ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

STRENGTH CAPACITY DETERMINATION OF T-GIRDER BRIDGE USING EXPERIMETAL METHOD.

A Thesis Submitted to School of Graduate Studies in Partial Fulfillment of the Requirement for the Degree of Master of Science in structural Engineering

By: Ibrahim Dilsebo

Advisor: Dr. Asnake Adamu

July, 2016

ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

STRENGTH CAPACITY DETERMINATION OF T-GIRDER BRIDGE USING EXPERIMETAL METHOD.

By: Ibrahim Dilsebo

Approved by the Board of Examiners

____________________	____________________
Advisor	Signature
____________________	____________________
Internal Examiner	Signature
____________________	____________________
External Examiner	Signature
____________________	____________________
Chair Person	Signature

Addis Ababa, Ethiopia

DECLARATION

I, the undersigned, declare that the thesis is my original work, has not been presented for a degree in any other university and that all sources of materials used for the thesis have been dully acknowledged.

Ibrahim Dilsebo

Candidate

Signature

Date of submission: march 7,2016

This is to certify that the above declaration made by the candidate is correct to the best of my knowledge.

Advisor

Signature

ACKNOWLEDGEMENTS

First of all I would like to thank my God, for all things are for Him and by Him. I also thank my advisor Dr. Asnake Adamu for his guidance and encouragement from the beginning to the end. His cooperativeness and friendly manners made everything easy.

I would also like to thank Dr.Abraham G. for his initial suggestion and help throughout the work of this thesis paper. My thanks extends to my class friend and colleague for their comments which gave solutions to some difficult tasks.

Last but not least, I would like to thank my family and wife for their patience, encouragement and love.

ABSTRACT

Currently, Ethiopia is undergoing economic transformation and development which requires exporting of local commodities and importing heavy machineries. Reinforced concrete bridges on different routes are sustaining these equipments loads safely. But there are bridges where their design data is not available for some reasons (i.e. construction time and work document).In the absence of such design data, determination of strength capacity of reinforced concrete bridges are difficult and it is a critical problem that currently engineers are facing.

E.R.A manual has developed strength evaluation method for bridges where it has its own way of estimating concrete strength, area of reinforcement and yield strength of concrete. But, based on the current condition of bridges approximate method should be developed to obtain construction data thereby determine the strength capacity of Girder bridges. In this study, experimental investigation of reinforced concrete T-Girder bridges will be carried out based on the existing condition of the bridges. For verification of results, similar bridges with detailed design document will be investigated.

The results obtained show lesser reinforcement area and yield strength of steel when compared with original data. it is good with regard of safety while determining what amount of load should pass over z bridge.

TABLE OF CONTENTS PAGES

LIST OF FIGURES PAGES

LIST OF TABLES PAGES

1. INTRODUCTION

1.1 BACKGROUND

Today, Ethiopia is undergoing transformation with the aim of achieving its millennium goals by constructing Abay Dam, Gilgel Gibe I ,II, and III electric power projects, etc. These projects require heavy construction equipments which are being imported through Djibouti. The roads linking Djibouti and the respective project sites contain bridges where neither construction time nor design data is known. Challenges are arising whether these reinforced concrete bridges can sustain the equipments loads without failure.

In this study, effort will be made to assess the strength of these reinforced concrete bridges. Actual Load test will be performed on reinforced concrete bridges. In these load test deflection as a main data will be measured. Using these measured collected deflection data, a numerical method will be developed to come up with a result evaluating the current condition of the structure, and hence the strength capacity will be computed accordingly.

The carrying capacity of the reinforced concrete bridge must be adequately quantified so that relative retrofitting can be applied whenever the structure is under capacity which Otherwise failure is eminent.

Expectation from this study is to come up with a tool of determining the load capacity of reinforced concrete bridges based on observations and measurements.

1.2 OBJECTIVE AND SCOPE OF STUDY

Current condition of bridges will be determined based on observation and measurements. But the main target is providing quick solution for determining strength capacity of reinforced concrete T-Girder bridges when problem arises where their design data can’t be retrieved.

The scope of these study is limited to analysis obtained from results by reading vertical deflection data. Other strain reading(i.e horizontal) are difficult to take and the instruments are not available as well as expensive.

1.3 THESIS ORGANIZATION

The Thesis is organized in to chapters.

Chapter 1 deals with introduction where objective and scope of the study is addressed.

Chapter 2 deals with literature review. In the literature review History of Ethiopian bridge development, Bridge capacity assessment problem, strength evaluation and Theoretical deflection of response of Existing structures will be discussed.

Chapter 3 deals with Experimental Evaluation of Bridge capacity. Inside these chapter Background of evaluation problem, experimental setup, structure selection , structure dimension, devises to be used and loads are discussed.

Chapter 4 deals with determination of capacity from test. Copy of excel spreadsheet

On which calculation is done is provided.

Chapter 5 deals with conclusion and recommendation. Conclusion will be made on how much accurate the numerical solution is and recommendation will be made if anyone wants to widen the scope of the thesis.

2. LITERATURE REVIEW

2.1 BRIDGE CAPACITY ASSESSMENT PROBLEM.

Bridges, However constitute the most vulnerable elements of the road network, are neglected for long time in-terms of Inspection and Maintenance. It is mainly due to shortage of skilled manpower and absence of any kind of ready made bridge information in the country. This may be true for most of African countries as well [4].

The path that will be followed in these research work to assess the bridge capacity is shown in the figure below.

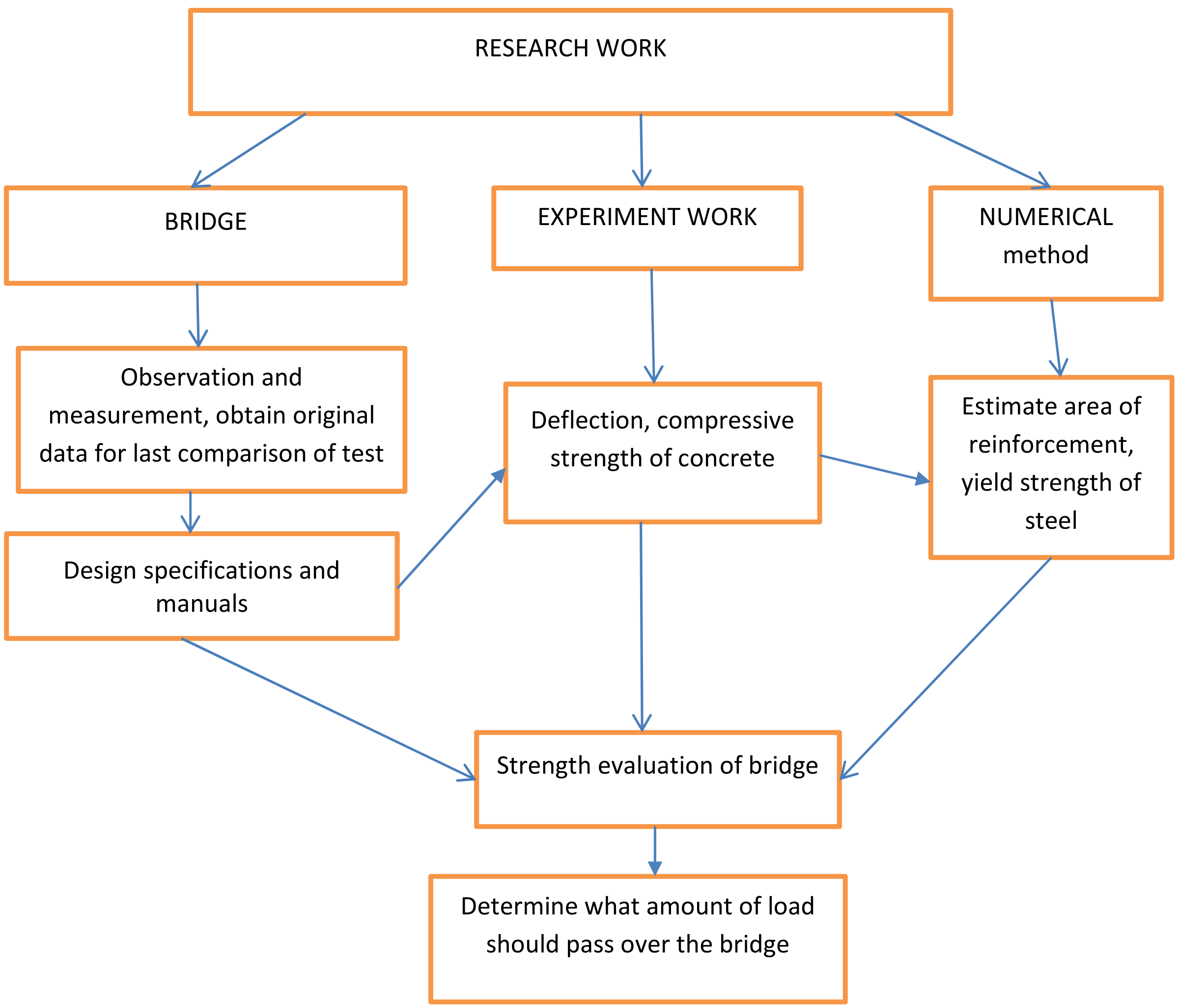


Fig 2.1 Research Work

According to the Road Sector Development Program (RSDP) document, which was launched in 1997 to speed up improvement and expansion of Ethiopian road network, only 21 % of nearly 25000 km road network found in `Good ` condition.

However the program emphasized the need for rehabilitation of bridges along the road network, while implementing the formulated projects, due to absence of the bridge data-base since establishment of ERA in 1951, this historical document could not say anything about number and condition of bridges in the country. Therefore, any bridge improvement plan in the RSDP was highly in deficient of bridge data like total numbers, length, type, name and location, service condition, etc [4]

The bridge condition data can be analyzed by the software for prioritization purpose which is vital for improvement intervention. The prioritization criteria can be different. As per the preset prioritization criteria, Ethiopian bridges are categorized as in ` GOOD` `FAIR` and `BAD` conditions.

Condition of bridges	Federal Roads - ERA Sep. 2006, %	Anticipated Improvement plan. 2010, %	Anticipated condition of bridges in absence of intervention. 2010,
GOOD Still sound, adequate and functional	54	79	41
FAIR Inadequate and require rehabilitation	36	20	36
BAD Critical ,requiring immediate intervention	10	1	23

Table 2.1 condition of bridges [4]

Bridge Management System(hereinafter referred to as BMS) means to manage bridges lifetime throughout design,construction, operation and maintenance of the bridges.

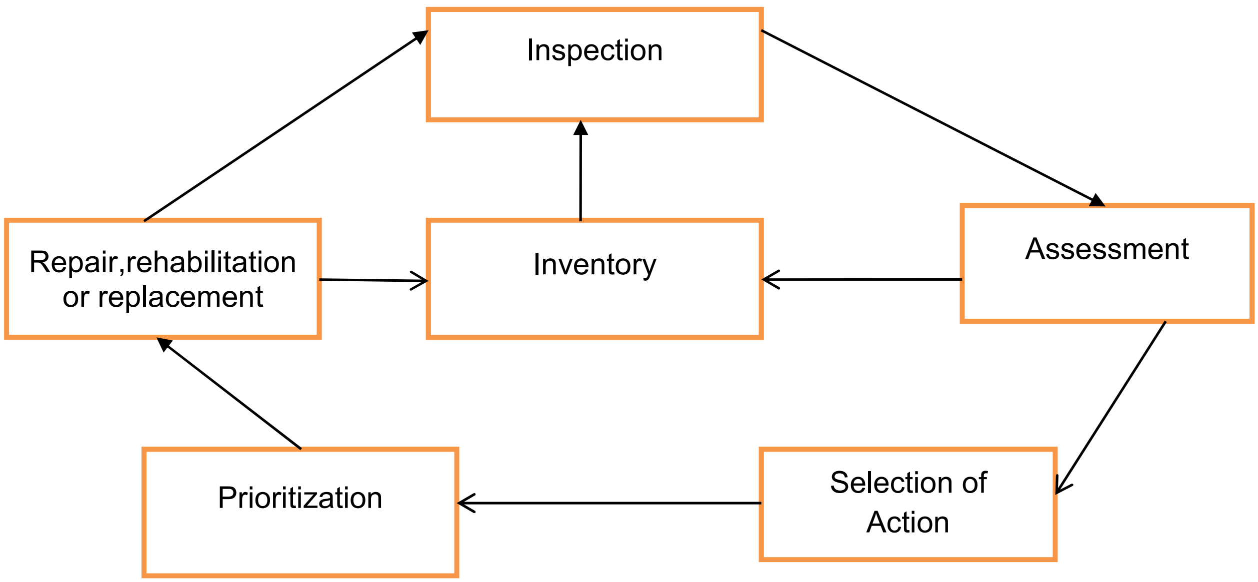


Fig 2.2 Bridge management system [4]

In the developed world [3], they use different technology that helps in determining strength evaluation of bridge. They are listed and described below.

A) Ultrasonic Defect Detection

The University of Manchester Institute of Science and Technology (UMIST) performed a comprehensive research effort sponsored by the National Research Council (NCHRP 10-30) in the late1980s and early 1990s. This research was aimed at development of nondestructive test and evaluation methods to detect the deterioration of prestressing steel tendons embedded in concrete. The case of sevenwire strands inside metal ducts was of particular interest. Based on a survey of various technologies available at the time, UMIST researchers identified ultrasonic testing and corrosion rate monitoring techniques as having the most potential for obtaining information on embedded prestressing steel. Since the latter technology was at the time the subject of Strategic Highway Research Program study, UMIST efforts concentrated on adaptation of ultrasonic techniques for strand evaluation. This effort consisted of theoretical analyses as well as experimental work. A prototype system was built, which included rolling transducers and incorporated advanced signal processing techniques.

B) Pulsed Eddy Current

Pulsed eddy current is a new NDT method for inspection and identification of hidden corrosion in layered structures such as aircraft lap-splices, as well as thickness and conductivity measurements of conductive coatings on metal plates. This method was developed at the Center for Nondestructive Evaluation of the Iowa State University in Ames, Iowa.The major advantage of this method over conventional eddy current testing is the ability to cover a wide range of frequencies rapidly (i.e., greater information) due to pulsed excitation. It also uses simple, relatively inexpensive equipment. The eddy current method is based on the principle of electromagnetic induction. Flaws introduce changes in current induced by an induction coil. Eddy currents can penetrate into subsurface layers, even when those layers are not mechanically bonded. This is an advantage over ultrasonic methods where mechanical contact between layers is required. The pulsed eddy current method is reportedly capable of detecting metal loss in a 2-or 3-layer structure and can distinguish metal loss from metal separation.

C) coustic Emission (Prestressing Wire Break Monitoring)

Acoustic emission testing is a "passive" monitoring method in which the detection system waits for the occurrence and capture of stress wave emissions associated with cracking, corrosion, or wire breaks. By contrast, classical flaw detection methods, such as ultrasonics, are considered "active" in that a stress wave is sent into the test object to identify the presence of defects.

D) Surface Spectral Resistivity Method

This method uses a multi-electrode electrical resistivity array to determine reinforcing bar locations and the state of corrosion. The method is based on surface measurements of the frequency dependence of the complex impedance of reinforcing bars embedded in concrete. Complex impedance can be directly related to the corrosion rate of reinforcing steel in concrete. The advantage of this method is that it does not require removal of concrete cover to attach electrodes directly to the reinforcing steel. Also, resistivity of concrete can be determined in areas where reinforcing bars are away from the surface.

E) Nonlinear Vibro-acoustic Method

A paper by Sutin and Donskoy presents the non-linear vibro-acoustic method. The conventional linear acoustic method includes effects of reflection, scattering, transmission, and absorption of acoustic energy. Presence of a defect changes the phase and/or amplitude of signal while the frequencies of the received signals are unchanged (same as emitted signal). The nonlinear technique correlates the presence and characteristics of a defect with acoustical signals whose frequencies differ from the frequencies of the emitted signal. This is due to the nonlinear transformation of the acoustic energy by a defect.

F) Electrical Time Domain Reflectometry (ETDR)

ETDR has been defined as "closed-loop" radar. It has been extensively used for flaw detection in power transmission lines. The technology has also been used in sensing systems in geotechnical engineering applications. The method involves sending a high-frequency electrical pulse through the sensing cable. Impedance discontinuities (or mismatches) along the cable result in partial reflection of the pulse. These reflections are monitored using TDR cable test equipment. Impedance is mainly a function of the inductance *(L)* and capacitance *(C)* of the line. A typical TDR sensor is a coaxial cable for which the critical TDR parameter is the impedance between the center wire and the outside shield. TDR sensor (e.g., coaxial cable) can be developed for embedment inconcrete for detection of concrete cracking or corrosion, or as a continuous strain gauge.

G) X-Ray Diffraction for Direct Stress Measurements

Carfagno present the first use of X-ray diffraction technique to measure PT tendon stresses of prestressed deck slabs at La Guardia Airport in New York. The authors present a brief theory of X-ray diffraction. In this method, strains are estimated by measuring the elastic atomic lattice spacing (distance between atomic planes). The existing prestress was measured on these slabs to find the cause of longitudinal hairline crack over the negative moment region.

H) Strain Relief for Prestress Measurements

Strain relief methods for concrete are similar to residual stress measurements in metals using hole-drilling concepts. While not comparable in principle to nondestructive flaw detection methods, they offer potential benefits as global evaluation techniques for evaluating and quantifying extent of time dependent prestress losses.

I) Imaging and Tomographic Systems

It present a comprehensive summary of various imaging technologies for reinforced concrete. Different procedures are explained, and the advantages/disadvantages of each method are presented. Methods considered include radiography, radioactive computed tomography, infrared thermography, microwave imaging (including microwave tomography), and acoustical imaging (including acoustic tomography).

j) Magnetostrictive Sensors

Bartels, Kwun, and Hanley and Kwun and Teller report on research performed at the Southwest Research Institute on the use of Magnetostrictive Sensors (MsS) to characterize corrosion in prestressing strands and reinforcing bars and on the detection of fractures in steel cables, respectively.

J) Power Focusing Ground Penetrating Radar

The Power Focusing Ground Penetrating Radar (PFGPR) technology was developed recently for use in real-time detection of buried metallic and non-metallic land mines. It has been adapted for rapid inspection of pavements and bridge decks with the goal of allowing deck/pavement evaluations at higher vehicle speeds.

Above mentioned technology will help in determining strength evaluation of bridge if available.

Non-destructive testing can be applied to both old and new structures. For new structures, the principal applications are likely to be for quality control or the resolution of doubts about the quality of materials or construction. The testing of existing structures is usually related to an assessment of structural integrity or adequacy. In either case, if destructive testing alone is used, for instance, by removing cores for compression testing, the cost of coring and testing may only allow a relatively small number of tests to be carried out on a large structure which may be misleading[14].

Bridge owners and engineers are urged to ask themselves some important questions as to what exactly will be done with the field data prior to taking any measurements[15]

Non-destructive load testing can be used as a tool to better understand the field behavior of bridges. In general no detrimental effects of deterioration could be determined from the results of the bridge load tests[11].

There is consensus on the importance of objectively and reliably assessing the condition and load capacity of aged bridges. Although each bridge may be considered as a unique structure, the behavior of many bridge types may be governed by only a few mechanisms and related parameters, especially if a population is constructed from standard designs. By identifying these parameters, and their variation within the population, it is possible to extend findings such as load rating obtained from a statistical sample to the entire population[4].

A Project was started with an aim to assist ERA to adopt the bridge management cycle, with which bridges are properly maintained and service level of road network is improved. Although the shortage of trained engineers had initially stagnated the progress of the Project activities, after the introduction of bridge management support service, the activities were steadily carried out and the execution of the rehabilitation and replacement works are likely to be steadily implemented[12].

2.2 STRENGTH EVALUATION OF BRIDGES

In these thesis, the procedure of strength evaluation of bridge is same as the procedure of E.R.A manual. But for estimation of concrete capacity, area of reinforcement and yield strength of steel will be established from numerical method developed by these theses.

These proposed guidelines establish a methodology for rating existing bridges. They are mainly based on AASHTO . The guidelines address several shortcomings of existing evaluation procedures. The methodology is developed within a framework that provides for a systematic rating improvement in the evaluation process. Moreover, the methodology can be used in conjunction with a wide range of engineering practices.

The methodology presented utilizes Load and Resistance Factor Design (LRFD). This procedure allows for combining probability theory, statistical data and engineering judgment into a rational decision making tool. In particular, the procedure allows the engineer to use site specific information in a consistent manner to improve, if necessary, his judgment on the safe rating level for a particular bridge. In addition, the format incorporates existing methodology for considering local laws and regulations and methods of calculation .

A load and resistance factor approach was also chosen as the basis for strength evaluation of existing bridges as it conforms to the design methods for new bridges specified earlier in this manual, while still allowing for a systematic consideration of the differences involved in bridge evaluation. This approach allows each variable to be addressed separately, analyzed in depth (if needed), and proportionally weighed in the overall rating process.

Conservative assumptions are made in each step of a strength design or checking procedure to safeguard against the worst possible conditions expected to occur during the lifetime of a structure. In other words, the probability of failure is made exceedingly small by providing large safety margins to cover the uncertainties in predicting load effects and resistance of a bridge. Reliability principle utilizing site data have been used to evaluate the uncertainties and the safety levels or indices implicit in current designs.

The rating methodology and load and resistance factors have been developed to maintain consistent safety levels for the above-mentioned uncertainties. Options for incorporating site specific traffic and loading data and higher levels of effort by the engineer are introduced since these lead to a reduction in the overall uncertainty. The lower safety margin required maintaining the same safety level means ratings that are more beneficial. At no stage is it necessary for the evaluation engineer to use probabilistic methods. The necessary reliability-based load and resistance factors have been tabulated for the evaluation.

Load and resistance factors were calculated from the coefficient of variation of actual load effects and resistances, the ratio of the mean value to nominally determined values (i.e., the bias) and the desired safety level.

Therefore, as the evaluator obtains more data on the distribution of actual load effects and resistances, more realistic load and resistance factors can be utilized.

This methodology is intended for evaluating almost all existing bridges. Steel spans bridges include simple and continuous girder bridges and trusses and floor systems. Concrete spans+ recognized include slab, girder, T-beam and box beam bridges with short to medium span length. Prestressed beams although of recent vintage are also included herein.

The procedure for rating existing bridges requires knowledge of the physical conditions of the bridge and the applied loadings. A safe level of rating presupposes that nominal strengths should be estimated from a detailed investigation of the structure's physical condition and any continuing attempts to alleviate any signs of deterioration. Further knowledge of traffic conditions including signs of overweight vehicle combinations combined with accurate methods of structural analysis should be used when necessary to estimate load effects.

The load and resistance factors (LRFD) that must be applied should rationally recognize the corresponding uncertainties in making these judgments on strength, analysis and loading. The concepts of structural reliability are a means for consistently representing these uncertainties and allowing bridge engineers to select proper load and resistance factors for rating specific bridges.

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

To compensate for this lack of knowledge and information, engineers have used safety factors to insure that failure does not occur. The Load and Resistance Factor Design (LRFD**)** has been introduced in design and rating to provide more uniform safety. The method implicitly recognizes that dead load effects may require lower safety margins than comparable live (truck) load effects due to their relative uncertainty. This probabilistic approach to safety is logically extended in the load and resistance factor methods used herein.

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 consideration. If the bridge rating is not acceptable, several options for a more detailed analysis are given. Each of these options are associated with an increasing level of effort and shall be done if the rating engineer warrants their use. An initial screening level, however, is provided for routine investigations[.

Some advantages of this method are:

a) It provides uniformly consistent procedures for evaluating existing bridges.
b) It permits suitable flexibility in making evaluations.
c) It provides uniform levels of reliability developed from performance histories.
d) It is based on extensive truck traffic and bridge response data.
e) It permits introduction of site specific data into the evaluation in a rational and consistent format.
f) It permits different levels of effort that involve progressively more work; with correspondingly greater rewards in terms of more beneficial ratings.
g) It includes the same nominal dead and live load calculations and resistances as in the design of new bridges.
h) It allows distinction between evaluation of redundant and no redundant components.

Evaluators will find options in these guidelines by which ratings can be improved by recommendations for more frequent and detailed inspection and maintenance, improved structural analysis and especially control of heavy overweight vehicles.

These guidelines are intended to produce rating factors for routine evaluation and posting considerations. Evaluation of live load for issuance of permits may require load factors different from rating and shall also utilize the actual vehicle size, weight and configuration.

Each of the steps in the evaluation process shall be performed in any one of several ways. Therefore, the proposed guidelines are general enough to accommodate the practices of different engineers and/or agencies. The load and resistance factors presented in the guidelines were developed on the principle that the accuracy of an evaluation was dependent, in part, on the methods used to perform the evaluation.

For economic reasons, it is desirable to keep the evaluation effort to a minimum. If the capacity of a bridge can be shown to be sufficient by making some approximations, there is no need to resort to an expensive evaluation procedure. On the other hand, if the sufficiency of a bridge cannot be reliably established using a more approximate method, an engineer may wish to resort to a more sophisticated approach in order to demonstrate the sufficiency of the bridge. Therefore, the evaluation process outlined in the guidelines is a cyclic process in which one or several of the steps shall be repeated.

The various options provided in the guidelines along with corresponding land/resistance factors have been developed so as to maintain an adequate level of safety based on calibration with existing performance experiences.

The evaluation procedures presented herein therefore provide a balance between safety and economics.

The single load rating value produced by these guidelines shall be greater than current operating ratings for well-maintained, non-deteriorated and redundant load path bridges having reasonably well enforced traffic. It may fall, however, even below existing inventory levels for heavily deteriorated bridges or those having non-redundant components and subjected to heavy truck traffic. A gradation of ratings between these two extremes will be obtained depending on the condition of the bridge, type and volume of traffic, the quality of inspection and the regularity of maintenance. Thus, a deficient bridge shall be made to rate sufficiently if certain preventive measures such as load control restriction, inspection, etc. are undertaken. A variety of options may exist and the engineer could choose one of them depending on the economics of the situation and the amount of effort the engineer is willing to expend.

Evaluators will find options in these guidelines by which ratings can be improved by recommendations for more frequent and detailed inspection and maintenance, improved structural analysis and especially control of heavy overweight vehicles[1].

These guidelines are intended to produce rating factors for routine evaluation and posting considerations. Evaluation of live load for issuance of permits may require load factors different from rating and shall also utilize the actual vehicle size, weight and configuration.

Each of the steps in the evaluation process shall be performed in any one of several ways. Therefore, the proposed guidelines are general enough to accommodate the practices of different engineers and/or agencies. The load and resistance factors presented in the guidelines were developed on the principle that the accuracy of an evaluation was dependent, in part, on the methods used to perform the evaluation.

For economic reasons, it is desirable to keep the evaluation effort to a minimum. If the capacity of a bridge can be shown to be sufficient by making some approximations, there is no need to resort to an expensive evaluation procedure. On the other hand, if the sufficiency of a bridge cannot be reliably established using a more approximate method, an engineer may wish to resort to a more sophisticated approach in order to demonstrate the sufficiency of the bridge. Therefore, the evaluation process outlined in the guidelines is a cyclic process in which one or several of the steps shall be repeated.

The various options provided in the guidelines along with corresponding land/resistance factors have been developed so as to maintain an adequate level of safety based on calibration with existing performance experiences. The evaluation procedures presented here in therefore provide a balance between safety and economics.

The single load rating value produced by these guidelines shall be greater than current operating ratings for well-maintained, non-deteriorated and redundant load path bridges having reasonably well enforced traffic. It may fall, however, even below existing inventory levels for heavily deteriorated bridges or those having non-redundant components and subjected to heavy truck traffic. A gradation of ratings between these two extremes will be obtained depending on the condition of the bridge, type and volume of traffic, the quality of inspection and the regularity of maintenance. Thus, a deficient bridge shall be made to rate sufficiently if certain preventive measures such as load control restriction, inspection, etc. are undertaken. A variety of options may exist and the engineer could choose one of them depending on the economics of the situation and the amount of effort the engineer is willing to expend.

2.2.1 Available record and data gathering

Although the selected bridge are built recently, because of lack of proper filing system, getting full design data with updates are difficult to find. Thus hard copy is the option that is left with.

With regard to traffic data, many type of traffic loading pass over them. i.e trucks, lorries, heavy machineries, house automobiles, e.t.c. currently the most available loading type is sinotruck because there are big construction works .

.

There are many T- girder bridges in Ethiopia. T-girder bridges located in addis abeba are preferred because of their location to perform test with minimum transport cost to reach the site. Out of these T-girder bridges, those whose design data or hard copy of their construction are selected so that the result of deflection test can be compared with first design data.

Available records of the selected bridges are hard copies. In the data gathering of these T-girder bridges, dimension, cross section, compressive strength, area of reinforcement and yield strength of concrete are mentioned on the hard copy. Cross section and longitudinal dimension can be measured from site.

Data for four girder bridges were obtained from A.A.C.R.A but three girder bridges are tested due to technical issues. The bole bridge has a longitudinal span of 18m,crossectional width of 5m. The T-section has web width of 0.47m and height of 1.6m. Most of the time outomobile of less axle are seen passing on these bridge but Construction machineries and trucks pass through these road.

The bulbula bridge has a longitudinal span of 20m ,crossectional width of 4m. The T-section has web width of 0.47 and height of 1.6m. Most of the time Construction machineries and sinotrucks pass through these road.

The saletemeheret bridge has a longitudinal span of 15m ,crossectional width of 5m. The T-section has web width of 0.38 and height of1.4m. Most of the time Construction machineries and sinotrucks pass through these road.

2.2.2 Capacity determination

With regard to E.R.A manual, strength evaluation is done with calculation of rating factor. Then the result will allow ratio of legal truck mentioned in the manual to be pass over the bridge or not.

When coming to this Thesis, first concrete strength is evaluated using hammer test. Second deflection of bridge girder is measured. Using numerical method, area and yield strength of reinforcement will be estimated. Using these newly developed values we go to rating equation of E.R.A manual and calculate the rating factor. Then the result will allow ration of legal truck mentioned in there manual to pass over the bridge or not.

2.2.3 Determination of load effects

DEAD LOAD

Dead load for the middle strip is calculated from the weight of wearing surface and deck slab. Dead load for the edge beam is calculate form the weight of wearing surface, deck slab, curbs/edge beams, posts and railing.

LIVE LOAD

The three legal loads and legal lane loading are used to determine the live load action effects. The approximate method of analysis as given in ERA Bridge design manual is used.

- interior longitudinal beam (live load distribution for moment)

  - One design lane loaded:

$$0.06+\left[\frac{S}{4300}\right]^{0.4}\quad\left[\frac{S}{L}\right]^{0.3}\quad\left[\frac{K_g}{Lt_s{}^3}\right]^{0.1}\qquad(2.1)$$

- Two or more design lanes loaded:

$$0.075+\left[\frac{S}{4300}\right]^{0.06}\quad\left[\frac{S}{L}\right]^{0.2}\quad\left[\frac{K_g}{Lt_s{}^3}\right]^{0.1} \tag{2.2}$$

If number of beams is equal to three, use lesser of the values obtained from the equation above or the lever rule.

$$K_g = n(I + Ae^2) \tag{2.3}$$

$$n = E_B/E_D \tag{2.4}$$

➢ exterior longitudinal beam (live load distribution for moment)

- One design lane loaded: Use lever rule
- Two or more design lanes loaded:

$$g = e \text{ x } g_{\text{interior}} \tag{2.5}$$

$$e=0.77+\frac{d_e}{2800} \tag{2.6}$$

- If number of beams is equal to three, use lesser of the values obtained from the equation above or the lever rule.

➢ exterior longitudinal beam (live load distribution for shear)

- One design lane loaded

$$0.36+\frac{S}{2800} \tag{2.7}$$

- Two or more design lanes loaded:

$$0.2+0.36+\frac{S}{3600}-\left[\frac{S}{10700}\right]^{2} \tag{2.8}$$

- If number of beams is equal to three, use the lever rule.

➢ exterior longitudinal beam (live load distribution for shear)

- One design lane loaded: Use lever rule

- Two or more design lanes loaded

$$g = e \times g_{nterior} \qquad (2.9)$$

$$e=0.6+\frac{d_e}{3000} \qquad (2.10)$$

- If number of beams is equal to three, use the lever rule.

Where:

L = span of beam (mm)

S = spacing of supporting components (mm)

t_S = deck slab thickness (mm)

K_g = 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)

➢ *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:

$$V_n = V_{e+} \ V_s \qquad (2.11)$$

or

$$V_n = 0.25f\ bd \qquad (2.13)$$

For which

$$V_c = 0.083\ \beta \sqrt{fc'} b_v b_v \qquad (2.14)$$

And

$$V_s = \frac{A_V f_V d_V (\cot\theta + \cot\alpha)\sin\alpha}{s}$$

where

b_V = effective web width taken as the minimum web width within the depth d_V (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:

$$\beta = 2.0$$

$$\theta = 45^o$$

- Moment

flexural strength is calculated by taking the rectangular stress block .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 ofthe steel reinforcement required for a balanced condition.

Deflection, neutral axis,Area of steel [2],

$$\Delta=\frac{Pbx}{6EIeL}*[L^2 \ - b^{\ 2} \ - x^2 \ ] \qquad (2.16)$$

$$I_e=\left(\frac{Mcr}{\mathrm{Ma}}\right)^3 I_g+\left[1-\left(\frac{Mcr}{\mathrm{Ma}}\right)^3\right]*I_{cr} \qquad (2.17)$$

$$I_{cr}=\frac{bx^3}{3}*nA_s\,(d\text{-}x)^2 \qquad (2.18)$$

$$X=\frac{nAs}{b_e}+\sqrt{\left(\frac{nAs}{b_e}\right)^2+\frac{2nAs*d}{b_e}} \qquad (2.19)$$

- Compressive strength of concrete is determined using hammer test. One of the most popular non-destructive methods of concrete testing is the Baltic states is carried out by using the Schmidt rebound hammer. The use of this method is practiced on large scale on building sites throughout Lativia,Lithuania and Estonia. This method has gained its popularity by its simple use and the possibility of using it on a single concrete surface without requiring accesss to the construction from both sides, as is necessary for ultrasonic testing methods.

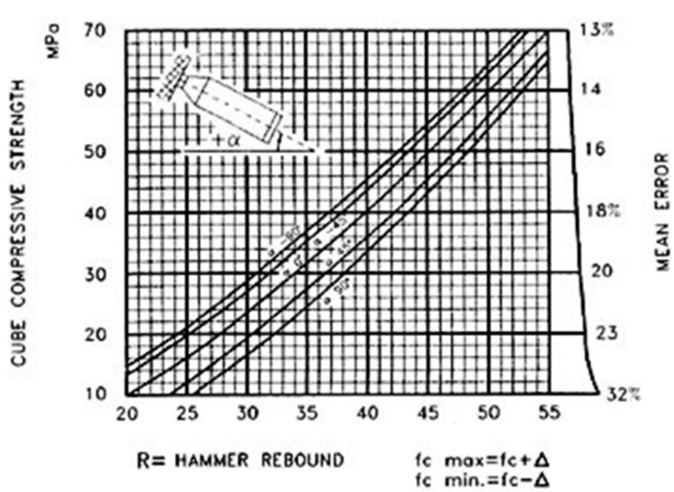


Fig 2.3 hammer rebound [9]

2.2.4 Rating for evaluating safety

Each bridge carrying vehicular traffic shall be rated to determine its safe load carrying capacity. If it is determined that the maximum legal load configurations exceeds the load allowed at the bridge, then the structure shall be posted for load restriction. The notice shall caution all persons against driving on the bridge a loaded conveyance of greater weight than the bridge's carrying capacity. The use of warning signs shall be based on an engineering study. Bridges that do not have sufficient capacity under the legal loads rating should be posted for load restriction for immediate solution and considered for strengthening for the future.

2.2.5 The rating equation [1]

The evaluation is carried out with a comparison of the factored live load effects and the factored strength or resistance. The load factors are used to account for uncertainties in load effects due to uncertainties in analysis as well as load magnitudes. The dead load factor includes normal variations in material dimensions and densities.

The live load factor accounts for uncertainties in expected maximum vehicle loading effect, impact and distribution of loads during a time period between inspection. The resistance factor accounts for uncertainties in strength prediction theories, material properties and deterioration influences over time periods between inspection. Furthermore, the load and resistance factors are adjusted to produce an overall safety margin which leads to an adequate level of safety considering all uncertainties described above.

The rating procedure is carried out for all strength checks (moment, shear, etc.) at all potentially critical sections with the lowest value determining the rating factor for the entire span. The rating equation to be used throughout the application of these guidelines is [1]:

$$\emptyset R_n = \gamma_D D + \gamma_L (RF)\, L\, (1 + I) \tag{2.20}$$

$$RF = \frac{\emptyset R_n - \gamma_D D}{\gamma_L * L(1+I)} \tag{2.21}$$

Where: RF = rating factor (the portion of the rating Legal Truck allowed on the bridge)

Ø= resistance factor

Rn = nominal resistance

YD = dead load factor

D = nominal dead load effect

YL = live load factor

l = nominal traffic live load effects

L = nominal live load effect

I = live load impact factor

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. In determining load and resistance factors for the rating equation, the following steps shall be carried out in evaluating a bridge span:

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

A flowchart for the rating procedure is also provided in Figure 2-1. The evaluator/designer should note that potential improvement in the rating factor may came from selecting options in each step. These generally provide a less conservative factor provided additional evaluation effort is performed and no unsatisfactory information is uncovered. The basic structural engineering equation states that the resistance of a structure must equal or exceed the demand placed on it by loads. Stated mathematically:

$$R = \sum Q_K \qquad (2.22)$$

Where: R = resistance

Qk = effect of load k

The solution of this simple equation encompasses the whole art and science of structural engineering including the disciplines of strength of materials, structural analysis and load determination. This equation applies to design as well as evaluation. In structural evaluation, the objective is to determine the maximum allowable live load. In the case of bridge evaluation, this usually means the maximum vehicle weight.

Any rational and tractable approach to the analytical solution of the basic structural engineering equation requires that the modes of failure be identified to establish the resistance. The location, types and extent of the critical failure modes must be determined. The checking equation must be solved for each of these potential failure checking modes.

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:

$$RF = \frac{\emptyset R_n - \sum_{i=1}^{m} \gamma_i{}^D * D_i - \sum_{i=1}^{m} \gamma_j{}^L L_j(1+I)}{\gamma^{LR} * L_R(1+I)} \tag{2.23}$$

Where: RF = rating factor (the portion of the rating Legal Truck allowed on the bridge)

φ = resistance factor

m = number of elements included in the dead load

R_n= nominal resistance

n = number of live loads other than the rating vehicle

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

D^i = nominal dead load effect of element “i”

$\gamma_j{}^D$= live load factor for live load “j” other than the rating vehicle(s)

Lj = nominal traffic live load effects for load “j” other than the rating vehicle(s)

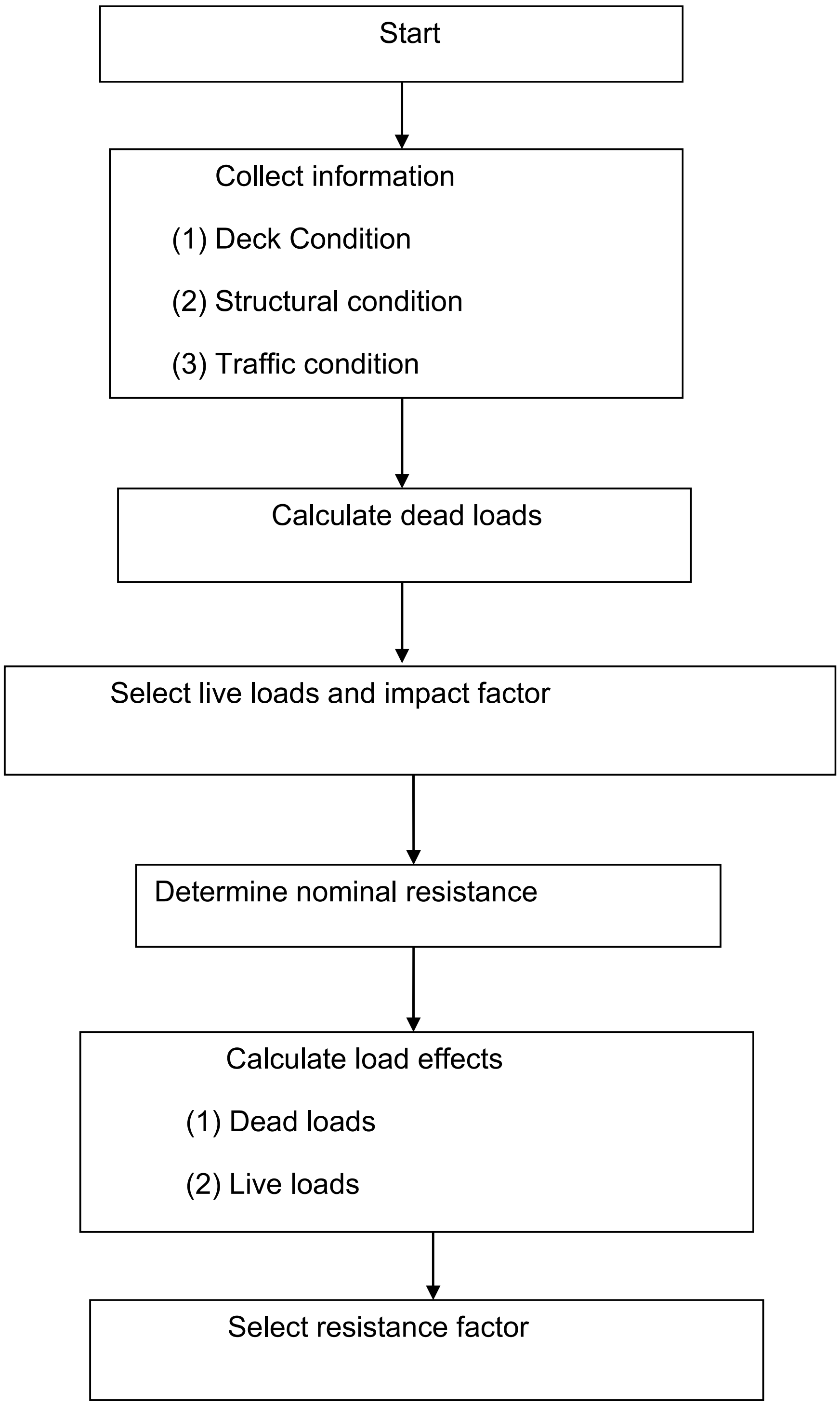
γ^{LR} = live load factor for rating Legal Truck

L_R = nominal live load effect for the rating Legal Truck

I= live load impact factor

The maximum permitted traffic live load effect will be the total resistance minus the effect of loadings other than the rating Legal Truck. This will include dead loads, non-vehicular live loads, and, in the case of unsupervised permit loading, the vehicular live load and the impact of normal traffic that could mix with the rating Legal Truck.

Flow chart is presented below where it start with collecting information of bridge and end up with determining the safelty level of bridge is acceptable or not. Detail information of the flow chart are presented in following pages after the chart.



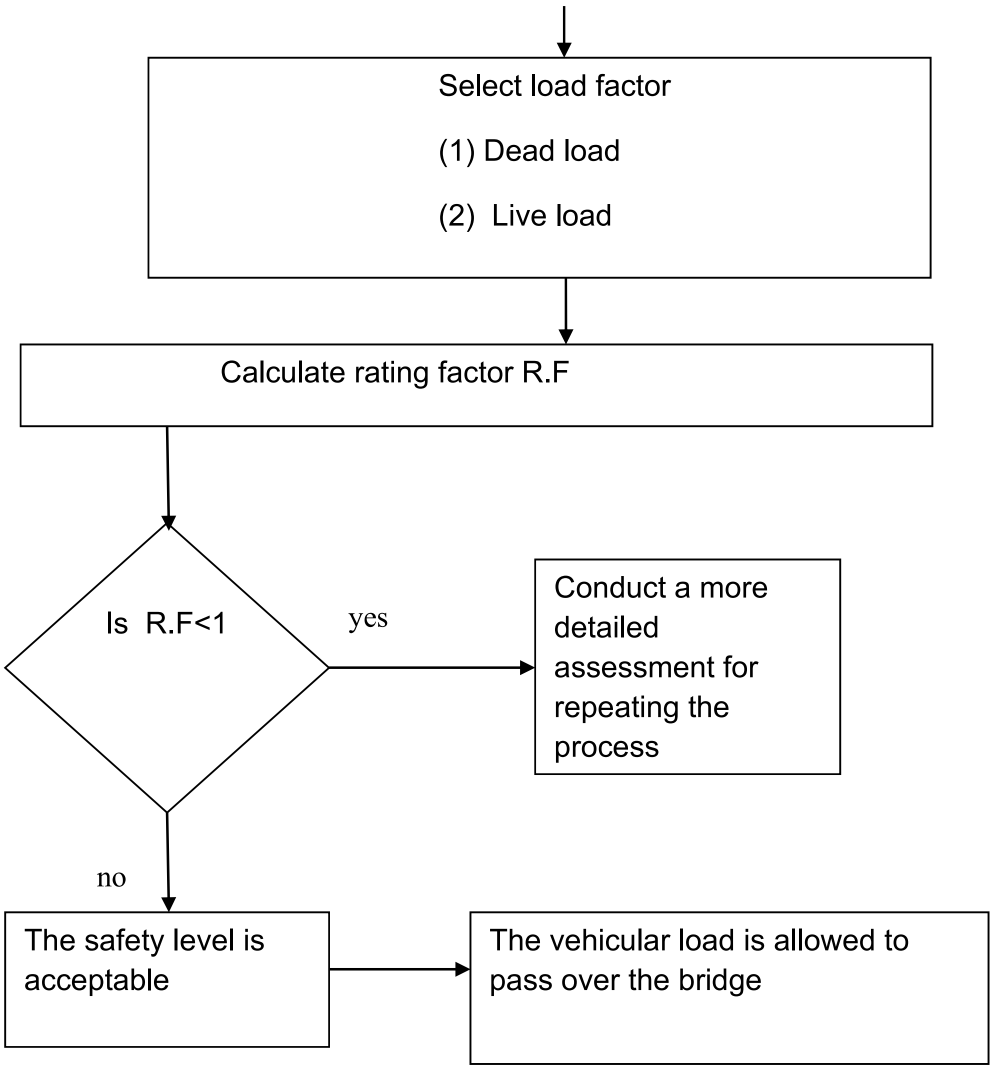


Fig 2.4 Flow Chart for Rating Procedure [1]

2.2.6 Collection of information

Before the load rating of a specific bridge can be conducted, a certain amount of information has to be gathered. The extent to which the engineer is required to collect information will have a direct influence on the load rating of the bridge due to the selection of the proper category for the load and resistance factors[1].

This task shall be the same as the provisions in the existing except that the following items should be noted since they can have an influence on the selection of load and resistance factors[1].

- Deck condition – *Live Loads* are deliberately selected to be conservative with respect to most conditions. 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.

- 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. Conversely, maintenance efforts to mitigate such deterioration should also be noted. An allowance for structural deterioration should note whether this is either an expected or conservative estimation since further deterioration may increase the uncertainty regarding reliable section properties and strength during the next inspection interval.

- Traffic Condition - The expected loading during the inspection internal is affected by the truck traffic at the site. In the best instance, data will be available from traffic surveys including objective truck weight operations. Alternatively, advice should be sought from the traffic division regarding truck traffic volume, composition, permit activities, overload sources, and degree of enforcement.

2.2.7 Selection of nominal loading and resistances

Loads consist of concentrated or distributed forces that are applied directly to the bridge or result from deformations or the constraint of deformations. 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, *the Load Requirements* load factors for design shall be used.

2.2.8 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 to be used in computing the dead load should be as in Table 2-1 :

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

TABLE 2-2 Unit Weights of Materials [1]

The dead load of the structure is computed in accordance with the conditions existing at thetime of the analysis.

Dead load can usually be determined more accurately than any other type of loading. One major source of error is failure to consider some of the elements that will contribute to dead load. Some items that are often overlooked are:

- Wearing surfaces
- Railings and Utilities
- Structure modifications not shown on plans

Other items that can affect the calculation of dead load are dimensional variations in the concrete section and variations in the unit weight of material. 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.

2.2.9 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. When unusual conditions exist, adjustments to the specified number of vehicles should be made.

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 Figure 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.

The moving loads to be applied on the deck for calculating maximum nominal live loading effects shall be the three Legal Trucks. The spacing and axle weights chosen for these vehicle types were selected from actual trucks. It is believed that these typical vehicles correspond better to existing traffic and will provide more uniform reliability than the old standard AASHTO H or HS design trucks. Hence, the latter are not recommended for bridge posting purposes.

In computing load effects, one Legal Truck shall be considered present in each lane. It is unnecessary to place more than one vehicle in a lane since the load factors shown below have been modeled for this possibility. These load factors shall be considered applicable for spans up to 60m.

For longer spans, a lane loading is specified in the evaluation. Reduction factors for live loading of more than two traffic lanes are provided. These rationally account for the lower possibility of such occurrences.

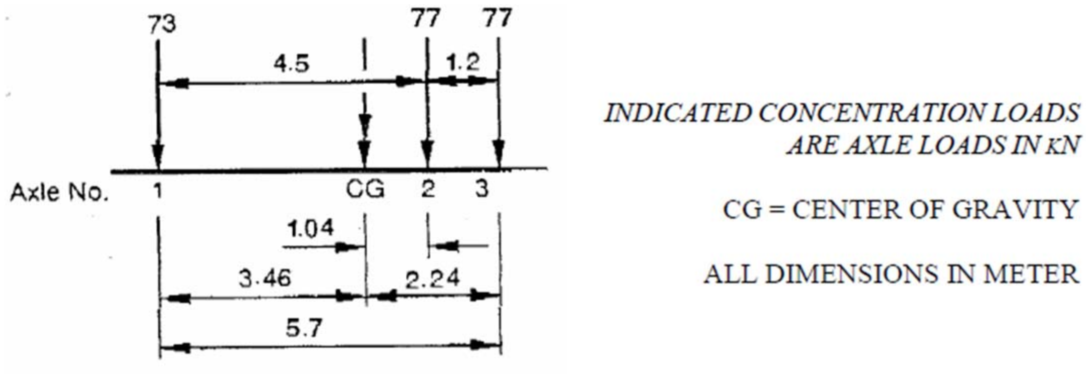


Fig 2-5 Truck Type 3 Unit Weight = 227 kN [1]

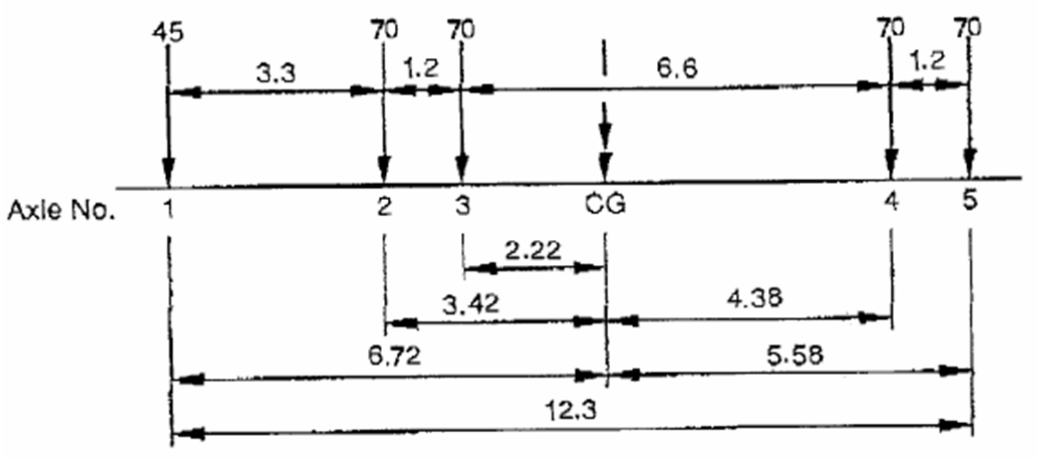


Fig 2-6 Truck Type 3-2 Unit Weight = 325 kN [1]

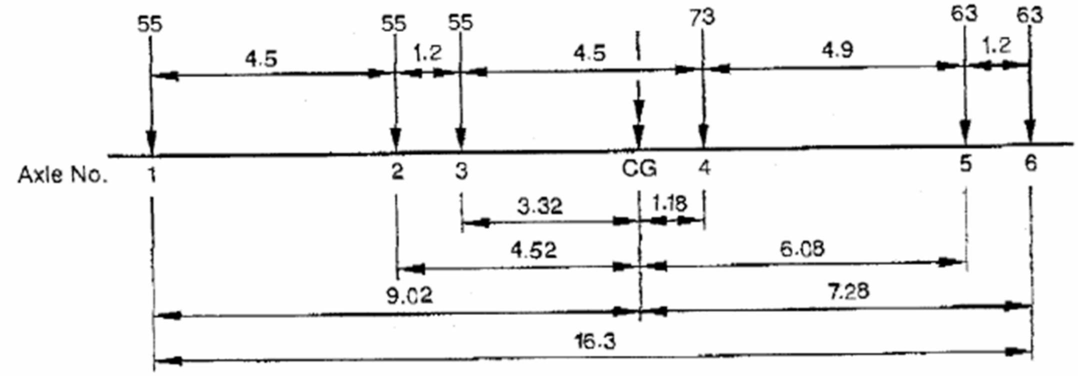


Fig 2-7 Truck Type 3-3 Unit Weight = 364 kN [1]

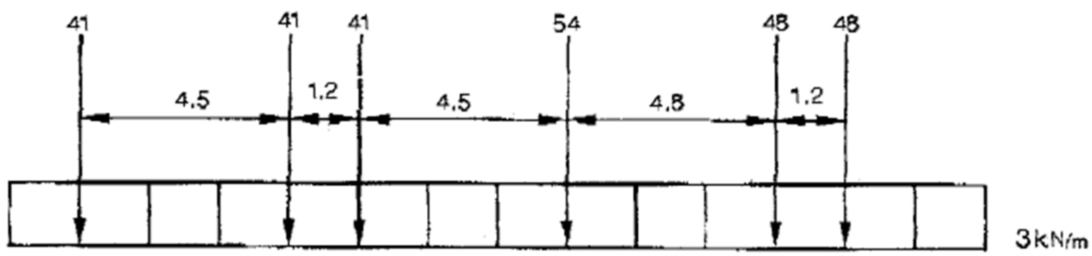


Fig 2-8 The Legal Lane Loading (mainly for large spans) [1]

For longer spans, the Legal Lane Loading given in Figure 2-5 will govern the evaluation (up to 90 m). 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 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 Figure 2-5. Where maximum load effects in any member are produced by loading a number of traffic lanes simultaneously, reduction factors as given in Table 2-8 should be applied.

In checking special permits, the actual vehicle weights and dimensions shall be used. If the number of such permits in one year are frequent, then it shall be assumed that two lanes are occupied by such a vehicle. Otherwise, standard vehicles shall be placed in the other lanes. When the engineer determines that conditions of traffic movement and volume warrant it, the standard vehicles shall be eliminated. Upon special investigation, the load factor for a controlled permit use is reduced below the value taken for ordinary traffic conditions.

Since overload permissible vehicles typically have very different axle configurations, it is very important that this be considered when issuing permits.

Judgment must also be exercised concerning sidewalk loadings. The likelihood of the maximum sidewalk loading is small. A unit loading for the sidewalk for the purposes of load limit evaluation will generally be less than the design unit loading.

The probable maximum sidewalk loadings should be used in calculations for safe load capacity ratings. This loading will vary from bridge to bridge, depending generally upon its location. Because of this variation, the Engineer must use his judgment to make the final determination of the unit loadings to be used.

E.R.A (Ethiopian road authority) has data of axle load that comes in and out of addis abeba and other cities. The way it is collected is not for one vehicle but axle data is collected for each wheel from different vehicles. The clear one is the data where allowed vehicle load is described for sino trucks. It consist of F1=80kN, R1=140kN, and R2=140KN (F=front, R=rear) .

2.2.10 Impact

An impact allowance shall be added to the static loads used for rating as shown in Equation 2.1. Impact values 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. 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. For span ≤ 12.0 m, where the measured deflection exceeds 1/90 of the span, 0.10 should be added to these values. See Table 2-2. If such a judgment cannot be made, refer to the bridge inspection report and relate impact to the condition of the wearing surface.

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 continue	0.2
4 - Critical condition	Item no longer functioning as designed	0.3

Table 2-3 Condition of Wearing Surface and Impact Value [1]

2.2.11 Resistances

Nominal strength calculations shall take into consideration the observable effects of deterioration, such as loss of concrete or steel crosssectional area, loss of composite action or corrosion.

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.

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 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.

Reinforcing Steel	Yield Stress F_y (MPa)
Unknown steel (prior to 1954)	228
Structural Grade	248
Intermediate Grade 300 and unknown after 1954 (former Grade 40)	276
Hard Grade (former Grade 50)	314
Grade 420 (former Grade 60)	614
Grade 520 (former Grade 75)	517

Table 2-4 Reinforcing Steel Yield Stresses [1]

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.

These limit states should have a very low probability of occurrence because they can lead to loss of life as well as to major financial losses. They include:

• Loss of equilibrium of all - or part of - the structure considered as a rigid body (e.g. overturning, sliding, uplift, etc.);

• Loss of load-bearing capacity of members due to insufficient material strength, buckling, fatigue, fire, corrosion, or deterioration;

• Overall **i**nstability of the structure (e.g., P-delta effect, wind flutter, seismic motions, etc.);

• Very large deformation (e.g., transformation into a mechanism).

Determination of the true safety limit state involves very complicated and difficult analytical procedures. In most cases, the use of these procedures for routine evaluation of bridges is not economically feasible. The ultimate member capacity shall be a lower bound of the ultimate capacity in shear or in flexure. Different methods for considering the observable effects of deterioration were studied in developing the guidelines. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths. An alternate approach is to calculate resistance based on plan dimensions and use a smaller capacity reduction factor.

Nominal resistances for members in the proposed guidelines are based on the load factor section. This resistance depends on both the current dimensions of the section and the nominal material strength. Specifications for both these factors have been provided. Options exist for incorporating data on structural conditions obtained from the site. Careful estimation of losses and deterioration are awarded a higher resistance factor. Similar gains are also given for vigorous maintenance and inspection schedules, which may prevent further deterioration during a normal inspection interval. Options also exist for obtaining more precise material strength through tests.

Structural Steel

Nominal unit stresses must depend on the type of steel used in the structural member. When tests are performed to assess yield stress, the mean values shall be reduced by 10% to produce nominal values for strength calculations. Nominal values shall be nominal strength computed without any resistance factor applied.

2.2.12 Distribution of Loads

The fraction of vehicle load effect transferred to a single member shall be selected in accordance with Load Requirements. 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). Loadings shall be placed in positions causing the maximum response. Further, if such a measurement or analysis is made and the expected distribution value is obtained, then this shall be adjusted by the factors shown in Table 2-4. The latter are needed to adjust for the expected bias in distribution factors for different material types.

Distribution of Loads		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 2-5.

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

Table 2-5 Correction Factor for Analysis* [1]

Lateral distribution refers to the fraction of the live load carried by the member under consideration. Options exist for using tabulated values, more refined analysis (e.g. finite elements) and field measurements. Each of these options involves a greater level of effort and more accuracy, so adjustments to the basic live load factors are provided. These adjustments implicitly recognize that more refined analysis may in some instances remove the implicit conservativeness present in some simplified distribution formulas and are therefore treated accordingly.

2.2.13 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 ensures that this acceptable level of safety is achieved or exceeded.

I. Load Factor

The load factors shall be taken from Table 2-5. These are intended to represent conditions existing at the time this specification is written based on field data obtained from a variety of locations using weight-in-motion and other data gathering methods. The live load factor accounts for the likelihood of extreme loads side-by-side and following in the same lane and the possibility of overloaded vehicles. Since one aim of this chapter is to protect the investment in the bridge structure, the live load factors do recognize the presence of overweight trucks on many highways. An option to reflect effective overload enforcement is contained herein with a reduced live load factor. The presence of illegal loads has also been noted, and if such vehicles are present in large numbers at the site, the higher load factors may lead to unacceptable ratings and enforcement efforts should be instituted.

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

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 weigh limits.

Table 2-6 Load Factors [1]

When the Rating Factor (R.F.) is less than 1.0, the loads are to be restricted. In such instances, consideration should be given to truck weight surveys and vigorous enforcement programs. If there is a reason to believe that truck posting signs are being ignored then consideration should be given to further raising the live load factor.

Dead load factors are used to account for 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.

Factory produced girders cast in steel forms obviously have less variation than an asphalt overlay placed on the bridge deck. 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. A large amount of filed data has been modeled to estimate the maximum live load effect together with its uncertainty. Based on this data, degree of enforcement, volume and type of traffic are isolated as the major factors influencing the live load effect. The live load factors have been derived from this data for bridges with a single lane, two lanes and three and four lanes. Instead of providing different sets of load factors for different numbers of lanes, only one set of load factors are provided with corresponding reduction factors for other cases. Three categories of live load are provided in Table 2-5 with varying volumes and degrees of enforcement, each with its corresponding live load factor. Site truck traffic data recorded by the engineer may also be included.

II. RESISTANCE FACTORS

A capacity reduction factor **($\emptyset$)** 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.

The resistance factors or capacity reduction factors are intended for new components with current methods of high quality control. The nominal (unfactored) strengths to be used for evaluation represent an estimate of strength using data pertaining to member properties and conditions at the time of inspection. The resistance factor shall consider both the uncertainties in estimating these member properties and also any bias or conservativeness deliberately introduced into these estimates. Because further changes may occur to the section during the inspection interval, there is some dependence of these properties on the quality of maintenance. Also, the level and detail of inspection is important since it may reveal actual properties to be used in section calculations.

A table of resistance factors for all combinations of conditions encountered is given in Table 2-7. A flow chart for obtaining the resistance factors is also presented in Figure 2-6 .

A basic set of resistance factors is provided. The reliability levels are calibrated to produce different resistance factors for redundant and non-redundant spans with the latter having lower (more conservative) factors. The resistance factors can be further modified depending on the amount of deterioration and type of inspection and maintenance. Options exist for conducting detailed measurements of strength losses. Also included are benefits for vigorous maintenance schedules. This allows the evaluation to be flexible enough and also covers a large range of types and conditions of members that shall be encountered .

III. CALCULATION OF RATING FACTORS

The rating factor is to be calculated from Equation 2.20. If it exceeds 1.0, the span is satisfactory for the legal loads in Ethiopia. In the present Bridge Design Specifications, there is only one single rating value (eliminating the operating and inventory levels) which determines the allowable loads.

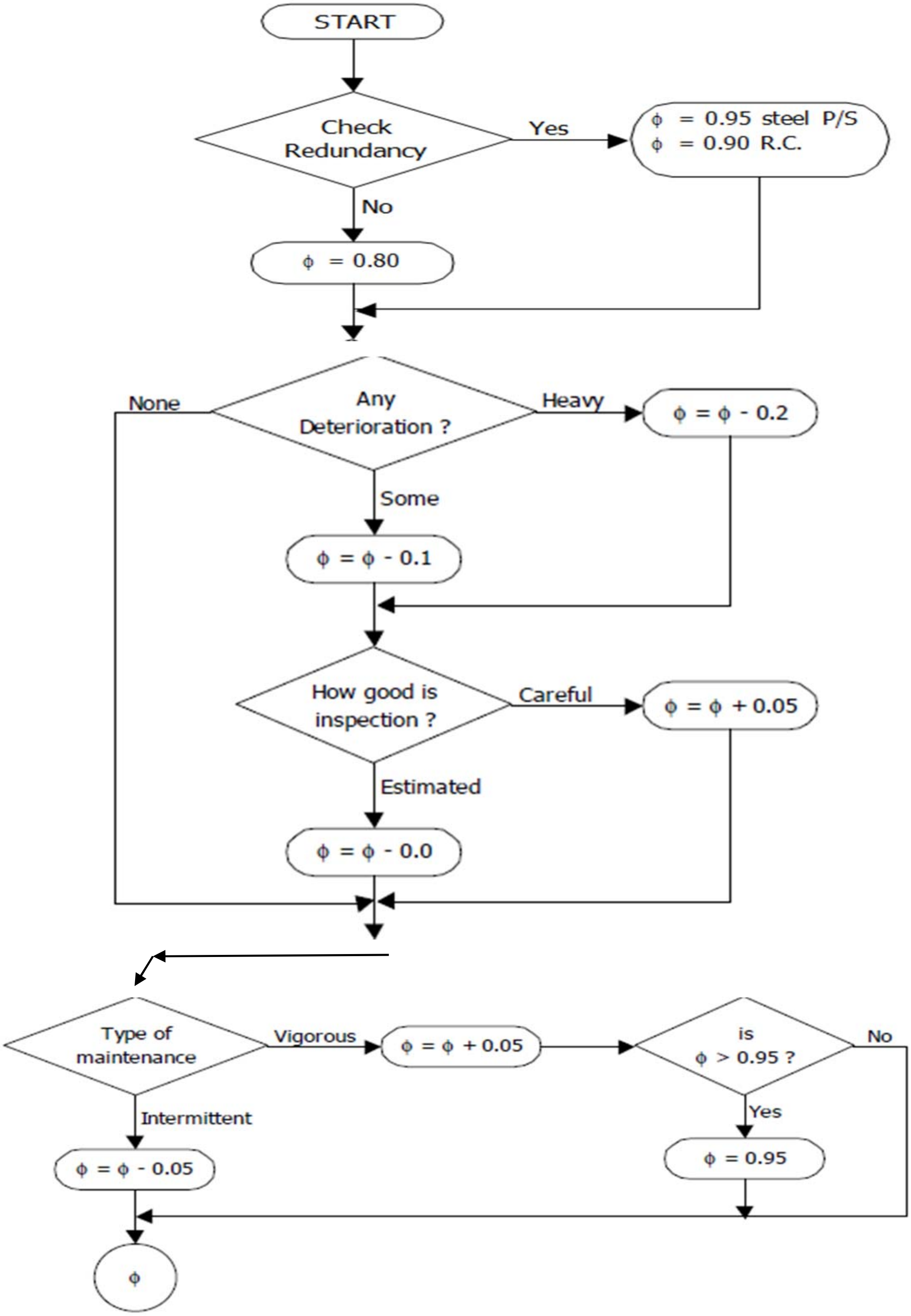


Fig 2-9 Flowcharts for Selecting Resistance Factors [1]

A. Resistance Factors - Good condition

Nominal resistance equations are to be those indicated in *Load Requirements*. Resistance (capacity reduction) factors are to be applied to the following for the case where members are in good condition. Redundant * Steel Members: $\varphi = 0.95$; Nonredundant Steel Members: $\varphi = 0.80$; Prestressed concrete beams: $\varphi = 0.95$; Reinforced concrete beams: $\varphi = 0.90$

B. Influence of Deterioration

- Where field inspection and condition survey reports indicate **no deterioration**, the provisions of this section should not be used.
- Where field inspection and condition survey reports indicate slight deterioration with some possible loss of section, the resistance factor values above shall be decreased by 0.1.
- Where field inspection and condition surveys report significant deterioration and heavy section loss, the resistance factor values shall be reduced by 0.2.

- If such information is not available then bridge records shall be used. Reduce the resistance factor values by 0.1 for superstructure condition of 5 or 6. Reduce the resistance factor values by 0.2 for a superstructure condition of 4 or less. If these reductions are made then the next two sections should be omitted.

C. Inspection*

- Where field inspection and condition survey reports indicate no deterioration, the provisions of this section should not be used.
- Where section losses have been carefully estimated in the calculation of remaining section areas the resistance factors shall be increased by 0.05.
- Where material yield stress has been estimated by physical testing, a mean value of 0.90 shall be used for calculating strength together with the resistance factor contained in the design rules.

D. Maintenance**

- Where maintenance activity is vigorous and likely to correct deficiencies which may lead to further section loss, increase φ by 0.05.
- Where maintenance activity is intermittent and may not correct defects that have lead to section loss, decrease φ by 0.05.

*Examples of redundant members include parallel stringers (three or more), parallel eye bars (four or more). Example of nonredundant component includes two-girder system(s) and trusses with single members.

** In no instance shall Ø be taken to exceed 0.95.

Superstructure Condition	Redundancy		Inspection		Maintenance		Steel, P/S Concrete	Reinforced Concrete
	Yes	No	Careful	Estimated	Vigorous	Intermittent		
Good or Fair	x		x		x		0.95	0.95
	x		x			x	0.90	0.85
	x			x	x		0.95	0.95
	x			x		x	0.90	0.85
		x	x		x		0.85	0.80
		x	x			x	0.75	0.70
		x		x	x		0.85	0.80
		x		x		x	0.75	0.70
Deteriorated	x		x		x		0.95	0.90
	x		x			x	0.85	0.80
	x			x	x		0.90	0.85
	x			x		x	0.80	0.75
		x	x		x		0.80	0.80
		x	x			x	0.70	0.70
		x		x	x		0.75	0.75
		x		x		x	0.65	0.65
Heavily Deteriorated	x		x		x		0.85	0.80
	x		x			x	0.75	0.70
	x			x	x		0.80	0.75
	x			x		x	0.70	0.65
		x	x		x		0.70	0.70
		x	x			x	0.60	0.60
		x		x	x		0.65	0.65
		x		x		x	0.55	0.55

Note : For ratings using data obtained from plans only, the capacity reduction factor should be calculated based on judgment of the engineer supplemented by any additional information obtained.

Table 2-7 Resistance Factors φ for All Conditions [1]

The load and resistance factors have been calibrated to provide adequate safety under the inspection, maintenance, analysis, redundancy, and loading conditions cited. These provisions have the capability for evaluations to be improved by utilizing options related to more intensive inspection and maintenance or control of heavy overloads.

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 ofpermits that shall be issued. Fatigue life should be a consideration in the issuance of overload permits.

Number of Lanes	Reduction Factor
One or two lanes	1.0
Three lanes	0.8
Four lanes	0.7

Table 2-8 Reduction Factors for Live Load [1]

2.3 THEORETICAL DEFLECTION RESPONSE OF EXISTING STRUCTURES.

2.3.1 Permanent loads

The deflection formula[6.12] for dead load deflection is $\Delta=\frac{5wL^4}{384EI}$ …(2.24) but it is assumed that the bridge has already deflected once the formwork of bridge construction is taken off.

2.3.2 Vehicular loads

The deflection formula[6.2] for live load deflection is $\Delta=\frac{Pbx}{6EIeL}*[L^2 - b^2 - x^2]$…(2.25). The live load moment is calculated by putting axle load at position where they give maximum load effect using influence line. The live load moment is added with dead load moment to give total moment. Total moment is used in effective moment of inertia during calculation of deflection.

3. EXPERIMENTAL EVALUATION OF BRIDGE CAPACITY

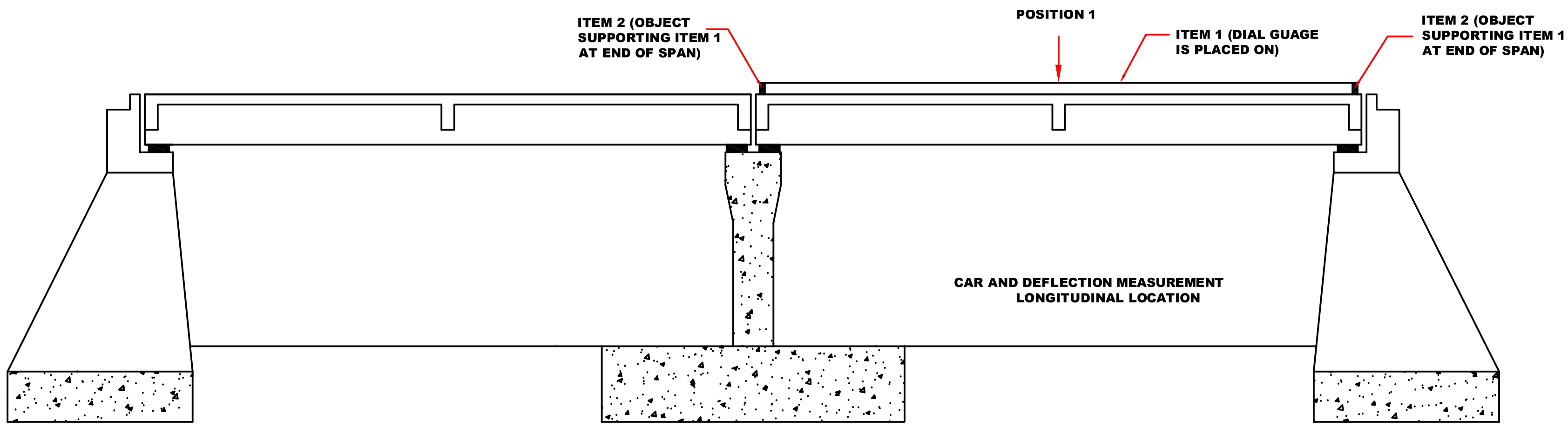
3.1 Background of evaluation problem

In the evaluation problem, deflection test is perfrormed. Deflection reading is taken when vehicular load is applied. From the deflection data, area and yield strength of reinforcement is estimated. Concrete strength is measured using hammer test. These result are used as input to calculate the R.F(rating factor) in E.R.A manual. Then results are posted according to the procedure E.R.A manual.s

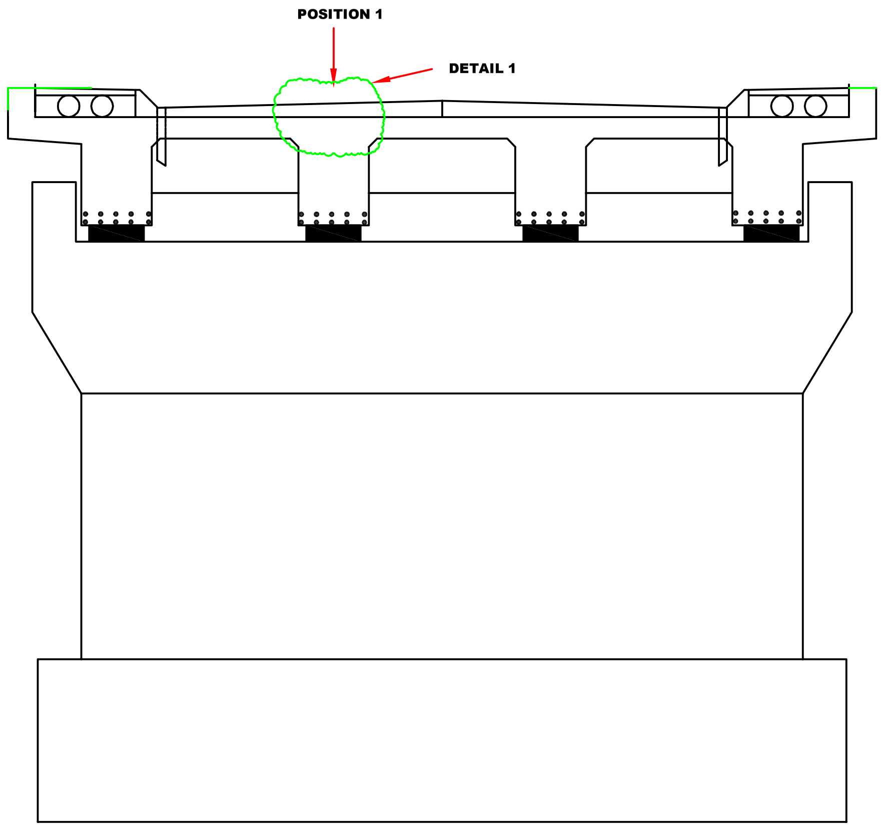
3.2 Experimental setup

On first trial, dial guage was to be put on RHS section(ITEM 1) where the RHS section is supported at the end of the bridge as shown in the fig 3.1. but the RHS section was deflecting from its own dead weight. The minimum size of RHS not to deflect from calculation was 500mm and that is not easy to get and makes it difficult for sinotruck to pass over it since the clearance of sinotruck axle (chasis) supporting the weight of the sinortruck and the road surface is less than 300mm. It lead to a loss of 2,000 birr and other mechanism was sought.

On second trial , the dial guage was place on steel wire where the wire is pulled and pinned at support. Position of dial guage(Item 1) placement and point where deflection data is taken is shown in Fig 3.1.The support (ITEM 2) are shown on fig 3.2 and fig 3.3. Big stones are placed on this item so that they don't topple when the steel wire is stretched to carry the dial guage.



CAR AND DEFLECTION MEASUREMENT
LONGITUDINAL LOCATION



CAR AND DEFLECTION MEASURMENT
CROSSECTIONAL LOCATION

VEHICLE AND DEFLECTION MEASURING
DEVICE LOCATION (FIG 3.1)

Fig 3.2 -steel wire end support

Fig 3.3-steel wire end support with rolling effect

3.2.1 Structure selection

Getting T-Girder bridge design data from E.R.A or A.A.C.R.A is very difficult. There are many T-Girder bridges in addis ababa but getting the design data is difficult. To solve the problem first drawings of civil and reinforcement drawing were searched from library of A.A.C.R.A. Data for four bridges is obtained. Then the bridges on drawing are located on their perspective sites. The other good side is that currently built bridges are according to E.R.A design manual thus we can cross refers with E.R.A manual templates is there is any missing data.

Four sites were selected. Their location is summit road, bole road, bulbula road and akaki. Permission for performing load test on akaki was dismissed on the last day of performing the test. So we are left with three bridges to perform test.

3.2.2 Structure dimension

The Structure dimensions and reinforcement are obtained from hard copies obtained from A.A.C.R.A. common feature are listed below

Summit road T-girder bridge (span=15m, girder depth=1.4m, girder spacing=2.4m)

Bole road T-girder bridge (span=18m, girder depth=1.6, girder spacing=2.4m)

Bulbula road T-girder bridge (span=20m, girder depth=1.6m, girder spacing=3m)

3.2.3 Devise to be used

Dial guage is a device which read deflection. The setup is discussed in section above in experimental setup. The picture is shown below.

Fig 3.4 Dial guage

3.2.4 loads

The vehicular load is a sinotruck measured at sululta car weighting station .The Truck axles are as follows F1=85kN, R1=123KN and R2=120KN.

3.3 Recording and possible problem forecast

When recording is taken, R1 is placed at center of span of beam. The deflection at center of span is taken. Necessary precaution is taken while taking data but error might occur due slight movement of ITEM 2 support, elongation of steel wire at support that are bent.

Traffic flow is stopped not for the whole period where sinotruck comes in and out of bridge but at the time of reading. This is because of the birocracy of traffic chiefs from one kifleketema to another. But still the reading was stable and good enough to show the amount of bridge deflection.

3.4 Discussion on Results with Theory

Deflection from recording for bulbula,bole and summit road bridges are 6.9,3.9 and 4mm respectively, While Deflection using next formula

$$\Delta=\frac{Pbx}{6EIeL}*[L^2 - b^2 - x^2] \quad \text{…. (3.1)}$$

gives 9.1mm,3.9mm and 5.1mm respectively. Then creep deflection is added. While calculating deflection, effective moment of inertia is calculated based on A.C.I code.
The following figure shows possible position of effective moment of inertia.

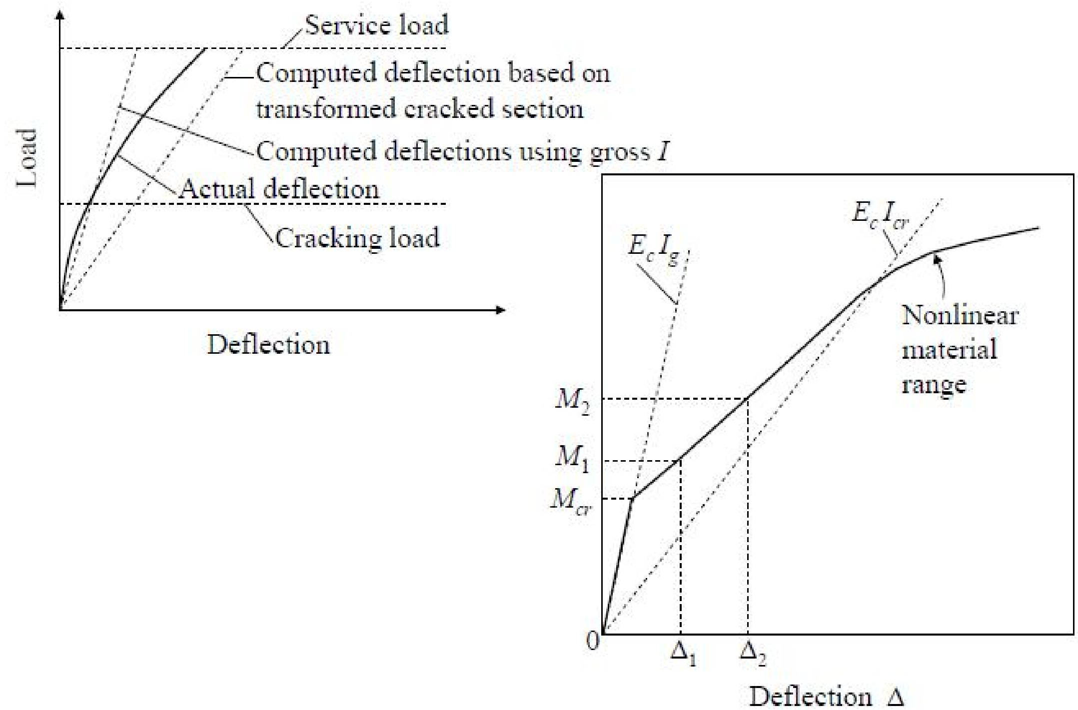


Fig 3.5 variation of flexural rigidity [17]

As we can see from figure above , actual deflection is different from computed deflection using gross moment of inertia as well as cracked moment of inertia. We should expect that since A.C.I code gives formula for estimation of effective moment of inertia, theoretical deflection and actual deflection will not be the same.

Yield strength of reinforcement for bulbula,bole and summit road bridges is calculated using next formula

$$f_s = \frac{nM(d-x)}{Icr} \quad ..... (3.2)$$

Table are shown below to make comparison b/n the result of these thesis and original data as well as data to be assumed using E.R.A manual.

Site	Original data(Mpa)	Hammer test (Mpa)
Bulbula	20	Varies (25-40)
Bole	20	Varies (25-40)
Summit	24	Varies (25-40)

Table 2-9 Concrete strength comparison

Site	Original data	Numerical evaluation For fc'=25Mpa
Bulbula	14476	14500
Bole	14476	14200
Summit	11260	11000

Table 2-10 Area of steel comparison

Site	Original data	E.R.A manual	Numerical evaluation For fc'=25Mpa
Bulbula	410	276	232
Bole	410	276	179
Summit	420	276	193

Table 2-11 steel stress comparison

Site	E.R.A manual	Numerical evaluation For fc'=25Mpa
Bulbula	1.94	1.42
Bole	1.94	1.2
Summit	1.78	1.12

Table 2-12 Ratiing factor comparison

The main objective of these study is to determine the strength capacity of where the above calculated parameters are not known. We see that the area of steel and yield strength of reinforcement have less value than original document. But, the result from this thesis help us to be confident when coming to how much load should pass over the bridge. We are on the safe side.

Hammer test give quite large value of compressive strength than original document . But we have to take into account that compressive strength of concrete increase with time, during construction the supervisor might have made a decision to make it higher grade, e.t.c

Taking into consideration different values of hammer test, the following charts are provides showing variation of reinforcement area as well as yield strength of steel for different values of compressive strength of concrete obtained from hammer test.

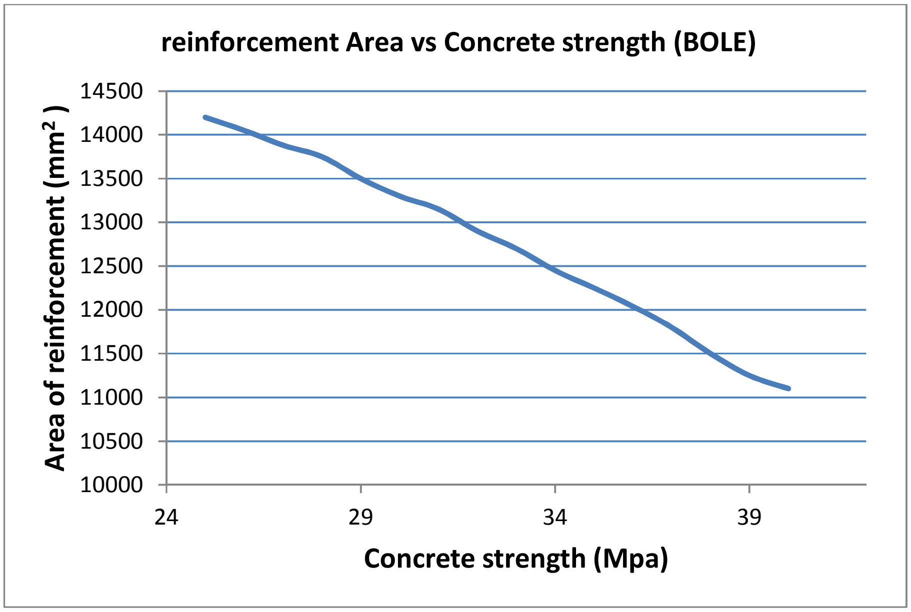


Fig 3.6 Reinforcement area Vs Concrete strength (Bole)

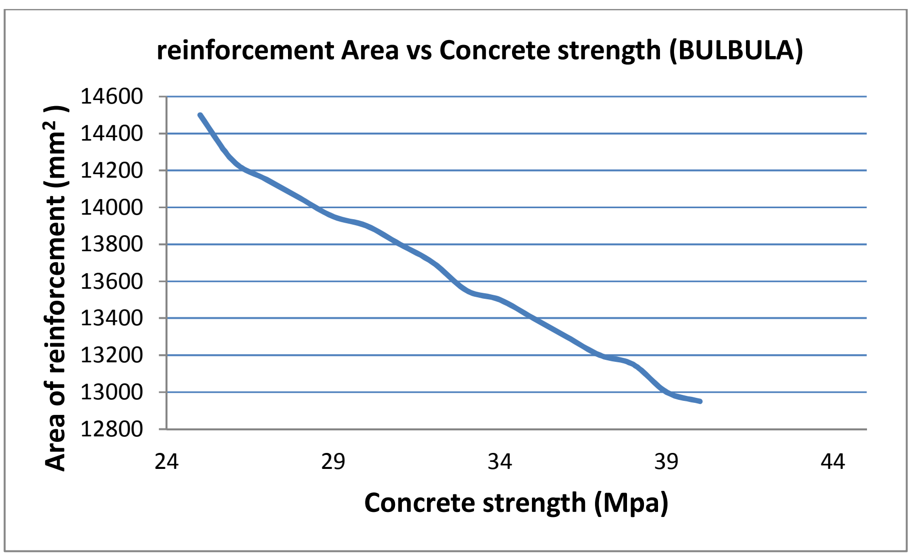


Fig 3.7 Reinforcement area Vs Concrete strengh (Bulbula)

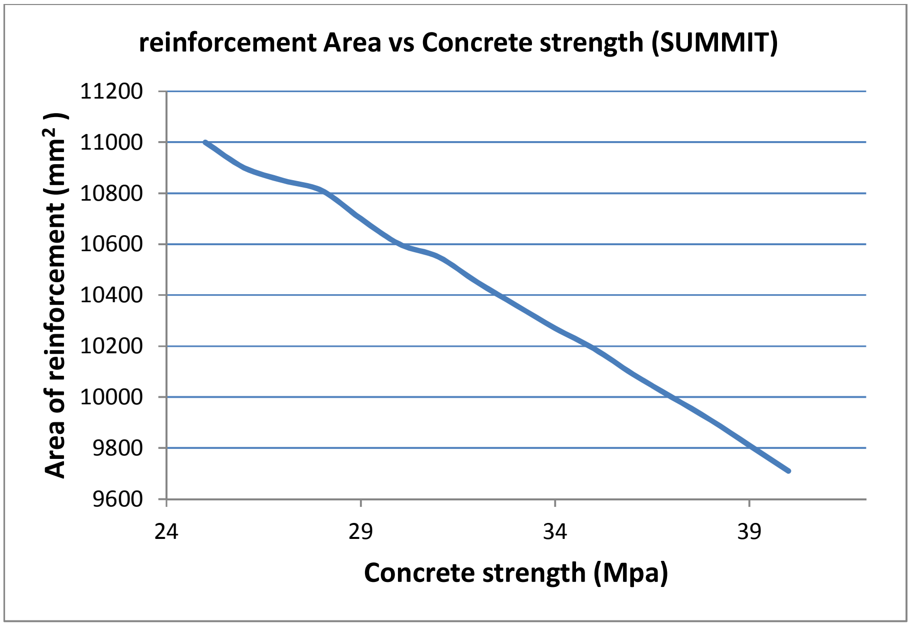


Fig 3.8 Reinforcement area Vs Concrete strengh (Summit)

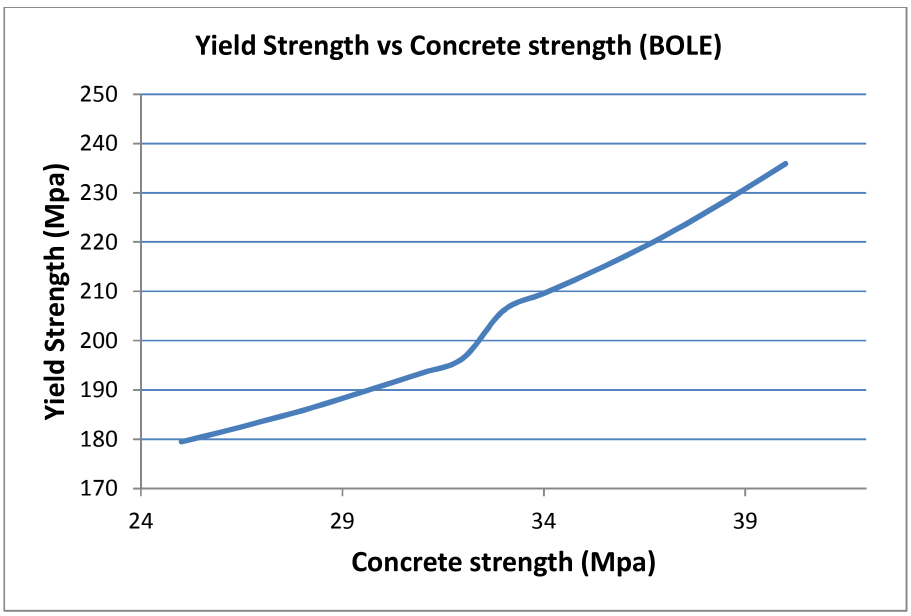


Fig 3.9 Yield strength Vs Concrete strength (Bole)

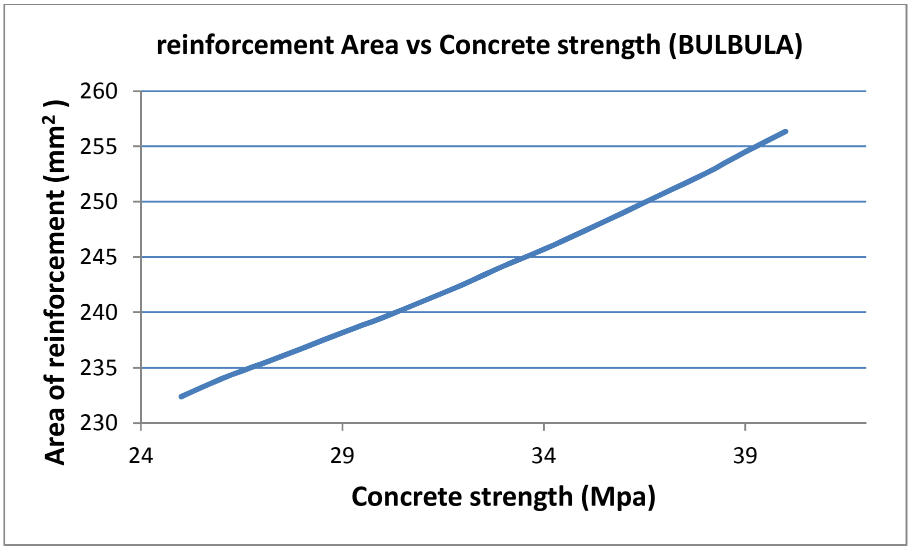


Fig 3.10 Yield strength Vs Concrete strength (Bulbula)

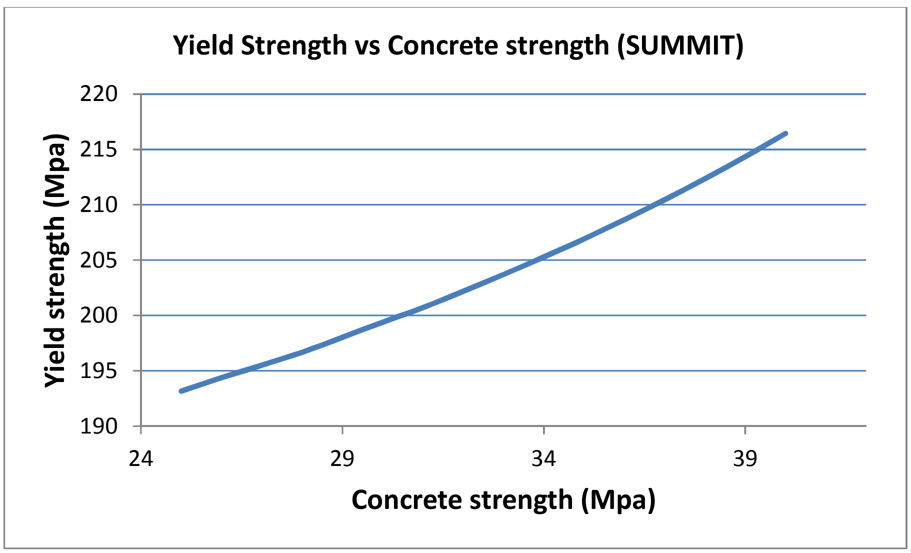


Fig 3.11 Yield strength Vs Concrete strength (Summit)

4. DETERMINATION OF CAPACITY FROM TEST

The chart below show steps followed in the numerical method during determination of capacity.

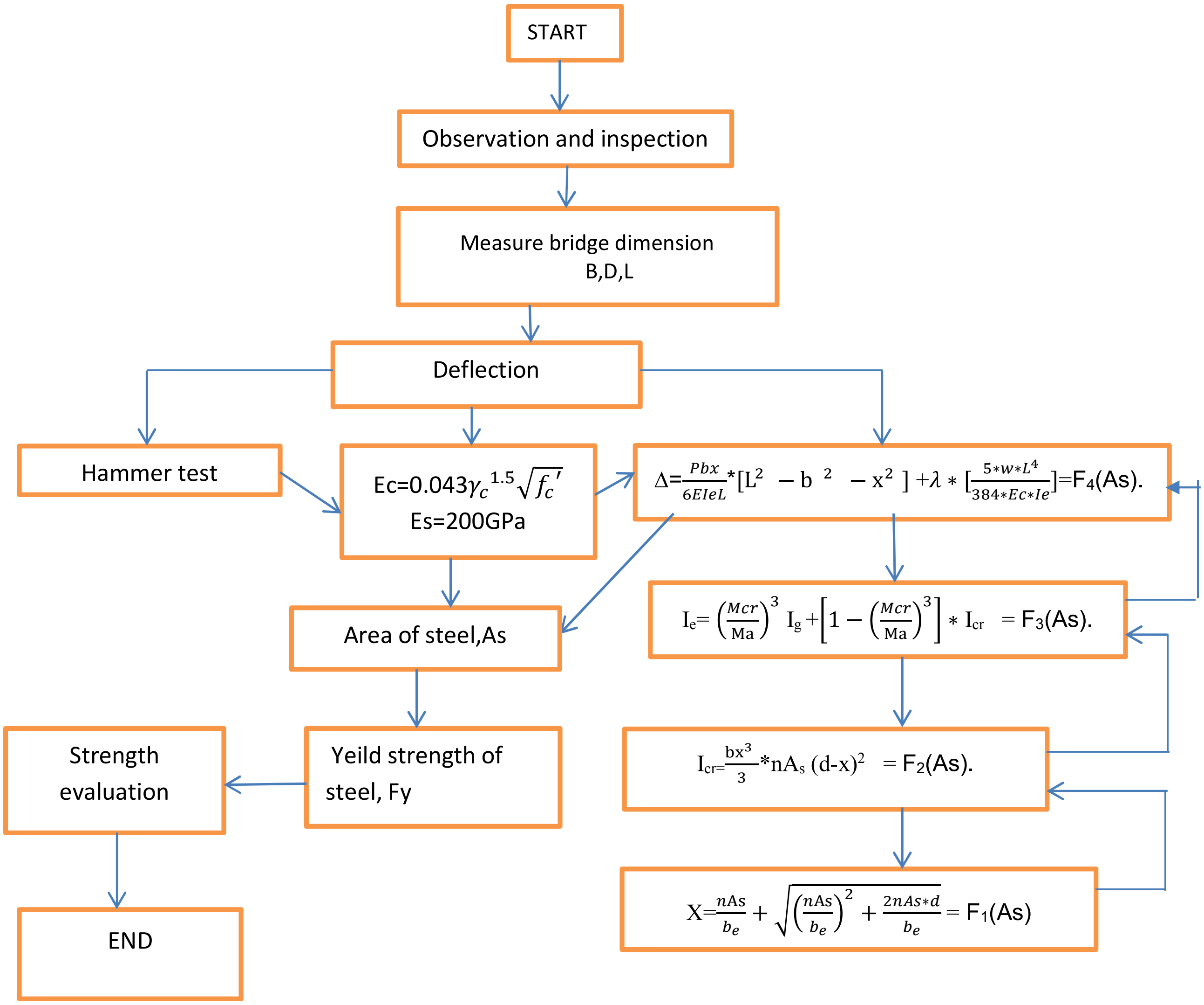


Fig 4.1 numerical method concept

Determination of capacity from test is done of excel spreadsheet. The task is done by calculating deflection as a function of area of steel.

a) neutal axis i.e

$$X=\frac{nAs}{b_e}+\sqrt{\left(\frac{nAs}{b_e}\right)^2+\frac{2nAs*d}{b_e}} \quad .....(4.1)$$

Can be written as $X=F_1(As)$.

b) cracked moment of inertia,

$I_{cr}=\frac{bx^3}{3}*nA_s\,(d-x)^2$ .....(4.2)

can be written as $I_{cr}=F_2(As)$

c) Effective moment of Inertia is calculated from deflection data through the following

formula $I_e=\left(\frac{Mcr}{Ma}\right)^3 I_g+\left[1-\left(\frac{Mcr}{Ma}\right)^3\right]*I_{cr}$ ....(4.3)

can be written as $I_e=F_3(As)$

d) Deflection

$\Delta=\frac{Pbx}{6EIeL}[L^2-b^2-x^2]+\lambda*\frac{5*w*L^4}{384*Ec*Ie}$.... (4.4)

Can be written as $\Delta=F_4(As)$

e) Compressive strength of concrete is obtained from hammer test

f) Rating factor is calculated according to E.R.A manual. i.e

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

A typical excel spreadsheets showing calculation for the three bridges, i.e BULBULA, BOLE and SUMMIT for concrete strength (fc'=30Mpa) are shown in Appendix A.

5. CONCLUSION AND RECOMMENDATION

The result of determining the capacity of T-girder bridge from deflection does not give perfect result but an answer with approximation. Calculated values of area of reinforcement and Yield strength of steel were found to be lesser than their values in original documents. That makes the numerical method on safer side with regard to determining what amount of load should pass over the bridge.

During test, if the bridge undergoes small deflection than value obtained in this thesis, it indicates either the concrete strength is stronger or large amount of reinforcement. The yield strength depends on amount of reinforcement. If the bridge undergoes large deflection during test (within the range of allowable deflection), it indicates either less concrete strength or less amount of reinforcement.

The result of determining the capacity of T-girder bridge from deflection does not give perfect result but gives answer with approximation. For better understanding of the bridge behavior, if dial gauges with best sensitivity could be placed beneath the bridge, good result will be obtained, though it is costy for unreachable bridges.

6. REFERENCE

[1] E.R.A Bridge Design manual -2002

[2] Lecture note, Design of spillway bridge, Ato fekadu melese-2010

[3] Evaluation of Bridge Performance and Rating through

[4] Nondestructive Load Testing

[5] Load Testing for Assessment and Rating of Highway Bridges

[6] AASHTO LRFD Bridge Design Specifications, third edition, 2004

[7] Unified theory of Concrete Structures, Thomas T.C. Hsu, Y.L.Mo

[8] Reinforced concrete, James k.wight,James G. MacGREGOR

[9] use of the schmidt rebound hammer for non-destructive concrete structure testing in field,

[10] Review of GPR Rebar Detection,Xian-Qi He, Zi-Qiang Zhu, Qun-Yi Liu and Guang-Yin Lu,2009

[11] Evaluation of Bridge Performance and Rating through Non-destructive Load Testing

[12] The Capacity Development Project on Bridge Management, JICA Ethiopia,2010

[13] Nondestructive Methods for Condition Evaluation of Prestressing Steel Strands in Concrete Bridges,1999

[14] Guidebook on non-destructive testing of concrete structures,2002

[15] Load Testing for Assessment and Rating of Highway Bridges, Research Project No. 655,2006

[16] Bridge Management in Ethiopia,E.R.A

[17] Lecture note, Reinforced concrete design, Dr-Esayas G/Yohannes-2014

APPENDIX A

BULBULA BRIDGE

1) Bridge parameters

span		20		c	1.74
total depth		1.6		s	3
slab		0.22			
T-section	w		0.47		
Diaphragm	w		0.3		
	h		1.13		

2) Load effects

a) dead load

64mm asphalt we	0.064*22KN/m3	1.408		
rc slab	0.2*24KN/m3	5.28	6.688	
Overhang slab	(0.25+0.45)/2*24KI	12.06	12.06	
Curb(200mm abo	0.2*24	4.8		
Posts and railing	[(0.85*0.3*24+0.08	6.2136		

Posts LEFT	5.04			
			span/2	10
web	15.2706384	KN/m	x	10
Diaphram load				
exrerior girder	10.09649124			
interior girder	20.19298248			

- girder reaction

Rb	18	33.2706384

- shear and moment of bridge

	interior		
M(x)	1663.53192		
V(x)	146.1602095		
span	20000	bw	470
ts	220		

b) Live load effect

	Effective flange width				
Bi	1/4*Leff	5000			
	12ts+bw	3110			
	S	3000			
Be-bi/2	1/8*Leff	2500			
	6ts+bw/2	1555			
	overhang	1735			
taking minimum	3055	be	span	20	
			X	10	

truck(test)	
S1	1.3
S2	3.5
load1	63
load2	90
load3	90
M(x)	1046.25

- ratio,neutral axis, critical moment of inertia, Area of reinforcement

	interior		D	1600	
M(ll+dl)	2709.78192		d	1534	
			As	M/fsjd	
n	7.476635514				
X	296.1597086				
Icr	1.92296E+11		As(prov)	14476.45895	int
h	1380				
s	2.7				
slab	220				
T-section	w	470			
diaphragm	w	300			
	h	1000			
be	3055				
be	3055				
ts	220				
Ybar	1097.117438				
Igross	3.16889E+11	36580000000	0.31689		
Fc'	25	Mpa			
Fr	3.15				
Mcrext=fr*Ig/Yt	909838301.6	909.8383016			
Mcrint=fr*Ig/Yt	909838301.6	909.8383016			
Ma(int)	1663.53192				
E	2.48E+07				
Ie(int)	2.1268E+11		Eie(int)	5.26766E+18	5267657

3) Deflection test and Iteration

Δ(test)+Δcreep		0.047	
L	20	20	20
b	10	8.7	13.5
x	10	11.3	6.5
E	2.53E+07		
p	90	90	63
Eie	5.38E+18		
I_e	2.04E-01		
Mcr	909.8383016		
Ma	2709.78192		
I_g	3.17E-01		
I_{cr}	2.00E-01		
I_{cr}	0.200972834		
n	7.91E+00		
As	14400		
be	3055		
d	1534		
x	303.0089279		

4) Determination of Stress in steel

Mtot	5013.60774		
Fyd	2.33E+08	232.5722396	Mpa

5) Determination of Rating factor

Bulbula+++(asphalt=370Kn/m)+++(other=1294)

d(rev)	1475	
As	14400	
Fyd	232.5722396	
Mu	4432439821	4432.439821
R.F	1.420694588	

BOLE BRIDGE

1) Bridge parameters

span		18	c	1.625
total depth		1.6	s	2.4
slab		0.3		
T-section	w	0.47		
diaphragm	w	0.3		
	h	1.05		

2) Load effects

a) dead load

64mm asphalt	0.064*22KN/m3		1.408		
rc slab	0.2*24KN/m3		7.2	8.608	
Overhang slab	0.2*24KN/m3		7.2	12	
Curb(200mm	0.2*24		4.8		
Posts and raili	[(0.85*0.3*24+0.08		6.2136		
Posts LEFT		5.04			
				span/2	9
web		14.385384 KN/m		x	9
Diaphram load					
exrerior girder		7.1567874			
interior girder		14.3135748			

• girder reaction

Rb	15
(include web)	29.385384

• shear and moment of bridge

	interior
M(x)	1190.108052
V(x)	117.9206532

span	18000	bw	470
ts	300		

	Effective flange width	
Bi	1/4*Leff	4500
	12ts+bw	4070
	S	2400
Be-bi/2	1/8*Leff	2250
	6ts+bw/2	2035
	overhang	1625

taking minimu	2825	be	span	18
			X	9

truck test)

S1	1.3
S2	3.5
load1	56
load2	79
load3	77

M(x)	805.95

- ratio,neutral axis, critical moment of inertia, Area of reinforcement

	interior						
M(ll+dl)	1996.058052			D		1600	
				d		1534	
n	8			As	M/fsjd		
X	316.0119144					0	int
Icr	29717178158			As(prov)		14476.4589	
h	1300						
s	2.7						
slab	300						
T-section w		470					
diaphragm w		300					
h	1300						
be	2825						
be	2825						
ts	300						
Ybar	1114.861159						
Igross	3.1963E+11	36580000000	0.31963				
Fc'	34 Mpa						
Fr	3.673499694						
Mcrint=fr*Ig/Y	1053188595	1053.188595					
	Ma						
Ma(int)		1996.058052	837.87				
E	2.26E+07						
Ie(int)	72303064436		Eie(int)	1.63477E+18		1634771.36	

3) Deflection test and Iteration

Δ	0.027			
L	18	18	18	
b	9	7.7	12.5	
x	9	10.3	5.5	
E	2.95E+07			
p	79	77	56	
Eie	2.13E+18			
I_e	1.78E-01			
Mcr	1053.188595		1.25	`
Ma	1996.058052			
I_g	3.20E-01			
I_{cr}	1.53E-01			
I_{cr}	0.15346664			
n	6.78E+00			
As	12450			
be	2825			
d	1534			
x	274.4428928	59.50448076		

4) Determination of stress in steel

determine Fs

Mtot	3867.50752	
Fs	2.03E+08	202.929605

5) Determination of Rating factor

Bole===(asphalt=150kn/m)++++(other=1040)

As	12450	
Fs	202.929605	
Mu	3.69E+09	3691.264801
R.F	1.206058581	

SUMMIT BRIDGE

1) Bridge parameters

span		15			c	1.5
total depth		1.3			s	2.4
slab		0.22				
T-section	w		0.38			
	h		1.08			
diaphragm	w		0.3			
	h		0.78			

2) Load effects

a) dead load

64mm asphalt we	0.064*22KN/m3		1.408			
rc slab	0.2*24KN/m3		5.28	6.688		
Overhang slab	(0.25+0.45)/2*24KI		12.06	12.06		
Curb(200mm abo	0.2*24		4.8			
Posts and railing	[(0.85*0.3*24+0.08		6.2136			
Posts LEFT		5.04				
	dead load				span/2	7.5
web		9.6624576 KN/m			x	7.5
Diaphram load						
exrerior girder		5.56438896				
interior girder		11.12877792				

- girder reaction

Rb	10	19.6624576

- shear and moment of bridge

	interior
M(x)	553.00662
V(x)	62.60543808

span	15000	bw	380
ts	220		

b) Live load effect

	Effective flange width	
Bi	1/4*Leff	3750
	12ts+bw	3020
	S	2400
Be-bi/2	1/8*Leff	1875
	6ts+bw/2	1510
	overhang	1500
taking minimum	2700 be	

	truck(test)		
		span	15
S1	1.3	X	7.5
S2	3.5		
load1	56		
load2	79		
load3	77		
M(x)	646.95		

- ratio,neutral axis, critical moment of inertia, Area of reinforcement

	interior			D	1400
M(ll+dl)	1199.95662			d	1334
				As	M/fsjd
n	7				4283.418 int
X	251.4050883			As(prov)	11259.47
Icr	1.06675E+11				

total depth	1300		
slab	220		
h	1080		
s	2.7		
T-section	w	380	
diaphragm	w	300	
h	1000		
be	2700		
be	2700		
ts	220		
Ybar	924.4086022		
Igross	1.44832E+11	36580000000	0.14483
Fc'	30 Mpa		
Fr	3.450652112		
Mcrext=fr*Ig/Yt	540630176.7	540.6301767	
Mcrint=fr*Ig/Yt	540630176.7	540.6301767	
Ma(int)	1199.95662		
E	26750000		

Ie(int)	1.10164E+11	Eie(int)	2.94689E+18	2946891

3) Deflection test and Iteration

Δ(test)+Δcreep	0.017		
L	15	15	15
b	7.5	6.2	11
x	7.5	8.8	4
E	2.77E+07		
p	78	76	55
Eie	3.05E+18		
I_e	1.08E-01		
Mcr	540.6301767		1.25
Ma	1199.95662		
I_g	1.45E-01		
I_{cr}	1.04E-01		
I_{cr}	0.104019737		
n	7.22E+00		
As	10600		
be	2700		
d	1334		
x	248.1493394	50.66244948	

4) Determination of stress in steel

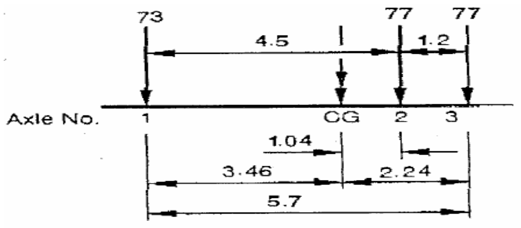
Mtot	2644.692667	
Fs	1.99E+08	199.3964635

5) Determination of Rating factor

Summit+++(asphalt=112Kn/m)+++(other=525Kn/m)

d(rev)	1275	
As	10600	
Fs	199.3964635	
Mu	2432598671	2432.598671
R.f	1.125549688	

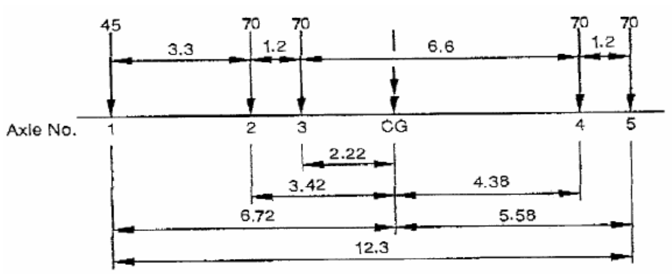
MOMENT CALCULATIO FOR LEGAL TRUCKS USING INFLUENCE LINE



span		15
77	3.5	7.5
77	2.94	6.3
73	1.4	3
M		598.08

span		18
77	4.5	9
77	3.9	7.8
73	2.25	4.5
M		811.05

span		20
77	5	10
77	4.4	8.8
73	2.75	5.5
M		924.55



span 15
spac at end 1.35

Trial 1			M
70	0.823846	1.35	57.669231
70	2.837692	4.65	198.63846
70	3.57	5.85	249.9
70	3.57	9.15	249.9
70	0.994918	2.55	69.644262
45	0.526721	1.35	23.702459
	M		849.45441

span 15
spac at end 1.35

Trial 2			M
70	1.224745	2.35	85.73212
70	2.944599	5.65	206.1219
70	3.57	6.85	249.9
70	3.57	8.15	249.9
70	0.678957	1.55	47.52699
45	0.153313	0.35	6.89908
	M		846.0801

span 18
spac at end 2.85

Trial 1			M
70	1.68625	2.85	118.0375
70	3.63875	6.15	254.7125
70	4.34875	7.35	304.4125
70	3.57	10.65	249.9
70	1.357606	4.05	95.032394
45	0.955352	2.85	42.990845
	M		1065.0857

span 18
spac at end 2.85

Trial 2			M
70	2.303883	5.07	161.2718
70	3.803452	8.37	266.2417
70	4.34875	9.57	304.4125
70	3.57	8.43	249.9
70	0.774982	1.83	54.24875
45	0.266797	0.63	12.00587
	M		1048.081

span 20
spac at end 3.85

Trial 1			M
70	2.242625	3.85	156.98375
70	4.164875	7.15	291.54125
70	4.863875	8.35	340.47125
70	3.57	11.65	249.9
70	1.547511	5.05	108.32575
45	1.179785	3.85	53.090343

span 20
spac at end 3.85

Trial 2			M
70	2.793162	6.07	195.5213
70	4.311685	9.37	301.8179
70	4.863875	10.57	340.4713
70	3.57	9.43	249.9
70	1.071379	2.83	74.9965
45	0.617084	1.63	27.76877

M	1200.3123	M	1190.476

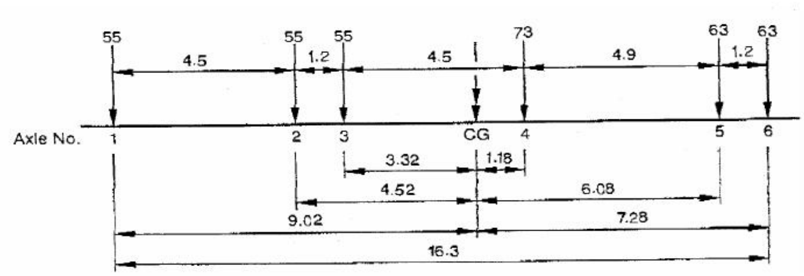


Figure 14-4 Truck Type 3-3 Unit Weight = 364 kN

span	18		
spac at end	0.85		
	Trial 1		M
63	0.466283	0.85	29.37584
63	1.124565	2.05	70.84761
73	3.81255	6.95	278.3162
	4.459861	8.13	
	4.459861	9.87	
55	2.959685	6.55	162.7827
55	2.417453	5.35	132.9599
55	0.384081	0.85	21.12447
			316.867

span	20		
spac at end	1.85		
	Trial 1		M
63	1.005475	1.85	63.34493
63	1.657675	3.05	104.4335
73	4.320825	7.95	315.4202
	4.962155	9.13	
	4.962155	10.87	
55	3.446575	7.55	189.5616
55	2.898775	6.35	159.4326
55	0.844525	1.85	46.44888
			395.4431