2.2.3.2 Fatigue of concrete

Fatigue resistance of concrete is defined by a pair of stresses, that is the maximum and minimum stress values as the most important fatigue relevant parameters. The effect of this pair of stresses as a function of load cycles is best represented by Goodman diagram (Fig.2.8). Other fatigue relevant parameter include the concrete strength and structural size effect which are taken into account by the nominal design values fc and τc for static compressive and shear strength, respectively. The fatigue action effect in the concrete is described by the maximum and minimum stress values due to fatigue loading and dead load of the structure including permanent loads [1].

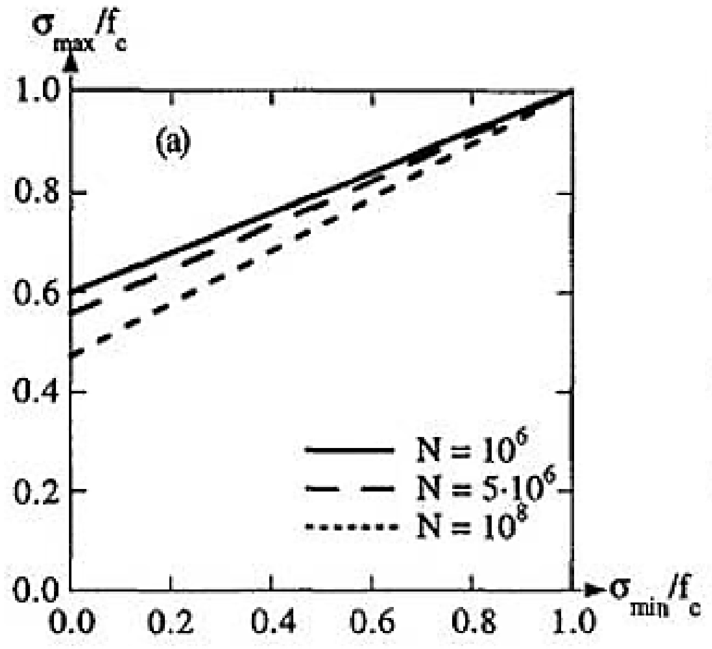

Figure 2.8 Fatigue strength (compressive) diagram for concrete [1]

2.2.4 Fatigue life evaluation Methods

This section introduces two fatigue analysis methods such as stress-life (S-N) method and the linear elastic fracture mechanics approach. The two methods were studied because they are commonly employed to evaluate the fatigue life of bridges.

2.2.4.1 Fatigue life evaluation based on S-N curves

The first approach to fatigue assessment is based on stress S versus number of load cycles N curves (S-N curves). These curves are constructed by testing numerous specimens at different stress ranges and determining the number of cycles it takes to fail the specimen. By varying the stress amplitude, the number of cycles to failure also varies allowing different points on the curve to be plotted, the higher the stress amplitude the fewer cycles it takes to reach failure, and vise-versa [1]**.** This stress-based approach involves establishing an empirical relationship between stress range amplitudes and number of cycles to failure.

There some formula previously derived to predict a fatigue life of rebar, based on S-N curves, among which one is given in equation 2.21. In the calculation of fatigue life of steel bars, there are basically two things needed; the fatigue strength of the reinforcement bar and the stress range in the reinforcement [14].

Moss *et al*. (1982) [14], derived the following fatigue life relationship (a relationship between stress range and cycles to failure) from analysis of experimental results for axially and laterally loaded reinforcing bars embedded in concrete:

$$N_f \sigma_r^m = K \tag{2.21}$$

Where σ_r = stress range within tensile reinforcing steel bar in MPa; N_f = number of cycles to failure; m = inverse slope of S-N curve. K=is mean line of S-N curve relationship which varies depending on loading and bar diameter, given in table 2.10.

Table 2.11 Values of K for different action and bar diameter [14]

Type of loading	$K \times 10^{27}$	
	16 mm diameter	32 and 40 mm diameter
Axial	0.75	0.11
Flexural (bending)	3.09	0.31

2.2.4.2 Fatigue Life Evaluation Based On Fracture Mechanics

The use of fracture mechanics requires the determination of the material's fracture toughness, nominal stress range, flaw size, and geometry. The stress field near the tip of a crack is characterized by the stress intensity factor, KI, having units of MPa√m. This factor takes into account the nominal stress, σ, crack size, a, among other factors, Y.

The fatigue life of rebar can be predicted based on the principles that the stress state near the crack tip is described by a single parameter, the stress intensity factor, K or under cyclic loading condition the stress intensity factor range, ΔK [15] as:

$$\Delta K = Y.\Delta\sigma.\sqrt{\pi.a} \tag{2.22}$$

Where Δσ= is applied cyclic stress range, Y= is a shape factor that depends on the crack geometry and a=is crack size

The cyclic stress intensity factor ΔK associated to the Paris law provides the number of fatigue cycles to propagate a crack under an applied stress range.

In order to determine how long it will take a crack, once detected, to reach its critical length, it is useful to determine the crack propagation rate. The fatigue crack growth rate is essentially the increase in crack length (a) per cycle (N) resulting in the ratio (Da/DN). However, since the change in length per cycle is small, the growth rate can be considered as the derivative, da/dN. In 1964 Paris proposed the Paris Law, which correlates the crack propagation rate, da/dN, and the stress intensity factor as described in equation as:

$$\frac{da}{dN} = C.(\Delta K)^m \qquad (2.23)$$

Where C and m are material constants and the range of stress intensity actor, ΔK is determined as equation (2.22) above.

To study the fatigue crack propagation of the rebar using the elastic fracture mechanics approach, the presence of an initial flaw, a_o on a cylindrical steel bar in form of a semi-circular crack at the surface and perpendicular to the steel bar axis is assumed (Fig. 2.9). Stable crack growth is assumed from the initial flaw and the Paris law is applied for the crack growth calculations. Accordingly, fracture of rebar occurs when the depth of crack reaches the critical crack depth $a=a_{cr}$ or the applied stress is equal to the resistance of the remaining cross section [2].

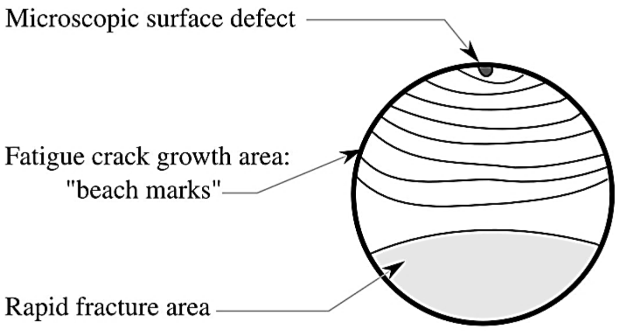

Figure 2.9 Rebar cross section with initial flaw and crack at fracture [21]

The critical crack depth could be determined from equation (2.22) by substituting $Y=Y_{cr}$, $a=a_{cr}$ and $K=K_{IC}$ as below:

$$a_{cr} = \frac{1}{\pi}\left(\frac{K_{IC}}{Y_{cr}\sigma_{max}}\right)^2 \qquad (2.24)$$

Where σ_{max} =is the maximum stress of dead and live load.

The fracture toughness is determined experimentally from pre-cracked specimens.

The shape factor Y for a semicircular crack in round bars is given by the expression (BS7910, 1999) [11]:

$$Y = \frac{\frac{1.84}{\pi}.\left[\frac{\tan(\pi a/4r)}{(\pi a/4r)}\right]^{0.5}}{\cos\left(\frac{\pi a}{4r}\right)}.\left[0.752 + 2.02\left(\frac{a}{2r}\right) + 0.37\left\{1 - \sin\left(\frac{\pi a}{4r}\right)\right\}^3\right] \qquad (2.25)$$

Where a is the flaw/crack depth and r is the radius of the bar.

The relation between the crack propagation rate and the stress intensity factor range is made up of three regions: threshold region (Region-I), steady growth (Region-II), and unstable growth/fracture (Region-III) as shown in Figure 2.10.

In Region I, a threshold stress intensity factor, ΔK_{th}, is shown below which crack growth cannot be detected. Below ΔK_{th}, cracks do not grow under cyclic loading. In Region III, the crack growth rate is so high that cracks grow rapidly until the component fractures. Because cracks grow so fast in Region III, the crack growth behavior in this region does not significantly affect the total fatigue life. Region II is the most important region involving crack propagation that affects fatigue analysis.

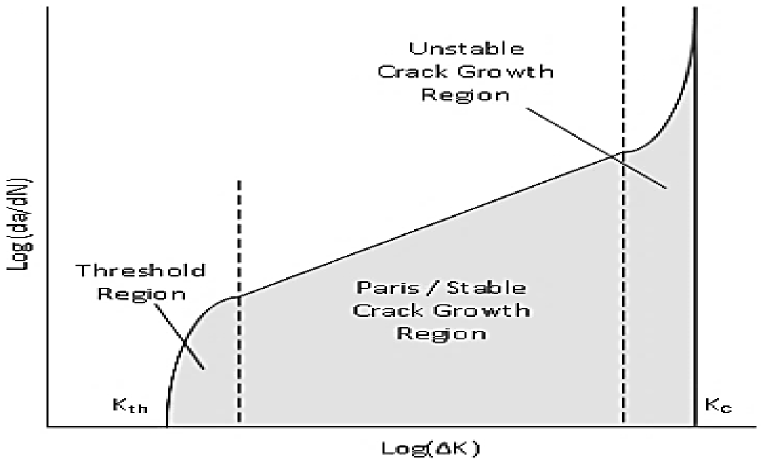

Figure 2.10 Crack Growth Rate versus Stress Intensity Factor Range [11]

The method of linear elastic fracture mechanics (LEFM) relates the growth of an initial crack of size a to the number of fatigue cycles, N_f.

The Paris Law, Equation 2.23, can be rewritten so that the number of fatigue cycles from an initial crack length to the critical crack length can be determined. An integration procedure must be utilized to compute the number of cycles to failure, N_f, it takes for a crack to grow from an initial crack size, a_o, to a failure crack size, a_f, it follows equation (3.8) [2]:

$$N_f = \int_{a_i}^{a_{cr}} dN = \int_{a_i}^{a_{cr}} \frac{1}{C.\Delta K^m} da = \int_{a_i}^{a_{cr}} \frac{1}{C.Y^m.\Delta\sigma^m.\pi^{m/2}.a^{m/2}} da \quad (2.26)$$

Where K_{IC} is Fracture toughness of steel bar in concrete and N_f is the number of fatigue cycles from the initial cycle to the final cycle. It is also known as total fatigue life in cycles.

The number fatigue cycles to unstable crack growth is derived from Paris law, above as:

$$N_f = \frac{2 * \left(a_{cr}^{\frac{2-m}{2}} - a_i^{\frac{2-m}{2}}\right)}{(2-m)C\left(Y\Delta\sigma\sqrt{\pi}\right)^m} \tag{2.27}$$

2.2.5 Fatigue load model

The loading model is an important parameter in a fatigue evaluation. The fatigue truck is typically used to represent truck traffic at a given site with a variety of gross weights and truck configurations. Its configuration should be selected so that the fatigue damage caused by the fatigue truck is the same as the fatigue damage caused by actual truck traffic with an equivalent number of passages [16]. Researchers have modeled many fatigue trucks with different axles as representative trucks, among which one is discussed below because it is used in this thesis.

2.2.5.1 AASHTO Fatigue Truck Model

AASHTO Guide Specifications (1990) stipulate a 54-kip (240 kN) gross weight of the fatigue truck for a basic evaluation procedure. This gross vehicle weight represents the actual truck traffic spectrum obtained from WIM studies, from more than 27,000 trucks and 30 sites. The AASHTO fatigue truck model has front and rear axle spacings of 14 ft (4.3 m) and 30 ft (9.1 m), respectively, with a 6-ft (1.83 m) axle width, as shown in Figure 3.4. The National Cooperative Highway Research Program Report 299, which is based on extensive nationwide WIM data, states that this truck can be used for bridges located in routes for which no WIM data is available.

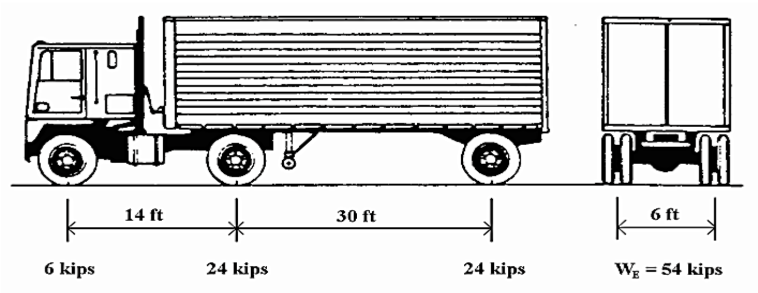

Figure 2.11 AASHTO Fatigue Truck (AASHTO, 1990)

2.2.6 Method of Analysis

There are several methods of bridge analysis, including line girder analysis and more refined methods, such as the finite element methods. The girder line analysis method is discussed below because it was utilized in this thesis.

2.2.6.1 Line Girder Analysis Methods

This method is based on the assumption that the maximum load effects on a girder or strip of unit width (in the case of a slab bridge), can be determined by treating it as a one dimensional beam subjected to the load of one line of wheels of the design vehicle multiplied by a load fraction. The distribution factors sometimes known as girder distribution factors are used and applied to distribute wheel loads to adjacent girders. Girder distribution factor (GDF) is defined as the ratio of the load effect in a girder to the total moment or shear force. The purpose of the distribution factors used in the 1998 AASHTO LRFD Standard Specifications method of lateral distribution is to reduce the complex analysis of a bridge subjected to one or more vehicular loads to a simple analysis of a beam. The girder distribution factor can be applied to one dimensional-analysis method to obtain the moment or shear value per girder [16].

Once the total design fatigue life in cycles is determined, to calculate the remaining fatigue life, R, in years, following steps a, b, c, and d below need to be followed [16]:

a. Determine the past growth factor, GF1. This may be estimated or calculated provided that the ADTT values for two separate years are known.

$$\mathrm{GF1} = \sqrt[n_3]{\frac{\mathrm{ADTT}(n_2)}{\mathrm{ADTT}(n_1)}} - 1 \tag{2.28}$$

b. Calculate the ADTT for the year the bridge was built,

$$\mathrm{ADTT(year\ built)} = \frac{\mathrm{ADTT}(n_2)}{(1+\mathrm{GF1})^n} \tag{2.29}$$

c. Calculate the number of cycles, M, accumulated up to year n_2,

$$M = 365\,\frac{\mathrm{days}}{\mathrm{year}} * [\mathrm{ADTT(year\ built)}] * \frac{(1+\mathrm{GF1})^n - 1}{\mathrm{GF1}} \tag{2.30}$$

d. Determine the future growth factor, GF2. This may be estimated or calculated provided that the estimated ADTT for the future year, n_f, is entered.

$$\mathrm{GF2} = \sqrt[n_4]{\frac{\mathrm{ADTT}(n_f)}{\mathrm{ADTT}(n_2)}} - 1 \tag{2.31}$$

e. Calculate the remaining fatigue life, R, in years,

$$R = \frac{\ln\left[\frac{(N_f - M)*\mathrm{GF2}}{365\frac{\mathrm{days}}{\mathrm{year}}*\mathrm{ADTT}(n_2)*(1+\mathrm{GF2})} + 1\right]}{\ln[1+\mathrm{GF2}]} \tag{2.32}$$

Where n_1 = previous year n_2 = recent year $n_3 = n_2 - n_1$ $n = n_2$ - year built $n_4 = n_f - n_2$.

CHAPTER THREE

3. METHODOLOGY AND MATERIALS

3.1 Study area

The proposed bridge of this study is found in Eastern Wollega Zone between Bako and Nekemte town, at 289.36 km from Addis Ababa. The bridge is called Oda Bridge.

3.2 Description of the bridge

Figure 3.1 shows Oda Bridge, a two lane girder bridge constructed in 1954 E.C by Italians. It is consisted of a series of three simple spans with a multi-girder reinforced concrete deck girder bridge. The bridge was constructed on a curved road from three straight spans. It has a carriageway width of 7.00 meters, each span 12 meters long through 4 main longitudinal girders. The girders are 0.86 meters deep and 0.40 meter wide spaced at 2.33 meters. The slab thickness of the bridge is 0.20 meters.

Figure 3.1 Side View of Oda Bridge

3.3 Data collection process

Both the primary and secondary data that show the present condition of the bridge are collected. The secondary data are taken from inspection reports provided by ERCC Nekemte District. For primary data collection, non-destructive tests (NDT) like hammer testing and visual inspections are employed on the bridge. Then, necessary information was collected. The following methods were utilized to do so.

3.3.1 Visual Inspection

Visual inspections are commonly used nowadays. Visual testing is probably the most important of all non-destructive tests. It is very important to know the recent condition of the bridge under consideration. Due to this, visual inspection has done to Oda Bridge and information such as cracks, spalling, disintegration, colour change and surface blemishes are gathered to indicate the condition of the structure. In addition, number of girders, spans, lanes, type of railings, and element dimensions are measured. But damages inside the structure that are not visible are difficult to identify. These are major drawback of the method.

3.3.2. Tools for Visual Inspection

The visual survey was equipped with tools to facilitate the inspection. These are notebook, measuring ruler and camera.

Figure 3.2 Taking dimensional Measurements of Oda bridge

3.3.3 Rebound Hammer Testing

The rebound hammer is a simple, handy tool which is used to measure the hardness and predict the strength of the concrete. It is a spring-loaded impacting device that incorporates a scale to measure the energy of the rebound following the impact. The extent of rebound gives an indication of the strength of the concrete at the surface position tested. But the internal cracks, flaws or heterogeneity across the cross section will not be indicated by rebound numbers. This is a major defect of this method.

3.4 Principle of Testing

The method is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which mass strikes. When the plunger of rebound hammer is pressed against the surface of the concrete, the spring controlled mass rebounds and the extent of such rebound depends upon the surface hardness of concrete.

For taking a measurement, the hammer should be held at right angles to the surface of the structure. The test thus can be conducted horizontally on vertical surface and vertically upwards or downwards on horizontal surfaces. The average of about 10 to 20 impacts would give an approximate indication as to the compressive strength of concrete at that location. If one reading differs from the average by plus or minus ten, the number should be ignored and the average value should be recalculated.

The surface hardness and therefore the rebound are taken to be related to the compressive strength of the concrete. The rebound value is read off along a graduated scale and is designated as the rebound number or rebound index. The compressive strength can be read directly from the graph provided on the body of the hammer.

In this thesis, rebound hammer test was conducted to determine the likely concrete compressive strength (fck) from which the bridge is built. The hammer readings are taken at 16 different locations both horizontal surface (curbs and slabs) and vertical surfaces (girders). The tool used to take the readings is called Schmidt rebound hammer. The compressive strength was read directly from the graph provided on the body of the hammer.

Figure 3.3 Taking Rebound hammer readings on Oda Bridge

3.3 Live Load Analysis

The actual traffic that a bridge subjected to, during its design life, is difficult to determine. This may be due to variation in vehicle weights, number of axle and its configuration. To overcome this problem, a single representative vehicle is required. These trucks are assumed to induce the same effects as that of the actual traffic. ERA 2002 Bridge Design Manual proposed different legal trucks used for rating calculations of existing bridges as explained in Chapter two, section 2.1.6.2.

There is also the most important live load for fatigue analysis. Fatigue truck is required for analysis of fatigue life of bridges. Fatigue truck is an equivalent fatigue truck that when passed over the bridge will generate the equivalent stress ranges for fatigue analysis as provided in section 2.3.

In this thesis, standard legal trucks provided by ERA 2002 and AASHTO 1990 fatigue trucks are used in the analysis. Then the capacity of girders is determined from the analysis output and fatigue life of the whole bridge is predicted.

3.4 Life prediction

Using the specified truck models, the available life is determined for both S-N curve and LEFM methods. Stress life method used a relation given by equation 2.21.

The following is a general outline of the fatigue evaluation method using LEFM

1. Determination of stress intensity factor, K_{IC}
2. Calculate stress intensity, σ_{max} from dead and live loads. The stress will be calculated from the equivalent fatigue truck model for live loads case.
3. Determination of initial crack length by assumption
4. Solve for critical crack length as equation (2.24).
5. Determine the total fatigue life in cycles, N, for the nominal stress range, σ_{max}, which is the maximum stress of loads. The total fatigue life in cycles, N, may be calculated as equation (2.26).
6. Estimating the number of trucks traveling across the bridge in the past, present and future. This can be represented as ADTT and traffic growth rate.
7. Determine the remaining life of the bridge, in years (equations 2.28 to 2.32)

3.5 Computer Programs

To accomplish the analysis, two computer programs are utilized. These are Excel and Mathcad Prime 2.0. The excel program is used to calculate girders' shear and moment under the analysis live loads by influence line principle.

3.6 Traffic Volume Analysis

Due to time constraints, the available traffic data is used to determine a number of vehicle loads a bridge will experience in its lifetime. The average daily traffic volume (ADT) of the area was predicted for the design of the pavement during rehabilitation of the route in 2001-2003 E.C.

3.6.1 Lifetime Truck Volume

A lifetime average truck volume is very important in the fatigue because it is major source of fatigue inducing stresses. The present average truck traffic is given by $ADTT=ADT.F_TF_L$.

Where F_T=fraction of trucks F_L= fraction of trucks in outer lane

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1 Capacity Evaluation

This section introduces a live load carrying capacity of a case study bridge. Rating factors for moments and shears in interior girder are calculated. The tonnage it can carry is determined using the analysis results.

4.1.1 Bridge Property

After site visiting, all these properties of the bridge are identified and recorded as in Fig.3.2. Then, the value from measurement was compared with the site inspection report from ERA Wollega District, bridge management department office, done during the rehabilitation of the Bako- Nekemte route section in 2003 E.C. Some differences in girder depth and spacing have been recognized from the comparison. Hence, dimensions from the inspection results are used for analysis.

The bridge has no visible cracks as well as almost no section losses. The railings of the bridge were constructed from I shaped steel structures. The data on dimensions of the components are presented in Chapter three, section 3.1.

4.1.2 Materials Strengths

There was no documented plan for the bridge. Hence, information such as strength of concrete and rebar are difficult to determine. The ERA Design Manual is employed to approximate the strength of steel bars and rebound hammer test result is used to determine the strengths concrete compressive strength.

The hammer readings are taken at 16 different points from girder and top curbs as listed in table below.

Table 4.1 Hammer test readings taken at different points

S. Number	Hammer Reading values	S. Number	Hammer Reading values
1	34	9	40
2	32	10	34
3	28	11	38
4	34	12	32
5	38	13	36
6	30	14	28
7	36	15	38
8	40	16	40
Average	**34.6**		

The Average Rebound value is 34.6. The Concrete compressive strength, as interpreted from the rebound value, is 24.85 MPa. Hence, it can be estimated that the concrete grade used for the bridge was C-30. Accordingly, fck=24.85 MPa and fy=276 MPa (Table 2.7) are used for concrete and steel bar strengths, respectively.

4.1.3 Load Calculation

Once all the properties were known or estimated as closely as possible, the next task executed in load rating was to determine all the loads on the bridge.

Dead loads- This includes loads from curbs, railings, posts and wearing course overlays. The bridge has railings made of steel structures anchored on exterior girders. The railing loads were assumed to be 0.05 kN/m and equally distributed on all girders. Slab loads and self-weights of the beams were also calculated from their as inspected dimensions. Based on this load, which is calculated in kN/m for each individual beam, the moment and shear effects due to the dead loads was determined (equations 4.1 & 4.2).

The total dead load **w**, is distributed on girders and the moment and shear effects are calculated as the following.

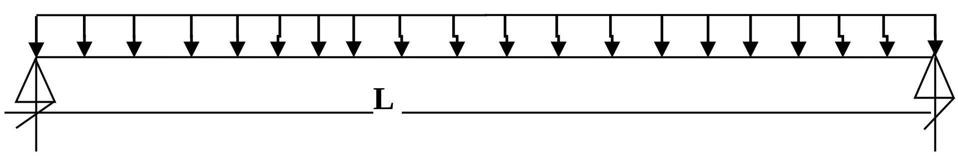

Figure 4.2 Dead load distributions on girders of the bridge

$$V_{DL} = \frac{w*L}{2} \tag{4.1}$$

$$M_{DL} = w * \frac{L^2}{8} \tag{4.2}$$

Where w = uniformly distributed dead load

L = span length center to center of support

V_{DL}= dead load shear force

M_{DL}= dead load moment at mid-span

The maximum effects of dead loads were calculated at mid-span of the girders.

- **Live loads**-are loads from vehicle loads.

A truck type 3 was used as a rating load. Type 3 truck load is critical for this bridge because of its large GVW and short over all axle spacing. Longer legal truck Types 3-2 and 3-3 vehicles would not be expected to govern due to the limited span length for this bridge.

The shear and moment effects of this truck are determined from influence lines. To obtain a critical point of load effects, the beam is analyzed at 0.05 of the span length. The truck load model on typical girder looks like the following.

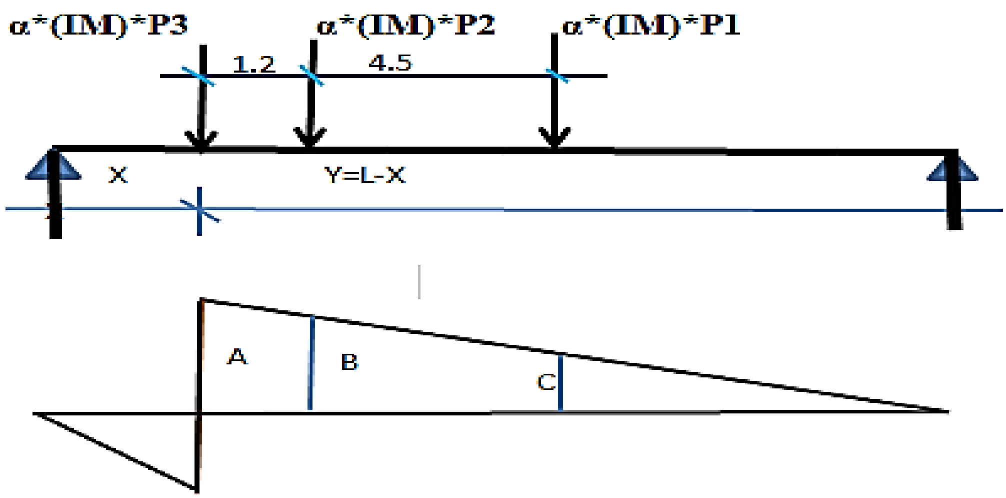

Figure 4.3 Type 3 Legal truck moving on girders of the bridge (Rear wheel position)

Where **α** is the lateral live load distribution factors for shears and moments and IM=is impact factors. P1=73, and P2=P3=77 KN (from Figure 2.2).

4.1.4 Moment and Shear Live Load Distribution Factors

The live load distribution factors are calculated and applied to determine how much of the live load is applied to each girder when a truck is on the bridge. The calculation considers single or two or more than two lane loading cases and the governing value is used. The DF calculations are carried out using Mathcad and the governing shear and moment distribution factors are used for the analysis. There are given in table 4.2 (see appendix-A).

Table 4.2 Summary of girder Distribution Factors

ACTION	DISTRIBUTION FACTORS (DF)	
	Interior Girder	Exterior girder
For moment(MDF)	0.624	0.624
For shear(SDF)	0.667	0.467

Using these distribution factors with impact factor, the moment and shears on girders due to live loads and dead loads are calculated. The results are listed in table below.

Table 4.3 summary of loads effect on girders

LOAD TYPE	LOAD EFFECTS			
	MOMENT (KNm)		SHEAR (KN)	
	Interior girder	Exterior girder	Interior girder	Exterior girder
DEAD LOAD	350.8	223.5	100.9	64.3
LIVE LOAD	246.2	239.2	143.24	100.34

4.1.5 Resisting Strength

This is consisted of shear and moment resisting of the section. The resisting capacity of the section is very important to know the available capacity of the member under investigation.

4.1.4.1 Calculating Moment and Shear Capacity of Girder

Since the bridge is a multi-span bridge, the analysis method for simply supported reinforced concrete beam is utilized for analyzing girders to get nominal shear and moment capacity. The nominal capacity of the interior and exterior girders is summarized in table below for both moments and shears.

Table 4.4 Summary of resistance capacity of girders

NOMINAL RESISTANCE			
MOMENT (KNm)		SHEAR (KN)	
Interior girder	Exterior girder	Interior girder	Exterior girder
1175	1103	363.2	394.8

4.1.6 Rating Factor Calculation

Once the nominal capacity of the structure is determined, the capacity available to resist live load can be evaluated in terms of rating factor. The Rating Factors was determined using LRFR method for the type 3 legal truck. Then, a capacity of the structure available to carry live load was determined using equation 2.1 (ERA, 2002).

The Rating Factors and Capacity of the girders obtained from type 3 legal load analysis is summarized in the table below.

Table 4.5 Summary of RF for Oda Bridge due to Type-3 Legal truck

GIRDER TYPE	RATING FACTORS (RF)		AVAILABLE CAPACITY (RF*W) IN TONS	
	FOR MOMENT	FOR SHEAR	MOMENT	SHEAR
INTERIOR	1.987	1.105	45.105	25.084
EXTERIOR	2.331	2.133	52.194	48.419

4.2 Fatigue Life Analysis

In this section, the fatigue life of the bridge is determined using the two common methods, S-N curve and LEFM.

4.2.1 General

In this section, fatigue life of Oda Bridge was determined using the LEFM approach and S-N curves. There are some points that are considered in the analysis:

- Fatigue analysis of the bridge is executed for reinforcement bars under the 240 kN fatigue truck (1990 AASHTO LRFD fatigue truck). This truck is used to calculate flexural stress ranges in girders.
- For this load case, the stress range was calculated from a line girder analysis with distribution factors. A single lane of the bridge was loaded by single fatigue truck so that it will produce maximum effect in the girders.

- Longitudinal steel bars in the bottom part of girders are assumed to be 20 mm in diameter. Since the steel bars in the lower position are subject to more fatigue damage, the lowest longitudinal steel bar are selected to evaluate the fatigue life for the bridge. Moreover, steel bars in the interior girder govern the analysis, because it was assumed that an interior is the most critical member.

4.2.2 Truck traffic analysis (ADTT)

It is assumed that only truck traffic will cause stress cycles of significant to the fatigue damage calculation, therefore, the average daily truck traffic, ADTT factor is used. The ADTT of the area was determined from the ADT predicted for the design of the pavement during rehabilitation of the route in 2001-2003 E.C. The ADT at 2008 is 808. Using two estimated different growth rates, 3 % for the past and 5% for future, the total number of traffic expected to cross the bridge in 100 years is 4,238,000.

ADTT is consisted of truck types of small, medium, heavy and truck-trailer. Using this data, the present ADTT is determined. Accordingly, the $ADTT_{2008}$ become 422 using 5 per cent growth rate. The $ADTT_{1954}$ was then estimated back using annual growth rate of 3 per cent. The total truck traffic that a bridge can experience in its lifetime (say 100 years) is 1,608,000 trucks.

4.2.3 Stress Calculation

The live load stress range is calculated from the fatigue truck model provided by AASHTO. This stress is used in remaining life prediction.

The dimensions and loads of AASHTO fatigue truck model on the bridge was shown as follows.

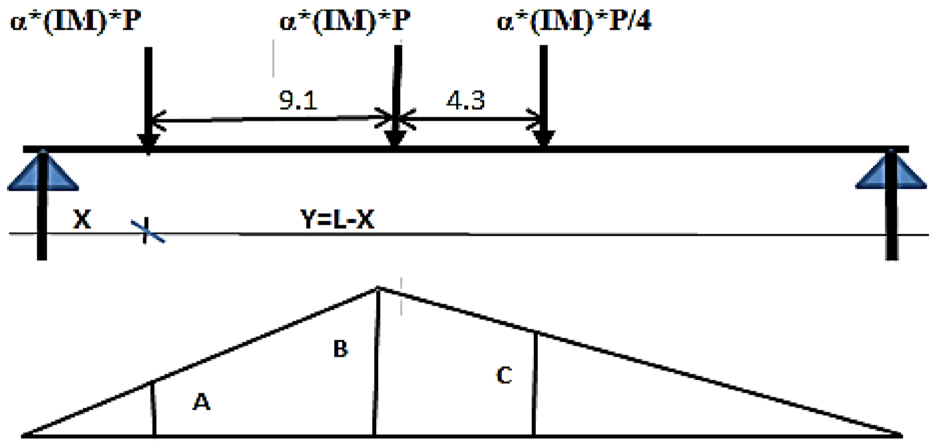

Figure 4.4 Fatigue truck model on the girder for fatigue stress analysis

Where α =is moment distribution factor for interior girder single loaded case and IM= impact factor for fatigue and P is axle load of 106.8 KN. The analysis result is σ_{max}=19.3 MPa.

4.2.4 Fatigue life prediction by S-N approach

The fatigue life is determined using equation 2.21 above for flexural loading types. The value of K was calculated by interpolation method for rebar of diameter 20 mm. K=2.395 $x10^{27}$ and m=4. Accordingly, the total remaining life is R= 742.5 years (see Appendix-B).

4.2.5 Fatigue Life prediction by LEFM approach

According to appearance detection, no crack was found on Oda Bridge, so initial crack was assumed to evaluate the fatigue life. In this section, the fatigue life of reinforced concrete bridges is evaluated based on the steel bar fatigue failure using Paris Law because the remaining service life of the bridge is controlled by the reinforcement. The following Paris law constants are used in the analysis.

Fracture constants are $C=2x10^{-13}$ and m=4 according to previous researches, threshold of crack propagation of steel bar is $\Delta K_{th} = 2\ \mathrm{MPa}.\sqrt{\mathrm{m}}$, and fracture toughness of the steel bar is taken as $K_{IC}=50MPa.m^{1/2}$. Using these constants, the available service life became 38.6 years.

Table 4.6 Summary of fatigue analysis results

Stress range due to fatigue truck (MPa)	Fatigue Life (in Years)	
	S-N curve	LEFM
19.23	742.50	38.60

4.4 Discussion of Results

In this thesis, a superstructure part of Oda Bridge is evaluated for two types of failure modes. Firstly, it was checked for the available capacity. Secondly, its fatigue life was evaluated using two different methods such as S-N curves and fracture mechanics.

The ERA type 3 legal truck model is used in rating of the bridge. The capacity obtained from the analysis of girder is expressed in tons and presented in the table 4.5. As it can be seen from the table, the bridge can carry 45 tons for flexure and 25 tons for shear which is greater than the weight of the rating truck.

The bridge is also analyzed to determine its remaining service life. For fatigue life prediction the AASHTO 1990 fatigue truck model is utilized. The fatigue analysis results are presented in the table 4.6. As it can be seen from the table, two very far different results are obtained with respect to the analysis methods. From the S-N curve analysis method, the bridge will live for 742.50 years while the LEFM result shows that the remaining life of the bridge is about 38.6 years.

This difference is due to that the stress based (S-N curves) evaluation method does not consider the presence of actual crack. In another way, uncracked component is assumed at the beginning of fatigue analysis so that the crack growth of such components is not accounted for. It simply depends on the magnitude of stress range induced by the fatigue truck. This limits their use and accuracy as a fatigue life prediction method. The fracture mechanics is, on the other hand, related to rate of growth of crack of specific size with time. It is used to analyze fatigue in cracked components.

CHAPTER FIVE

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This thesis utilized a one dimensional analysis method with a combination of field tests (Non-Destructive Tests), for rating as well as for fatigue life evaluation of Oda Bridge, from which the followings can be concluded:

(1) Load rating of Oda Bridge is performed under a Type 3 legal truck. The interior girder capacity is 45 tons for flexure and 25 tons for shear. The value for flexure and shear are greater than the rating truck load weight. Hence, the bridge is able to withstand the stresses from trucks greater or equal to the rating trucks.

(2) Fatigue safety and fatigue life evaluation based on elastic fracture mechanics and stress life methods is given. Fatigue life of Oda Bridge based on fracture mechanics is about 38.6 years, whereas 742.5 years based on stress-life method. Fatigue failure may occur during design service life.

(3) Based on the fatigue safety evaluation utilized in this thesis, it was concluded that LEFM approach could be effectively applied to evaluate the fatigue safety of reinforced concrete girder bridges than S-N curve method.

5.2 Recommendations

(1) In order to obtain more accurate and comprehensive results in bridge rating and fatigue life analysis, a three-dimensional structural analysis should be carried out instead of a one dimensional analytical analysis.

(2) The evaluation was only performed for superstructure. But an inspection result showed that the substructure of the bridge was highly deteriorated and eroded. Hence, it is difficult to say safe unless remedial action is taken to abutments and piers of the bridge. It is recommended that evaluation is required for substructure at all.

(3) The result obtained by LEFM may be conservative in fatigue life evaluation due to the uncertainties in initial crack size estimation, rebar size, truck traffic, fatigue loading and lack of well documented plan of the bridge.

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