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

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Abstract

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

The objective of this study is to assess a performance of concrete bridge. Load rating was made to determine the available capacity under ERA type-3 legal truck and fatigue life of the bridge was estimated based on linear elastic fracture mechanics and Stress-based approaches under AASHTO 1990 fatigue truck model in which a girder line analysis method was utilized for both load rating and fatigue analysis. A Non-destructive test was done on a case study bridge to determine concrete strength from which the bridge has constructed.

The rating values determined were greater than the rating truck load weight both for flexure and shear. The available life of the bridge was also predicted. The evaluation results show that the bridge can serve for 38.6 years. Finally, it was concluded that the bridge is safe in carrying the current live loads but fatigue failure is likely to happen in its lifetime.

Keywords: *existing concrete bridges, fatigue life, load rating, Non-destructive test, stress-life.*

1. Introduction

Bridges are very important components of highway transportation. They are not simply structures but crucial for life as foods and water. Existing 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 causes safety problems. Performance assessment is, therefore, required for older bridges to estimate their available capacity and remaining life as it 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 [1-2].

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. It is a phenomenon of weakening of a material as a result of accumulation of damages from repeated loads.

For long, fatigue effect in concrete structures was not considered and even not well studied. Under cyclic loads, cracks known as fatigue cracks will develop in reinforced concrete structures. 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].

2. Rating And Fatigue Life Evaluation Methods

2.1 Bridge load rating

Bridge load rating provides a basis for determining an existing bridge's safe load carrying capacity. 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. Although any bridge component can be load rated, it is generally assumed that the bending moment and shear in the girders will give the critical rating values [5].

2.1.1 Bridge Load Rating Methods

There are three methods of bridge ratings: the Allowable Stress Rating (ASR), the Load Factor Rating (LFR) and the Load and Resistance Factor Rating (LRFR). Among these, the load and resistance factor rating method is the newer method and was directly related with the LRFD philosophy. In this method, there are three distinct levels of evaluation according to FHWA-2012. These are: design load rating (first level evaluation); legal load rating (second level evaluation); permit load rating (third level evaluation).

2.1.2 The Rating Equation

The capacity evaluation is done with a comparison of the factored live load effects and the factored strength or resistance. The rating procedure 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. 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.

The basic rating equation used in the ERA Bridge Design Manual 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 and written as follow:

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

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

Before the load rating of a specific bridge can be conducted, a certain amount of information has to be gathered. These includes: 1. Deck condition 2. Structural Condition 3. Traffic Condition. From these information, dead load effects have been determined especially using as built dimensions of the bridges.

2.1.3 Live Loads

Since 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, 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 (Fig.1) to (Fig.3) are recommended in Ethiopia 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.

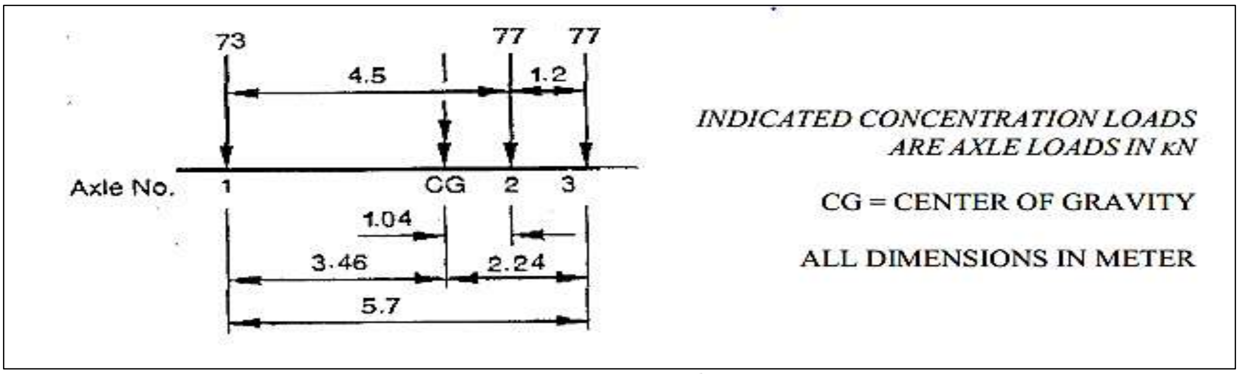

Figure 1. Truck Type 3-1 Unit Weight = 227 kN

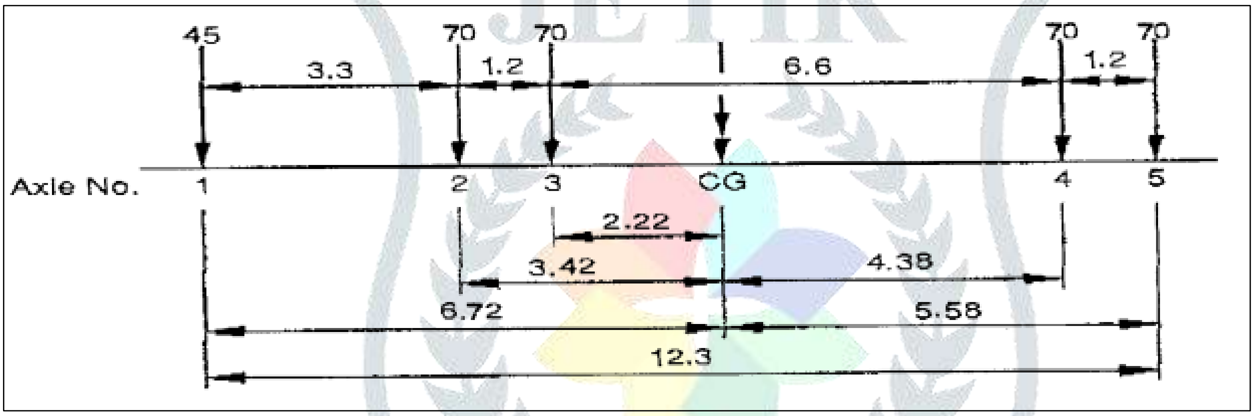

Figure 2. Truck Type 3-2 Unit Weight = 325 kN

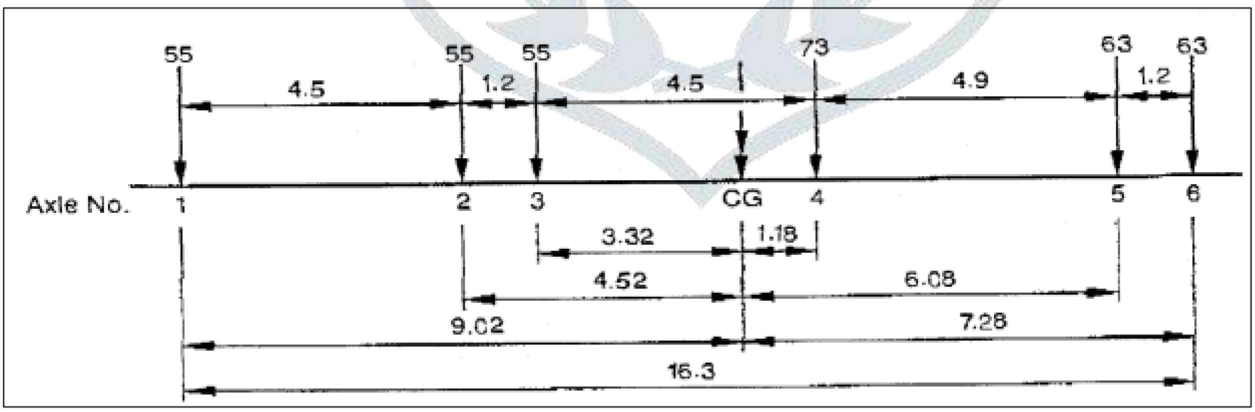

Figure 3. Truck Type 3-3 Unit Weight = 364 KN

2.1.4 Rating Factors (RF)

The rating factor is calculated from Equation (1). If it exceeds 1.0, the span is satisfactory for the legal loads. If not, the bridge is incapable to resist the load.

2.2 Bridge Fatigue Evaluation

Fatigue damage in concrete structures is a complex area and not as well researched as in steel. As a result, to determine fatigue of reinforced concrete bridges, the fatigue resistances of steel bars and concrete were studied separately.

2.2.1 Fatigue of Steel Reinforcement

It was proved by many researchers that the fatigue strength of reinforcing steel is a vital parameter of reinforced concrete members subjected to cyclic loading and also similar to that of steel structures [1]. For steel reinforcement, similar to steel structures, the fatigue relevant parameters are: **(1) The stress ranges $\Delta\sigma$. (2) The number of load cycles, N** and **(3) discontinuities.** 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 Fig .4.

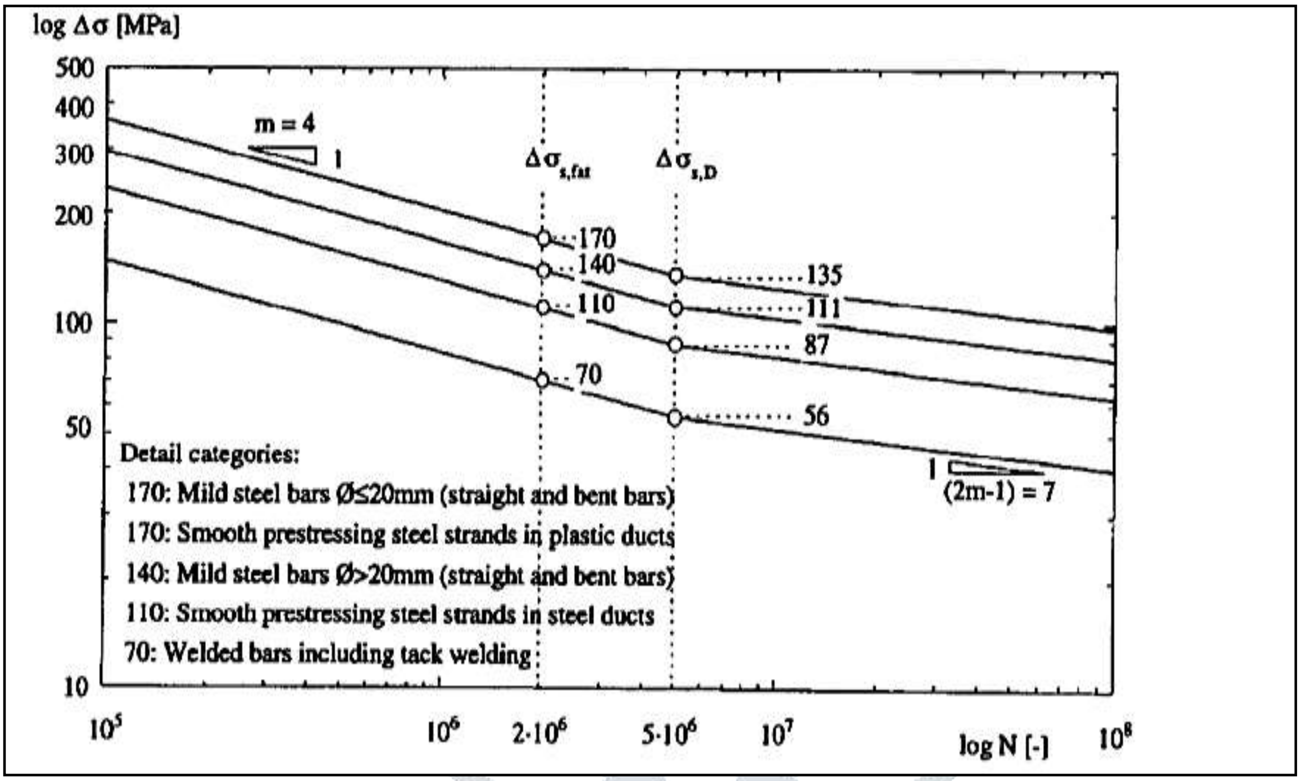

Figure 4. Fatigue strength of steel reinforcement [1]

2.2.2 Fatigue of concrete

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]. The effect of this pair of stresses as a function of load cycles is best represented by **Goodman** diagram (Fig.5). Other fatigue relevant parameter include the concrete strength and structural size effect which are taken into account by the nominal design values f_c and τ_c for static compressive and shear strength, respectively.

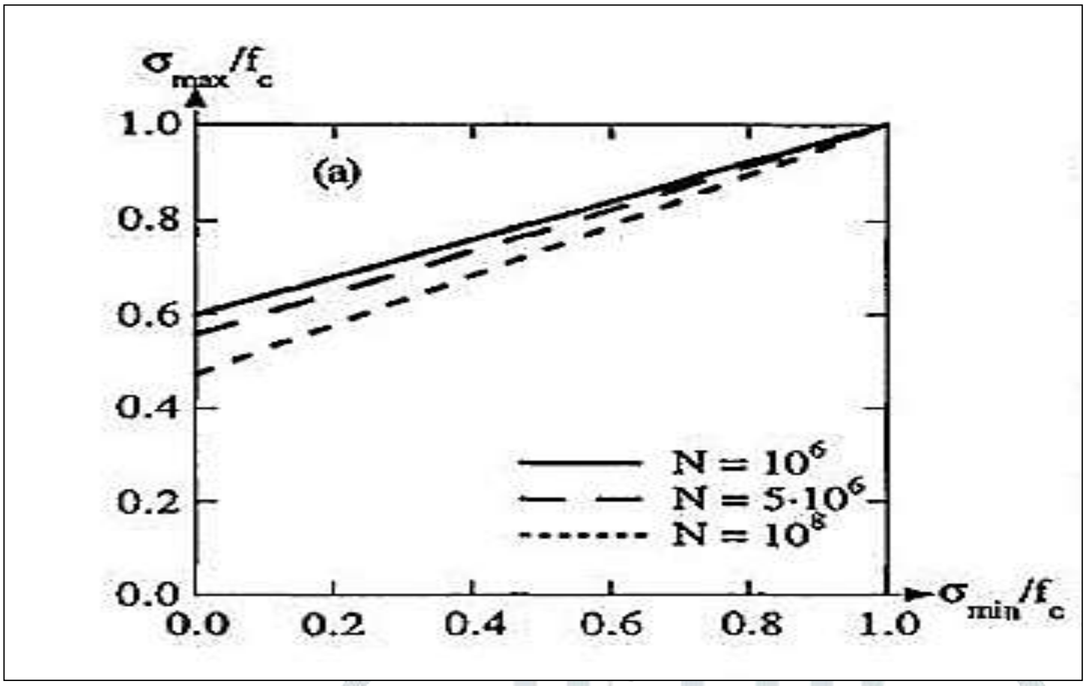

Figure 5. Fatigue strength (compressive) diagrams for concrete [1]

2.3 Fatigue life evaluation Methods

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

2.3.1 Fatigue life evaluation based on S-N curves

This stress-based approach involves establishing an empirical relationship between stress range amplitudes and number of cycles to failure. To do that, a formula previously derived to predict a fatigue life of rebar, was employed. 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:

$$\mathbf{N_f \sigma_r^m = K} \quad \mathbf{(2)}$$

Where σr = 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 1.

Table 1 Values of K for different action and bar diameter [14]

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

2.3.2 Fatigue Life Evaluation Based On Fracture Mechanics

The use of this method requires the determination of the material's fracture toughness, nominal stress range, flaw size, and geometry.

To study the fatigue crack propagation of the rebar using this 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.2.6). And also a stable crack growth is assumed from the initial flaw. 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].

The stress field near the tip of a crack is characterized by the stress intensity factor, KI.

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:

$$\Delta K = Y.\Delta\sigma.\sqrt{\pi.a} \tag{3}$$

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:

$$\frac{da}{dN} = C.(\Delta K)^m \tag{4}$$

Where C and m are material constants and the range of stress intensity actor, ΔK is determined as equation (3) above.

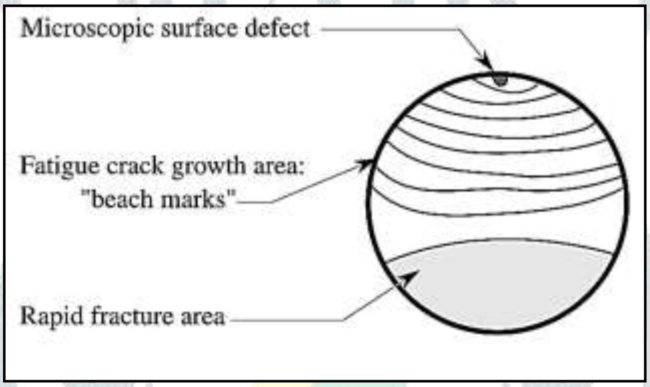

Figure 6. Rebar cross section with initial flaw and crack at fracture [21]

The critical crack depth could be determined from equation (3) by substituting $Y=Y_{cr}$, $a=a_{cr}$ and $K=K_{IC}$ as below:

$$a_{cr} = \frac{1}{\pi}\left(\frac{K_{IC}}{Y_{cr}\sigma_{max}}\right)^2 \tag{5}$$

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]:

$$Y = \frac{\frac{1.84}{\pi}.\left[\tan\frac{(\pi a/4r)}{(\pi a/4r)}\right]^{0.5}}{\cos\left(\frac{\pi a}{4r}\right)}.\left[0.752 + 2.02\left(\frac{a}{2r}\right) + 0.37\left\{1 - \sin\left(\frac{\pi a}{4r}\right)\right\}^3\right] \tag{6}$$

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 (Fig. 7). 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.

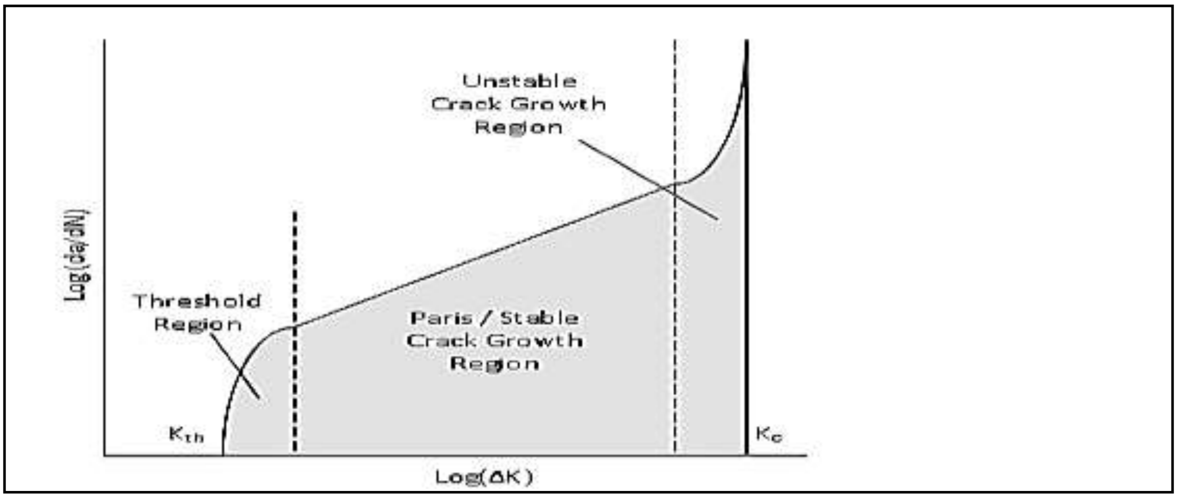

Figure 7. Crack Growth Rate versus Stress Intensity Factor Range [11]

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

The Paris Law 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, as the following equation [2]:

$$\boldsymbol{N_f = \int_{a_i}^{a_{cr}} dN = \int_{a_i}^{a_{cr}} \frac{1}{C.\Delta K^m} da = \int_{a_i}^{a_{cr}} \frac{1}{C.Y^m.\Delta\sigma^m.\pi^{\frac{m}{2}}.a^{\frac{m}{2}}} da} \quad (7)$$

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.

2.3.3 Analysis Fatigue Life in Years

The number of stress cycles was also determined using the specified equation (7). Once the total number of cycles is determined, to calculate the remaining fatigue life, **R**, in years, following steps has followed.

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

$$\mathbf{GF1 = \sqrt[n_3]{\frac{ADTT(n_2)}{ADTT(n_1)}} - 1} \quad (8)$$

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

$$\mathbf{ADTT(year\ built) = \frac{ADTT(n_2)}{(1+GF1)^n}} \quad (9)$$

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

$$\mathbf{M = 365\frac{days}{year} * [ADTT(year\ built)] * \frac{(1+GF1)^n - 1}{GF1}} \quad (10)$$

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.

$$\mathbf{GF2 = \sqrt[n_4]{\frac{ADTT(n_f)}{ADTT(n_2)}} - 1} \quad (11)$$

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

$$\mathbf{R = \frac{\ln[\frac{(N_f - M)*GF2}{365\frac{days}{year}*ADTT(n_2)*(1+GF2)} + 1]}{\ln[1+GF2]}} \quad (12)$$

Where

n_1 = previous year

n_2 = recent year $n_3 = n_2 - n_1$

$n = n_2$ - year built $n_4 = n_f - n_2$.

3. 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 pavement design during rehabilitation of Bako-Nekemte raod segnment in 2009-2011. The ADT at 2016 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.

Using this data, the present ADTT is determined. Accordingly, the $ADTT_{2016}$ become 422 using 5 per cent growth rate. The $ADTT_{1962}$ was then estimated back using annual growth rate of 3 per cent. The total truck traffic that a bridge can experience in its lifetime (100 years) is 1,608,000 trucks.

4. Nondestructive Test

A non-destructive test, rebound hammer test was conducted to determine the likely concrete compressive strength (fck) from which the bridge was built. The hammer readings are taken at 16 different locations both horizontal surface (curbs and slabs) and vertical surfaces (girders) per square meter. Accordingly, fck is estimated to be 24.89 MPa which means the concrete grade used for the bridge was C-30. A steel bar grade was taken from ERA Bridge Design Manual 2002 and it is fy=276 MPa based on the year of construction.

5. A CASE STUDY BRIDGE

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 as it is constructed on Oda River.

5.1 Description of the bridge

Fig.8 shows Oda Bridge, a two lane girder bridge constructed in 1962. The bridge was constructed on a curved road from straight road consisted of three simple spans. It has a carriageway width of 7.00 meters, each span 12 meters long through three main longitudinal girders. The girders are 0.86 meters deep and 0.40 meter wide spaced at 2.33 meters center to center. The slab thickness of the bridge is 0.20 meters.

Figure 8. Side View of Oda Bridge

5.2 Stress Analysis

The maximum shear and moments were calculated for both live loads and dead loads using girder line analysis. 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 Type 3-2 and Type 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 were determined from influence lines. To obtain a critical point of load effects, the beam is analyzed at 0.05 of the span length. The influence line for shear force under type-3 truck load model on typical girder looks like the following.

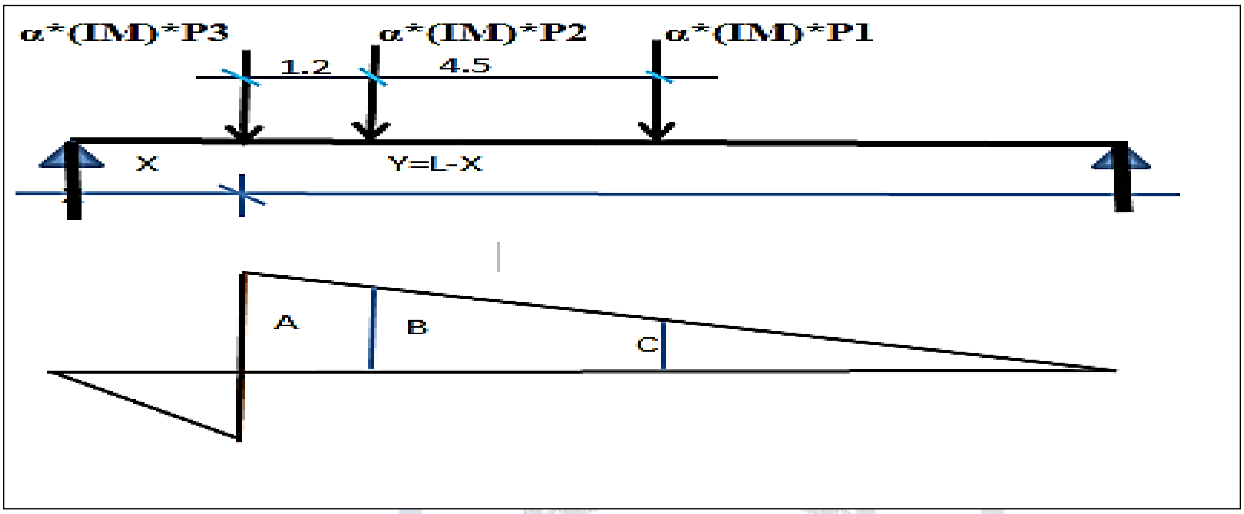

Figure 9. Type 3 Legal truck moving on girders of the bridge (Rear wheel position)

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).The moment and shears on girders due to live loads and dead loads are calculated. The results are listed in table below.

Table 2. 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

5.2.1 Resisting Strength

Since the bridge is a multi-span bridge, the analysis method for simply supported reinforced concrete beam was 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 3. 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

5.3 Rating and Fatigue Life prediction

5.3.1 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 the available capacity was determined.

Table 4. 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

5.4 Fatigue Life Analysis

The fatigue life of Oda Bridge was determined using the two common methods, S-N curve and LEFM.

To determine stresses, the live load stress range was 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.

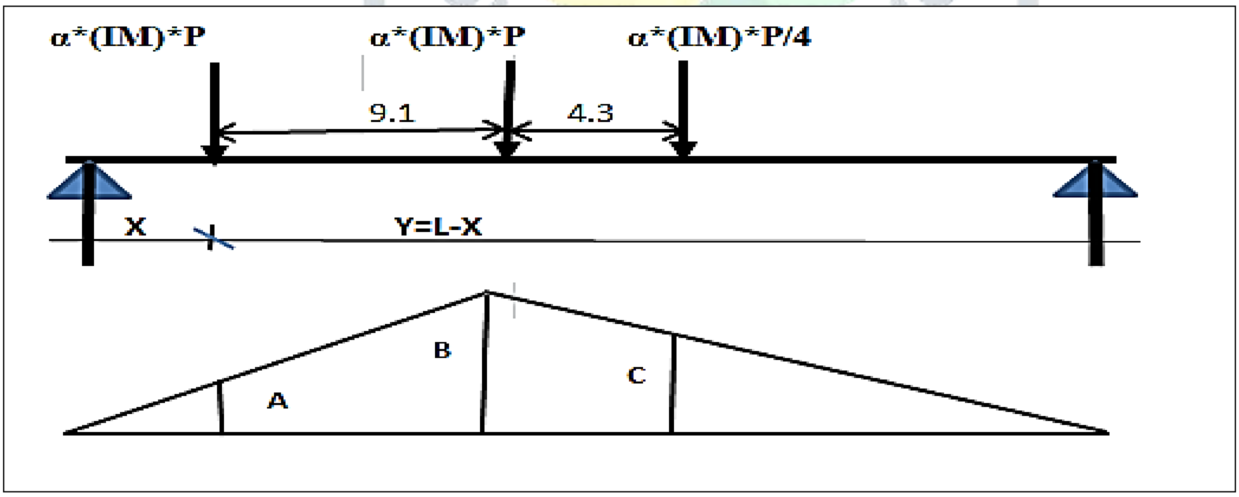

Figure 10. Fatigue truck model on the girder for fatigue stress analysis

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 was σ_{max}=19.3 MPa.

5.4.1 Fatigue life prediction by S-N approach

The fatigue life is determined using equation (2) above for flexural loading types. The value of K was calculated by interpolation method for rebar Φ20 mm. K=2.395 x10^{27} and m=4. Accordingly, the total remaining life in years is, R= 742.5 years.

5.4.2 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 the bridges was 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 were 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 $K_{IC}=50\mathrm{MPa.m}^{1/2}$. Using these constants, the available service life became 38.6 years.

Table 5 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

6. Discussion And Conclusion

6.1 Discussion of Results

In this thesis, a superstructure part of Oda Bridge was evaluated for two types of failure modes. Firstly, it was checked for the available capacity. Secondly, its fatigue life was evaluated based on 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. 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 to mean that the bridge has enough capacity.

The bridge is also analyzed to determine its remaining service life. The fatigue analysis results are presented in the Table 5. 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. It simply considers the magnitude of stress range induced by the fatigue truck. This limits its use and accuracy as a fatigue life evaluation method. The fracture mechanics is, on the other hand, related to rate of growth of crack of specific size with time. Therefore, it was used to analyze fatigue of the bridge.

6.2 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 (type-3) trucks.

(2) Fatigue safety and fatigue life evaluation based on elastic fracture mechanics and stress life methods was given. Fatigue life of Oda Bridge based on fracture mechanics is about 38.6 years, whereas 742.5 years based on stress-life method. Therefore, 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.

7. Acknowlegments

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