ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

# CONNECTION STUDY OF LONG SPAN STEEL TRUSS BRIDGES

LEULE MEBRATIE

APRIL 2012

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**BY LEULE MEBRATIE**

**THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF ADDIS ABABA UNIVERSITY, INSTITUTE OF TECHNOLOGY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING.**

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**Approved by Board of Examiners**

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# Acknowledgement

It is fortunate to explore something with you, Dr. Asnake Adamu; and I am really grateful to have you here with all your genuine guidance and advice. I always wish you be a means to the development of science and technology in Ethiopia.

And my brother, Wondwossen Abebe; I am always happy that God picked you to be my brother and am grateful for you being the means to the success of this research.

## List of Symbols

***a***= width

***A*** = cross-sectional area

$A_{bm}$ = cross sectional area of the base material

***Av*** = the shear area

***Aw*** = cross sectional area of the weld

***b*** = length

***BR*** = vehicular braking force

***CE*** = vehicular centrifugal force

***CR*** = creep

***Ct*** = torsion constant

***CT*** = vehicular collision force

***d*** = diameter of pin/bolt

***DC*** = dead load of structural components

***DD*** = down drag

***DW*** = dead load of wearing surfaces and utilities

***E*** = modulus of elasticity

***EH*** = horizontal earth pressure load

***EL*** = accumulated locked-in effects resulting from the construction process

***EQ*** = earthquake load

***ES*** = earth surcharge load

***EV*** = vertical pressure from dead load of earth fill

$f_b$ = maximum normal stress due to bending

$F_{bm}$ = nominal strength of the base material

$F_{b,RD}$ = bearing capacity of a pin

$F_{EXX}$ = nominal tensile strength of the weld metal

$F_{V,RD}$ = specified minimum ultimate strength of pin

$F_w$ = nominal strength of the weld electrode

$f_y$ = yield strength

***h*** = overall depth

***IM*** = vehicular dynamic load allowance

***L*** = length

***LL*** = vehicular live load

***LS*** = live load surcharge

***M*** = bending moment due to the applied loads

$M_D$ = design moment

$M_{RD}$ = moment capacity of the pin

$M_{sd}$ = the design bending moment

***WA*** =water load and stream pressure

***Wel*** = elastic section modulus

***WL*** = wind on live load

***Wpl*** = the required section design plastic resistance

***WS*** = wind load on structure

$\beta A$ = factor

$\eta i$ = load modifier: a factor relating to ductility, redundancy, and operational importance

$\eta D$ = a factor relating to ductility, as specified below

$\eta R$ = a factor relating to redundancy as specified below

$\eta I$ = a factor relating to operational importance as specified below

$\sigma x$ = stress in the x direction

$\sigma xy$ = shear stress

$\sigma y$ = stress in the Y direction

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# Abstract

Safe and economic design of a structure is an indication of good computational process which has been followed in this work. This intensive computation of analysis and design of structural elements had great value in the transfer of loads. During the structural analysis of truss elements and design of the elements; joints between the truss chords play great role in transferring the force.

The very custom of designing a connection element is standard detail information and even without an adaptation of the connection design and this situation is hardly very risky, for especially to Ethiopian case; because we even do not have a manual or a guide line for such a long span steel truss bridge connections.

This problem calls for the study of long span steel truss bridge connections. Failure stresses of Connections of steel structural elements are verified and have got recommendations to handle or to minimize it. Thus, this research is intended to assess the reliable connection design process for all designers and for concerned governmental body who are responsible in code book or standard preparation.

# Table of Contents

# 1.0 Introduction

## *1.1 Background*

Countries characterized by topographic conditions such as a mountainous rough terrain, deep gorges, river crossings and other factors that make construction of crossways such as culverts and bridges mandatory in road construction [1]. Bridges that have long span is needed in such areas for safe transportation of civilization from one nation to another. The study of this Thesis mainly focused on literature assessment of connections of long span steel truss bridges in view of addressing causes of failure, addressed in hand books, design standards and recent papers published journals of bridges and structures. Based on the main objective of the Thesis, the study focused on developing analysis and design procedure for long span steel truss bridge connections.

The increase in span length of long span bridges results in remarkable decrease in their natural frequencies and the ratio between the fundamental torsion and vertical mode frequencies [2]. This renders long span bridges very susceptible to the actions of strong wind.

Connections are an intimate part of a steel structure, their proper treatment is essential for a safe and economic structure [3]. An intuitive knowledge of how a system will transmit loads (the art of load paths) and an understanding of structural mechanics (the science of equilibrium and limit states) are necessary to achieve connections which are both safe and economic.

## *1.2 Objectives of the Thesis*

The objective of this study is to assess the design and construction behavior of long span steel truss bridge connections and assessing manual recommendations that are often given by manufacturers.

The specific objective of the thesis is to develop analysis and design procedure for long span steel truss bridges with emphasis on the design and construction of the connections that can serve as a guideline for designers, and illustrate the application using a practical design example.

### *1.3 Thesis Content*

The study has focused on developing design and construction procedure for connections of a long span steel truss bridge and assessment of failure stresses during construction. In addition to this, the study reviews manual recommendations of manufacturers. In this view the study is organized as follows.

The first chapter is devoted to introduction dealing with background of the study, objectives and scope.

The second chapter deals with the review of long span steel truss bridges.

The third chapter discusses the international and Ethiopian construction practices of long span steel truss bridge.

The fourth one deals on analysis and design procedures of the long span steel truss bridge.

The fifth chapter deals on the discussion of the results.

Finally, the sixth chapter addresses the conclusion and recommendations of the Thesis.

### *1.4. Applications and Limitations*

The result of this study can be applied to design of steel truss bridge connections. The study shall benefit the designers and contractors to design and construct safe joint connections between members and can be an input for the preparation of codes and standards. The scope of the study has been limited to the preparation of analysis and design procedure for selected connections only.

# 2.0 Brief Review of Long Span Steel Bridges

## *2.1 Historical Development and Design Standards*

Trusses are triangulated frameworks used as spanning or bracing elements in buildings, bridges, transmission towers, and other structures [2]. What distinguishes the truss from other structural forms is precisely its triangulation, from which two benefits accrue:

- ✧ first, the triangular geometry is inherently stable;
- ✧ second, all internal stresses — at least in "ideal" trusses whose bars are pinned together at the vertices of each triangular panel and whose loads are applied only at these pinned joints — are axial, i.e., limited to pure tension and pure compression.

Aside from its web of triangular panels, the truss has no intrinsic formal identity. Put another way, it is the specific pattern of internal diagonal, vertical, or horizontal bars (patterns that in many cases bear the names of their nineteenth-century inventors: Pratt, Howe, Town, Warren, etc.) that makes the structure a truss, not its overall shape. One may design an arch as a truss, a beam or column as a truss, or any number of tower forms — essentially beams cantilevered from the ground plane — as trusses. Advantages of truss construction include the following:

1) Large trusses can be assembled from small members pinned together, facilitating production, transportation, and erection;

2) Because all internal stresses are axial, with no bending stresses present, the truss is an extremely efficient structural form;

3) Because trusses are typically assembled from individual elements bolted, welded, or nailed together, it is relatively easy to customize the overall shape of the truss in relation to external loads and spans; and to adjust the cross-sectional area of each member in relation to anticipated internal stresses.

Trusses have been used for many centuries; Andrea Palladio illustrates truss bridges in his *Four Books of Architecture* as early as 1570. However, it is in the nineteenth century that industrial expansion in particular the need for long-span exhibition and market halls, railroad terminals, and bridges together with the development of engineering theory and improvements in the production of cast and wrought iron, and later steel, provide the motive and means for most of the advances in truss design that are exploited within early twentieth-century architecture [2].

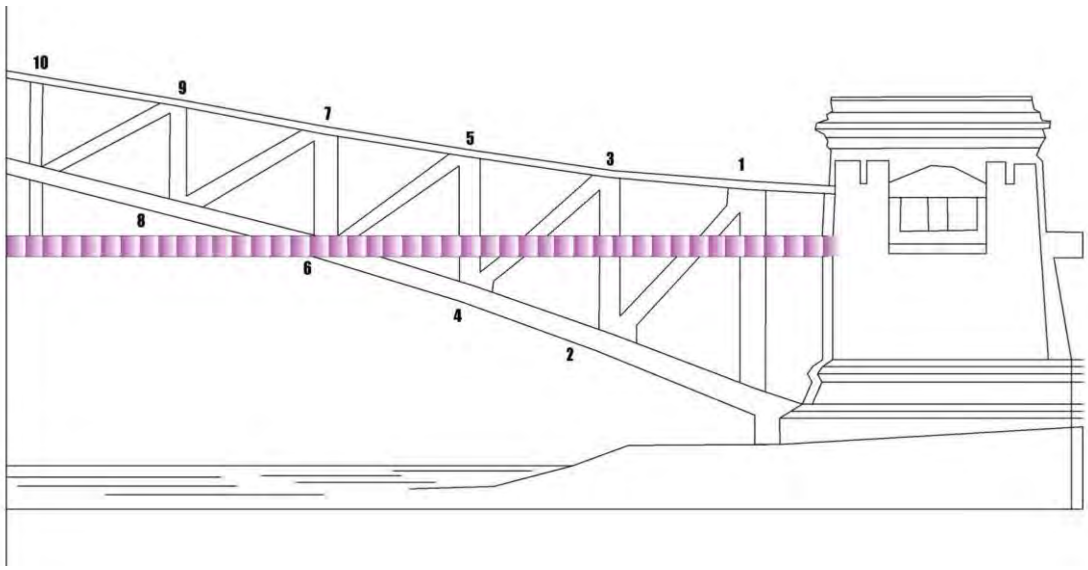



Figure 2.1: Lindenthal: Hell's Gate Bridge;

Early twentieth-century trusses are also important as infrastructure (bridges); and as industrial (long-span factory roofs), vernacular (ordinary wooden gable roofs) or pragmatic (hidden bracing or support) building elements. Gustav Lindenthal's arched Hell's Gate Bridge in New York City (1916) utilizes steel trusses that were the longest spans of their type when constructed. Steel trusses are used to transfer loads over large spans allowing hotel rooms to be placed on top of lower-floor ballrooms, or office buildings over railroad tracks — without themselves being expressed as part of the architectural form. William LeMessurier's development in the 1960s of a "staggered truss" system is another example of an entirely pragmatic invention utilizing trusses to minimize floor-to-floor dimensions in multi-story steel-framed structures.

Trusses are commonly featured in so-called "high tech" architecture of the late twentieth century. Buildings within this genre reprise to some extent the Constructivists' interest in industrial production, but differ from Constructivist projects in at least two respects: high tech buildings tend to be less influenced by abstract compositional formulas; and often evidence a more theoretically grounded appreciation of structural systems as potential sources of architectural expression. In Renzo Piano and Richard Rogers's Pompidou Center in Paris (1977), virtually the entire architectural concept relies on exposed truss work. Long-span interior floor trusses define

column-free exhibition zones, and exterior wind-bracing trusses create the gridded diagonal pattern of the facades [2].
Though there is no definite definition of long span bridge, in the NSBA (National Steel Bridge Alliance) prize bridge brochure, they define short, medium and long-spans:

| | |
|---|---|
| Short span | less than (37m) 120 ft |
| Medium span | up to (122m) 400 ft |
| Long span | over (122m) 400 ft |

Use of structural steel in bridges exploits its advantageous properties of economically carrying heavy loads over long spans with minimum dead weight. Steel is however suitable for all span ranges, categorized [4].

### *2.2 Materials and Loadings on Long Span Steel Bridges*

- **Materials Properties**

Variability of loading is only one aspect of uncertainty relating to structural behavior. Another important one is the variability of the structural material which is reflected in variations in strength of the components of the structure. Again, the variability is formally accounted for by applying appropriate partial safety factors to characteristic values. For structural steel, the most important property in this context is the yield strength [5].

- **Characteristic Values of Material Properties**

The characteristic yield strength is defined as that value below which only a small proportion of all values would be expected to fall. Theoretically this can only be calculated from reliable statistical data. In the case of steel, for practical reasons a nominal value, corresponding typically to the specified minimum yield strength is generally used as the characteristic value for structural design purposes.

- **Design Values of Material Properties**

The design value for the strength of steel is defined as the characteristic value divided by the appropriate partial safety factor. Other material properties, notably modulus of elasticity, shear modulus, Poisson's ratio, coefficient of linear thermal expansion and density, are much less variable than strength and their design values are typically quoted as deterministic.

In addition to the quantified values used directly in structural design, certain other material properties are normally specified to ensure the validity of the design procedures included within codified rules. For instance Eurocode 3 stipulates minimum requirements for the ratio of ultimate to yield strength, elongation at failure and ultimate strain if plastic analysis is to be used [5].

- **Geometric data**

Geometrical data are generally represented by their nominal values. They are the values to be used for design purposes. The variability, for instance in cross-section dimensions, is accounted for in partial safety factors applied elsewhere. Other imperfections such as lack of verticality, lack of straightness, lack of fit and unavoidable minor eccentricities present in practical connections should be allowed for. They may influence the global structural analysis, the analysis of the bracing system, or the design of individual structural elements and are generally accounted for in the design rules themselves.

- ***Loadings:***

The various types of loading which need to be considered can broadly be classified as permanent, and transient (variable). Permanent loads are those due to the weight of the structure itself and of any other immovable loads that are constant in magnitude and permanently attached to the structure. They act on the bridge throughout its life. Transient loads are those loads that vary in position and magnitude and act on the bridge for short period of time such as live loads, wind loads and water loads etc. Some of these loadings include [5]:

1. Permanent loads
    - dead load of structure
    - superimposed dead loads
2. Transient loads
    - vehicular live loads
    - pedestrian live loads
    - impact loads

• wind loads

• earthquake loads

These loads are factored and combined to produce extreme adverse effect on the member being designed.

***Dead Load***

The dead load on superstructure is the weight of all structural elements and nonstructural parts of the bridge above the bearing. This would include the main supporting trusses or girders, floor beams, stringers, the deck, sidewalk, bracings, parapets and road surfacing.

***Live Loads***

The live load for bridges consists of the weight of the applied moving load of vehicles and pedestrians. The traffic over a highway bridge consists of a multitude of different types of vehicles. To form a consistent basis for design, standard loading conditions are applied to the design model of structure. These standard loadings are specified in codes and manuals. Some of these are American Association of State Highway and transportation Officials (AASHTO) Bridge Design Manual, Ethiopian Roads Authority Bridge Design Manual 2002; Addis Ababa City Roads Authority Bridge Design Manual 2004. In Ethiopia the design vehicle live load on roadways of bridges, designated HL-93, shall consist of a combination of the [5]:

• design truck plus design lane load

• design tandem plus design lane load

The live load model consists of either a truck or tandem with a lane load of 9.3kN/m at the same place. The lane load is assumed to occupy 3m transversely within the design lane.

***Design Truck***

The weight and the spacing of the axle and wheel for design truck are as specified in Figure 2.2 [5]

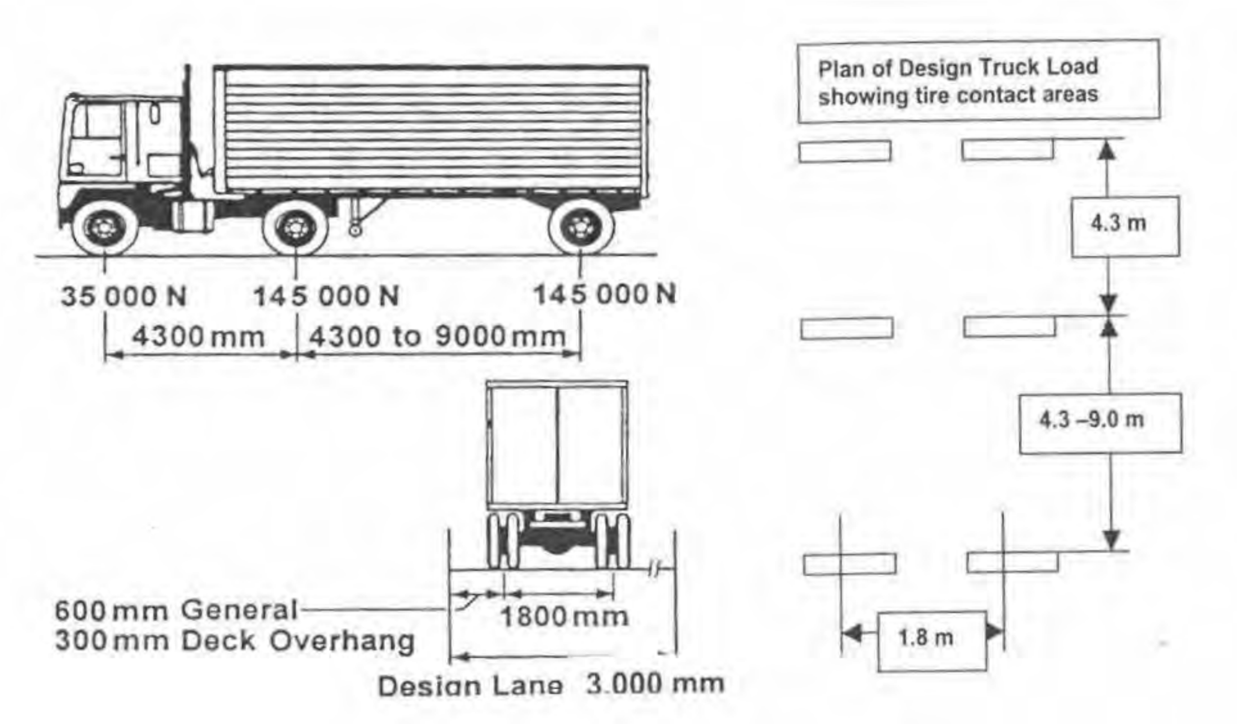



Figure 2.2: The Design Truck

A dynamic load allowance shall be considered [5]. Unless otherwise specified, the spacing between the two 145kN axels shall be varied between 4.3 and 9m to produce extreme force effect.

***Design Tandem***

The design tandem used for strategic bridge shall consist of a pair of 110kN axles spaced at 1.2m apart. The transverse spacing of wheels shall be taken as 1.8m. A dynamic load allowance shall be considered [5]. The design tandem load is shown in the Figure 2.3

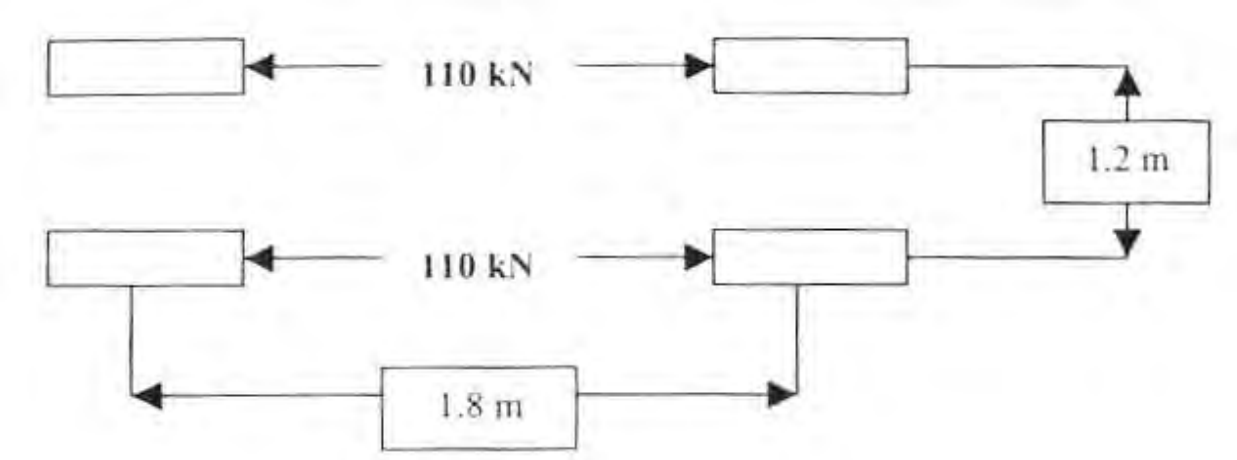



Figure2.3: Design Tandem Loads

***Design Lane Load***

The design lane load shall consist of a load of 9.3kN/m, uniformly distributed in longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over 3m width. A dynamic load allowance shall not be considered [5].

***Dynamic Load Allowance (Impact)***

The truckloads on bridges are applied not gently and gradually but rather violently, causing stress increase. Therefore, additional loads called impact loads must be considered [6].

These are taken into account by increasing the static effects of design truck or tandem, with the exceptions of centrifugal and breaking forces, by the Dynamic Load Allowance, IM percentage specified in Table 2.1 [5].

The factor to be applied to the static load shall be taken as: (1 + IM/100). The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load.

Table 2.1 Dynamic Load Allowance, IM

| **Component** | **IM** |
|---|---|
| Deck Joints – All Limit States | 75% |
| All Other Components<br>Fatigue and Fracture Limit State | 15% |
| All Other Limit States | 33% |

***Wind Load***

Wind forces are resisted by the bracing systems for a through bridge. This shall suffice the recommendations of the ERA 2002 code book.

***Earthquake Loading***

Seismic analysis is not required for single-span Bridge, regardless of the seismic Zone [5].

***Load Factors and Combinations***

The LRFD (Load and Resistance Factor Design) design method as per the provision of ERA 2002 Bridge Design Manual is used. Load factors are applied to the loads and resistance factors to the internal resistances or capacities of sections. The load combinations, load factors and force effects are considered according to ERA 2002 Bridge Design Manual clause 2.8 and shall satisfy Eq. (2.1).

$$\Sigma \eta_i \gamma_i Q_i < \varphi R_n = R_f \quad (2.1)$$

Where:

for loads for which a maximum value of $\gamma i$ is appropriate:

$\eta i = \eta_D \eta_R \eta_I \geq 0.95$

for loads for which a minimum value of $\gamma i$ is appropriate:

$\eta i = 1/( \eta_D \eta_R \eta_I) \leq 1.0$

Where:

$\eta i$ = load modifier: a factor relating to ductility, redundancy, and operational importance

$\gamma i$ = load factor: a statistically based multiplier applied to force effects

Qi = force effect

$\varphi$= resistance factor: a statistically based multiplier applied to nominal resistance

Rn = nominal resistance

$\eta_D$ = a factor relating to ductility, as specified below

$\eta_R$ = a factor relating to redundancy as specified below

$\eta_I$ = a factor relating to operational importance as specified below

$R_f$ = factored resistance: $\varphi$ Rn

Table 2-2 - Load Combinations and Load Factors

| Load Combination<br><br>Limit State | DC<br>DD<br>DW<br>EH<br>EV<br>ES | LL<br>IM<br>CE<br>BR<br>PL<br>LS<br>EL | WA | WS | WL | FR | TU<br>CR<br>SH | TG | SE | Use one of these at a time | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | EQ | CT |
| **STRENGTH I** (Unless noted) | $\gamma_p$ | 1.75 | 1.00 | - | - | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - |
| **STRENGTH II** | $\gamma_p$ | 1.35 | 1.00 | - | - | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - |
| **STRENGTH III** | $\gamma_p$ | - | 1.00 | 1.40 | - | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - |
| **STRENGTH IV** EH, EV, ES, DW, DC ONLY | $\gamma_p$<br><br>1.5 | - | 1.00 | - | - | 1.00 | 0.50/1.20 | - | - | - | - |
| **STRENGTH V** | $\gamma_p$ | 1.35 | 1.00 | 0.50 | 1.0 | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - |
| **EXTREME EVENT I** | $\gamma_p$ | $\gamma_{EQ}$ | 1.00 | - | - | 1.00 | - | - | - | 1.00 | - |
| **SERVICE I** | 1.00 | 1.00 | 1.00 | 0.30 | 1.0 | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - |
| **SERVICE II** | 1.00 | 1.30 | 1.00 | - | - | 1.00 | 1.00/1.20 | - | - | - | - |
| **SERVICE III** | 1.00 | 0.80 | 1.00 | - | - | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | - | - |
| **FATIGUE** LL, IM and CE ONLY | - | 0.75 | - | - | - | - | - | - | - | - | - |

Where: $\Upsilon_{SE}$ = load factor for settlement, TU = uniform temperature, WA =water load and stream pressure, FR = friction, IM = vehicular dynamic load allowance, BR = vehicular braking force, DC = dead load of structural components, CE = vehicular centrifugal force, DD = down drag, LL = vehicular live load, LS = live load surcharge, CR = creep, DW = dead load of wearing surfaces and utilities, CT = vehicular collision force, EH = horizontal earth pressure load, EL = accumulated locked-in effects, $\gamma_{EQ}$ = load factor for live load with seismic load resulting from the construction process, EQ = earthquake load, ES = earth surcharge load, EV =vertical pressure from dead load of earth fill, $\gamma_p$ = load factor for permanent loading, $\gamma_{TG}$ = load factor for temperature gradient, PL = pedestrian live load, SE = settlement, SH = shrinkage, TG = temperature gradient, WL = wind on live load, WS = wind load on structure

### *2.3 Analysis and Design Concept of Long Span Steel Bridges*

Rational design of long span bridges often is inhibited by the lack of a design code. The live loading in this case is left to experienced designer or consultant. Certain types of bridges are relatively insensitive to errors in estimating these loads, because their design is governed by the dead loads (e.g., heavy truss bridges). Other types, however, are very sensitive to errors or to changes in the live load, and examples include single box girders and cable-stayed bridges. Although most of the world's long – span bridges are found in North America, very little has been made to establish representative loads for long-span bridges with few exceptions [Invy et al., 1953] for any bridge or component of a bridge that is outside the range of the code normally appropriate for a shorter bridge under the same conditions.

The credible load occurring on a short- span bridge is the heaviest trucks that can travel on the bridge deck, but this is not the case for a long-span bridge because this structure will not be entirely covered by the heaviest possible vehicles. This claim was tested in a bridge in Vancouver, British Columbia, Canada (Navel et al., 1976), with a main span of 473 m. Two methods were used to predict long – span bridge loading: a purely analytical solution of probability equations, and a procedure based on the random-scatter capability of a computer to simulate incoming traffic (Buckland, Navin, Zidek, and McBryde, 1980). Basic assumptions included stationary traffic where maximum loading will occur only when the traffic is stationary and bumper-to-bumper. When traffic begins to move, vehicle distance increase and load intensity decreases. For the loading, therefore, the traffic is stationary and allowance is not made for impact.

AASHTO, also states the consecutive truck arrangement for long span or multiple span is considered 15m after the rear axle.

### *2.4 Types of Connections, Analysis and failure Stresses*

**General**

The major connections in bridge trusses occur at the truss nodes where the web members are connected to the chord members. This connection usually incorporates a splice in the chord member and sometimes also in one or both of the minor truss connections joining the cross girder and the lateral system to the truss [7].

Site connections can be made by high strength friction grip bolts for reasons of economy and speed of erection. Good site welds are difficult to achieve where access is difficult and fatigue life of welded joints is lower than that of bolted joints.

However, in several countries, the connections are now usually butt-welded on site. Figure 2.4 shows different gusset geometries which are used to obtain durability in view of the fatigue-governing effects [7].

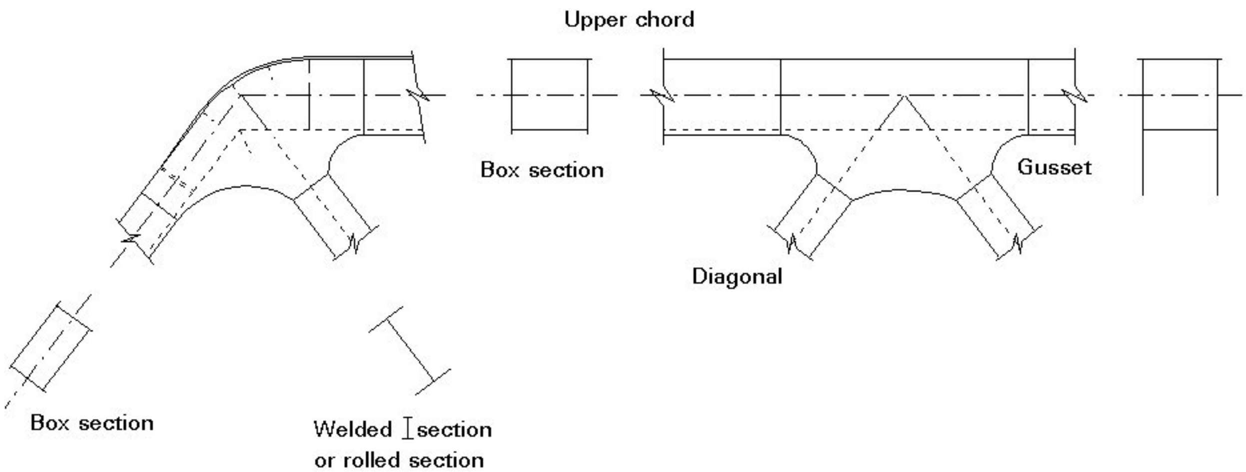



Figure 2.4: Butt-welded connections and gusset geometries used to avoid fatigue in the connection

When a concrete slab is cast in place to support the highway or the railway, the horizontal forces caused by the shrinkage of the concrete should be taken into account in the design of the lower chord connection joints.

**Truss Joints**

At the nodes of a truss where the web members are connected to the chords, there is a change in load in the chord which necessitates a change in its cross-section area. The node is, therefore, the point at which there is a joint in the chord as well as being the connection point of the web members [7].

The web members are connected to the chords by vertical gusset plates. They are usually bolted to the chord webs and the web members fit between them (Figure 2.5a).

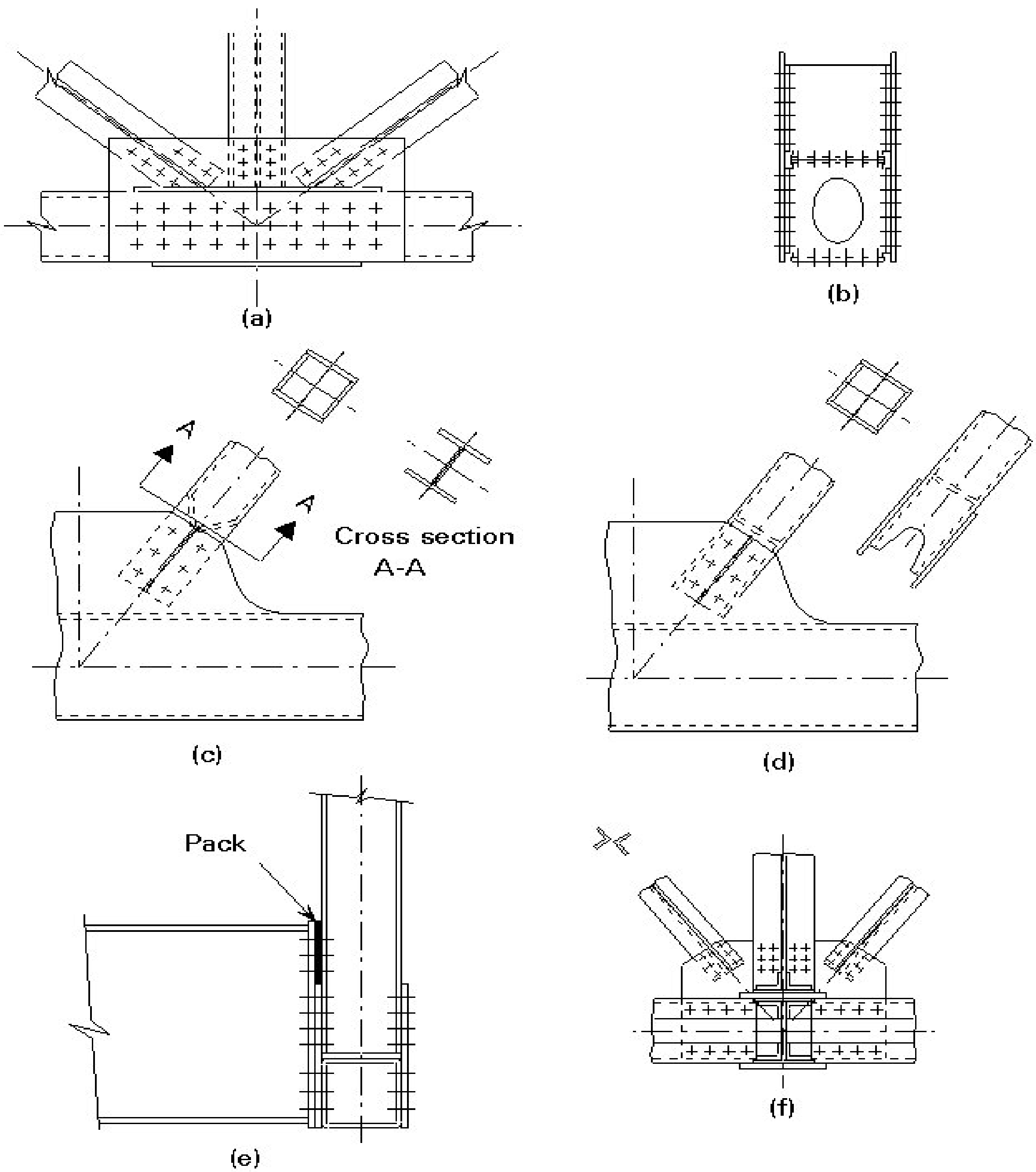



Figure 2.5: Bolted connections

The chord joint is effected by providing cover plates. They should be so disposed, with respect to the cross-section of the member, as to transfer the load in proportion to the respective parts of the section (Figure 2.5b). The gusset plates form the external web cover plates. Since they work in the dual capacity of cover plate and web connector, their thickness takes this into account. The joint is designed to carry the coexistent load in the lesser loaded chord plus the horizontal component of the load in the adjacent diagonal. The load from the other diagonal is transferred to the more heavily loaded chord through the gussets alone.[7] In compression chords which have fitting abutting ends in contact, design codes allow up to 75% of the compressive load to be carried through the abutting ends.

Sometimes the gusset is formed by shop-welding a thicker shaped plate to the chord in place of the chord web. The web members are then all narrower than the chords and the chord splice is offset from the node. An advantage occurs in erection as the web connections can be made before the next chord is erected.

At the connections of all tension members and elements, care has to be taken in the arrangement of bolt holes to ensure that the critical net section area of the section is not so small that fracture will govern. If necessary, staggering the lines of bolts helps to increase the effective net area. Remember that the critical net section is usually at the ends of the section or the centre of the cover plates, and that elsewhere some of the load has been transferred to the other parts of the joint and more bolt holes can be tolerated.

Connections of web members to gussets are quite straightforward and special treatment such as the use of lug angles is rarely required. In connecting rectangular hollow sections the method shown in Figure 2.5d is preferable to that of Figure 2.5c.

Unsupported edges of gussets should be such that the distance between connections does not exceed about 50 times the gusset plate thickness (Figure 2.5a). If this is unavoidable, the edge should be stiffened [7].

***Cross Girder Connections***

They are quite straightforward. The 2 or 4 rows of bolts in the cross girder end plate are made to correspond with the equivalent central rows of bolts in the gusset. Packing plates may be required to accommodate the difference in height of gussets and cross girders (Figure 2.5e).

*Lateral Bracing Connections*

As recommended in (ii) the axes of the lateral systems should be in the same planes as those of the truss chords. This requirement is met in 2 of the 3 types of lateral members and connections described below [7]:

i. For long and medium spans, the lateral members are frequently made from two rolled channel sections connected by lacing to give an overall depth the same as the chords. They are connected to the chords by gussets bolted to the chord flanges exactly as the main web members are connected to the main joint gussets.

ii. For medium spans, laterals consisting of two rolled angles arranged toe to toe in "star" formation and with intermediate battens are often ideal. They are connected to the chords by gussets positioned at the chord axis (Figure 2.5f). Note, angles "back-to-back", but separated by a small gap should never be used because of maintenance problems.

iii. On short spans single laterals often suffice. They can be connected by a gusset to the upper or lower chord flange, as the moments due to eccentricity are small.

*TYPES OF SPLICE*

There are two basic methods of making splices. Welding, using butt welds or fillet welds, and bolting. Where the main elements of the splice can be connected together with full strength butt welds, the design is simple and the effect of any loss of section due to the bolt holes does not arise.

When making a decision as to whether welding or bolting is to be used, the following are some of the points that should be considered [7]:

- Aesthetics: -Butt-welded connections are normally less obtrusive than bolted connections.
- Access:-Adequate and safe access is required for both methods of connection; but protection from wind and rain is also required for satisfactory welding.

- Temporary support: - The support of the member while the connection is being made has to be considered. This is particularly significant in a welded splice, where the location and alignment of the elements to be spliced must be maintained during welding. This often requires the use of temporary erection cleats and, if these are welded, the effect of the welding needs to be taken into account when making any fatigue checks (even if they are removed after erection).
- Corrosion: - Particular care is required to ensure that the corrosion protection prevents rusting between the plates in a bolted connection and that the weld area is properly cleaned before painting with a welded connection. Both types of connection should then perform adequately as far as resistance to corrosion is concerned.
- Details: - Bolted cover plate splices take up additional space, compared with butt welded splices. This could be a problem, for example, where deck plates are fixed to top flanges, particularly when a relative thin wearing surface is to be applied to the deck plates.
- Cost: - The cost of the various options should also be taken into account when making decisions regarding the type and position of connections.

***Welded Splices***

In addition to the pre-planning and care that is required to deal with the problems of temporary support, access, location and good fit consideration must also be given to the potential effects of weld shrinkage.

[7] These points can be illustrated by reference to the staggered connection shown in Figure 2.6 Alignment of the flange can be helped by omitting the flange-to-web welds for a short distance X on either side of the connection before assembly. The flange-to-web welds are completed after the other welds.

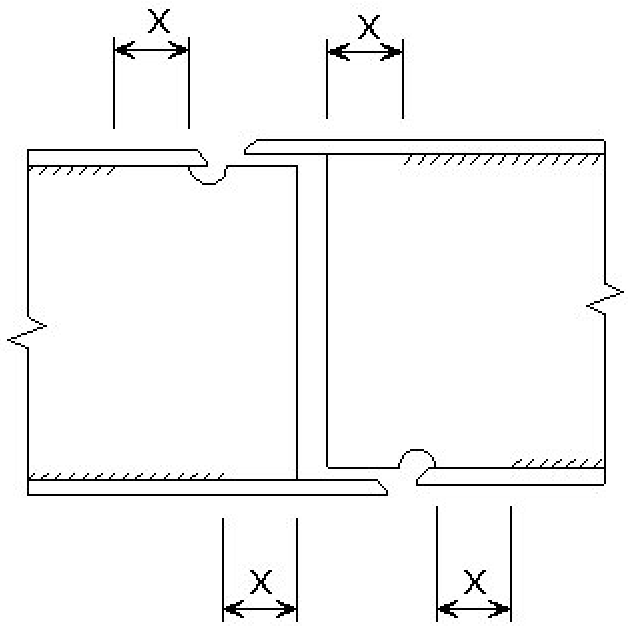



Figure 2.6: plate girder: welded splice

One method of reducing the effects of the shrinkage of the transverse welds is to complete the flange welds before carrying out the web welds. Due to the slenderness of the web, the shrinkage of the flange welds could cause buckling of the web if it is welded first. An alternative procedure is to make individual runs of weld on the flanges and web in sequence, starting with the flanges. This should tend to balance the shrinkages between the elements [7].

The cope holes in the web adjacent to the flange welds improve the access for welding the flanges and should result in a better stress pattern. Normally, the cope hole should not be filled in, although filling may be necessary for corrosion protection in box girders.

***Bolted Splices***

Untorqued, bearing bolts in normal (2mm) clearance holes are not generally used for splices in bridges. In most splices the deformation associated with slip into bearing would be unacceptable. To avoid the slip, fitted bolts, in close tolerance holes, or High Strength Friction Grip (HSFG) bolts are required. Generally HSFG bolts are used, since this avoids the need to match and ream the holes. The pre tensioning of the bolts also improves their fatigue life and prevents the nuts working loose due to vibration.

It is important that, when HSFG bolts are to be used, adequate clearances are provided to allow the use of suitable tightening tools.

A typical bolted cover plate splice is illustrated in Figure 2.7. It is possible that variations in the profiles of the two parts of the girder may occur due to rolling tolerances, differences in the overall depths and relative twisting or warping of the flanges. This may result in a mismatch such as that illustrated in Figure 2.8. The possibility of this occurring should be considered in the design of the splice and in the corrosion protection. Adding extra bolts, at the detailing stage, is a simple way of ensuring that the slip resistance would still be adequate if a mismatch should occur [7].

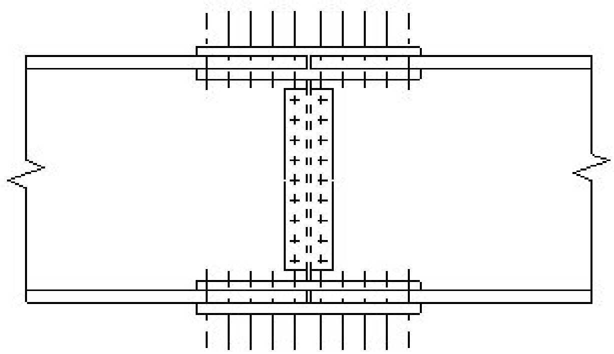


Figure 2.7: Plate girder: bolted splice

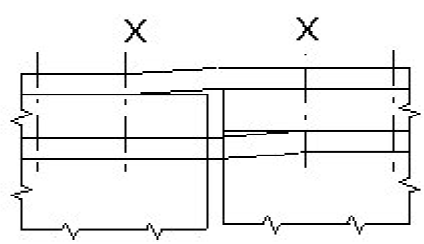



Figure 2.8: lack of fit in the flange

Shims or packs are used, for example, where there is a change of flange plate thickness. It is essential that the surfaces of the packs or shims comply with the requirements assumed for the faying surfaces in the design.

*Hybrid Splices*

A combination of welds and bolts could be used in a splice. A possible hybrid connection, for a light beam, is illustrated in Figure 2.9. In this splice the cover plates are each attached to one half of the beam in the fabricating shop and the bolting is used to complete the splice on site. One disadvantage of the connection is that each part requires both drilling and welding in the fabrication shop.

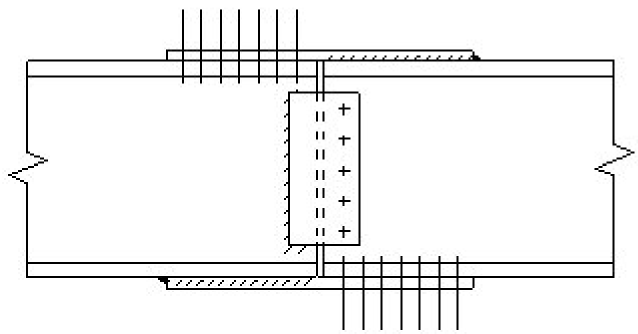


Figure 2.9: hybrid connection

If a combination of different types of bolt or of bolts and welds, with different load/deformation characteristics, is used in the same part of a connection, the load will tend to be carried by the stiffer connecting elements. Consequently, one type of bolt or weld should be assumed to transmit the entire load. An example of this would be if a bolted cover plate, using bolts in clearance holes, were to be "strengthened" by fillet welding the cover plate to the flange. The bolts would be ignored and the weld would have to be designed to carry the entire load. There is an exception to this rule which is that HSFG bolts designed as slip resistant at the ultimate limit state may be assumed to share the load with welds, provided that the final tightening of the bolts is carried out after the welding is complete [7].

As of Eurocode 3: Design of steel structures-part 1-8, failure modes and a design requirement for pinned and welded connections is given in table 2.3 below [8]:-

Table 2.3 Failure mode and design requirement

| Failure mode | Design requirements |
|---|---|
| Shear resistance | $F_{v,Rd} = 0.6Af_{up}/\gamma_{M2} \geq F_{v,Ed}$ |
| Bearing resistance of the plate and the pin<br>If the pin is intended to be replaceable this requirement should also be satisfied. | $F_{b,Rd} = 1.5tdf_y/\gamma_{M0} \geq F_{b,Ed}$<br>$F_{b,Rd,ser} = 0.6tdf_y/\gamma_{M6,ser} \geq F_{b,Ed,ser}$ |
| Bending resistance of the pin<br>If the pin is intended to be replaceable this requirement should also be satisfied. | $M_{Rd} = 1.5Welf_{yp}/\gamma_{M0} \geq M_{Ed}$<br>$M_{Rd,ser} = 0.8Welf_{yp}/\gamma_{M6,ser} \geq M_{Ed,ser}$ |
| Combined shear and bending resistance of the pin | $[M_{Ed}/M_{Rd}]^2 + [F_{v,Ed}/F_{v,Rd}]^2 \leq 1$ |
| $d$ is the diameter of the pin<br>$f_y$ is the lower of the design strengths of the pin and the connected part;<br>$f_{up}$ is the ultimate tensile strength of the pin;<br>$f_{yp}$ is the yield strength of the pin;<br>$t$ is the thickness of the connected part;<br>$A$ is the cross- sectional area of a pin. | |

**Failure mode for hollow section connections**

The design joint resistances of connections between hollow sections and of connections between hollow sections and open sections should be based on the following failure modes as applicable [8]:

a) **Chord face failure** (plastic failure of the chord face) or chord plastification (plastic failure of the chord cross-section):
b) **Chord side wall failure** (or chord web failure) by yielding, crushing or instability (crippling or buckling of the chord side wall or chord web) under the compression brace member:
c) **Chord shear failure:**
d) **Punching shear** failure of a hollow section chord wall( crack initiation leading to rupture of the brace members from the chord member):
e) **Brace failure** with reduced effective width( cracking in the welds or in the brace members):
f) **Local buckling** failure of a brace member or of a hollow section chord member at the joint location.

Although the resistance of a joint with properly formed welds is generally higher under tension than under compression, the design resistance of the joint is generally based on the resistance of the brace in compression to avoid the possible excessive local deformation or reduced rotation capacity or deformation capacity which might otherwise occur.

Thus the following figures illustrate the above failure modes [8]:-

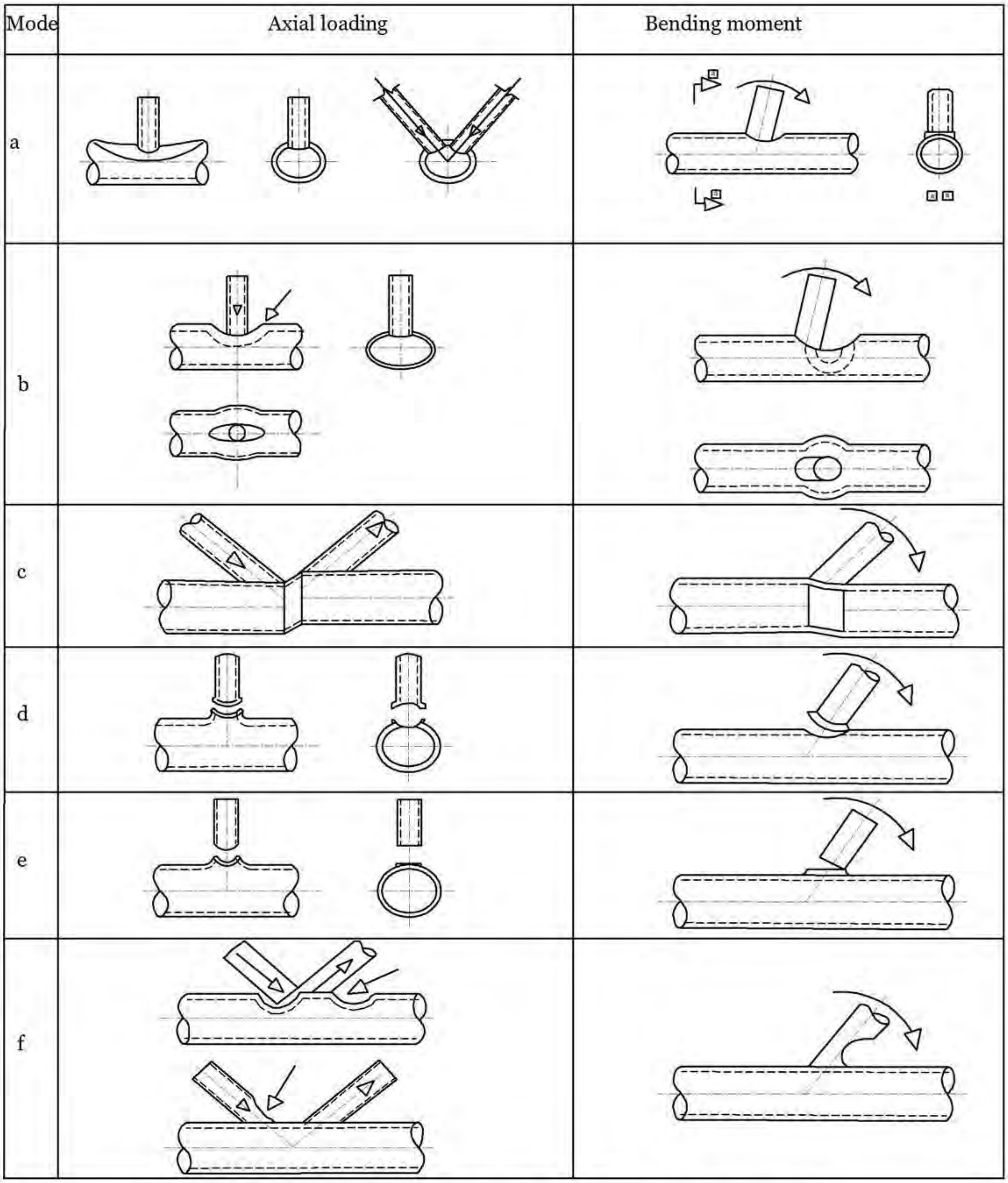



Figure 2.10:Failure modes for joints between CHS members

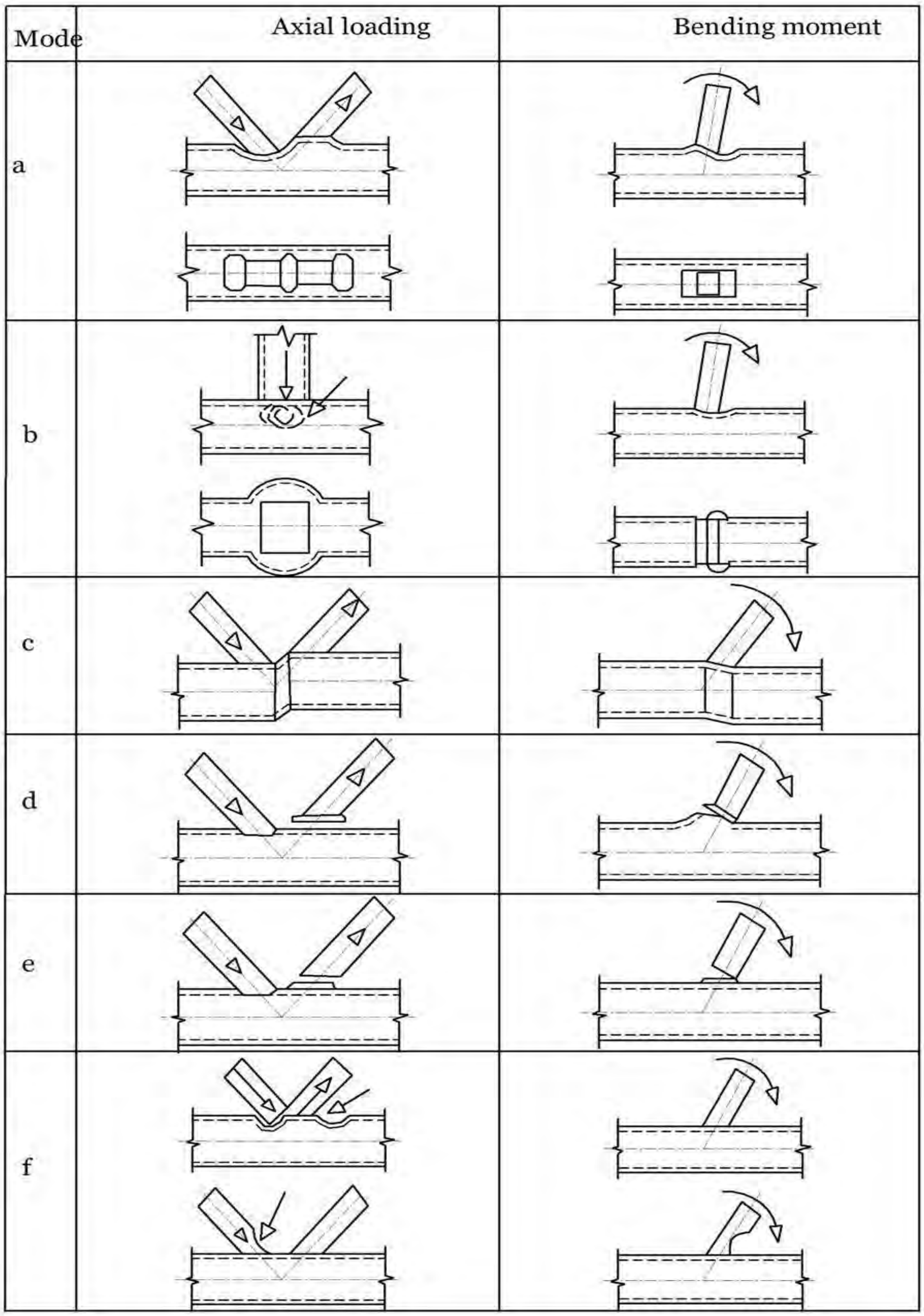



Figure 2.11:Failure modes for joints between RHS brace members and RHS chord members

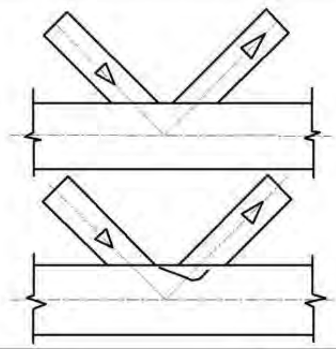
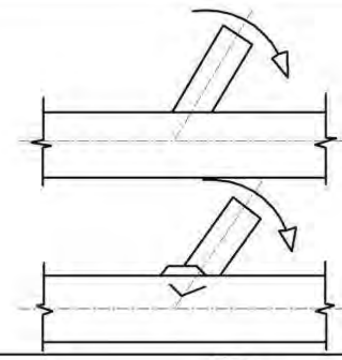
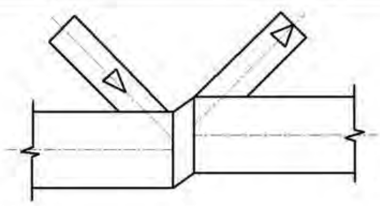
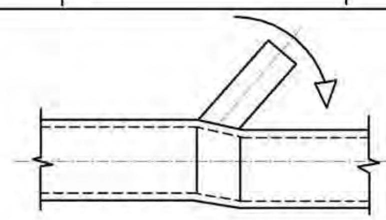
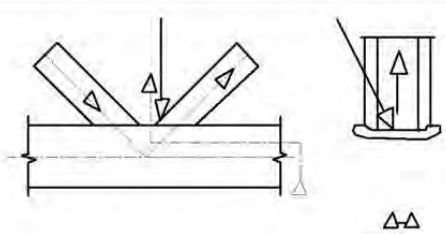
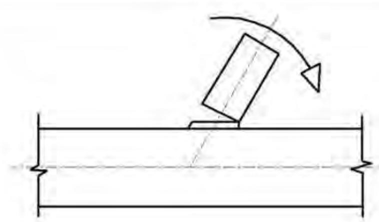
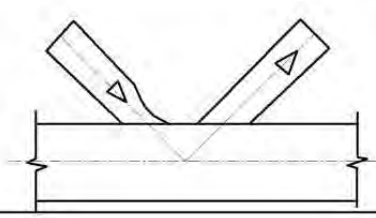
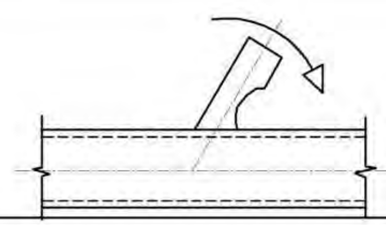
| Mode | Axial loading | Bending moment |
| --- | --- | --- |
| a | – | – |
| b |  |  |
| c |  |  |
| d | – | – |
| e |  |  |
| f |  |  |

Figure 2.12:Failure modes for joints between CHS or RHS brace members and I or H section chord members

### *2.5 Design Concept of Connections*

Structural engineering can never be exact science and design philosophies should recognize and accommodate this uncertainty. Even in a carefully controlled laboratory experiment, perfect correlation between behavior and analysis will not be achieved. In a practical structure the divergence between behavior and prediction is generally greater. In addition to environment, the degree of uncertainty also depends on the type of structure or structural element [9].

Uncertainties in connection behavior are frequently much greater, even in the laboratory. In a study of short end- plate beam/column connections measurements were taken of the prying forces that the design method predicted would develop between the end-plate and the face of the column. In one specimen no such forces developed- an error of 100%. This error was due to bad fit [9].

The conventional design method for beam splices, which assume that the web splice is subject to a shear force with a given eccentricity, cannot be modeling the true behavior of this connection, whichever method of analysis is used. In practice, there must be a complex interaction of moments and shears between flanges and web as differing parts of the connection reach their limiting capacity [10].

The main reasons why connection behavior is more uncertain and more complex than that of other steel elements include the following [9]:

***Geometric imperfections and lacks of fit***

All steel components contain geometric imperfections and lacks of fit but the differing degree of uncertainty between elements and connections is mirrored in their varying degree of imperfection. The significant imperfection in abeam or column is a bow or twist with a maximum permitted amplitude of length/100.

***Residual stresses and strains***

Almost all steelwork contains residual stresses, that is, sets of self-equilibrating stresses that are locked in during manufacturing and fabrication. A stiffened plate panel is likely to have compressive residual stresses of 0.2 x $\sigma_y$ and tensile residual stresses of $\sigma_y$. A hot rolled section may have residual stresses of 0.3 x $\sigma_y$ in compression and tension. These two situations correspond to elastic strain incompatibilities of 0.15% and 0.08%, respectively.

***Geometric complexity***

It is axiomatic that there is greater geometric complexity within a connection than along the length of a structural member. This complexity has two important influences on connection behavior. It causes considerable elastic stress concentrations within the connection.

Any rational design philosophy has to take account of both the complexity and variability of practical connection behavior. It is the uncertainty that presents the greater difficulty. If connection behavior was merely complex it would be possible for designers to make a suitable choice for a particular situation. Detailed analysis could be used that took account of the complexity, thus permitting economic design with small load factors. Alternatively, simple analysis could be used in conjunction with higher load factors to account for possible variations between analysis and behavior. Because of the variability of behavior the first alternative cannot lead to economic and safe design. However refined the analysis, it would not be appropriate to reduce the load factors significantly.

The following design philosophy, based on simple analysis, would seem to be the most appropriate way of dealing with the problem. This approach has been developed over many years, based on the experience of connection design and research into connection behavior. As put forward here in its most general form it is widely applicable [9].

1) *Taking account of overall connection behavior, carry out an appropriate simple analysis to determine a realistic distribution of forces within the connection.*

For many connections this analysis should be based on the concept of 'force paths'. Here the overall loads acting on the connection are replaced by equivalent systems of forces which can then be assigned specific paths through the connection. In carrying out this analysis take account of:

a) The distribution of forces in the elements to be connected. For example, if the connection involves a beam carrying shear and moment, then remote from the connection the shear will be concentrated in the beam web and the flanges will carry most of the moment. In many instances this will be a satisfactory basis for the analysis of the forces within the connection. Indeed, it is a common simplifying assumption that the flanges carry all the moment; this is quite satisfactory provided it does not lead to an overstress in the flanges.

b) The flexibility of the components of the connection. It is the most flexible components that will govern the distribution of forces. For example, in an end plate connection, if the bolts are of small diameter and the end plate is thick, it is the bolt flexibility that will govern the distribution of forces-as is indicated by conventional analysis. However, if the bolts are stiff compared to the end plates it is the flexural action of the latter that will primarily govern the distribution of forces in the connection, including the distribution of forces in the bolts.

It follows from the above that the conventional methods of analysis may be used in the context of this overall philosophy. They are most appropriate when the dominant flexibility is that of the connectors themselves.

It is most important to ensure that the analysis is consistent throughout the connection. In general, this is achieved by carrying out a single analysis of the most critical part of the connection and using that to determine the distribution of forces in other parts of the connection. Surprisingly, it is not uncommon to see designs where serious inconsistencies in analysis have occurred. The most commonly arise when more than one analysis has been used to determine the distribution of forces [9].

2) *Ensure that each component of each force path has sufficient strength to transmit the required force.*

This is self –evident, and yet it is surprising how frequently designers leave a weak link somewhere in a connection. A major disadvantage of traditional methods of analysis is that they concentrate on the distribution of forces in the connectors. Many codes of practice only give guidance on connector strength. Unwary on inexperienced designers are thus lured into thinking that, provided they have checked the bolts and/or welds, the connection is satisfactory. In reality, more design effort has frequently to be devoted to the other components than to the connectors themselves.

The only way to be certain that a design is satisfactory is for designers to have a clear understanding of how they wish the connection to behave and for them to ensure that all the components and critical sections have the capacity for this mode of behavior [9].

3) *Recognizing that the above procedure can only give a connection where equilibrium is capable of being achieved but where compatibility is unlikely to be satisfied, ensure that the components are capable of ductile behavior*

This may be expressed alternatively as follows. Steps 1 and 2 have ensured that there is a reasonable way in which the connection can behave and hence have adequate strength. Ductility must now be ensured so that it will attain this condition without any rupture or buckling.

The incompatibilities may arise either from simplifications of the analysis or because of some lack of fit; their cause is not important. However, because of their possible presence, it is essential that the connection is capable of sufficiently ductile behavior for plastic deformation to remove them prior to failure. Provided that this precaution is taken, even if it has not been possible to predict elastic response accurately, the connection will redistribute forces until it is acting in the way that was assumed in design. Fortunately, it is usually a straightforward matter to ensure that the components can achieve the necessary ductility [9].

4) *Recognizing that the preceding steps only relate to static ultimate capacity, ensure that the connection will achieve satisfactory serviceability, fatigue resistance, etc.*

For connections in buildings that have been designed by conventional elastic approaches this step may generally be omitted. However, in the following cases further calculations will be necessary:

a) Where either overall analysis or individual component design has been based on simple rigid-plastic methods it will be necessary to ensure that only limited plastic deformation has taken place at working levels.

b) Where the connection is subject to significant repeated loading, a separate assessment of fatigue resistance should be carried out. This can create considerable difficulties because it requires both detailed consideration of the elastic response of the connection and an evaluation of important stress concentration factors. In extreme circumstances (for example for the case of tubular connections) design for fatigue resistance should govern the overall design procedure and the sequence outlined above should be reversed. In less extreme circumstances (for example, cross- girder connections in bridges) static strength should still govern overall design but the connection layout should be arranged to minimize stress concentrations because of the importance of fatigue considerations [9].

### *2.6 Detailing concept of Connections*

Society rightly demands a high standard of safety from civil engineering structures. When structure fails, it may claim many lives, and its reinstatement may require considerable resources. Structures rarely fail from a single cause; there are usually several contributing factors to failure. Structures are frequently at greatest risk during construction and structures may fail due to in one or two of the reasons [11]:

- Poor communication
- Design error or lack of understanding of structural behavior.
- A material related problem causing failure in a structure even though its behavior is reasonably well understood by the designer.
- Errors in detailing or poor detailing rules caused by lack of understanding or checking.
- Inadequate temporary works, lack of thoughts about a temporary condition or about the process of erection.

Failure is by no means the prerogative of ignorance or incompetence. Even in routine work according to recognized codes, failure is more often the consequence of a rare lapse which team work and vigilance have for once failed to remedy. This lapse may be compounded by ill-luck, by inadequate consideration of the fundamental behavior of the proposed structure, by safety margins too small to allow for human fallibility, and by in exact methods of calculation or construction. Success or failure is ultimately the work not of codes but people; the engineer and his team are responsible for both effects.

Again Failure is defined as the onset of unacceptable deflections [12].

These divided into:-

a) The occurrence of excessive or uncomfortable deflections under service loads or

b) The collapse of the structure, where the bridge deck falls to the ground.

Failure is generally a consequence, not only of the geometry of the structure, but also the nature of the material of which it is composed.

So poor detailing of connections will have failure effect of the system due to the mechanism of failure propagation.

### *2.7 Reliability Design of Connections*

Reliability is defined in several ways [12]:

- The idea that something is fit for a purpose with respect to time;
- The capacity of a device or system to perform as designed;
- The resistance to failure of a device or system;
- The ability of a device or system to perform a required function under stated conditions for a specified period of time;
- The probability that a functional unit will perform its required function for a specified interval under stated conditions.
- The ability of something to "fail well" (fail without catastrophic consequences)

Reliability theory is the foundation of reliability engineering. For engineering purposes, reliability is defined as: the probability that a device will perform its intended function during a specified period of time under stated conditions. Mathematically, this may be expressed as,

$$R(t) = Pr\{T>t\} = \int_t^{\infty} f(x)\, dx$$

Where $f(x)$ is the failure probability density function and

$t$ is the length of the period of time (which is assumed to start from time zero).

Reliability engineering is concerned with four key elements of this definition:

- First, reliability is a probability. This means that failure is regarded as a random phenomenon: it is a recurring event, and we do not express any information on individual failures, the causes of failures, or relationships between failures, except that the likelihood for failures to occur varies over time according to the given probability function. Reliability engineering is concerned with meeting the specified probability of success, at a specified statistical confidence level.
- Second, reliability is predicated on "intended function:" Generally, this is taken to mean operation without failure. However, even if no individual part of the system fails, but the system as a whole does not do what was intended, then it is still charged against the system reliability. The system requirements specification is the criterion against which reliability is measured.
- Third, reliability applies to a specified period of time. In practical terms, this means that a system has a specified chance that it will operate without failure before time t. Reliability engineering ensures that components and materials will meet the requirements during the specified time. Units other than time may sometimes be used. The automotive industry might specify reliability in terms of miles; the military might specify reliability of a gun for a certain number of rounds fired. A piece of mechanical equipment may have a reliability rating value in terms of cycles of use.
- Fourth, reliability is restricted to operation under stated conditions. This constraint is necessary because it is impossible to design a system for unlimited conditions. Many tasks, methods, and tools can be used to achieve reliability. Every system requires a different level of reliability. A reliability program plan is used to document exactly what tasks, methods, tools, analyses, and tests are required for a particular system. The reliability program plan is essential for a successful reliability program and is developed early during system development [12].

***Reliability modeling***

Reliability modeling is the process of predicting or understanding the reliability of a component or system. Two separate fields of investigation are common: The physics of failure approach uses *an understanding of the failure mechanisms involved, such as crack propagation or chemical corrosion*; The parts stress modeling approach is *an empirical method for prediction based on counting the number and type of components of the system, and the stress they undergo during operation* [12].

For systems with a clearly defined failure time (which is sometimes not given for systems with a drifting parameter), the empirical distribution function of these failure times can be determined. This is done in general in an accelerated experiment with increased stress.

***Design of reliability***

Design for Reliability (DFR), is an emerging discipline that refers to the process of designing reliability into products. This process encompasses several tools and practices and describes the order of their deployment that an organization needs to have in place to drive reliability into their products. Typically, the first step in the DFR process is to set the system's reliability requirements. Reliability must be "designed in" to the system. During system design, the top-level reliability requirements are then allocated to subsystems by design engineers and reliability engineers working together [12].

## *2.8 Design procedure considerations of Connections*

**Reliability Considerations**

Limit states for connection elements are arrived at in a similar fashion as those for main-member design [15]. Limit states that could result in sudden, fracture-type failures are required to have greater safety factors, or greater reliabilities, than limit states associated with yielding. Bolt and weld failures are treated as fracture-type failures, and are therefore required to be designed at the higher reliability level. Plates, angles, and other connection elements are designed to reliabilities based on the individual modes of failure in the same way that main members are designed. Generally, connections are not required to be designed to a higher reliability than the members they connect.

Proposed connection design method shall be justified in the following ways [13]:-

a) ***Through precedence***: Precedence simply means that there is sufficient historical record of adequate performance of a connection configuration or an assumption to justify its use. Many valid arguments can be made against accepting a connection design method based on precedence. Because of the conservatism built into design loads and load factors and the fact that loads can often redistribute, connections in service may rarely see their full design loads. Therefore, a history of satisfactory service may not correlate directly to a safe design. However, some assumptions implicit in AISC, "Manual of Steel Construction," 2005 (referred to herein as the AISC Manual), are based largely on precedence. For instance, the bolts at the supported member of a double-angle connection are typically not designed to resist any eccentricity, though logically an eccentricity could exist. An argument can be made that the flexing of the angles relieves the eccentricity, and therefore the bolts do not have to be designed to resist this rotation. However, the support must now take this neglected moment. The argument can then be made that the eccentricity is small, and the supporting member probably has some excess capacity. All of these are qualitative arguments with little analytical basis. The only real justification that can be found to support this assumption is decades of satisfactory performance. Precedence should not be overlooked as a valid justification for engineering practices, but it must be used with caution and must be evaluated whenever paradigm shifts occur in design philosophies, especially when these shifts involve load determination or resistance factors.

b) ***Testing:*** This approach has been used to develop a handful of essentially prescribed connections, the standard single plate shear connection being the most notable. For many, this approach may be considered the "gold standard" for justifying a connection design, but it requires a great deal of financial investment, sometimes with relatively little return, since results are often valid only for a range of strictly defined parameters. Greater benefits from testing are more often achieved when an analytical model can be found to predict the results of testing. This analytical model can then be applied to a wider range of conditions. Often testing is performed to determine the effects of a single limit state. These data are then used to develop a model for use with more complex conditions.

c) ***Analysis:*** The final and most common way to justify a connection design is through analysis. Precedence, testing, and engineering theory and judgment are coalesced to produce a rationale to justify the connection to be used. This is the art of connection design. Simple tests are extrapolated to more complex configurations. Load paths are analyzed and optimized. Assumptions are scrutinized to ensure their validity. In some cases these procedures are clearly codified. In many others they are not. The tools are essentially the same as those used in main-member design: statics to satisfy equilibrium, mechanics of materials to confirm strength and determine load paths, and statistical analysis to determine reliability. When combined with sound engineering judgment, these tools allow the connection design engineer to provide safe and economical connections for structural steel.

**Economic Considerations**

For any given connection situation, it is usually possible to arrive at more than one satisfactory solution. Where there is a possibility of using bolts or welds, let the economics of fabrication and erection play a role in the choice. Fabricators and erectors in different parts of the country have their preferred ways of working, and as long as the principles of connection design are followed to achieve a safe connection, local preferences should be accepted. Some additional considerations which will result in more economical connections are as follows [13].

**1.** For shear connections, design for the specified factored loads and allow the use of single-plate and single-angle shear connections. Do not specify full-depth connections or rely on the AISC uniform load tables.

**2.** For moment connections, design for the specified factored moments and shears. Also, provide a "breakdown" of the total moment, that is, give the gravity moment and lateral moment due to wind or seismic loads separately. This is needed to do a proper check for column web double plates. If stiffeners are required, allow the use of fillet welds in place of complete joint-penetration welds. To avoid the use of stiffeners, consider redesigning with a heavier column to eliminate them.

**3.** For bracing connections, in addition to providing the brace force, also provide the beam shear and axial transfer force. The transfer force is the axial force that must be transferred to the opposite side of the column. The transfer force is not necessarily the beam axial force that is obtained from a computer analysis of the structure. A misunderstanding of transfer forces can lead to both uneconomic and unsafe connections [13].

**Computational Considerations**

As it is often observed from computation of the connection analysis and design; every situation of inability for transfer of actions needs be checked intensively so that all the capacity of the parts shall be a little more than the force systems being transferred [2].

And in the computation process, we can deduce that it is on the benefit of having economic plate and connection parts for every connected long spanned bridge.

# 3.0 Construction Practices of Long Span Steel Truss Bridges

## *3.1 Background*

This chapter addresses some of the principles and practices applicable to the construction of medium and long-span steel bridges; structures of such size and complexity that construction engineering becomes an important or even the governing factor in the successful fabrication and erection of the superstructure steelwork.

Construction engineering is not so well known [14]. It involves governing and guiding the fabrication and erection operations needed to produce the structural steel members to the proper cambered or "no-load" shape, and get them safely and efficiently "up in the air" in place in the structure, so that the completed structure under the dead load conditions and at normal temperature will meet the geometric and stress requirements stipulated on the design drawings.

The construction phase of the total life of a major steel bridge will probably be much more hazardous than the service- use phase. Experience shows that a large bridge is more likely to suffer failure during erection than after completion. Many decades ago, steel bridge design engineering had progressed to the stage where the chance of structural failure under service loadings became altogether remote. However, the erection phase for a large bridge is inherently less secure, primarily because of the prospect of inadequacies in construction engineering and its implementation at the job site [14].

## *3.2 International Practice*

The practices done in construction industry are fabrication, assembling and then erection. There are a number of methods for the erection of bridges [7]. These include the lifting of the whole bridge (or the main girders) in one piece, the use of false work to support the parts while the site splices are made, sliding the bridge into position using jacks and winches and free-cantilevering.

The size of the parts shipped to site depends on the capacity of the fabrication shop, the transport facilities and the erection equipment on site. The maximum size of a part to be transported is typically up to 4.5m wide, up to 24m long and up to 40 tons in weight.

Where larger pieces can be lifted or moved into position on site than can be handled in the fabrication shop (or transported to site) it is often advantageous to sub-assemble the parts into larger pieces on site before final erection.

The cost of shipping (transportation) is frequently a function of the volume occupied, as well as the weight. Consequently, when shipping long distances, the parts of, say, a truss girder may be shipped as individual members and then be assembled on site to form the truss. The designer would need to consider this requirement when designing and detailing the truss.

A trial assembly of the bridge (or adjacent parts of the bridge if it is large) at the fabrication works may be advisable to ensure that unnecessary problems of alignment and lack of fit do not occur on site [7].

**Methods of Erection**

It has long been appreciated that a designer must consider at the design stage the method by which a bridge will be erected [7]. Indeed it is not infrequently the case that such consideration should be made even at the time of conceptual choice, since it can happen that the superficially most attractive design is impossible to erect in a particular location. For example,  a design that relies on being erected in large pieces (such as a major box girder), may be ruled out because of the impossibility of transporting such pieces to a remote site with inadequate access roads.

Frequently, particularly on large structures, it is possible to adjust the distribution of moment and forces in a structure by choosing a particular erection sequence. This possibility can affect the conceptual choice, e.g. the designer of a major three span estuarial crossing with a central span of 200m, may consider that the best conceptual choice is a steel box girder hunched at the internal piers thus carrying high hogging bending moments at these piers and comparatively low sagging moments at mid span. However, the most convenient erection method for this site may be to float out and lift most of the central span in one piece, thus causing high dead load sagging moments at mid span. By building the end supports high and jacking the ends of the bridge down after the connection of the main span is made, an overall hogging moment can be induced to counter the unwanted sagging moment. Certainly this solution requires very careful analysis, and calculation of fabrication and pre-cambering dimensions to obtain the correct carriageway profile, but at least the concept will be right!

Many methods of erection of steel bridges exist; five typical ones are [7]:

- Assembly in situ
- Launching
- Lifting
- Cantilevering
- Sliding or rolling-in.

Combinations of these methods are possible.

**a) Assembly in situ**

This method involves assembling the bridge from its individual components or sub-assemblies in its final position, usually on false work or some other form of temporary support, making the site splices and removing the false work. Adequate cranage must be provided to cover the whole of the deck area. The presence of false work may temporarily block a road, railway or river over which a bridge is built. Because large individual pieces are not normally involved, it is a method which may be practicable when access to a site is difficult. Assembly in situ may be used in conjunction with other methods of erection.

**b) Launching**

This method involves assembling a bridge on rollers, slide-tracks or skates on its final alignment but at the side of the obstacle to be crossed. When complete it is pushed or pulled forward to cross the obstacle and land on bearings on the far side (Figure 3.1).

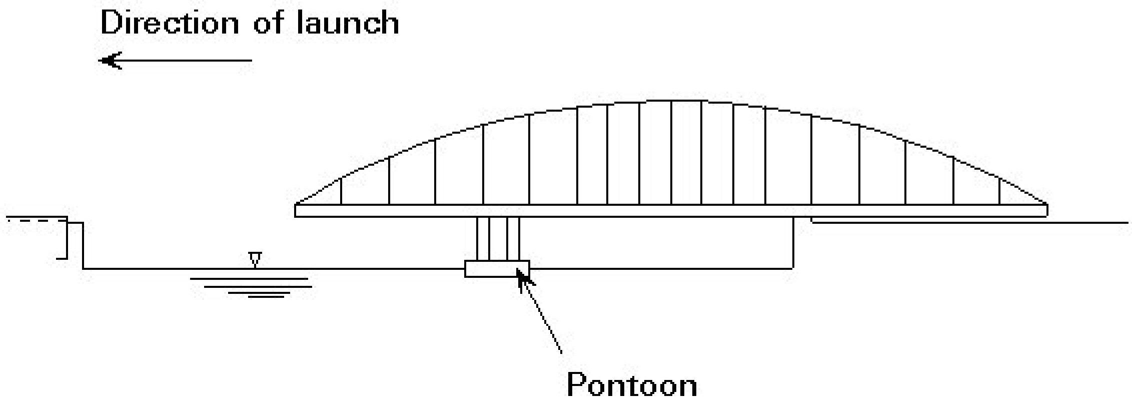



Figure 3.1: Typical launching arrangement using pontoon

Whilst simple in principle, launching requires a site where large pieces of the bridge can be constructed in line with the final position but on the shore. The operation also requires very careful control and detailed analysis since, at various stages, bridge sections may be subjected to loadings differing greatly from those in service.

**c) Lifting**

This method involves lifting a self-supporting part or the whole of a bridge into or near to its final position (Figure 3.2). Pieces lifted can vary from a small footbridge weighing a few tons, to a large section of a major crossing weighing over 1000 tons. Lifting may be a complete operation in itself, or part of a cantilever erection scheme.

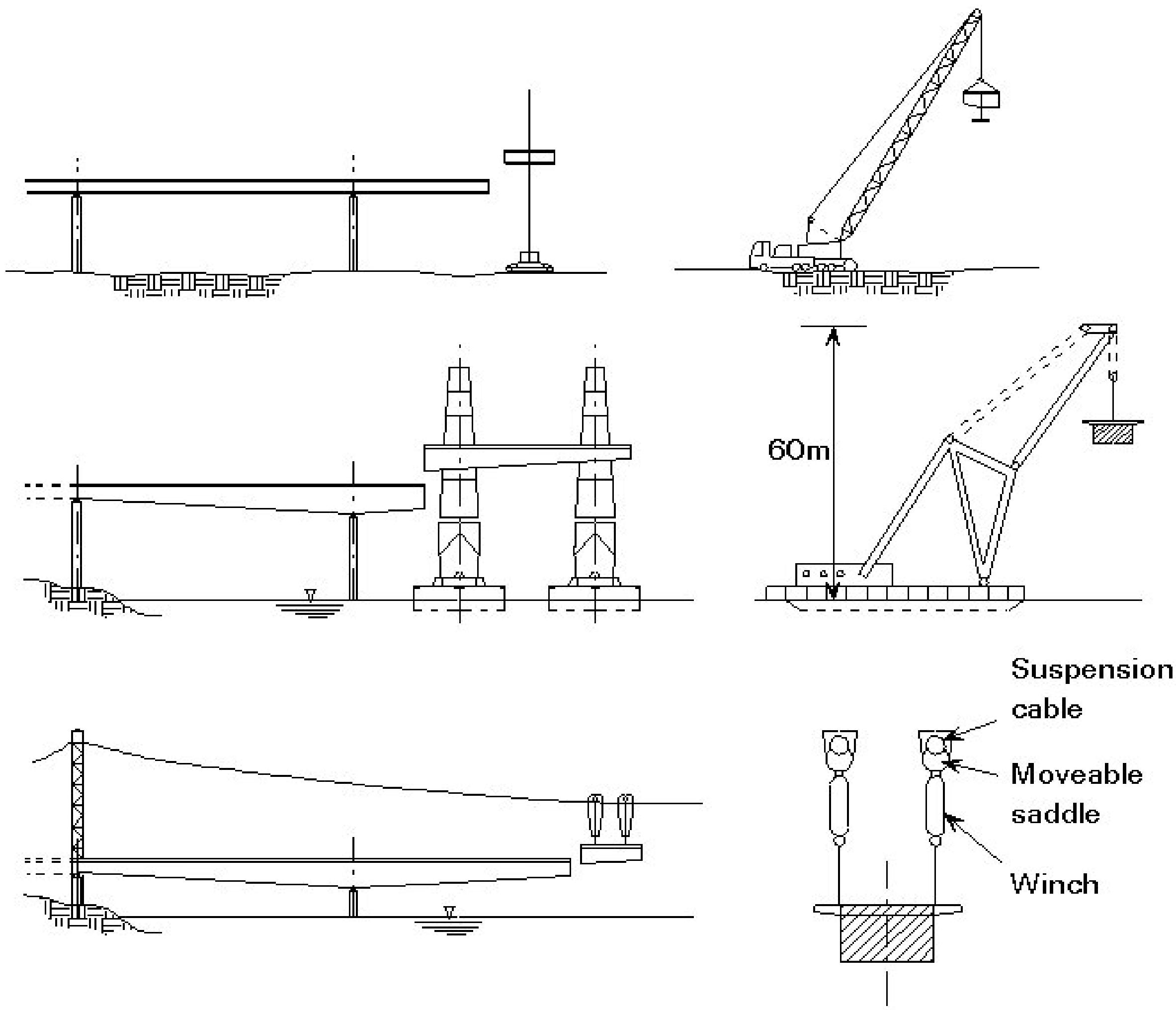



Figure 3.2: Typical methods of lifting bridge sections

Lifting plant may range from small cranes for minor bridges, to very large floating cranes for major parts of estuarial bridges; alternatively winches or jacks on the already erected part of the bridge may be used. Hence, the position and topography of the site will have a significant effect on the conceptual choice.

**d) Cantilevering**

This method involves constructing a bridge, normally continuous over several spans, progressively from one or both abutments, by attaching sections to the end of already erected portions (Figure 3.3). An anchor span is lifted or assembled in situ, and sections then cantilevered from this by either lifting from ground level, or running along the deck and lowering from the end. The position of the site and the access to it will determine the size of the pieces erected and this in turn will have a bearing on the original choice of the structural concept. Cantilevering is an ideal method for erecting cable-stayed bridges, using the stays as supports for the cantilever as work progresses.

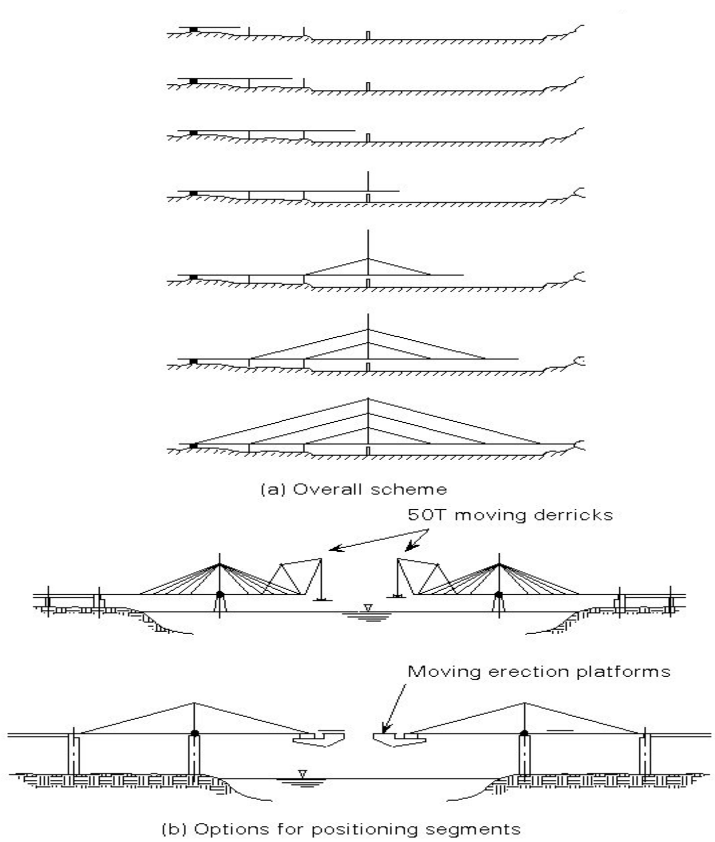



Figure 3.3: Typical arrangement of cantilevering

**e) Sliding or rolling-in**

This method involves building the bridge offset laterally from the final site and then jacking and winching it sideways into its final position. It is typically used for replacing an existing bridge which cannot be taken out of service for a long period. For obvious reasons it is only possible to use it for a very strictly limited type of site [7].

### *3.3 The Ethiopian Practice*

There is a lack of definite documented set of principles or methods to a long span steel bridge construction and even there is no definite procedure of adaptation to the country's case; it is only an adoption of construction techniques or practices from different countries; In fact this adoption has an effect on the life of the structure and the economy of the country.

But, the construction engineering of the country has experiencing different methods for the fabrication, erection of long spanned bridges, Bailey bridges, and ware houses and every methods of the construction incorporates the basic principles of mechanics i.e. Static and dynamic equilibrium for the truss element by balancing the overturning effects and also by following the basic customs of international practices.

The erection method of Callendar-Hamilton Bridges are capable of being built by several well proven techniques- the method adopted being dependent upon such factors as site conditions, span lengths and crane age availability. The three main methods of constructions most commonly used are:-

a) Construction in-situ:

Span built on top of temporary works e.g. scaffolding.

b) Cantilever construction:

The span is built out from a temporary anchor tail on the abutment approach. The bridge is gradually built from the anchor until reaching the far abutment.

c) Launch construction :

In this method the bridge is constructed behind a light weight Launch Nose and is pushed/ pulled into place, the bridge being supported on rollers or sliding skates.

Figure 3.4: Tekeze River steel truss bridge

Figure 3.5: Bailey bridge around Dukem.

# 4.0 Analysis of Long Span Steel Truss Bridge.

## *4.1 Loadings*

### 4.1.1 The Role of case studies in bridge design.

Bridge design, even of highway overpasses, often involves standard problems but always in different situations. Case studies can help in the design of these standard problems by showing models and points of comparison for a large number of bridges without implying that each such bridge be mere imitation [15].

The primary goals of a case study are to look carefully at all major aspects of the completed bridge, to understand the reasons for each design decision, and to discuss alternatives, all to the end of improving future designs. Such cases help to define more general ideas or principles. Case studies are well recognized by engineers when designing for acceptable performance and low cost; they can be useful when considering appearance as well.

A common organization of these studies will help identify standard problems and make comparisons easier. First comes to an overall evaluation of the bridge as a justification for studying it. Is it a good example that can be better? Is it a model of near perfection? Is it a bad example to be avoided?, Second comes a description of the complete bridge, which is divided into parts roughly coinciding with easily identifiable costs and including modifications to each part as suggested improvements. Is this major description section? , there is an order to the parts that implies a priority for the structural engineer: concept and form of the entire structure, superstructure, supports, deck, and landscaping.

Third, the case study can give a critique of the concept and form by comparison with other similar bridges or bridge designs for similar conditions, including those with very different forms, as a stimulus to design imagination.

Fourth and finally, the case should conclude with some discussion of the relationship of this study to a theory of bridge design. Clearly, any such study must be based upon a set of ideas about design which often implicitly bias the writer who should make these ideas explicit. This conclusion should show how the present study illustrates a theory and even at times forces a modification of it. General ideas form only out of specific examples [15].

### 4.1.2 Assessment (Description) of the Bridge.

Site visit was not conducted on existing Omo bridges in Southern region of the country but consulted Ethiopian Roads Authority (Client) and appreciate the design and construction challenge. During the consultation of the bridge engineering department of the Client; the picture shown below is taken.

Figure 4.1: Omo river bridge 3D view (callendar- Hamilton Bridge)

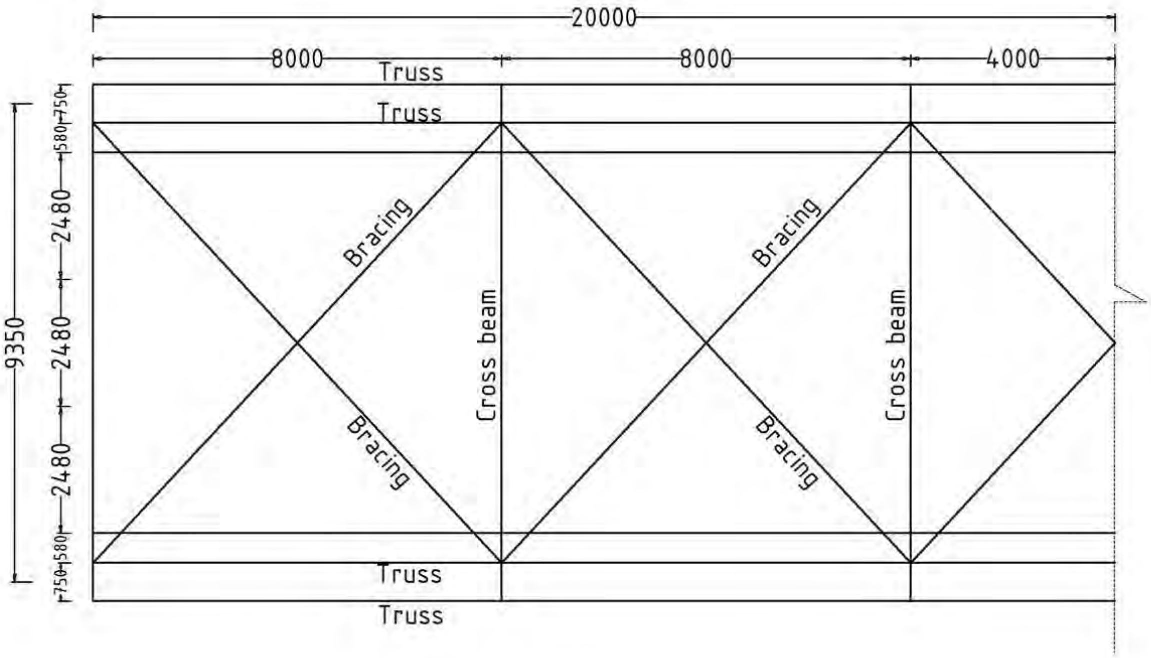



Figure 4.2: Top chord layout of Omo River Bridge.

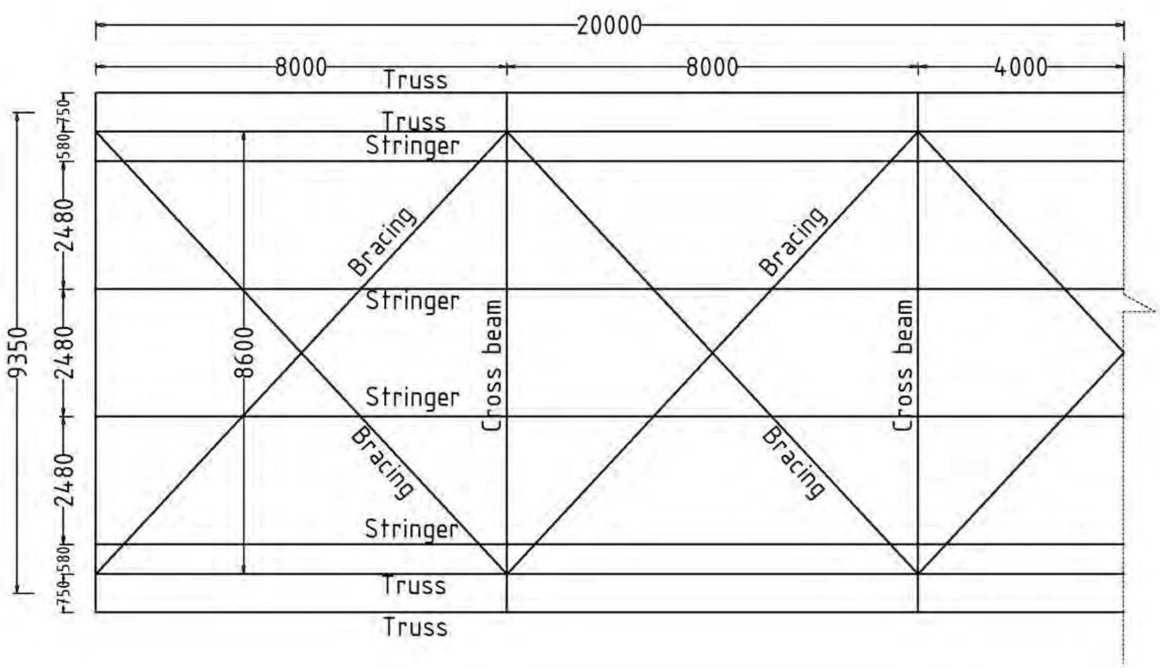



Figure 4.3: Bottom chord layout of Omo River Bridge

Bridge Location**:** At Southern Nations People, Peoples Republic, Jinka area.

Crossing: Omo River

Bridge Type: Callendar- Hamilton with steel plate deck:

*Callendar-Hamilton Bridge System:* The modern day prefabricated Panel/Floor Beam/Deck system was first patented by A.M. Hamilton in 1935. The bridge was used for quick mobilization to allow military access to remote locations or to replace destroyed bridges in times of conflict. The design was centered on a series of gusset plates that allowed the direct attachment of the longitudinal, diagonal, vertical, and cross framing members. The centralizing of connection points increased the speed of construction and also allowed identical panels to be fabricated from identical members and then installed on site. Figures 4.4 and 4.5 are original design drawings as recorded by the U.S. Patent and Trademark Office. This system is currently known as the Callendar-Hamilton System.

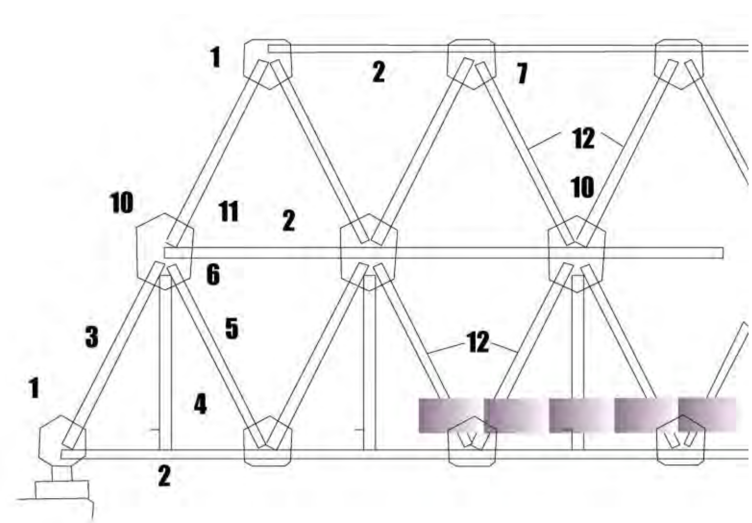



Figure 4.4: A.M.Hamilton Patent Information, Elevation.

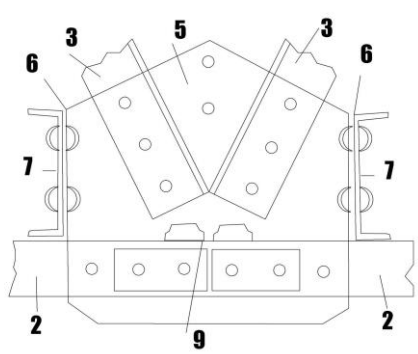



Figure 4.5: A.M.Hamilton Patent Information, Gusset Plate Detail.

Since the gusset plate carried the direct attachment of the vertical, diagonal, and cross members, the lateral stiffness carried by the floor beams is isolated and thereby increased. The members and connection points are modular in that many similar components could be erected to meet various applications. Truss panels that are stacked on top of each other can easily be attained by attaching two prefabricated gusset plates together, forming a central location for all connection members [2].

Number of spans**:** 3

Span Length**:** 32.00m, 64.00m and 128.00m

Roadway width**:** 9.35m

Truss height: 8.60m

Sub-structure**:** concrete column piers

#### 4.1.3 Assigned loads for analysis

Loads are generally assumed to be applied at the intersection points of the truss members, so that they are principally subjected to direct stresses [4]. And loads are assigned in the model of finite element software.

### *4.2 Analysis of member internal forces*

To simplify the analysis the weights of the truss members are assumed to be proportioned to the top and bottom chord panel points and the truss members are assumed to be pinned at their end, even though this is usually not the case. Normally chords are continuous and the connections are either welded or contain multiple bolts; such joints tend to restrict relative rotations of the members at the nodes and end moments develop [4]. However, the axes of the members at a joint are concurrent.

### *4.3 Design of Connections*

There are basically three types of connections used for connecting truss elements to each other, that is, welding, bolting and riveting.

Generally in steel construction bolted site splices are much preferred to welded splices for economy and speed of erection. And bolted connections are more widely used in bridge trusses, particularly medium to large span road bridges and rail way bridges, due to their improved performance under fatigue loading [4].

Gusset plates enable the incoming members to be positioned in such a way that their centroidal axes meet at a single point thus avoiding load eccentricities. Ideally for all types of trusses the connections should be arranged so that the centres of gravity of all incoming members meeting at the joint coincide. If this is not possible the out-of-balance moments caused by the eccentricities must be taken into account in the design.

### 4.3.1 Design of welded connections

The required strength of a connection is determined from analysis of the entire structure with factored loads acting on it. A detailed analysis of the connection produces required strengths for its components. The components of connections are the connectors (i.e., welds and pins) and the connecting elements (plates or the members themselves).

According to [16] article 10.19.1.1,and [8] article 10.3.2 connection shall be designed for not less than the average of the required strength at the point of connection and the strength of member at the point but in any event not less than 75% of the strength of the member . Groups of welds that transmit axial force into a member should preferably be proportioned so that the center of gravity of the group coincides with the centroidal axis of member. Of the different types of welds, fillet welds are chosen for connecting members. The design strength of welds is the lower value of

$$\phi F_{bm} A_{bm} \text{ and } \phi F_w A_w$$

Where,

$F_{bm}$ = nominal strength of the base material, kN/m$^2$

$F_w$ = nominal strength of the weld electrode, kN/m$^2$

$A_{bm}$ = cross sectional area of the base material, m$^2$

$A_w$ = cross sectional area of the weld, m$^2$

$\phi$= resistance factor

Values for $\phi$, $F_{bm}$, and $F_w$ are given in Table 4.1 [16].

Table 4.1 Values for $\phi$, $F_{bm}$, and $F_w$

| Fillet welds | | | | |
|---|---|---|---|---|
| Types of weld or stress | Material | Resistance factor, $\phi$ | Nominal strength, $F_{bm}$ or $F_w$ | Required weld strength level |
| Stress on effective area | base weld electrode | 0.75 | $0.6F_{Exx}$ | Weld metal with a strength level equal to or less than "matching" weld metal may be used. |
| Compression or tension parallel to axis of weld | Base | 0.9 | $f_y$ | |

Where, $F_{EXX}$ = nominal tensile strength of the weld metal, kN/m$^2$

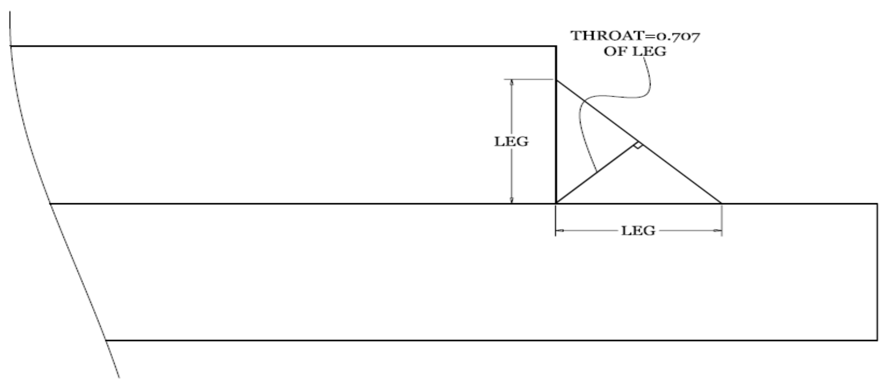



Figure 4.6: Weld throat

$A_w$ = (length of weld) (throat)

The throat length is shown in Fig. 4.6

For fillet weld the minimum weld size is determined by the thicker of the two parts joined.

The minimum size of fillet weld is given in Table 4.2 below [17].

Table 4.2 Minimum size of fillet weld

| Material thickness of the thicker parts joined ,t , mm | Minimum size of fillet weld,* mm |
|---|---|
| $t \leq 6.35$ | 3.18 |
| $6.35 < t \leq 12.70$ | 4.76 |
| $12.70 < t \leq 19.05$ | 6.35 |
| $t \geq 19.05$ | 7.94 |

* Leg dimension of fillet welds

### 4.3.2 Design of bolts and plates

The capacity of pin connection shall be determined from the shear capacity of the pin at the shear plane and the bearing capacity on each connected ply. The bending moment on pin shall also be checked [18].

## *4.4 Design example*

Omo River Bridge, a steel truss bridge configured below is taken as an exemplary study of analysis of the bridge structure and the design of truss connections.

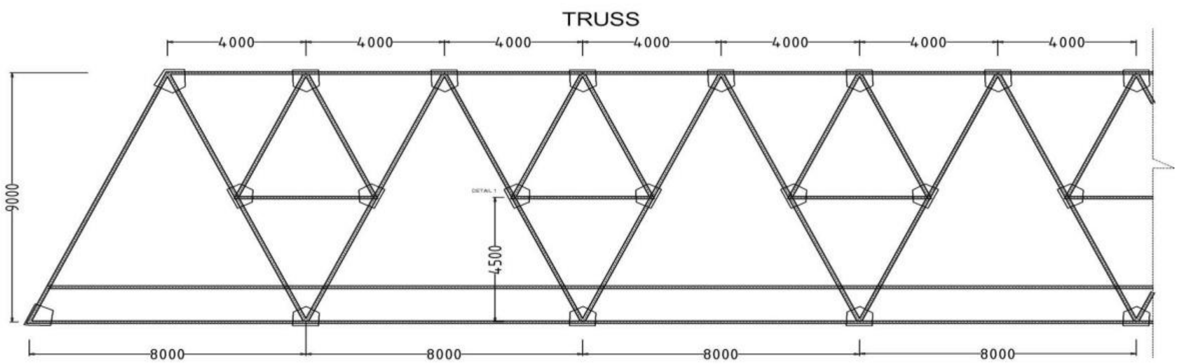



Figure 4.7: Part of truss panel

-Bottom/ top chord

-made of two angles sections with the following measured dimensions (see Fig. 4.8).

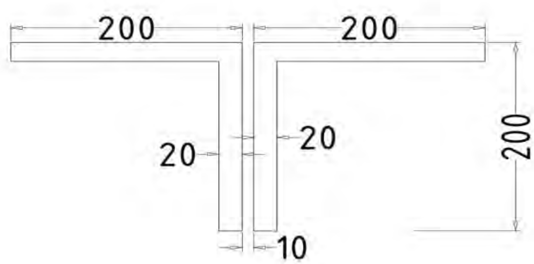



Figure 4.8: Measured dimensions in mm of bottom/top chord

- Cross-sectional area = 15000 mm2

- Diagonal members

-made of two angles with the following measured dimensions (see Fig.4.9).

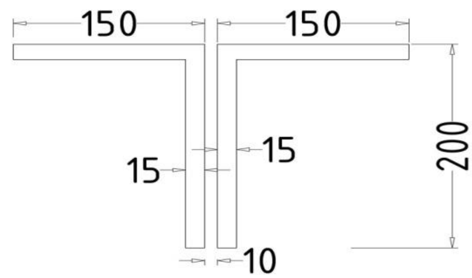



Figure 4.9: Measured dimensions in mm of diagonal chord

- Cross-sectional area = 9900 mm2

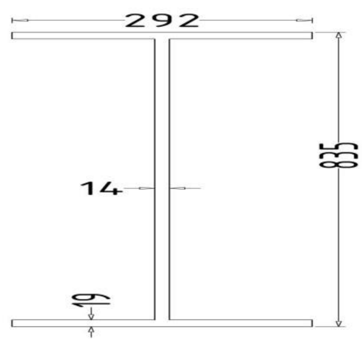



Figure 4.10: Measured dimensions in mm of floor beam

| | | |
|---|---|---|
| h=834.9mm | A=224 $cm^2$ | $w_{el.y}$ =5890 $cm^3$ |
| b=291.6mm | $I_y$ =246000 $cm^4$ | $w_{el.z}$ =534 $cm^3$ |
| $t_w$ =14.0mm | $I_z$ =7790$cm^4$ | $w_{pl.y}$ =6810 $cm^3$ |
| $t_f$ =18.8mm | $i_y$ =33.10 cm | $w_{pl.z}$ =842 $cm^3$ |
| | $i_z$ = 5.90 cm | |

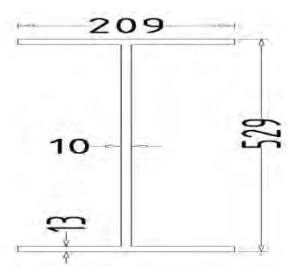



Figure 4.11: Measured dimensions in mm of stringers

| | | | |
|---|---|---|---|
| h=528.3mm | A=104 $cm^2$ | $w_{el.y}$ =1800 $cm^3$ | |
| b=208.7mm | $I_y$ =47500 $cm^4$ | $w_{el.z}$ =192 $cm^3$ | |
| $t_w$ =9.6mm | $I_z$ =2010 $cm^4$ | $w_{pl.y}$ =2060 $cm^3$ | |
| $t_f$ =13.2mm | $i_y$ =21.30 cm | $w_{pl.z}$ =300 $cm^3$ | $i_z$ = 4.38 cm |

Steel deck section is used and standard ERA specification loading conditions are applied on it. These loads are transferred to the stringers and floor beams, which are universal beam sections, and then load is transferred to the truss.

The steel truss bridge of 128m span with cross section shown below is considered for analysis and design procedures.

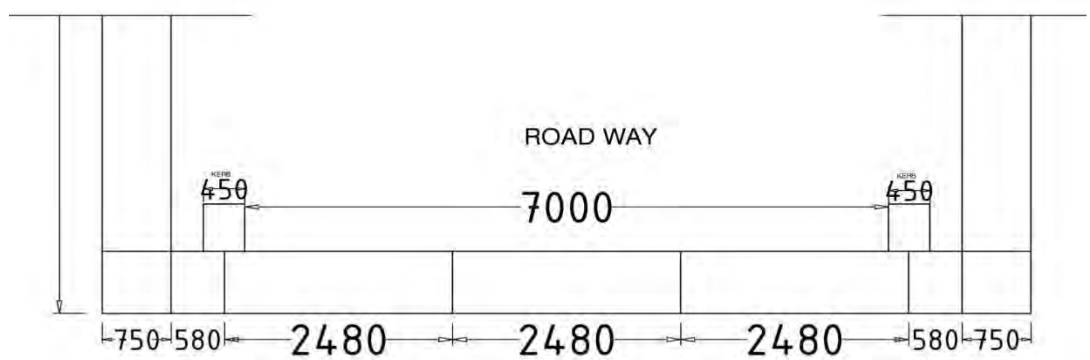



Figure 4.12: cross section of steel bridge showing deck and stringer in mm

**Design Data**

The following assumptions are used.

- The 128.00m span is made up of 16 panels each 8.00m long.
- The road way width is 7.00m.
- The center-to-center distance of truss is 8.60m.
- The center-to-center distance of floor beams is 8.00m.
- The center-to-center distance of stringers is 2.48m.
- Structural sections with yield strength of 355MPa and ultimate tensile strength of 510MPa are used to construct the stringers and floor beams.

-Structural sections with yield strength of 450MPa and ultimate tensile strength of 800MPa are used to construct the trusses.

- A steel deck with steel curb designed is taken from ERA manual.
- For live loads, ERA live loading of HL-93 are used.

Interior and exterior stringers are designed separately.

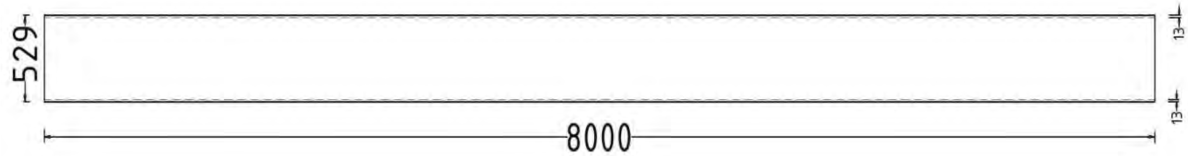



Figure 4.13: stringer longitudinal layout

### 4.4.1 Interior stringer design

**Dead loads**

Deck weight = (1018.37) (2.48) = 2525.56 N/m

Span of stringers = panel length = 8.00m

UB 533X210 stringer with $f_y$ = 355 MPa and mass 82 kg / m (see Fig. 4.11& see Fig. 4.13)

Total length of stringer members = 8.00 m

Total weight of stringer members = (82) (9.81) (8) = 6435.36 N

Weight per meter of stringer = 6435.36 / 8.00 = 804.42 N/m

Stringer weight = 804.42 N/m

Total dead load = 2525.56 + 804.42 = 3329.98 N/m=3.33 kN/m

Dead load moment = (3329.98) (8) (8) / 8 = 26.64 kNm

**Live Loads**

Truck Load with impact (33%):

$P1$ = (1+33/100) (145) = (1.33) (145) = 192.85 kN; for axle load of 145 kN

$P2$ = (1+33/100) (35) = (1.33) (35) = 46.55 kN; for axle load of 35 kN

Tandem Load with impact (33%):

$P3$= (1+33/100) (110) = (1.33) (110) = 146.30 kN; for tandem load of 110 kN

Lane Load: 9.3kN/m uniformly distributed in the longitudinal direction and occupying 3m width transversely. The line load per meter on stringer is (2.48/4.30) (9.30) = 5.36kN/m.

Fraction of wheel loads carried by the interior stringer as specified by AASHTO clause 3.23.2.2 is, S /4.5

Where, $S$ is average stringer spacing in feet.

Fraction of wheel loads carried by the interior stringer $= \frac{(2.48/0.3048)}{4.5} = 1.81$

**For truck load**

The axle spacing is 1.8m and so both axles can be placed on the stringer at a time. The maximum load occurs when the center of span is mid way between the resultant load and the heavier axle load as shown in Fig. 4.14 and Fig. 4.15. And the reaction load transferred is $0.5P1[1 + 0.68/2.48] = 1.28\text{P1}$.

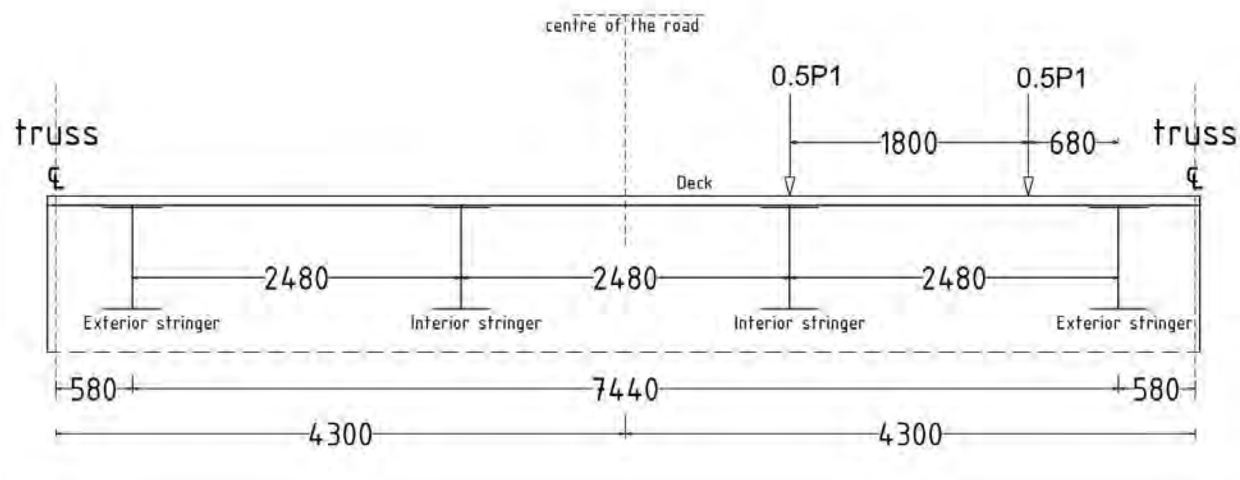



Figure 4.14: Truck load

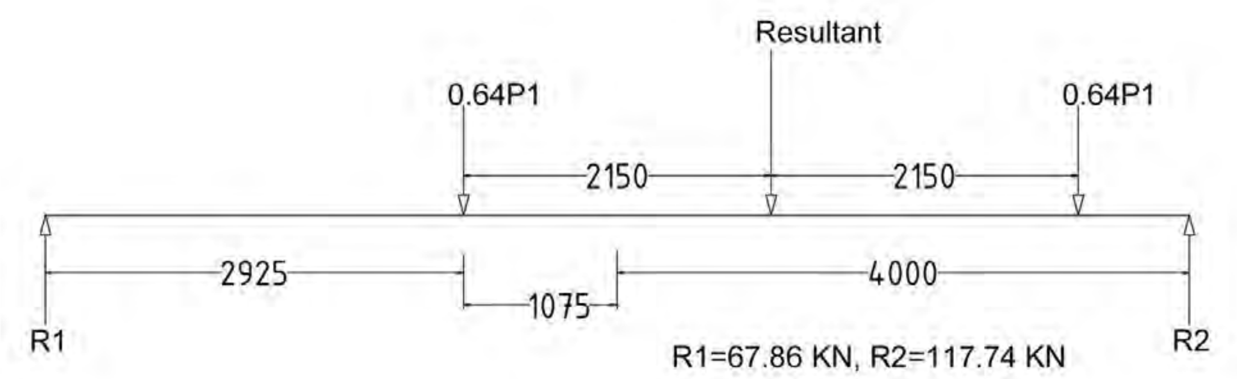



Figure 4.15: Truck load on interior stringer

Maximum truck load moment = (67.86) (2.925) (1.81) = 359.27kNm

The design moment is taken by applying limit load factors 1.25 for dead loads and 1.75 for live loads for strength I limit state, and also service load factor of 1.00 for dead loads and 1.3 for live loads for service II limit state and then by combining the effects of dead loads and live loads. The maximum design moment is the one, which is the maximum of limit or service state moments.

Total limit truck and dead moment = (1.25) (26.64) + (1.75) (359.27) = 662.02kNm

Total service truck and dead moment = (1.00) (26.64) + (1.30) (359.27) = 493.69kNm

**For tandem load**

The axle spacing is 1.2m and so both axles can be placed on the stringer at a time. The maximum load occurs when the center of span is mid way between the resultant load and the heavier axle load as shown in Fig. 4.16 and Fig. 4.17. And the reaction load transferred is $0.5P3[1+1.28/2.48] = 1.52\ \mathrm{P3}$.

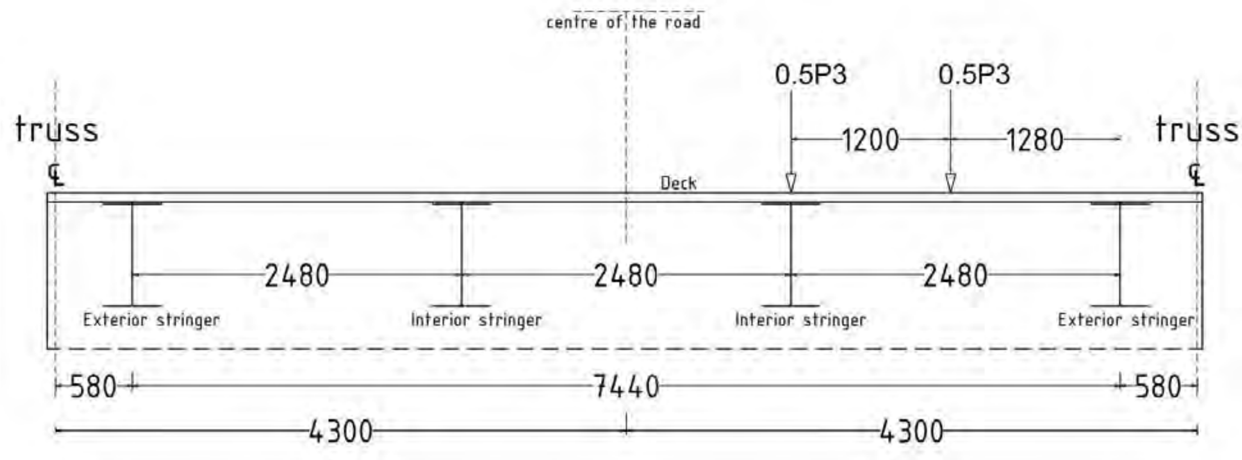



Figure 4.16: Tandem load

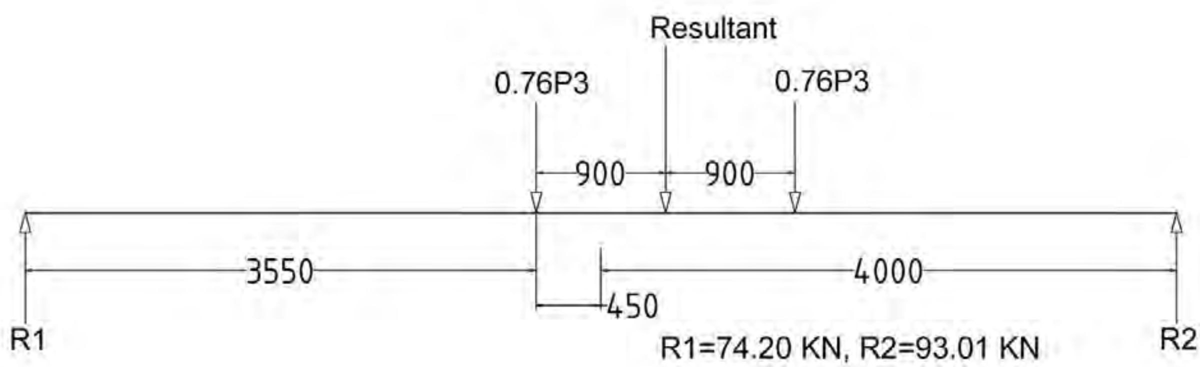



Figure 4.17: Tandem load on interior stringer

Maximum tandem load moment = (74.20) (3.55) (1.81) = 476.77 kNm

Total limit tandem and dead moment = (1.25) (26.64) + (1.75) (476.77) = 867.65kNm

Total service tandem and dead moment = (1.00) (26.64) + (1.30) (476.77) = 646.44kNm

**For lane load** (see Fig. 4.18)

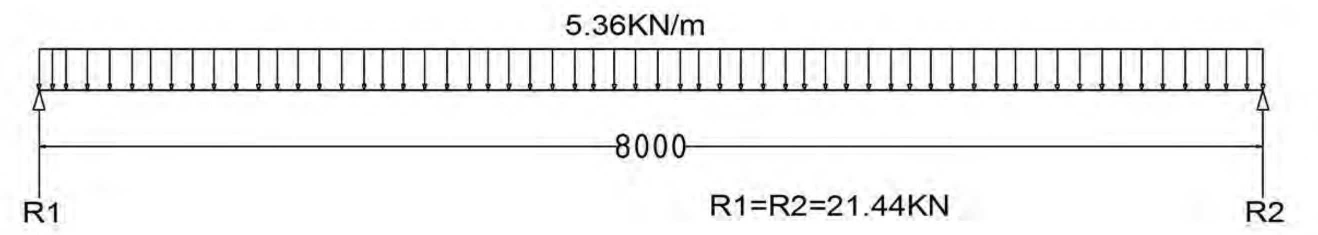



Figure 4.18: Lane load on interior stringer

Maximum lane load moment = (5.36) (8) (8) / 8 = 42.88 kNm

The design moment is the maximum of the truck or tandem moment combined with lane and dead moment. A summary of the combination is shown in Table 4.3.

Table 4.3 combination of truck or tandem with lane and dead moment on interior stringer

| No | Loading Type | Combined moment(in KNm) | Remark |
|---|---|---|---|
| **1** | Truck load and dead load with lane load( service) | 493.69+ (42.88) (1.75) =568.73 | |
| **2** | Truck load and dead load with lane load( limit) | 662.02+ (42.88) (1.75) =737.06 | |
| **3** | Tandem load and dead load with lane load( service) | 646.44+ (42.88) (1.75) =721.48 | |
| **4** | Tandem load and dead load with lane load( limit) | 867.65+ (42.88) (1.75) =942.69 | Maximum |

Therefore the design moment is

$M_D$=942.69 kNm

The required section modulus for the above moment is

$$S = {}^{M}/_{f_b}$$

Where:

*S* is elastic section modulus

*M* is bending moment due to the applied loads

$f_b$ is maximum normal stress due to bending

$$fb = (fy / \gamma M1) = 355 / 1.1 = 322.73\ MPa, \text{ where } \gamma M1 = 1.10$$

Thus, $s = \frac{942.69}{(322.73)(1000)} = 2.92 \times 10^6\ mm^3$

Check section modulus of H section stringer:

UB 533X210X82kg/m (see fig.4.11)

| | | |
|---|---|---|
| h=528.3mm | A=104 $cm^2$ | $w_{el.y}$ =1800 $cm^3$ |
| b=208.7mm | $I_y$ =47500 $cm^4$ | $w_{el.z}$ =192 $cm^3$ |
| $t_w$ =9.6mm | $I_z$ =2010 $cm^4$ | $w_{pl.y}$ =2960 $cm^3$ |
| $t_f$ =13.2mm | $i_y$ =21.30 cm | $w_{pl.z}$ =300 $cm^3$ , $i_z$ = 4.38 cm, |

***The section is sufficient.***

### 4.4.2 Exterior stringer design

**Dead loads**

Steel deck weight = (1018.37) (2.48) (0.5) = 1262.78 N/m

Weight of steel curbs = (1026.09) (0.45) = 461.74 N/m

Span of stringers = panel length = 8.00m

UB 533X210 stringer with $f_y$ = 355 MPa and mass 82 kg / m (see Fig. 4.11& see Fig. 4.13)

Stringer weight = 804.42 N/m

Total dead load = 1262.78 +461.74 + 804.42 = 2528.94N/m=2.53 kN/m

Dead load moment = (2528.94) (8) (8) / 8 = 20.23kNm

**Live Loads**

The axle load arrangement for maximum moment is the same as for interior stringers. The position of outer wheel load as specified by ERA clause 3.9.1 is shown in Fig. 4.19.

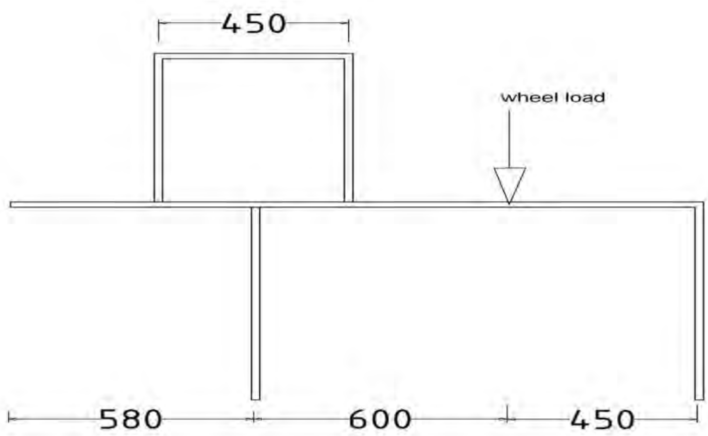



Figure 4.19: Position of wheel loads for exterior stringer

Part of wheel load supported by the exterior stringer as specified by AASHTO clause 3.23.2.3.1.2 is, $(0.45/1.05) = 0.43$

**For truck load** (see Fig. 4.20)

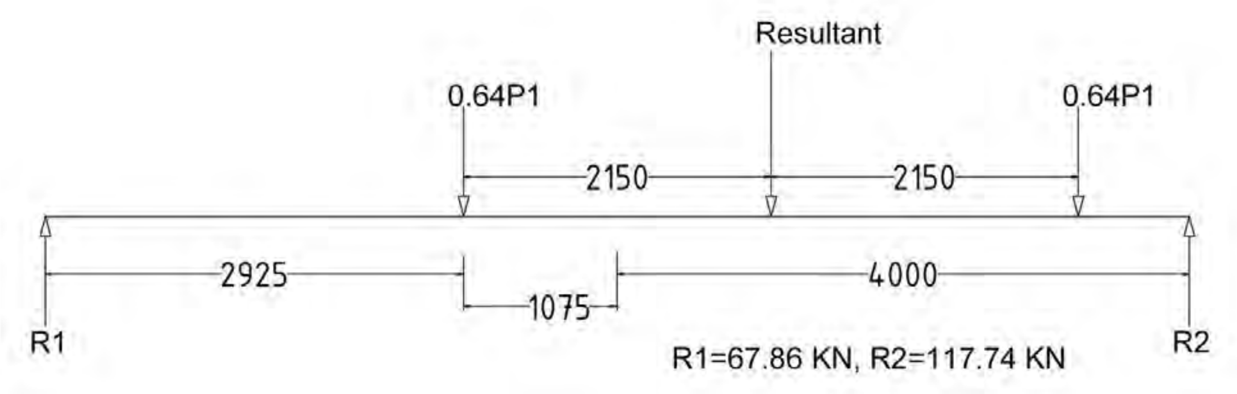



Figure 4.20: Truck load on exterior stringer

Maximum truck load moment = (67.86) (2.925) (0.43) = 85.35 kNm

Total limit truck and dead moment = (1.25) (20.23) + (1.75) (85.35) = 174.65 kNm

Total service truck and dead moment = (1.00) (20.23) + (1.30) (85.35) = 131.19 kNm

**For tandem load** (see Fig. 4.21)

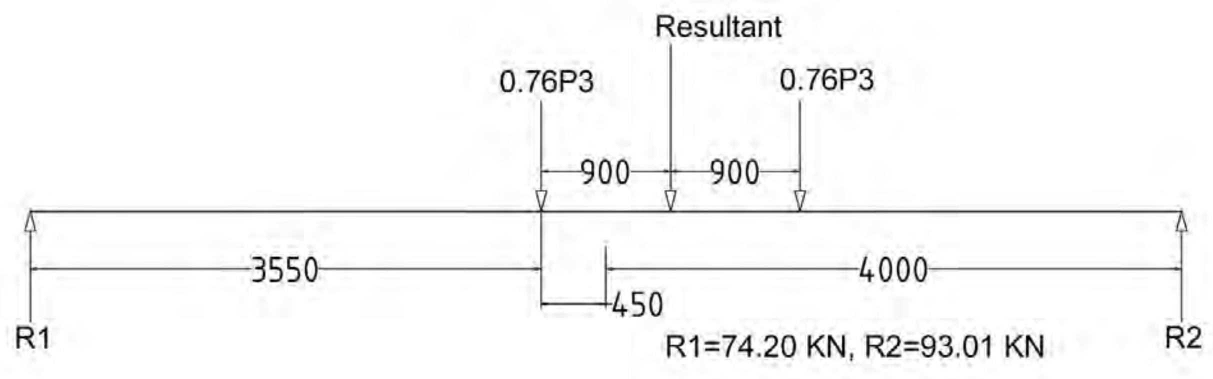



Figure 4.21: Tandem load on exterior stringer

Maximum tandem load moment = (74.20) (3.55) (0.43) = 113.27 kNm

Total limit tandem and dead moment = (1.25) (20.23) + (1.75) (113.27) = 223.51 kNm

Total service tandem and dead moment = (1.00) (20.23) + (1.30) (113.27) = 167.48 kNm

**For lane load** (see Fig. 4.22)

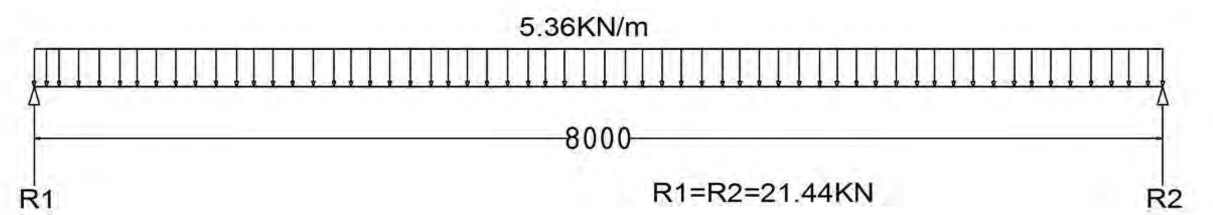



Figure 4.22: Lane load on exterior stringer

Maximum lane load moment = (5.36) (8) (8) / 8 = 42.88 kNm

A summary of the combination of the truck or tandem moment with lane and dead moment is shown in Table 4.4.

Table 4.4 combination of truck or tandem with lane and dead moment on exterior stringer

| No | Loading Type | Combined moment(in KNm) | Remark |
|---|---|---|---|
| **1** | Truck load and dead load with lane load( service) | 131.19 + (42.88) (1.75) =206.23 | |
| **2** | Truck load and dead load with lane load( limit) | 174.65+ (42.88) (1.75) =249.69 | |
| **3** | Tandem load and dead load with lane load( service) | 167.48 + (42.88) (1.75) =242.52 | |
| **4** | Tandem load and dead load with lane load( limit) | 223.51+ (42.88) (1.75) =298.55 | Maximum |

Therefore the design moment is

$M_D$=298.55kNm

The required section modulus for the above moment is:

$s = \frac{298.55}{(322.73)(1000)}$= 0.93x $10^6$ $mm^3$

***The section is sufficient.***

The required section modulus allows us to use a stringer of lower depth. But AASHTO clause 3.23.2.3.1.4 specifies that an exterior stringer shall not have lesser carrying capacity than the interior one. Therefore, use the same stringer as the interior one.

The total reaction force at the point of connection of stringer and floor beam is 408.31 KN (see attached SAP2000 analysis result.)

### 4.4.3 Floor beam design

The floor beam used is a universal beam made from H beam as shown in Fig. 4.23 and Fig.4.10

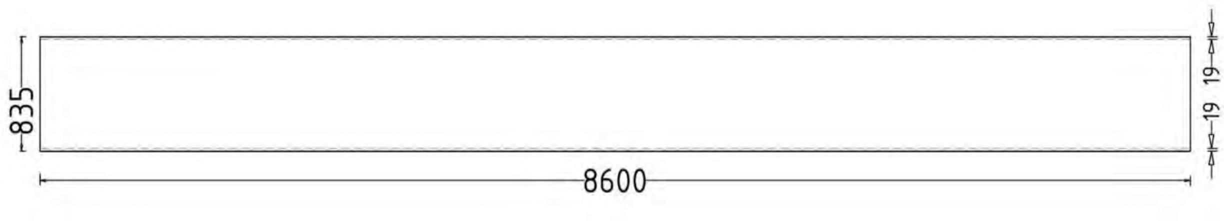



Figure 4.23: Floor beam longitudinal layout

**Dead loads**

Span of floor beam = 8.60m

The center-to-center distance of floor beam is 8.00m.

UB 838X292 with $f_y$ = 355 MPa and mass 176kg/m (see fig.4.10)

Total length of floor beam members = 8.60 m

Weight of floor beam members = (176) (9.81) (8.60) = 14848.42 N/m

Weight per meter of floor beam = 14848.42 / 8.60 = 1726.56 N/m

Floor beam weight = 1.73 kN/m

Dead load reaction from interior stringer = (3.33) (8.00) = 26.64 kN

Dead load reaction from exterior stringers = (2.53) (8.00) = 20.24 kN

The dead load reactions from stringers are applied as concentrated loads to floor beam as shown in Fig. 4.24

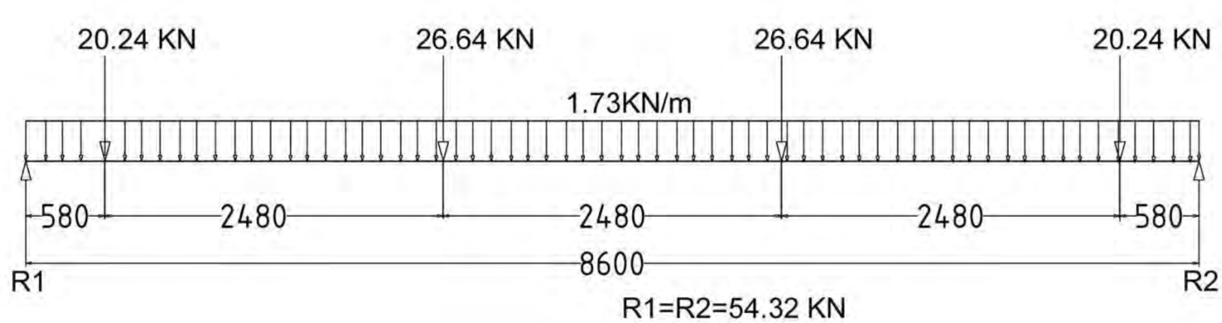



Figure 4.24: concentrated dead loads on floor beam

Maximum dead load moment = (54.32) (4.30) - (20.24) (3.72) - (26.64) (1.24) - (1.73) (4.30) (4.30) / 2 = 109.26 kNm

**Live Loads**

Article 3.23.3.1 of AASHTO doesn't permit transverse distribution of wheel loads in calculating bending moments for floor beams. The maximum corresponding load from the stringers is truck load and thus follows loading:-

**For truck load**

The maximum load occurs when the heavier axle load is on the floor beam transferred from the respective stringer reactions as shown in Fig. 4.25.

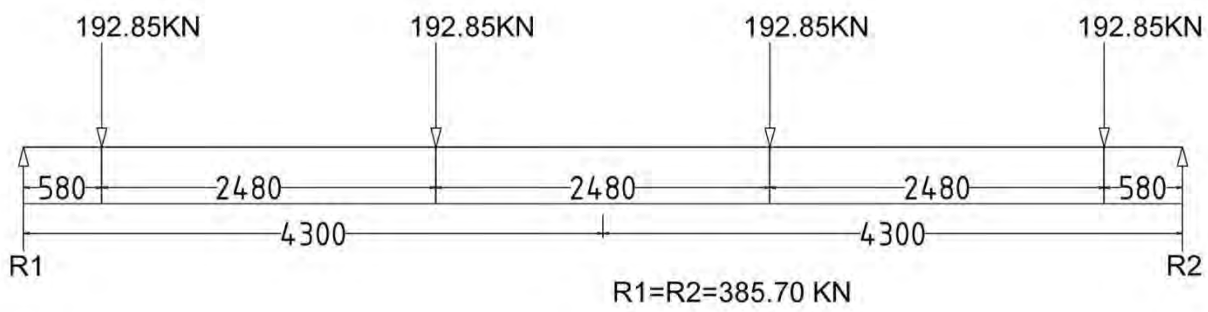



Figure 4.25: Theavier axle load on the floor beam

Maximum truck load moment = (385.70)(4.30) - (192.85)(3.72) - (192.85)(1.24) = 701.97 kNm

Total limit truck and dead moment = (1.25) (109.26) + (1.75) (701.97) = 1365.02 kNm

Total service truck and dead moment = (1.00) (109.26) + (1.30) (701.97) = 1021.82 kNm

**For lane load**

Lane load transferred to floor beams (see Fig 4.26)

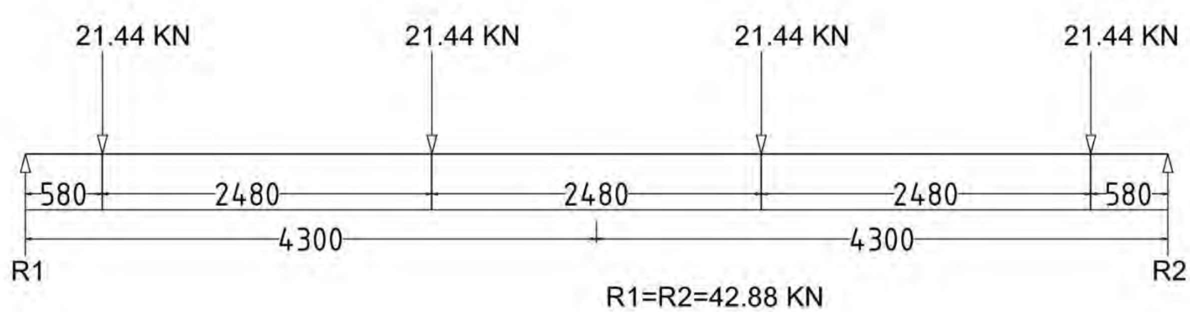



Figure 4.26: Lane load on the floor beam

$$Maximum\ lane\ load\ moment = (42.88)\,(4.30) - (21.44)\,(3.72 + 1.24) = 78.04\ kNm$$

A summary of the combination of the truck or tandem moment with lane and dead moment is shown in Table 4.5.

Table 4.5 combination of truck with lane and dead moment on floor beam

| No | Loading Type | Combined moment(in KNm) | Remark |
|---|---|---|---|
| 1 | Truck load and dead load with lane load( service) | 1021.82 + (78.04) (1.75) =1158.39 | |
| 2 | Truck load and dead load with lane load( limit) | 1365.02 + (78.04) (1.75) =1501.59 | Maximum |

Therefore the design moment is

$$MD = 1501.59kNm$$

The required section modulus for the above moment is

$$S = \frac{1501.59}{(322.73)(1000)} = 0.004654130 \text{ m}^3 = 4.65\text{x } 10^6 \text{ mm}^3$$

Check section modulus of H floor beam shown in Fig. 4.10.

UB 838X292X176kg/m

| | | |
|---|---|---|
| h=834.9mm | A=224 $cm^2$ | $w_{el.y}$ =5890 $cm^3$ |
| b=291.6mm | $I_y$ =246000 $cm^4$ | $w_{el.z}$ =534 $cm^3$ |
| $t_w$ =14.0mm | $I_z$ =7790$cm^4$ | $w_{pl.y}$ =6810 $cm^3$ |
| $t_f$=18.8mm | $i_y$ =33.10 cm | $w_{pl.z}$ =842 $cm^3$ |
| | $i_z$ = 5.90 cm | |

***The section is sufficient***.

The capacities of the members are determined first and then iterative procedures of design continue.

The total reaction force at the point of connection of truss and floor beam with having eccentricity of 375mm is 818 KN (see attached SAP2000 analysis result.)

### 4.4.4 Truss member design

**Dead loads - super imposed loads**

Total dead load transferred to truss at panel points = reaction of floor beam = 54.32kN.

The dead load at panel point transferred from floor beam (see Fig. 4.27) = 54.32kN

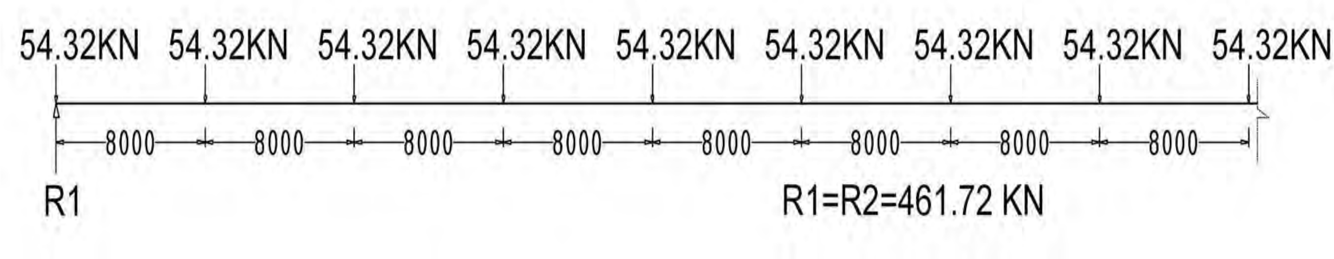



Figure 4.27: concentrated dead loads on truss at panel points

Maximum moment = (461.72) (64) - (54.32) (64) - (54.32) (56) - (54.32) (48) - (54.32) (40) - (54.32) (32) - (54.32) (24) - (54.32) (16) - (54.32) (8) = 13905.92 kNm

**Dead loads - self weight**

Top and Bottom chord truss members, 200x200x20L with :-( see Fig.4.8)

$f_y$ = 450 MPa and

Mass =48.92 kg / m

Length=496m

Diagonal truss members, 200x150x15L with :-( see Fig.4.08)

$f_y$ = 450 MPa and

Mass =36.40 kg / m

Length=1045.60m

$$Weight\ of\ steel\ truss\ = [\ (1045.60)(9.81)(36.40) + (496)(9.81)(48.92)] =\ 611.40KN$$

Span of steel truss = 128.00m

Steel truss weight = 611.40 / 128.00 = 4.78 kN/m

Dead load- self weight moment = (4.78) (128) (128) / 8 = 9789.44 kNm

Total dead load moment = 13905.92 + 9789.44 = 23695.36 kNm.

**Live Loads**

These loads are the transferred loads from floor beams.

**For truck loads** (see Fig. 4.28) this has maximum stress on the truss element.

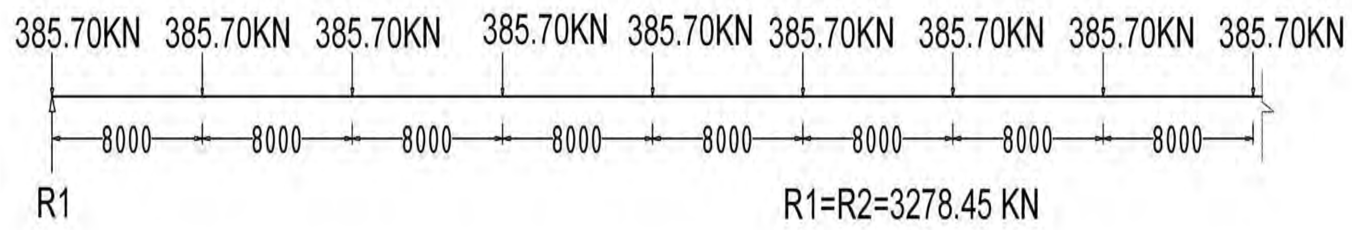



Figure 4.28: Concentrated Truck load on the truss

$$\begin{aligned} Maximum\ truck\ load\ moment &= (3278.45)\,(64) - (385.70)\,(64) - (385.70)\,(56) - (385.70)\,(48) \\ &- (385.70)\,(40) - (385.70)\,(32) - (385.70)\,(24) - (385.70)\,(16) \\ &- (385.70)\,(8) = 98739.20\ kNm \end{aligned}$$

Total limit truck and dead moment = (1.25) (23695.36) + (1.75) (98739.20) = 202412.80 kNm

Total service truck and dead moment = (1.00) (23695.36) + (1.3) (98739.20) = 152056.32 kNm

**For lane loads** (see Fig. 4.29) transferred from floor beam lane load reaction.

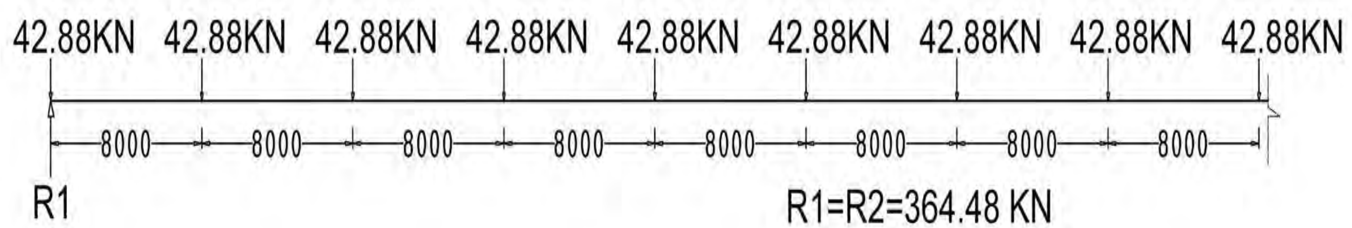



Figure 4.29: Lane load on the truss

$$\begin{aligned} \text{Maximum lane load moment} &= (364.48)\,(64) - (42.88)\,(64) - (42.88)\,(56) - (42.88)\,(48) \\ &- (42.88)\,(40) - (42.88)\,(32) - (42.88)\,(24) - (42.88)\,(16) \\ &- (42.88)\,(8) = 10977.28\ \text{kNm} \end{aligned}$$

A summary of the combination of the truck moment with lane and dead moment is shown in Table 4.6.

Table 4.6 combination of truck with lane and dead moment for truss

| No | Loading Type | Combined moment(in KNm) | Remark |
|---|---|---|---|
| **1** | Truck load and dead load with lane load( service) | 152056.32 + (10977.28) (1.75) =171266.56 | |
| **2** | Truck load and dead load with lane load( limit) | 202412.80 + (10977.28) (1.75) =221623.04 | Maximum |

Therefore the design moment is

$M_D = 221623.04\, kNm$

The required section modulus for the above moment is

$$S = {}^{M}/_{f_b}$$

Where:

*S* is elastic section modulus

*M* is bending moment due to the applied loads

$f_b$ is maximum normal stress due to bending

$$fb = (fy / \gamma M1) = 450 / 1.1 = 409.09\, MPa, \text{ where } \gamma M1 = 1.10$$

Thus, $S = \frac{221623.04}{(409.09)(1000)} = 0.541746413 \text{ m}^3$ = 541.75 x $10^6$ $\text{mm}^3$

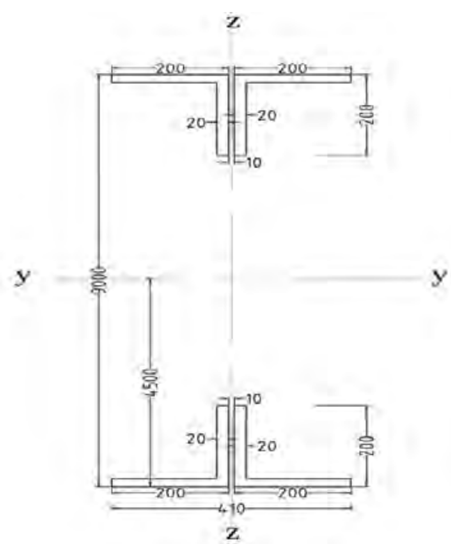



Figure 4.30: Cross section of the truss in mm

The truss is modeled for analysis using SAP2000 version 14 software; The top and bottom chords are modeled using 2x200x200x20L,angle section and the vertical and diagonal members using 2x200x150x15L,angle section. The design loads are loaded on the truss joints. Following analysis of the truss; the member forces nearest to joint shown below are:-

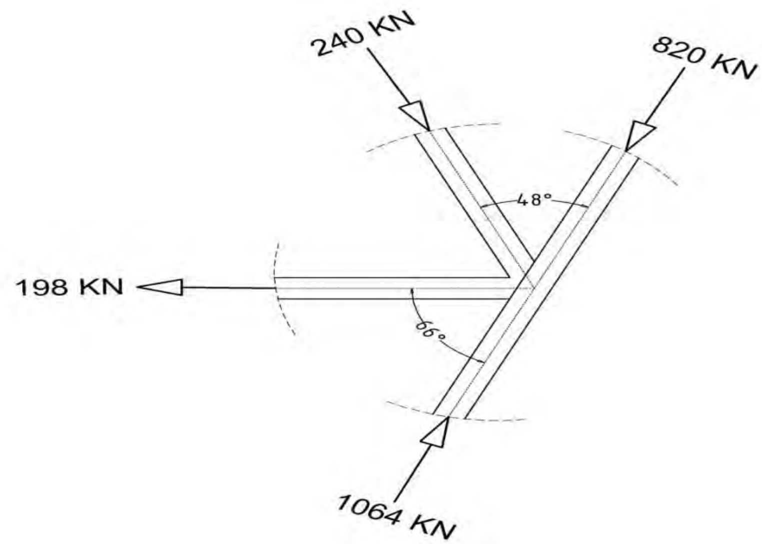



Figure 4.31: Member forces at a joint

### 4.4.5 Connection design

The design of a connection is shown below for the given preliminary setting joint.

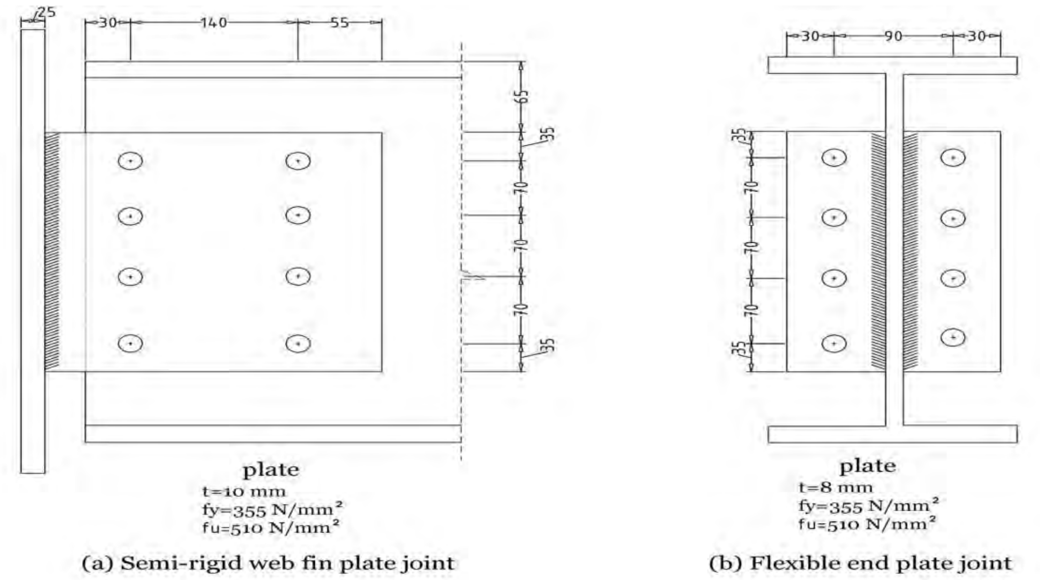



Figure 4.32: Member joint

***a) in-plane analysis of a bolt group:*** The semi-rigid web side plate (fin plate) joint shown in Figure 4.30 is to transmit factored design actions equivalent to a vertical downwards force of $Q=408$ kN acting at the centroid of the bolt group and a clockwise moment of $My= 7$ kNm.

For the bolt group $\sum(y_i^{\,2} + z_i^{\,2}) = 4\text{x}\,(\text{p2}^2 + (\text{p1} + \text{p2})^2) + 4\text{x}\,(\text{p2}^2 + \text{p1}^2)$

$$\sum(y_i^{\,2} + z_i^{\,2}) = 4\text{x}\,(70^2 + (105)^2) + 4\text{x}\,(70^2 + 35^2) = 88\,200\text{mm2}.$$

Using the expression, $y_r = \dfrac{-Q\ X10^3\ X \sum(y_i^{\,2} + z_i^{\,2})}{-My\ X\ 10^6\ X\ t}$

$$y_r = (-408x10^3x\ 88200)/(-7x10^6x8) = 643\ mm$$

And so the most heavily loaded bolts are at the top and bottom of the right-hand row.

For these $r_i = \sqrt{\{(y_r + 70)^2 + 105^2\}}$

$$r_i = \sqrt{\{(643 + 70)^2 + 105^2\}} = 720\text{mm}$$

and the maximum bolt force is:

$$F_{v,ED} = \frac{-M_y\ X10^6\ X\ r_i}{\sum(y_i^{\,2} + z_i^{\,2})}$$

$$F_{v,ED} = \frac{-7\ X10^6\ X\ 720}{88200}\text{N} = -57.14\text{KN}.$$

***b) in-plane design resistance of a bolt group***

For 20mm Grade 8.8 bolts, $A_s = A_t = 245\text{mm}^2$,

$\alpha_v = 0.6$, $fub = 800$ N/mm$^2$, and so,

$$F_{v,Rd} = \frac{\alpha_y \; fub \; At}{\gamma M2}$$

$$F_{v,Rd} = \frac{0.6 \; x \; 800x \; 245}{1.25} = 94.1KN.$$

Using this with the solution of (a), $57.14 \leq 94.1$, and ok!

***c) plate-bearing resistance: web fin plate resistance***

$$\alpha_d = \frac{35}{(3X22)} = 5.30\,, \qquad \frac{f_{ub}}{f_u} = \frac{800}{510} = 1.569 > 0.530$$

and so $\alpha_b = 0.530$.

$$\frac{2.8e_2}{d_0} - 1.70 = \frac{2.80X55}{22} - 1.70 = 5.30 > 2.50, and\ so\ k1 = 2.50$$

$$F_{b,Rd} = \frac{k_1 \; \alpha_b \; fu \; dt}{\gamma M1}$$

$$F_{b,Rd} = \frac{2.5X\ 0.53X\ 510X20X10\ \ N}{1.1} = 122.9KN.$$

Using this with the solution of (a), $57.14 \leq 122.90$, and ok!.

***d) Plate shear and tension resistance***

If the yield criterion is used with the elastic stresses on the gross cross-section of the plate, then the elastic bending stress may be determined using an elastic section modulus of $bd^2/6$, so that

$$\sigma_y = \frac{M_y\ X10^6\ X\ a}{(t\ \text{X}\ w^2)} \text{N}/mm^2$$

$$\sigma_y = \frac{7\ X10^6\ X\ 6}{(10\ \text{X}\ 280^2)} = 53.59N/mm^2$$

and the average shear stress is

$$\tau_{yz} = \frac{Q\ X10^3}{(t\ \text{X}\ w^2)} \text{N}/mm^2$$

$$\tau_{yz} = \frac{408\ X10^3}{(10\ \text{X}\ 280^2)} = 145.66\ N/mm^2.$$

$$(53.59)^2 + 3 \times (145.66)^2 \leq 355^2$$

Thus 258 ≤ 355, ok!

If the equivalent fracture criterion is used with the elastic stresses on the net cross-section of the plate, then

$$I_p = (w^3 \times t)\frac{1}{12} - 2 \times hole \times t \times l^2 - 2 \times hole \times t \times p1^2 mm^4$$

$$I_p = (280^3 \times 10)\frac{1}{12} - 2 \times 22 \times 10 \times 105^2 - 2 \times 22 \times 10 \times 35^2 = 12.90 \times 10^6 mm^4$$

so that,

$$w_{el,p} = (Ip) \times \frac{2}{w} mm^3$$

$$w_{el,p} = (12.90 \times 10^6)\frac{2}{280} = 92.20 \times 10^3 mm^3$$, and

$$\sigma_y = \frac{M_y\, X10^6}{(\ w_{el,p})} \text{N}/mm^2$$

$$\sigma_y = \frac{7\, X10^6}{(92.20 \times 10^3)} = 75.95\ \text{N}/mm^2$$ and the average shear stress is

$$\tau_{yz} = \frac{Q\, X10^3}{((w - 4X\ \text{hole})\, X\, t)} \text{N}/mm^2$$

$$\tau_{yz} = \frac{408\ X10^3}{\{(280 - 4X\ 22)\, X\, 10\}} = 212.57\ \text{N}/mm^2.$$

Using the equivalent of equation above for fracture with *fu* = 510N/mm$^2$

$$\sqrt{\{(\ \sigma_y)^2 + 3 \times (\ \tau_{yz})^2\}} \leq \sqrt{fu}$$

$(75.95)^2 + 3 \times (212.57)^2 \leq 510^2$, so that $376 \leq 510$

However, these calculations are conservative, since the maximum bending stresses occur at the top and bottom of the plate, where the shear stresses are zero. If the shear stresses are ignored, then 53.59≤355 and *75.95≤510*

***e) Fillet weld resistance:*** Weld forces per unit length.

At the welds, the design actions consist of a vertical shear of *408* kN and a moment of

$$M_w = \left(-7 - 408 \times \frac{25 + 30 + 70}{1000}\right) kNm = -58.00\ kNm.$$

$$l_{eff} = 280 - 2 \times \frac{8}{\sqrt{2}} = 268.70 mm$$

The average shear force per unit weld length can be determined as:

$$F_{L,ED} = \frac{Q\ X10^3}{(2\,X\ \ l_{eff})}\ \text{N}/mm$$

$F_{L,ED} = \frac{408\ X10^3}{\{2X\ 268.70\}} = 759.29\ \text{N}/mm$, and the maximum bending force per unit weld length is:-

$$F_{Ty,ED} = \frac{(M_w\ X10^6\ )\ X\ (\frac{l_{eff}}{2})}{(2\,X\ \ l_{eff}{}^3\ /12)}\ \text{N}/mm$$

$F_{Ty,ED} = \frac{(-58\ X10^6\ )\ X\ (\frac{268.7}{2})}{(2\ X\ 268.70^3\ /12)} = -2410\ \text{N}/mm$ , and the resultant of these forces is:-

$$F_{w,ED} = \sqrt{[(\ F_{L,ED})^2 + (\ F_{Ty,ED})^2]}\ N/mm$$

$$F_{w,ED} = \sqrt{[(759.29)^2 + (2410)^2]} = 2526.78\ N/mm$$

*Using Simplified method of Eurocode 3 article 1-8(EC3-1-8).*

For Grade S355 steel, $\beta w = 0.9$

$$f_{vw,d} = \frac{fu/\sqrt{3}}{(1.25\ X\ \ \beta_w)}\ \text{N}/mm^2$$

$$f_{vw,d} = \frac{510\ /\ \sqrt{3}}{(1.25\ X\ 0.9)} = 261.70\ \text{N}/mm^2$$

$$F_{w,Rd} = \ f_{vw,d}\ X\ (8/\sqrt{2})\ \text{N}/mm$$

$F_{w,Rd} = 261.70\ X\ (8/\sqrt{2}) = 1481.00\ \text{N}/mm$, since 2526.78 > 1481.00, needs revision.

*Using Directional method of Eurocode 3 article 1-8(EC3-1-8) the following equations can be applied,*

$$\sigma\perp = \frac{(M_w\ X10^6\ )\ X\left(\frac{l_{eff}}{2}\right)\sin 45°}{\{2\ X\ (l_{eff}{}^3\ /12)X8/\sqrt{2}\}}\ \text{N}/mm^2$$

$$\sigma\perp = \frac{(-58\ X10^6\ )\ X\left(\frac{268.70}{2}\right)\sin 45°}{\{2\ X\ 268.70^3\ /12)X8/\sqrt{2}\}} = -301.24\ \text{N}/mm^2$$

$$\tau\perp = \frac{(M_w\ X10^6\ )\ X\left(\frac{l_{eff}}{2}\right)\cos 45°}{\{2\ X\ (l_{eff}{}^3\ /12)X8/\sqrt{2}\}}\ \text{N}/mm^2$$

$$\tau\perp = \frac{(-58\ X10^6\ )\ X\left(\frac{268.70}{2}\right)\cos 45°}{\{2\ X\ 268.70^3\ /12)X8/\sqrt{2}\}} = -301.24\ \text{N}/mm^2$$

$$\tau|| = \frac{QX\ 10^3}{2\ X\ \ l_{eff}X\ (8/\sqrt{2})}\ \text{N}/mm^2$$

$\tau|| = \frac{408X\ 10^3}{2\ X\ 268.70X\ (8/\sqrt{2})} = 134.23$ N/$mm^2$, the strength condition is:-

$$\sqrt{\{(\sigma \perp)^2 + 3 \times [(\tau \perp)^2 + (\tau||)^2]\}} \leq \frac{fu}{\beta_w X1.25}$$

$\sqrt{\{(-301.24)^2 + 3 \times [(-301.24)^2 + (134.23)^2]\}} \leq \frac{510}{0.9X1.25} (= 453.33),$

So that, $645.78 > 453.33$,thus it agrees with the simplified method.

Therefore the fillet weld shall be revised, and the governing joint resistance welds shall be determined.

***f) bolt slip:*** If the bolts of the semi-rigid web fin plate joint shown in Figure 4.30 are preloaded, then we need to determine the value of vertical load at which the first bolt slip occurs, if the joint is designed to be non-slip in service and the friction surface is Class B.

$F_{p,c} = 0.7 \times 800 \times 245N = 137.2\,kN$

$k_s = 1.0$

$\mu = 0.4$

$\gamma_{M3,ser} = 1.1$

$F_{s,Rd} = 1.0 \times 0.4 \times 137.2/1.1 = 49.9\ kN$; And using this, $Q_{SL} = 49.90$KN.

***g) out-of-plane resistance of a bolt group:*** The flexible end plate joint shown in Figure 4.30 is to transmit factored design actions equivalent to a downwards force of *Q= 408* kN acting at the centroid of the bolt group and an out-of-plane moment of *Mx* = −7 kNm. Thus the maximum bolt forces and the design resistance of the bolt group are:-

*Bolt group analysis.*

$\sum Zi^2 = 4 \times (\mathrm{p1} + \mathrm{p2})^2 + 4 \times (\mathrm{p1})^2$ mm2

$\sum Zi^2 = 4 \times (105)^2 + 4 \times (35)^2$ mm2 $= 49\,000$mm2 ,and using equation:

$$F_{t,Ed} = \frac{-M_x\, X10^6(-p1 - p2)}{\sum {z_i}^2} N$$

The maximum bolt tension is:

$$F_{t,Ed} = \frac{-7\ X10^6(-35-70)}{49000} = 14.98kN$$

(A less-conservative value of $F_{t,Ed} = 14$ kN is obtained if the centre of compression is assumed to be at the lowest pair of bolts so that the lever arm to the highest pair of bolts is 175 mm). The average bolt shear is:

$$F_{v,Ed} = \frac{Q\ X10^3}{n} N\text{, where n= number of bolts.}$$

$$F_{v,Ed} = \frac{408\ X10^3}{8} = 51.00kN$$

*Plastic resistance of plate*

If the web thickness is 8 mm,

$$m = \frac{90}{2} - \frac{8}{2} - 0.8X6 = 36.20mm$$

$$l_{eff} = 2 \times 36.2 + 0.625 \times 30 + 35 = 126.15mm$$

$$M_{Pl,1,Rd} = \frac{0.25\ X\ t^2 X\ l_{eff}\ Xfy}{1.00} Nmm$$

$$M_{Pl,1,Rd} = \frac{0.25\ X\ 8^2 X\ 126.15\ X\ 355}{1.00} = 0.72\ kNm$$

$$F_{T,1,Rd} = \frac{4\ X\ M_{Pl,1,Rd}\ X\ 10^6}{30.00} N$$

$F_{T,1,Rd} = \frac{4\ X\ 0.72\ X\ 10^6}{30.00} = 96.00\ KN$, and so $F_{T,1,Rd} \geq 2$ x14.98 (=29.96).

*Bolt resistance.*

At plastic collapse of the plate, the prying force causes a plastic hinge at the bolt line, so that the prying force; $F_{prying} = \frac{M_{Pl,1,Rd}\ X\ 10^6}{30.00} N$

$F_{prying} = \frac{0.72\ X\ 10^6}{30.00} kN = 24.00$ kN.

Thus, bolt tension is given by the expression; $F_{t,bolt} = F_{t,Ed} + F_{prying}$,

$F_{t,bolt}$ = 14.98 + 24.00 = 38.98 kN.

Using the solution of (b), $F_{v,Rd}$ = 94.1 kN. $F_{t,bolt} \leq F_{v,Rd}$, ok!

For 20mm Grade 8.8 bolts, $A_t = 245mm^2$ and *fub* = 800 $N/mm^2$ and using the resistance formula:

$$F_{t,Rd} = \frac{\beta w \times fu,b \times At}{1.25} \text{N}$$ , we have:-

$F_{t,Rd} = \frac{0.9 \times 800 \times 245}{1.25}$ N $=$ 141.1 kN. And the governing resistance shall be determined using the expression below.

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,bolt}}{1.4XF_{t,Rd}} < 1.00$$

Hence,

$$\frac{51}{94.10} + \frac{38.98}{1.40X141.10} = 0.74 < 1.00, \text{and so the bolt resistance doesn't govern.}$$

***h) plate-bearing resistance for the above situation***

$\alpha_d = \frac{35}{3 \times 22} = 0.530, \frac{fub}{fu} = \frac{800}{510} = 1.569 > 0.530,$

and so $\alpha_b$ = 0.530.

$k1 = 2.8 \times \frac{30}{22} = 3.82 > 2.5 \text{ so that } k1 = 2.5$

$$F_{b,Rd} = \frac{2.5 \times \alpha_b \times fu \times \text{d} \times \text{t}}{\gamma\text{M0}} \text{N}$$

$$F_{b,Rd} = \frac{2.5 \times 0.530 \times 510 \times 20 \times 8}{1.1} \text{N} = 98.3 \text{ kN}$$

Using this with the analysis solution of (g), 98.30 < 408.00,

***i) fillet weld resistance***

$lw = 268.7mm \text{ and } F_{L,ED} = 759.29\ N/mm, as\ in\ (e).$

The maximum bending force per unit weld length can be determined from:-

$$F_{Ty,ED} = \frac{(M_X\ X10^6)\ X\ (\frac{l_{eff}}{2})}{(2\ X\ l_{eff}^3\ /12)} \text{N}/mm$$

$F_{Ty,ED} = \frac{(-7\ X10^6)\ X\ (\frac{268.7}{2})}{(2\ X\ 268.70^3\ /12)} = -290.92$ N$/mm$ , and the resultant of these forces is:-

$$F_{w,ED} = \sqrt{[(F_{L,ED})^2 + (F_{Ty,ED})^2]}\ N/mm$$

$$F_{w,ED} = \sqrt{[(759.29)^2 + (290.92)^2]} = 813.11 \text{ N/mm.}$$

*Using Simplified method of Eurocode 3 article 1-8(EC3-1-8).*

For S355 Grade steel, $\beta w = 0.9$

$$f_{vw,d} = \frac{fu/\sqrt{3}}{(1.25 \text{ X } \beta_w)} \text{ N}/mm^2$$

$$f_{vw,d} = \frac{510/\sqrt{3}}{(1.25 \text{ X } 0.9)} = 261.70 \text{ N}/mm^2$$

$$F_{w,Rd} = f_{vw,d} \; X \; (6/\sqrt{2}) \text{ N}/mm$$

$F_{w,Rd} = 261.70 \; X \; (6/\sqrt{2}) = 1110.00$ N/$mm$, but $813.11 < 1110.00$, ok!

*Using Directional method of Eurocode 3 article 1-8(EC3-1-8) the following equations can be applied*,

$$\sigma \perp = \frac{(M_X \; X10^6) \; X\left(\frac{l_{eff}}{2}\right) \sin 45°}{\{2 \text{ X } (l_{eff}{}^3/12)X6/\sqrt{2}\}} \text{ N}/mm^2$$

$$\sigma \perp = \frac{(-7 \; X10^6) \; X\left(\frac{268.70}{2}\right) \sin 45°}{\{2 \text{ X } 268.70^3/12)X6/\sqrt{2}\}} = -48.44 \text{ N}/mm^2$$

$$\tau \perp = \frac{(M_X \; X10^6) \; X\left(\frac{l_{eff}}{2}\right) \cos 45°}{\{2 \text{ X } (l_{eff}{}^3/12)X6/\sqrt{2}\}} \text{ N}/mm^2$$

$$\tau \perp = \frac{(-7 \; X10^6) \; X\left(\frac{268.70}{2}\right) \cos 45°}{\{2 \text{ X } 268.70^3/12)X6/\sqrt{2}\}} = -48.44 \text{ N}/mm^2$$

$$\tau|| = \frac{Q X \; 10^3}{2 \text{ X } l_{eff} X \; (6/\sqrt{2})} \text{ N}/mm^2$$

$\tau|| = \frac{408X \; 10^3}{2 \text{ X } 268.70X \; (8/\sqrt{2})} = 178.95$ N/$mm^2$, the strength condition is:-

$$\sqrt{\{(\sigma \perp)^2 + 3 \times [(\tau \perp)^2 + (\tau||)^2]\}} \leq \frac{fu}{\beta_w X1.25}$$

$$\sqrt{\{(-48.44)^2 + 3 \times [(-48.44)^2 + (178.95)^2]\}} \leq \frac{510}{0.9X1.25} (= 453.33),$$

so that *324.74* $\leq$ 453.33; and $48.44 \leq \frac{0.9 \times 510}{1.25} (= 367.20)$.Thus the connection capacity is not governed by the shear and bending resistance of the welds.

**Truss connections**.

Design a bolted gusset plate connection for the members and factored loads shown in figure 4.33

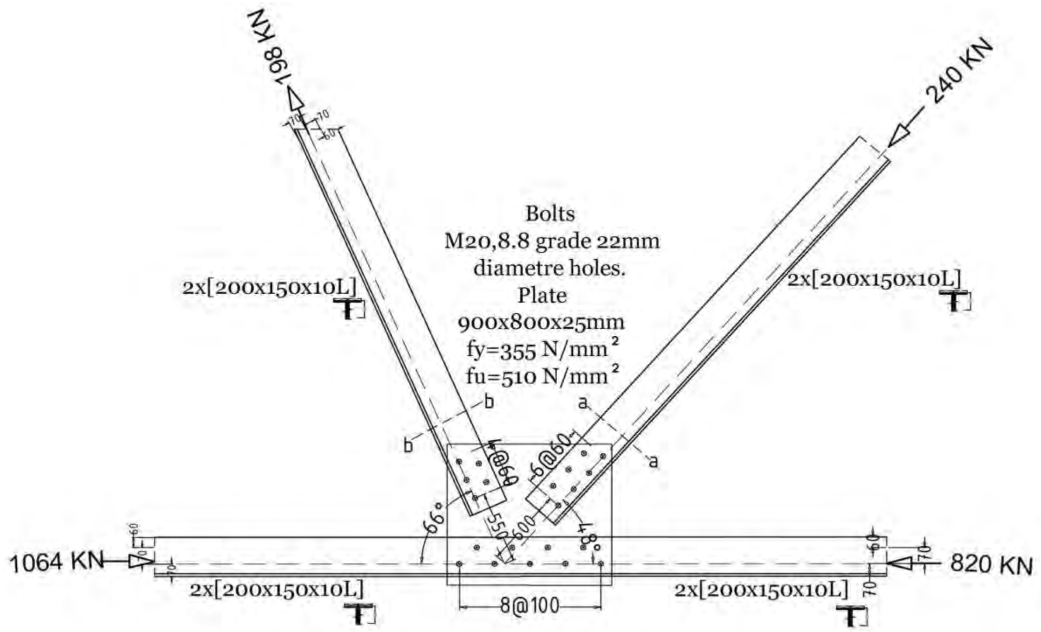



Figure 4.33: Typical truss joint

**Bolts**

Use M20, 8.8 bolts in double shear.

Shear capacity per bolts Ps=$p_s A_s$

$$=(2X375)\frac{245}{1000} = 183.75kN$$

Bearing capacity $P_{bs}$ =$dtp_{bs}$

$$P_{bs}=183.75kN$$

Minimum thickness of plate in bearing t = $\frac{183.7X1000}{20x355}$

$$t = 25.88mm$$

Use 25mm thick gusset plate.

Minimum end distance for gusset plate p1=$\frac{2X183.75X1000}{20X355} = 51.76mm$

Combined thickness of attached legs of the tension diagonal=$2X10 = 20 < 25mm$

Therefore, capacity per bolt for tension diagonal (bearing capacity of connected plies).

$=\frac{2X20X10X355}{1000}$ =142.00 kN

**Compression diagonal chord (2[200x150x10 ┘└ ]**

Number of bolts required ignoring eccentricity=$\frac{240}{183.75}=1.31$; try for 4 bolts.

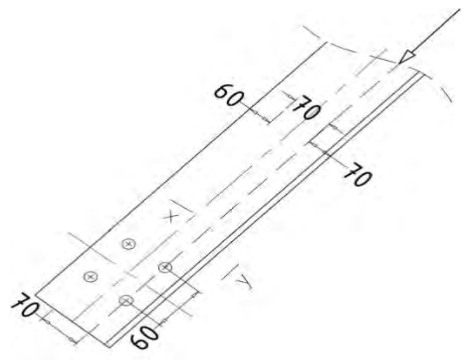



Figure 4.34: Compressed diagonal chord o the truss

Thus from figure 4.34, we can find the centroid of the bolt group:

$\bar{x}=\frac{2X70}{4}=35mm$

$\bar{y}=\frac{2X60}{4}=30mm$

$\sum(x_i{}^2+y_i{}^2) = 4\text{x}\ (35^2)+4\text{x}\ (30^2)= 8500\text{mm2}$

Moment $=\frac{240X(70-30+35)}{1000}=18000.00\ Nm$

Maximum resultant shear on bolt

$$=\sqrt{\left[\frac{240}{4}+\frac{18000X35}{8500}\right]^2+[\frac{18000X30}{8500}]^2} \quad = 148.41\ \text{kN} < 183.75\text{KN},$$

thus use 4 /M20 8.8 bolts in compression diagonal chords.

**Tension diagonal chord (2[200x150x10 ┘└])**

Number of bolts required ignoring eccentricity =$\frac{198}{142}=1.39$; use 4 bolts

Check tension capacity.

Effective area=$2(3400-22X10)$

$=3180mm2$

Tension capacity=$\frac{3180X355}{1000} = 1100.00\ kN$ >198Kn, hence ok! Therefore use 4/M20, 8.8 grade bolts in tension diagonal chord.

**Bottom chord (2[200x150x10 ┘└])**

Number of bolts required ignoring eccentricity=$\frac{1064}{183.75} = 5.78$; try using 8 bolts.

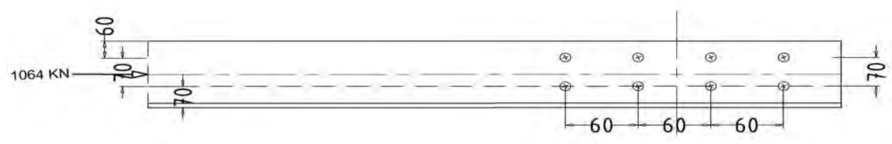



Figure 4.35: Compressed bottom chord of the truss

Thus from figure 4.35, we can find the centroid of the bolt group:

$\bar{x}=\frac{4X70}{8} = 35mm$

$\bar{y}=\frac{2X60+2X120+2X180}{8} = 90mm$

$$\sum(x_i^2 + y_i^2) = 8x\,(35^2) + 4x\,(90^2) + 4x\,(30^2) = 45800\text{mm2}$$

Moment =$\frac{1064X(70-60+35)}{1000} = 47880.00\ Nm$

Maximum resultant shear on bolt

$$=\sqrt{\left[\frac{1064}{8} + \frac{47880X35}{45800}\right]^2 + \left[\frac{47880X90}{19300}\right]^2} = 193.94\text{ kN} > 183.75\text{KN},$$

thus, 8 number of bolts are not enough so we need to increase the number of bolts by 2 and we need to check this also with same procedure.

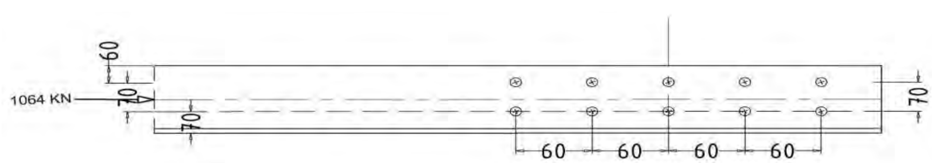



Figure 4.36: Revised Compressed bottom chord of the truss

$\bar{x}=\frac{5X70}{10} = 35mm$

$\bar{y}=\frac{2X60+2X120+2X180+2X240}{10} = 120mm$

$$\sum(x_i^2 + y_i^2) = 8x\,(35^2) + 4x\,(120^2) + 4x\,(60^2) = 81800\text{mm2}$$

Moment $=\frac{1064X(70-60+35)}{1000} = 47880.00\ Nm$

Maximum resultant shear on bolt

$$=\sqrt{\left[\frac{1064}{10} + \frac{47880X35}{81800}\right]^2 + \left[\frac{47880X120}{81800}\right]^2} = 145.03 \text{ kN} < 183.75\text{KN},$$

thus use 10 /M20, 8.8 grade bolts in compression bottom chord.

**Gusset plate**

Check section a-a, taking $30^0$ spread from extreme bolts,

Effective width=$60\ tan30 + 70 + 120\ tan30 - 2\ X\ 22 = 130mm$

Tension capacity=$\frac{130X20X355}{1000} = 923kN\ > 240kN\ is\ ok!$

Thus use gusset plate of having the following dimensions,$400X300X25mm$

# 5.0 Discussions

## *5.1 Detailing of connections*

The process of detailing in connections is very important to execute reliable and economic construction engineering; and detailing is not only a drafting process but it is also a science of transferring loads in an optimal situation.

And using proper detailing methods, it is helpful to avoid connection failures.

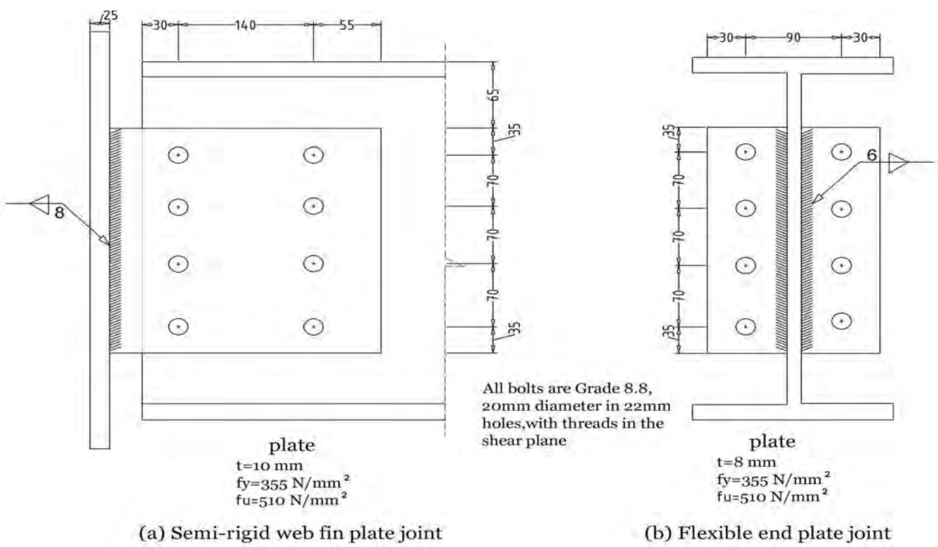



Figure 5.1: Frame connection detail

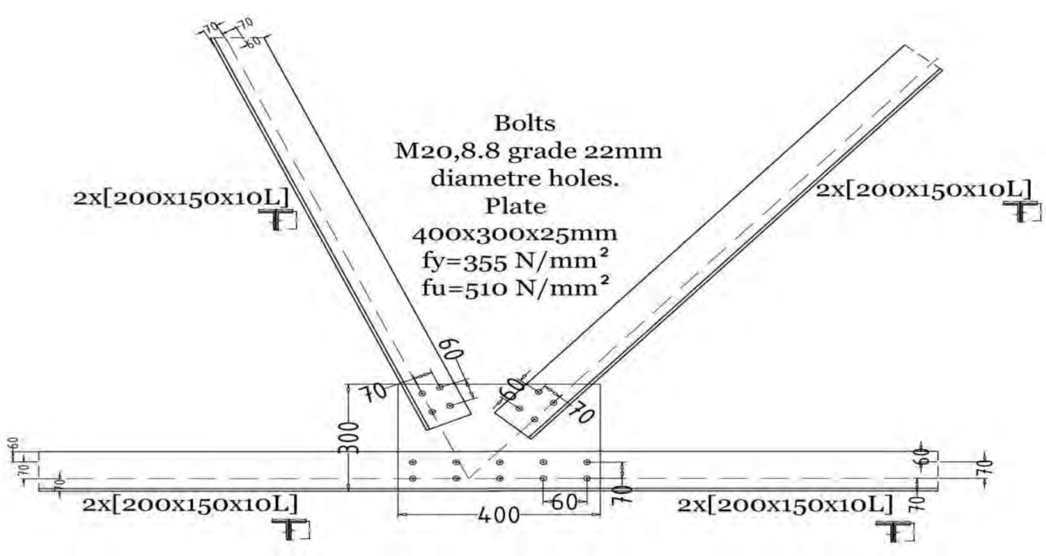



Figure 5.2: Truss connection detail

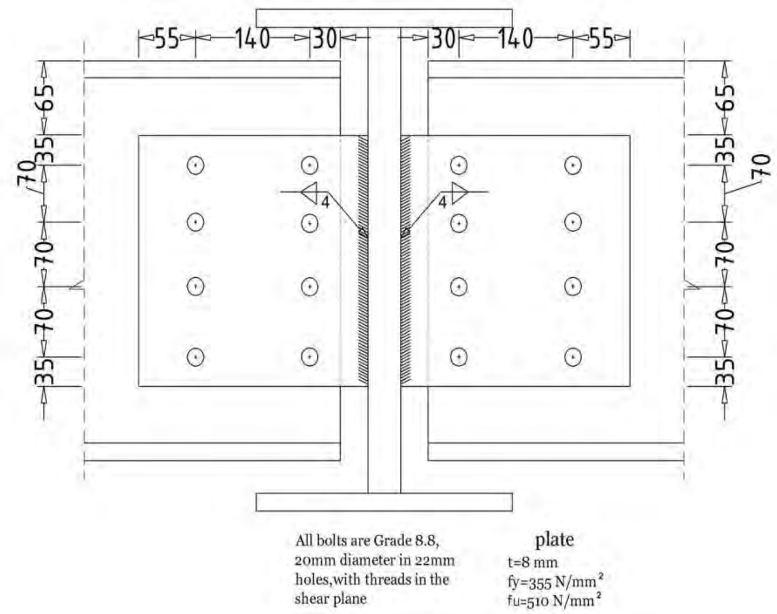



Figure 5.3: beam connection detail

### 5.1.1 Rigid connection

The joints that are having less rotation with high resistance of moment are a type of connections which are rigid to allow rotation.

The connection between Floor beams and truss joints have a tendency of being fixed and also for splice members, the rotation amount shall be negligible so that it can transfer or withstand the applied moment.

Thus, such connection type is achieved by providing high stiffness connection element and fixed-fixed connection modeling and by attaining full strength connection but no partial strength is advised.

The qualitative representation suggests that fully welded connections, extended end plate, and top and bottom flange splices could be considered as rigid restraints.

### 5.1.2 Semi rigid Connection

This type of connection as it has been shown in Fig.5.1(a) is having a rotation tendency with some amount of moment of rotation at the joint and this type of connection is susceptible to fail during high rotational value.

Semi-rigid connections are used in joints that are having less moment, like the connections of stringers with floor beams.

For semi-rigid connections, some degree of flexibility can be allowed, Partial strength connections are considered to be within the semi-rigid or flexible range.

### 5.1.3 Flexible Connection

Flexible connections are type of connections that are totally free to rotate (see Fig. 5.1(b) and Fig. 5.2) and these types of connections are having less stiffness to rotation capacity.

The main connections of the trusses are considered as having high degree of flexibility and also it is a partial strength connection, which is pinned type of connection.

### 5.1.4 Summary of Results

The connections are ideally classified as rigid, semi-rigid or flexible according to stiffness behavior and also according to strength; they are classified as partial or full strength connection. And the total system of connections is debuted in Fig.5.4.

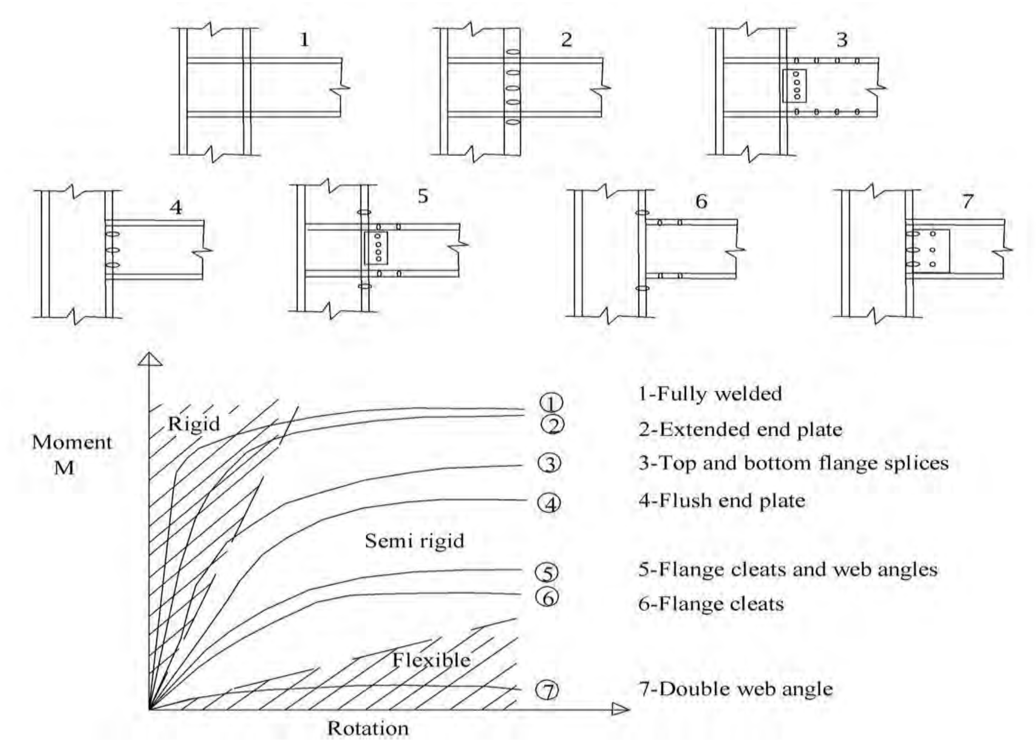



Figure 5.4: Moment and Rotation relations of connections

But connections are classified in realty as flexible or rigid, there is no in between and this is shown in the Figure 5.5, with qualitative measures.

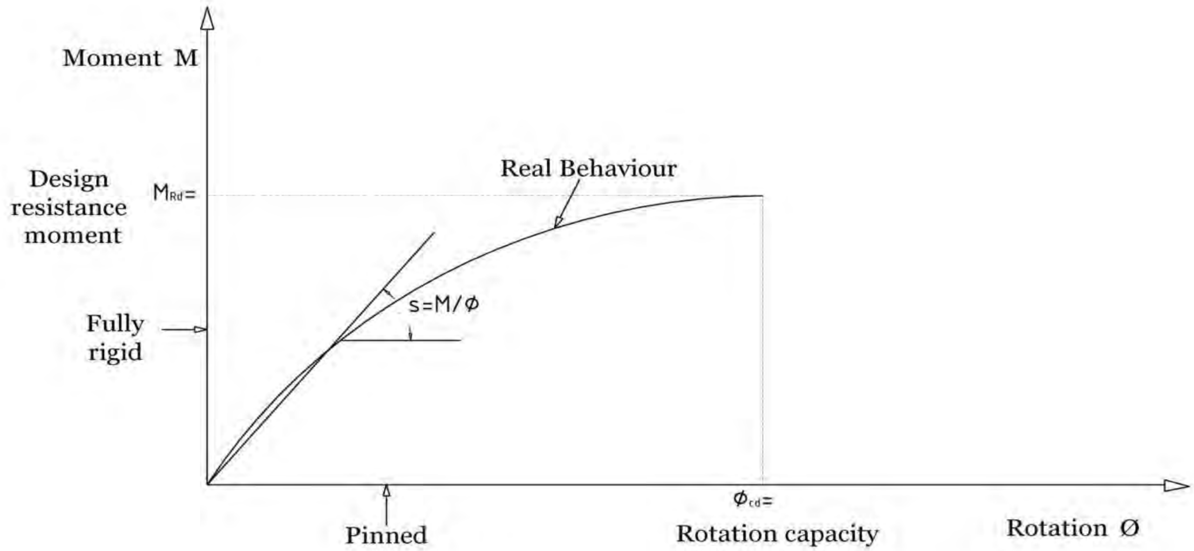



Figure 5.5: Qualitative characteristic of connections

# 6.0 Conclusion and recommendation

### *6.1 Conclusion*

The study of connections has addressed the need of proper detail computational analysis. All the analysis, design and construction process of structural connections is very helpful to have a reliable and economic truss joint; that could withstand or transfer stresses. For many long span steel truss bridges in road or highway construction projects, the demand of proper computational methods and standards is high. In this study the possible approaches of steel truss bridge connections has been assessed. Each connection parts are responsible for safe dissipation of energy or loads and this will be very important when the span of the assembled chords is large and same time when dead load shall not be large to protect excessive deflections.

The fabrication and erection of connections is one of the things discussed in the study and found, the methods adopted from international practices shall be improved in away to have the specific country cases which comprises the level of the workmanship skill and the country's economy. Connection construction is not as simple as drafting the joints detail.

All the concepts, theories, principles discussed in the study and the practical mechanics that has been dealt for the clear understanding of the whole state of the art was properly stated.

Connections shall have proper design procedures and detailing process to ensure the balance of safety and economy within the margin of available manufacturing recommendations and the design values.

The study of this research might be an input to standard (manual) preparation for responsible governmental bodies and for organizations that are working on the areas of design and construction engineering.

### *6.2 Recommendation*

Based on the study the following recommendations are made:

1. In ERA Bridge design manual, there need to be said more about connections of long span steel truss bridge.

2. A counter check on quality of connections needs to be conducted on imported steel bridges which are already connected.

3. The reliability measurement of connections shall be studied and standardized for minimizing the failures.

4. The design and construction sector of the country needs to be aware of the science and art of connections for long span steel truss bridge.

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**Declaration**

I the undersigned, declare that this thesis is my work and all sources of materials used for this Thesis have been properly acknowledged.

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**Signature:**

**Place:** Addis Ababa University

**Date of Submission:** April 2012