Sudan University of Science and Technology

College of Graduate Studies

Assessment and Upgrading of Temporary Bridges in Upper Atbara and Setit

تقييم وترفيع الجسور المؤقتة في أعالي عطبرة وستيت

A Thesis submitted in partial Fulfillment of the Requirements for the Degree of MSc in Civil Engineering (Construction Engineering)

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بسم الله الرحمن الرحيم

صدق الله العظيم

Dedication

- To my parents with gratitude and respect for struggling to let me learn and be educated.
- To every one stand on his feet to teach me while I'm sitting in rest and calm, those are my teachers.
- To gentle Engineers whom supported and inspired me to continue working in bridges domain more than ten years.
- And before all, A lot of thanks to my God whom I depend on always.

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ABSTRACT

This research aims to structural assessment and upgrading for the upper Atbara of 5 spans with 198.12m of total length and Setit bridge of 7 spans with 260m length. Both bridges are temporary baily bridges used during the construction of the dam complex of upper Atbara. The purpose of the upgrading is to allow using the two bridges as two lanes with permanent concrete deck instead of the current one lane steel plate's deck.

The rating at inventory and operation level for the design and legal loading and bridge posting were performed. The rating results shows that the entire baily truss might not adequate to sustain the loads induced by the concrete deck and two lane live load, so it's probably feasible to be replaced by an alternative superstructure and in the same time the capacity of the substructure should investigated whether it could withstand the loads from the new superstructure or it might need some sort of strengthening.

Two types of alternative superstructures were done using the commercially available structural design software SAP 2000. A preliminary design for prestressed concrete T-girder was first option and steel plate girders was second one. The AASHTO-LRFD method was used. Main design steps with illustrative internal forces were displayed to check the capacity of the girders. After verifying the suitability of the new selected girders, the substructure's load carrying capacity was investigated, first the bent caps resistance capacity calculated and compared against the ultimate loads produced by the new superstructure, but both upper Atbara and Setit bridges bent caps require to be strengthened between the two shaft columns supports by adding more reinforcement to increase the resistance to the positive moments induced by the girders.

The shaft column demonstrates that it has enough capacity withstand the axial loads and also the pile shaft foundation.

This study to consider keeping the traffic moving through a temporary pass way and also the simultaneity of removing and strengthening of the truss and bent cap with the manufacturing or casting process of girders in order to shorten the time of the construction and traffic interruption.

مستخلص البحث

يهدف هذا البحث لعمل تقييم إنشائي ومن ثم الترفيع لكبري أعالي عطبرة و الذي يتكون من خمسة مجازات بطول كلي يبلغ 198.12 متروكبري سيتيت المكون من سبعة مجازات بطول كلي يبلغ 260 مترا، و كلا الجسران مكونان من منشأ علوي من نوع جملونات الفولاذ (Baily Truss) تم استخدامهما كجسور مؤقتة للمساعدة في تشييد مجمع سدي أعالي عطبرة و سيتيت. و الهدف من الترفيع هو تحويلهما الي كباري دائمة من حارتين للمرور و ذات بلاطة خرسانية بدلا عن وضعهما الحالي حيث تستخدم حارة واحدة للمرور علي أرضية من ألواح الحديد.

تم القيام بعملية تقييم إنشائي لأعضاء الجمالون في حالة مستوي الحمل التصميمي و التشغيلي و عند حالات تقييد المرور لحمل أقصي معين. و قد أظهرت النتائج احتمالية عدم قدرة نظام الجملون هذا علي حمل الأحمال التي ستنتج من البلاطة الخرسانية و كذلك الأحمال الحية التي ستنتج من المرور في حارتين مما يطرح إمكانية التفكير في إقتراح بدائل لحل محل الجمالون الفولاذي الموجود. وفي نفس الوقت سيتم بحث قدرة الأعضاء التحتية للمنشأة لمعرفة ما اذا كانت تستطيع تحمل القوي الناتجة من العناصر العلوية الجديدة أم تحتاج الي تقوية.

تم عمل مقترح تشييد أحد من منشأين علويين باستخدام النسخة التجارية لبرنامج التصميم SAP 2000. تصميم أولي الأول يتكون من أبيام علي شكل T من الخرسانة المسبقة الصب و الثاني يتكون من أبيام من الفولاذ و قد إستخدام الكود الأمريكي AASHTO LRFD. تم عمل عرض توضيحي لخطوات التصميم الرئيسية و للقوي الداخلية لإختبار سعة الأبيام التحميلية.

بعد التأكد من كفاءة نوع العارضة المختارة تم مراجعة سعة تحمل المنشآت السفلية. أولا تمت مراجعة السعة التحميلية للابيام العرضية الحاملة للعارضة الرئيسية و مقارنتها مع الأحمال القصوي الناتجة من المنشأ العلوي الجديد إتضح أنه توجد ضرورة لعمل تقوية لهذه الأبيام العرضية لكل من كبريي أعالي عطبرة و سيتيت في المسافة بين العمودين الساندين بزيادة حديد التسليح لمقاومة العزوم الموجبة الناتجة من العارضات الرئيسية.

بالنسبة للأعمدة الحاملة للمنشأ العلوي فقد أظهرت قدرة كبيرة علي تحمل القوي المحورية و كذلك القاعدة المكونة من الخوازيق .

يوصي البحث علي ضرورة دراسة كيفية إستبدال هذا المنشأ العلوي المكون من الفولاذ و مراعاة عمل معابر مؤقتة أثناء التشييد و أيضا لكسب الوقت يجب بدء عمليات الصب العارضات المسبقة الصب أو التصنيع بالتزامن مع عملية ازالة الجمالون الفولاذي و تقوية العارضات العرضية.

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List of Symbols and Abbreviations

A = cross-sectional area
Ag = Gross area of the basic beam (mm2)
A_{PS} = area of prestressing steel
AP = Area of tip
As = Surface area of pile
As = area of nonprestressed tension reinforcement
As` = area of compression reinforcement
Av = the shear area
b = width of the effective compression block of the member
beff= Effective width of the flange
bw = web width
BR = vehicular braking force
β1 = stress block factor
C = Member Capacity
C = The distance from the neutral axis to the compression face of the member
CE = vehicular centrifugal force
CR = creep
CT = vehicular collision force
d = distance between two axes
DC = dead load of structural components
DFM= Distribution factor for moment
DFV= Distribution factor for shear
dp = distance from bottom of beam to location of P/S steel force
d_S = distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement
D_W = dead load of wearing surfaces and utilities
E = modulus of elasticity
EC= Concrete slab modulus of elasticity
Eg = Elastic modulus of girder
e_g= Distance between the centroid of girder and slab
Es = Elastic modulus of deck slab
ES= Steel girder modulus of elasticity
EC= Concrete slab modulus of elasticity
E_Q = earthquake load
e_0 = Distance between the neutral axis of the noncomposite girder and the center of gravity of the prestressing steel
fb = maximum normal stress due to bending
f_{DL}= Stress due to permanent dead load
f_{DW} = Stress due to superimposed dead load
f_{LL} = Stress due to live load
f_{PS} = average stress in prestressing steel
f c = specified strength of concrete at 28 days, unless another age is specified
fy = yield strength

h = overall depth
HSE = Health and safety environment
hf = compression flange depth of an I or T member
i = radius of gyration about the relevant axis
I = Second moment of inertia of beam
IM = vehicular dynamic load allowance
Ixx = second moment of inertia about the major axis
Iyy = second moment of inertia about the major axis
Kg = Factor for axle load distribution
KL = Effective length of the truss member
KL/r = The slenderness ratio
ks = buckling factor corresponding to stress ratio
L= Span Length
L = actual unbraced length of the truss member
L_{eff} = Effective length of the truss member
LL = vehicular live load
LS = live load surcharge
M = bending moment due to the applied loads
M_{DL} = Moments due to read loads
M_{DW} = Moment due to superimposed dead loads
M_{LL} = Moment due to live loads
Mn = Nominal resistance moment
M_R = Factored resistance moment
Mu= Maximum bending moment due to the applied loads
P = Permanent loads other than dead loads
P= Truck axle loading
P_o = nominal axial resistance of a section at zero eccentricity
Pn = nominal axial resistance, with or without flexure
P_R = factored axial resistance
Pt = Initial prestressing force
P_L = pedestrian live load
Qs= Pile shaft resistance
q_p = Unit tip resistance of pile
q_s = Unit shaft resistance of pile
r = radius of gyration of the truss member cross section
RF = Rating factor
S = average girders spacing
S_{bot} = Girder bottom elastic section modulus
S_{top} = Girder bottom elastic section modulus
S = elastic section modulus
SH = shrinkage
t = plate thickness
t_f = thickness of flange
ts = thickness of flange
TU = uniform temperature
tw = thickness of web.

V_n = Factored plate shear capacity
V_p = Nominal shear strength
Y_{bot} = ordinate to the bottom of the total area
Y_{top} = ordinate to the top of the total area
$Y_{c.g}$ = ordinate to the centroid of the total area
γDC = LRFD load factor for structural components and attachments
γDW = LRFD load factor for wearing surfaces and utilities
γp = LRFD load factor for permanent loads other than dead loads
γLL = Evaluation live load factor
Φc = Condition factor
Φs = System factor
φ_n = LRFD resistance factor

CHAPTER I

INTRODUCTION

CHAPTER ONE

INTRODUCTION

1.1 Statement of the Problem

The dam complex of upper Atbara project includes two bridges on Upper Atbara river and Settit river, which are tributaries of the Nile River in the republic of the Sudan. The two bridges were constructed in 2010 as temporary to enable and ease the transportation of materials, labors, plants... etc. during the construction of the dam. The bridges are located in Gedarif state, at Wad Alhilaio locality where Setit and Atbara rivers runs toward north hindering the community at the west bank from getting their needs from Gedarif and Showak cities, that they have to travel many hours to Khasm Elgirba, but fortunately when the bridges were constructed, all the vicinity community around the dam get their navigation easy.

The bridges have a high economic value that most of the demography structure is a farming and pastoral society, so although the bridges were constructed as temporary, it was used extensively by locals to transport and cross through their crops and Cattles.

In order to sustain the community development and benefits, the Dams Implementation Unit decided to upgrade the two bridges to be permanently used with more consideration to the bridge management and maintenance issues, when the project handed over and all left.

The superstructure of the bridges is baily type with orthotropic steel plate deck bolted on a transverse floor beams, the steel plates bolts and stiffeners are always require inspection and maintenance on a weekly basis and this was done by the contractor during the construction as well as there is a deflection in many points along the bridge and more over the design reference and method was not verified due to the lack of clarity and references.

Although the structures have performed satisfactorily over the past years but many problems have raised due to fatigue, excessive truck loads, corrosion or extreme environmental conditions.

These factors have many causes. They may affect appearance only, or they may indicate significant structural distress or a lack of durability.

Repair and rehabilitation work for steel structures can broadly be classified into two categories:

a) Repair in which damage due to deterioration and corrosion is corrected to restore the original structural shape, and
b) Repair which is necessary to strengthen the structural capacity of members whose load carrying capacity is either inadequate or whose strength has been severely impaired due to sustained damage.

While the former is essential a cosmetic restoration aimed at compliance with serviceability and structural integrity criteria, the second category deals primarily with the enhancement of strength and therefore complies predominantly with strength criteria , whereas the upgrading might be the best option when comparing the costs of maintenance and rehabilitation.

1.2 Research Objectives

Assessment and upgrading of the bridge depends upon knowing the previous design standards used, design loads and material properties. Comprehensive investigation was carried out included reviewing technical information, existing design calculations, reinforcement details, and laboratory tests results for concrete and pile loads carrying capacity. Based on this information, the bridges were analyzed to determine the loads on the members, redesigned to different alternatives, and evaluated by load rating. The objectives of this study are:

1. To evaluate and assess the two bridges by analysis and rating procedures,
2. To upgrade it by strengthen or changing the entire superstructure to be permanent.

These procedures include visual inspections and bridge analysis and ratings using structural analysis and design software. The outcome should be introducing the best alternative super structure as well as the bridge deck.

1.3 Limitations of the Study

This research presents a case study of assessment and evaluation of setit and Atbara bridges. The assessment and evaluation rely on the available bridges documents information that may help to fulfill the upgrading requirements. This study is limited to the structural evaluation and upgrading depending on the available information because there are technical and financial difficulties of performing full inspection and field tests, which are required when the recommendation of bridge maintenance is one of the objectives.

1.4 Research Methodology

In this research, the way to achieve the objectives is by analytical work. Basic principles of how the structures behave were studied. In order to achieve a realistic assessment of the bridge condition, a good understanding of the bridge's behavior is necessary. To achieve the main goal of the research, several steps should be taken, including:

1. Modeling the existing bridge according to AASHTO LRFD traffic loads and interpretation of the behavior and identify possible problems:
   - Recalculation of the design dead and live loads and load combinations for the existing bridge superstructure and substructure because the available design is generally an empirical design and has no clear reference standard.
   - A member forces, distribution of sectional forces and the member's response during loading and/or failure.
   - Verification of the model in correlation with deflections and stresses.
   - Check members with respect to fatigue in order to get a rough representation of the existing condition.
   - Calculate the self-weight of the existing deck.
   - Estimate load capacity of the bridge members with exiting condition.
2. Analysis of the upgraded bridge model with a new superstructure and investigation of advantages and disadvantages associated with the types of superstructure:
   - A member forces, distribution of sectional forces and the member's response during loading and/or failure.
   - Verification of the model in correlation of deflections and stresses.
   - Contribution of the deck type in load carrying capacity.
   - How much of the superstructure weight could affect the piers and foundations carrying capacity and safety factors due to the lighter or heavier deck.
   - Load carrying capacity of the upgraded bridge members.

1.5 Thesis Outline

This thesis is trying to apply the load rating and upgrading principles on the cases of Upper Atbara Bridges, in the following sequences:

In Chapter 3:

- Load rating for the Bridge: by applying the AASHTO LRFD truck loads at inventory and operation level to get the maximum loading effect on the baily truss members and then calculate the member's capacities to apply the load rating equation in which its main concept is to get the ratio by subtracting the load effect from the member capacity.

- Upgrading of the Bridge: by changing the superstructure to the option of prestressed concrete girder or steel girder in order to get the bridge works as two lane permanent bridge so a preliminary design was introduced for each option by checking the suitability of the selected sections to sustain the bridge loads.
- Results of the calculations were presented and discussed in which it was found that the existing Baily truss needs to be replaced by either concrete or steel girder super structure.
- In Chapter 4:

  Conclusions and Recommendations: In the conclusion for the ease and accelerated construction process the steel girder is preferred and plan for traffic control during the construction stage was recommended.

CHAPTER II

BACKGROUND AND LITERATURE REVIEW

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Today, many existing structures are replaced or strengthened because their reliability and functionality cannot be guaranteed based on the structural assessments made. This leads to great environmental stresses and a bad usage of the society's resources. The purpose of this research is to evaluate two bridges designs to replace the deck slab or upgrade them after comparing the alternatives. A baily type design were evaluated based on a set of established evaluation criteria. These criteria included considerations for design live loads, cost, environmental impact, timeline and ability for ethical design. After each of the alternatives was evaluated and compared to a "no change" scenario, the superior alternative was selected and a design of the superstructure and evaluation of substructure will completed based on this alternative.

2.2 Literature and Review

Structural assessments for bridges are normally made using simplified structural models, based on information from drawings. This information is sometimes complemented through material tests from the existing structure and by studying of the original design and construction documentation. Information from field tests and measurements of the real response of the bridge is generally not used and improvements of the structural models through testing or monitoring have rarely been utilized.

By using modern analysis methods for structural assessment, the intrinsic load carrying capacity can be utilized during the entire lifetime of the structure. By structural verification through field tests and measurements, a better knowledge of the structural response and performance will be achieved, resulting in an improved base also for inspection and maintenance. By extending the lifetime of the structures and by optimizing the maintenance, great environmental benefits will be achieved, with less raw-material consumption, reduction of transportation and energy consumption, decreased pollution and less deposit. At the same time substantial costs are avoided, both for the society and for the owner or administrator of the structure.

2.2.1 Bridge Management

After the completion of a bridge, it is managed by a bridge administrator during its lifetime. Three main types of measures taken during the bridge management phase can be distinguished according to (Plos, Report 2008:5) :

- The inspections are planned and are repeated with predicted intervals. Normally, they include visual inspection, but they can also include testing and measurements. In some cases, continuous monitoring using built-in or permanently installed gauges on the bridge is used.
- An assessment is only made when called for. It can be a structural assessment with respect to the safety or the function of the bridge. It can also be an assessment of the condition of the bridge.
- Maintenance and repair: This can either be periodical maintenance or consist of measures called for by an assessment. A structural model of a bridge is made as a part of the design process. However, this model is often simplified and based on a priori assumptions of the bridge. For the management phase, an improved structural model is often required. After the bridge is constructed, there is a possibility to improve the structural model and to update it through testing and measurements. Such an improved structural model is usually made as a part of a structural assessment. It is not needed only to determine the safety or function of the bridge, but also if the bridge needs to be repaired or reconstructed. It can also be used for improved planning and for decisions regarding inspection and maintenance.

2.2.2 Structural Assessment

The reasons to perform a structural assessment of a bridge can be subdivided in four main categories according to (Plos, Report 2008:5):

1. Changed requirements: Requirements for increased traffic loads are the dominating reason for structural assessments in Sweden. Other examples in this category can be changes in codes and regulations, or changed requirements due to a change in use.
2. Planned reconstruction: A reconstruction often involves interventions into the load carrying structure, which requires a structural evaluation of the bridge.

3. Damage: A bridge may become damaged due to extreme events like floods, storms and earthquakes. Scour is the main cause for bridge damage in many parts of the world. Damage can also occur due to events that the bridge was not designed for, such as overloading, traffic or ship impact, fire and explosions.
4. Deterioration. Deterioration can be caused by external environmental loading, e.g. chloride penetration, corrosion, frost, carbonation or fatigue. It can also be caused by reactions inside the material.

A structural assessment is made with respect one or more of the following Aspects:

- Safety: The load carrying capacity is evaluated with respect to the risk for failure or collapse. It is normally expressed as the load carrying capacity for traffic loads, but can also be expressed by a safety index for given design
- Function: An evaluation of the function can be made with respect to e.g. deformations or vibrations.
- Condition: An assessment of the condition of a bridge can be made with respect to e.g. cracking in concrete bridges, or the state and development of the deterioration.

The measures or activities included in a structural assessment vary from case to another and may consist of one or more of the following parts:

- Structural modelling and analyses: To be able to evaluate safety and function, or to be able to do a more close evaluation of the condition, structural analyses and calculations are needed. A structural assessment of the load carrying capacity includes traditionally this part only.
- More accurate inspections: The regularly inspections made may need to be complemented, e.g. for a more careful survey of the extension and cause for damage or deterioration.
- Testing and measurements: To better determine the properties of the bridge, testing and measurements can be conducted. These can include determination of material properties, real geometry, bridge condition, damage extensions, and traffic.

2.2.3 Inspection for Assessment

The assessment of a structure for its load carrying capacity involves not only analysis and calculations but also the inspection of the structure concerned. Such inspection is necessary to verify the form of construction, the dimensions of the structure and the nature and condition of the structural components. Inspection should cover not only the condition of individual components but also the condition of the structure as an entity and especially noting any signs of distress and its cause. Prior to undertaking the inspection of a structure, all existing information pertaining to the structure should be collected including as-built drawings, soils data and past inspection reports. This may be of use in determining what further information should be obtained from the inspection and which items require special attention.

The structure shall be inspected to determine the density and dimensions needed to calculate the nominal loads Q_K. Care shall be taken to obtain an accurate estimate of dead and superimposed dead loading by undertaking a detailed geometric survey of the structure, reference being made to as-built drawings when available. Loads due to excessive fill, previous strengthening operations and installation of services shall be included. Trial holes or boreholes may be required. The live loading depends on the number of traffic lanes that can be accommodated. The clear width of carriageway and position of lane markings shall be recorded. Similarly, the horizontal road alignment, when curved on the structure shall be determined to permit the calculation of centrifugal loads.

The structure shall be inspected to record all the parameters needed to determine the strength of members and elements, including possible deficiencies, eg. cracks, corrosion, settlement, defective materials, damage, etc. The inspection should provide confirmation of the information obtained from documents, particularly (AGENCY, May 2001):

(i) Dimensions of internal sections that may not be related to external features;

(ii) Previous strengthening;

(iii) Reduction in strength due to services laid through or near the structure.

All constituent parts of the superstructure shall be inspected to determine their respective strengths. Members susceptible to fatigue shall be closely examined for cracks. Samples may be required for testing to determine yield stresses of metal members and reinforcement or strengths of concrete, brickwork, stone masonry and mortar.

It is recommended that, for initial assessment, the appropriate values of the material properties should be used. However, in cases where the initial assessment shows inadequacies or there is doubt about the particular material, the material properties should be verified by testing.

For initial assessment the characteristic strength of materials should be taken as specified in the design tables. Testing should normally only be carried out if the initial assessment is considered inadequate or if there is some doubt about the nature and quality of the materials. The strength values obtained from a limited number of tests shall be considered as only an indication of whether the characteristic strength values are applicable to the material present in the structure. For any particular structure the determination of appropriate characteristic strength values that are statistically valid will usually require extensive testing. The strength of materials in a particular structure may be known from records (AGENCY, May 2001).

2.2.4 Bridge Load Rating Required Data

The safe live load carrying capacity of a highway structure is called its load rating. It is usually expressed as a Rating Factor (RF) or in terms of tonnage for a particular vehicle.

Some of the components of good bridge records are described below (THE MANUAL FOR BRIDGE Evaluation, 2008). It is recognized that, in many cases (particularly for older bridges), only a portion of this information may be available. The components of data entered in a bridge record should be dated and include the signature of the individual responsible for the data presented.

- **Construction Plans**

Each bridge record should include one full-size or clear and readable reduced-size set of all drawings used to construct or repair the bridge.

- **Shop and Working Drawings**

Each bridge record should include one set of all shop and working drawings approved for the construction or repair of the bridge.

- **As-Built Drawings**

Each bridge record should include one set of final drawings showing the "as-built" condition of the bridge, complete with signature of the individual responsible for recording the as-built conditions.

- **Specifications**

Each bridge record should contain one complete copy of the technical specifications under which the bridge was built. Where a general technical specification was used, only the special technical provisions need be incorporated in the bridge record. The edition and date of the general technical specification should be noted in the bridge record.

- **Correspondence**

Include all pertinent letters, memoranda, notices of project completion, daily logs during construction, telephone memos, and all other related information directly concerning the bridge in chronological order in the bridge record.

- **Photographs**

Each bridge record should contain at least two photographs, one showing a top view of the roadway across and one a side elevation view of the bridge. Other photos necessary to show major defects or other important features, such as utilities on the bridge, should also be included.

- **Materials and Tests**

Material Test Data, Reports of nondestructive and laboratory tests of materials incorporated in the bridge, during construction or subsequently, should be included in the bridge record.

- **Load Test Data**

Reports on any field load testing of the bridge should be included in the bridge record.

- **Maintenance and Repair History**

Each bridge record should include a chronological record documenting the maintenance and repairs that have occurred since the initial construction of the bridge. Include details such as date, description of project, contractor, cost, contract number, and related data for in-house projects.

- **Coating History**

Each bridge record should document the surface protective coatings used, including surface preparation, application methods, dry-film thickness and types of paint, concrete and timber sealants, and other protective membranes.

- **Accident Records**

Details of accident or damage occurrences, including date, description of accident, member damage and repairs, and investigative reports should be included in the bridge record.

- **Posting**

Each bridge record should include a summary of all posting actions taken for the bridge, including load capacity calculations, date of posting, and description of signing used.

- **Permit Loads**

A record of the most significant special single-trip permits issued for use of the bridge along with supporting documentation and computations should be included in the bridge record.

- **Flood Data**

For those structures over waterways, a chronological history of major flooding events, including high-water marks at the bridge site and scour activity, should be included in the bridge record where available.

- **Traffic Data**

Each bridge record should include the frequency and type of vehicles using the bridge and their historical variations, when available. Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT) are two important parameters in fatigue life and safe load capacity determination that should be routinely monitored for each bridge and each traffic lane on the bridge. Weights of vehicles using the bridge, if available should also be included in the bridge record.

- **Inspection History**

Each bridge record should include a chronological record of the date and type of all inspections performed on the bridge. The original of the report for each inspection should be included in the bridge record. When available, scour, seismic, and fatigue evaluation studies; fracture-critical information; deck evaluations; and corrosion studies should be part of the bridge record.

- **Inspection Requirements**

To assist in planning and conducting the field inspection of the bridge, a list of specialized tools and equipment as well as descriptions of unique bridge details or features requiring non-routine inspection procedures or access should be provided. Special requirements to ensure the safety of the inspection personnel, the public, or both should be noted, including a traffic management plan.

- **Structure Inventory and Appraisal Sheets**

The bridge record should include a chronological record of Inventory and Appraisal Sheets used by the Bridge Owner. A sample Structure Inventory and Appraisal Sheet is shown in Appendix A4.1 of the AAEHTO manual for bridge evaluation, 1st edition, 2008.

- **Inventories and Inspections**

The bridge record should include reports and results of all inventories and bridge inspections, such as construction and repair inspections.

- **Rating Records**

The bridge record should include a complete record of the determinations of the bridge's load-carrying capacity.

2.2.5 Load and Resistance Factor Rating

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

The rating factor is the multiple of the vehicular live-load effect (for example, moment or shear) that the bridge can carry when the limit-state under investigation is satisfied. The weight of the live-load in tons multiplied by the rating factor is the tonnage that the bridge can safely carry.

All superstructure spans, and main or primary components of the span and their connections shall be load rated until the governing component is established. The load and resistance factor rating procedures of Part A provide a methodology for load rating a bridge consistent with the load and resistance factor design philosophy of the AASHTO LRFD bridge design specifications. The specific load ratings are used in identifying the need for load posting or bridge strengthening and in making overweight-vehicle permit decisions.

Bridge ratings are based on information in the bridge file, including the results of a recent field inspection. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or loading noted during the inspection.

This section of the manual for bridge evaluation, 1st edition, 2008 is consistent with the current AASHTO LRFD Bridge Design Specifications. Where this Section of the AAEHTO manual is silent, the current AASHTO LRFD bridge design Specifications shall govern. Where appropriate, reference is made herein to specific articles in the AASHTO LRFD bridge design Specifications .Where the behavior of a member under traffic is not consistent with that predicted by the governing specifications, as evidenced by a lack of visible signs of distress or excessive deformation or cases where there is evidence of distress even though the specification does not predict such distress, deviation from the governing specifications based on the known behavior of the member under traffic may be used and shall be fully documented. Material sampling, instrumentation, and load tests may be helpful in establishing the load capacity for such members.

This manual for bridge evaluation, 1st edition, 2008 provides analytical and empirical methods for evaluating the safe maximum live load capacity of bridges or for assessing their safety under a particular loading condition. Empirical methods are load ratings by load testing. Only the specific analytical method,

Load and resistance factor rating of bridges, is discussed in this Part A of Section 6 of the manual AAEHTO manual for bridge evaluation , 2nd edition, 2010. Other analytical methods are discussed in Part B, and load testing is discussed in Section 8 of the manual.

I. Component-Specific Evaluation

- **Decks**

Stringer-supported concrete deck slabs and metal decks that are carrying normal traffic satisfactorily need not be routinely evaluated for load capacity. The bridge decks should be inspected regularly to verify satisfactory performance. The inspection of metal decks should emphasize identifying the onset of fatigue cracks. Timber decks that exhibit excessive deformations or deflections under normal traffic loads are considered suitable candidates for further evaluation and often control the rating. Capacity of timber plank decks is often controlled by horizontal shear.

- **Substructures**

Members of substructures need not be routinely checked for load capacity. Substructure elements such as pier caps and columns should be checked in situations where the engineer has reason to believe that their capacity may govern the load capacity of the entire bridge.

Where deemed necessary by the engineer, load rating of substructure elements and checking of stability of substructure components, such as abutments, piers, and walls, should be done using the Strength I load combination and load factors of AASHTO LRFD Design Specifications SI Units 2005 Article 3.4.1, including all permanent loads and loads due to braking and centrifugal forces, but neglecting other transient loads such as wind or temperature. The permanent load factors shall be chosen from AASHTO LRFD Design Specifications SI Units 2005- Table 3.4.1-2 so as to produce the maximum factored force effect. Where longitudinal stability is considered inadequate, the structure may be posted for restricted speed.

Careful attention shall be given to substructure elements for evidence of distress or instability that could affect the load-carrying capacity of the bridge. Main elements and components of the substructure whose failure is expected to cause the collapse of the bridge shall be identified for special emphasis during inspection.

II. Loads For Evaluation

- **Dead Loads: DC and DW**

The dead load effects on the structure shall be computed in accordance with the conditions existing at the time of analysis. Dead loads should be based on dimensions shown on the plans and verified with field measurements. Where present, utilities, attachments, and thickness of wearing surface should be field verified at the time of inspection. Minimum unit weights of materials used in computing dead loads should be in accordance with LRFD Design Table 3.5.1-1, in the absence of more precise information.

- **Permanent Loads Other Than Dead Loads**

Secondary effects from post-tensioning shall be considered as permanent loads.

- **Load Factors**

Load factors for permanent loads are as given in Table 6A.4.2.2-1. If the wearing surface thickness is field measured, γ_{DW} may be taken as 1.25.A load factor of 1.0 shall be applied to the secondary effects from post-tensioning, cited in Article 6A.2.2.2 (γp = 1.0).

Table (2.1): Limit States and Load Factors for Load Rating

Table B6A-1—Limit States and Load Factors for Load Rating (6A.4.2.2-1)

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

- **Transient Loads**

  a- Vehicular Live Loads (Gravity Loads): LL

The nominal live loads to be used in the evaluation of bridges are selected based on the purpose and intended use of the evaluation results. Live load models for load rating include:

- Design Load: HL-93 Design Load per LRFD Design Specifications.
- Legal Loads:
  1- AASHTO Legal loads, as specified in Article 6A.4.4.2.1a.
  2- The Notional Rating Load as specified in Article 6A.4.4.2.1b or State legal loads.
- Permit Load: Actual Permit Truck.

  Load factors for vehicular live loads appropriate for use in load rating are as specified in table 3.1 above state legal loads having only minor variations from the AASHTO legal loads should be evaluated using the same procedures and factors specified for AASHTO trucks in this Manual.

State legal loads significantly heavier than the AASHTO legal loads should be load rated using load factors specified for routine permits in this Manual, if the span has sufficient capacity for AASHTO legal loads.

- **Application of Vehicular Live Load**

  In the past, a distance as little as 1ft between wheel load and edge of the roadway was used for rating by some agencies. This deviation from design is considered overly conservative and especially affected the rating of exterior stringers. The design of exterior stringers in many older bridges, especially those designed prior to 1957, may not have included a minimum live load distribution to the outside stringers.

  The number of traffic lanes to be loaded and the transverse placement of wheel lines shall be in conformance with the AASHTO LRFD Bridge Design Specifications and the following:

  - Roadway widths from 18 to 20 ft shall have two traffic lanes, each equal to one half the roadway width.
  - Roadway widths less than 18 ft shall carry one traffic lane only.
  - The center of any wheel load shall not be closer than 2.0 ft from the edge of a traffic lane or face of the curb.
  - The distance between adjacent wheel lines of passing trucks shall not be less than 4.0 ft.
  - The standard gage width, distance between the wheels of a truck shall be taken to be 6.0 ft unless noted otherwise

- **Dynamic Load Allowance**

  The dynamic load allowance for evaluation shall be as specified in Articles 6A.4.3.3, 6A.4.4.3, and 6A.4.5.5 of the manual.

  In the AASHTO Standard Specifications, the dynamic load allowance was termed impact. Part A allows the use of reduced dynamic load allowance for load rating under certain conditions.

- **Pedestrian Live Loads: PL**

  Pedestrian loads on sidewalks need not be considered simultaneously with vehicular loads when load rating a bridge unless the Engineer has reason to expect that significant pedestrian loading will coincide with maximum vehicular loading. Pedestrian loads considered simultaneously with vehicular loads in calculations for load ratings shall be the probable maximum loads anticipated, but in no case should the loading exceed the value specified in LRFD Design Article 3.6.1.6.

- **Wind Loads: WL and WS**

  Wind loads are not normally considered in load rating. However, the effects of wind on special structures such as movable bridges, long-span bridges, and other high-level bridges should be considered in accordance with applicable standards.

- **Temperature Effects: TG and TU**

  Temperature effects need not be considered in calculating load ratings for nonsegmental bridge components that have been provided with well-distributed steel reinforcement to control thermal cracking.

- **Earthquake Effects: EQ**

  Earthquake effects need not be considered in calculating load ratings.

- **Creep and Shrinkage: CR and SH**

  Creep and shrinkage effects do not need to be considered in calculating load ratings where there is well distributed reinforcement to control cracking in nonsegmental, nonprestressed components.

2.2.6 Global modelling approaches

Steel bridges come in a wide range of structural arrangements such as baily trusses, half-through girders, and steel girders composite with RC decks and so on. The method of analysis used must be appropriate to the structural behavior of the bridge in question, and more highly developed models - better representing the likely real behavior of the structure - become appropriate as a multi-stage assessment reaches its final phases. Assessments almost universally commence with a linear elastic analysis, used to evaluate the load-carrying capacity of the structure at ULS. Although hand calculations are used, often FE software is involved early on, when it is typically not known if nonlinear analysis will be required later. However, software which has nonlinear capabilities should be selected at the outset, enabling analysis models to be modified and developed, rather than re-created from scratch, as the assessment progresses from stage to stage. Switching software late in the process often leads to conflicting results from models which are only somewhat similar in their set-up, and these are time-consuming to resolve since they might arise from a range of sources.

Even without recourse to nonlinear analysis techniques, the idealization adopted can significantly affect the peak load effects identified in the structure. Therefore modelling assumptions should be challenged early in the assessment process and the analysis approach modified to give more realistic results. The following subsections consider some analysis approaches which have been adopted for specific projects.

Assumptions concerning boundary conditions can have a profound influence on calculated internal force and on member resistances. The assumption that translations and rotations are either rigidity restrained or free at support locations is as crude as it is commonplace. While the attention of the bridge engineer is quite reasonably drawn to the deck structure, and issues described in the following sections are principally concerned with deck analysis, it is important to devote as much care to the representation of the boundary conditions as to the deck structure.

- **Modelling Levels For Design and Assessment of Bridges**

Normally, structural design and assessment of bridges are made in two steps. Through a linear analysis of a structural model, cross-sectional forces and moments are determined for a large number of load combinations. These are then used to design or analyze cross-sections, structural elements and connections of the bridge in a local analysis.

Often, separate analysis is made in the two main directions and the structure is designed or analyzed separately in these directions.

For more complicated geometries like curved bridges, 3D beam or frame analyses can be used. For slab types of bridges, 3D analyses with shell finite elements are sometimes used. With such a geometrically improved structural analysis the geometrical modelling is of course more correct, but the analysis is still linear.

Analysis on this level, as well as on the standard level, is used both for design and assessment.

Assessment on this level, in particularly with shell FE analysis, has in practice shown to be unfavorable compared to standard level assessment. Linear analysis leads to high stress concentrations, that need to be re-distributed, and different effects that give increased cross-sectional forces and moments occur in 3D analysis. This is mainly a problem for concrete bridges, and in particularly for analysis of slab bridges or bridges where beams and slabs are interacting.

Non-linear finite element (FE) analysis has proven a great potential for improved bridge assessment, with increased load carrying capacity in many cases. On this level of accuracy, the real response of the bridge is traced in the analysis. The non-linear material response is modelled and effects like cracking, yielding and failures are reflected in the analysis. A non-linear FE analysis can be made either using structural or continuum finite elements. Depending on the level of detailing, conventional local analysis are sometimes needed to check failure risks that are not reflected in the nonlinear analysis.

This type of simulations are normally much more demanding and time consuming, and can in practical bridge assessment only be performed for critical load combinations determined through simplified analysis. On the other hand, in addition to a higher capacity, they can give a much deeper insight into the real structural response and hence are very valuable for further decisions regarding further assessment, maintenance or strengthening.

CHAPTER III

THE CASE STUDY

CHAPTER THREE

THE CASE STUDY

3.1 Introduction:

This chapter aims to provide an overview of how load rating of the Upper Atbara and Setit was developed. The load rating was accomplished following the guidelines presented in the AASHTO Manual for Condition Evaluation of Bridges [AASHTO 2008]. Different Load ratings levels were determined at the design level, Legal level and posting. After that designing the superstructure, steel and concrete girders designs were introduced as an options to determine which system would be the best to replace the existing baily superstructure.

3.2 Previous Design

The dam Complex of Upper Atbara Project includes two bridges on Upper Atbara River Bridge is 5×40m. Bridge Total length is 200m， and Settit River Bridge is (30m, 5×40m, 30m）Bridge length is 260m.

Table 3.1: Upper Atbara River Bridge Components.

Portion	Bridge Elements	No.	Remark
Foundation	Pier cast-in-situ Pile	8	ϕ1.6m
	Abutment cast-in-situ Pile	4	ϕ1.4m
Substructure	Main Pier Pillar	8	ϕ1.6m column pier
Superstructure	Baily Truss Bridge Deck	664.2 (t)	

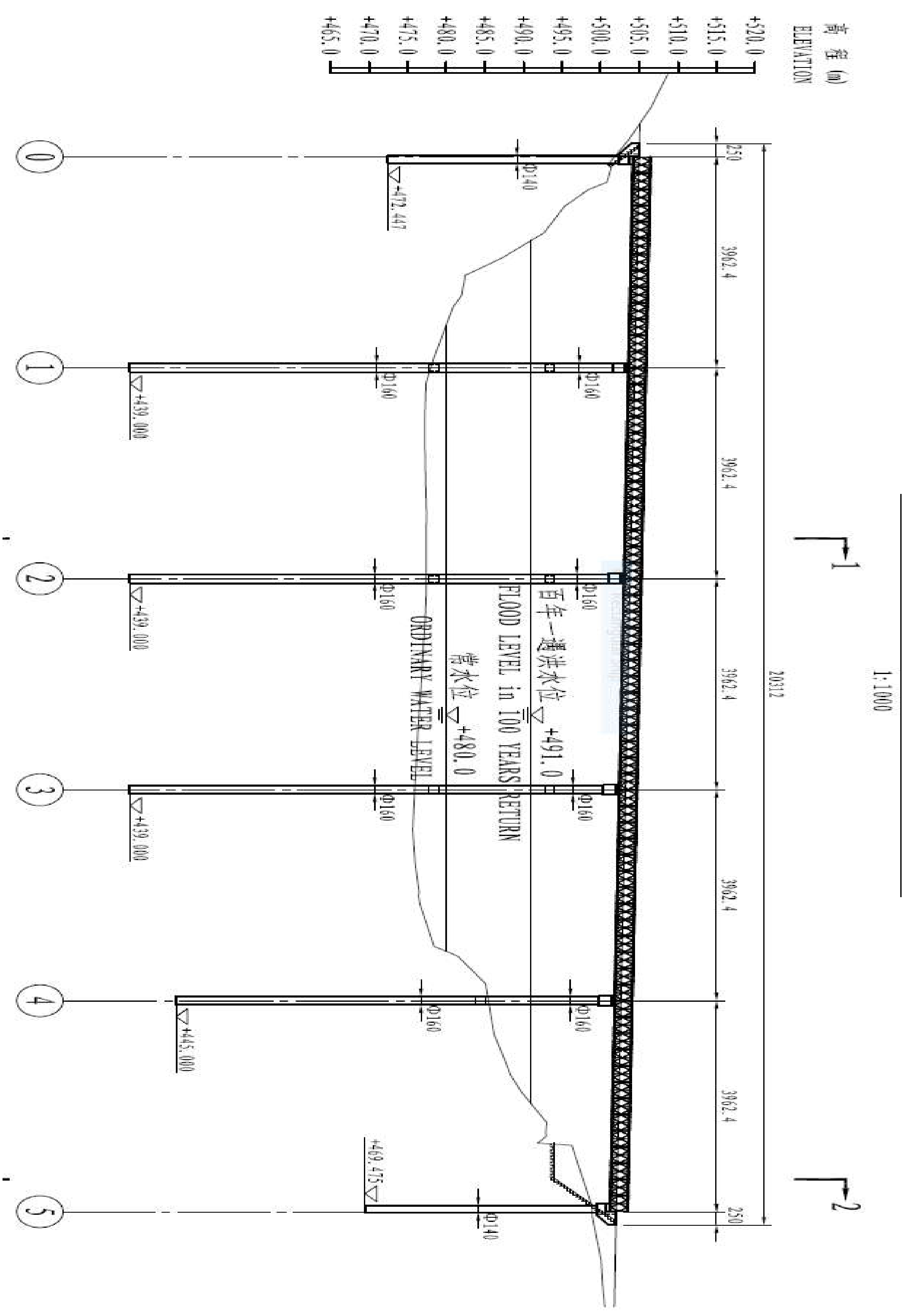


Figure 3.1: Upper Atbara Bridge

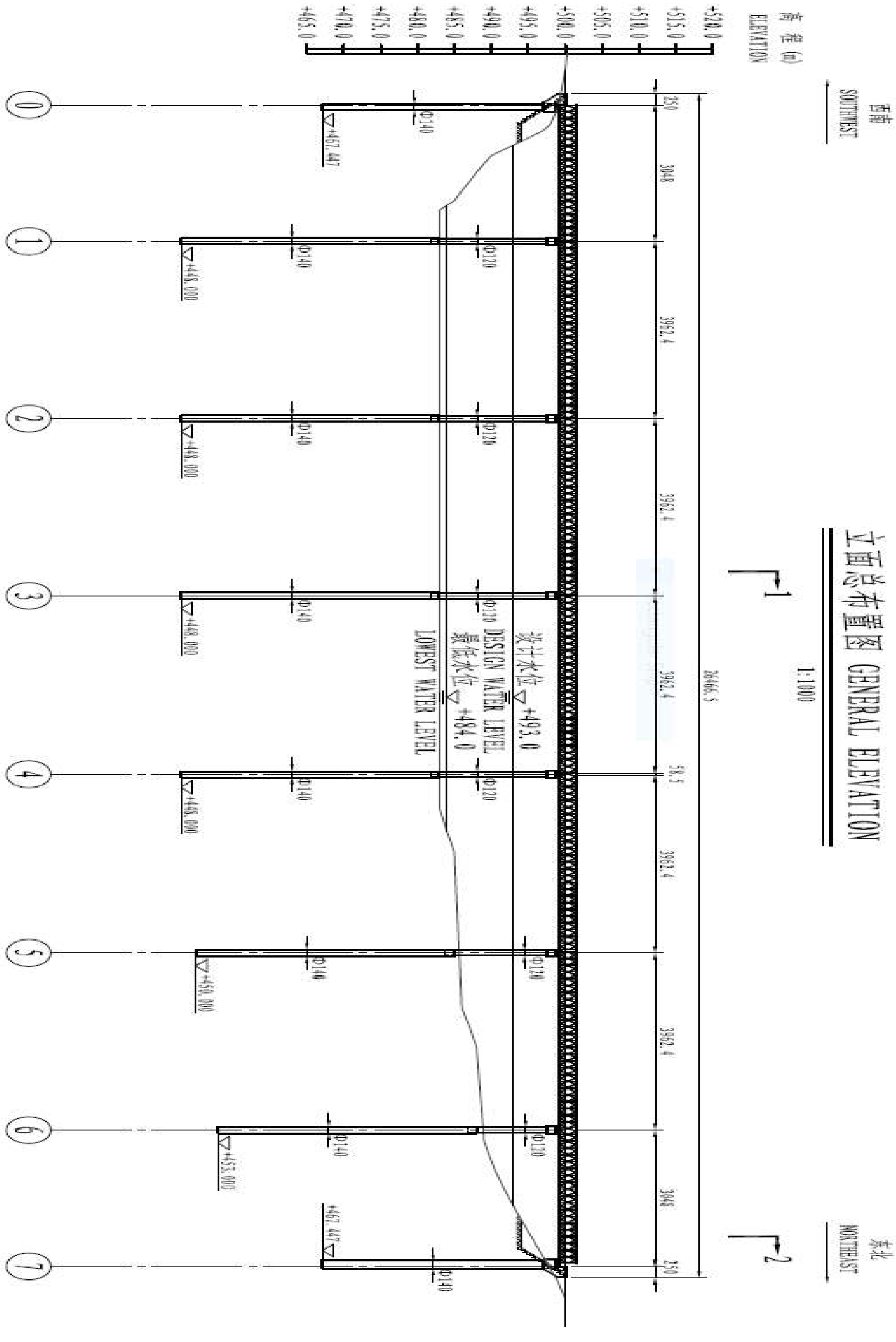


Figure 3.2: Setit Bridge Bridge

Table (3.2): Setit River Bridge Components.

Portion	Bridge Elements	No.	Remark
Foundation	Pier cast-in-situ Pile	12	ф1.4m
	Abutment cast-in-situ Pile	4	ф1.4m
substructure	Main Pier	12	ф1.2m column pier
Superstructure	Baily Truss Bridge Deck	1951(t)	

The Upper Atbara and Setit Bridges were posted by the contractor since it was constructed to be one lane, the control of the traffic flow is organizing by the HSE department at both bridge entrances, and however a regular maintenance for steel deck plates is performed. The concern of continuous maintenance for the deck after handing over of the bridges arises an idea of eating and upgrading the deck to concrete.

3.3 Load Rating Methodology

3.3.1 Load-rating Steps

Three load-rating procedures that are consistent with the load and resistance factor philosophy have been provided in for the load capacity evaluation of in-service bridges:

- **Design load rating** (first level evaluation) Normally the rating starts by the design load rating and when the rating factor is less than 1 (RF<1) , that means the member is not adequate to carry the loads then,
- **Legal load rating** (second level evaluation) the legal rating is required in which the vehicle loads are less than the design load, these vehicles are introduced by AASHTO and they are three types of vehicle , at this level if also the rating factor is less than 1 (RF<1) that means the member is not adequate to carry the loads also then there should be a decision either to do:

a- Posting, and posting means to determine the safe load the bridge can carry and then put the signs and prevent the heavier vehicles from passing through the bridge.

b- Upgrading: Strengthening or Replace the members or the entire superstructure by a new capable one.

- **Permit load rating** (third level evaluation) this only used when requested by the owner for the review of permit applications for the passage of vehicles above the legally established weight limitations.

Each procedure is geared to a specific live load model with specially calibrated load factors aimed at maintaining a uniform and acceptable level of reliability in all evaluations.
The load rating is generally expressed as a rating factor for a particular live load model, using the general load rating equation provided below.

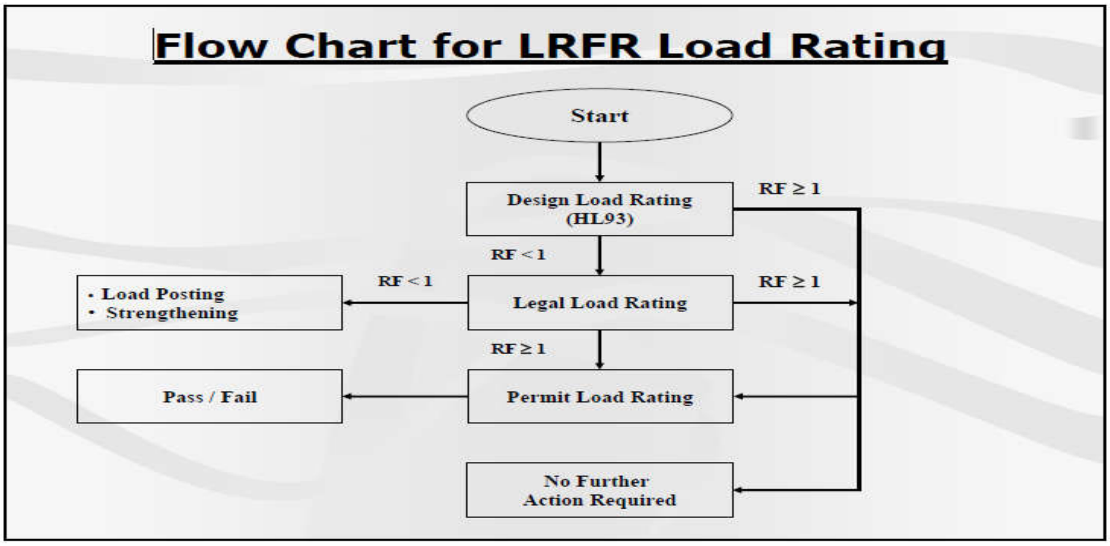


Figure 3.3: Load Rating Chart LRFD.

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect of factored dead loads deducted from the member capacity and divided by the factored live load (i.e., axial force, flexure, or shear):

$$RF = \frac{C-(\gamma_{DC}\times DC)-(\gamma_{DW}\times DW)\pm(\gamma_{P}\times P)}{\gamma_{L}(\, LL\times IM)}$$

Where:

C = Member Capacity (Axial or Moment or Shear)

$C = \Phi_c \Phi_S \phi_n R$

Where the following lower limit shall apply:

$$\Phi_c \Phi_S \geq 0.85$$

RF = Rating factor

DC = Dead load effect due to structural components and attachments

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

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

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

γ_{p} = LRFD load factor for permanent loads other than dead loads = 1.0

γ_{LL} = Evaluation live load factor

Φ_c = Condition factor

Φ_S = System factor

ϕ_n = LRFD resistance factor

System Factor Φ_S:

Non-redundant Bridges $\Phi_S = 0.85$

Condition Factor Φ_c:

· Condition Factor Φ_c is tied to the condition of the member being evaluated:

· Good or Satisfactory $\Phi_c = 1.00$

· Fair $\Phi_c = 0.95$

· Poor $\Phi_c = 0.85$

- **The main steps for each load rating procedure are:**

1- Apply the specific dead and vehicle loads on the bridge to calculate the effect (Axial, Shear and moments) on a selected members using one of structural analysis methods for the bridge loading as per the general expression below.

$$\boldsymbol{Total\ Load} = (\boldsymbol{\gamma_{DC}} \times \boldsymbol{DC}) + (\boldsymbol{\gamma_{DW}} \times \boldsymbol{DW}) + (\boldsymbol{\gamma_{P}} \times \boldsymbol{P}) + \boldsymbol{\gamma_{L}}(\ \boldsymbol{LL} \times \boldsymbol{IM})$$

And the factors in the table (2.1).

2- Calculate the selected member's capacity either for axial loads or shear or moments.

3- Use the load rating equation to calculate the ratting factor (RF) in which the factored dead loads deducted from the member capacity and divided by the factored live load.

3.3.2 The Selected Members for the Load Rating

The superstructure of Upper Atbara River Bridge and Setit is a continuous bailey steel bridge of 5& 7 spans for each bridge, one span is 39.624m.

Analysis of the case study bridge was divided between analysis of the truss and analysis of the floor beam. In each case, the analysis was conducted by using a structural analysis computer software. The computer program used for the case study bridge was SAP2000 [SAP2000 V 14], a commercially available frame analysis software. This program was chosen because it was readily available.

The baily truss consists of 4 rows for each side and 13 panels , the length of the panel is 3.048 m , all spans are symmetric then one row taken all members of trusses were modeled thus 4 members were chosen for the rating considering that the maximum axial forces will be there :

1- Vertical member at the support at panels 1 and 13
2- Diagonal member at the support at panels 1 and 13
3- Bottom chord at the center of the truss at panel 7
4- The floor beam.

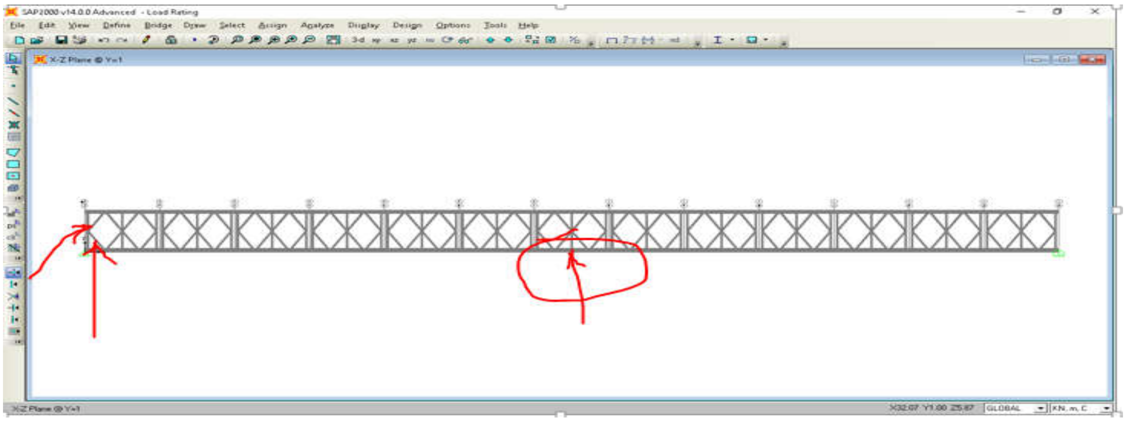


Figure 3.4 Selected Critical Truss Members.

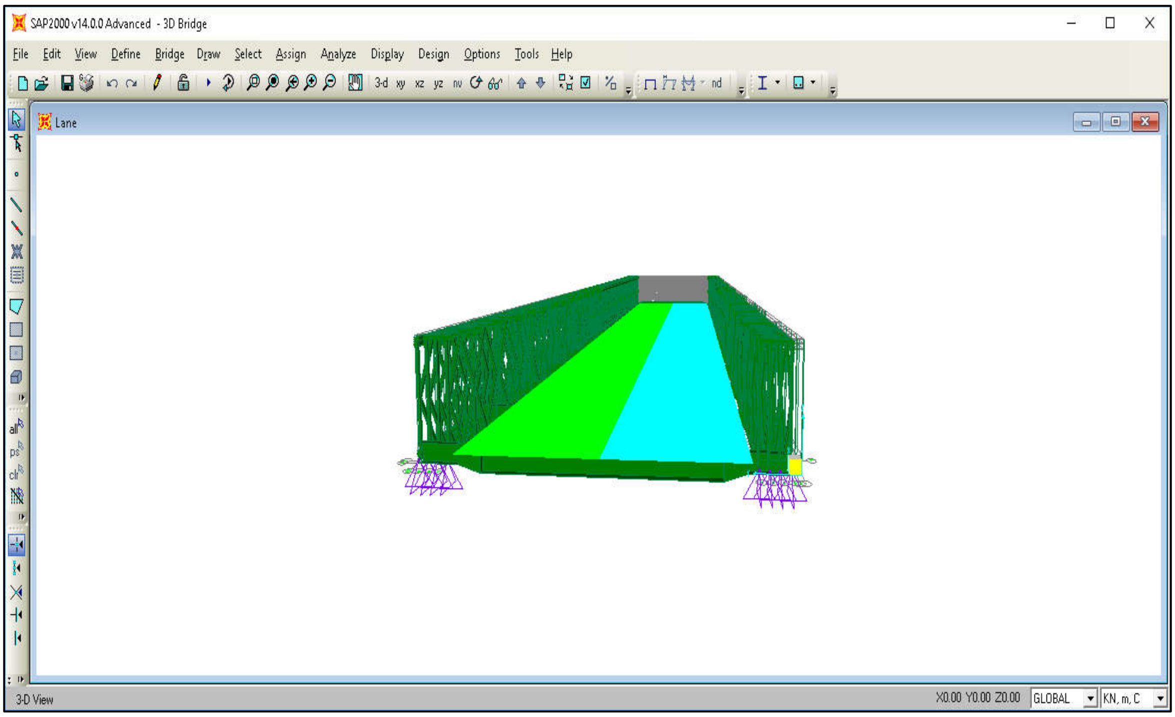


Figure (3-5): 3D model for One Span – 2 Lanes.

Simple 2-D model

All the truss members were modeled as pin-ended truss elements. The truss end supports were modeled as a hinge support at one end of the bridge and as a roller support at the other end to represent the idealized support conditions for the actual bridge. Figure 4.1 shows the model, the frame elements with end releases, and the supports.

3.3.3 Design Load Rating

Design load rating is a first-level assessment of bridges based on the HL-93 Truck loading (The effect of one design truck with variable 4.3 m axle spacing (35.6 kN, 142.3 kN, 142.3kN) combined with the effect of the 9.34 kN/M lane loading. (HL-93K in SAP) and LRFD design standards, using dimensions and properties of the bridge in its present as-inspected condition.

Two levels of rating are performed using two Values of load factors at each level for applying the live truck loads on the bridge:

- Inventory Level:
  Live Load Factor γ_L = 1.75
- Operating Level:
  Live Load Factor γ_L = 1.35

1- Apply Loads to Calculate selected Critical members forces:

a- Loads on Truss Members:

- <u>DL = Self wt. Of truss</u>

= 4.19KN/m distributed load.

Dead load factor for load rating from table 6A 4.2.2-1 of LRFD specs is 1.25

- <u>LL - Live Load</u>

HL-93 Truck and Lane Load, plus impact load multiplied by 1.75 at the inventory level and 1.35 at the operation level and divided over four rows that due to the symmetry only one truss row was taken.

The truck was placed at the center of the span this situation could produce a maximum loads on the truss as well as it will produce a maximum deflection also.

Apply the Truss Distribution Factor:

Assume that each truss carries 75% of the HL-93 load effect the loads thus the applied truck and lane loads will be:

1.1 For the front axle:

- 35.6 over 4
- (IM)(Truck) + Lane: (0.75) x(1.33)x8.9 = 8.9

@ Inventory Level 1.75x8.9 = <u>15.67</u> kN

@Operation Level 1.35x0.75x8.9 = 9.0 kN

1.2 For the two Rear axles:

- 142.3/4 = 35.575 KN
- (IM)(Truck) + Lane: (0.75) x(1.33)x35.575= 35.5

  @ Inventory Level 1.75x35.5 = 62.125 kN

  @Operation Level 1.35x0.75x35.575 = 36.0 kN

1.3 For the lane loading:

- 9.34/4 = 2.335 kN/m

  @ Inventory Level 1.75x2.335 = 4.09 kN

  @Operation Level 1.35x0.75x2.335 = 2.4 kN

The resulted axial member's maximum forces due to the applied dead and live truck loads will be recorded in the tables of results for the **vertical** and **diagonal** members at the supports and for the **bottom chord** at the mid span.

b- Floor Beam

1) Dead load : Self-weight of Beam and Bridge Deck:

- Self-weight of Beam =1.85kN/m
- Self-weight of Bridge Deck =4.14kN/m

2）HL-93 Truck and Lane Live Load:

As the minimum distance of vehicle axle is 4.3m and the spacing of transom floor beam is 3.048m, therefore only one axle factored load (P=145kN) of the design truck will be carried by transom floor beam. The force diagram of transom is shown below, in which For the floor beam the moment and shear forces to be calculated.

- For the 2 lanes axles: P=145/2=72.5 kN.
- (IM)(Truck) + Lane: (1.33)x72.5 = 96.43

  @ Inventory Level 1.75x96.43 = 168.75 kN

  @Operation Level 1.35x72.5 = 97.875 kN
- For the lane loading:

  @ Inventory Level 1.75x9.34x1.33 = 21.74 kN/m

  @Operation Level 1.35x9.34 = 12.6 kN/m

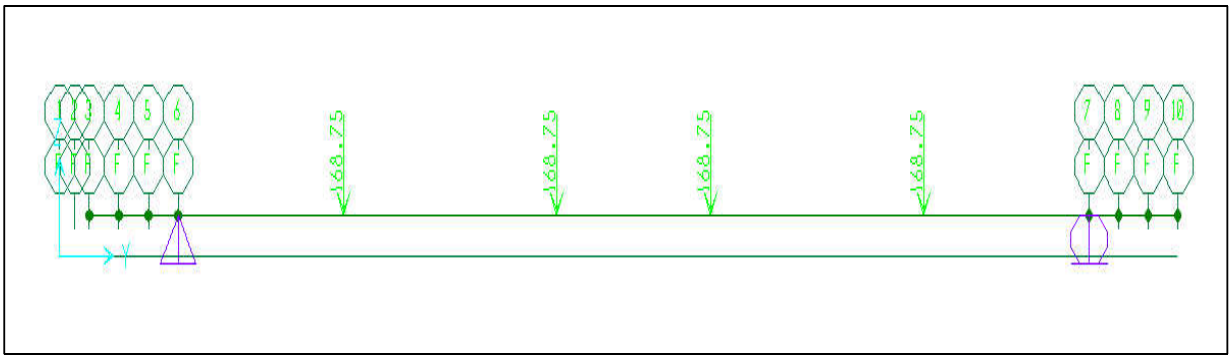


Figure (3-6): Floor Beam Loading.

2- Calculate Members Capacities

The calculation of nominal capacity, C, for the truss members and the floor beam. The capacity was calculated for vertical, diagonal and bottom chord as steel tension and compression members therefore the capacity of the axial forces calculated whereas the floor beam was considered as flexure then the flexure and shear capacities were considered.

Table (3-3): Truss Members Properties.

Member	Length(m)	Area (mm^2)	I_{xx} (mm^4)	I_{yy} (mm^4)	r_x	r_y	kl
241- Bottom Chord(100x50x5)	3.048	1320	$1.5x10^6$	$1.4x10^5$	34.33	10.43	2.74
Vertical Member (C8 Channel 80x43)	2.334	10.248	$1.01X10^6$	$1.66x10^5$	31.39	12.722	2034
Diagonal Member (Tube 80x60)	1.279	1160	$1.07x10^6$	$6.2x10^5$	30.42	23.11	1137

The member's capacities were calculated as per AISC. The allowable stress for tension members was taken as 0.95 times the yield stress. The allowable stress for compression members was calculated based on the effective slenderness ratio (KL/r min), the K factor for compression members was taken as 0.9 for pinned connected members and 0.75 for the continuous top chord members.

- **Truss Tension and compression members Chord**
  - Check Minimum Slenderness Ratio $\frac{L}{\mathrm{r}}$
  - Compute the Design Strength:

Gross Section Yielding Pn =

$$Pn = Fy\,Ag$$
$$\Phi Pn = (0.95\ Fy\,Ag$$

- **Floor Beam:**

1- Flexure Capacity :

For Investigate Compression Flange Local Buckling:

Investigate the compactness of the compression flange.

$$\gamma_f = \frac{b_{fc}}{2t_{fc}}$$

$$\gamma_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}}$$

$if\ \ \gamma_f < \gamma_{pf}$ The flange is compact and,

$$\boldsymbol{M_y = S_x\ F_y}$$

$$\boldsymbol{M_p = Z_x\ F_y}$$

$$\boldsymbol{R_{pc} = \frac{M_p}{M_{yc}}}$$

- **Moment Capacity** = $\boldsymbol{M_{nc\,(FLB)} = R_{pc}\ M_{yc}}$

2- Shear Strength Capacity:

Referring to Section 6.10.9.2 of the Specification, Check Design Shear Strength:

$V_n = \phi V_p$

$V_p = 0.58FyDtw$

Where:

$$D = d - 2t_f$$

For shear yield Strength, ϕ=1

- Shear Capacity = $V_n = \phi V_p$

3- Find The Load Rating factor :

The rating will be done using the dead and HL-93 Truckloads at inventory level and also at operation level foe each member (Vertical, diagonal, bottom chord and floor beam).

$$RF = \frac{C - (\gamma_{DC} \times DC) - (\gamma_{DW} \times DW) \pm (\gamma_P \times P)}{\gamma_L(LL \times IM)}$$

- If the load rating factor RF >1, at the inventory or operation level, the member's has a sufficient capacity.
- If the load rating factor RF < 1, at the inventory or operation level, the member is adequate to retain the load at this level then further calculation for the legal load rating is required.

3.3.4 Legal Load Rating

Bridges that do not have sufficient capacity under the design-load rating when (RF<1) shall be load rated for legal loads to establish the need for load posting or strengthening. Load rating for legal loads determines the safe load capacity of a bridge for the AASHTO family of legal loads and State legal loads, using safety and serviceability criteria considered appropriate for evaluation. A single safe load capacity is obtained for a given legal load configuration.

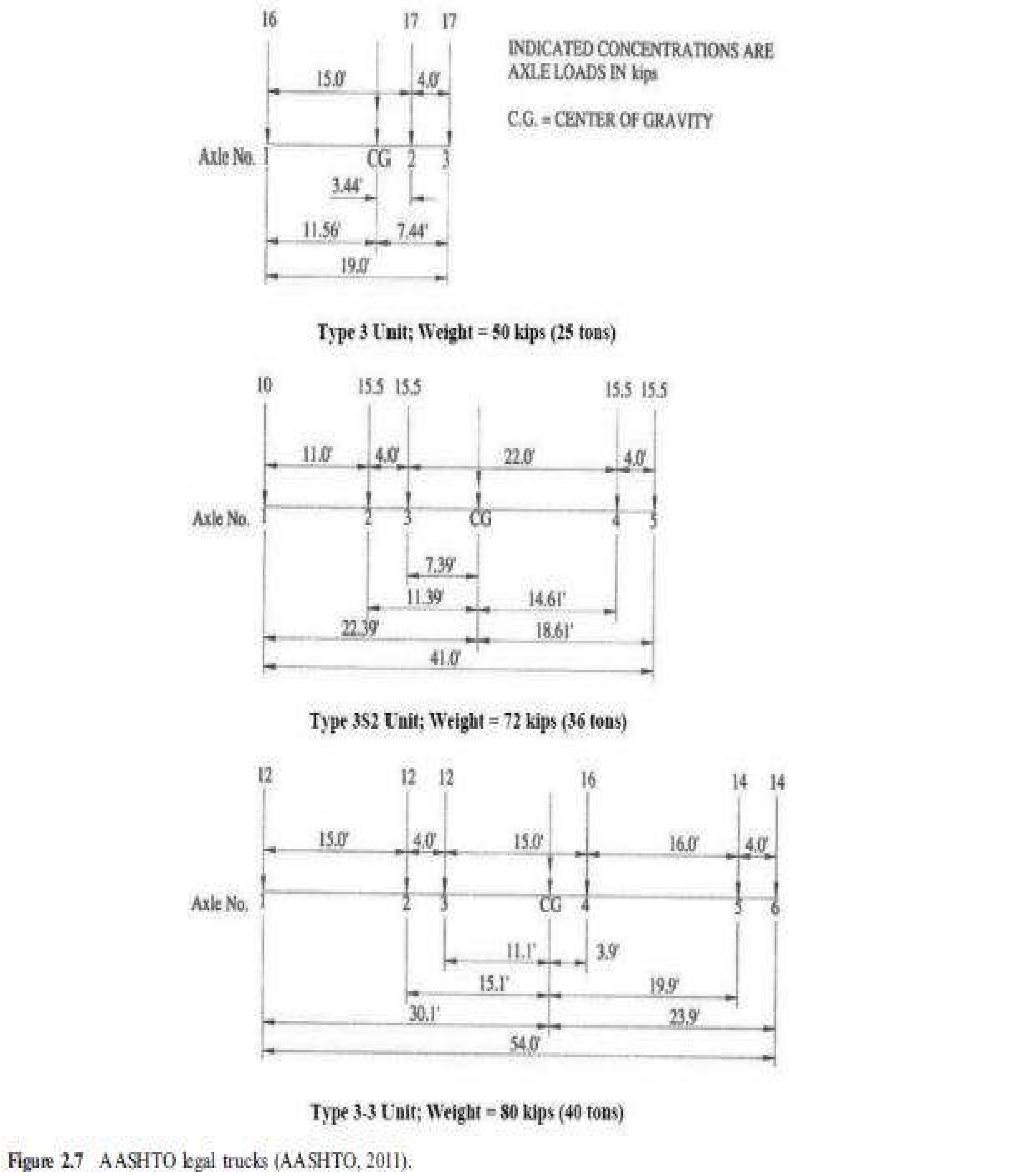


Figure (3-7): AASHTO Legal Loads.

The legal load rating uses three types of AASHTO truck illustrated in figure (3-31,

1- 25 tons of 3 axles truck named (Type 3)
2- 36 tons truck of 5 axles named (Type 3S2).
3- 40 tons truck of 6 axles named (Type 3-3).

Each one of these loads will be applied individually to find the members forces, the same members at the design load rating will be subjected to these loads, then the rating factor will be calculated.

1- Apply loads to calculate selected critical members forces:

Loads on Truss Members:

- DL = Self wt. Of truss

  4.19KN/m distributed load.

- LL - Live Load

  Apply the Truss Distribution Factor assuming that each truss carries 75% of the trucks load effect and dynamic impact factor of 1.33, for one single truss out of four rows, the applied truck loads will be:

  - 25 tons of 3 axles truck named (Type 3)
    18.85, 18.85, 17.76 kN
  - 36 tons truck of 5 axles named (Type 3S2).
    4 axles @ 17.18 and 11.1 kN
  - 40 tons truck of 6 axles named (Type 3-3).
    3 axles @ 13.3 kN , 17.76, 17.76 kN

The live load factor taken from the table below was 1.8.

Table (3.4): Traffic Volume Load Factor

ADTT > 5000	**1.80**
ADTT = 1000	**1.65**
ADTT < 100	**1.40**

2- Calculate members Capacities

The member's capacities were already calculated in the above design load rating.

3- Find The Load Rating factor

The rat factor was calculated for each truck type 3, 3S2 and 3-3, the rating results would help in the determining the safe truck load to be used on the bridge

3.4 Upgrading of Superstructure Methodology

3.4.1 Introduction

Based on the findings and results of the load rating and analysis for the baily truss members, the existing superstructure configuration is considered not capable of withstanding the AASHTO HL- 93 LRFD Loadings. The analyses and rating shows that the bridge cannot be subjected to a 2-lane loading neither concrete deck also.

Even with the current condition of the posting, the bridge demonstrate low capability for sustaining the loads, particularly the bottom chords.

Bridge strengthening could be considered but it might be costly that dismantling and changing the sizes of chosen panels chords and vertical members are the supports and midspan is probably not practical.

Thinking about alternatives of changing the entire superstructure is an option which is to some extent could be a practical solution to allow the permanent use of the bridge and increase the durability of the structure.

After the comparison of construction alternatives was completed, a prestressed Bulb T-Girder Concrete system and Steel Plate Girder was identified as two best solutions. The design of the bridge superstructure was completed first by developing a preliminary design, and should be followed by completing a more detailed design later, where the aspects of the preliminary design were adjusted to meet certain requirements. These requirements included moment capacities developed by the loading conditions as well as section properties to accommodate prestressing steel in the primary girders. SAP software was utilized during the design process and helped facilitate adjustment of the design.

The preliminary design of the chosen bridge superstructures will include:

- Preliminary girder design (cross-sectional dimensions)
- Potential girder spacing
- Sketch including primary superstructure components (girders, deck, sidewalks and parapets)

3.4.2 Prestressed Girders Superstructures Design:

1- Concrete Precast T-Girder Size Selection:

This preliminary design was based on assumed dimensions obtained from the PCI Bridge Design Manual (2003). Preliminary design was used to determine initial dead load and live load conditions.

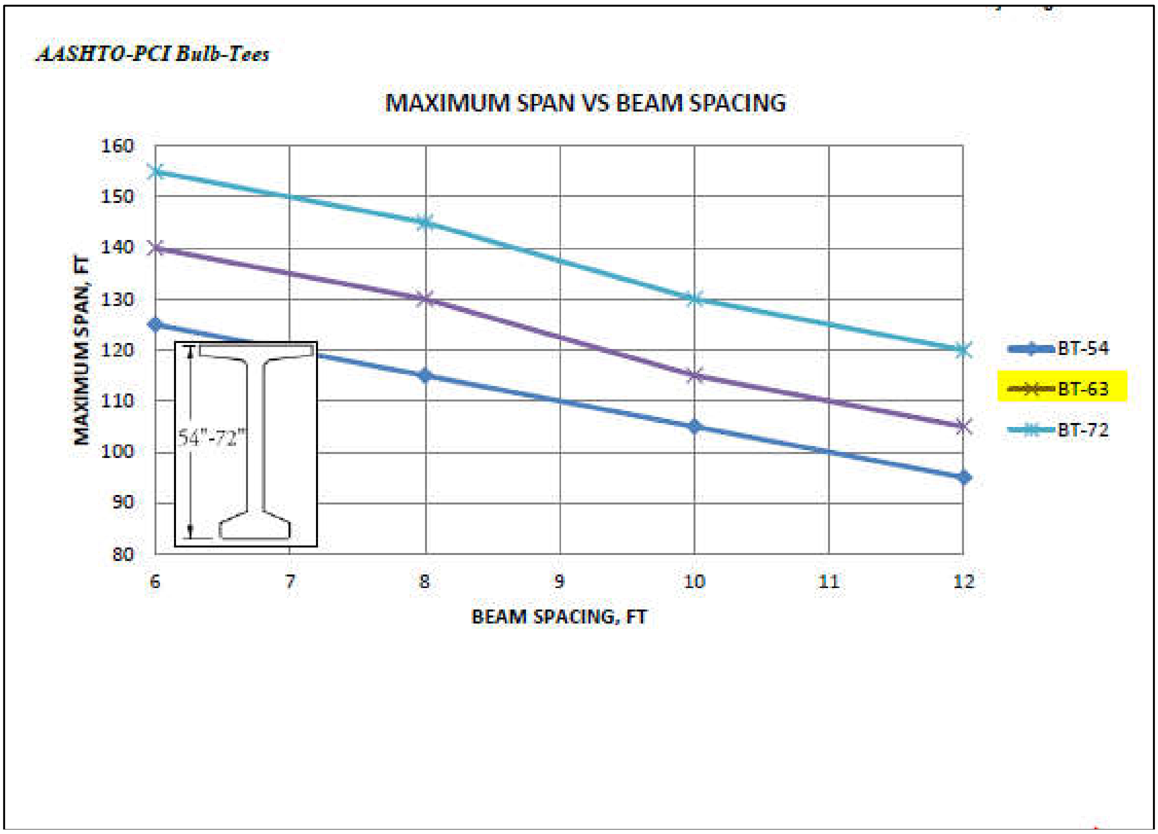


Figure 3.8: Selection of Preliminary Girder Charts (PSC Institute).

Developing this initial design provided cross-section drawings that were used to evaluate the structure as a whole and determine the viability of the initial design. In order to develop the final design, information from the preliminary design would be utilized in conjunction with dead and live load calculations as well as prestressed steel calculations in order to develop the proposed design.

2- Dead Load Calculations

The dead load of the concrete deck, asphalt wearing surface, and railings were calculated and distributed to the bridge girders. Table displays the various dead loads calculated for the design.

Table 3.5 – Dead Loads for Final Design

Bridge Component	Dead Load (kN/m)
Concrete Deck (DC1)	12
One Girder (DC1)	11.0
Railings & Barrier (DC2)	2.74
Asphalt Wearing Surface (DW)	3.22

The following Dead Loads used to obtain from the SAP 2D analysis:

• The maximum positive dead-load moments occur at mid spans.

• The maximum negative dead-load moments occur over the center of supports.

3- Live Load Calculations

The live load conditions of the superstructure were determined using the HL-93 truck load. The distribution factors for the interior and exterior beams were calculated based on the equations then applied to these two loading conditions. The resulting distribution factors and adjusted loads are shown in table 3.6 below. For the interior girders, the larger resulting value of the two distribution factor equations was chosen to be used during the design in order to produce the maximum loading condition. The calculations that were completed and the results are shown in Table 3.6 can be seen in Appendix B.

- **Distribution Factor for Bending Moment**

For two lanes or more:

$$\text{DFM} = 0.075 + \left(\frac{S}{2900}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{Kg}{L\,t_s{}^3}\right)^{0.1}$$ LRFD Table 4.6.2.2.2b-1

Where:

DFM= Distribution factor for moment

S = Girder spacing c/c = 2800 mm

L= Span Length = 39624 mm

$Kg = n\ (I+A.e^2_g)$

$$n = \frac{Eg}{Es},\ \frac{\text{Elastic modulus of girder}}{\text{Elastic modulus of deck slab}}$$

A= Cross section area of girder

e_g= Distance between the centroid of girder and slab

I = Moment of inertia of beam

For the simplicity and the ease of calculations, consider the following values for the entire girders:
For the Moments DFM = 0.62
For the Shear forces DFV = 1

Table 3.6– Moments Distribution Factors for Live Load Conditions

Interior Girder	Value
Distribution Factor	0.62
Rear Axle Load (KN)	88.23
Front Axle Load (KN)	22.07
Design Lane Load (k/m)	5.79

- **Distribution Factor for Shear Force:**

From LRFD Table 4.6.2.2.2b-1:

$$DFV = 0.2 + \left(\frac{S}{3600}\right)\ \left(\frac{S}{10700}\right)^2$$

Table 3.7– Shear Distribution Factors for Live Load Conditions

Interior Girder	Value
Distribution Factor	1
Rear Axle Load (KN)	142.3
Front Axle Load (KN)	35.6
Design Lane Load (k/m)	9.34

The live loads were applied to girders using the SAP software. The design lane load was applied longitudinally to the girders as a distributed load and the design truck axle loads were applied as a point loads. The loads of the design truck were then moved along the length of the girder, and the reaction forces within the girder were calculated for each location.

4- Load Combinations

Factored Shear and Moment Envelopes

The following load combinations were considered:

- **Strength I:**

The general equation used to obtain the different loads combinations and their effect on the structure:

1.25 M DL. + 1.5 M DW + 1.75 MLL

- Positive Moment
- Negative Moment
- Shear

- **Service I:**

Load combination relating to the normal operational use of the bridge with a 55 MPH wind and all loads taken at their nominal values.

"Applicable for maximum compressive stresses in beam ONLY. For tension, see Service III."

The general equation used to obtain the different loads combinations and their effect on the structure:

1.3(LL + IM) + 1.0DC1 + 1.0DC2 + 1.0DW

- Positive Moment:
- Negative Moment:

- **Service III:**

Load combination for longitudinal analysis relating only to tension in prestressed concrete structures with the objective of crack control.

"Applicable for maximum tension at midspan ONLY. For compression, see Service I."

Service3 = 1.0DC + 1.0DW + 0.8LL +1.0(CR, SH)

5- Prestressed Steel Design

Referring to Table S5.9.4.1.2-1 of (AASHTO LRFD Design Specifications-SI Units 2005), the stress limit in areas with bonded reinforcement sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section.

The strength parameters were based on a specified concrete compressive strength of 40 Mpa. The values of the other parameters used in the design of the girders and prestressed steel are shown in table below.

Table 3.8- Design Values for Prestressed Steel Design

Parameter	Symbol	Value (MPa)
Specified compressive strength of concrete (Mpa)	f'_c	40
Compressive strength of concrete at time of initial prestressing (Mpa)	f'_{ci}	30
Allowable compressive stress immediately after transfer (Mpa)	$0.6f_{ci}$	18
Allowable tensile stress immediately after transfer	$0.63\sqrt{fci}$	3.6
Allowable compressive stress at service load	$0.45f`_c$	18
Allowable tensile stress at service load	$0.25\sqrt{f`c}$	1.58

- Stress calculations at transfer:

$$f_{top} = \frac{-P_t}{A_g} + \frac{P_{t\,e0}}{S_t} - \frac{M_g}{S_t}$$

P_t = Initial prestressing force taken from (AASHTO LRFD Design Specifications-SI Units 2005) Table 5.9.3-1 =$0.75f_{pu}$

A_g = Gross area of the basic beam (mm^2)

e_0 = Distance between the neutral axis of the noncomposite girder and the center of gravity of the prestressing steel = 1100 (mm).

Table 3.9 - Steel Strand Properties

Property	Value
Grade of steel (Mpa)	1860
Required area of prestressed steel (mm^2)	5922
Size of steel strands – diameter (mm)	11.21
Cross-sectional are of strands (mm^2)	98.71
Total number of required strands	60

As can be seen in Table 3.6, a strand size of 11.21 mmm was chosen to be used for the bridge design. This value was obtained per the PCI Bridge design manual because this size would result in fewer required strands and would take full advantage of the concrete strength. The steel grade shown in table was also obtained from the PCI Bridge design manual 2003 because this grade has a minimum ultimate strength of 1860 Mpa (270 ksi) and is most often used in prestressed bridges.

Once the size and required number of strands were determined, the spacing requirements of the prestressed strands were designed. According to the PCI Bridge Design Manual, steel strands used in post-tensioning are often formed into tendons that include a minimum of one strand to a maximum of 55 strands (2003). In addition, these tendons are then placed in post-tensioning ducts that must be at least ¼ inch larger than the nominal diameter of the tendon, unless multiple tendons are used in which case the ducts should be at least twice the cross-sectional area of the tendons (PCI Bridge design manual, 2003)

6- Calculation of Girder's Moment Capacity

The girder section moment capacity was calculated and compared to the maximum factored applied moment from the above load combinations to check the adequacy of the section to sustain the loads.

- **Stress in prestressing strands**

$$f_{ps} = f_{pu}\{1 - k(\frac{c}{d_p})$$

Where:

$k = 2(1.04 - f_{py}/f_{pu}) = 0.28$ (LRFD5.7.3.1.1-2)

d_p = h – (distance from bottom of beam to location of P/S steel force)

For "c": The distance from the neutral axis to the compression face of the member may be determined as follows:

Assuming rectangular section behavior with no compression steel or mild tension reinforcement:

$$C = \frac{A_{ps} * f_{pu}}{0.85\, f`_c \beta_1 b + k * A_{PS}(\frac{f_{PU}}{d_P})}$$

A_{PS}= Area of prestressed steel.

- **The factored flexural resistance:**

M_R , shall be taken as ϕMn, where M_n is determined using LRFD Eq. S5.7.3.2.2-1:

$$M_n = A_{Ps} * f_{PS}\left(d_P - \frac{a}{2}\right) + A_{S`} f_{y`}\left(d_{S`} - \frac{a}{2}\right) + 0.85\, f_{c`}(b - b_w)\beta_1 h_f(\frac{a}{2} - \frac{h_f}{2})$$

Where:

A_{PS} = area of prestressing steel = 5922 mm^2

F_{PS} = average stress in prestressing steel = 1735.29 MPa

(As = area of nonprestressed tension reinforcement = 0.0 mm^2

f_y = specified yield strength of reinforcing bars = 460 MPa

d_S = distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement (mm), NA

$A_{S`}$ = area of compression reinforcement = 0.0 mm^2

$f_{y`}$ = specified yield strength of compression reinforcement, NA

$d_{S`}$ = distance from the extreme compression fiber to the centroid of compression reinforcement (in.), NA

fc` = specified compressive strength of concrete at 28 days= 40 MPa

b = width of the effective compression block of the member = 660 mm

bw = web width = 200 mm

β1 = stress block factor specified in S5.7.2.2, NA

h_f = compression flange depth of an I or T member (in.), NA

a = β 1c; depth of the equivalent stress block (in.) = 0.85(538) = 457.3 mm

M_R > Mu @ Strength limit state, the section was Adequate

3.4.3 Steel Plate Girders Bridge

1- Steel Girder Size Selection

A steel Plate I-girder bridge design was the second alternative design considered in this thesis. Steel I-girder bridges are supported by one or more welded steel plates. The cross section consists of web plates connected to a bottom and top flange plates.

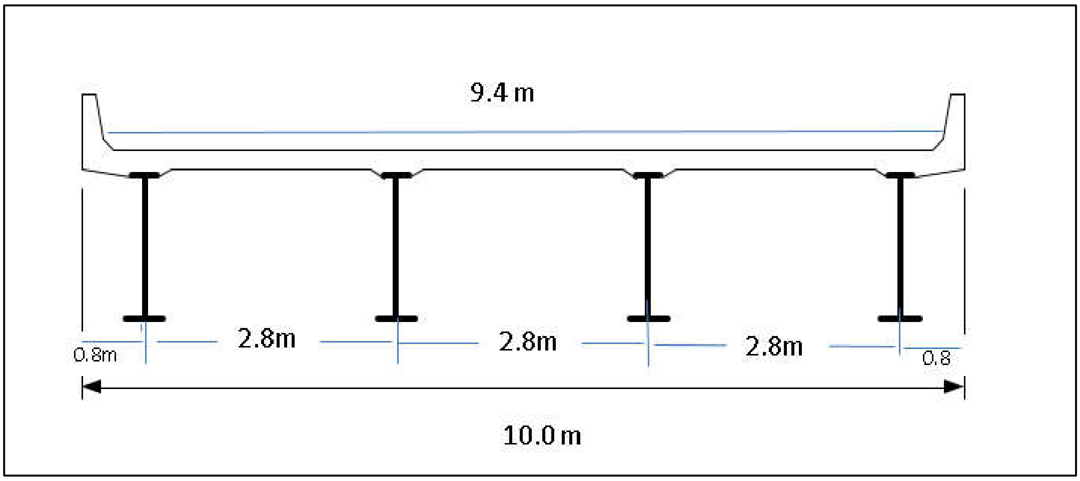


Figure 3.9: Deck Section

2- Dead Load Calculations

The dead load of the concrete deck, asphalt wearing surface, and railings were calculated and distributed to the bridge girders. Table 3.10 displays the various dead loads calculated for the design. The girder weight was calculated during the iterative design process for multiple girder cross-sections. The calculation process can be seen in Appendix B where the dead load of other investigated cross-sections is displayed.

Table 3.10– Steel Girder Dead Loads for Final Design

Bridge Component	Dead Load (kN/m)
Concrete Deck (DC1)	13.44
One Girder (DC1)	11.8
Railings & Barrier (DC2)	2.74
Asphalt Wearing Surface (DW)	3.22

The following dead load results were obtained from the SAP 2D analysis:

• The maximum positive live-load moments occur at mid spans.

• The maximum negative live-load moments occur over the center of supports.

3- Live Load Calculations

The live load conditions of the superstructure were determined using theHL-93 design truck shown in as well as a 9.34 k/m design lane load. The distribution factors for the interior and exterior beams were calculated based on the equations of LRFD Table 4.6.2.2.2b-1 demonstrated in the previous section and then applied to these two loading conditions. The resulting distribution factors and adjusted loads are shown in table 4.22 below. For the interior girders, the larger resulting value of the two distribution factor equations was chosen to be used during the design in order to produce the maximum loading condition.

All live load calculations were performed in SAP 2000 and CSi Bridge using a beam line analysis. The nominal moment data from SAP was then input into Excel. An Impact Factor of 1.33 was applied to the truck and tandem loads and an impact factor of 1.15 was applied to the fatigue loads within SAP.

It was determined that the maximum moment the girder experienced occurred in the center of the beam when the rear axles of the truck were spaced 14 feet (4.3m) apart. The resulting maximum moments for the interior and exterior girders are shown in tables in chapter 4.

4- Load Combinations

- Factored Shear And Moment Envelopes

- The following load combinations were considered:

a- Strength I:

1.25 M DL. + 1.5 M DW + 1.75 MLL

- Positive Moment
- Negative Moment

b- Service II:

$1.3(LL + IM) + 1.0DC1 + 1.0DC2 + 1.0DW$

- Positive Moment:

- Negative Moment:

- **Section Properties:**

For non-composite section:

Table 3.11: Non Composite Girder Section Properties

$Y_{c.g}$	y_{bot}	y_{top}	S_{bot}	S_{top}
553.9771	553.9771	553.971	4.283E+07	4.283E+07

S= Section modulus for the extreme top and bottom fiber

3.1: Effective Flange Width, b_{eff}:

For an interior beam, b_{eff} is the lesser of:

1- $L_{eff}/4 = 39.62/4$ = 9.905
2- 12ts+bf/2 = 12/.2+0.428/2 = 2.6
3- S= Average distance between girders = 2.8
4-

❖ b_{eff} = 2.6 m = 2600 mm

Calculate for Short-Term Composite (n = 8)

ES= Steel girder modulus of elasticity

EC= Concrete slab modulus of elasticity

Transformed flange width = b_{eff}/n = 2600/8 = 325 mm

Transformed flange Area = 325 x 200 = 65000 mm^2

For composite section:

Table 3.12: Composite Girder Section Properties.

Yc.g	y_{bot}	y_{top}	$y_{topslab}$	S_{bot}	S_{top}	$S_{topslab}$
794.99	794.99	312.96	512.96	6.349E+07	1.613E+08	9.840E+07

5- Shear And Moment Stresses Envelopes:

Combined shears, and flexural stresses can be computed at the controlling locations. The resulted stresses will be compared to the yield strength of the steel girder of **345 Mpa where the permissible stresses is 0.9*345.**

i. **Maximum stress in the top of the girder due to positive moment (located at 0.5L) for the Strength I Limit State is computed as follows:**

- Non-composite dead load :

  f_{DL} non-comp DL = $\frac{M_{DL}}{S_{top}}$

  Railing Composite DL moment:

  $M_{parapet}$ = 537.7 KN.m, S_{topgdr} = 1.613 x10^8 mm³

  $f_{Par} = \frac{M_{Par}}{S_{top}}$

- Wearing composite DL:

  M_{DW} = 632 KN.m

  $fDW= \frac{MDW}{Stop} =$

- Live Load: HL-93+IM :

  M_{LL}= 2121 KN.m

  $f_{LL}= \frac{M_{LL}}{S_{top}}$

- Calculation of top stresses for strength limit state :

$1.25\, f_{DL} + 1.5\, f_{DW} + 1.75 f_{LL}$ = 182.12 N/mm^2 < 345 Mpa O.K.

Similarly compute for the bottom and then the negative moment.

- Calculation of bottom stresses for strength limit state :

  For non-composite:

  S_{bottom} = 6.349E+07 mm³

  $1.25\, f_{DL} + 1.5\, f_{DW} + 1.75 f_{LL}$ < 345 Mpa O.K.

ii. **Maximum stress in the top of the girder due to negative moment (located at 0.5L) for the Strength I Limit State is computed as follows:**

- Calculation of top stresses for strength limit state:

For non-composite:

S_{top} (Non-composite) = 4.283E+07 mm³

S_{top} (Composite) = 1.613E+08 mm³

- Compute for the top negative moment:

  f_{DL}=119.3, f_{par} = 8.47 , f_{DW} = 10 , fLL = 33.4

  1.25 f DL. + 1.5 f DW + 1.75 fLL= 266 N/mm^2 < 345 MPa O.K.

6- Conclusion:

By the same, the service limit could be checked, and obviously it will be within the permissible limits.

The yield strength of the steel girder is **345 MPa**, so the section is adequate.

Strength II is not considered since this deals with special permit loads. Strength III and V are not considered as they include wind effects, which will be handled separately as needed. Strength IV is considered but is not expected to govern since it addresses situations with high dead load that come into play for longer spans. Extreme Event load combinations are not included as they are also beyond the scope of this example. Service I applies to wind loads and is not considered (except for deflection) and Service III and Service IV correspond to tension in prestressed concrete elements and are therefore not included in this example.

3.4.4 Concrete Deck Design

- **Width of Equivalent Interior Strips [LRFD 4.6.2.1.3]**

The deck is designed using equivalent strips of deck width. The equivalent strips account for the longitudinal distribution of LRFD wheel loads and are not subject to width limitations. The width in the transverse direction is calculated for both positive and negative moments. The overhangs will not be addressed because it's very small.

Width of equivalent strip for +Ve positive moment = 660+0.55S

Width of equivalent strip for –Ve negative moment = 1220+0.25S

- **Live Loads for Equivalent Strips**

All HL-93 wheel loads shall be applied to the equivalent strip of deck width, since the spacing of supporting components in the secondary direction (longitudinal to beams) exceeds 1.5 times the spacing in the primary direction (transverse to beams). [LRFD 4.6.2.1.5]

HL-93 wheel load.................................. P = 72 kN

HL-93 wheel load for negative moment..... P_{neg}

HL-93 wheel load for positive moment..... P_{pos}

$$= \frac{P}{\text{Eq.strip neg}} \; X\, IM$$

Location of Negative Live Load Design Moment is taken at a distance from the supports....... Loc negative = min (1/3 b_{tf} , 380 mm)

$$= 380 \text{ mm}$$

- **HL-93 Live Load Design Moments**

Instead of performing a continuous beam analysis, Table A4-1 in AASHTO LRFD-SI Units) Appendix A4 may be used to determine the live Load design moments.

The dimensions are in feet and an interpretation is required:

S= 2.8X3.048 = 8.5 feet, Distance of neg. moment = 3.8x3.048 = 11.5

- For positive moment: between S is 8.3 and 8.6:
  M +Ve = 5.99 kip/ft = 8.1 KN.m/m

- For negative moment: is at 11.5 in between :
  M –Ve = 3.6 Kip/ft = 4.9 KN.m/m

- **Dead Load Design Moments**

Design width of deck slab....................... = 1m

"DC" loads include the dead load of structural components and non-structural attachments.

Self-weight of deck slab............... = t_{slab} x b_{slab} x γ_{conc} = 0.2x1x24 = 4.8 kN/m

Weight of traffic barriers......... = $Wt_{barrier}$ x b_{slab} = 2.74x1 = 2.74 kN/m

Weight of Future Wearing Surface.....= t_{WS} x b_{slab} x γ_{ws} = 0.05x1x23 = 1.15kN/m

- **Analysis Model for Dead Loads**
  SAP-2D software was used to produce a maximum negative and positive moment for the deck using the dead loads calculated above tables below shows the calculations results.

The deck is modeled as a beam supported at the girder locations.

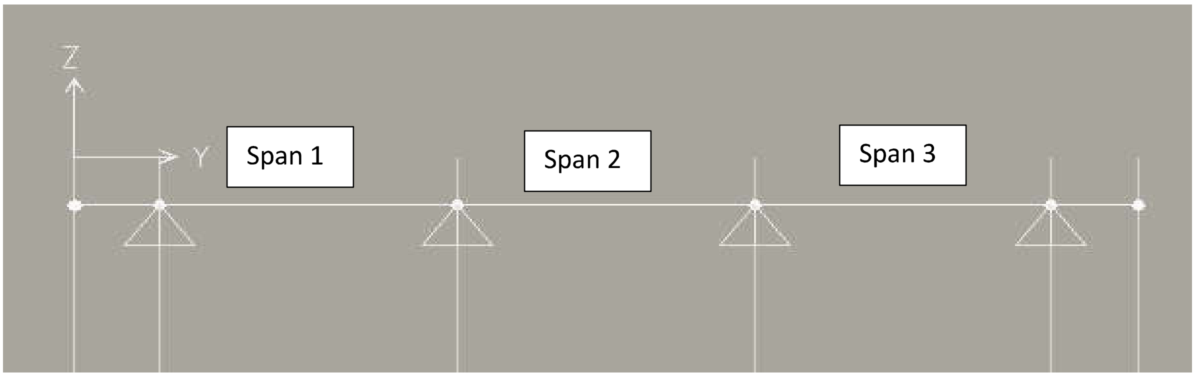


Figure 3.10: Deck Slab Transverse Section Model in SAP

3.4.5 Bearing Pad Design

The bearing pad design began once the superstructure live and dead loads were finalized. The design process consisted of four main steps:

1. Determined what type of bearing pad would be used.
2. Assumed bridge dimensions and site conditions.
3. Calculated the minimum thickness of the chosen bearing pad.
4. Made conclusions based on the bearing capacity of the chosen pad.

Since a bearing translates the loads from the superstructure to the substructure, it was important to design the bearing properly. For reassurance in the final design, the bearing capacity was recalculated although no dimensions and loads had changed from its original design.

1- Types of Bearing Pads

As there are several different types of bridge bearings, the first step was to choose a type for application. The choice was made based off prior background research. Table below shows, detailed bridge bearings' capacities, translations, rotational maximums, and costs. An elastomeric bearing pad was chosen to as it has the lowest costs compared to the other bearing types.

Table 3.13 - Bearing Type Capacities (Chen & Duan, 2000)

Bearing Type	Load		Relative Translation		Rotational Max (rad.)
	Min (kips)	Max (kips)	Min (in)	Max (in)	
Sliding Plate	0	>2,250	1	>0.40	0
Single Roller	0	100	1	>0.40	>0.04
Multiple Roller	115	2,250	4	>0.40	>0.04
Pin and Link	270	1,000	0	0	>0.04
Elastomeric	0	100	0	0.60	0.01
Pot	270	2,250	0	0	0.02

2- Calculations Steps for Bearing Pad Design

- Design Step 1 - Obtain Design Criteria
- Design Step 2 - Select Optimum Bearing Type.
- Design Step 3 - Select Preliminary Bearing Properties.
- Design Step 4 - Select Design Method (A or B)
- Design Step 5 - Compute Shape Factor
- Design Step 6 - Check Compressive Stress
- Design Step 7 - Check Compressive Deflection.
- Design Step 8 - Check Shear Deformation.
- Design Step 9 - Check Rotation or Combined Compression and Rotation.
- Design Step 10 - Check Stability.
- Design Step 1 - Check Reinforcement.

For the details of the calculations see appendix D

3.4.6 Substructure Design Verification

A- Pier Cap

1- Geometry

This section provides the design dead loads applied to the substructure from the superstructure. The self-weight of the substructure is generated by the analysis program for the substructure model.

2- Pier Dead Loads:

The reaction were calculated due to the Dead Load at pier for interior and exterior Beam:

The calculations for one girder:

- Reaction due to dead load of girder
- Reaction due to dead load of the deck slab
- Reaction due to the dead load of the wearing surface
- Total dead load reaction for one girder

3- Live load transmitted from the superstructure to the substructure:

The live load reaction is applied to the deck at the pier location. The load is distributed to the girders assuming the deck acts as a series of simple spans supported on the girders. The girder reactions are then applied to the pier. In all cases, the appropriate multiple presence factor is applied.

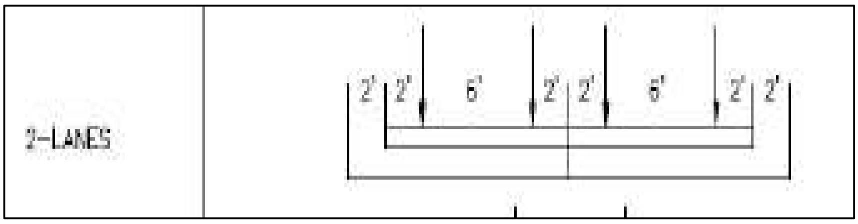


Figure 3.11: HL-93 truck Load above pier Cap for 2-Lanes.

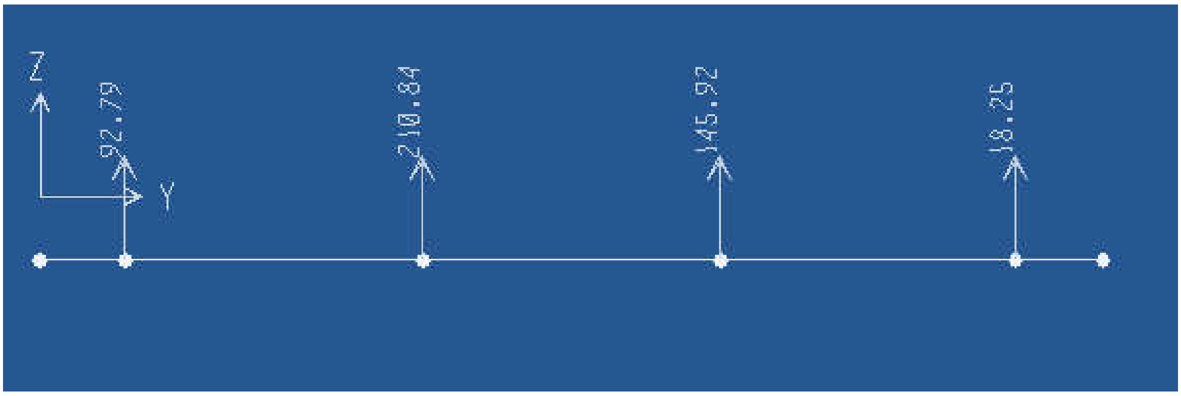


Figure 3.12: The worst case for the reaction induced by HL-93 truck (including impact factor) and lane load.

4- Braking Force

The Breaking Force, BR, is taken as the maximum of:

A) 25% of the Design Truck

B) 25% of the Design Tandem

C) 5% of the Design Truck with the Lane Load.

D) 5% of the Design Tandem with the Lane Load.

In the normal practice always the truck loading in case (A) govern thus:

BR = 0.25x2(35+ 142.3+ 142.3) = 159.8 kN

BR Force on piers = Kpier x BR

$K_{pier} = \frac{Npads.pier\ .\ Kpad}{\sum(Npads.pier+Npads.end\ bent)Kpad}$

$= \frac{2}{2+2} = 0.5$

Braking force on piers = 0.5x159.8= 79.9 kN

Braking force on one pier = 079.9/4= 20 kN

Load on the Pier = µx79.9= 0.03X79.9=2.4 KN on each bearing point

5- Wind Loads

For the calculation of wind loads, assume that the bridge is located in the "open country" at an elevation of more than 40feet (13.2m) above the ground.

Table 3.14: The Wind Loads Initial Values

SECTION 3 (SI): LOADS AND LOAD FACTORS

Table 3.8.1.1-1 Values of V_0 and Z_0 for Various Upstream Surface Conditions.

CONDITION	OPEN COUNTRY	SUBURBAN	CITY
V_0 (km/hr.)	13.2	17.6	19.3
Z_0 (mm)	70	1000	2500

Take Z = 15m V_o = 13.2 Km/h

Z_o = 0.075m

- Horizontal Wind Load on Structure: (WS)

Design Pressure according to LRFD:

$$P_D = P_B\left(\frac{V_{DZ}}{V_B}\right)^2 = P_B\frac{V_{DZ}^{\ 2}}{25\ 600} \qquad (3.8.1.2.1\text{-}1)$$

PB - Base Pressure - For beams, PB = 0.0036N/mm^2 when VB = 168Km/h.

VB - Base Wind Velocity, typically taken as 168Km/h.

V_{10} - Wind Velocity at an elevation of Z = 15m , V10 = 168Km/h

VD_Z - Design Wind Velocity (Km/h)

$$V_{DZ} = 2.5\ V_0 \left(\frac{V_{10}}{V_B}\right) \ln\left(\frac{Z}{Z_o}\right) \qquad (3.8.1.1\text{-}1)$$

$VD_Z = 2.5X13.2\ (\frac{168}{168})\ \ln\frac{15}{0.07} = 177.12$ Km/h

$P_D = 3.6(\frac{177.12}{168})^2$

$= \underline{\underline{4}}$ KN/m^2

6- Stream water pressure

The flowing water pressure can be calculated according LRFD equation:

$P=5.14x10^4\ C_D\ V^2$

P= Stream pressure Mpa

C_D = Drag coefficient for piers = 0.7 for circular pier

V = Velocity of flowing water = 4.12 for our case

$P= 5.14x10^4\ x0.7x4.12=6.1x10^{-6}$ Mpa = $\underline{\underline{\mathbf{6.1}}}$ KN/m^2

7- Factored Limit States Loading On The Pier Cap:

- **Strength I Limit State:**

Strength1 = 1.25DC +1.5DW + 1.75LL + 1.75BR + 0.50(TU, CR, SH)

Beam Moment & Shear:

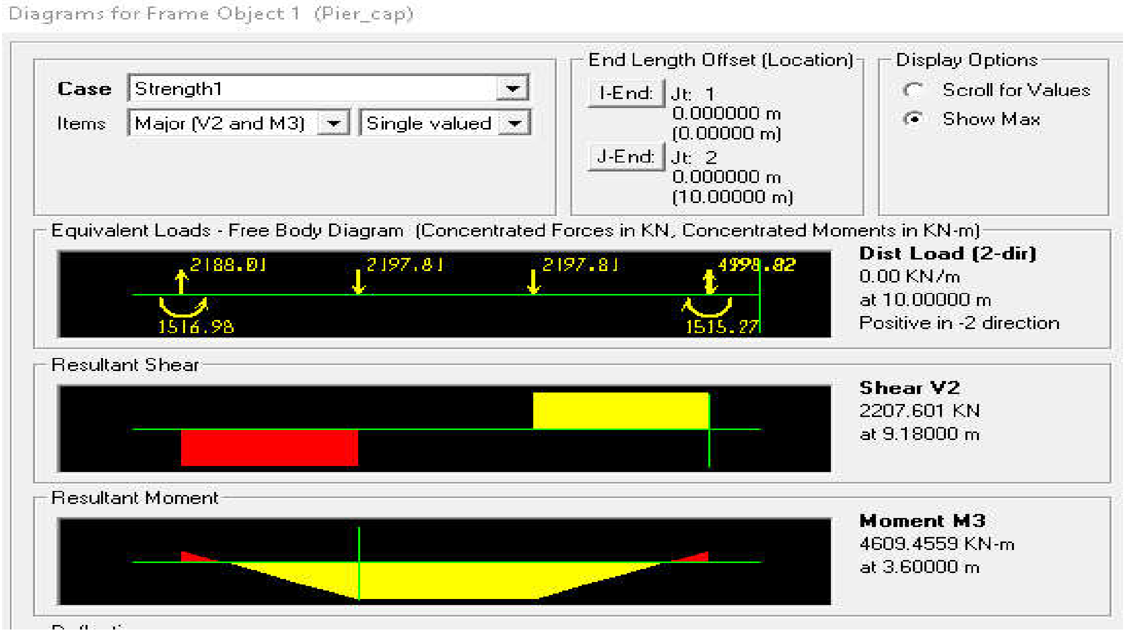


Figure 3.13: The Strength I Limit States Moments and Shear on the Pier Cap.

- **Strength V Limit State**

StrengthV = 1.25DC +1.5DW + 1.35LL + 1.35BR + 0.50(TU, CR, SH) + 1.30WS + 1.0WL

Figure 3.14: The Strength V Limit States Moments and Shear on the Pier Cap.

- **Service I Limit State:**

Strength1 = 1.0DC +1.0DW + 1.0LL + 1.0BR + 1.0(TU, CR, SH) + 1.0WS + 1.0WL

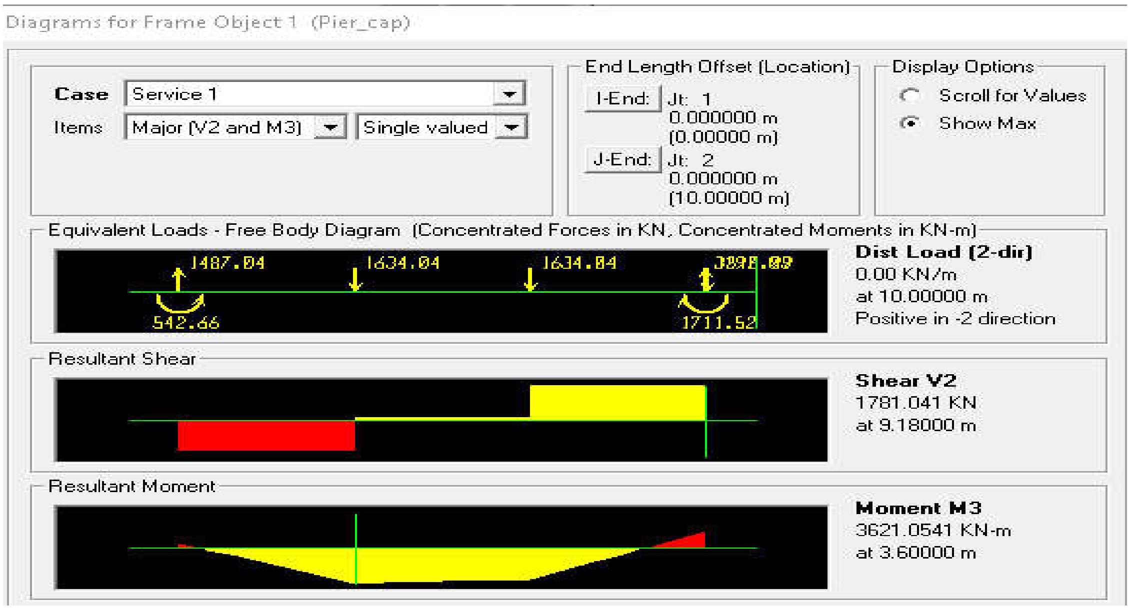


Figure 3.15: The Service I Limit States Moments and Shear on the Pier Cap.

8- Pier Cap Capacity & Ultimate Loading

The procedure to compare the capacity of the section required to satisfy the design moment.

The procedure is the same for both positive and negative moment regions.

Mu Positive = 4609 kN.m

Mu Negative = - 2181 kN.m

Factored resistance:

$$M_R = \phi Mn$$

1. Positive Moment Capacity:

For a rectangular, non-prestressed section:

$$Mn = As.\ fy\ (ds - \frac{a}{2})$$

$$a = \frac{As.fy}{0.85f\prime cb}$$

Assume fs = fy [LRFD 5.7.2.1]

Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

$$M_R = \phi As \cdot fy \{ds- \frac{1}{2}(\frac{As.fy}{0.85f'cb})\}$$

The resistance moment M_R was compared to the applied moment M_U and found that

If $M_R < Mu$ **The section needs to be strengthened by increasing the area of the reinforcement in the positive moment's region.**

2- Negative Moment Capacity:

$$M_R = \phi As \cdot fy \{ds- \frac{1}{2}(\frac{As.fy}{0.85f'cb})\}$$

B- Pier Column Capacity

Maximum axial force at the bottom of the column from the analysis:

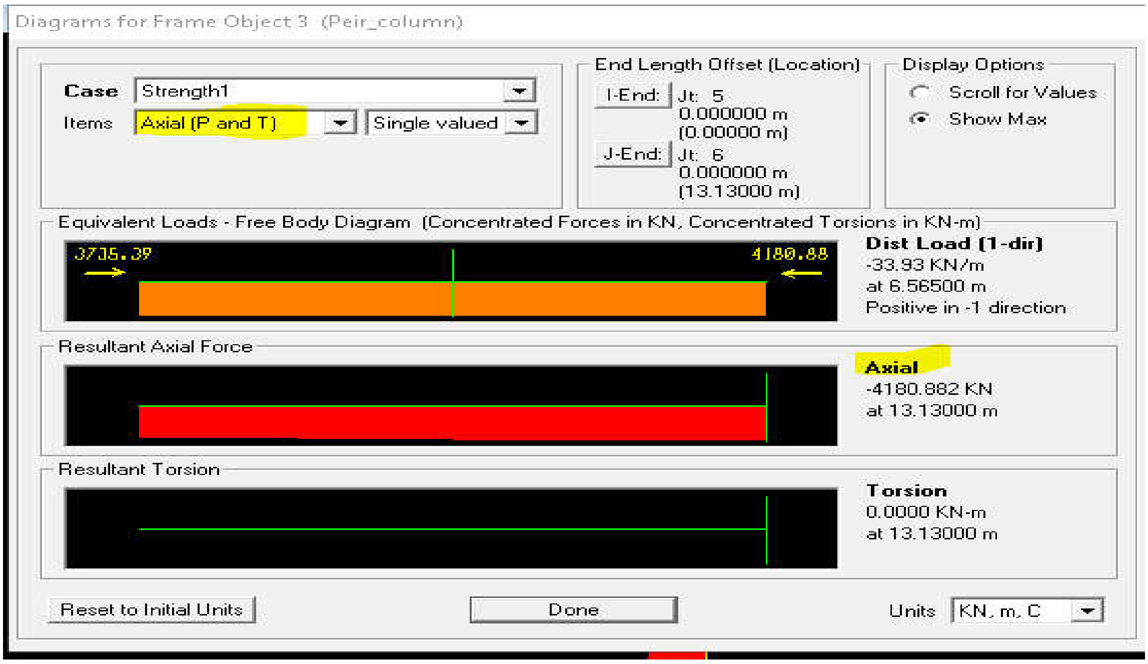


Figure 3.16: Axial Loads on Pier Column.

$P_{max} = 4180$ kN

- **Column Axial Compression ~~Resistance~~**

Pu = -4180 KN

For members with spiral transverse reinforcement, the axial resistance is based on:

$P_r = \phi P_n = \phi\ 0.85P_o = (0.850)\ [0.85f`_c\ (Ag-A_{st}) + A_{st}\ f_y]$

Where:

P_r = factored axial resistance

P_n = nominal axial resistance, with or without flexure

ϕ= resistance factor specified in AASHTO LRFD Design Specifacations 5.5.4.2

P_o = nominal axial resistance of a section at zero eccentricity

f`c = specified strength of concrete at 28 days, unless another age is specified

A_g = gross area of section

A_{st} = total area of main column reinforcement

f_y = specified yield strength of reinforcement

For this pier column:

f`c = 30 Mpa , $A_g = 2.0x10^6$, $A_{st} =$ 9818 mm^2 , f_y = 460 mm^2

C- Shaft Pile Capacity

The pile are drilled shaft socketed in a sandstone and mudstone rocks only one pile under one column.

The soil-pile friction resistance value q_s in the below table are quoted from the Ground Investigation Report prepared by NCEII.

The verification is illustrated in the table below for the piers according to the soil report.

The factored bearing resistance according to AASHTO LRFD Design Specifications article 10.7.3.2:

$Q_R = \phi Q_n = \phi_p Q_p + \phi_s Q_s$

$Q_p = q_p\ A_p$

$Q_s = q_s\ A_S$

Qp = Pile Tip resistance

Q_s= Pile shaft resistance

q_p = Unit tip resistance of pile = 280 Kpa

q_s = Unit shaft resistance of pile = see the table below

A_s = Surface area of pile = Π x 1.6 x L_i (layer thickness)

A_P = Area of tip = 2.01 m^2

ϕ_p , ϕ_s = resistance factors from table 10.5.5-3 = 0.55 & 0.65

CHAPTER IV

ANALYSIS AND DISCUSSION OF RESULTS

CHAPTER FOUR

ANALYSIS AND DISCUSSIONS OF RESULTS

4.1 Introduction

Based on the findings and results of the load rating and analysis for the baily truss members, the existing superstructure configuration is considered not capable of withstanding the AASHTO HL- 93 LRFD Loadings. The analyses and rating shows that the bridge cannot be subjected to a 2-lane loading neither concrete deck also.

Even with the current condition of the posting, the bridge demonstrate low capability for sustaining the loads, particularly the bottom chords.

Bridge strengthening could be considered but it might be costly that dismantling and changing the sizes of chosen panels chords and vertical members are the supports and midspan is probably not practical.

Thinking about alternatives of changing the entire superstructure is an option which is to some extent could be a practical solution to allow the permanent use of the bridge and increase the durability of the structure.

The option of

4.2 Load Rating Results

1- **Design Load Rating Results:**

a- For Selected Critical Truss member's forces and capacities:

After Applying Loads to Calculate selected Critical member's forces and capacities:

- Inventory Level:

  Live Load Factor γ_L = 1.75

Summary of truss members internal forces due to dead and live load & nominal capacity:@ inventory Level

Table (4-1): Baily Truss Forces & Member's Capacities at Inventory Level

Member	**Axial force due to Dead load, kN**	**Axial force due to HL-93 Live Load, kN**	**Nominal Capacity**
Bottom Chord 2C10	**238**	**470**	**433**
Vertical C-08	**-118**	**-172**	**336**
Diagonal Tube 80x60	**67**	**98**	**380**

- Summary of truss members forces due to dead and live load & Nominal Capacity: @ Operation Level

  - Operating Level:

    Live Load Factor γ_L = 1.35

Table (4.2): Baily Truss Forces & Member's Capacities at Operation Level.

Member	Axial force due to Dead load, in kN	Axial force due to HL-93 Live Load, in kN	Nominal Capacity in kN
Bottom Chord 2C10	238	362	432
Vertical C-08	-118	-137	335
Diagonal Tube 80x60	67	75	380

- Summary of floor beam forces due to dead and live load & nominal capacity:@ inventory level

Table (4.3): Floor beam forces & member's capacity.

Member	due to Dead load, kN	due to HL-93 Live Load + IM , kN	Nominal Capacity in kN
Moment @ Mid Span	54	867	2156
Shear @ Supports	28	384	1696

a- Load Rating Factors:

Table (4.4): Summary of rating factors —truss members

Limit State	Member	Design Load Rating (HL-93)		Remarks
		Inventory	Operating	
Strength I	Bottom Chord	0.09	0.12	
	Vertical at Support	0.34	0.45	
	Diagonal	1.05	1.36	

Table (4.5): Summary of rating factors —floor beam.

Limit State		Design Load Rating (HL-93)		Remarks
		Inventory	Operating	
Strength I	Flexure	0.87	1.13	
	Shear	1.57	2.04	

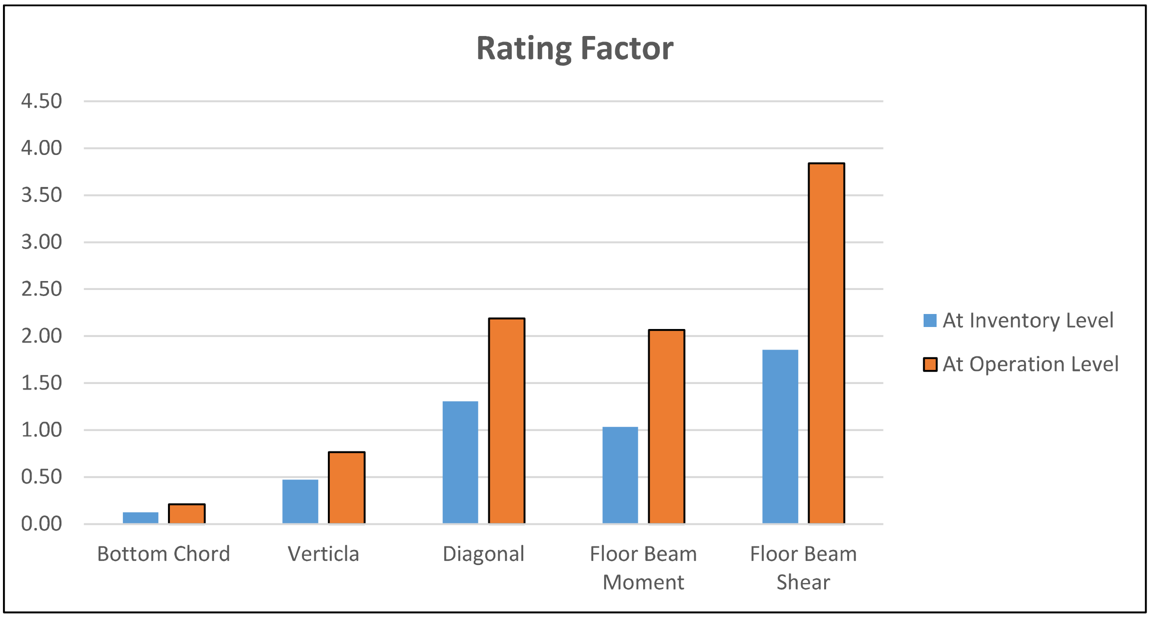


Figure (4.1): Load Rating Factors Chart.

2- Legal Load Rating

Bridges that do not have sufficient capacity under the design-load rating when (R_F<1) shall be load rated for legal loads to establish the need for load posting or strengthening. Load rating for legal loads determines the safe load capacity of a bridge for the AASHTO family of legal loads and state legal loads, using safety and serviceability criteria considered appropriate for evaluation. A single safe load capacity is obtained for a given legal load configuration.

Table (4.6): Type 3 (25 Tons Truck) Dead & Truck Forces Effects.

Sr. No	Panel No	Location	Forces (kN)		
			DL	LL	Total
1	1	Vertical at Support	-118	-55	-173
2	1&13	Diagonal	67	34	101
4	13	Bottom Chord	238	128	366

Table (4.7): Type 3S2 (36 Tons Truck) Dead & Truck Forces Effects.

Sr. No	Panel No	Location	Forces (kN)		
			DL	LL	Total
1	1	Vertical at Support	-118	-72	-190
2	1&13	Diagonal	67	47	111
4	13	Bottom Chord	238	158	397

Table (4.8): Type 3-3 (40 Tons Truck) Dead & Truck Forces Effects.

Sr. No	Panel No	Location	Forces (kN)		
			DL	LL	Total
1	1	Vertical at Support	-118	-77	-195
2	1&13	Diagonal	67	44	114
4	13	Bottom Chord	238	160	398

- **Load Rating Factors:**

Table (4.9): Summary of Legal Rating Factors —Truss Members.

Limit State	Member	AASHTO LEGAL LOADS			Remarks
		Type 3	Type 3S2	Type 3-3	
Strength I	Bottom Chord	0.23	0.18	0.18	
	Vertical at Support	1.03	0.79	0.75	
	Diagonal	2.93	2.24	2.11	

4.3 Upgrading of Superstructure Results

4.3.1 Introduction

Based on the findings and results of the load rating and analysis for the baily truss members, the Existing Super Structure configuration is considered not capable of withstanding the AASHTO HL- 93 LRFD Loadings. The analyses and rating shows that the bridge cannot be subjected to a 2-lane loading neither concrete deck also.

Even with the current condition of the posting, the bridge demonstrate low capability for sustaining the loads, particularly the bottom chords.

Bridge strengthening could be considered but it might be costly that dismantling and changing the sizes of chosen panels chords and vertical members are the supports and midspan is probably not practical.

Thinking about alternatives of changing the entire superstructure is an option which is to some extent could be a practical solution to allow the permanent use of the bridge and increase the durability of the structure.

After the comparison of construction alternatives was completed, a prestressed Bulb T-Girder Concrete system and Steel Plate Girder was identified as two best solutions. The design of the bridge superstructure was completed first by developing a preliminary design, and should be followed by completing a more detailed design later, where the aspects of the preliminary design were adjusted to meet certain requirements. These requirements included moment capacities developed by the loading conditions as well as section properties to accommodate prestressing steel in the primary girders. SAP software was utilized during the design process and helped facilitate adjustment of the design.

The preliminary design of the chosen bridge superstructures will include:

- Preliminary girder design (cross-sectional dimensions)
- Potential girder spacing
- Sketch including primary superstructure components (girders, deck, sidewalks and parapets)

4.3.2 Prestressed Girders Superstructures Results:

1- Design Data:

- Deck Slab:

  Deck Slab Thickness ts = 200mm

Concrete Compressive Strength at 28 days fc` = 40 Mpa

- Precast Prestressed Girder:
- Concrete Strength at Transfer fci = 0.75x40=30 Mpa

Concrete Strength at 28 days fc` =40 Mpa

Concrete Unit Weight = 24KN/m³

Overall Beam Length = 39.624

Design Span = 39.624

- Prestressing Strands:

12.7 dia 7 wire low relaxation strand

Area of Strands = 98.7mm^2

No. of Strands in one cable = 10

No. of cables = 6

Ultimate Strength fpu = 1860 Mpa

Yield Strength fpy = 0.9x1860 = 1674 Mpa

Table 4.10: LRFD stress Limits

Table 5.9.3-1 Stress Limits for Prestressing Tendons.

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars	Tendon Type: Low Relaxation Strand	Tendon Type: Deformed High-Strength Bars
Pretensioning			
Immediately prior to transfer (f_{pbt})	$0.70\ f_{pu}$	$0.75\ f_{pu}$	—
At service limit state after all losses (f_{pe})	$0.80\ f_{py}$	$0.80\ f_{py}$	$0.80\ f_{py}$
Post-Tensioning			
Prior to seating—short-term f_{pbt} may be allowed	$0.90\ f_{py}$	$0.90\ f_{py}$	$0.90\ f_{py}$
At anchorages and couplers immediately after anchor set	$0.70\ f_{pu}$	$0.70\ f_{pu}$	$0.70\ f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70\ f_{pu}$	$0.74\ f_{pu}$	$0.70\ f_{pu}$
At service limit state after losses (f_{pe})	$0.80\ f_{py}$	$0.80\ f_{py}$	$0.80\ f_{py}$

2- Cross-Section Details

Although it was decided to use interior and exterior girders with the same cross-sectional dimensions, through independent analysis of the interior and exterior girders it was determined that the interior girders would meet the proper strength requirements. However, for constructability it was decided that the height of the interior girders should be same to the exterior girders. This allowed for a more conservative design of the girders as well as resulted in the need for only one prestressing design.

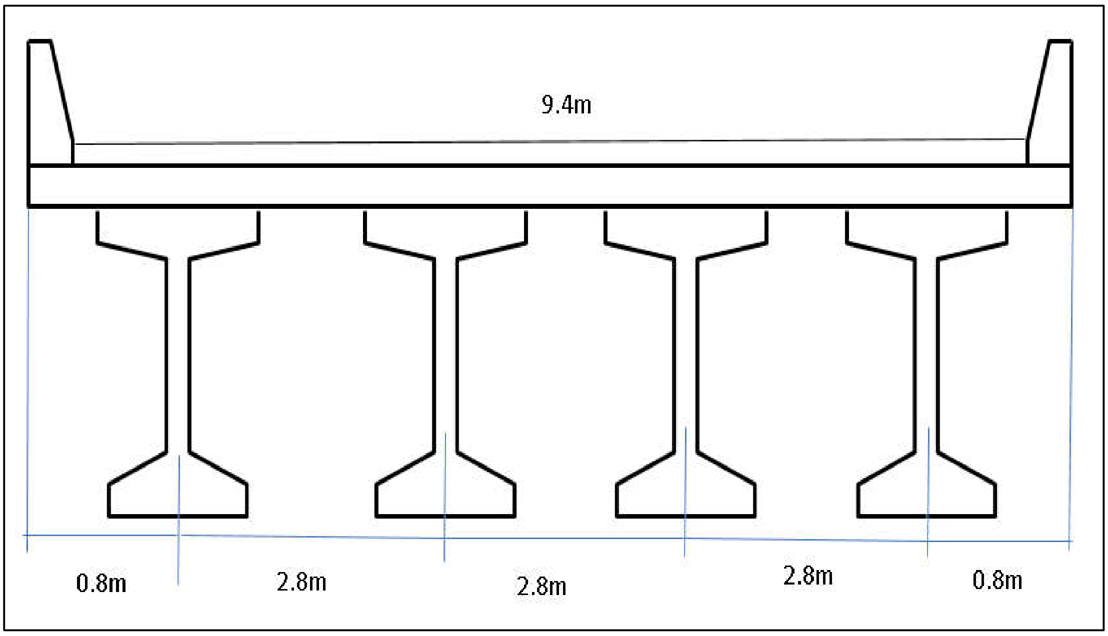


Figure 4.3: Deck Section

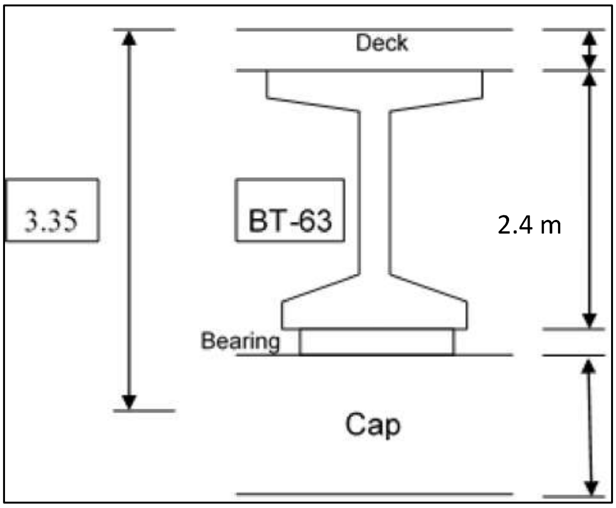


Figure 4.4: Elevation Section

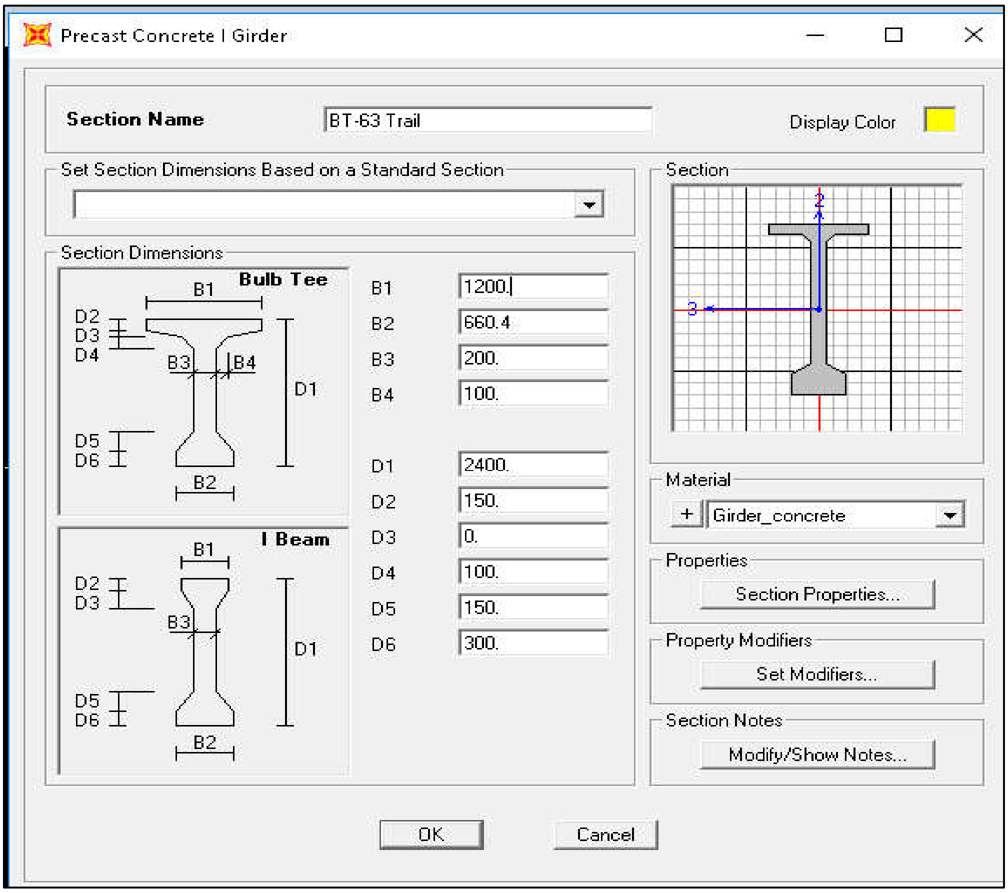


Figure 4.5: Girder Final Section Properties.

➢ Non-Composite Section Properties

Here is the summary of the section properties, the detailed will be found in Appendix C

Table 4.11: Non-Composite Section Properties.

I	$Y_{c.g}$	y_{bot}	y_{top}	S_{bot}	S_{top}
5.992E+11	1219.146	1219.146	1180.854	4.915E+08	5.075E+08

S= Section modulus for the extreme top and bottom fiber

- **Composite Section Properties**

- Effective Flange Width, b_{eff}:

For an interior beam, b_{eff} is the lesser of:

5- $L_{eff}/4 = 39.62/4$ = 9.905
6- 12ts+bf/2 = 12/.2+0.428/2 = 2.933
7- S= Average distance between girders = 2.800

- b_{eff} = 2.800 m = 2800 mm

Modular ratio n: $n = Es/Eg = \frac{4800\sqrt{30}}{4800\sqrt{40}} = 0.87$

Eg= girder modulus of elasticity

EC= Concrete slab modulus of elasticity

Transformed flange width = b_{eff} x n = 2800x0.87 = 2436 mm

Transformed flange Area = b_{eff} x n x t_s =2436 x 200 = 487200 mm^2

Table 4.12: Composite Section Properties.

I	$Y_{c.g}$	y_{bot}	y_{top}	S_{bot}	S_{top}
9.098E+11	1703.710534	1703.710534	96.28946592	5.340E+08	9.449E+09

3- Dead Load Calculations

• The maximum positive dead-load moments occurs at mid spans.

• The maximum negative dead-load moments occurs over the center of supports.

Table 4.13: The Maximum Positive and Negative Dead-Load Moments.

	Max (+) Moment @Mid- Spans (kN.m)	Max (-) Moment @ Supports (kN.m)

Girder Concrete:	3827.73	-2199
Deck:	2637	-2655
Railing:	537.7	-543.3
Total:	7002.4	-5397.3
Wearing:	632	-638.5

4.14: The Dead-Load Shear

	Max (+) Shear **@Supports (kN)**
Concrete Girder :	285.8
Deck:	333.5
Railing:	68
Total:	687.3
Wearing:	80

The dead load moment applied to the superstructure was calculated in two steps. A moment was first calculated based on the dead load of the railings, girder weight and concrete deck and was constant for each of the evaluated cross-sections. This moment was calculated based on the assumption that each girder would support an equal portion of the load. Additionally, a separate moment was calculated due to the wearing surface for the final design.

4- Live Load Calculations

- **Distribution Factor for Bending Moment:**

For the simplicity and the ease of calculations, consider the following values for the entire girders:

For the Moments DFM = 0.62

For the Shear forces DFV = 1

Table 4.15 – Moments Distribution Factors for Live Load Conditions

Interior Girder	Value
Distribution Factor	0.62
Rear Axle Load (KN)	88.23
Front Axle Load (KN)	22.07
Design Lane Load (k/m)	5.79

- **Distribution Factor for Shear Force:**

Table 4.16 – Shear Distribution Factors for Live Load Conditions

Interior Girder	Value
Distribution Factor	1
Rear Axle Load (kN)	142.3
Front Axle Load (kN)	35.6
Design Lane Load (k/m)	9.34

The live loads were applied to girders using the SAP software. The design lane load was applied longitudinally to the girders as a distributed load and the design truck axle loads were applied as a point loads. The loads of the design truck were then moved along the length of the girder, and the reaction forces within the girder were calculated for each location.

It was determined that the maximum moment the girder experienced occurred in the center of the beam when the rear axles of the truck were spaced 14` (4.3 m) apart. The resulting maximum moments for the interior and exterior girders are shown in table 4.17 and 4.18.

Table 4.17: Maximum Live Load Positive and Negative Moments

	Max (+) Moment @Mid- Spans (kN.m)	Max (-) Moment @ Supports (kN.m)
HL-93 K (Truck Loading)	3402	-2705.7

Table 4.18 - Maximum Live Load Shear for Interior and Exterior Girders

	Max Shear **(kN)**
HL-93 K (Truck Loading)	390

5- Load Combinations :

Factored Shear and Moment Envelopes:

The following load combinations were considered:

- **Strength I:**
  - Positive Moment :

    1.25 M DL. + 1.5 M DW + 1.75 MLL

    = **15654** kN.m
  - Negative Moment:

    1.25 x5025.7 + 1.5x 616.7 + 1.75x 4238.7 = **12439** kN.m
  - Shear :

    1.25 M DL. + 1.5 M DW + 1.75 MLL

= **<u>1661.62</u>** kN

- **Service I:**

Load combination relating to the normal operational use of the bridge with a 55 MPH wind and all loads taken at their nominal values.

"Applicable for maximum compressive stresses in beam ONLY. For tension, see Service III."

- Positive Moment:

1.3(LL + IM) + 1.0DC1 + 1.0DC2 + 1.0DW

= 5644 + 632 + 1.3x2121 = **<u>9350</u>** kN.m

Negative Moment:

= 5025.7 + 616.7 + 1.3x4238.7 = **<u>8751.8</u>** kN.m

- **Service III:**

Load combination for longitudinal analysis relating only to tension in prestressed concrete structures with the objective of crack control.

"Applicable for maximum tension at midspan ONLY. For compression, see Service I."

Service3 = 1.0DC + 1.0DW + 0.8LL +1.0(CR, SH) = **<u>8699.6</u>** kN.m

6- Prestressed Steel Design Results

From Table S5.9.4.1.2-1 of AASHTO LRFD, the stress limit in areas with bonded reinforcement sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section.

The strength parameters were based on a specified concrete compressive strength of 40 Mpa. The results of the design of the girders and prestressed steel stresses are shown in table below.

Table 4.20: Schedule of Stresses.

SN	Description	Axial Forces	Bending Moments	Stress @ Girder Bottom	Stress @ Girder Top	Stress @ Deck Slab Top
	At Transfer Stresses Check					
1	Moment Due to self-weight of Girder		3.83E+09	-7.79	-7.54	
2	Axial Force due to Prestressing	8.26E+06		10.32	-10.32	
3	Moments due to Prestressing Moment		9.09E+09	18.49	17.91	
4	Loss of Prestressing due to Friction 15%			-4.32	-1.20	
5	Loss of Prestressing due to Elastic Shorting 0.3%			-0.86	-0.20	
	Sub Total 1 Allowable Compression 0.6fci = 18 N/mm^2 Allowable Tension $0.63\sqrt{fci}$ = 3.5 N/mm^2			15.83 O.K	-1.35 O.K	
	At Service Limit Stresses Check					
1	Moment Due to Deck		2.64E+09	5.36	5.20	
2	Moment Due to Wearing Surface		6.32E+08	1.29	1.25	0.07
3	Moment Due to Railing		5.38E+08	1.09	1.06	0.06
4	Moment Due to Live Load		3.40E+09	6.92	6.70	0.36
	Total Stresses at Service Load Condition Allowable Compression 045 f`c = 18 $N/mm^2$ Allowable Tension $0.5\sqrt{f`c}$ = 3.5			14.67	14.21	0.48

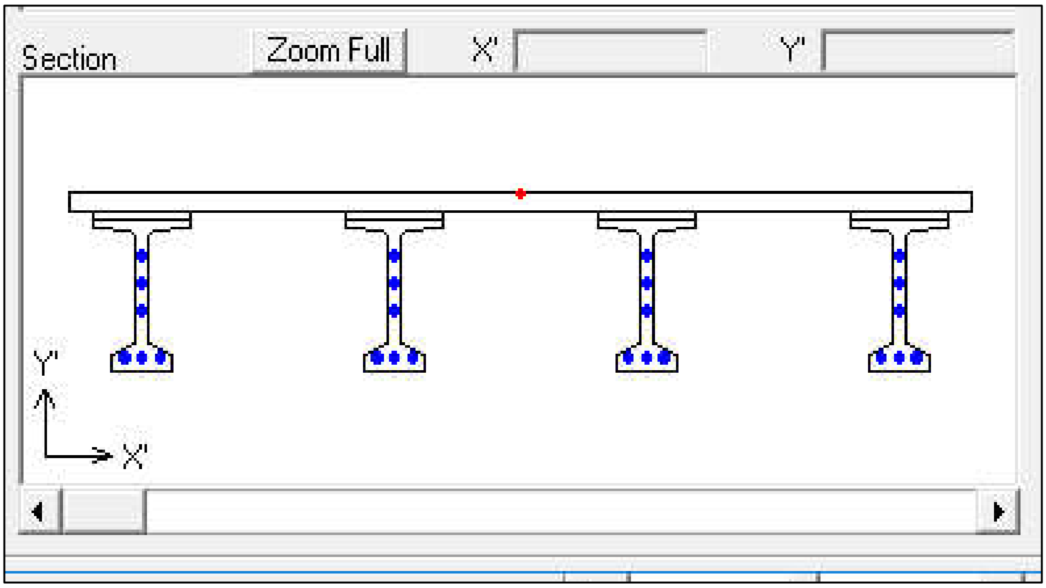


Figure 4.6- Prestressed Steel Section

This section has presented the prestressed design of the girder. Some details of design could have been considered and explored further, but were not part of the design due to the limitation of this preliminary design like the detailing of the stress permissible zone, tendons profiles, shear reinforcement etc.

7- Calculation of Girder's Moment Capacity

The girder section moment capacity was calculated and compared to the maximum factored applied moment from the above load combinations to check the adequacy of the section to sustain the loads.

- **The factored flexural resistance**

Mr = **20941** kN.m > Mu = **15354** kN.m @ Strength limit state

The section is Adequate

4.3.3 Steel Plate Girders Bridge Results

A steel Plate I-girder bridge design was the second comparative design considered in this thesis. Steel I-girder bridges are supported by one or more welded steel plates. The cross section consists of web plates connected to a bottom and top flange plates.

4.3.3.1 Cross-Section Details

Although it was decided to use interior and exterior girders with the same cross-sectional dimensions, through independent analysis of the interior and exterior girders it was determined that the interior girders would meet the proper strength requirements. However, for constructability it was decided that the height of the interior girders should be same to the exterior girders. This allowed for a more conservative design of the girders as well as resulted in the need for only one prestressing design.

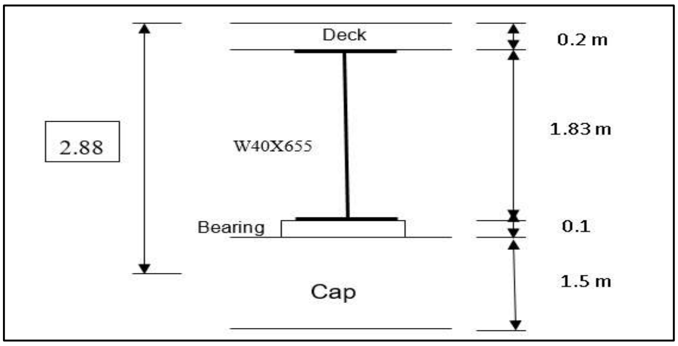


Figure 4.7: Deck Section

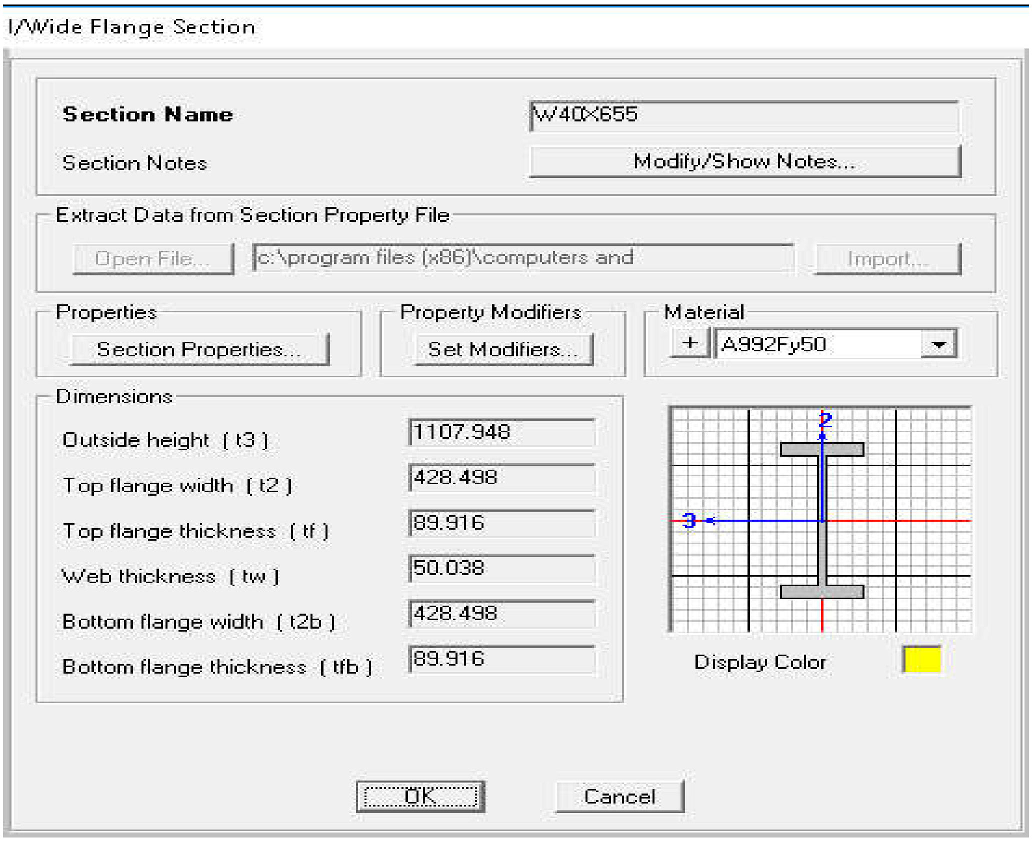


Figure 4.8 - Preliminary Design Trail Girder.

- **Section Properties**

  For non-composite section:

Table 4.21: Non Composite Girder Section Properties.

$Y_{c.g}$	y_{bot}	y_{top}	S_{bot}	S_{top}
553.9771	553.9771	553.971	4.283E+07	4.283E+07

S= Section modulus for the extreme top and bottom fiber

3.1: Effective Flange Width, b_{eff}:

For an interior beam, b_{eff} is the lesser of:

8- $L_{eff}/4 = 39.62/4$ $= 9.905$

9- 12ts+bf/2 = 12/.2+0.428/2 = 2.6

10- S= Average distance between girders = 2.8

11-

- b_{eff} = 2.6 m = 2600 mm

Calculate for Short-Term Composite (n = 8)

ES= Steel girder modulus of elasticity

EC= Concrete slab modulus of elasticity

Transformed flange width = b_{eff}/n = 2600/8 = 325 mm

Transformed flange Area = 325 x 200 = 65000 mm^2

For composite section:

Table 4.22: Composite Girder Section Properties.

$Y_{c.g}$	y_{bot}	y_{top}	$y_{topslab}$	S_{bot}	S_{top}	$S_{topslab}$
794.99	794.99	312.96	512.96	6.349E+07	1.613E+08	9.840E+07

4.3.3.2 Dead Load Calculations

The following Dead Load results were obtained from the SAP 2D analysis:

- The maximum positive live-load moments occur at mid spans.
- The maximum negative live-load moments occur over the center of supports.

Table 4.23 – Dead Loads Maximum Moments.

	Max (+) Moment **@Mid- Spans (kN.m)**	Max (-) Moment **@ Supports (kN.m)**
Girder Steel:	2470	-1868
Deck:	2637	-2633
Railing:	537.7	-524.7

Total:	5644	-5025.7
Wearing:	632	-616.7

Table 4.24 – Dead Loads Maximum Shear.

	Max Shear **@Supports (kN)**
Girder Steel:	245.5
Deck:	262.2
Railing:	53.4
Total:	561
Wearing:	62.8

4.3.3.3 Live Load Calculations

It was determined that the maximum moment the girder experienced occurred in the center of the beam when the rear axles of the truck were spaced 14 feet (4.3m) apart. The resulting maximum moments for the interior and exterior girders are shown in tables below.

Table 4.25 – Maximum Live Load Moments

	Max (+) Moment **@Mid- Spans (kN.m)**	Max (-) Moment **@ Supports (kN.m)**
HL-93 K (Truck Loading)	2121	-4238.7

Table 4.26 - Maximum Live Load Shear for Interior and Exterior Girders

Max Shear (kN)	
HL-93 K (Truck Loading)	539.4

As has been discussed previously, the determination of these moments required the use of an iterative process. Changes in the size and spacing of the girders also changed the distribution factors, and ultimately the applied loads. As a result, once a new girder size was selected, the loads and girder cross-section were input into the software to determine the new maximum moment. If this new moment required a new girder size, the process was repeated until an adequate girder size and spacing was obtained. The moments shown in Table, are those that were obtained for the final girder size. This iterative process was aided by the use of optimization technique in the CSi bridge evaluation version software in which the ratio of the demand moment or shear (Ultimate) to the section capacity calculated to be less than 1.

4.3.3.4 Factored Shear And Moment Envelopes

- **The following load combinations were considered**

Strength I:

- Positive Moment :

1.25 M DL. + 1.5 M DW + 1.75 MLL

=1.25x5644 + 1.5x632+ 1.75x2121 = **11714.75** kN.m

- Negative Moment:

1.25 x5025.7 + 1.5x 616.7 + 1.75x 4238.7 = **14624** kN.m

Service II:

- Positive Moment:

1.3(LL + IM) + 1.0DC1 + 1.0DC2 + 1.0DW

= 5644 + 632 + 1.3x2121 = **9033.3** kN.m

Negative Moment:

$= 5025.7 + 616.7 + 1.3x4238.7 \qquad = \underline{\mathbf{11152.7}}$ kN.m

4.3.3.5 Shear And Moment Stresses Envelopes

Combined shears, and flexural stresses can be computed at the controlling locations. A summary of those combined load effects for an interior beam is presented in the following three tables, summarizing the results obtained using the procedures demonstrated in the above computations, the resulted stresses will be compared to the yield strength of the steel girder of **345 Mpa where the permissible stresses is 0.9*345.**

a- Maximum stress in the top of the girder due to positive moment (located at 0.5L) for the Strength I Limit State is computed as follows:

- Non-composite dead load :

  $$fDL\ \text{non-comp DL} = \frac{MDL}{Stop} = \frac{5106x10^6}{4.283x10^7} = 119.3\ \text{N/mm}^2$$

- Railing Composite DL moment:

  Mparapet = 537.7 KN.m, S topgdr = 1.613 x10^8 mm^3

  $$f_{Par} = \frac{MPar}{Stop} = \frac{537.7x10^6}{1.613x10^8} = 3.33\ N/mm2$$

- Wearing composite DL:

  M_{DW} = 632 KN.m

  $$fDW = \frac{MDW}{Stop} = \frac{632x10^6}{1.613x10^8} = 3.92\ N/mm2$$

- Live Load: HL-93+IM :

  M_{LL}= 2121 KN.m

  $$fLL = \frac{MLL}{Stop} = \frac{2121x10^6}{1.613x10^8} = 13.14\ N/mm2$$

- Calculation of top stresses for strength limit state :

1.25 f DL. + 1.5 f DW + 1.75 fLL = 182.12 N/mm^2 < 345 MPa O.K.

Similarly compute for the bottom and then the negative moment.

- Calculation of bottom stresses for strength limit state :

For non-composite:

S_{bottom} = 6.349E+07 mm³

fDL=79.16 , Fdw = 9.7 , fLL = 66.75

1.25 f DL. + 1.5 f DW + 1.75 fLL = <u>230.31</u> N/mm^2 < 345 Mpa O.k

b- Maximum stress in the top of the girder due to Negative moment (located at 0.5L) for the Strength I Limit State is computed as follows:

• Calculation of top stresses for strength limit state:

For non-composite:

S_{top} (Non-composite) = 4.283E+07 mm³

S_{top} (Composite) = 1.613E+08 mm³

- Compute for the top negative moment:

  fDL=119.3 , fpar = 8.47 , Fdw = 10 , fLL = 33.4

1.25 f DL. + 1.5 f DW + 1.75 fLL= 266.34 N/mm2 < 345 MPa .

By the same, the service limit could be checked, and obviously it will be within the permissible limits.

The yield strength of the steel girder is **345 MPa**, so the section is adequate.

4.3.4 Concrete Deck Design Results

- **Analysis Model for Dead Loads**

SAP-2D software was used to produce a maximum negative and positive moment for the deck using the dead loads calculated above tables below shows the calculations results.

The deck is modeled as a beam supported at the girder locations.

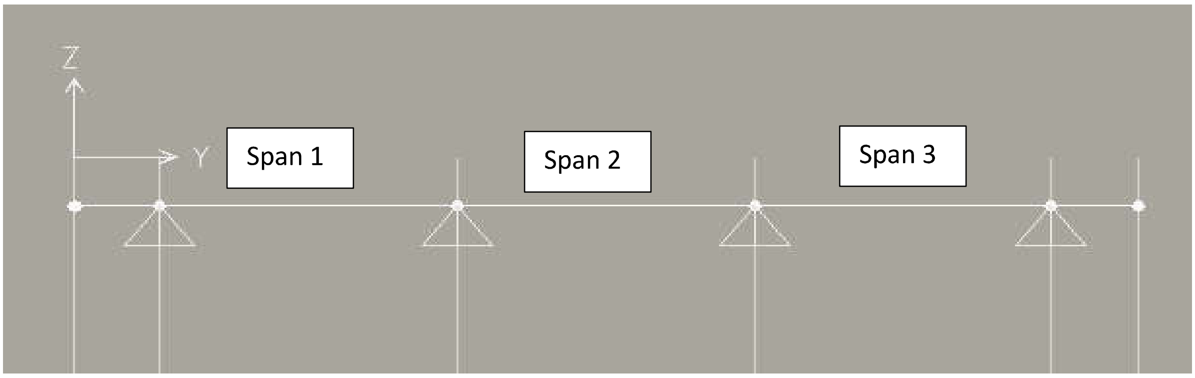


Figure 4.9: Deck Slab Transverse Section Model in SAP.

❖ **Design Moments for Deck Slab**

Table 4.27: Deck Slab Maximum +ve Moment.

span	+Ve moment
1	2.29
2	1.18
3	2.21

Table 4.28: Deck Slab Maximum -ve Moment.

Support	-Ve moment
1	1.5
2	3.47
3	3.4
4	1.6

Take:

M positive = 2.29 kN.m , M negative = 3.47 kN.m

- **Design Moments for Wearing Surface**

Table 4.29: Wearing Surface Maximum +ve Moment

span	+Ve moment
1	1.28
2	0.12
3	0.21

Table 4.30 : Wearing Surface Maximum +ve Moment

Support	-Ve moment
1	1.88
2	1.98
3	1.98
4	1.88

Take :

M positive = 1.28 kN.m , M negative = 1.98 kN.m

- **Limit State Moments:**

The service and strength limit states are used to design the section

1- Service I Limit State

- Positive Service I Moment :

M DL. + M DW + MLL. = 2.29+1.28+8.1 = 11.67 kN.m/m

- Negative Service I Moment:

$$M\,DL. + M\,DW + MLL. = 1.98+3.47+4.9 = 10.35 \text{ kN.m/m}$$

2- Strength I Limit State:

- Positive Service I Moment :

  1.25 M DL. + 1.5 M DW + 1.75 MLL = 18.97 kN.m/m

- Negative Service I Moment:

  1.25 M DL. + 1.5 M DW + 1.75 MLL = 16.26 kN.m/m

❖ Positive and Negative Moments for Design the values of Strength I governs ,Use:

Mpositve = 18.97 kN.m/m

Mnegaitve = 16.26 kN.m/m

To calculate reinforcement:

$$Mr = \phi Mn = \phi\, As.fs \left\{ds - \frac{1}{2}\left(\frac{As.fy}{0.85 f'cb}\right)\right\}$$

Use:

Mr = Mpositve for reinforcement in positive regions,

= Mnegaitve for reinforcement in negative regions,

4.3.5 Bearing Pad Design Results

- **Selected Bearing Type and Properties**

  Steel-reinforced elastomeric bearing was selected.

  The bearing properties are obtained from the Specifications, as well as from past experience. The following preliminary bearing properties were selected from a worked example:

➢ Steel reinforcement thickness: $h_{\mathbf{reinf}}$ = 3mm

- Number of steel reinforcement layers: $N_{stlayers} = 9$
- Elastomer internal layer thickness: $hr_{internal} =$ 9.5mm
- Elastomer cover thickness: $hr_{cover} = 6.3$ mm
- Pad width (bridge transverse direction): $W_{pad} =$ 380mm
- Pad length (bridge longitudinal direction): $L_{pad} =$ ~~355~~ 380mm
- Materials Properties:
  - Elastomer shear modulus: G = 0.66 Mpa S Table 14.7.5.2-1
  - Elastomer hardness: HshoreA = 50 S14.7.5.2
  - Creep deflection Cd= 0.25
  - Steel reinforcement yield Strength = 345 MPa

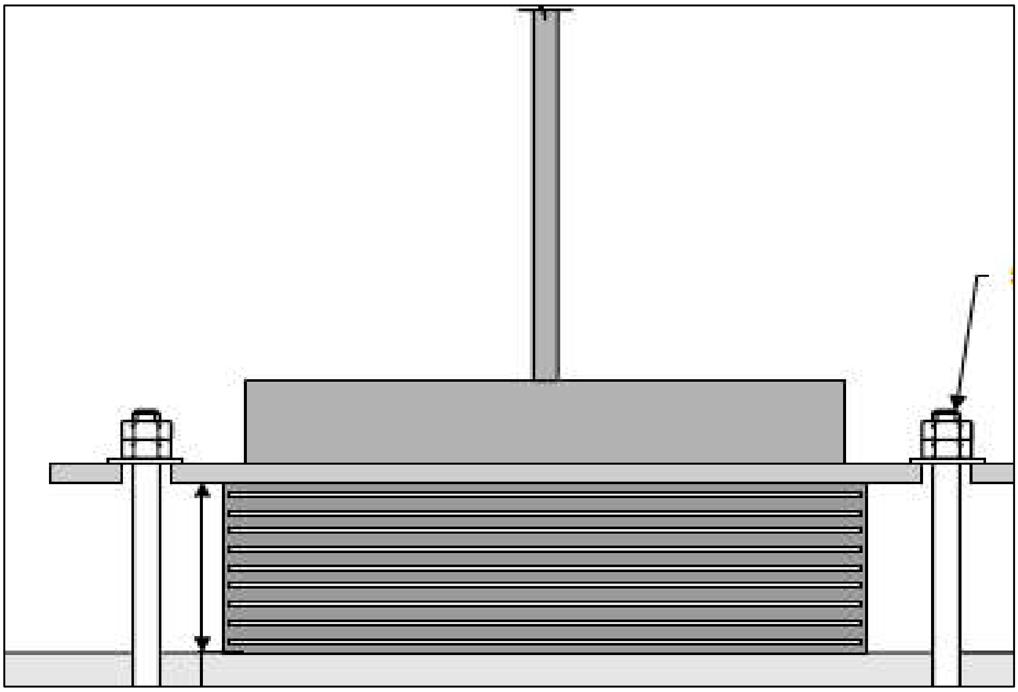


Figure 4.10: Beating Pad.

4.3.6 Substructure Design Verification Results

4.3.6.1 Pier Cap

- **Summary of Results :**

Table 4.31: Summary of the Factored Limit States Loads on the Pier Cap

	STRENGTH I		STRENGTH V		SERVICE I	
	Positive	Negative	Positive	Negative	Positive	Negative
MOMENT	4609.46	1555.1	4644.85	2181.18	3621.05	1741.84
SHEAR	2207.6	2188.01	2279.81	1946.1	1781.04	1487.04

4.3.6.2 Pier Cap Capacity & Ultimate Loading

`

The procedure to compare the capacity of the section required to satisfy the design moment.

The procedure is the same for both positive and negative moment regions.

Mu Positive = 4609 kN.m

Mu Negative = - 2181 kN.m

Factored resistance:

$$Mr = \phi Mn$$

i. Positive Moment Capacity:

For a rectangular, non-prestressed section:

$$Mn = As.fs(ds - \frac{a}{2})$$

$$a = \frac{As.fy}{0.85f\prime cb}$$

Assume fs = fy [LRFD 5.7.2.1]

Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

$$Mr = \phi\ As.fs\{ds- \frac{1}{2}(\frac{As.fy}{0.85f'cb})\}$$

f'c=30 Mpa , fy= 460 N/mm^2 , h_{cap}=1500mm , $\phi = 0.9$

bcap= 1800mm , AS= 6872 mm^2 (from existing drawings)

ds=h-cover- $\frac{\emptyset}{2}$ - Tie= 1500-60- $\frac{\emptyset 25}{2}$ - 12= 1415.5

Mr=2.85x10^6 (1413.25) = 4027.77 KN.m < Mu = 4644.85 KN.m

The section needs to be strengthened by increasing the area of the reinforcement in the positive moment's region.

- **For Setit Bridge :**

  As=4909, b=1500,

  M_R= 2227.37 < Mu,

  The section needs to be strengthened.

ii. Negative Moment Capacity:

$$Mr = \phi\ As.fs\{ds- \frac{1}{2}(\frac{As.fy}{0.85f'cb})\}$$

For Upper Atbara Bridge

f'c=30 Mpa, fy= 460 N/mm^2 , h_{cap}=1500mm , $\phi = 0.9$

b_{cap}= 1800mm , AS= 6872 mm^2 (from existing drawings)

ds=h-cover- $\frac{\emptyset}{2}$ - Tie= 1500-60- $\frac{\emptyset 25}{2}$ - 12= 1415.5

Mr=2.85x10^6 (1413.25) = 4027.77 KN.m > Mu = 2181 kN.m

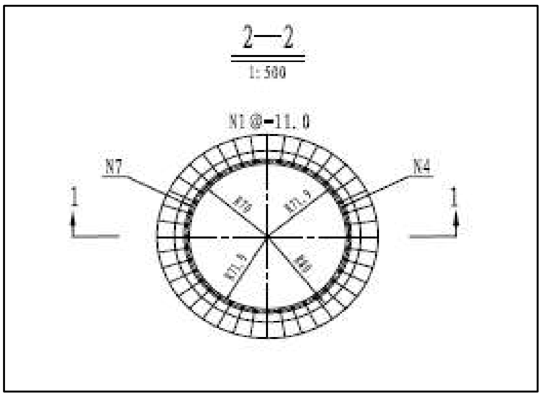
- **For Setit Bridge :**

  As=4909, b=1500,

M_R= 2227.37 > Mu, **O.K**

4.3.6.3 Pier Column Capacity:

Figure 4.11: Pier Column Plan Section



4.3.6.4 Column Axial Compression Resistance:

Pu = -4180 kN

For members with spiral transverse reinforcement, the axial resistance is

based on:

$P_r = \phi P_n = \phi\ 0.85P_o = (0.850)\ [0.85f`_c\ (Ag - A_{st}) + A_{st}\ f_y]$

For this pier column:

$f`c = 30$ Mpa , $A_g = 2.0 \times 10^6$, $A_{st} = 9818\ mm^2$, $f_y = 460\ mm^2$

$P_r = 0.85[(0.85 \times 30\ (2.0 \times 10^6 - 9818) + 9818 \times 460] =$ **55265.9** kN > Pu -4180 **O.K**

4.3.6.5 Shaft Pile Capacity:

- **Pile Capacity Calculations:**

Table 4.32: Summary of the Design Calculations for the Shaft Pile

Pier NO.				1# Pier	2# Pier	3# Pier
P（kN）Max. Vertical Force				4180	4180	4180
Difference between pile self-weight and replaced soil weight ΔG（kN）				465.66	465.66	465.66
pile loading: P+ΔG （kN）				4180	4180	4180
Top Pile Level （m）				4645	4645	4645
Depth and Parameter of Soil	Soil		qs (kPa)			
	1	Backfill	0	9.98	9.98	9.98
	2	Medium Sand	40			4.38
	3	Gravel	140	5.25	9.89	
	4	Wholly weathered Mudstone	80	5.40	2.25	7.20
	5	Medium weathered mudstone	80	2.20		
	6	Medium weathered sandstone	85	9.90	16.48	17.04
	7	Medium weathered mudstone	85	2.90		
	8	Medium weathered sandstone	90	2.97		
	Total Length （m）			38.600	38.600	38.600
	Total Friction Resistance			7456	7456	8194
	Pile Tip Resistance			365.8	365.8	365.8
	Total Pile Capacity			7821.74	7821.74	8559.79

4.3.7 Discussions:

i. Load Rating Results

According to legal rating results we can observe that:

- The Diagonal Member is adequate.
- The vertical member just RF passed the type 3 (25 Tons Truck) Legal Load.
- The Chord member does not satisfy any of rating vehicles and the RF falls below 0.3.

The results of this investigation and the detailed evaluation of the case study bridge have demonstrated a number of bridge actual situations for load rating of truss members , while the Chord members exhibit low rating factors RF< 1 for all load rating conditions a number of apparent structural deficiencies, are seen at the site like the deflection and deteriorations.
However, in other member's cases, only the design load rating conditions are not satisfied, but some of members satisfied the legal load rating RF>1 condition.
It's possible to think about strengthening or replacing members but it might be more feasible to look forward and upgrade the bridge by replace the whole super structure, this option was investigated in which the bridge was modelled and alternatives of other superstructures introduced with verification for the adequacy load capacity of the substructure components.

ii. Upgrading Results

Alternative of replacing superstructure in was introduced, and a cross-section dimensions of the concrete girders and steel girders were chosen and checked for the capacity against the strength and service limits stresses and moments. The most influential part of the superstructure design was the effect of the live loads on the superstructure.
SAP software used to facilitate the rating and design process. Also using excel for the selection of the suitable section was useful. Once the selected cross-section updated, design dead applied loads within the software could easily be changed.

CHAPTER V

RECOMMENDATIONS AND CONCLUSION

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

1- Introduction

The Upper Atbara and Setit River Bridges needs to be rated in order to strengthen the superstructure or replaced to satisfy the requirements of the safely usage for long time was discussed through this research and some results were obtained after applying the load rating principles on the bridges.

2- Conclusion

a) The load rating for Upper Atbara and Setit Bridges Baily superstructure was done and the evaluation shows that the rating results shows that the existing Baily truss is not adequate enough to sustain the two lanes loading and also the option of concrete deck as well. The load effect is Larger than members capacities , when applying the full design load the rating factor RF was less than one, especially the bottom chords which was not pass the inventory and legal rating at all and this explains the observed deflection of the bridge on site now.

b) Preliminary design and selection process was developed to determine concrete and steel girders were performed. Later there will be a chance to choose between the steel or concrete girders based on which is more ethical, economical, and sustainable and the capacity of the available contractor.

c) Alternative of replacing steel truss superstructure by Concrete prestressed girder, or steel girder. A cross-section dimensions of the concrete girders and steel girders were chosen and checked for the capacity against the strength and service limits stresses and moments. Moment and shear distribution factor equations were used for determination of the live load effects on a single girder. However, these equations rely on the spacing and depth of the girders, which was under investigation during the design process. As a result, the distribution factors and live loads were recalculated each time a satisfactory girder size was determined, then cross-section dimensions were re-checked to determine if they were still adequate.

The utilization of SAP software to facilitate the rating and design process. Also using excel for the selection of the suitable section was useful. Once the selected cross-section updated, design dead applied loads within the software could easily be changed.

3- Recommendations

To sustain the use of Upper Atbara and setit bridges it's recommended that:

- To replace the superstructure with new durable one taking into account the implementation of this change in a short time, so that the steel girder superstructure is preferred.
- It is also recommended that the superstructure be prepared in parallel to the removal of the old Baily truss and the strengthening of the substructure weather if it's going to be manufactured or casted in-situ.
- It's recommended that to strengthen the x-head Beam by increasing the size and adding additional reinforcement at the bottom for the positive moment, the option of the materials to use might be a normal concrete or fiber as well.
- Have a complete plan for the operation and maintenance of the bridge to increase the time life of the usage.
- Control the traffic at the entrance to avoid any overweight as this bridges linking the road to Ethiopia and many heavy trucks are passing through.

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Appendix A : **Previous Design:**

The dam Complex of Upper Atbara Project includes two bridges on Upper Atbara River and Settit River, which are tributaries of the Nile River in the republic of the Sudan. The total length of the bridges is 460m.

1. **Bridge overall layout:**

a. Bridge spans : Upper Atbara River Bridge is 5×40m. Bridge Total length is 200m, bridge longitudinal slope is 1.5% ; Settit River Bridge is (30m, 5×40m, 30m) Bridge length is 260m, bridge longitudinal slope 0%.

Table A1: Upper Atbara River Bridge Components

Portion	Bridge Elements	No.	Remark
Foundation	Pier cast-in-situ Pile	8	ϕ1.6m
	Abutment cast-in-situ Pile	4	ϕ1.4m
Substructure	Main Pier Pillar	8	ϕ1.6m column pier
	Main Pier Cap	4	
	Abutment Cap	2	
Superstructure	span	5	
	Bridge Deck	664.2 (t)	

Table A2: Setit River Bridge Components

Portion	Bridge Elements	No.	Remark
Foundation	Pier cast-in-situ Pile	12	ϕ1.4m
	Abutment cast-in-situ Pile	4	ϕ1.4m

substructure	Main Pier	12	ф1.2m column pier
	Main Pier Cap	6	
	Abutment Cap	2	
superstructure	span	7	
	Bridge Deck	1951(t)	

2. Materials properties：

a. Concrete Properties:

Table A3: Concrete Properties

Concrete Strength Grade	C30

b. Reinforcement Steel:

Table A4: Reinforcement Steel properties

Steel Type		R235	HRB335
Standard Value of Tensile Strength (fsk)	(MPa)	235	335

c. Structural Steel：

China-made Q345B steel is used for structural steel, complying with the material code of GB/T 1591-2008. The physical and chemical properties are as follows:

Table A5 Structural Steel Physical property

Thickness	Tensile Strength （MN/m2）	Yielding Strength （MN/m2）	Elongation Ratio （δ5）

＜16mm	470~630	345	20%
16~40mm	470~630	335	20%

3. Dead Load Calculations:

a. Upper Atbara Bridge:

The superstructure of Upper Atbara River Bridge is a continuous bailey steel bridge of 5x40m, with calculating span of 39.624m. Bailey steel bridge has total length of 198.12m and total weight of 664156.8 kg

b. Setit River Bridge

The superstructure of Setit River Bridge is a continuous bailey steel bridge of two units; one unit is 30m+3x40m and the other is 2x40m+30m, with calculating span of 39.624m for span of 40m and 30.48m for span of 30m

4. Live Load Calculations:

The live load is calculated as per HL 93 load in AASHTO, which comprises the following four items:

The designer has compared load effect between AASHTO and Chinese Design Code, the Class-II Live Load in Chinese Design Code used as its equivalent to AASHTO LRFD loading specifications.

Dead Load + Live Load Calculating Results (Unit：KN, m)

5. Calculation of Bailey Components:

a- Calculation of Bailey Panels:

The Designer has considered that the bailey truss is made of standard bailey panels, connected by pins. All the forces are transferred by pins. In the calculation, for the sake of simplification, the bailey panel can be defined as an internally statically-determinate structure.

According to the calculations above, the designer stated that most unfavorable case would be in Upper Atbara River Bridge under the load of HL-93, for one bailey truss, the maximum reaction force is 1839kN, maximum shear force is 1129kN, and maximum moment is 6440kN.m.

Verifications were performed for shear in the vertical and diagonal members and for moments by considering these loads.

6. Piers:

- Basic Data:

Table A6: Basic Data

Item	1# Pier	2# Pier	3# Pier	4# Pier
Level of Road Crown (m)	504.400	503.806	503.212	502.617
Distance from Road Crown to Bottom of Bearing (m)	1.57	1.57	1.57	1.57
Pier Top Level H (m)	502.330	501.736	501.142	500.547
Cross Beam Top Level (m)	479.380	478.786	478.192	485.000
Pier Height h (m)	22.95	22.95	22.95	15.55

- Dimensions of pier :

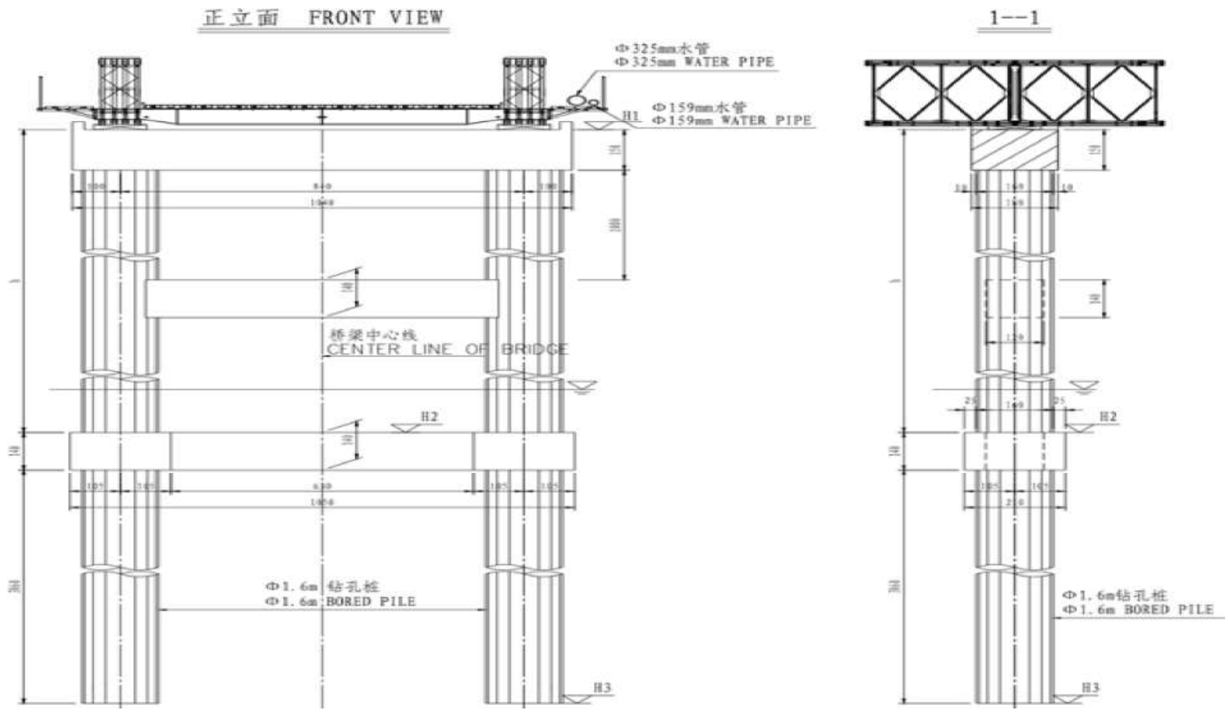


Figure 2.1: Pier Cap Sections

- Calculation Model :

A calculation model is established in MIDAS/Civil.

7. Pile Foundations:

Table A7: Pile General Data

Item	Unit	Value
Top Cross Beam Level	(m)	479.38
Depth of Cross Beam	(m)	1.4
Top Pile Level	(m)	477.98
Level of general scouring	(m)	470.50

Local Scour Level	(m)	468.00
Bottom Pile Level	(m)	439.38
D Pile Diameter	(m)	□1.6
L Pile Length	(m)	38.6
L_0 Free Length of Pile	(m)	10.0
h_m Embedded Length	(m)	28.6

i. **Piles Design Calculations:**

The pile bearing capacity can be calculated according to Clause 5.3.3 of ***Code for Design of Ground Base and Foundation of Highway Bridges and Culverts (JTG D63-2007):***

The soil-pile friction resistance value q_{ik} in the below table are quoted from the Ground Investigation Report prepared by NCEII:

Table A8:Summary of Pile Design Calculations

Pier NO.				1# Pier	2# Pier	3# Pier
P（kN）Max. Vertical Force				5140.60	5140.60	5140.60
Difference between pile self-weight and replaced soil weight ΔG（kN）				465.66	465.66	465.66
pile loading: P+ΔG （kN）				5606.26	5606.26	5606.26
Top Pile Level（m）				477.98	477.98	477.98
Depth and Parameter of Soil	Soil		q_{ik} (kPa)			
	1	Backfill	0	9.98	9.98	9.98

	2	Medium Sand	40			4.38
	3	Gravel	140	5.25	9.89	
	4	Wholly weathered Mudstone	80	5.40	2.25	7.20
	5	Medium weathered mudstone	80	2.20		
	6	Medium weathered sandstone	85	9.90	16.48	17.04
	7	Medium weathered mudstone	85	2.90		
	8	Medium weathered sandstone	90	2.97		
	Total Length（m）			38.600	38.600	38.600
Bottom Pile Level（m）				439.380	439.380	439.380
Pile Diameter（m）				1.600	1.600	1.600
Pile Circumference （m）				5.027	5.027	5.027
Bottom Pile Area A_p（m^2）				2.011	2.011	2.011
Correction factor k_2				1.000	1.000	1.000
Coefficient λ				0.700	0.700	0.700
Coefficient m_0				0.850	0.850	0.850
Embedded Depth h（m）				38.600	38.600	38.600

Weighted average density of soil beyond pile toe γ_2（kN/m^3）	19.000	19.000	19.000
Allowable bearing capacity of soil at pile toe $[f_{a0}]$（kPa）	280.00	280.00	280.00
Allowable friction bearing capacity（kN）	6781.57	7452.86	5528.20
Allowable End-bearing Capacity q_r（kN）	1144.16	1144.16	1144.16
Allowable Bearing Capacity $[R_a]$（kN）	7925.73	8597.02	6672.36

ii. Conclusion:

This summary is for Atbara Bridge, and Setit bridge calculations is typical with just the difference of the member's dimensions.

As summarized above , the existing design of the two bridges is not consistent with one standard that it's almost an imperial design calculations depends on interpreted wide variety parameters furthermore the parameters for the piles foundations is always controversial and could questioned, because of all above the evaluation of the two bridges became more essential and fundamental.

However the normal practice is to design demonstrate the truss member's forces and the critical members, and deflections at mid span.

Appendix B: Some Illustrative Calculations for Chapter III

1- Load member Forces Calculations by influence Line:

It's obviously very clear that this baily truss is indeterminate structure hence

$m+R > 2j$
$m+R < 2j$
m= number of truss members
j= number of truss joints
R= number of reactions

For the panel below is at length of 3.048 and the truss composed of 13 panel connected by pin joints:

$m=15$, $j= 13$, $R = 3$

$m+R < 2j$

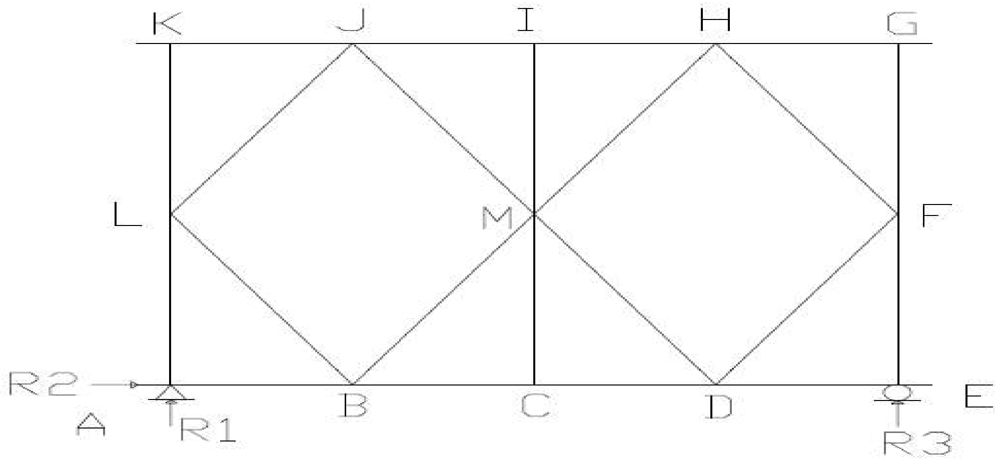
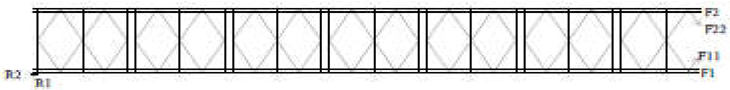


Figure B-1: Baily Panel

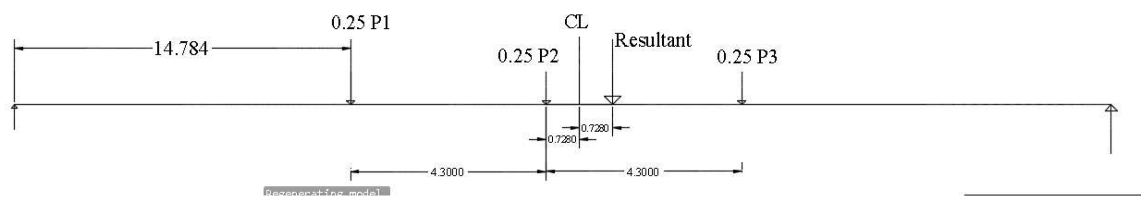
BRIDGE SPAN

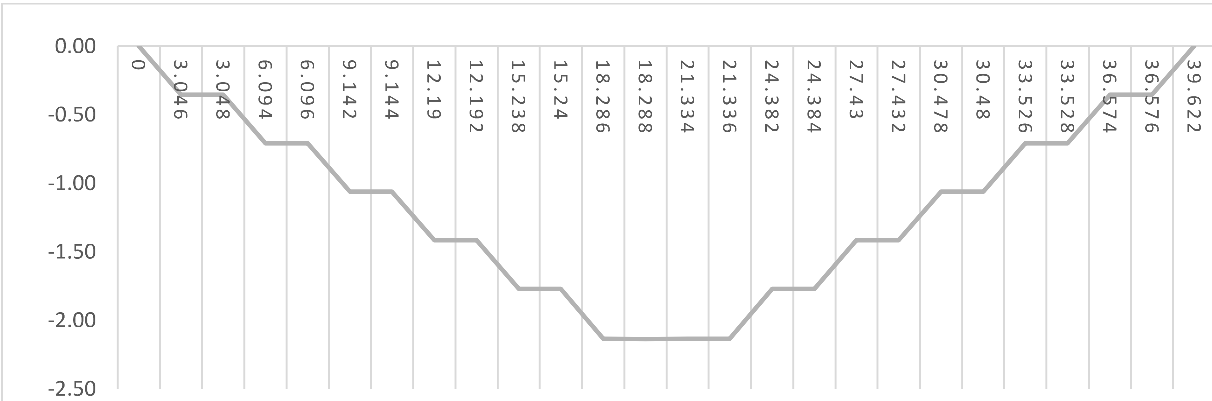


Section cut to find forces at Midspan
Bottom Chord

The influence lines Values for Bottom Chord were obtained from SAP For Live Load:

HL-93 TRUCK LOADING PATTERN AT MID SPAN





Front Axle P1 @ 14.784 = 1.6* 8.9 = 14.24 KN

Rear Axle P2 @ 19.084=2.25*35.5 = 79.88 KN

Rear Axle P3 @ 23.384 = 35.5*2.1 = 74.55 KN

Force due to 2.335 Lane Loading = 2.335*(Area of diagram)

= 2.335 *39.624*2.25/2 = 104.1 KN

Total Unfactored Member Force = 272.76 KN

- **Factored Member force at Inventory Level = 1.75*272.76 = <u>436.411 KN</u>**

  The Force value obtained from direct SAP calculations in Table.. = **<u>469.723 KN</u>**

Slight difference that the program is more accurate than hand calculation

2- Load Rating Calculations Tables:

General Load Rating Equation:

$$RF = \frac{C - (\gamma_{DC} \times DC) - (\gamma_{DW} \times DW) \pm (\gamma_P \times P)}{\gamma_L(LL \times IM)}$$

$C = \phi_c\phi_s\phi_n R$

Where the following lower limit shall apply:

$\phi_c\phi_s \geq 0.85$

R = Nominal Capacity

RF = Rating factor

C = Capacity

f_R = Allowable stress specified in the LRFD code

Rn = Nominal member resistance (as inspected)

DC = Dead load effect due to structural components and attachments

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

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

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

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

γLL = Evaluation live load factor

φ_c = Condition factor

φ_s = System factor

φ_n = LRFD resistance factor

2.1 Design Load Rating Excel Sheets :

AT INVENTORY LEVEL											
MEMBER	C	Ydc	DC	YDW	DW	YP	P	YL	LL	IM	RF
BOTTOM CHORD	432.63	1.25	238.303	0	0	0	0	1.75	326.58	1.33	0.18
VERTICAL	285.4963	1.25	118.087	0	0	0	0	1.75	172.161	1.33	0.34
DIAGONAL	323.1615	1.25	67.169	0	0	0	0	1.75	97.733	1.33	1.05
FLOOR BEAM MOMENT	1832.813	1.25	54.7463	0	0	0	0	1.75	867.871	1.33	0.87
FLOOR BEAM SHEAR	1441.64	1.25	28.25	0	0	0	0	1.75	384.796	1.33	1.57
AT OPERATION LEVEL											
MEMBER	C	YDC	DC	YDW	DW	YP	P	YL	LL	IM	RF
BOTTOM CHORD	432.63	1.25	238.303	0	0	0	0	1.35	362.2	1.33	0.21
VERTICAL	335.878	1.25	118.087	0	0	0	0	1.35	137.183	1.33	0.76
DIAGONAL	380.19	1.25	67.169	0	0	0	0	1.35	75.4	1.33	2.19
FLOOR BEAM MOMENT	2156.25	1.25	68.73	0	0	0	0	1.35	558.37	1.33	2.07
FLOOR BEAM SHEAR	1696.047	1.25	28.25	0	0	0	0	1.35	241	1.33	3.84

2.2 Legal Load Rating Excel Sheets :

TYPE 3

COLUMN1	C	Ydc	DC	YDW	DW	YP	P	YL	LL	IM	RF
BOTTOM CHORD	367.7355	1.25	238.303	0	0	0	0	1.8	128	1.33	0.23
VERTICAL	285.4963	1.25	118.087	0	0	0	0	1.8	55.7	1.33	1.03
DIAGONAL	323.1615	1.25	67.169	0	0	0	0	1.8	34.13	1.33	2.93
FLOOR BEAM MOMENT	1832.813	1.25	54.7463	0	0	0	0	1.8	867.871	1.33	0.85
FLOOR BEAM SHEAR	1441.64	1.25	28.25	0	0	0	0	1.8	384.796	1.33	1.53

TYPE 3S3

COLUMN1	C	Ydc	DC	YDW	DW	YP	P	YL	LL	IM	RF
BOTTOM CHORD	367.7355	1.25	238.303	0	0	0	0	1.8	158	1.33	0.18
VERTICAL	285.4963	1.25	118.087	0	0	0	0	1.8	72.8	1.33	0.79
DIAGONAL	323.1615	1.25	67.169	0	0	0	0	1.8	44.62	1.33	2.24
FLOOR BEAM MOMENT	1832.813	1.25	54.7463	0	0	0	0	1.8	867.871	1.33	0.85
FLOOR BEAM SHEAR	1441.64	1.25	28.25	0	0	0	0	1.8	384.796	1.33	1.53

TYPE 3-3

COLUMN1	C	Ydc	DC	YDW	DW	YP	P	YL	LL	IM	RF
BOTTOM CHORD	367.7355	1.25	238.303	0	0	0	0	1.8	160	1.33	0.18
VERTICAL	285.4963	1.25	118.087	0	0	0	0	1.8	77.03	1.33	0.75
DIAGONAL	323.1615	1.25	67.169	0	0	0	0	1.8	47.27	1.33	2.11
FLOOR BEAM MOMENT	1832.813	1.25	54.7463	0	0	0	0	1.8	867.871	1.33	0.85
FLOOR BEAM SHEAR	1441.64	1.25	28.25	0	0	0	0	1.8	384.796	1.33	1.53

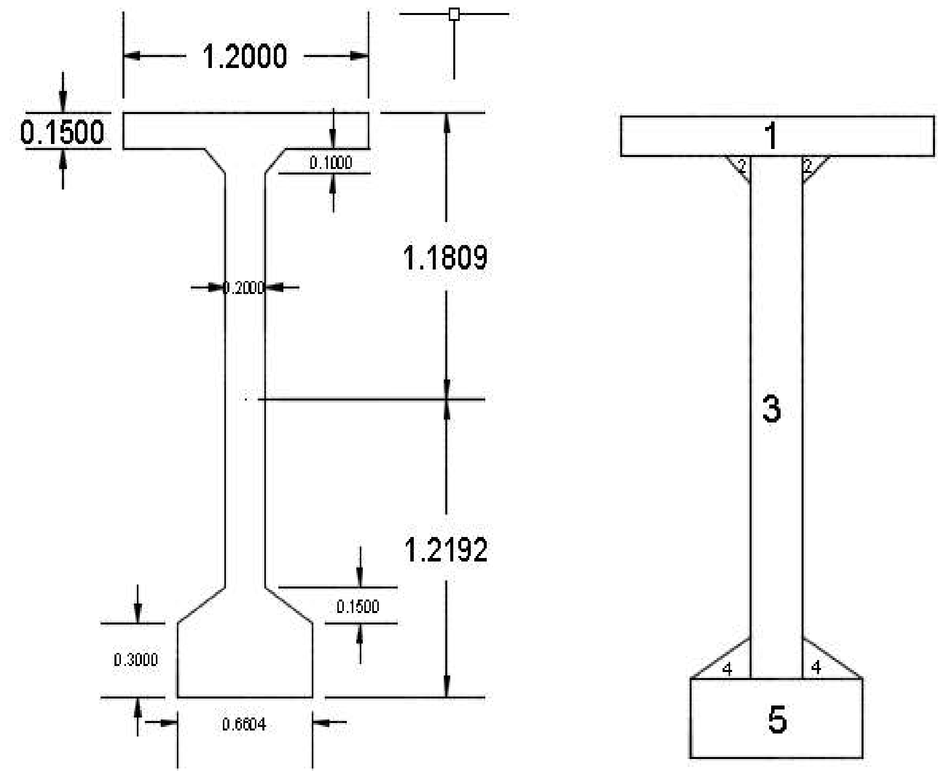
Appendix B: Some Illustrative Calculations for Concrete Precast Bulb T-Girder Superstructure in Chapter IV:

3- **Section Properties:**

- **Non Composite Section:**

Part	h	b	A	yi	Ayi	y^	Ix	d	Ad²	I
1	150	1200	1.8E+05	2325	4.2E+08		3.4E+08	1.1E+03	2.2E+11	2.2E+11
2	100	100	1.0E+04	2216.7	2.2E+07		8.3E+06	1.0E+03	1.0E+10	1.0E+10
3	1950	200	3.9E+05	1275	5.0E+08	1219.15	1.2E+11	5.6E+01	1.2E+09	1.2E+11
4	150	150	2.3E+04	375	8.4E+06		4.2E+07	-8.4E+02	1.6E+10	1.6E+10
5	300	660.4	2.0E+05	150	3.0E+07		1.5E+09	-1.1E+03	2.3E+11	2.3E+11
			8.0E+05		9.8E+08					6.0E+11

I	$y_{c.g}$	y_{bott}	y_{top}	S_{bott}	S_{top}
5.992E+11	1219.146	1219.146	1180.854	4.915E+08	5.075E+08

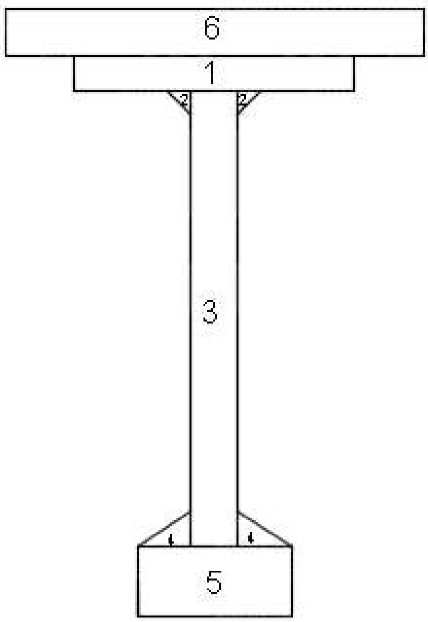


- **For Composite Section:**

The properties of composite were taken when calculating stresses at final stage that the stress at the top of deck is taken instead of top of beam.

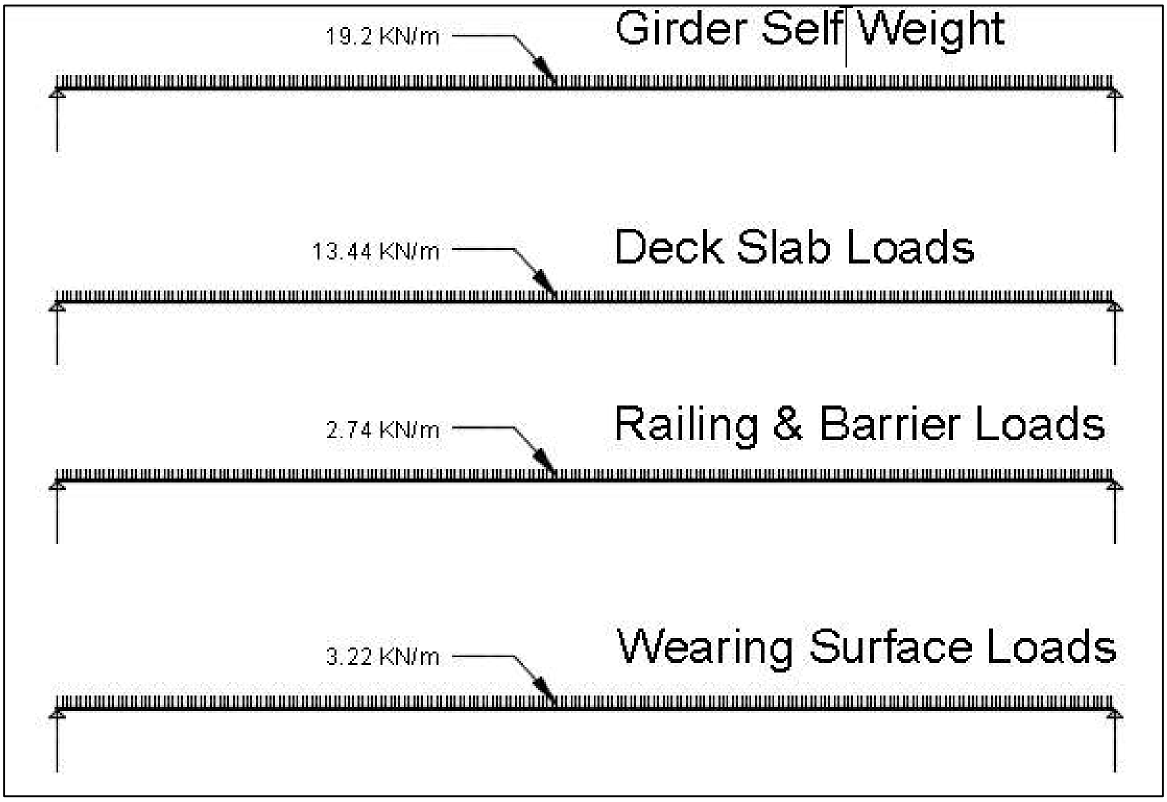
.	h	b	A	yi	Ayi	y^	Ix	d	Ad²	Ixx
1	150	1200	2E+05	2325	4.2E+08		3.4E+08	1105.85	2.2E+11	2E+11
2	100	100	1E+04	2216.7	2.2E+07	1703.71	8.3E+06	997.55	1.0E+10	1E+10
3	1950	200	4E+05	1275	5.0E+08		1.2E+11	55.85	1.2E+09	1E+11
4	150	150	2E+04	375	8.4E+06		4.2E+07	-844.15	1.6E+10	2E+10
5	300	660.4	2E+05	150	3.0E+07		1.5E+09	-1069.15	2.3E+11	2E+11
6	200	2436	5E+05	2500	1.2E+09		1.6E+09	796.29	3.1E+11	3E+11
			1E+06		2.2E+09					9E+11

I	yc.g	Ybot	Ytop	Sbot	Stop
9.10E+11	1703.7	1703.7	696.3	5.34E+08	1.31E+09



4- Dead and Live Loads Moments:

1.1 : Dead Loads Calculations:



$$M = \frac{WL^2}{8}$$

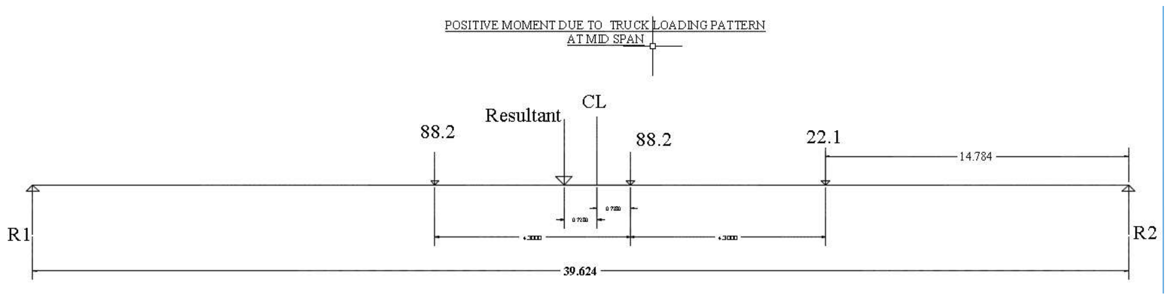
- Moment Due to girder self-weight = $\frac{19.2*39.624^2}{8} = 3768.15\ KN.m$
- Moment Due to Deck Slab = $\frac{13.44*39.624^2}{8} = 2637\ KN.m$
- Moment Due to Railing = $\frac{2.74*39.624^2}{8} = 537.7\ KN.m$
- Moment Due to Wearing Surface = $\frac{3.22*39.624^2}{8} = 632\ KN.m$

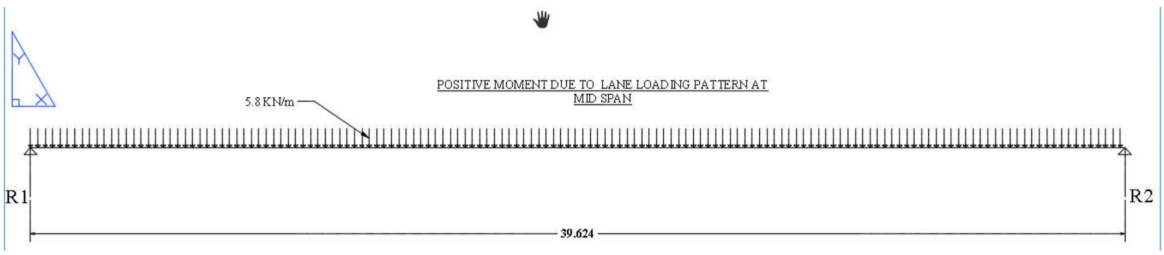
The results are typical comparing to the 2-D software calculations

1.2 Live Loads Calculations:

The Live loads distribution factor were calculated before in chapter 3

- Moment distribution factor DFM = 0.65
- Shear distribution factor DFV = 1.04





R1=(22.1x 14.784 + 88.23x19.084 + 88.23x23.384)/39.624 = 102.8 KN

Moment at Mid Span Due to Truck Loading = 102.8x19.812 – 92.3x3.57 = 1721.69 KN.m

Moment x Impact = 1721.69x 1.33 = 2289.85 KN.m

Due to Lane Load $= \frac{5.8*39.624^2}{8} = 1136.49\ KN.m$

Total Positive Moment = 3426 KN.m

Moment from SAP calculations = 3402 KN.m

5- Girder Section Selection:

-At Transfer the Bottom and Top of Girder Sheet Calculations:

Only the stresses due to girder self-weight moments used to choose the suitable section.

- Stress calculations at transfer:
  ftop = -Pt/Ag + Pte$_0$'/Stop – Mg/Stop

  fbot = -Pt/Ag - Pte$_0$'/Sbot + Mg/Sbot

  e_0 = 1219.2- 119 = 1100 (Distance between c.g of Girder and c.g of prestressed steel)

Station	Girder Self wt Moment	Pt	Ag	St	Sb	e_0	ftop	fbot
9.906	2.9E+09	8E+06	8.0E+05	5.1E+08	4.92E+08	1100	1.931556	22.9658
19.812	3.8E+09	8E+06	8.0E+05	5.1E+08	4.92E+08	1100	0.045857	21.0189
29.718	2.9E+09	8E+06	8.0E+05	5.1E+08	4.92E+08	1100	1.931556	22.9658

- Tendon Profile is Calculated as :
  e_{top} < (Md + St(Ft+Pt/A))/Pt = 1342 mm for Upper Zone
  e_{bot}< (Md- Sb (Fci +Pt/A))/Pt= 2197 mm For Lower Zone
  for any point calculate to produce the profile

6- Prestressed Loses:

Loss of prestress can be characterized as that due to instantaneous loss and time dependent loss. Losses due to anchorage set, friction and elastic shortening are instantaneous.
Losses due to creep, shrinkage and relaxation are time-dependent.

7- Moments and Shear Distribution Factors:

- Distribution Factor for Bending Moment:

For two lanes or more:

$$DFM = 0.075 + \left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{Kg}{L\,t_s{}^3}\right)^{0.1}$$ LRFD Table 4.6.2.2.2b-1

Where:

DFM= Distribution factor for moment

S = Girder spacing c/c = 2800 mm

L= Span Length = 39624 mm

Kg = n(I+A.e^2_g)

$$n= \frac{Eg}{Es} , \frac{Elastic\ modulus\ of\ girder}{Elastic\ modulus\ of\ deck\ slab} = \frac{\sqrt{40}}{\sqrt{30}} = 1.15$$

A= Cross section area of girder = 4.457E+05

e_g= Distance between the centroid of girder and slab= 439.48

I = Moment of inertia of beam = 1.567E+11

Kg = 1.8 E+11

DFM= 0.075 + (0.97 x 0.59 x 0.95) = 0.62

- Distribution Factor for Shear Force:

From LRFD Table 4.6.2.2.2b-1:

DFV = $0.2 + \left(\frac{S}{3600}\right) \left(\frac{S}{10700}\right)^2 = 0.2 + 0.77 + 0.068$ = 1.04

Appendix C: Some Illustrative Calculations for Steel Girder Superstructure in Chapter IV:

8- Section Properties:

- **Non Composite Section:**

Part	t	b	A	yi	Ayi	y^	Ix	d	Ad²	Ixx
Top Flange	89.92	428.5	3.9E+04	1063.0	4.1E+07		2.6E+07	509.01	1.0E+10	1.0E+10
Web	50.04	928.1	4.6E+04	554.0	2.6E+07	553.98	3.3E+09	89.92	3.8E+08	3.7E+09
Bottom Flange	89.92	428.5	3.9E+04	45.0	1.7E+06		2.6E+07	-509.02	1.0E+10	1.0E+10
			1.2E+05		6.8E+07					2.4E+10

Y	ybot	Ytop	Sbot	Stop
553.98	553.98	553.97	4.3E+07	4.3E+07

- **For Composite Section:**

The properties of composite were taken when calculating stresses at final stage that the stress at the top of deck is taken instead of top of beam.

Part	t	b	A	Yi	Ayi	y^	Ix	d	Ad²	Ixx
Top Flange	89.9	428.5	3.9E+04	1063.0	4.1E+07		2.6E+07	223.04	1.92E+09	1.9E+09
Web	50.0	928.1	4.6E+04	553.97	2.6E+07	794.99	3.3E+09	241.02	2.70E+09	6.0E+09
Bottom Flange	89.9	428.5	3.9E+04	44.96	1.7E+06		2.6E+07	750.03	2.17E+10	2.2E+10
Slab			6.5E+04	1252.92	8.1E+07		2.1E+10			2.1E+10
			1.9E+05		1.5E+08					5.0E+10

Y	y_{bot}	y_{top}	$y_{topslab}$	Sbot	Stop	Stop slab
794.99	794.99	312.96	512.96	6.349E+07	1.613E+08	9.840E+07

9- Check Cross-sections Proportion Limits:

- **Web Proportions:**

$$\frac{D}{t_w} \leq 50 \quad , \quad \frac{1107.95}{89.92} = 12.3 \quad <150 \quad \text{O.K}$$

- **Flange Properties**

$$\frac{b_f}{2t_f} \leq 12 \quad , \quad \frac{428.498}{2*50.038} = 4.28 \; < 12 \quad \text{O.K}$$

t_{fmin} = 1.1 t_w , t_{fmin} = 1.1x50.038 = 55.04 O.K

$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$, $0.1 \leq \frac{428.498 \text{ x } 89.916^3}{428.498 \text{ x } 89.916^3} = 1 \leq 10$ O.K

10- Check Flexure at Strength Limit State with 345 Mpa Flanges:

Check section compactness

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}} = \sqrt{\frac{200 \times 10^6}{345}} = 154.9$$

Find Dcp, the depth of the web in compression at Mp (compression rebar in the slab is ignored).

$P_t = F_{yt} b_t t_t$ = 345x428.498 x 89.916 = 13.292 10^6 N
$P_w = F_{yw} D \ t_w$ = 345x 928.116 x 50.04 = 16.02 x 10^6 N
$P_c = F_{yc} b_c t_c$ = 345x428.498 x 89.916 = 13.292 10^6 N
$P_s = 0.85 F_c b_s t_s$ = 0.85x30 x 2614.3 = 66.66 10^6 N

But:

Pt + Pw + Ps = 42.6x10^6 < Ps , Plastic N.A lies in the slab

$\bar{y} = t_s \left\{\frac{Pt+Pw+Pc}{Ps}\right\} = 200 * \frac{42.6 * 10^6}{66.66 * 10^6}$ = 127.8 mm from top of slab.

Dp = ȳ = 127.8mm

Since all web is not within compression zone, Dcp = 0, and the section is compact.

- ❖ Mu + $\frac{1}{3} f_l S_x \leq \phi_f M_n$

Dt = 1017.948 , 0.1 Dt = 110.8 < Dp

- ❖ Mn = Mp (1.07 – 0.7 $\frac{D_p}{D_t}$)

In this case (where the PNA is in the slab):c

Mp = Ps x a_1

$a_1 = \frac{D_{st}}{2} + t_s - \frac{a_c}{2}$, (D_{st} = Depth of steel Girder Only)

$a_c = \frac{A_{st}\,Fy}{0.85\,fc\,be}$ (A_{st} = Area of Steel Girder)

$= \frac{12.4 \text{ x } 10^4 \text{ x } 345}{0.85\,x30x\,2614.3} = 641.7$ mm

$a_1 = \frac{1107.948}{2} + 200 - \frac{641.7}{2} = 433.124$ mm

- Mp = 66.66 10^6 x 433.124 = 2.89 x 10^{10} N.mm
  Mn = 2.89 x 10^{10} (1.07-0.7 $\frac{127.8}{1307.948}$) = 7.0725x 10^{10}

Mn = **30725** KN.m > Mu = **11714.75** KN.m @ Strength Limit **O.K**

Appendix D: Some Calculations for Bridge Bearings in Chapter IV:

1- Bearing Design as per LRFD Loads:

From chapter 4 tables 4.21 and 4.22 of the research

- DL Due to service I = 623.9 KN
- DL Due to service I = 539.4
- Ɵs Rotation due to Service I load (From SAP) = 0.008 rad
- Psd = Vertical force due to permanent loads(deduct wearing surface) = 561 KN

c- Select Design Method (A or B):

As per LRFD specifications, there are two design methods, A&B, method A will be used. Method A usually results in a bearing with a lower capacity than a bearing designed with Method B. However, Method B requires additional testing and Quality control. Method A is described in LRFD S14.7.6, while Method B is described in S14.7.5.

d- Bearing Type and Properties:

Steel-reinforced elastomeric bearing was selected.

The bearing properties are obtained from the Specifications, as well as from past experience. The following preliminary bearing properties were selected from a worked example:

- Steel reinforcement thickness: h_{reinf} = 3mm
- Number of steel reinforcement layers: $N_{stlayers}$ = 9
- Elastomer internal layer thickness: $hr_{internal}$ = 9.5mm
- Elastomer cover thickness: hr_{cover} = 6.3 mm
- Pad width (bridge transverse direction): W_{pad} = 380mm
- Pad length (bridge longitudinal direction): L_{pad} = 355 mm
- Materials Properties:
  Elastomer shear modulus: G = 0.66 Mpa STable 14.7.5.2-1
  Elastomer hardness: HshoreA = 50 S14.7.5.2
  Creep deflection Cd= 0.25
  Steel reinforcement yield Strength = 345 Mpa
  Design Computations:

1- Compute Shape Factor:

$$Si = \frac{L*W}{2h_{ri}(L+W)}$$

- Shape factor for cover layer :

$$S_{cover} = \frac{L*W}{2h_{Cover}(L+W)} = 14.57$$

- Shape factor for internal layer :

$$h_{internal} = \frac{L * W}{2h_{internal}(L+W)} = 9.67$$

2- **Check Compressive Stress**

For bearing Subjected to shear deformations :

$\sigma s \leq 1.66\ G*S \leq 11$ Mpa

$\sigma L \leq 0.66\ G*S$

The compressive stress is taken as the total reaction at one of the pier bearings for the service limit state divided by the elastomeric pad plan area. The service limit state dead and
live load reactions are obtained from the tables of superstructure output. The shape factor used in the above equation should be for the thickest elastomer layer.

$$\sigma_S = \frac{DLserv+LL\ serv}{(L+W)} = \frac{(623.9+539.4)*10^3}{(380*355)} = 8.6 < 10.6 < 11 \quad O.K.$$

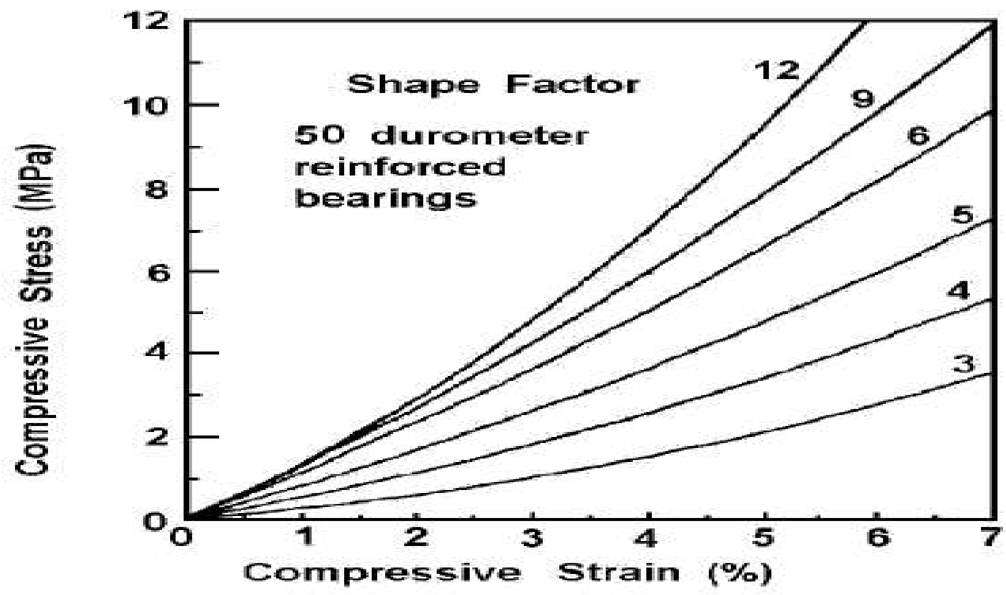
$$\sigma_L = \frac{LL\ serv}{(L+W)} = \frac{539.4*10^3}{(380*355)} = 4.0 < 4.2 \quad O.K$$

3- **Check Compressive Deflection**

The compressive deflection due to the total load at the service limit state is obtained from the following equation:

$$\delta = \sum \varepsilon_i h_{ri}$$

ε_i = The instantaneous compressive strain was approximated from LRFD CTable 14.7.5.3.3-1 for 50 durometer reinforced bearings using a compressive stress of 8.6 Mpa and a shape factor of 9.67:



From Table: $\varepsilon_i = 0.05$

$$\delta_{inst} = 2\varepsilon_{inst}\, h_{r.cover} + 8\, \varepsilon_{inst}\, h_{r.internal}$$

Note: The cover is 2 layers and 8 of internal layers.

$\delta_{inst} = 2x0.05x6.3 + 8x0.05x9.67 \quad = \mathbf{4.43}$ mm

$\delta_{creep} = Cd^* \, \delta_{inst} = 0.25x4.43 = 1.1$ mm

$\delta_{Total} = \delta_{inst} + \delta_{creep} = 5.53$ mm

4- **Check Shear Deformation**

The shear deformation is checked to ensure that the bearing is capable S14.7.6.3.4 of allowing the anticipated horizontal bridge movement.

The bearing must satisfy:

$$h_{rt} \geq 2\, \Delta s$$

$$h_{rt} = 2\, h_{r.cover} + 8\, h_{r.internal} = 89.96 \text{ mm}$$

To find Δs:
Expansion calculation:

- $\varepsilon = 11x10^{-6}$
- Initial Deign temperature = 35 C^o
- Max Deign temperature = 70 C^o
  Δries rise of temp. = 70-35 =35

$\Delta exp = \varepsilon \cdot \Delta rise\ Lspan = 11x10^{-6}$ x 35 x39624 = 15.3 mm

Contraction calculation:

$\Delta cont = \varepsilon \cdot \Delta t\ Lspan = 11x10^{-6}$ x 35 x39624 = 15.3 mm

$\Delta t = \Delta fall = 35$ (fall of temperature to zero)

$\Delta cont$ = 15.3mm

$\Delta t = 1.2\ \Delta cont$ = 18.3 mm

$h_{rt} \geq 2\ \Delta s$, 89.96 > 2x18.3 = 36.6 O.K

5- **Check Rotation or Combined Compression and Rotation**

Since Design Method A was chosen, combined compression and rotation does not need to be checked. The rotation check ensures that no point in the bearing undergoes net uplift and is as follows (S14.7.6.3.5).

$$\sigma_s \geq 0.5\ G\ S \left(\frac{\text{Lpad}}{\text{hr}}\right)^2 \frac{\theta s}{n}$$

$\theta s = 0.008$, n =9 No of layers +half top and bottom cover

- $\sigma_s \geq 4.3$ O.K

6- **Check Stability:**

The total thickness of the pad shall not exceed the least of L/3 or W/3.

$\frac{Lpad}{3} = 126.66$, $\frac{Wpad}{3} = 118.33$

The total thickness of the pad based on the preliminary dimensions is:

$h_{total} = 2 \cdot hrcover + 8 \cdot hrinternal + Nstlayers$ x hreinf

$= 2x6.3 + 8x9.5 + (9x3) =$ **121.9,** $> \frac{\boldsymbol{Wpad}}{\boldsymbol{3}}$

Increase Wpad to be 380 mm

7- **Check Reinforcement**

The thickness of the steel reinforcement must be able to sustain the tensile stresses induced by compression in the bearing. The reinforcement thickness must also satisfy the requirements of the AASHTO LRFD Bridge Construction Specifications S14.7.6.3.7.

$$hs \geq \frac{3hmax * \sigma s}{F_y}$$

hs= hreinf, hmax = hr internal = 9.5mm , σs = 8.6 , Fy = 345

$$\frac{3hmax * \sigma s}{F_y} = 0.7\text{mm}$$

- $h_{reinforcement}$ = 3mm > 0.7mm O.K

8- **Conclusion:**

Use 380 x 380 mm bearing with 8 internal layers and 9 reinforcement plates