ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

ANALYSIS AND DESIGN OF ASSEMBLED STEEL TRUSS BRIDGES

TIHITINA SIYOUM

OCTOBER 2007

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

PROF. NEGUSSIE TEBEDGE
THESIS ADVISOR

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<u>Approved by Board of Examiners</u>

Prof . Negussie Tebedge		
Advisor	Signature	Date
External Examiner	Signature	Date
Internal Examiner	Signature	Date
Chairman	Signature	Date

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1. Introduction

1.1. Background

Ethiopia is a mountainous country characterized by topographic conditions such as rough terrain, deep gorges, river crossings and other factors that make construction of crossways such as culverts and bridges mandatory in road construction. The ease of assembling and erecting panelized steel truss bridge makes it a good alternative for use in remote areas. But these are usually imported from abroad. The high cost of importing it is one of the reasons that prohibit its widespread application in inaccessible areas and for temporary works.

The design and fabrication of this panelized steel bridge with locally produced steel sections is a possible solution for this problem. This research explores this possibility. It gives analysis and design procedures clarified by an example.

1.2. Objective

In our country most of the permanent bridges are made from reinforced concrete and imported steel bridges being used mainly as temporary replacement for failed bridges and in locations where access is not easy. The main objective of this thesis is to promote the use of locally available steel profiles for the production and use of assembled steel truss bridges.

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

1.3. Thesis Content

The study of the thesis mainly focuses on literature review of books and design standards of bridges. Based on the main objective of the thesis, the study has focused on developing analysis and design procedure for a steel truss bridge. In addition to this the developed guideline will be utilized in the design of up to 21m span steel truss bridge.

1.4. Applications and Limitations

The result of this study can be applied to design steel truss bridge with local steel profiles. The study shall benefit the client and constructors as these bridges would only take a short time to produce and assemble. This enables the quick restoration of traffic.

The scope of the study has been limited to the preparation of analysis and design procedure for main panel only. This research may be used as a basis for future study to include the design of other parts of the bridge so that the whole bridge could be designed and manufactured using local steel profiles. Finally, the application of analysis and design procedure is illustrated through practical design example of a sample and common span of a bridge.

2. Bridge Structure

2.1. General

A bridge is a structure that crosses over a gorge, road, river, railway, or other obstructions, permitting smooth and safe passage of vehicles, trains and pedestrians. A bridge structure is divided into an upper part (the superstructure), which consists of the slab, the floor system, the main trusses or girders, and a lower part (the substructure), which are columns, piers, towers, footings, piles and abutments [3].

The super structure provides horizontal spans such as deck and girders and carries the traffic loads directly. The function of sub structure is to support the super structure of the bridge.

2.2. Classification of Bridges

Bridges can be classified using different criteria, some of which are listed below.

1. Classification by function

    Highway bridges: bridges on highway

    Railway bridges: bridges on railway

    Aqueduct bridges: bridges supporting pipes

    Viaduct bridges: bridges over a highway

    Foot bridges: bridges carrying pedestrian traffic

2. Classification by material

    Timber bridges: wooden bridges are used when the span is relatively short.

    Concrete bridges: This can be reinforced or pre-stressed concrete.

    Steel bridges: A steel bridge may use a wide variety of structural steel components and systems.

    Masonry Bridge: This may be made of brick or stone masonry in arch form.

3. Classification by relative position of floor

    This classification is based on the location of flooring deck with respect to the supporting structures.

    Deck bridge: the deck or flooring is supported at the top of supporting structure.

    Semi-through bridge: The flooring is supported at the intermediate level of the supporting structures.

    Through bridge: The flooring is supported at the bottom.

4. Classification by method of providing clearance for navigation

   Swing bridge: It is supported on a center pier and is rotated horizontally.

   Vertical lift bridge: The movable span is lifted vertically above the navigation area.

   Bascule bridge: The span is turned vertically up at one end, usually by some type of counterweight system.

5. Classification by structural system

   Slab bridge: This is usually of solid rectangular section. Such a bridge is generally constructed of reinforced concrete or pre-stressed concrete.

   T-beam girder bridge: a number of reinforced concrete T-beams are placed side by side to support the load.

   Box girder bridge: the single (or multiple) main girder consists of a box beam, which resists not only bending and shear but also torsion effectively.

   Composite girder bridge: the concrete deck slab works in conjunction with the steel girders to support the load as a united beam.

   Truss bridge: Truss bar members are assumed to be connected with pins at their ends. Each member resists an axial force, either in compression or tension.

   Arch bridge: The arch is a structure that resists load mainly in axial compression.

   Cable Stayed bridge: The highly strengthened cables that stem directly from the tower support the girders. These are most suited to bridge long distances.

   Suspension bridges: Hangers tied to the main cables, which hang from the towers, suspend the girders. The load is transmitted mainly by tension in cable.

6. Classification by support condition

   Simply supported bridge: The main girders or trusses are simply supported; thus they can be analyzed using conditions of equilibrium.

   Continuously supported bridge: Girders or trusses are continuously supported, resulting in a structurally indeterminate system.

   Cantilever bridge: a continuous bridge is rendered determinate by placing intermediate hinges between the supports.

7. Classification depending on the life of the bridge

Temporary bridge: A bridge that is used for short time and is then demolished and used in other areas whenever the need arises as in military bridges.

Permanent bridges: Bridge that is used through out its lifetime.

Based on the above classification, the study of the thesis will focus on simply supported from support condition, on through type from relative position of floor, on steel from material type and on truss from structural system. Therefore, a simply supported through truss bridge made of steel is studied in this thesis.

2.3. Selection of Bridge Type

Selection of bridge type requires the collection of extensive data from which possible options are chosen.

Factors Affecting the Selection of Type of Bridge

The following factors govern the selection of type of bridges: [4]

1) Volume and nature of the traffic
2) The nature of the river and its bed soil
3) The availability of materials and fund
4) Time available for construction of the bridge.
5) Physical feature of the country.
6) Whether the river is used for navigation purposes or not
7) Availability of skilled and unskilled workers
8) Facilities available for erection of bridge and maintenance.
9) Economic length of the span.
10) Level of high flood level and the clearance required
11) Climatic condition
12) Strategic condition
13) Hydraulic data
14) Length and width of the bridge
15) Foundation conditions for piers and abutment
16) Live load on the bridge
17) Appearance of bridge from aesthetic point of view

The above factors are taken into account and the type that is most economical and can give maximum service and satisfaction is designed.

Some of the factors considered to be the main criteria for the selection of type of bridge studied in the thesis are:

- Physical feature of the country.
- Availability of construction materials and fund
- Time available for construction of the bridge
- Availability of skilled and unskilled manpower

2.4. Steel Truss Bridges

2.4.1. Description

A truss is an assemblage of long, slender structural steel elements that are connected at their ends. In design theory, individual members of a truss are subject to axial tension or compression forces and not to bending forces. Trusses find substantial use in modern construction, for instance as bridges, towers, roof trusses etc.

These trusses can be made of timber, steel or can be composite structure. In this thesis, steel trusses used for building bridges are considered. Steel has higher strength, ductility and toughness than many other structural materials such as concrete or wood. However steel must be painted to prevent rusting. [3].

Like other bridge types, there are both simple and continuous truss bridges. The members of a truss can be arranged in an almost unlimited number of ways, but the vast majority of trusses encountered in bridge or building work belong to one of the common types listed below. Some of these common types of trusses are the Warren truss, Warren truss with verticals, subdivided Warren truss, the Pratt truss, subdivided Pratt (Baltimore) truss, K truss, and the Howe truss.

2.4.2. Why use Steel Truss bridges?

Steel truss bridges are made of panels that are pin connected or fixed type to form the bridge. Fabrication of panels by welding is conducted in the shop (at production site) where the bridge components are prepared before being assembled on the construction site. The relatively small size of individual parts makes it the ideal bridge for places where it is difficult to transport large parts or where large cranes and heavy equipment can not be used during erection or at remote places where it becomes very expensive to build reinforced concrete bridges.

2.4.3. Steel Bridges in Ethiopia

According to information gathered from Ethiopian Roads Authority, steel bridges were in use as early as 1945. The types of steel bridges used in Ethiopia are steel truss bridges, steel girder bridges and temporary steel panel bridges. Currently the federal road network has 50 temporary panel bridges, 7 permanent steel bridges and 11 steel girder bridges under it [see Appendix A]. Some of these bridges have reinforced concrete deck, others timber deck and some are of steel deck. Temporary steel bridges imported as close as 10 years back were with timber deck. Usually such kind of deck serves only for short time depending on volume of traffic. When it wears out it is replaced by local timber made from Eucalyptus tree, which brings about undesired dynamic effect and movement of the span while vehicles are passing. Thus overload plus over speed coupled with irregular driving sometimes brings about failure of these bridges. Rohalomi bridge is an example of such a failure. It was a 20mt long, single span steel truss bridge (Bailey type) with timber deck located in Gonder on the Gonder-Humera road.

3. Analysis and Design of Steel Truss Bridges

3.1. Bridge Loading

The various types of loading which need to be considered can broadly be classified as permanent, or 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 through out 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 are

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

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

3.1.1 Dead Load

The dead load on superstructure is the weight of all structural elements and non-structural 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.

3.1.2 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-2002, Ethiopian Roads Authority

Bridge Design Manual-2004, Addis Ababa City Roads Authority

In Ethiopia the design vehicle live load on roadways of bridges, designated HL-93, shall consist of a combination of the:[1]

- design truck or design tandem
- 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 shall be as specified in Fig. 3.1.

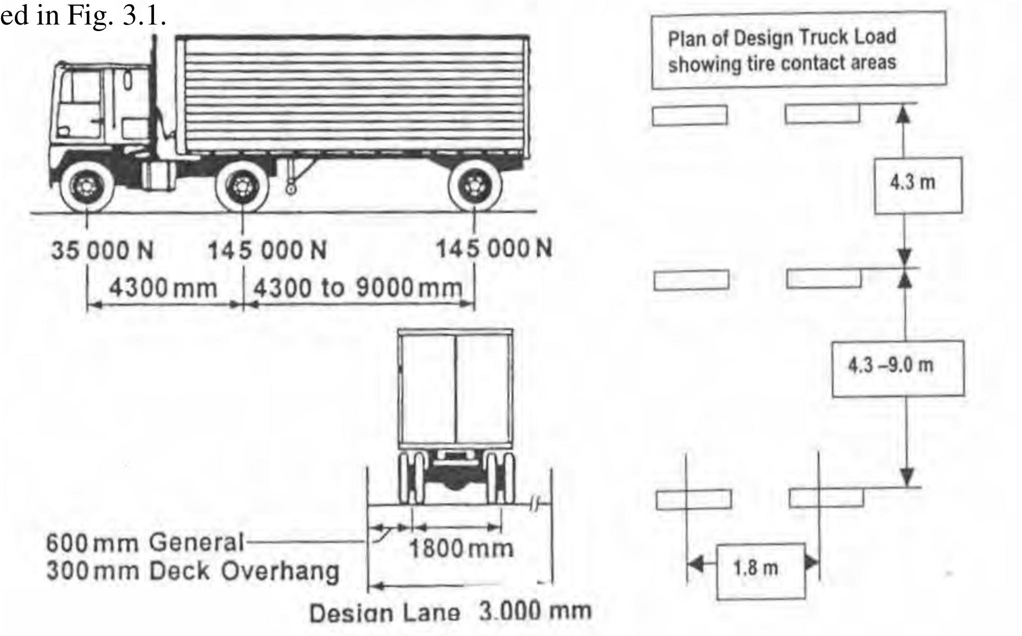


Figure 3.1 The Design Truck

A dynamic load allowance shall be considered [1]. 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 [1]. The design tandem load is shown in the Figure 3.2

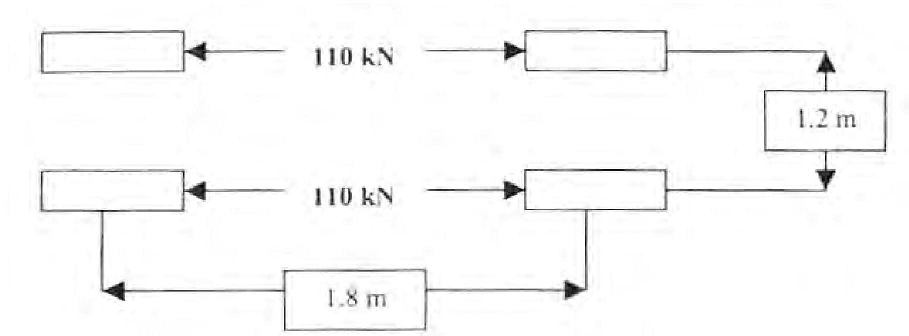


Figure 3.2 Design Tandem Loads

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 [1].

3.1.3 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[5].

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 3.1 [1].

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 3.1 Dynamic Load Allowance, IM

Component	IM
Deck Joints – All Limit States	75%
All Other Components	
Fatigue and Fracture Limit State	15%
All Other Limit States	33%

3.1.4 Wind Load

Wind forces are resisted by the bracing systems for a through bridge. The bracing systems are neither analyzed nor designed in this thesis since the load is considered insignificant.

3.1.5 Earthquake Loading

Seismic analysis is not required for single-span Bridge, regardless of the seismic Zone [1].

3.2. 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 3.3 and shall satisfy Eq. (3.1).

$$\sum \eta_i \gamma_i Q_i < \varphi R_n = R_f \tag{3.1}$$

Where:

for loads for which a maximum value of γ_i is appropriate:

$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$

for loads for which a minimum value of γ_i is appropriate:

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

where:

η_i = load modifier: a factor relating to ductility, redundancy, and operational importance

γ_i = load factor: a statistically based multiplier applied to force effects

Q_i = force effect

φ= resistance factor: a statistically based multiplier applied to nominal resistance

R_n = nominal resistance

η_D = a factor relating to ductility, as specified below

η_R = a factor relating to redundancy as specified below

η_I = a factor relating to operational importance as specified below

R_f = factored resistance: φR_n

Table 3-2 - Load Combinations and Load Factors

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

where:

BR = vehicular braking force

CE = vehicular centrifugal force

CR = creep

CT = vehicular collision force

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

ES = earth surcharge load

γ_p = load factor for permanent loading

FR = friction

LL = vehicular live load

PL = pedestrian live load

SH = shrinkage

TU = uniform temperature

WL = wind on live load

γ_{SE} = load factor for settlement

DC = dead load of structural components

DD = down drag

DW = dead load of wearing surfaces and utilities

EH = horizontal earth pressure load

γ_{EQ} = load factor for live load with seismic load

EQ = earthquake load

EV =vertical pressure from dead load of earth fill

γ_{TG} = load factor for temperature gradient

IM = vehicular dynamic load allowance

LS = live load surcharge

SE = settlement

TG = temperature gradient

WA =water load and stream pressure

WS = wind load on structure

3.3. Load Application

To understand the manner of application of loads to a supporting truss it is necessary for the arrangement of the members of the floor system to be carefully studied. The most common type of bridge floor is supported by a series of stringers parallel to traffic and running the length of each panel. The stringers frame into and are supported by the floor beams (transoms). The floor beams frame into panel points of the supporting trusses. The stringers and floor beams are for simplicity assumed to have simple end supports (see Fig. 3.3).

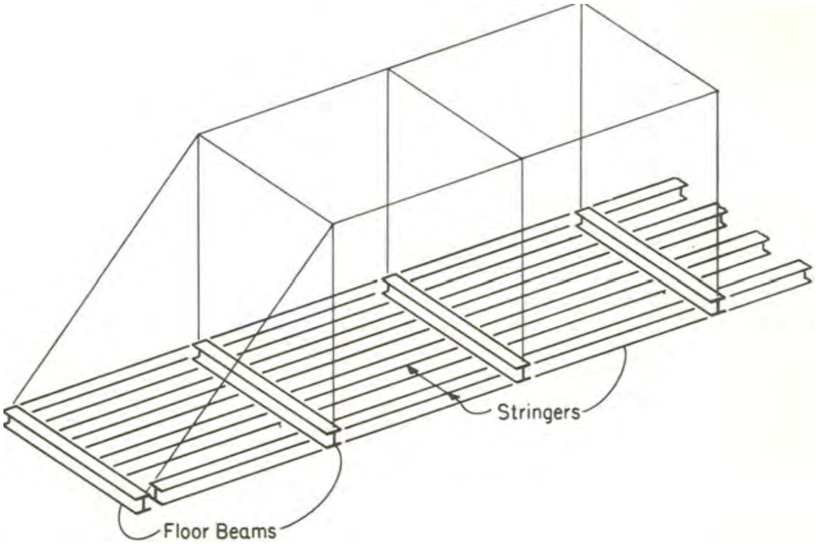


Figure 3.3 Arrangement of floor system

A few explanatory remarks are made here concerning the application of the wheel loads. Even though a wheel load is directly placed over an interior stringer it causes the adjacent stringers as well as the stringer in question to deflect due to the stiffness of the bridge floor. The obvious result is the distribution of the load and the AASHTO Article 3.23 and Table 3.23.1 specifications attempt to estimate the effect of this distribution. They say the percentage of a wheel load to be taken is the center to center spacing of the stringers divided by a certain value which is determined from the

- type of floor
- spacing of stringers
- number of traffic lanes

For steel bridge with steel grid (less than 4" thick)

$DF = S/4.5$ For bridge designed for one traffic lane

$DF = S/4.0$ For bridge designed for two or more traffic lanes

where S is the average stringer spacing in feet.

In calculating bending moment due to live load in stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows.

3.4. Interior Stringers

Interior stringers are subjected to dead load of the bridge floor and their own weight. Live load bending moments are computed using one set of front and rear wheels multiplied by distribution factor given in AASHTO Table 3.23.1. Impact is then calculated. Live load bending moment coming from lane load is also computed. The moments are summed up and used to calculate the section modulus required. A suitable interior stringer is then selected.

3.5. Exterior Stringers

Exterior stringers are subjected to dead load coming from bridge floor, their own weight, curbs, and railings. The live load bending moment is then calculated by applying to the stringer the reaction of the wheel load determined by assuming the flooring to act as a simple span between stringers (AASHTO Art. 3.23.2.3.1.1). Live load bending moment coming from lane load is also computed. The calculation of impact and use of total moments for the section modulus requirement is the same as in the interior stringer. A suitable exterior stringer is then selected. In no case shall an exterior girder have less carrying capacity than an interior stringer.

3.6. Floor beams

These are subjected to dead load reactions from outside and interior stringers. These reactions are applied as concentrated loads to floor beams. AASHTO Article 3.23.3 specifications do not allow transverse distribution of wheel loads for design of floor beams. Therefore, concentrated live loads applied to floor beams from stringers are calculated and placed in the design lane to cause maximum live load moment. Live load bending moment coming from lane load is also computed. Impact moment is then calculated, the total moment summed up, the required section modulus calculated and used for the selection of suitable floor beam.

3.7. Live load truss stresses

The live load stresses are to be computed for the HL93 loading. The design lane loading is 9.3 kN/m uniformly distributed in the longitudinal direction and across 3m width and is to be placed in design traffic lane. The number of design traffic lane is determined by taking the integer part of *w*/3000 where *w* is the clear roadway width between curbs and/or barriers. The lane loading is moved as close as possible to one of the trusses to cause maximum loads on that truss. From this arrangement, the live load proportion going to the truss can be determined. The same is done for wheel loads and the maximum percentage of the concentrated wheel loads going to the truss can be determined. The impact percentage for various truss members will be determined as given in Table 3.1.

3.8. Truss dead loads

The dead load of a truss bridge consists of the weight of the floor system, truss, and bracing. The weight of the floor system, which comprises a large percentage of the total weight, can closely be estimated by making a preliminary design of the floor. From the weight of the floor system and bracing the load applied to the truss can be determined. Before dead load analysis of the truss can be made of the main trusses the weights of the trusses must be estimated.

When using softwares, the self weight of the truss is calculated by the software. When using hand calculation, the weight of the truss can be estimated by increasing the other dead loads by some percentage or by using some approximate formula. The Hudson formula, given as Eq. (3.2), can be used for estimating the weight of the truss.

$$W = 17\,S\,L/\,s \tag{3.2}$$

where:

W is the total weight of the bridge truss including its bracing

S is the maximum total tensile stress in the most stressed chord member

L is the length of the truss in feet

s is the allowable tensile stress

Since S is the maximum total tensile stress due to live load plus impact plus dead load, a value has to be assumed for the truss weight to apply the formula. For this effect, the largest tensile chord member is assumed to extend for the full length of the truss and its weight is assumed to equal 20 percent of the weight of the entire truss and bracing.

3.9. Selection of truss member

Article 10.7 of AASHTO need to be studied before the truss members are selected. In this article the limiting lengths of members are specified as the maximum slenderness ratio, KL/r, for compression and tension members. For the main tension members 200 is the maximum value and 240 for bracing members. For compression members the comparable values are120 and 140.

3.10. Bridge-truss deflection

Members having simple or continuous spans preferably should be designed so that the deflection due to service live load and impact shall not exceed 1/800 of span [2]. According to ERA clause 2.6, deflection due to vehicular load and impact shall not exceed 1/500 of span. Trusses with horizontal bottom chords will sag down in the middle, affecting their appearance and perhaps worrying the users of the bridge. These trusses should desirably be cambered to avoid such sagging.

4. Selected Bridges Survey and Design Example

4.1. Assessment of Locally Built Baily Bridges

Site visit was conducted on existing Baily bridges in Addis Ababa and on different parts of panel bridge out side of Addis Ababa to appreciate the design problem.

Site visits were made at the Alemgena panel bridge parts store of Ethiopian Roads Authority in Alemgena district and the visit area within Addis Ababa was the Baily bridge at the entrance of Addis Ababa Tennis Club (see Fig. 4.1).

Figure 4.1 Baily bridge at the entrance to the Addis Ababa Tennis Club

Bridge Location: At the entrance to the Addis Ababa Tennis Club
Crossing: Filwoha River
Bridge Type: Baily bridge with timber deck
Number of spans: 1
Span Length: 15.20m
Roadway width: 3.35m
Foundation: Stone masonry head wall and wing wall

During the site visit at Alemgena, pictures (see Fig. 4.2) and measurements of different parts were taken for the accompanying study. The measurements are listed below.

Figure 4.2 Panel bridge parts at ERA's store in Alemgena District.

-Panel

-made of top chord, bottom chord, verticals, diagonals and connection plates with the following measured dimensions (see Fig. 4.3).

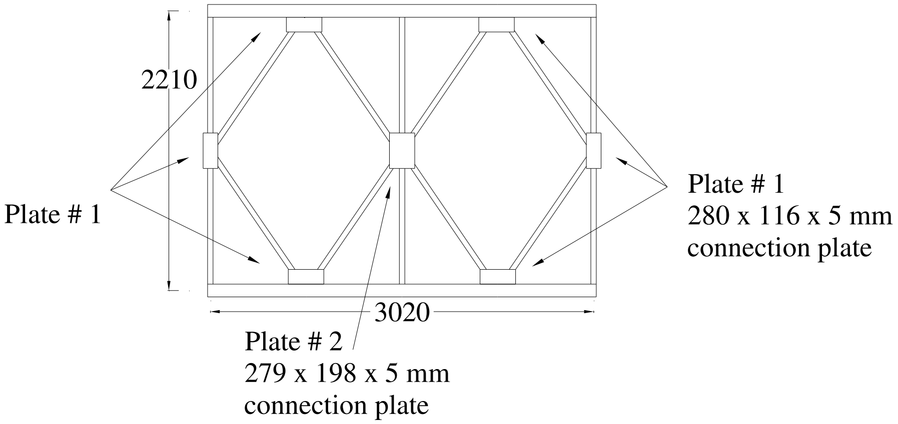


Figure 4.3 Measured dimensions of locally available panel bridge part at ERA's store in Alemgena District

-Bottom/ top chord

-made of two channels with the following measured dimensions (see Fig. 4.4).

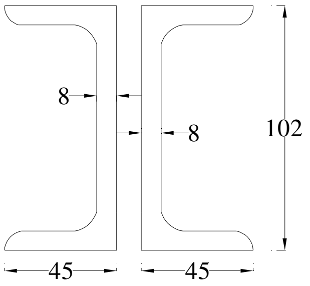


Figure 4.4 Measured dimensions in mm of bottom /top channel

- Cross-sectional area = 1368 mm^2

-Vertical/ diagonal members

-made of single channel with the following measured dimensions (see Fig. 4.5).

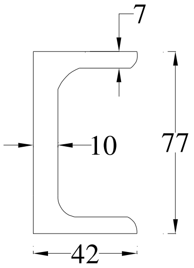


Figure 4.5 Measured dimensions in mm of vertical / diagonal channel

- Cross-sectional area = 1129.00 mm^2

Using the above data as a starting point a local trial panel section shown in Fig. 4.6 is chosen. The panel members, stringers and floor beams are made of hollow rectangular sections chosen from Kality Metal Products Factory products. The hollow rectangular sections used in the trial panel are shown in Fig. 4.7 and Fig. 4.8. Steel deck section is assumed and standard ERA specification loading conditions are applied on it. These loads are transferred to the stringers and floor beams, which are lattice girders, and then analyzed. The section capacities of the stringers and floor beams are checked for the applied loading and the load is transferred to the truss. The section capacities of the truss are then checked for sufficiency in the following structural arrangement.

-Panel

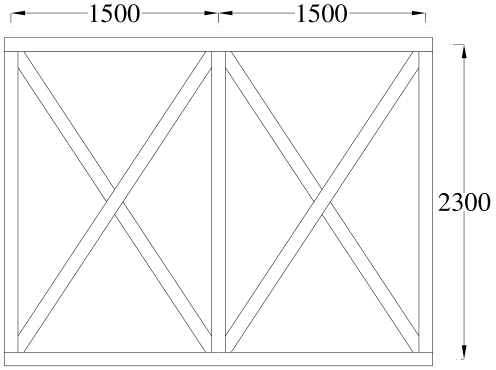


Figure 4.6 Assumed dimensions of trial panel in mm

-Bottom/ top chord

-made of two RT128.

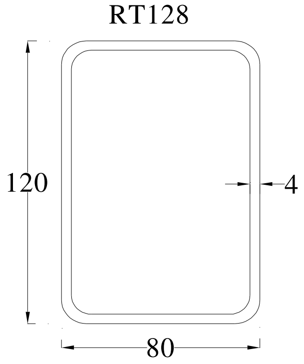


Figure 4.7 Dimensions of RT128 in mm

-available cross-sectional area = 1495 mm^2

-Vertical/ diagonal members

-made of single ST100.

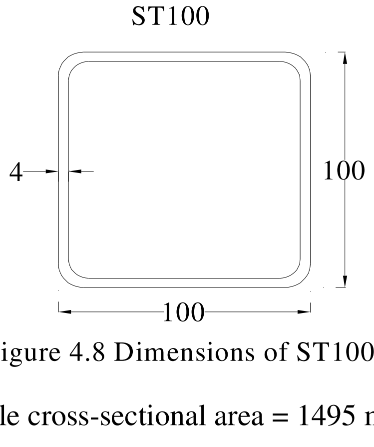


Figure 4.8 Dimensions of ST100 in mm

-available cross-sectional area = 1495 mm^2

4.2 Analysis and Design Example

4.2.1 X-Shaped Panel

To illustrate the application of the design principles of the panel, a steel truss bridge of 21m span with cross section shown in Fig. 4.9 is considered.

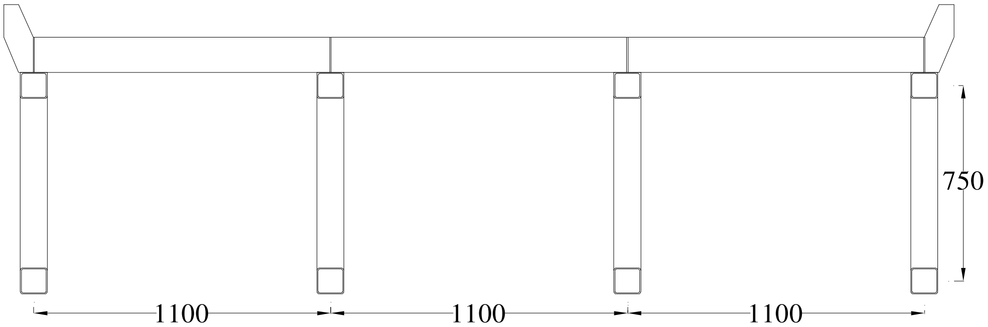


Figure 4.9 Cross section of steel bridge showing deck and stringer in mm

4.2.1.1 Design Data

The following assumptions are used.

- The 21m span is made up of seven panels each 3m long.
- The road way width is 3300mm.
- The center-to-center distance of truss is 4060mm.
- The center-to-center distance of floor beams is 3000mm.
- The center-to-center distance of stringers is 1100mm.
- Structural hollow sections of Kality Metal Products Factory with yield strength of 250MPa and ultimate tensile strength of 343MPa are used to construct the trusses, stringers and floor beams.
- A steel deck with steel curb at edge currently used by ERA is assumed designed and only its load is considered [6].
- A preliminary design of stringers and floor beams will be done.
- For live loads, ERA live loading of HL-93 are used.

4.2.1.2 Design of stringers

Interior and exterior stingers are designed separately.

Interior stringer design

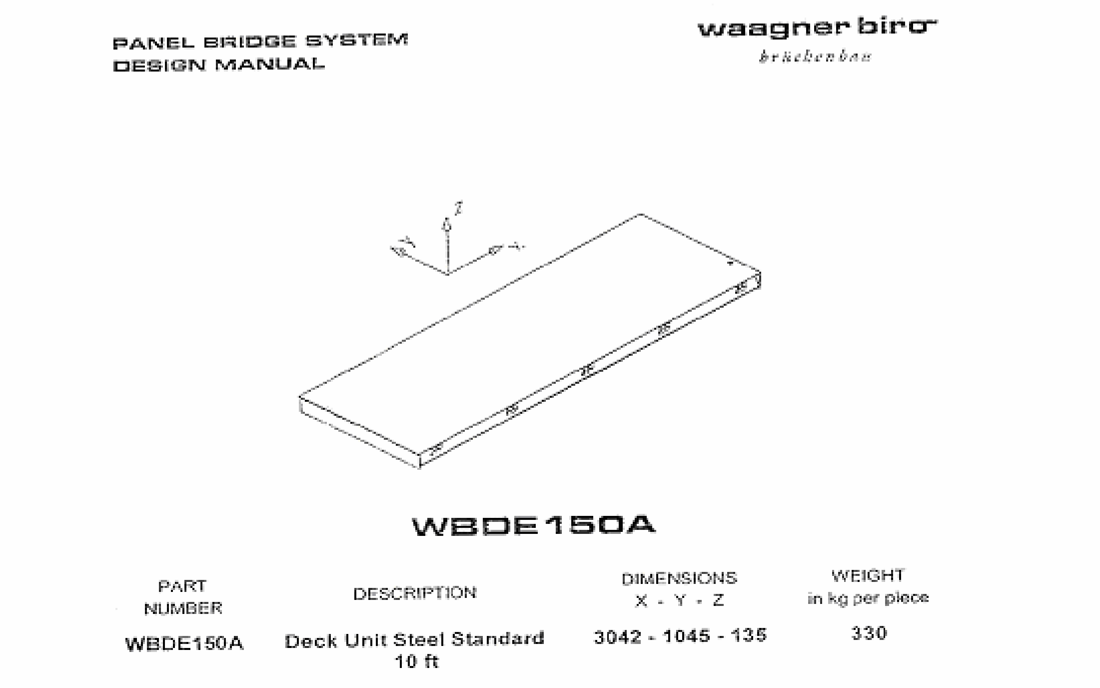
Dead loads

Steel deck weight = (1018.37) (1.10) = 1120.21 N/m

Span of stringers = panel length = 3m

Assume built up RHS lattice stringer from ST100 with f_y = 250 MPa and mass 11.73 kg / m (see Fig. 4.12 and Fig 4.13)

The steel deck and curb currently used by ERA is shown in Fig. 4.10 and Fig 4.11 respectively.



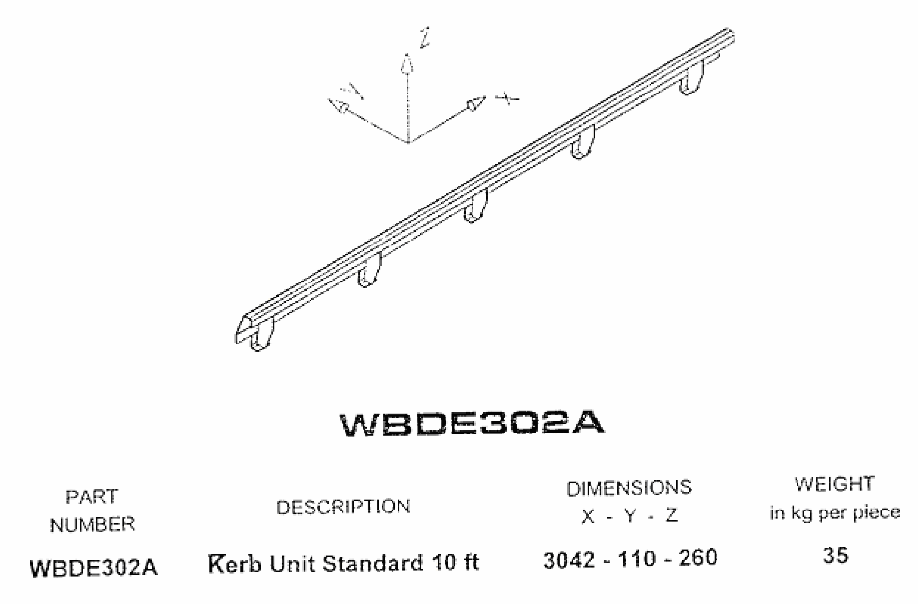
PANEL BRIDGE SYSTEM
DESIGN MANUAL

waagner biro
brückenbau

WBDE150A

PART NUMBER	DESCRIPTION	DIMENSIONS X - Y - Z	WEIGHT in kg per piece
WBDE150A	Deck Unit Steel Standard 10 ft	3042 - 1045 - 135	330

Figure 4.10 Steel deck unit currently used by ERA



WBDE302A

PART NUMBER	DESCRIPTION	DIMENSIONS X - Y - Z	WEIGHT in kg per piece
WBDE302A	Kerb Unit Standard 10 ft	3042 - 110 - 260	35

Figure 4.11 Steel curb unit currently used by ERA

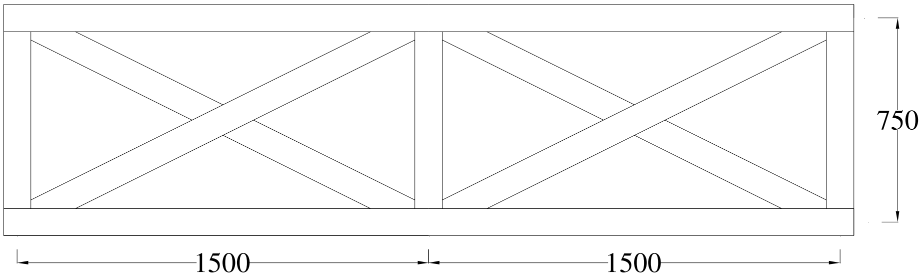


Figure 4.12 Lattice stringer made from 100mm SHS section

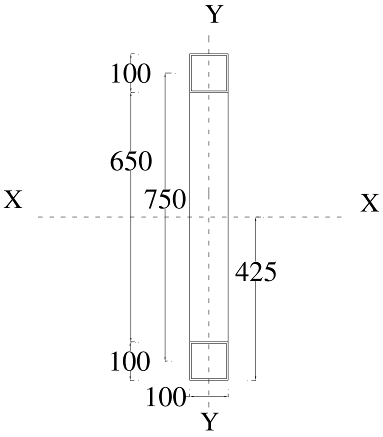


Figure 4.13 Cross section of lattice stringer

Total length of stringer members = 14.96 m

Total weight of stringer members = (11.73) (9.81) (14.96) = 1721.47 N/m

Weight per meter of stringer = 1721.47 / 3 = 573.82 N/m

Stringer weight = 573.82 N/m

Total dead load = 1120.21 + 573.82 = 1694.03 N/m

Dead load moment = (1694.03) (3) (3) / 8 = 1905.79 Nm = 1.91 kNm

Live Loads

Truck Load with impact (33%):

P_1 = (1.33) (145) = 192.85 kN; for axle load of 145 kN

P_2 = (1.33) (35) = 46.55 kN; for axle load of 35 kN

Tandem Load with impact (33%):

P_3= (1.33) (110) = 146.30 kN; for tandem load of 110 kN

Lane Load: 9.3 kN/m uniformly distributed in the longitudinal direction and occupying 3m width transversely. The line load per meter on stringer is (3/4.06) (9.3) = 6.64 kN/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 = (1.10/0.3048)/4.5 = 0.802

For truck load

As the axle spacing is greater than the span of the stringers, only one axle can be placed on the stringer at a time. The maximum load occurs when the heavier axle load is on the span (see Fig. 4.14).

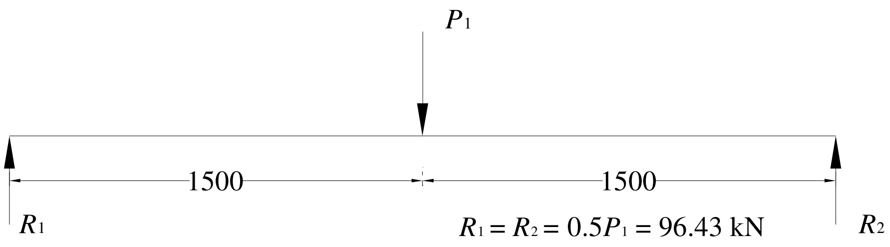


Figure 4.14 Truck load on interior stringer

Maximum truck load moment = (96.43) (1.5) (0.802) = 116.01 kNm

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) (1.91) + (1.75) (116.01) = 205.41 kNm

Total service truck and dead moment = (1.00) (1.91) + (1.30) (116.01) = 152.72 kNm

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. 4.15.

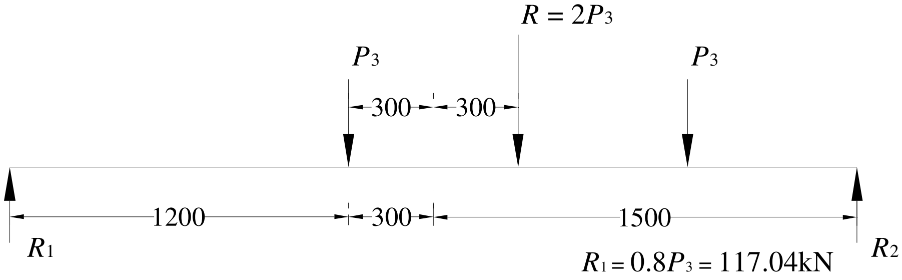


Figure 4.15 Tandem load on interior stringer

Maximum tandem load moment = (117.04) (1.2) (0.802) = 112.64 kNm

Total limit tandem and dead moment = (1.25) (1.91) + (1.75) (112.64) = 199.51 kNm

Total service tandem and dead moment = (1.00) (1.91) + (1.30) (112.64) = 148.34 kNm

For lane load (see Fig. 4.16)

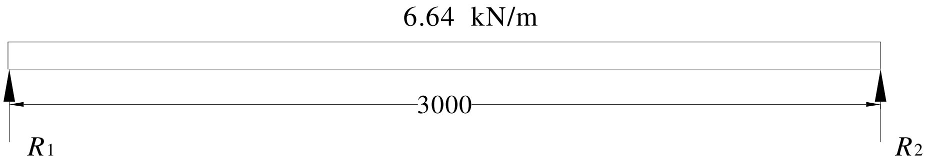


Figure 4.16 Lane load on interior stringer

Maximum lane load moment = (6.64) (3) (3) / 8 = 7.47 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 4.1.

Table 4.1 Combination of truck/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)	152.72 + (7.47) (1.75) = 164.79	
2	Truck load and dead load with lane load (limit)	205.41 + (7.47) (1.75) = 218.48	Maximum
3	Tandem load and dead load with lane load (service)	148.34 + (7.47) (1.75) = 161.41	
4	Tandem load and dead load with lane load (limit)	199.51 + (7.47) (1.75) = 212.58	

Therefore the design moment is

$MD = 218.48$ 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

$f_b = f_y / \gamma_{M1} = 250 / 1.1 = 227.27$ MPa

$S = 218.48 / ((227) (1000)) = 0.000962511 \text{ m}^3 = 962.51 \times 10^3 \text{ mm}^3$

Check section modulus of lattice stringer:

SHS 100 x 100 x 4 mm (see Fig. 4.13)

$A = 1495 \text{ mm}^2$ $I = 2{,}264{,}000 \text{ mm}^4$

$d = 800 / 2 = 400\text{mm}$

$I_{xx} = (2)\ (I_o + A\, d^2) = (2)\ (2264000 + (1495)(375)(375)) = 424.97 \times 10^6 \text{ mm}^4$

$C_y = 425$ mm

$S_{xx} = I_{xx} / C_y = 999.99 \times 10^3 \text{ mm}^3 > 962.51 \times 10^3 \text{ mm}^3$

The section is sufficient.

Exterior stringer design

Dead loads

Steel deck weight = (1018.37) (1.10) (0.5) = 560.10 N/m

Weight of steel curbs = (1026.09) (0.11) = 112.87 N/m

Span of stringers = panel length = 3m

Assume the same built up SHS lattice girder from ST100 with f_y = 250 MPa and mass 11.73 kg / m

Stringer weight = 573.82 N/m

Total dead load = 560.10 +112.87 + 573.82 = 1246.79 N/m

Dead load moment = (1246.79) (3) (3) / 8 = 1402.64 Nm =1.40 kNm

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. 4.17.

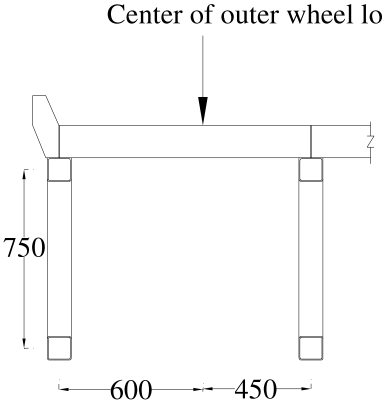


Figure 4.17 Position of wheel loads for exerior stringer

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. 4.18)

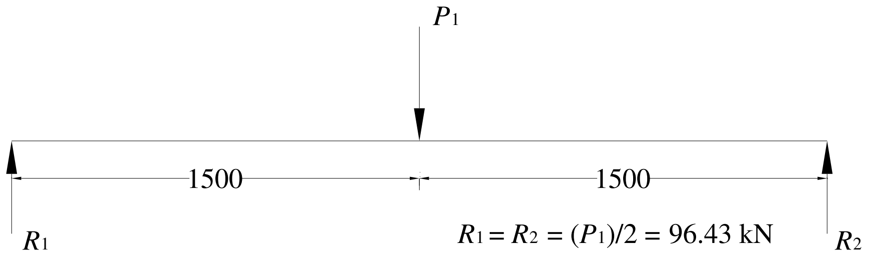


Figure 4.18 Truck load on exterior stringer

Maximum truck load moment = (96.43) (1.5) (0.43) = 62.20 kNm

Total limit truck and dead moment = (1.25) (1.40) + (1.75) (62.20) = 110.60 kNm

Total service truck and dead moment = (1.00) (1.40) + (1.30) (62.20) = 82.26 kNm

For tandem load (see Fig. 4.19)

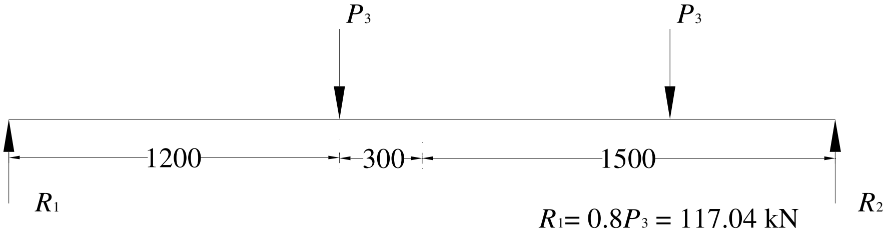


Figure 4.19 Tandem load on exterior stringer

Maximum tandem load moment = (117.04) (1.2) (0.43) = 60.39 kNm

Total limit tandem and dead moment = (1.25) (1.40) + (1.75) (60.39) = 107.44 kNm

Total service tandem and dead moment = (1.00) (1.40) + (1.30) (60.39) = 79.91 kNm

For lane load (see Fig. 4.20)

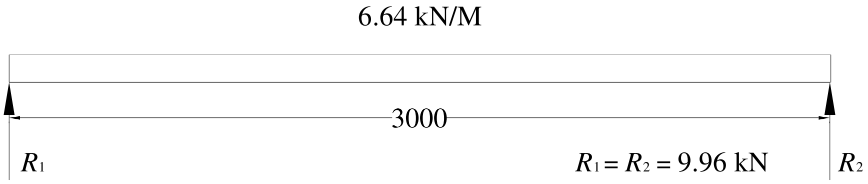


Figure 4.20 Lane load on exterior stringer

Maximum lane load moment = (6.64) (3) (3) / 8 = 7.47 kNm

A summary of the combination of the truck or tandem moment with lane and dead moment is shown in Table 4.2.

Table 4.2 Combination of truck or tandem with lane and dead moment for exterior stringer

No	Loading Type	Combined moment (in kNm)	Remark
1	Truck load and dead load with lane load (service)	82.26 + (7.47) (1.75) = 95.33	
2	Truck load and dead load with lane load (limit)	110.60 + (7.47) (1.75) = 123.67	Maximum
3	Tandem load and dead load with lane load (service)	79.91 + (7.47) (1.75) = 92.98	
4	Tandem load and dead load with lane load (limit)	107.44 + (7.47) (1.75) = 120.51	

Therefore the design moment is

MD = 123.67 kNm

The required section modulus for the above moment is

$S = 123.67 / (227{,}000) = (5.4480176)(10^{-4})\ m^3 = 544.80 \times 10^3\ mm^3$

The required section modulus allows us to use a lattice 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 lattice stringer as the interior one.

Check stringer members capacity by analyzing the stringer as a truss

The simply supported stringer is checked for maximum member forces by applying the truck load, the dead load and lane load at the joints on the top chord.

Maximum dead load on a joint = (1.25) (1.12) ((1.5/2) + (1.5/2)) = 2.10 kN.

Maximum truck and lane load = (1.75) (192.85+ (6.64) ((1.5/2) + (1.5/2)) = 354.92kN.

These loads are analyzed using SAP2000 version 8 and the maximum member forces are taken for design (refer to attached CD). From the analysis the maximum forces on chord are:

183.38 kN tension and

174.81 kN compression

diagonal are

195.51 kN tension and

205.09 kN compression

vertical are

92.08 kN tension and

124.79 kN compression

Design of stringer members

These are modeled as axially loaded compression or tension members. The axial capacities of the members are determined first.

Member capacities as compression members

This is when the stringer members are subjected to compressive forces applied through the centriodal axis of a member. A uniform compression force is developed at each cross section. The strength of compression members is limited by instability. This instability can be either local buckling or overall buckling.

Local buckling

The cross sections of structural steel elements are classified as plastic, compact, semi-compact or thin walled sections depending on the width-thickness ratios of their elements as specified in EBCS 3, 1995 Table 4.1.

Compression resistance of cross section

According to EBCS 3, 1995 Clause 4.5.4.1, for members in axial compression, the design value of the compressive force $N_{com,Sd}$ at each cross section shall satisfy:

$$N_{com,Sd} \leq N_{com,Rd}$$

where,

$N_{com,Rd}$ is the design compression resistance of the cross section, and may be determined as follows:

Class 1, 2, or 3 cross sections: $N_{com,Rd} = A f_y / \gamma_{mo}$

Class 4 cross sections: $N_{com,Rd} = A_{eff} f_y / \gamma_{m1}$

where,

for class 1, 2, or 3 cross sections: $\gamma_{mo} = 1.1$

Classification of section used in the design (see Fig. 4.21)

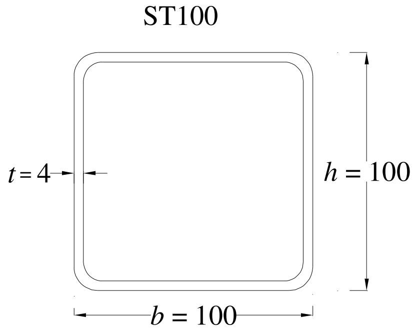


Figure 4.21 Cross-section of ST100 in mm

Check flange

$\varepsilon = \sqrt{(235/f_y)}$, f_y in MPa

$\varepsilon = \sqrt{(235/250)} = 0.9695$

For internal element of compression flange

$C / t_f \leq 23\,\varepsilon$

where $C = h - 3t_f$

$t_f =$ thickness of flange

$h =$ overall depth

$(100-(3)(4)) / 4 \leq (23)(0.9695)$

$22 \leq 22.30 \longrightarrow$ flange is plastic

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\,\varepsilon$

where $d = b - 3t_w$

$t_w =$ thickness of web.

$b =$ overall width

$(100-(3)(4)) / 4 \leq (28)(0.9695)$

$22 \leq 27.15 \longrightarrow$ web is plastic.

The whole section is plastic. Therefore, cross section used in the design is plastic. The design compression resistance is:

$N_{com,Rd} = A\, f_y / \gamma_{mo}$

$N_{com,Rd} = (A)(250) / 1.1 = 227.27\, A.$

For ST100, $N_{com,Rd} = (1495)(227.27) = 339.77$ kN

Overall buckling

The most significant parameter affecting truss member stability is the slenderness ratio *KL/r*, where *L* is the actual unbraced length of the truss member; *KL* is the effective length of the truss member; and *r* is the radius of gyration of the truss member cross section. *KL/r* is referred to as λ, *KL* as *L* and *r* as *i* in EBCS-3, 1995. According to AASHTO article 10.7.1 and 10.7.5, the slenderness ratio shall not exceed 120 for compression members and 140 for tension members subjected to stress reversal. Truss member strength equations are normally written for ideal "pin ended" truss member. To make the strength equations applicable to all truss members, an effective length factor *K* is used to account for the influence of end conditions on truss member stability. The design buckling resistance of a compression member shall be taken as:

$N_{b,Rd} = \chi \beta_A A f_y / \gamma_{m1}$

where $\beta_A = 1$ for class 1, 2 or 3 cross sections.

$\gamma_{m1} = 1.1$

χ is the reduction factor for the relevant buckling mode.

According to EBCS 3-1995 Table 4.11, the buckling curve about any axis for cold-formed hollow section is curve *b*. Use $\bar{\lambda}$ to determine the value of χ from EBCS3-1995 table 4.9 and then the buckling load $N_{b,Rd}$,

$\bar{\lambda} = (\lambda / \lambda_1)\,[\beta_A]^{0.5}$

where, $\bar{\lambda}$ is the non dimensional slenderness ratio

$\lambda = L / i$

where *i* is the radius of gyration about the relevant axis.

L is the buckling length of a compression member.

$\lambda_1 = \pi [E / f_y]^{0.5} = 93.9\varepsilon$

$\varepsilon = [235 / f_y]^{0.5}$ (f_y in MPa)

$\varepsilon = [235 / 250]^{0.5} = 0.9695$

$\lambda_1 = 91.04$

The stringer members are assumed as pin ended, so their buckling length is the same as their member length. The longest member length from the verticals is 750 mm diagonals is 1677 mm and from the chord members is 1500 mm. Buckling in the direction with the lower radius of gyration governs.

For ST100 diagonal, $i_x = i_y = 3.89$ cm , $L= 167.70$ cm , $\lambda = 43.11$

$\bar{\lambda} = (43.11 / 91.04) [1]^{0.5} = 0.47$

$\chi = 0.8968$, $N_{b,Rd} = (0.8968)\ (1)\ (1495)\ (250) / 1.1$

$N_{b,Rd} = 304.71$ kN

For ST100 vertical, $i_x = i_y = 3.89$ cm , $L = 75.00$ cm , $\lambda = 19.28$

$\bar{\lambda} = (19.28 / 91.04) [1]^{0.5} = 0.21$

$\chi = 0.9964$, $N_{b,Rd} = (0.9964)\ (1)\ (1495)\ (250) / 1.1$

$N_{b,Rd} = 338.55$ kN

For ST100 chord, $i_x = i_y = 3.89$ cm , $L= 150.00$ cm , $\lambda = 38.56$

$\bar{\lambda} = (38.56 / 91.04) [1]^{0.5} = 0.42$

$\chi = 0.9177$, $N_{b,Rd} = (0.9177)\ (1)\ (1495)\ (250) / 1.1$

$N_{b,Rd} = 311.81$ kN

Member capacities as tension members

When the truss members are subjected to tensile forces applied through the centriodal axis of a member, the result is a uniform tensile stress at each cross section. The design tensile strength of a structural steel depends on the appropriate cross sectional area. The axially loaded tensile members are checked for resistance of cross sections. EBCS 3-1995 clause 4.4.1 specifies that for members in axial tension the design value of the axial tension force $N_{t,sd}$ at each cross section shall satisfy:

$$N_{t,sd} \leq N_{t,Rd}$$

where $N_{t,Rd}$ is the design tension resistance capacity of the cross section, taken as the smaller of:

(a) the design plastic resistance of the gross section

$$N_{pl,Rd} = A\, f_y / \gamma_{m1}$$

(b) the design ultimate resistance of cross section at the bolt hole

$$N_{u,Rd} = 0.9 A_{eff} f_y / \gamma_{m2}$$

The stringer members do not have holes on them and because of this checking the design plastic resistance is sufficient.

For ST100,

A = 1495 mm^{-2}

$N_{pl,Rd}$ = (1495) (250) / 1.1

$N_{t,Rd}$ = $N_{pl,Rd}$ = 339.77 kN

The stringer members are sufficient.

4.2.1.3 Design of floor beams

The floor beam used is a lattice beam made from 100mm SHS as shown in Fig. 4.22.

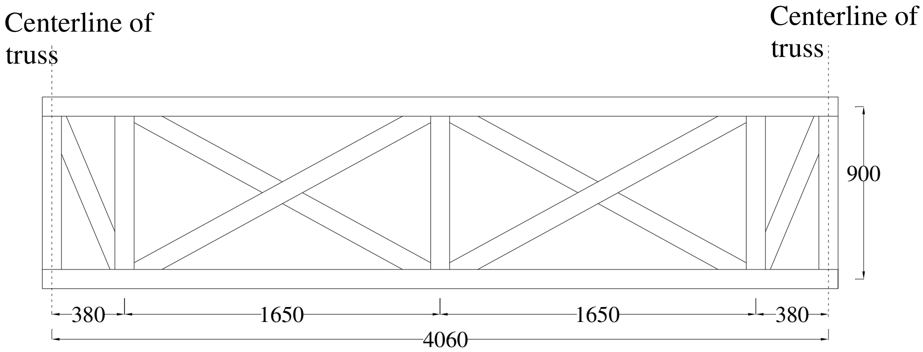


Figure 4.22 Lattice floor beam made from 4mm thick ST100 section

Dead loads

Span of floor beam = 4.06m

The center-to-center distance of floor beam is 3.0m.

Assume built up SHS lattice beam from ST100 with f_y = 250 MPa and mass 11.73 kg / m

Total length of floor beam members = 22.10 m

Weight of floor beam members = (11.73) (9.81) (22.10) = 2524.14 N/m

Weight per meter of floor beam = 2524.14 / 4.06 = 626.14 N/m

Floor beam weight = 0.63 kN/m

Dead load reaction from interior stringer = (1.69) (3.0) = 5.08 kN

Dead load reaction from exterior stringers = (1.25) (3.0) = 3.74 kN

The dead load reactions from stringers are applied as concentrated loads to floor beam as shown in Fig. 4.23.

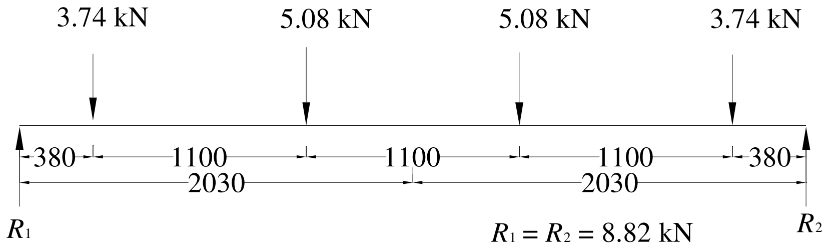


Figure 4.23 Concentrated dead loads on floor beam

Maximum dead load moment = (8.82) (2.03) - (3.74) (1.65) - (5.08) (0.55) + (0.63) (4.06) (4.06) / 8 = 10.24 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. As the floor beam is placed transversely on the trusses only one axle can be placed on the floor beam at a time as shown in Fig. 4.24.

For truck load

The truckload is placed so as to cause maximum live load moment (see Fig. 4.24 and Fig. 4.25). The maximum load occurs when the heavier axle load is on the floor beam as shown in Fig. 4.25.

Maximum truck load moment = (103.55) (2.78) – (96.43) (1.8))= 114.30 kNm

Total limit truck and dead moment = (1.25) (10.24) + (1.75) (114.30) = 212.83 kNm

Total service truck and dead moment = (1.00) (10.24) + (1.30) (114.30) = 158.83 kNm

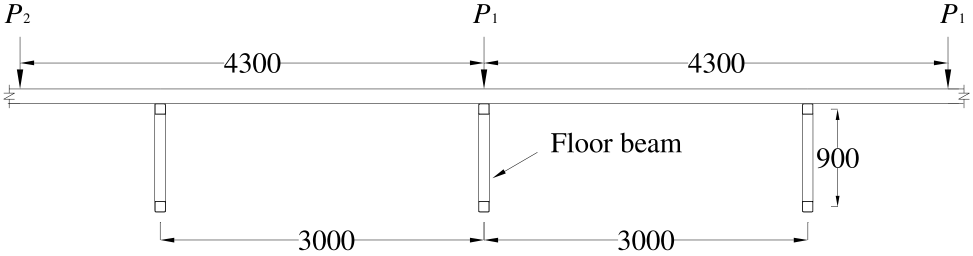


Figure 4.24 Truck moving to the left on the bridge

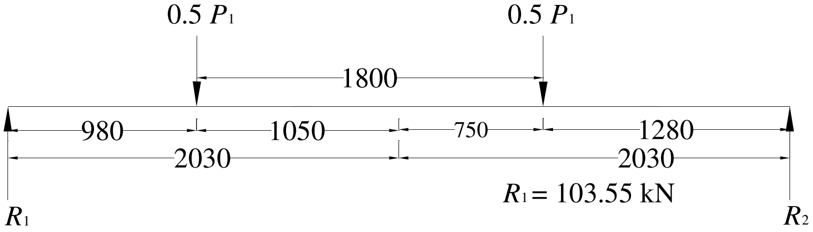


Figure 4.25 The heavier axle load on the floor beam

For tandem load

For maximum force effect, the first axle is placed on the floor beam and the same floor beam also carries a portion of the second axle as shown in Fig. 4.26. The concentrated loads applied to floor beams from tandem are (see Fig. 4.27):

$$P_3 + (1.8/3)\, P_3 = 1.6\, P_3$$

Maximum tandem load moment = (125.69) (2.78) – (117.04) (1.8) = 138.75 kNm

Total limit tandem and dead moment = (1.25) (10.24) + (1.75) (138.75) = 255.61 kNm

Total service tandem and dead moment = (1.00) (10.24) + (1.30) (138.75) = 190.61 kNm

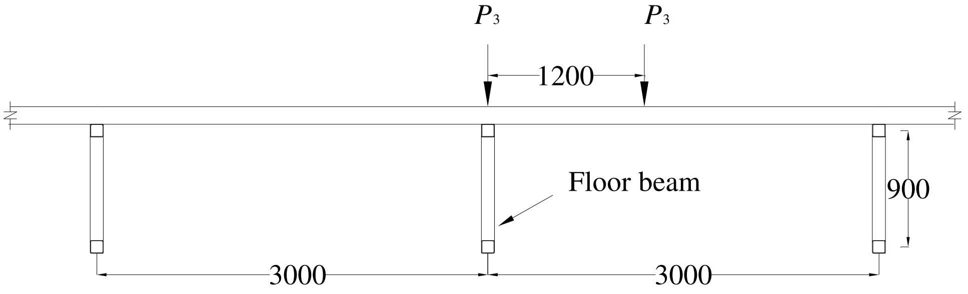


Figure 4.26 Tandem moving to the left on the bridge span

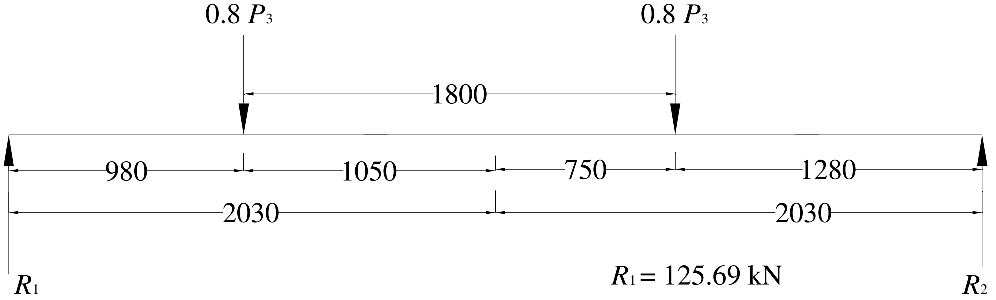


Figure 4.27 Tandem load on the floor beam

For lane load (see Fig 4.28)

Lane load in transverse direction 9.3 kN / 3m = 3.1 kN/m

Maximum lane load moment = (4.65) (2.03) -(3.10) (1.5) (1.5) / 2 = 5.95 kNm

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

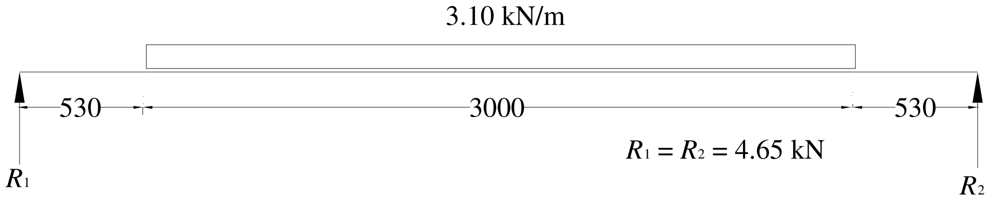


Figure 4.28 Lane load on floor beam

Table 4.3 Combination of truck or tandem with lane and dead moment for floor beam

No	Loading Type	Combined moment (in kNm)	Remark
1	Truck load and dead load with lane load (service)	158.83 + (5.95) (1.75) = 169.24	
2	Truck load and dead load with lane load (limit)	212.83 + (5.95) (1.75) = 223.24	
3	Tandem load and dead load with lane load (service)	190.61 + (5.95) (1.75) = 201.02	
4	Tandem load and dead load with lane load (limit)	255.61 + (5.95) (1.75) = 266.03	Maximum

Therefore the design moment is

$MD = 266.03$ kNm

The required section modulus for the above moment is

$S = 266.03 / (227{,}000) = 0.0011719163 \text{ m}^3 = 1.17 \times 10^6 \text{ mm}^3$

Check section modulus of floor beam shown in Fig. 4.29.

SHS 100 x 100 x 4 mm

$A = 1495 \text{ mm}^2$ $\quad$ $I = 2{,}264{,}000 \text{ mm}^4$

$d = 900/2 = 450$mm

$I_{xx} = (2)\ (I_o + A\ d^2) = (2)\ (2264000 + (1495)\ (450)\ (450)) = 610.00 \times 10^6 \text{ mm}^4$

$C_y = 500$ mm

$S_{xx} = I_{xx} / C_y = 1.22 \times 10^6 \text{ mm}^3 > 1.17 \times 10^6 \text{ mm}^3$

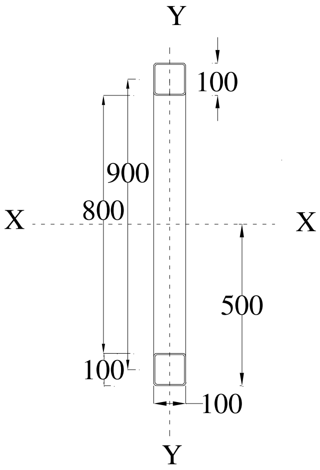


Figure 4.29 Cross section of floor beam in mm

The section is sufficient.

Check floor beam chord and diagonals capacity by analyzing the floor beam as a truss

The simply supported floor beam is checked for maximum member forces by applying the dead load (see Fig. 4.30), the tandem load (see Fig. 4.31) and lane load (see Fig 4.32) at the joints on the top chord. As the stringers are not placed at the floor beam joints, their loads are transferred to the two joints near the applied load in proportion to the distance from the joints. The same is done for the live load. Using factors of 1.25 for dead load and 1.75 for lane and tandem load these loads are factored.

These loads are analyzed using SAP2000 version 8 software and the maximum member forces are taken for design (refer attached CD). From the analysis the maximum forces on

diagonal are

262.03 kN tension and

126.85 kN compression

chord are

208.77 kN tension and

176.57 kN compression

vertical are

0.00 kN tension and

241.73 kN compression

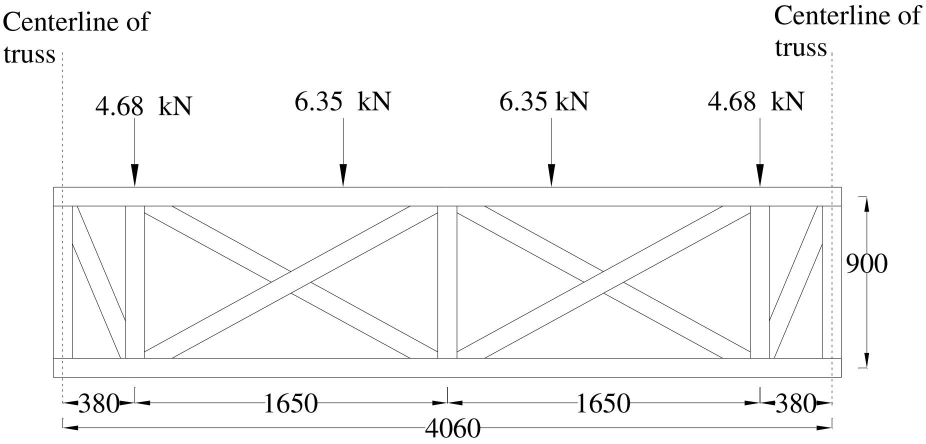


Figure 4.30 Factored dead load on floor beam

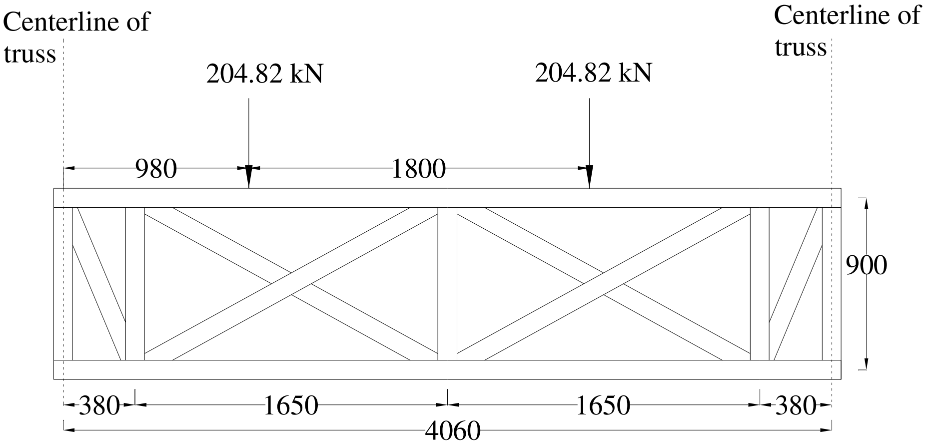


Figure 4.31 Factored tandem load on floor beam at closest position to left truss

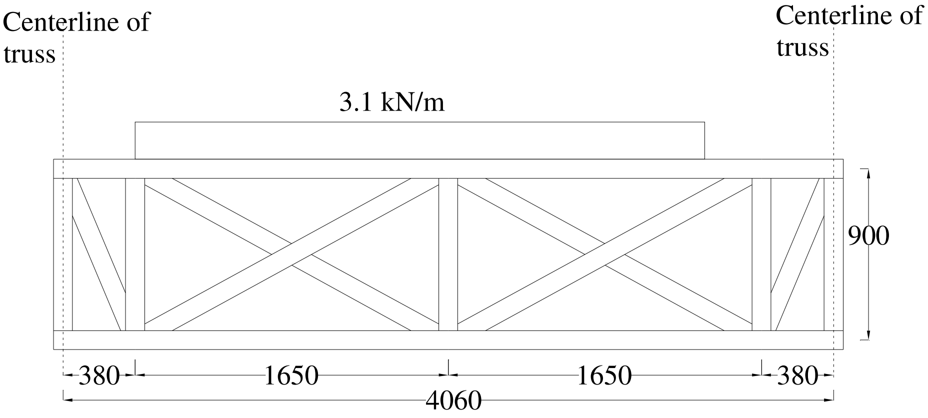


Figure 4.32 Lane load at closest position to left truss

Design of floor beam members

These are also modeled as axially loaded compression or tension members. The axial capacities of the members are determined first.

Member capacities as compression members

Check local buckling

Classification of section used in the design (See Fig. 4.33)

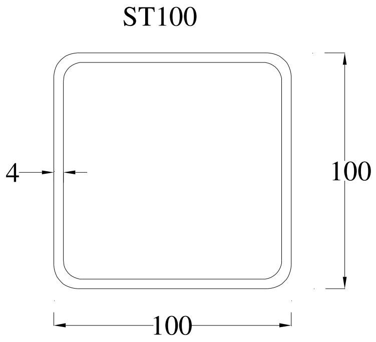


Figure 4.33 Cross-section of ST100 in mm

Check flange

$\varepsilon = 0.9695$

For internal element of compression flange

$C / t_f \leq 23\ \varepsilon$

$(100-(3)(4)) / 4 \leq (23)(0.9695)$

$22 \leq 22.30 \longrightarrow$ flange is plastic

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

$(100-(3)(4)) / 4 \leq (28)(0.9695)$

$22 \leq 27.15 \longrightarrow$ web is plastic.

The whole section is plastic.

The cross section used in the design is plastic. Therefore the design compression resistance is:

$$N_{com,Rd} = A\, f_y / \gamma_{mo}$$

$$N_{com,Rd} = (A)(250) / 1.1 = 227.27\, A.$$

$$N_{com,Rd} = (1495)(227.27) = 339.77 \text{ kN}$$

Check overall buckling

The design buckling resistance of a compression member shall be taken as:

$N_{b,Rd} = \chi \beta_A A f_y / \gamma_{m1}$

$\beta_A = 1$

$\bar{\lambda} = (\lambda / \lambda_1)\ [\beta_A]^{0.5}$

$\lambda_1 = 93.9\varepsilon$

$\varepsilon = 0.9695$

$\lambda_1 = 91.04$

The floor beam members are pin ended, so their buckling length is the same as their member length. The longest member length from the verticals is 900 mm, from the diagonals is 1880 mm and from the chord members is 1650 mm.

For diagonal, $i_x = i_y = 3.89$ cm , $L = 188$ cm , $\lambda = 48.33$

$\bar{\lambda} = (48.33 / 91.04)\ [1]^{0.5} = 0.53$

$\chi = 0.8701$,

$N_{b,Rd} = (0.8701)\ (1)\ (1495)\ (250) / 1.1 = 295.64$ kN

For chord, $i_x = i_y = 3.89$ cm , $L = 165.00$ cm , $\lambda = 42.42$

$\bar{\lambda} = (42.42 / 91.04)\ [1]^{0.5} = 0.47$

$\chi = 0.8968$,

$N_{b,Rd} = (0.8968)\ (1)\ (1495)\ (250) / 1.1 = 304.71$ kN

For vertical, $i_x = i_y = 3.89$ cm , $L = 90.00$ cm , $\lambda = 23.14$

$\bar{\lambda} = (23.14 / 91.04)\ [1]^{0.5} = 0.25$

$\chi = 0.9821$,

$N_{b,Rd} = (0.9821)\ (1)\ (1495)\ (250) / 1.1 = 333.69$ kN

Member capacities as tension members

The floor beam has no holes on it. Because of this the design resistance is taken as the design plastic resistance.

The design plastic resistance of the gross section

$N_{pl,Rd} = A f_y / \gamma_{m1}$

$A = 1495\ \text{mm}^{-2}$

$N_{pl,Rd} = (1495)\ (250) / 1.1 = 339.77$ kN

The floor beam chord and diagonal members are sufficient.

4.2.1.4 Design of truss

Dead loads - super imposed loads

Total dead load transferred to truss at panel points = 8.82 + (0.63) (4.06) / 2 = 10.10 kN.

The design dead load at panel point (see Fig. 4.34) = (1.25) (10.10) = 12.62 kN

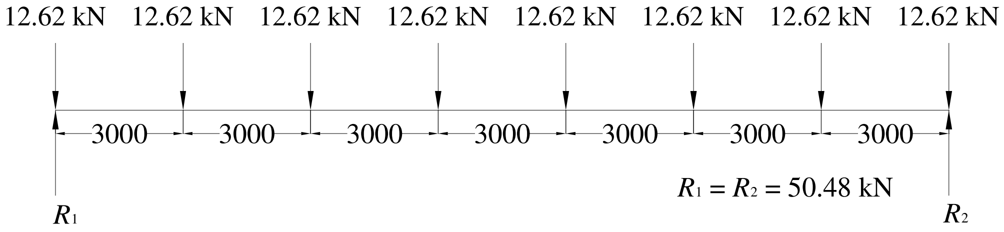


Figure 4.34 Concentrated dead loads on truss at panel points

Maximum moment = (50.48) (10.5) - (12.62) (10.5) - (12.62) (7.5) - (12.62) (4.5) - (12.62) (1.5) = 227.16 kNm

Dead loads - self weight

Assume built up hollow truss from ST100 and RT128 with f_y = 250 MPa and mass 11.73 kg / m

Panel length = (1.5) (4) (2) + (2.3) (3) + (4) √ ((1.5) (1.5) + (2.3) (2.3)) = 23.88 m

Total length of steel truss = (7) (29.88) = 209.16 m

Weight of steel truss = (209.16) (9.81) (11.73) = 24.07 kN/m

Span of steel truss = 21m

Steel truss weight = 24.07 / 21 = 1.15 kN/m

Dead load- self weight moment = (1.15) (21) (21) / 8 = 63.39 kNm

Total dead load moment = 227.16 + 63.39 = 290.55 kNm.

Live Loads

Place the uniform lane loadings as close as possible to one of the trusses to cause maximum loads on that truss as shown in Fig. 4.35.

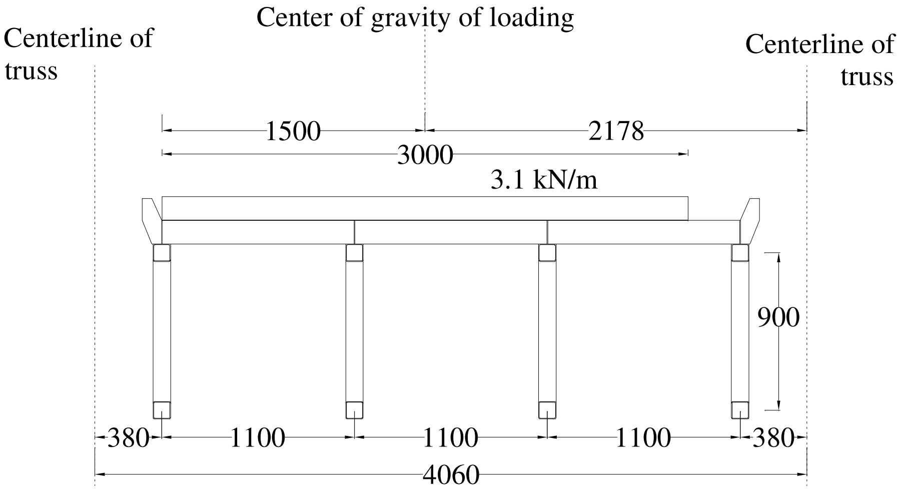


Figure 4.35 Lane load position for maximum load on left truss

From the figure part of the live load going to the left truss is determined to be 2.178/4.06 or 54 percent.

Uniform lane loading going to one truss = (3.1) (3) (0.54) = 5.02 kN/m

The lane loading transferred to a panel point = (5.02) (3) = 15.06 kN

Maximum lane load moment on truss = (5.02) (21) (21) / 8 = 276.73 kNm

Similarly determine the maximum percentage of concentrated truck loads going to the left truss from Fig. 4.36 and Fig. 4.37.

For truck loads (see Fig. 4.36 for rear axle and Fig. 4.37 for front axle)

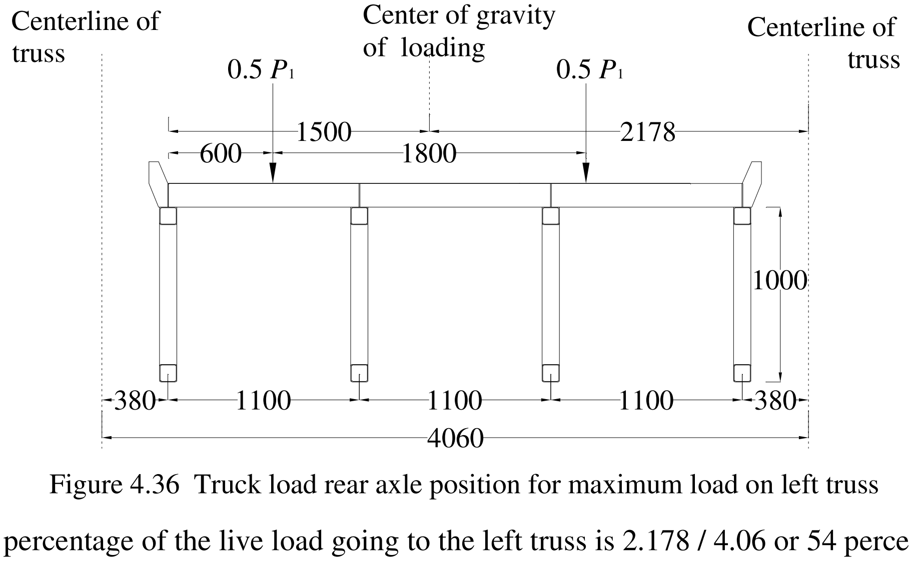


Figure 4.36 Truck load rear axle position for maximum load on left truss

The percentage of the live load going to the left truss is 2.178 / 4.06 or 54 percent.

Concentrated truck loads going to one truss = (192.85) (0.54) = 104.14 kN

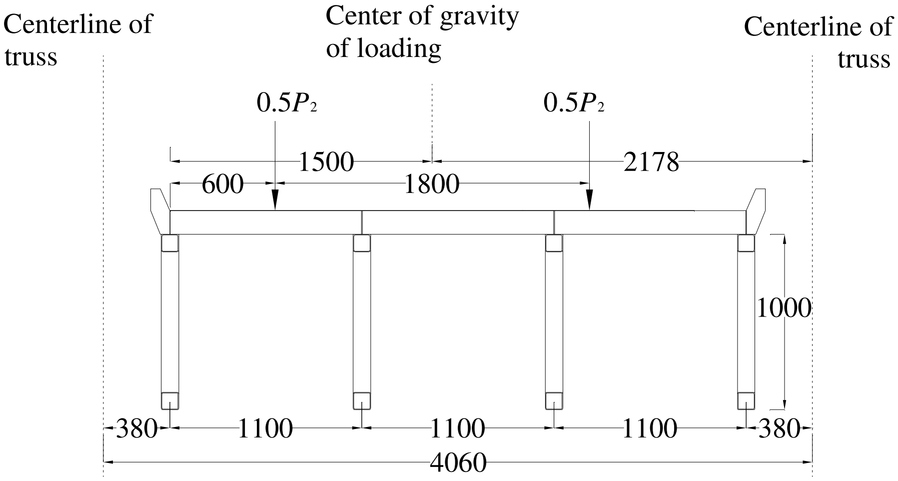


Figure 4.37 Truck load front axle position for maximum load on left truss

The percentage of the live load going to the left truss is 2.178/ 4.06 or 54 percent. Concentrated truck loads going to one truss = (46.55) (0.54) = 25.14 kN.

The maximum moment for truck load is under the first heavy axle when the mid-span is half way between the resultant and this axle as shown in Fig. 4.38.

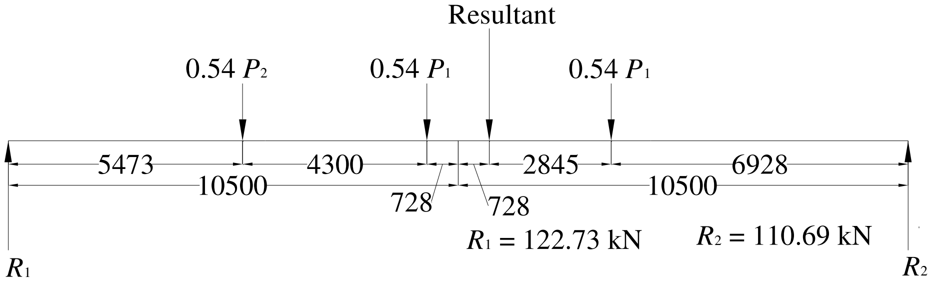


Figure 4.38 Truck load position on truss for maximum moment

Maximum truck load moment on truss = (122.73) (10.50 - 0.728) - (0.54) (46.55) (4.30) = 1091.29 kNm

Total limit truck and dead moment = (1.25) (290.55) + (1.75) (1091.29) = 2272.95 kNm

Total service truck and dead moment = (1.00) (290.55) + (1.3) (1091.29) = 1709.23 kNm

For tandem loads (see Fig. 4.39)

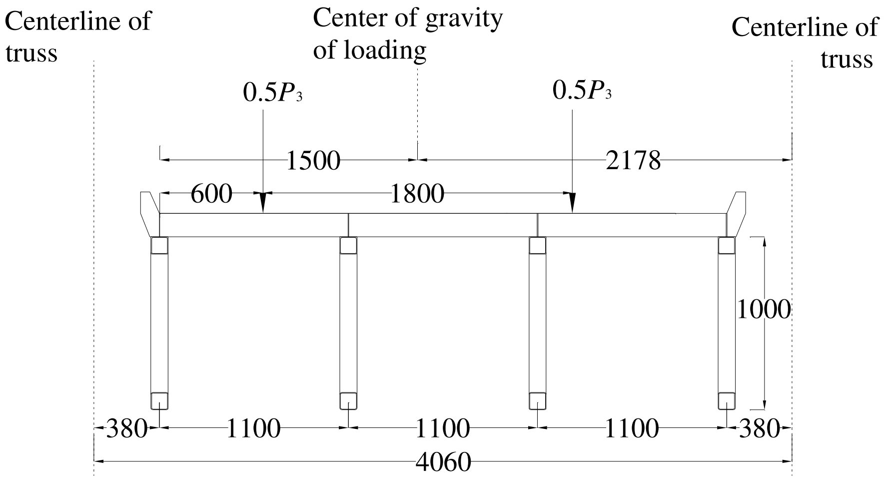


Figure 4.39 Tandem load position for maximum load on left truss

The percentage is 2.178 / 4.06 or 54 percent.

Concentrated tandem loads going to one truss = (146.30) (0.54) = 79.00 kN

The maximum moment for tandem load is under the first axle when the mid-span is half way between the resultant and this axle as shown in Fig. 4.40.

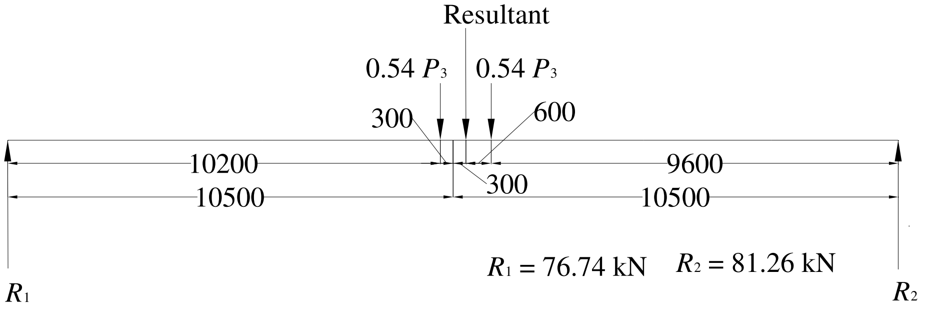


Figure 4.40 Tandem load position on truss for maximum moment

Maximum tandem load moment on truss = (76.74) (10.50 - 0.30) = 782.75 kNm

Total limit tandem and dead moment = (1.25) (290.55) + (1.75) (782.75) = 1733.00 kNm

Total service tandem and dead moment = (1) (290.55) + (1.30) (782.75) = 1308.13 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 for truss

No	Loading Type	Combined moment (in kNm)	Remark
1	Truck load and dead load with lane load (service)	1709.23 + (276.73) (1.75) = 2193.51	
2	Truck load and dead load with lane load (limit)	2272.95 + (276.73) (1.75) = 2757.23	Maximum
3	Tandem load and dead load with lane load (service)	1308.13 + (276.73) (1.75) = 1792.41	
4	Tandem load and dead load with lane load (limit)	1733.00 + (276.73) (1.75) = 2217.28	

Therefore the design moment is

$$MD = 2757.23 \text{ kNm}$$

The required section modulus for the above moment is

$S = M / f_b = 2757.23 / (227{,}000) = 0.012146387 \text{ m}^3 = 12.15 \times 10^6 \text{ mm}^3$

Check section modulus of truss shown in Fig. 4.41:

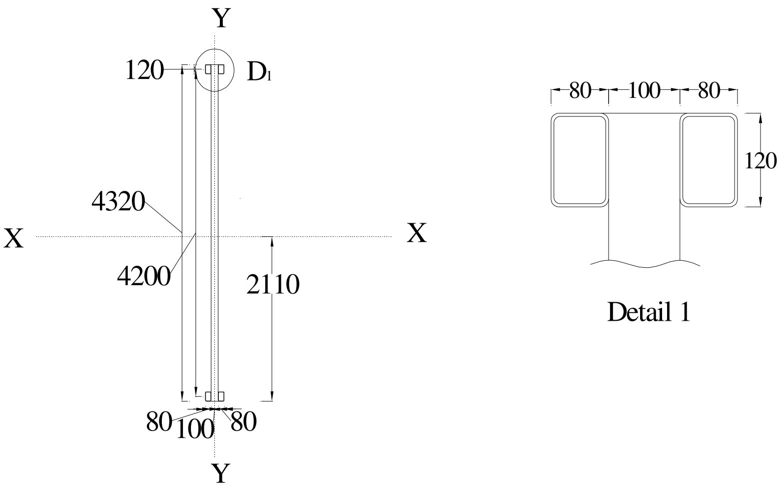


Figure 4.41 Cross section of truss in mm

RHS 120 x 80 x 4 mm

$A = 1495 \text{ mm}^2$ $\qquad I_x = 2{,}945{,}900 \text{ mm}^4$

$d = 4200 / 2 = 2100\text{mm}$

$I_{xx} = (4)\ (I_o + A\ d^2) = (4)\ (2945900 + (1495)\ (2100)\ (2100)) = 26{,}383.58 \text{ x } 10^6 \text{ mm}^4$

$C_y = 2160 \text{ mm}$

$S_{xx} = I_{xx} / C_y = 12.21 \text{ x } 10^6 \text{ mm}^3 > 12.15 \text{ x } 10^6 \text{ mm}^3$

The section is sufficient but too long. Consider using double truss with each truss carrying half the load. Therefore take half the required section modulus and check section capacity (see Fig. 4.42).

$S = 12.15. \text{ x } 10^6 \text{ mm}^3 / 2 = 6.07 \text{ x } 10^6 \text{ mm}^3$

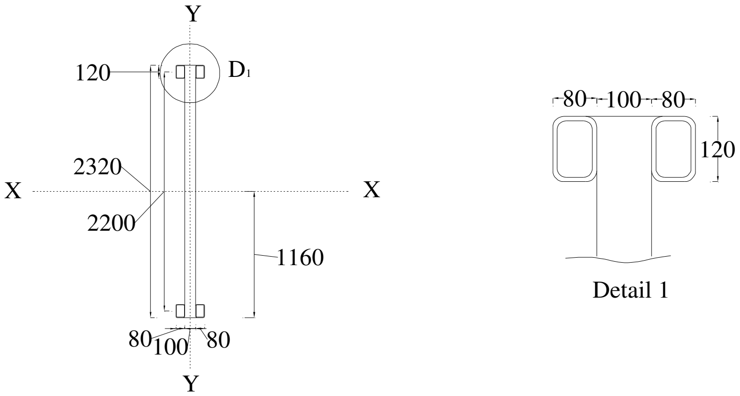


Figure 4.42 Cross section of truss in mm

RHS 120 x 80 x 4 mm

$A = 1495 \text{ mm}^2$ $\qquad I_x = 2{,}945{,}900 \text{ mm}^4$

$d = 2200 / 2 = 1100\text{mm}$

$I_{xx} = (4)\ (I_o + A\ d^2) = (4)\ (2945900 + (1495)\ (1100)\ (1100)) = 7{,}247.58 \text{ x } 10^6 \text{ mm}^4$

$C_y = 1160 \text{ mm}$

$S_{xx} = I_{xx} / C_y = 6.25 \text{ x } 10^6 \text{ mm}^3 > 6.07 \text{ x } 10^6 \text{ mm}^3$

The section is sufficient.

The bridge truss is modeled for analysis using SAP 2000 version 8 software. Four sections namely RT128, RT106, ST100 and ST80 are chosen and checked for carrying capacity and the two with better capacity are used as members of the panel. The top and bottom chords are modeled using RT128 and the vertical and diagonal members using ST100. The design dead load (see Fig. 4.34) is applied to all panel points, which are located on the bottom chord at the end of each panel. The design live load i.e. half the truck load shown in Fig. 4.38 is applied at few panel points at a time. Half the truck load (see Fig. 4.43) is taken as double truss each carrying half the load is assumed. The analysis is done repeatedly in three groups which apply the first, second and third axle load at panel points respectively and distribute the effects of the other axles by considering their distance from panel points near them. The maximum member forces for design of a typical panel are found by varying the position of live load at panel points for each group for maximum effect. The maximum member forces for the chord are when the rear (3rd) axle is placed on the third panel point from the left (refer to attached CD).

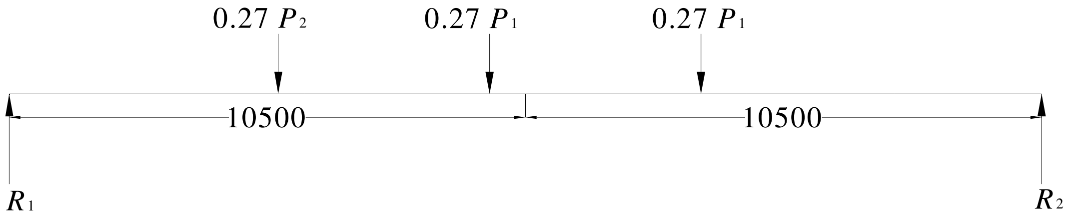


Figure 4.43 Half the truck load to be applied for single truss

From the analysis, the maximum member forces found are

- for chord members
    - 487.55 kN compression for RT128
    - 483.01 kN for tension for RT128
- for vertical members
    - 77.37 kN compression for ST100
    - 26.02 kN for tension for ST100
- for diagonal members
    - 136.79 kN compression for ST100
    - 123.43 kN for tension for ST100

Design of panel members

Types of members

There are three types of members in a typical panel; the chord (top or bottom), the vertical and the diagonal. These are axially loaded compression or tension members. The axial capacities of the members are determined first.

Member capacities as compression members

Check local buckling

Classification of sections used in the design (see Fig. 4.44, Fig. 4.45, Fig. 4.46, and Fig. 4.47).

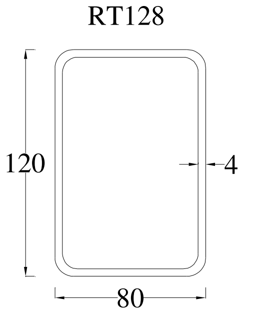


Figure 4.44 Cross section of RT128

Check flange

$\varepsilon = 0.9695$

For internal element of compression flange

$C / t_f \leq 23\ \varepsilon$

$(80-(3)\ (4))\ /\ 4 \leq (23)\ (0.9695)$

$17 \leq 22.30 \longrightarrow$ flange is plastic

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

$(120-(3)\ (4))\ /\ 4 \leq (28)\ (0.9695)$

$27 \leq 27.15 \longrightarrow$ web is plastic.

The whole section is plastic

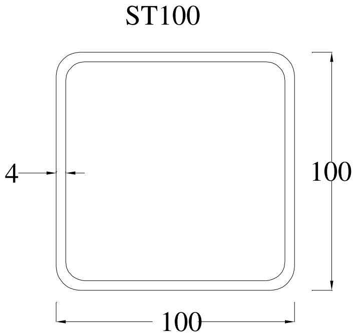


Figure 4.45 Cross-section of ST100

Check flange

$\varepsilon = 0.9695$

For internal element of compression flange

$C / t_f \leq 23\ \varepsilon$

$(100-(3)\ (4))\ /\ 4 \leq (23)\ (0.9695)$

$22 \leq 22.30 \xrightarrow{\quad}$ flange is plastic

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

$(100-(3)\ (4))\ /\ 4 \leq (28)\ (0.9695)$

$22 \leq 27.15 \xrightarrow{\quad}$ web is plastic.

The whole section is plastic.

RT106

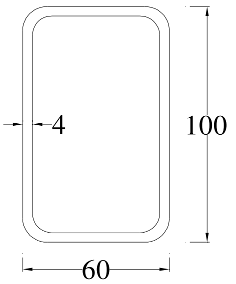


Figure 4.46 Cross-section of RT106

Check flange

$\varepsilon = 0.9695$

For internal element of compression flange

$C / t_f \leq 23\ \varepsilon$

$(60-(3)\ (4))\ /\ 4 \leq (23)\ (0.9695)$

$12 \leq 22.30 \xrightarrow{\quad}$ flange is plastic

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

$(100-(3)\ (4))\ /\ 4 \leq (28)\ (0.9695)$

$22 \leq 27.15 \xrightarrow{\quad}$ web is plastic.

The whole section is plastic.

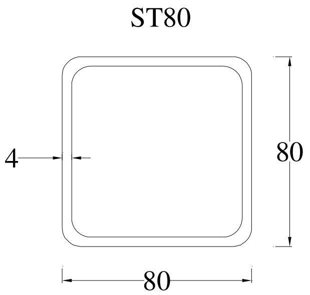


Figure 4.47 Cross-section of ST80

Check flange

$\varepsilon = 0.9695$

For internal element of compression flange

$C / t_f \leq 23\ \varepsilon$

$(80-(3)(4)) / 4 \leq (23)(0.9695)$

$17 \leq 22.30$ ⟶ flange is plastic

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

$(80 - (3)(4)) / 4 \leq (28)(0.9695)$

$17 \leq 27.15$ ⟶ web is plastic.

The whole section is plastic

The cross sections to be used in the design are plastic. Therefore the design compression resistance is:

$$N_{com,Rd} = A\, f_y / \gamma_{mo}$$

$$N_{com,Rd} = (A)(250) / 1.1 = 227.27\, A.$$

For RT128, $N_{com,Rd} = (1495)(227.27) = 339.77$ kN

For ST100, $N_{com,Rd} = (1495)(227.27) = 339.77$ kN

For RT106, $N_{com,Rd} = (1175)(227.27) = 267.04$ kN

For ST80, $N_{com,Rd} = (1175)(227.27) = 267.04$ kN

The top and bottom chord members are made from two RT128 members. Because of this the capacity of the chord is = (2) (339.77) = 679.54 kN.

Of these four cross-sections the ones with better resistance, that is RT128 and ST100 are chosen to form the members of the truss. RT128 is used for top and bottom chords while ST100 is used for the verticals and diagonals.

Check overall buckling

The design buckling resistance of a compression member is

$N_{b,Rd} = \chi \beta_A A f_y / \gamma_{m1}$

$\beta_A = 1$ for class 1, 2 or 3 cross sections.

$\bar{\lambda} = (\lambda / \lambda_1) [\beta_A]^{0.5}$

$\lambda_1 = 93.9\varepsilon$

$\varepsilon = 0.9695$

$\lambda_1 = 91.04$

The truss members are pin ended, so their buckling length is the same as their member length. The longest member length from the verticals is 2200mm, from the diagonals is 2663 mm and from the chord members is 1500 mm. Buckling in the direction with the lower radius of gyration governs.

For RT128 chord, $i_x = 4.44$ cm, $i_y = 3.24$, $L = 150.00$ cm , $\lambda = 46.30$

$\bar{\lambda} = (46.30 / 91.04) [1]^{0.5} = 0.51$

$\chi = 0.8795$, $N_{b,Rd} = (0.8795)\ (1)\ (1495)\ (250) / 1.1$

$N_{b,Rd} = 298.83$ kN

For ST100 diagonal, $i_x = i_y = 3.89$ cm , $L = 266.3$ cm , $\lambda = 68.46$

$\bar{\lambda} = (68.46 / 91.04) [1]^{0.5} = 0.75$

$\chi = 0.7541$, $N_{b,Rd} = (0.7541)\ (1)\ (1495)\ (250) / 1.1$

$N_{b,Rd} = 256.22$ kN

For ST100 vertical, $i_x = i_y = 3.89$ cm , $L = 220$ cm , $\lambda = 56.56$

$\bar{\lambda} = (56.56 / 91.04) [1]^{0.5} = 0.62$

$\chi = 0.8264$, $N_{b,Rd} = (0.8264)\ (1)\ (1495)\ (250) / 1.1$

$N_{b,Rd} = 280.79$ kN

The top and bottom chord members are made from two RT128 members. Because of this the capacity of the chord is = (2) (298.83) = 597.66 kN.

Member capacities as tension members

The panel has holes on the pin plates for connection with other panels. But the members of panel do not have holes on them and because of this checking the design tensile plastic resistance is sufficient.

$N_{pl,Rd} = A\, f_y \,/\, \gamma_{m1}$

For RT128,

$N_{pl,Rd} = A\, f_y \,/\, \gamma_{m1}$

$A = 1495 \text{ mm}^{-2}$

$N_{pl,Rd} = (1495)\ (250) / 1.1$

$N_{pl,Rd} = 339.77$ kN

For ST100,

$N_{pl,Rd} = A\, f_y \,/\, \gamma_{m1}$

$A = 1495 \text{ mm}^{-2}$

$N_{pl,Rd} = (1495)\ (250) / 1.1$

$N_{pl,Rd} = 339.77$ kN

Using similar reasoning as given above for chord member compression capacity, the tensile capacity of the chord is = (2) (339.77) = 679.54 kN. The sections used are sufficient.

4.2.2 Diamond Shaped Truss

To search for more efficient panel truss, a second trial panel with diagonals of diamond shape (see Fig. 4.48) and made from hollow sections with yield strength of f_y = 400 MPa chosen is checked for two types of decking. The first one uses steel imported decks (like the ones ERA uses but made to fit the bridge deck size) supported directly on floor beams spaced 1.5 m apart. The deck is not designed here and it's load is assumed to be equal to the steel decks currently used by ERA. The second type uses checkered plate for flooring supported on a system of steel stringers, spaced 550 mm apart, and floor beams with a spacing of 750 mm. Here the checkered plate is designed for truck and dead loadings. The first and second options are discussed below.

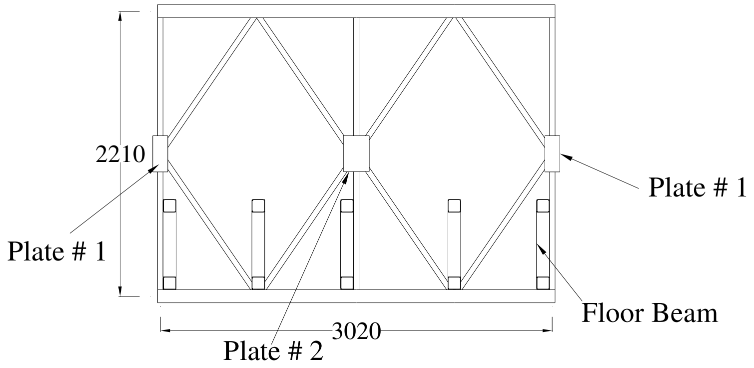


Figure 4.48 The second trial panel for the second option

4.2.2.1 The first option

For the first option floor beam at every 1.5 m (three per panel) made from hollow sections with yield strength of f_y = 400 MPa is used (see Fig. 4.49). This floor beam supports the deck directly.

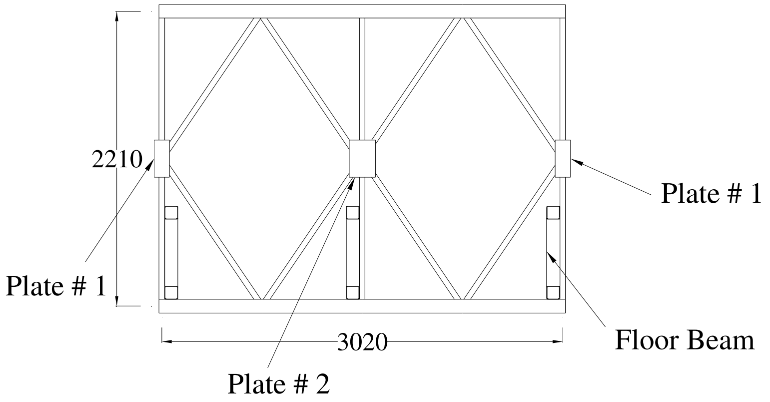


Figure 4.49 The second trial panel for the first option

4.2.2.1.1 Design Data

The following assumptions are used.

- The 21m span bridge is made up of seven panels each 3m long.
- The road way width is 3300mm.
- The center to center distance of truss is 4060mm.
- The center-to-center distance of floor beams is 1500mm.
- A steel deck and steel curb at edge currently used by ERA is assumed designed and only its load is considered here.
- The deck is directly supported on the floor beam.

- Structural hollow sections of Kality Metal Products Factory with yield strength of 400MPa are used to make the trusses, and floor beams.
- A preliminary design of floor beams will be done.
- For live loads ERA live loading of HL-93 are used.

4.2.2.1.2 Design of floor beams

The floor beam used is a lattice beam made from 4mm thick ST100mm as shown in Fig. 4.50.

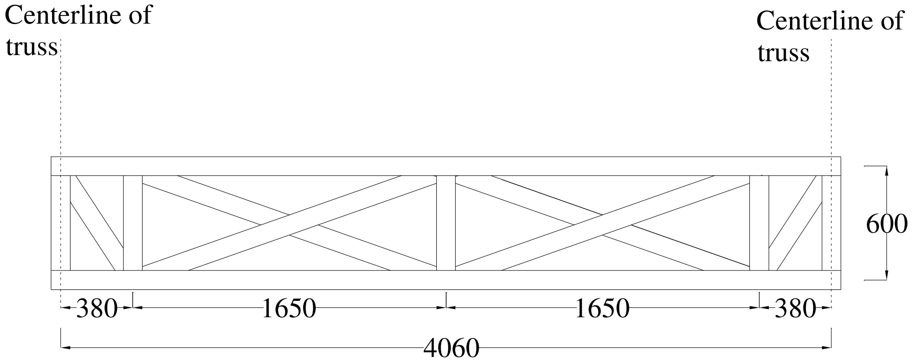


Figure 4.50 Lattice floor beam made from 4mm thick ST100 section

Dead loads

The center-to-center distance of floor beam is 1.5m.

Span of floor beam = 4.06m

Steel deck weight = (1018.37) (1.5) = 1527.56 N/m

Assume built up SHS lattice girder from 4mm ST100 with f_y = 400 MPa and mass 11.73 kg / m

Total length of floor beam members = 19.40m

Weight of floor beam members = (11.73) (9.81) (19.40) = 2232.38 N/m

Weight per meter of floor beam = 2232.38 / 4.06 = 547.15 N/m

Floor beam weight = 0.55 kN/m

Total dead load = 1527.56 + 547.15 = 2074.71 N/m

Dead load moment = (2074.71) (4.06) (4.06) / 8 = 4274.83 Nm = 4.27 kNm

Live Loads

Fraction of wheel loads carried by the interior floor beam as specified by AASHTO clause 3.23.2.2 is = S /4.5 = ((1.5/0.3048)/4.5) = 1.09

For truck load

The truck load is placed so as to cause maximum live load moment (see Fig. 4.51 and Fig. 4.52). The maximum load occurs when the heavier axle load is on the floor beam as shown in Fig. 4.52.

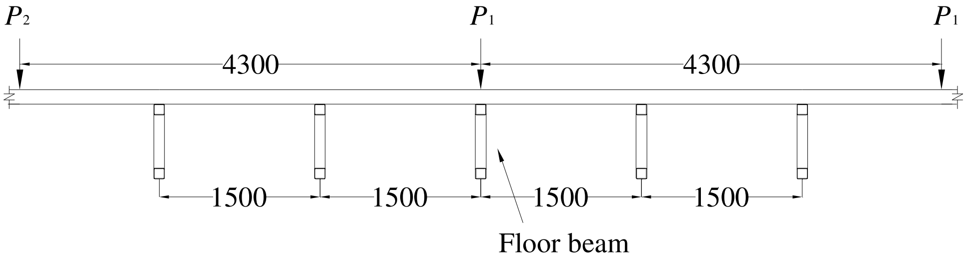


Figure 4.51 Truck moving to the left on the bridge

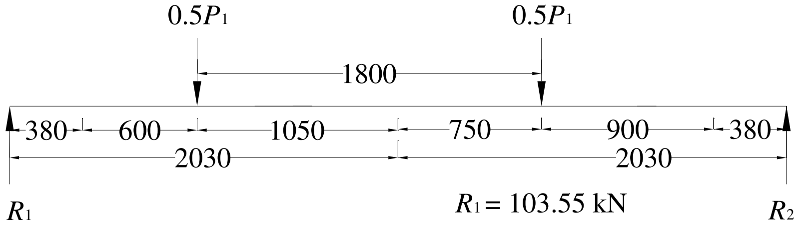


Figure 4.52 The heavier axle load on the floor beam

Maximum truck load moment = ((103.55) (2.78) – (96.43) (1.8)) (1.09) = 124.59 kNm

Total limit truck and dead moment = (1.25) (4.27) + (1.75) (124.59) = 223.37 kNm

Total service truck and dead moment = (1.00) (4.27) + (1.30) (124.59) = 166.24 kNm

For tandem load

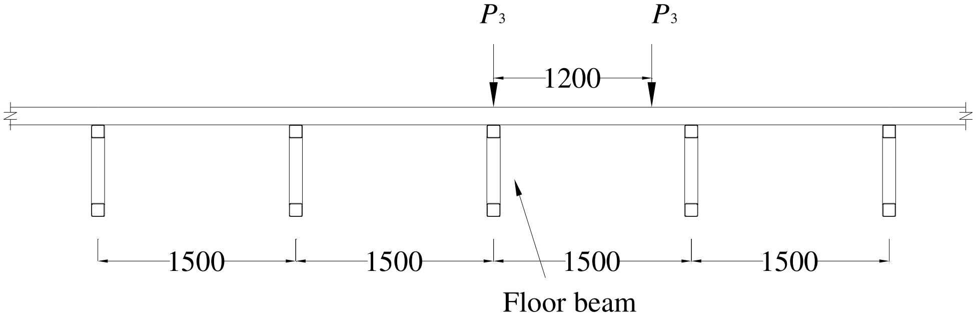


Figure 4.53 Tandem moving to the left on the bridge span

For maximum force effect, the first axle is placed on the floor beam and the same floor beam also carries a portion of the second axle as shown in Fig. 4.53. The concentrated loads applied to floor beams from tandem are (see Fig. 4.54):

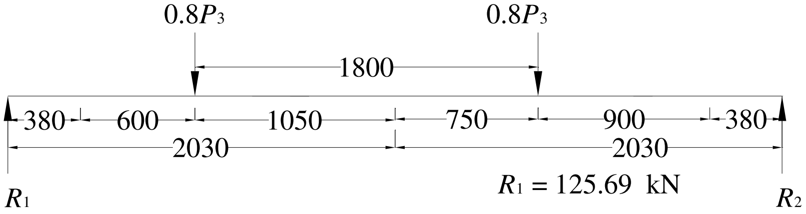


Figure 4.54 Tandem load on the floor beam

Maximum tandem load moment = ((125.69) (2.78) – (117.04) (1.8)) (1.09) = 151.23 kNm

Total limit tandem and dead moment = (1.25) (4.27) + (1.75) (151.23) = 269.99 kNm

Total service tandem and dead moment = (1.00) (4.27) + (1.30) (151.23) = 200.87 kNm

For lane load (see Fig. 4.55)

Lane load in transverse direction 9.3 kN/ 3m = 3.1 kN/m

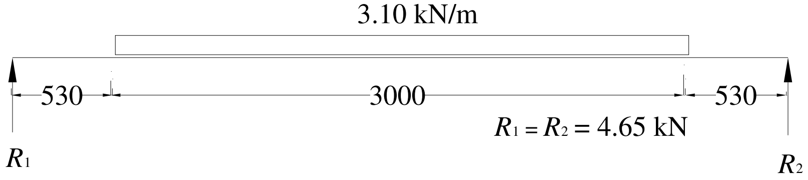


Figure 4.55 Lane load on floor beam

Maximum lane load moment = (4.65) (2.03) -(3.10) (1.5) (1.5) / 2 = 5.95 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 or tandem with lane and dead moment for floor beam

No	Loading Type	Combined moment (in kNm)	Remark
1	Truck load and dead load with lane load (service)	166.24 + (5.95) (1.75) = 176.65	
2	Truck load and dead load with lane load (limit)	223.37 + (5.95) (1.75) = 233.78	
3	Tandem load and dead load with lane load (service)	200.87 + (5.95) (1.75) = 211.28	
4	Tandem load and dead load with lane load (limit)	269.99 + (5.95) (1.75) = 280.40	Maximum

Therefore the design moment is

$$MD = 280.40 \text{ kNm}$$

The required section modulus for the above moment is

$$S = 280.40 / (363{,}000) = 0.000772458677 \text{ m}^3 = 772.46 \times 10^3 \text{ mm}^3$$

Check section modulus of floor beam shown in Fig. 4.56.

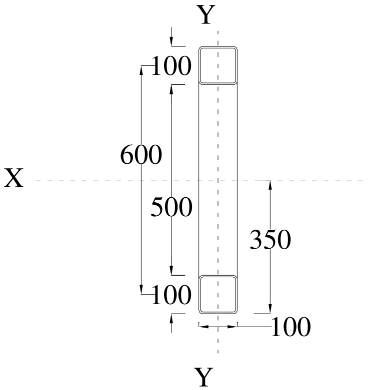


Figure 4.56 Cross section of floor beam in mm

SHS 100 x 100 x 4 mm

$A = 1495$ mm^2 $I = 2{,}264{,}000$ mm^4

$d = 600/2 = 300$ mm

$I_{xx} = (2)(I_o + A\,d^2) = (2)\ (2264000 + (1495)\ (300)\ (300)) = 273.628 \times 10^6$ mm ^{4}

$C_y = 350$ mm

$S_{xx} = I_{xx} / C_y = 781.79 \times 10^3$ mm$^3 > 772.46 \times 10^3$ mm ^{3}

The section is sufficient.

Check floor beam members' capacity by analyzing the floor beam as a truss

The simply supported floor beam is checked for maximum member forces by applying the dead load (see Fig. 4.57), the tandem load (see Fig. 4.58) and lane load (see Fig 60) at the joints on the top chord. The deck dead load, live and lane load are not placed on the joints and so their effects are applied to the joints near them in proportion to their distance from the joints. These loads are factored using factors of 1.25 for dead load and 1.75 for lane and live load.

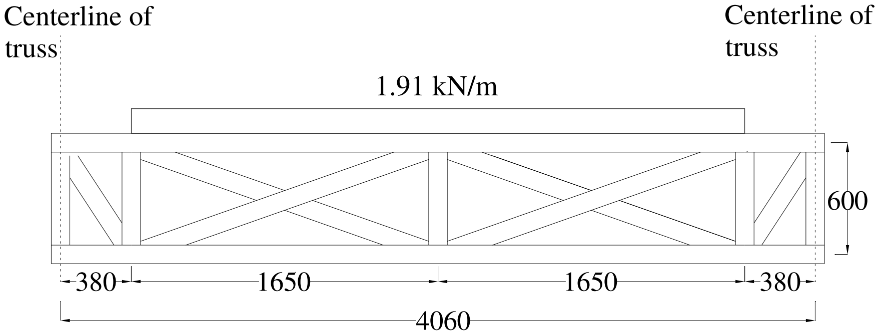


Figure 4.57 Factored dead load on floor beam

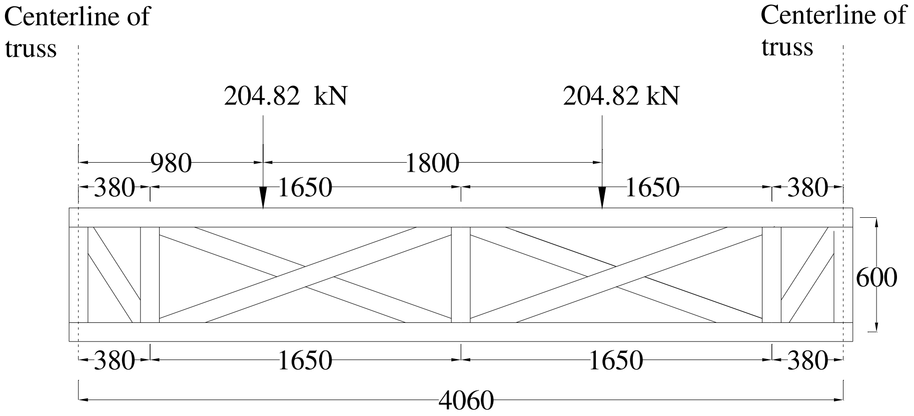


Figure 4.58 Factored tandem load with impact on floor beam at closest position to truss

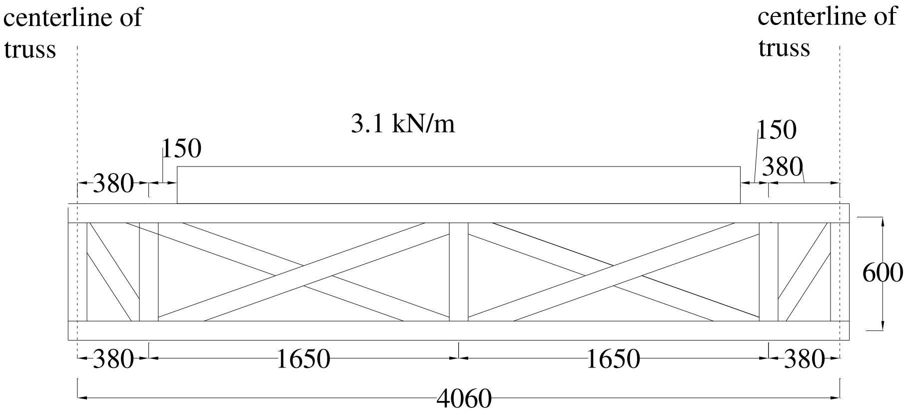


Figure 4.59 Lane load on floor beam

The tandem load in Fig. 4.58 is applied with the lane load in Fig. 4.59. These loads are analyzed together with the dead load in Fig. 4.57, using SAP2000 version 8 software and the maximum member forces are taken for design (refer to attached CD). From the analysis:

the maximum forces on diagonal are

279.57 kN tension and

161.15 kN compression

and the maximum forces on chord are

292.47 kN tension and

275.45 kN compression

and the maximum forces on verticals are

0 kN tension and

236.24 kN compression

Design of floor beam members

Member capacities as compression members

Check local buckling

Classification of section used in the design (see Fig. 4.60).

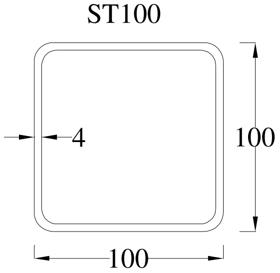


Figure 4.60 Cross-section of ST100

Check flange

$\varepsilon = \sqrt{(235/f_y)}$, f_y in MPa

$\varepsilon = \sqrt{(235/400)} = 0.7665$

For internal element of compression flange

$C / t_f \leq 28\ \varepsilon$

$(100 - (3)(4)) / 4 \leq (28)(0.7665)$

$22 > 21.46 \xrightarrow{\qquad}$ flange is class four.

$\bar{\lambda}_p = (C/t_f) / (28.4\ \varepsilon\sqrt{k_\sigma}) = 22/((28.4)(0.7665)(\sqrt{4})) = 0.51 < 0.673$

$\rho = 1$

$C_{eff} = \rho C = (1)(88) = 88$ mm

Total area that should be ignored from the two flanges is

$\Delta A_f = (2)(88-88)(4) = 0$

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

$(100 - (3)(4)) / 4 \leq (28)(0.7665)$

$22 \leq 21.46 \xrightarrow{\qquad}$ web is class four.

$\bar{\lambda}_p = (d/t_w) / (28.4\ \varepsilon\sqrt{k_\sigma}) = 22 / ((28.4)(0.7665)(\sqrt{4})) = 0.51 < 0.673$

$\rho = 1$

$C_{eff} = \rho C = (1)\ (88) = 88$ mm

Total area that should be ignored from the two webs is

$\Delta A_w = (2)\ (88\text{-}88)\ (4) = 0$

$A = A_{eff}$ i.e., the effective area is equal to total area.

The design compression resistance for ST100 is:

Class 4 cross section: $N_{com,Rd} = A_{eff}\ f_y\ /\ \gamma_{m1}$

For ST100, $N_{com,Rd} = (1495)\ (400)/1.1 = 543.64$ KN

Check overall buckling

The longest member length from the diagonals is 1756 mm, from the verticals 600 mm and from the chord members is 1650 mm.

For ST100 diagonal, $i_x = i_y = 3.89$ cm , $L = 175.6$ cm , $\lambda = 45.14$

$\lambda_1 = 71.97$

$\bar{\lambda} = (45.14\ /\ 71.97)\ [1]^{0.5} = 0.63$

$\chi = 0.8211$, $N_{b,Rd} = (0.8211)\ (1)\ (1495)\ (400)\ /\ 1.1$

$N_{b,Rd} = 446.38$ kN

For ST100 chord, $i_x = i_y = 3.89$ cm , $L = 165.00$ cm , $\lambda = 42.42$

$\bar{\lambda} = (42.42\ /\ 71.97)\ [1]^{0.5} = 0.59$

$\chi = 0.8418$, $N_{b,Rd} = (0.8418)\ (1)\ (1495)\ (400)\ /\ 1.1$

$N_{b,Rd} = 457.63$ kN

For ST100 vertical, $i_x = i_y = 3.89$ cm , $L = 60.00$ cm , $\lambda = 15.42$

$\bar{\lambda} = (15.42\ /\ 71.97)\ [1]^{0.5} = 0.21$

$\chi = 0.9964$, $N_{b,Rd} = (0.9964)\ (1)\ (1495)\ (400)\ /\ 1.1$

$N_{b,Rd} = 541.68$ kN

Member capacities as tension members

The floor beam members do not have holes on them and because of this checking the design plastic resistance is sufficient.

$N_{pl,Rd} = A\ f_y\ /\ \gamma_{m1}$

For 4 mm thick ST100,

$A = 1495\ \text{mm}^{.2}$

$N_{pl,Rd}$ = (1495) (400) / 1.1

$N_{t,Rd}$ = $N_{pl,Rd}$ = 543.64 KN

The floor beam members are sufficient.

4.2.2.2 The second option

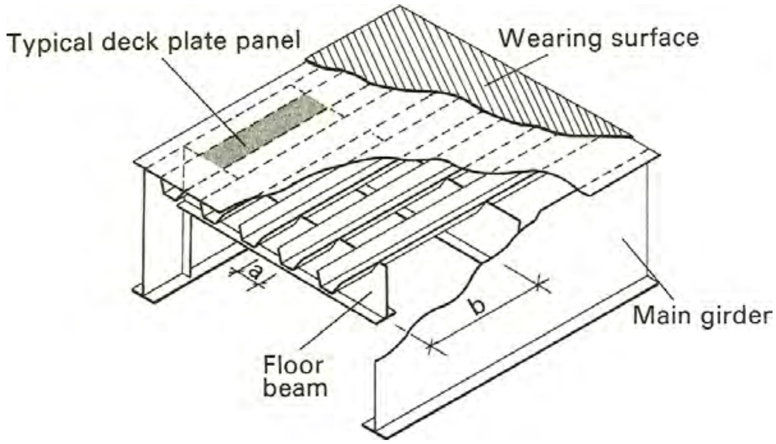
The second option for the deck of the steel bridge is to use checkered steel plate with a system of stringers and floor beams. The steel deck of a highway bridge is designed for wheel loads of truck using plate bending theory in two dimensions. Often in design practice, however, this plate theory is reduced to equivalent one dimensional beam theory [3].

Metal plate is one of the major components of many heavy structures such as ships, steel bridges, aircraft, sea platforms, etc. Most thin plate elements of plated structures undergo deflections ranging from about 0.1 to several times the plate thickness under working conditions. Due to this finite out of plate deflections *w*, the stiffness of the plate changes, resulting in greater resistance to loading not predicted by the classical plate theory. Such a plate is said to have gone large deflections, and design for these deformations is referred to as large-deflection design [7].

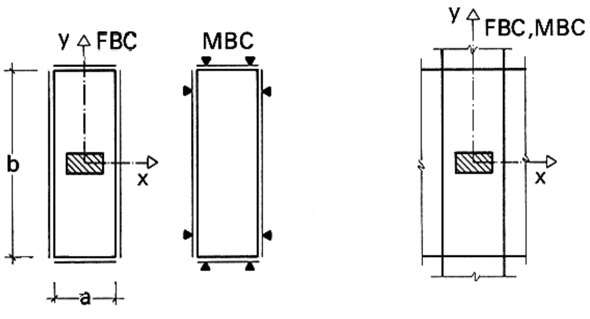
A large-deflection design or check for a panel of plating under transverse loading consists of the following steps:

1. Definition of panel geometry, boundary conditions and loading
2. Selection of design criteria and material.
3. Evaluation of plate thickness and/ or other selected design criteria.

A typical panel plate and idealizations is shown below in Fig. 4.61.



(i) Steel bridge deck showing typical plate panel



(ii) (iii)

(ii) and (iii) Possible boundary-condition idealizations of deck plate between the rib webs for the evaluation of local deck-plate stresses under wheel loading

Figure 4.61 Steel bridge decks and panel boundary-condition idealization

4.2.2.2.1 Definition of geometry, boundary conditions and loading

Aspect ratio of panel (ratio of length *b* to width *a*) = *b/a*

Flexural boundary conditions (FBC).

Membrane boundary conditions (MBC)

A combined flexural and membrane boundary conditions

Loading = Self weight and factored truck loading of HL-93 with impact.

4.2.2.2.2 Selection of design criteria

This can be in the control of out of plane deflections *w*, or the maximum equivalent surface stress $\sigma_{emax.}$ If it is desired to avoid permanent set in a plate panel, as a limit of serviceability, material yielding should not take place anywhere in the plate under working conditions. To use the yield criterion, first the biaxial state of stress in the element of plate under considerations is converted into an equivalent uniaxial state of stress σ_e, using the following relationship:

$$\sigma_e = (\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau^2)^{1/2}$$

The element of the plate is considered to yield when the when the magnitude of the equivalent stress thus obtained reaches the yield stress of the material in uniaxial tension, that is:

$$\sigma_e = \sigma_y = 400\text{MPa}$$

4.2.2.2.3 Evaluation of plate thickness t, and/ or other selected design criteria.

The non-dimensional parameters to be used in conjunction with the design curves given [7] are

1. Transverse loading

   For uniformly distributed loading q

   $Q = a^4 q / t^4 E$

   where Q is the non-dimensional parameter for uniformly distributed loading

2. Out of plane deflections

   Transverse deflection w is given by

   $W = w/t$

   where W is the non-dimensional parameter for transverse deflection

3. Maximum equivalent stress

   The maximum equivalent stress anywhere in the plate is given by the non-dimensional parameter $\overline{\sigma}_{e\ max}$, which is related to the actual maximum equivalent stress by the expression

   $\overline{\sigma}_{e\ max} = (a/t)^2\ \sigma_{e\ max} / E$

4.2.2.2.4 Design procedure

For a given value of plate thickness t, it is required to check the deflection and stresses resulting under the action of transverse loading. Using the given t, calculate the non-dimensional value of the transverse loading P or Q. Refer to design curves, and for the value of P or Q calculated read off the corresponding values of the non-dimensional stress and deflection coefficients ($\overline{\sigma}_{e\ max}, W$). Calculate the actual stresses.

4.2.2.2.5 Design of the deck plate

The deck is to be made up of 16mm thick checkered steel plate supported on stringers spaced 550mm apart which are in turn supported on floor beams with 750mm centerline spacing. To evaluate the central deflection and maximum equivalent stress, a steel panel of the deck with the dimensions as shown in Fig. 4.62 and boundary conditions as shown in Fig. 63, under a central truck wheel load is considered. The truck loading is distributed uniformly over the patch area (u) (v),

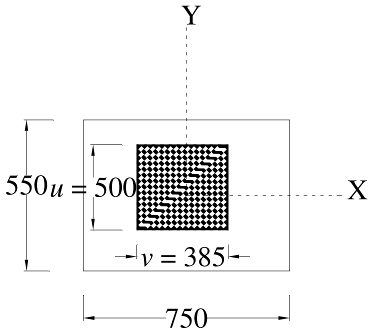


Figure 4.62 Panel under wheel loading distributed over patch area

Dimensions:

a = 550 mm

b = 750 mm

t = 16 mm

Material:

High tensile steel, E = 210 GPa,

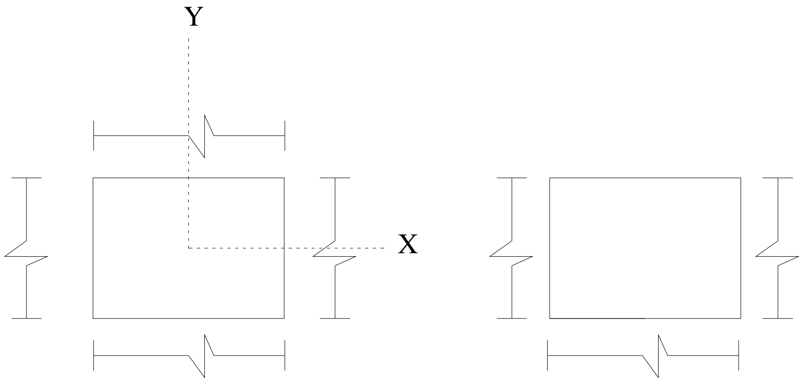
Poisson's ratio = 0.3

Yield stress = 400 MPa

Boundary conditions (see Fig. 4.63):

Two boundary conditions are necessary to represent the deck panels: one for the interior panels and one for edge panels.

Combined flexural and membrane boundary conditions: The plate panel is continuous over all edges in case (a) and on three sides in case (b).



(a) (b)

Figure 4.63 Idealizations of panel as continuous on three and four sides

Loading:

Truck Loading p = Factored truck loading of HL-93 with impact. The concentrated loading is assumed uniformly distributed over the patch area (u) (v) shown in Fig. 4.62. This load is assumed as distributed over the whole panel.

$p = (1.75)\ (1.33)\ (72.5) = 168.74$ kN.

Tire contact area according to ERA specifications clause 3.8.6 is

width = 500 mm

length = $(2.28)\ (10^{-3})\ (\gamma)\ (1+ IM/100)\ (P)$

where:

γ = load factor for the limit state under consideration

IM = dynamic load allowance

P = 72.5 kN for the design truck

length = $(2.28)\ (10^{-3})\ (1.75)\ (1+ 33/100)\ (72500) = 385$ mm

$u = 385$ mm

$v = 500$ mm

$P = p/uv = 168744 / ((500)\ (385)) = 0.876592$ N/mm^2

Lane loading: 9.3 kN/m^2 uniformly distributed over 3m width including a live load factor of 1.75.

$q = (3.1)\ (10^{-3})\ (1.75) = 5.43\ (10^{-3})$ N/mm^2

Dead load: unit weight of steel multiplied by thickness including a dead load factor of 1.25.

$$q = (7850)\ (9.81)\ (0.016)\ (1.25) = 1540.17\ \mathrm{N/m^2} = (1.54)\ (10^{-3})\ \mathrm{N/mm^2}$$

Total distributed load $q = 0.88\mathrm{N/mm^2}$

$$Q = a^4q/t^4E\ = (550)^4\ (0.88)\ \ /\ ((16^4)\ (210{,}000)) = 5.85$$

Aspect ratio of panel = b/a = 750 / 550 = 1.36

The related design curves are to be found in [7].

For maximum equivalent stress

	Case (a)	Case (b)
For $b/a = 1.0$		
	$\bar{\sigma}_{e\,max} = 1.88$	$\bar{\sigma}_{e\,max} = 1.90$
For $b/a = 2.0$		
	$\bar{\sigma}_{e\,max} = 2.19$	$\bar{\sigma}_{e\,max} = 2.30$
For $b/a = 1.36$ using linear interpolation		
	$\bar{\sigma}_{e\,max} = 1.99$	$\bar{\sigma}_{e\,max} = 2.04$

For central deflection

	Case (a)	Case (b)
For $b/a = 1.0$		
	$W = 0.275$	$W = 0.16$
For $b/a = 1.5$		
	$W = 0.313$	$W = 0.25$
For $b/a = 1.36$ using linear interpolation		
	$W = 0.289$	$W = 0.192$

Actual deflection and stress are:

$$\sigma_{e\,max} = E\,(t\,/a)^{2}\ \bar{\sigma}_{e\,max}$$

$$w = Wt$$

For case (a)

$\sigma_{e\ max} = (16/550)\ (16/550)\ (210000)\ (1.99) = 353.66\ \text{N/mm}^2$

$\sigma_{e\ max} < f_y\ /\ 1.1 = 363\ \text{MPa}$

$w = (0.289)\ (16) = 4.62\ \text{mm}$

For case (b)

$\sigma_{e\ max} = (16/550)\ (16/550)\ (210000)\ (2.04) = 362.55\ \text{N/mm}^2$

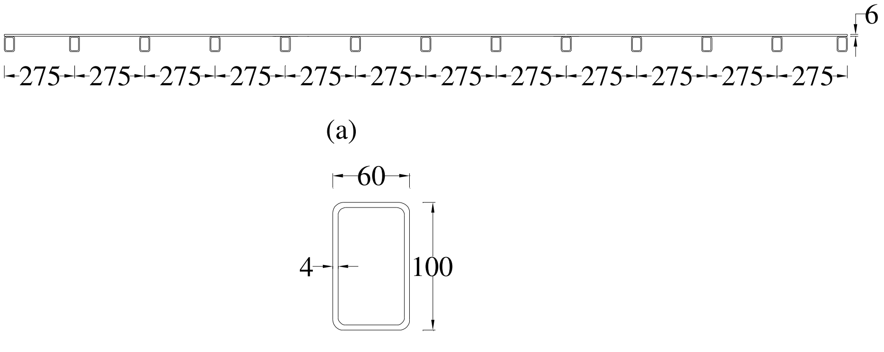
$\sigma_{e\ max} < f_y\ /\ 1.1 = 363\ \text{MPa}$

$w = (0.192)\ (16) = 3.08\ \text{mm}$

The thickness used is too big for deck, but lowering it would result in higher equivalent stress. Therefore, stiffen the deck using RT106 hollow sections and use 6 mm thick-checkered plate as shown in Fig. 4.64.

4.2.2.2.6 Stiffened Plate Deck

For steel bridges that utilize stiffened steel plate as deck, AASHTO article 10.41.1.2 specifies that an appropriate method of analysis such as the equivalent-othotropic-slab method or equivalent-grid method shall be used in designing the deck. And also the equivalent stiffness properties shall be selected to correctly simulate the actual deck.



(a)

(b)

Figure 4.64 Stiffened deck (a) cross-section;(b) dimension of RT106 beam

In modeling of the stiffened deck, a combined model is used which represents the checkered plate with plate elements and the RT106 beams with longitudinal grillage members (see Fig. 4.65).

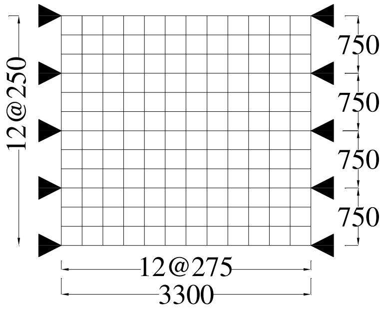


Figure 4.65 Combined finite element and grillage mesh

The checkered plate is modeled using isotropic elements which are assigned a thickness equal to the depth of the actual plate. They are also assigned the elastic properties of the plate. The longitudinal grillage members are then assigned the stiffness of the combined beam and associated portion of checkered plate minus those already provided through the plate elements (see Fig. 4.66).

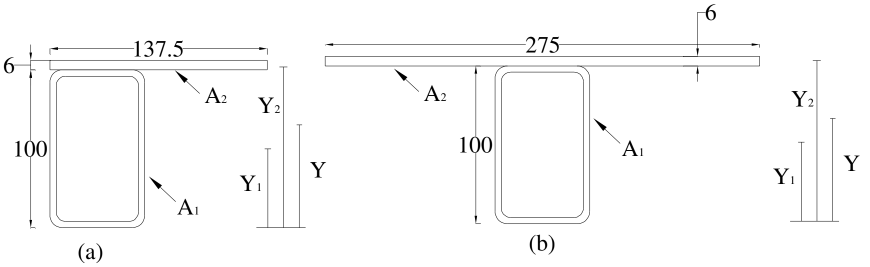


Figure 4.66 Combined RT106 beam and plate above it (a) edge (b) interior.

Location of neutral axis

(a) (b)

$Y = (A_1Y_1 + A_2Y_2) / (A_1+A_2)$

$Y = 71.86$ mm $Y = 80.96$ mm

Equivalent stiffness

$I_{comb} = I_{x1} + A_1d_1^2 + I_{x2} + A_2d_2^2 - I_{x2}$

$I_{comb} = 2,887,287.20\ \text{mm}^4$ $I_{comb} = 3,453,569.52\ \text{mm}^4$

According to AASHTO article 10.41.2 wheel load shall be uniformly distributed to the deck plate. A uniform load of factored live and dead load 0.88MPa is loaded on the combined model and analyzed using SAP2000 version8. From the analysis the following results are obtained.

Maximum stress on plate element

$$\sigma_{max} \leq f_y / 1.1 = 363 \text{ MPa}$$

$$\sigma_x = 12.64 \text{ MPa} < 363 \text{ MPa}$$

$$\sigma_y = 26.18 \text{ MPa} < 363 \text{ MPa}$$

$$\sigma_{xy} = 1.55 \text{ MPa} < 363 \text{ MPa}$$

Maximum and minimum principal stress on plate element

$$\sigma_{max} = 26.30 \text{ MPa} < 363 \text{ MPa}$$

$$\sigma_{min} = -26.30 \text{ MPa} < 363 \text{ MPa}$$

Maximum moments on beam

$M_{max} = -11.83$ kNm

$M_{min} = 11.17$ kNm

$T_{max} = -5.86$ Nm

$T_{min} = 5.86$ Nm

$V_{max} = -76.11$ kN

$V_{min} = 76.11$ kN

Design of beam

RT106 Section properties

$h = 100$mm	$I = 152.58$ cm^4	$I_t = 156.27$ cm^4
$b = 60$mm	$A = 11.75$ cm^2	$C_t = 38.68$ cm^3
$t_f = 4$mm	$i = 3.60$ cm	$W_{pl} = 37.94$ cm^3
$t_w = 4$mm	$W_{el} = 30.52$ cm^3	

Section classification

Web, with neutral axis at mid depth

$d / t_f \leq 79\,\varepsilon$

$\varepsilon = \sqrt{(235/400)} = 0.7665$

$(100 - (3)(4)) / 4 \leq (79)(0.7665)$

$22 \leq 60.55$

The section is plastic. → Class 1 cross-section

Determine the required section modulus

$W_{pl} = M_{sd} / (f_y / \gamma_{mo})$

where, W_{pl} = the required section design plastic resistance

M_{sd} = the design bending moment

$W_{pl} = ((11.83)(10^6)) / (400 / 1.1) = 30250 \text{ mm}^3 = 30.25 \text{ cm}^3 < W_{ply} = 37.94 \text{ cm}^3$

Check for shear

$V_{max} = 76.11$ kN

Shear resistance of section

$V_{pl, Rd} = A_v (f_y / \sqrt{3}) / \gamma_{mo}$

where, $V_{pl, Rd}$ = the design plastic shear resistance

A_v = the shear area

$A_v = (Ah) / (b+h)$

$A_v = (1175)(100) / (60+100) = 734.38 \text{ mm}^2$

$V_{pl, Rd} = 734.38) (400/ \sqrt{3}) / 1.1 = 154.18 \text{ kN} > 76.11 \text{ kN}$

Check for shear buckling

$d/t_w < 69\varepsilon$

$d/t_w = 100/4 = 25 < 69\varepsilon = (69)\sqrt{(235/400)} = 52.89$

No need for shear buckling check.

Check for reduction of design resistance moment

$V_{Sd} / V_{pl, Rd} < 0.5$

$76.11 / 154.18 = 0.494 < 0.5$

Therefore, no reduction of design resistance moment is required.

Check for deflection

Maximum deflection $\Delta = 0.53$ mm

$\delta_{all} = L / 500$

$\delta_{all} = 750 / 500 = 1.5$mm

$\Delta < 1.5$ Ok!

4.2.2.2.7 Design of floor beams

The floor beam used is a lattice girder made from 4mm thick ST100mm as shown in Fig. 4.67.

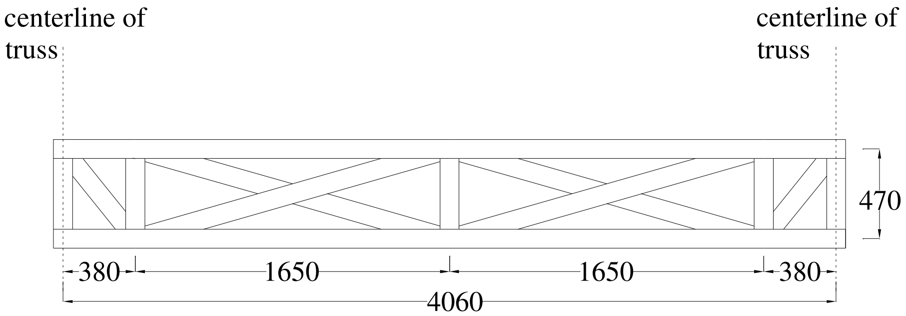


Figure 4.67 Lattice floor beam made from 4mm thick ST100 section

Dead loads

The center-to-center distance of floor beam is 0.75m.

Span of floor beam = 4.06m

Total steel deck weight per meter square = [(7850) (0.006) (3.3) (3.0) + (13) (9.22) (3.0)] (9.81) / ((3.3) (3.0)) = 818.36 N/m^2

Weight of steel deck = (818.36) (0.75) = 613.77 N/m

Assume built up SHS lattice girder from 4mm ST100 with f_y = 400 MPa and mass 11.73 kg / m.

Total length of floor beam members = 18.54 m

Weight of floor beam members = (11.73) (9.81) (18.54) = 2133.42 N/m

Weight per meter of floor beam = 2133.42 / 4.06 = 525.47 N/m

Floor beam weight = 0.53 kN/m

Total dead load = 613.77 + 525.47 = 1139.24 N/m

Dead load moment = (1139.24) (4.06) (4.06) / 8 = 2347.35 Nm = 2.35 kNm

The dead load is applied as distributed load to the floor beam as shown in Fig. 4.68.

Live Loads

Truck load

The truckload is placed so as to cause maximum live load moment (see Fig. 4.69 and Fig. 4.70). The maximum load occurs when the heavier axle load is on the floor beam as shown in Fig. 4.70.

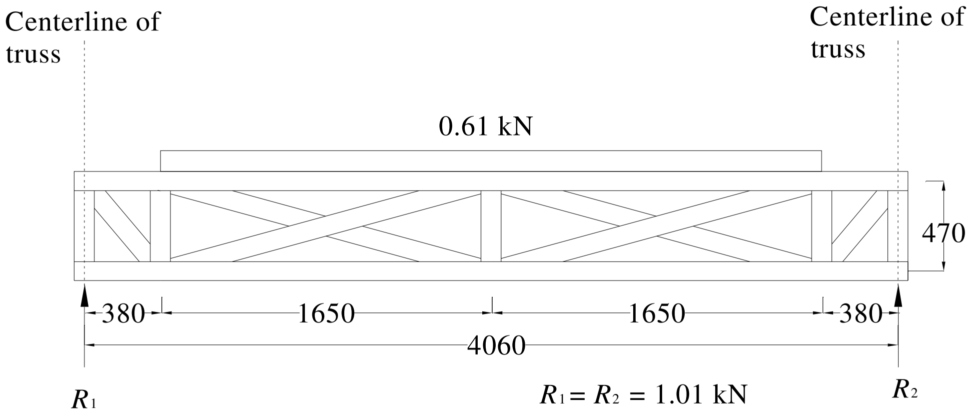


Figure 4.68 Uniform dead loads on floor beam

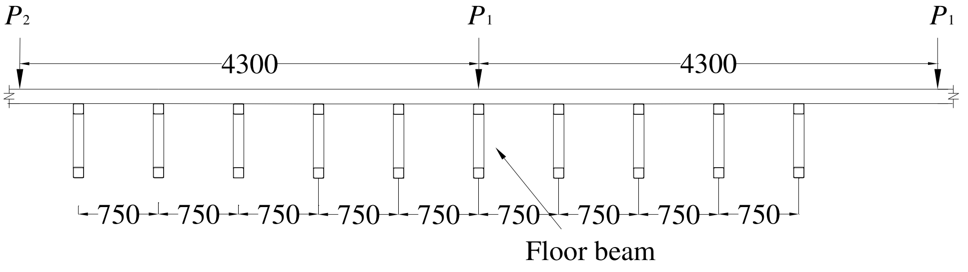


Figure 4.69 Truck moving to the left on the bridge

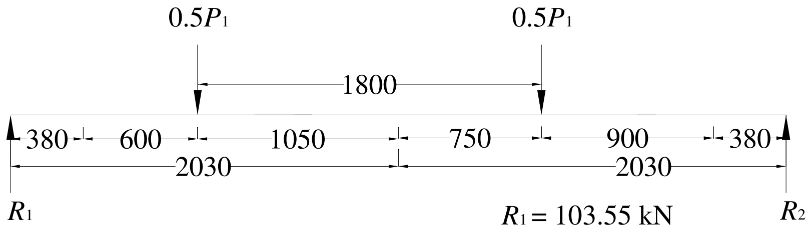


Figure 4.70 The heavier axle load on the floor beam

Maximum truck load moment = (103.55) (2.78) – (96.43) (1.8))= 114.30 kNm

Total limit truck and dead moment = (1.25) (2.35) + (1.75) (114.30) = 202.96 kNm

Total service truck and dead moment = (1.00) (2.35) + (1.30) (114.30) = 150.94 kNm

For tandem load

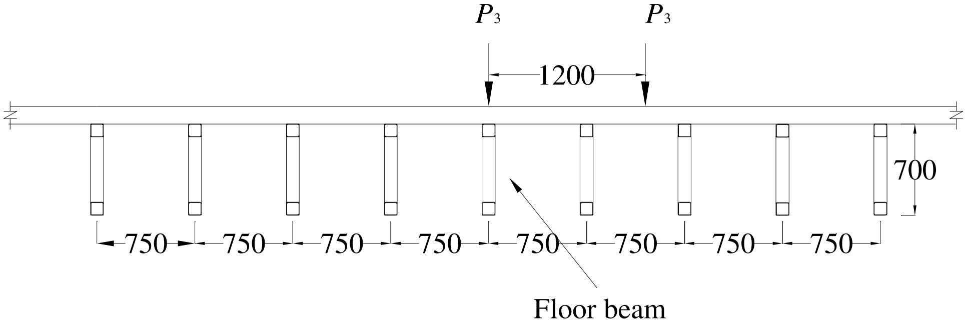


Figure 4.71 Tandem moving to the left on the bridge span

For maximum force effect, an axle is placed on the floor beam as shown in Fig. 4.71. The concentrated loads applied to floor beams from tandem are (see Fig. 4.72):

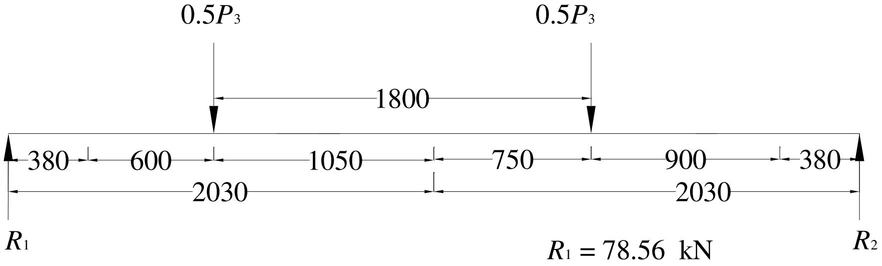


Figure 4.72 Tandem load on the floor beam

Maximum tandem load moment = (78.56) (2.78) – (73.15) (1.8) = 86.73 kNm

Total limit tandem and dead moment = (1.25) (2.35) + (1.75) (86.73) = 154.72 kNm

Total service tandem and dead moment = (1.00) (2.35) + (1.30) (86.73) = 115.10 kNm

For lane load (see Fig. 4.73)

Lane load in transverse direction 9.3 kN/ 3m = 3.1 kN/m

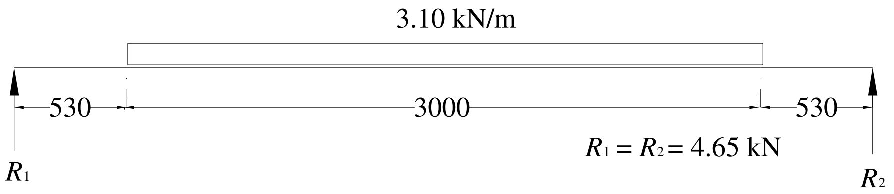


Figure 4.73 Lane load on floor beam

Maximum lane load moment = (4.65) (2.03) - (3.10) (1.5) (1.5) / 2 = 5.95 kNm

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

Table 4.6 Combination of truck or tandem with lane and dead moment for floor beam

No	Loading Type	Combined moment (in kNm)	Remark
1	Truck load and dead load with lane load (service)	150.94 + (5.95) (1.75) = 161.35	
2	Truck load and dead load with lane load (limit)	202.96 + (5.95) (1.75) = 213.37	Maximum
3	Tandem load and dead load with lane load (service)	115.10 + (5.95) (1.75) = 125.51	
4	Tandem load and dead load with lane load (limit)	154.72 + (5.95) (1.75) = 165.13	

Therefore the design moment is

MD = 213.37 kNm

The required section modulus for the above moment is

$S = 213.37 / (363{,}000) = 0.0005867769\ \text{m}^3 = 587.80 \times 10^3\ 14\ \text{mm}^{\ 3}$

Check section modulus of floor beam shown in Fig. 4.74.

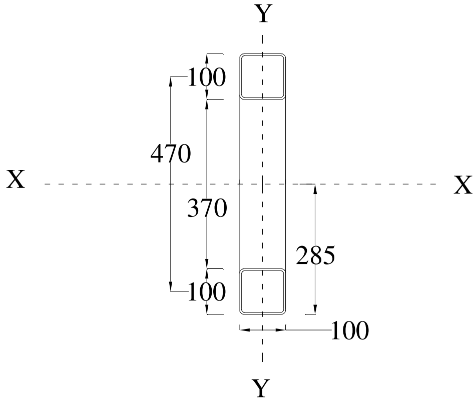


Figure 4.74 Cross section of floor beam in mm

SHS 100 x 100 x 4 mm

$A = 1495 \text{ mm}^2$ $\quad I = 2{,}264{,}000 \text{ mm}^4$

$d = 470/\,2 = 235$ mm

$I_{xx} = (2)\ (I_o + A\,d^2) = (2)\ (2264000 + (1495)\ (235)\ (235)) = 273.63 \times 10^6\ \text{mm}^4$

$C_y = 285$ mm

$S_{xx} = I_{xx} / C_y = 595.27 \times 10^3\ \text{mm}^3 > 587.80 \times 10^3\ 14\ \text{mm}^3$

The section is sufficient.

Check floor beam members' capacity by analyzing the floor beam as a truss

The simply supported floor beam is checked for maximum member forces by applying the dead load (see Fig. 4.75), the truck load (see Fig. 4.76) and lane load (see Fig 4.77) at the joints on the top chord. The deck dead load, live and lane load are not placed on the joints and so their effects are applied to the joints near them in proportion to their distance from the joints. These loads are factored using factors of 1.25 for dead load and 1.75 for lane and live load. These loads are analyzed using SAP2000 version 8 software and the maximum member forces are taken for design (refer attached CD). From the analysis:

the maximum forces on diagonal are

246.91 kN tension and

159.63 kN compression

and the maximum forces on chord are

299.22 kN tension and

290.44 kN compression

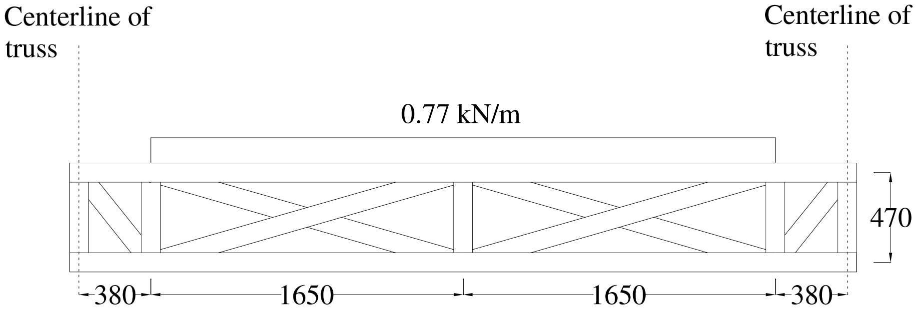


Figure 4.75 Factored dead load on floor beam

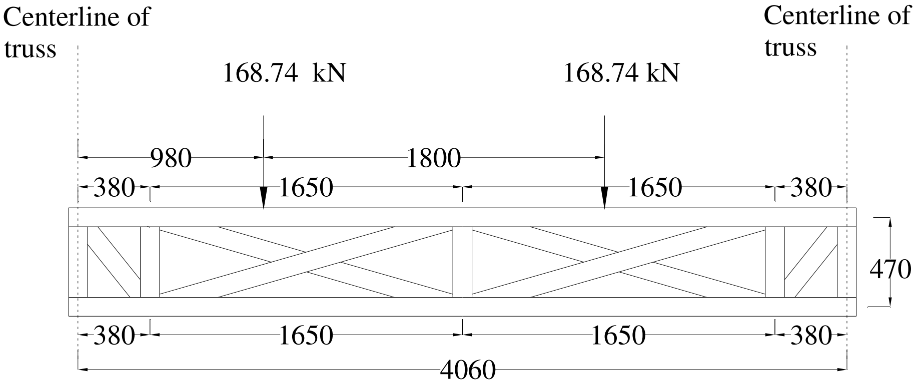


Figure 4.76 Factored truck load with impact on floor beam

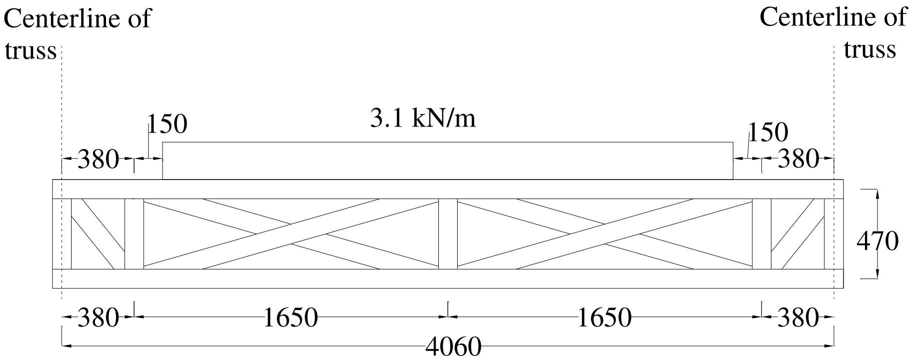


Figure 4.77 Lane load on floor beam

and the maximum forces on verticals are

0 kN tension and

192.06 kN compression

Design of floor beam members

Member capacities as compression members

Check local buckling

Classification of section used in the design (see Fig. 4.78).

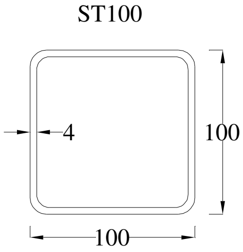


Figure 4.78 Cross-section of ST100

Check flange

$\varepsilon = \sqrt{(235/f_y)}$, f_y in MPa

$\varepsilon = \sqrt{(235/400)} = 0.7665$

For internal element of compression flange

$C / t_f \leq 28\ \varepsilon$

$(100-(3)(4)) / 4 \leq (28)(0.7665)$

$22 > 21.46 \xrightarrow{\qquad}$ flange is class four.

$\bar{\lambda}_\rho = (C/t_f)/(28.4\ \varepsilon \sqrt{k_\sigma}) = 22/((28.4)(0.7665)(\sqrt{4})) = 0.51 < 0.673$

$\rho = 1$

$C_{eff} = \rho C = (1)(88) = 88$ mm

Total area that should be ignored from the two flanges is

$\Delta A_f = (2)(88-88)(4) = 0$

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

$(100-(3)(4))/4 \leq (28)(0.7665)$

$22 \leq 21.46 \xrightarrow{\qquad}$ web is class four.

$\bar{\lambda}_\rho = (d/t_w)/(28.4\,\varepsilon\sqrt{k_\sigma}) = 22/((28.4)(0.7665)(\sqrt{4})) = 0.51 < 0.673$

$\rho = 1$

$C_{eff} = \rho C = (1)(88) = 88$ mm

Total area that should be ignored from the two webs is

$\Delta A_w = (2)(88-88)(4) = 0$

$A = A_{eff}$ i.e., the effective area is equal to total area.

The 4mm thick ST100 cross-section is class 4 and its design compression resistance is:

$N_{com,Rd} = A_{eff}\, f_y / \gamma_{m1}$

$N_{com,Rd} = (1495)(400)/1.1 = 543.64$ KN

Check overall buckling

The longest member length from the diagonals is 1716 mm, from the verticals 470 mm and from the chord members is 1650 mm.

For ST100 diagonal, $i_x = i_y = 3.89$ cm , $L = 171.6$ cm , $\lambda = 44.11$

$\lambda_1 = 71.97$

$\bar{\lambda} = (44.11 / 71.97)[1]^{0.5} = 0.78$

$\chi = 0.7363$, $N_{b,Rd} = (0.7363)(1)(1495)(400)/1.1$

$N_{b,Rd} = 400.28$ kN

For ST100 chord, $i_x = i_y = 3.89$ cm , $L = 165.00$ cm , $\lambda = 42.42$

$\bar{\lambda} = (42.42 / 71.97)[1]^{0.5} = 0.59$

$\chi = 0.8418$, $N_{b,Rd} = (0.8418)(1)(1495)(400)/1.1$

$N_{b,Rd} = 457.63$ kN

For ST100 vertical, $i_x = i_y = 3.89$ cm , $L = 47.00$ cm , $\lambda = 12.08$

$\bar{\lambda} = (12.08 / 71.97)[1]^{0.5} = 0.17$

$\chi = 1.0$, $N_{b,Rd} = (1.0)(1)(1495)(400)/1.1$

$N_{b,Rd} = 543.64$ kN

Member capacities as tension members

The floor beam members do not have holes on them and because of this checking the design plastic resistance is sufficient.

$N_{pl,Rd} = A\, f_y / \gamma_{m1}$

For 4 mm thick ST100,

A = 1495 mm$^{.2}$

$N_{pl,Rd}$ = (1495) (400) / 1.1

$N_{t,Rd} = N_{pl,Rd}$ = 543.64 KN

The floor beam chord and diagonal members are sufficient.

4.2.2.2.8 Design of truss

The Second option

Dead loads - super imposed loads (see Fig. 4.79)

The design dead load transferred to truss at panel points = 1.01 + (0.53) (4.06)/2= 2.08 kN.

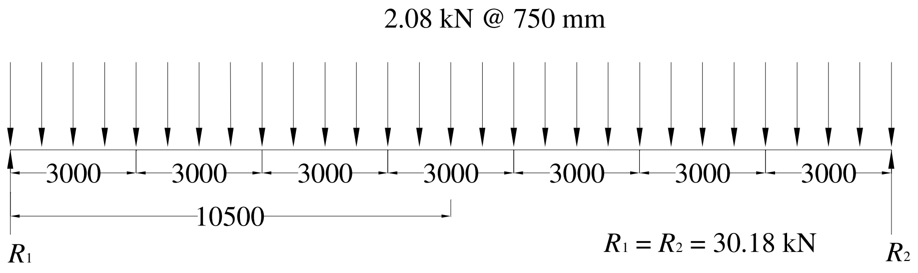


Figure 4.79 Concentrated dead loads on truss at panel points

Maximum moment = (30.18) (10.5) - (2.08) (10.5 + 9.75 + 9 + 8.25 + 7.5 + 6.75 + 6 + 5.25 + 4.5 + 3.75 + 3 +2.25 + 1.5 + .75) = 153.14 kNm

Dead loads - self weight (see Fig. 4.48)

Assume built-up SHS truss from ST100 with f_y = 400MPa and mass 11.73 kg/m

Span of steel truss = 21m

Panel length = (2) (0.75) (8) + (0.68) (6) + (8)√ ((0.75) (0.75) + (0.68) (0.68)) = 24.12 m

Total length of steel truss = (7) (24.12) = 168.86 m

Total weight of steel truss = (168.86) (9.81) (11.73) = 19.43 kN

Steel truss weight = 0.93 kN/m

Dead load- self weight moment = (0.93) (21) (21) / 8 = 51.27 kNm

Total dead load moment = 153.14 + 51.27 = 204.41 kNm.

Live Loads

Place the uniform lane loadings as close as possible to one of the trusses to cause maximum loads on that truss as shown in Fig. 4.80.

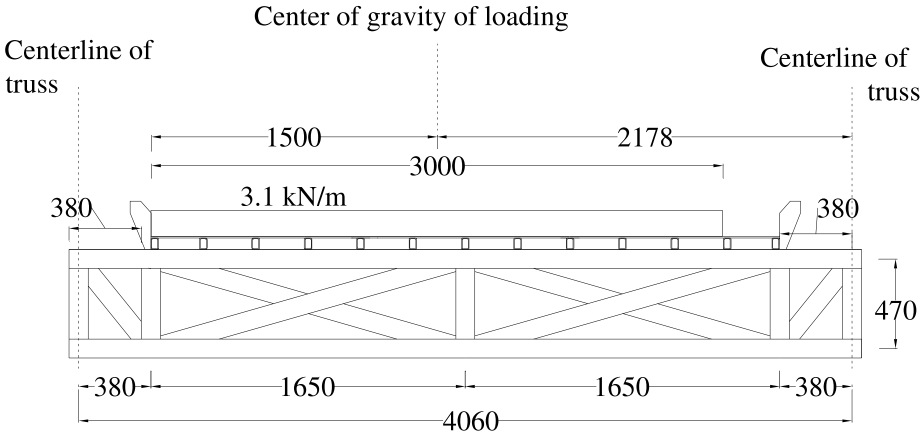


Figure 4.80 Lane load position for maximum load on left truss

From figure 4.80 part of the live load going to the left truss is determined to be 2.178/4.06 or 54 percent.

Uniform lane loading going to one truss = (3.1) (3) (0.54) = 5.02 kN per meter length

The lane loading transferred to a panel point = (5.02) (0.75) = 3.77 kN

Maximum lane load moment on truss = (5.02) (21) (21) / 8 = 276.73 kNm

Similarly determine the maximum percentage of concentrated wheel loads going to the left truss.

For truck loads (see Fig. 4.81 for rear axle and Fig. 4.82 for front axle)

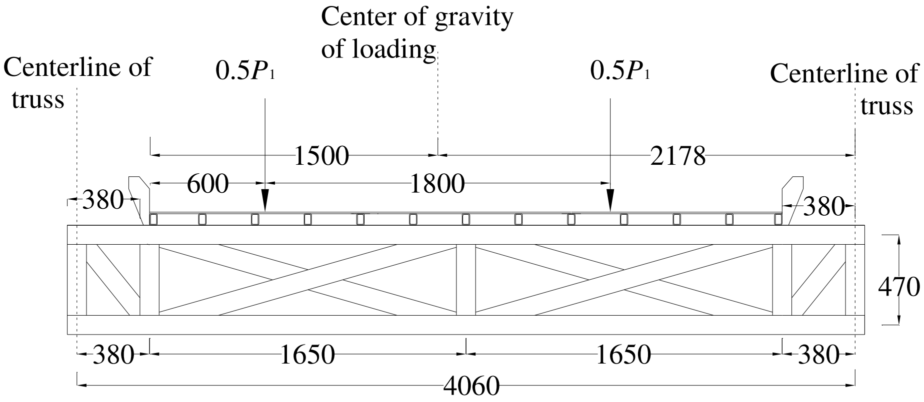


Figure 4.81 Truck load rear axle position for maximum load on left truss

The percentage of the live load going to the left truss is 2.178 / 4.06 or 54 percent.

Concentrated truck rear axle load going to one truss = (192.85) (0.54) = 104.14 kN

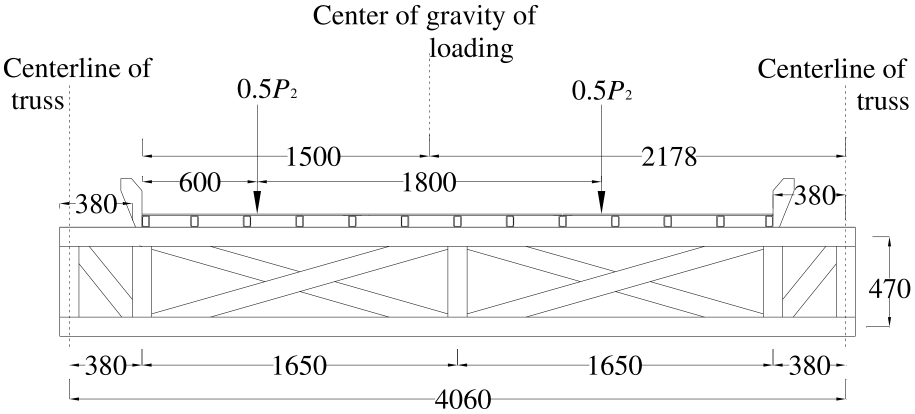


Figure 4.82 Truck load front axle position for maximum load on left truss

The percentage of the live load going to the left truss is 2.178/ 4.06 or 54 percent.

Concentrated truck front axle load going to one truss = (46.55) (0.54) = 25.14 kN.

The maximum moment for truck load is under the first heavy axle when the mid-span is half way between the resultant and this axle as shown in Fig. 4.83.

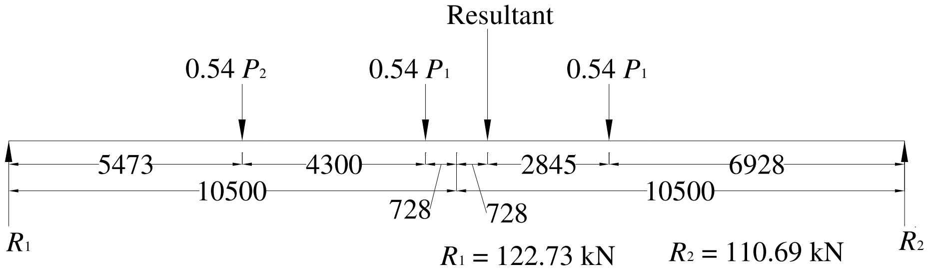


Figure 4.83 Truck load position on truss for maximum moment

Maximum truck load moment on truss = (122.73) (10.50 - 0.728) - (0.54) (46.55) (4.30) = 1091.29 kNm

Total limit truck and dead moment = (1.25) (204.41) + (1.75) (1091.29) = 2165.27 kNm

Total service truck and dead moment = (1.00) (204.41) + (1.3) (1091.29) = 1623.09 kNm

For tandem loads (see Fig. 4.84)

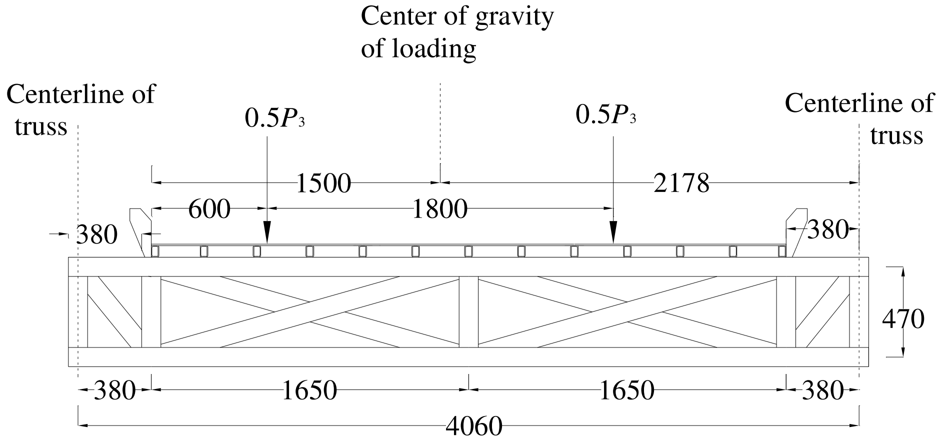


Figure 4.84 Tandem load position for maximum load on left truss

The percentage is 2.178 / 4.06 or 54 percent.

Concentrated tandem axle load going to one truss = (146.30) (0.54) = 79.00 kN

The maximum moment for tandem load is under the first axle when the mid-span is half way between the resultant and this axle as shown in Fig. 4.85.

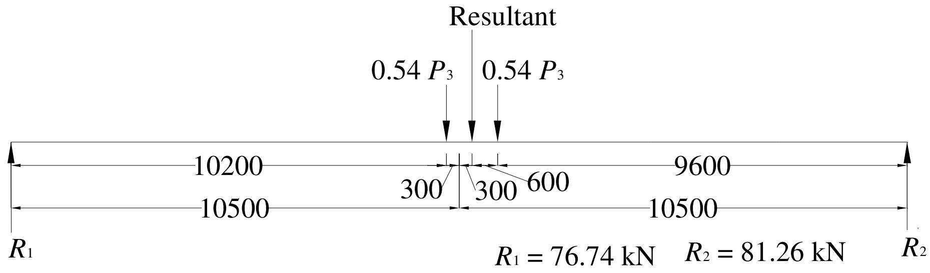


Figure 4.85 Tandem load position on truss for maximum moment

Maximum tandem load moment on truss = (76.74) (10.50 - 0.30) = 782.75 kNm
Total limit tandem and dead moment = (1.25) (204.41) + (1.75) (782.75) = 1625.33 kNm
Total service tandem and dead moment = (1) (204.41) + (1.30) (782.75) = 1221.99 kNm

A summary of the combination of the truck or tandem moment with lane and dead moment is shown in Table 4.7.

Table 4.7 Combination of truck or tandem with lane and dead moment for truss

No	Loading Type	Combined moment (in kNm)	Remark
1	truck load and dead load (service) with lane load	1623.09 + (276.73) (1.75) = 2107.37	
2	truck load and dead load (limit) with lane load	2165.27 + (276.73) (1.75) = 2649.54	maximum
3	tandem load and dead load (service) with lane load	1221.99 + (276.73) (1.75) = 1706.27	
4	tandem load and dead load (limit) with lane load	1625.33 + (276.73) (1.75) = 2109.61	

Therefore the design moment controlled by combination of truck load and dead load with lane load is

$$MD = 2649.54 \text{ kNm}$$

The required section modulus for the above moment is

$S = M / f_b = 2649.54 / (363 \text{ x } 10^3) = 0.00729900826 \text{ m}^3 = 7.30 \text{ x } 10^6 \text{ mm}^3$

Check section modulus of truss shown in Fig. 4.86:

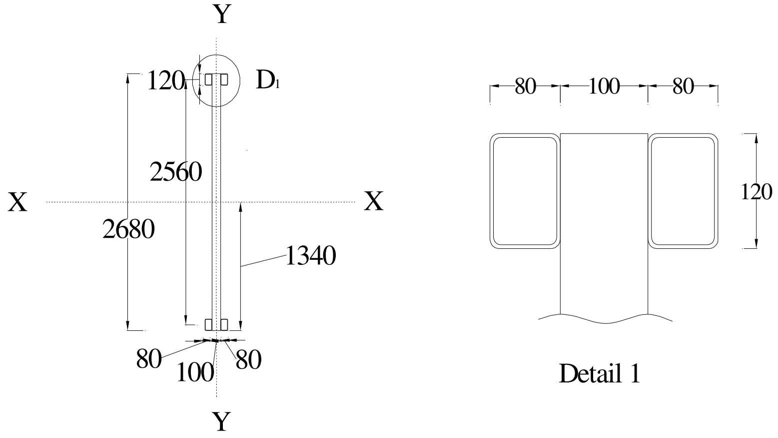


Figure 4.86 Cross section of truss in mm

RHS 120 x 80 x 4 mm

$A = 1495 \text{ mm}^2$ $\qquad I_x = 2{,}945{,}900 \text{ mm}^4$

$d = 2560/ 2 = 1280 \text{ mm}$

$I_{xx} = (4)\ (I_o + A\ d^2) = (4)\ (2945900 + (1495)\ (1280)\ (1280)) = 9{,}809 \text{ x } 10^6 \text{ mm}^4$

$C_y = 1340 \text{ mm}$

$S_{xx} = I_{xx} / C_y = 7.32 \text{ x } 10^6 \text{ mm}^3 > 7.30 \text{ x } 10^6 \text{ mm}^3$

The section is sufficient but too long. Consider using double truss with each truss carrying half the load. Therefore take half the required section modulus and check section capacity (see Fig. 4.87). 7,299,008.26

$S = 7.30 \text{ x } 10^6 \ / 2 = 3.65 \text{ x } 10^6 \text{ mm}^3$

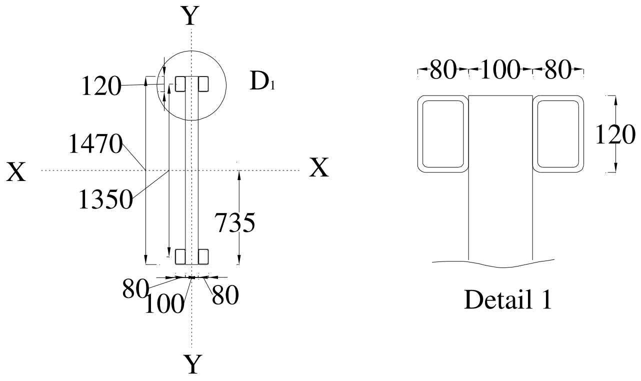


Figure 4.87 Cross section of truss in mm

RHS 120 x 80 x 4 mm

$A = 1495 \text{ mm}^2$ $\qquad I_x = 2{,}945{,}900 \text{ mm}^4$

$d = 1350 / 2 = 675 \text{ mm}$

$I_{xx} = (4)\ (I_o + A\ d^2) = (4)\ (2945900 + (1495)\ (675)\ (675)) = 2{,}736.42 \times 10^6 \text{ mm}^4$

$C_y = 735 \text{ mm}$

$S_{xx} = I_{xx} / C_y = 3.72 \times 10^6 \text{ mm}^3 > 3.65 \times 10^6 \text{ mm}^3$

The section is sufficient.

Check truss members' capacity by analyzing the truss using SAP 2000

The bridge truss is modeled for analysis using SAP 2000 version 8 software. The top and bottom chords are modeled using RT128 and the vertical and diagonal members using ST100. The design dead load (see Fig. 4.79) is applied to all panel points, which are located on the bottom chord of each panel. The design live load i.e. the truck load shown in Fig. 4.88 is applied on few panel points at a time. The analysis is done repeatedly in three groups which apply the first, second and third axle load at panel points respectively and distribute the effects of the other axles by considering their distance from panel points near them. The maximum member forces for design of a typical panel are found by varying the position of live load at panel points for each group for maximum effect.

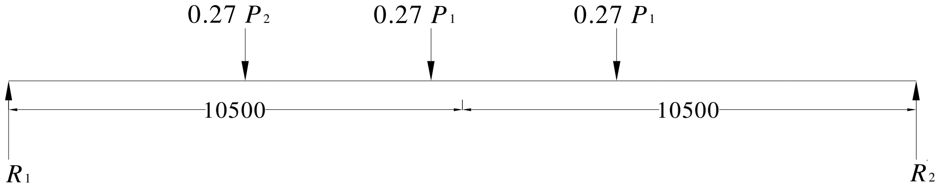


Figure 4.88 Half the truck load to be applied on truss

From the analysis, the maximum member forces found are

-for chord members

757.63 kN compression for RT128

738.22 kN for tension for RT128

-for vertical members

286.42 kN compression for ST100

96.16 kN for tension for ST100

-for diagonal members

171.79 kN compression for ST100

171.79 kN for tension for ST100

The First option

The first option differs from the second only by the amount of super imposed dead loads.

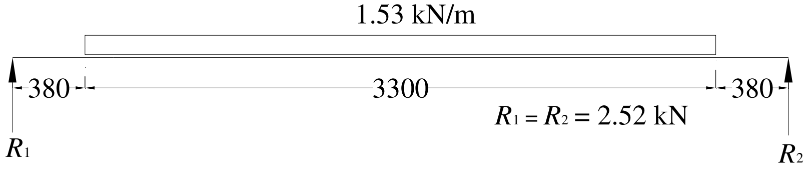


Figure 4.89 Super imposed dead load on floor beam

Dead loads - super imposed loads (see Fig. 4.89 and Fig. 4.90)

The design dead load transferred to truss at panel points = 2.52 + (0.55) (4.06) / 2

= 3.64 kN.

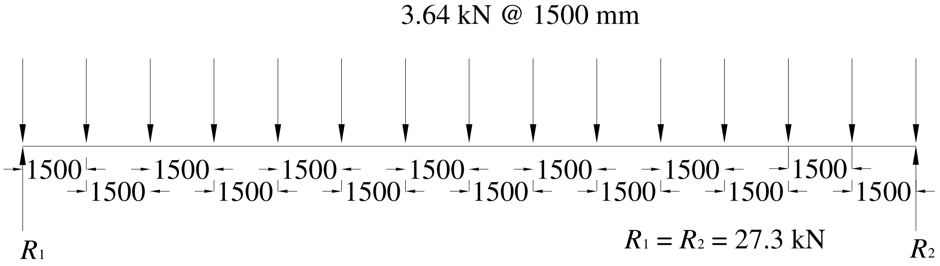


Figure 4.90 Concentrated dead loads on truss at panel points

Maximum moment = (27.30) (10.5) - (3.64) (10.5 + 9 + 7.5 + 6 + 4.5 + 3 +1.5) = 133.70 kNm < 153.14 kNm for the stiffened plate option. Therefore, the analysis for the first option is taken as being the same as the analysis for the second option.

Design of panel members

Member capacities as compression members

Check local buckling

Classification of sections used in the design (see Fig. 4.91, Fig. 4.92 and Fig. 4.91).

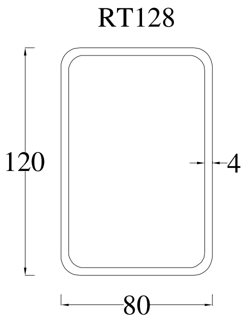


Figure 4.91 Cross section of RT128

Check flange

$\varepsilon = \sqrt{(235/ 400)} = 0.7665$

For internal element of compression flange

$C / t_f \leq 23\ \varepsilon$

(80-(3) (4)) / 4 ≤ (23) (0.7665)

$17 \leq 17.63 \xrightarrow{\qquad}$ flange is plastic

Check web

Web, where the whole section is subject to compression

$d / t_w \leq 28\ \varepsilon$

(120-(3) (4)) / 4 ≤ (28) (0.7665)

$27 > 21.46 \xrightarrow{\qquad}$ web is class four.

The whole section is thin walled

Stress distribution: Since the member is axially loaded, the stress distribution is uniform; i.e., $\sigma_1 = \sigma_2$. From table 4.3 of EBCS 3, 1995 for $\psi = \sigma_1 / \sigma_2 = 1.00$, the corresponding value of $k_\sigma = 4.0$ According to section 4.3.4(4) of EBCS 3, 1995,

$\bar{\lambda}_p = (d/t)/(28.4\ \varepsilon \sqrt{k_\sigma}) = 27/((28.4)\ (0.7665)\ (\sqrt{4})) = 0.62 < 0.673$

$\rho = 1$

$d_{eff} = \rho d = (1)\ (108) = 108$ mm

Total area that should be ignored from the two webs is

$\Delta A_w = (2)\ (108-108)\ (4) = 0$

This shows that the effective area is equal to total area.

ST100

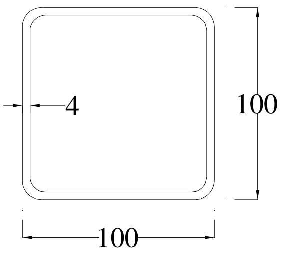


Figure 4.92 Cross-section of ST100

Check flange

$\varepsilon = \sqrt{(235/ 400)} = 0.7665$

For internal element of compression flange

$C / t_f \leq 28\ \varepsilon$

(100-(3) (4)) / 4 ≤ (28) (0.7665)

$22 > 21.46 \xrightarrow{\qquad}$ flange is class four.

$\bar{\lambda}_p = (C/ t_f)/(28.4\ \varepsilon \sqrt{k_\sigma}) = 22/((28.4)\ (0.7665)\ (\sqrt{4})) = 0.51 < 0.673$

$\rho = 1$

$C_{eff} = \rho C = (1)\ (88) = 88$ mm

Total area that should be ignored from the two flanges is

$\Delta A_f = (2)\ (88\text{-}88)\ (4) = 0$

Check web

Web, where the whole section is subject to compression

$d\,/\,t_w \ \leq 28\ \varepsilon$

$(100\text{-}(3)\ (4))\ /\ 4 \leq (28)\ (0.7665)$

$22 \leq 21.46$ → web is class four.

$\bar{\lambda}_p = (d/\,t_w\,)/\ (28.4\ \varepsilon\sqrt{k_\sigma}) = 22/\ ((28.4)\ (0.7665)\ (\sqrt{4})) = 0.51 < 0.673$

$\rho = 1$

$C_{eff} = \rho C = (1)\ (88) = 88$ mm

Total area that should be ignored from the two webs is

$\Delta A_w = (2)\ (88\text{-}88)\ (4) = 0$

$A = A_{eff}$ i.e., the effective area is equal to total area.

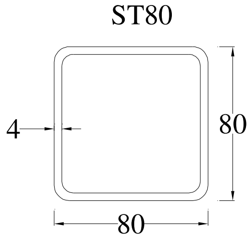


Figure 4.93 Cross-section of ST80

Check flange

$\varepsilon = \sqrt{(235/\ 400)} = 0.7665$

For internal element of compression flange

$C\,/\,t_f \ \leq 28\ \varepsilon$

$(80\text{-}(3)\ (4))\ /\ 4 \leq (23)\ (0.7665)$

$17 \leq 17.63$ → flange is plastic.

Check web

Web, where the whole section is subject to compression

$d\,/\,t_w \ \leq 28\ \varepsilon$

$(80\text{-}(3)\ (4))\ /\ 4 \leq (28)\ (0.7665)$

$17 \leq 21.46 \longrightarrow$ web is plastic.

The whole section is plastic.

The design compression resistance for RT128 and ST100 is:

Class 4 cross sections: $N_{com,Rd} = A_{eff}\ f_y / \gamma_{m1}$

For RT128, $N_{com,Rd}$ = (1495) (400)/1.1 = 543.64 KN

For ST100, $N_{com,Rd}$ = (1495) (400)/1.1 = 543.64 KN

The design compression resistance for ST80 is:

$N_{com,Rd} = A\, f_y / \gamma_{mo}$

For ST80, $N_{com,Rd}$ = (1175) (400) / 1.1 = 427.27 kN

Check overall buckling

The design buckling resistance of a compression member is

$N_{b,Rd} = \chi\beta_A\, A\, f_y / \gamma_{m1}$

where $\beta_A = A_{eff}/A$ for class 4 cross sections and β_A =1 for class 1, 2 or 3 cross sections.

$\beta_A = 1$

$\overline{\lambda} = (\lambda / \lambda_1)\ [\beta_A\ ]^{0.5}$

$\lambda_1 = 93.\ 9\varepsilon$

$\varepsilon = [235 / f_y]^{0.5}$ (f_y in MPa)

$\varepsilon = [235 / 400]^{0.5} = 0.7665$

$\lambda_1 = 71.98$

The truss members are pin ended, so their buckling length is the same as their member length. The longest member length from the verticals is 675 mm, from the diagonals is 1009 mm and from the chord members is 750 mm. Buckling in the direction with the lower radius of gyration governs.

For RT128 chord, i_x = 4.44 cm, i_y = 3.24 cm, L = 75.00 cm , λ = 23.15

$\overline{\lambda} = (23.15 / 71.97)\ [1]^{0.5} = 0.32$

$\chi = 0.9565$, $N_{b,Rd}$ = (0.9565) (1) (1495) (400) / 1.1

$N_{b,Rd}$ = 519.99 kN

For ST100 diagonal, $i_x = i_y$ = 3.89 cm , L = 100.9 cm , λ = 25.94

$\bar{\lambda} =$ (25.94 / 71.97) $[1]^{0.5} = 0.36$

$\chi = 0.9413$, $N_{b,Rd} = (0.9413)\ (1)\ (1495)\ (400) / 1.1$

$N_{b,Rd} = 511.72$ kN

For ST100 vertical, $i_x = i_y = 3.89$ cm , $L = 67.5$ cm , $\lambda = 17.35$

$\bar{\lambda} =$ (17.35 / 71.97) $[1]^{0.5} = 0.24$

$\chi = 0.9856$, $N_{b,Rd} = (0.9856)\ (1)\ (1495)\ (400) / 1.1$

$N_{b,Rd} = 535.81$ kN

For ST80 diagonal, $i_x = i_y = 3.07$ cm , $L = 100.9$ cm , $\lambda = 32.87$

$\bar{\lambda} =$ (25.94 / 71.97) $[1]^{0.5} = 0.46$

$\chi = 0.9010$, $N_{b,Rd} = (0.9010)\ (1)\ (1175)\ (400) / 1.1$

$N_{b,Rd} = 384.97$ kN

For ST80 vertical, $i_x = i_y = 3.07$ cm , $L = 67.5$ cm , $\lambda = 21.99$

$\bar{\lambda} =$ (17.35 / 71.97) $[1]^{0.5} = 0.31$

$\chi = 0.9603$, $N_{b,Rd} = (0.9603)\ (1)\ (1175)\ (250) / 1.1$

$N_{b,Rd} = 410.31$ kN

The top and bottom chord members are made from two RT128 members. Because of this the capacity of the chord is = (2) (519.99) = 1039.98 kN.

Member capacities as tension members

The design tension resistance capacity of the cross section is:

$N_{pl,Rd} = A\, f_y / \gamma_{m1}$

For RT128,

$A = 1495\ \text{mm}^{-2}$

$N_{pl,Rd} = (1495)\ (400) / 1.1$

$N_{pl,Rd} = 543.64$ kN

For ST100,

$A = 1495\ \text{mm}^{-2}$

$N_{pl,Rd}$ = (1495) (400) / 1.1

$N_{pl,Rd}$ = 543.64 kN

For ST80,

A = 1175 mm^{-2}

$N_{pl,Rd}$ = (1175) (400) / 1.1

$N_{pl,Rd}$ = 427.27 kN

Using similar reasoning as given above for the chord members in compression, the tensile capacity of the chord is = (2) (543.64) = 1087.28 kN.

The sections used are sufficient. The ST80 instead of ST100 can be used for the vertical and diagonal members to bring in a more economical design.

4.2.2.2.9 Design of connections

4.2.2.2.9.1 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 AASHTO article 10.19.1.1, 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. On the panel end vertical at mid height three members (one vertical and two diagonal) are connected while on mid panel vertical five members (one vertical and four diagonals) are connected. On the panel corners one vertical and two chord members are connected. The top and bottom chords also have two diagonals connected to them at quarter lengths.

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, ksi

F_w = nominal strength of the weld electrode, ksi

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

A_w = cross sectional area of the weld, in^2

ϕ = resistance factor

Values for ϕ, F_{bm}, and F_w are given in Table 4.8.

Table 4.8 Values for ϕ, F_{bm}, and F_w

Fillet welds				
Type of weld or stress	Material	Resistance factor , ϕ	Nominal strength, F_{bm} or F_w	Required weld strength level
Stress on effective area	Base weld electrode	0.75	0.6 F_{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	0.90	f_y	

where, F_{EXX} = nominal tensile strength of the weld metal, ksi

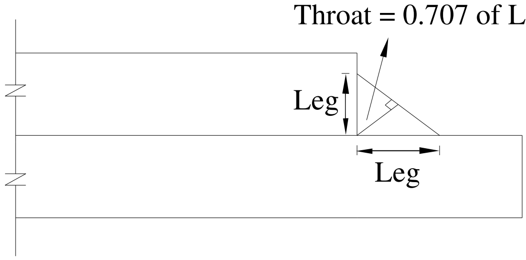


Figure 4.93 Weld throat

A_w = (length of weld) (throat)

The throat length is shown in Fig. 4.93

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 4.9 below [8].

Table 4.9 Minimum size of fillet weld

Material thickness of the thicker parts joined, t, in	Minimum size of fillet weld,* in
$t \leq 1/4$	1/8
$1/4 < t \leq 1/2$	3/16
$½ < t \leq 3/4$	1/4
$t > 3/4$	5/16

* Leg dimension of fillet welds

Design of fillet weld

The full and 75% member capacities of ST100, RT128 and ST80 are listed below. E70 electrodes are to be used. Tensile capacities of the main members are:

RT128: $N_{t,Rd} = 543.64$ kN 75% $N_{t,Rd} = 407.73$ kN
ST100: $N_{t,Rd} = 543.64$ kN 75% $N_{t,Rd} = 407.73$ kN
ST80: $N_{t,Rd} = 427.27$ kN 75% $N_{t,Rd} = 320.45$ kN

For E70 electrodes the design strength is:

$\phi F_w = (0.75)\ (0.6)\ (F_{EXX}) = (0.75)\ (0.6)\ (70) = 31.5$ ksi

$\phi F_w = (31.5)\ (6895) = 217.19$ MPa

For E60 electrodes the design strength is:

$\phi F_w = (0.75)\ (0.6)\ (F_{EXX}) = (0.75)\ (0.6)\ (60) = 27$ ksi

$\phi F_w = (27)\ (6895) = 186.17$ MPa

Capacity of weld = $\phi F_w A_w = (217.19)\ (3.54)\ (L) = 767.77\ L$

Capacity of weld / L = $\phi F_w A_w / L = 0.77$ kN/ mm

The plate connection is done using a set consisting of two plates, 5mm thick, one on top and the other at the bottom of the diagonal members and is located at mid height and on the chords of panel. It connects the vertical and diagonal members. The weld on one plate is to be designed to carry half the 75% capacity of the member, the other half being carried by the other plate weld. The vertical members are connected to the chord members with out the use of plates. One side of the connection carries half the 75% capacity of the member. The weld to be used is a fillet weld of size 5 mm on the sides and the end.

Check the length requirement to carry this force by the plate and weld.

Capacity of base plate = $\phi F_{bm} A_{bm} = (L/25.4)\ (5/25.4)\ (0.9)\ (400/6.895) = 0.40L$ kip

Capacity of base plate = $(0.40L)\ (4.448) = 1.80L$ kN

Capacity of base plate/L= 1.80 kN/mm > 0.77kN/mm.

Therefore, use weld capacity to determine weld length.

From Table 4.9 minimum leg dimension for $t = 5$ mm = 0.2 in is (1/8) (25.4) = 3.18mm

Use a leg dimension of 5mm.

Throat thickness = (0.707) (leg dimension)

= (0.707) (5) = 3.54 mm

The maximum required of the capacities is 407.73 /2 =203.87 kN

Length of weld required = 203.87 / $(\phi F_w A_w / L)$ = 203.87/0.77 = 264.77mm

Use 280 mm long weld.

Use end return length = 2 weld size = (2) (5) = 10mm for plate corners.

Plate types used

A. Plate #1

Plate #1 is a 44 x 23 cm, 5 mm thick plate with f_y = 400 MPa that connects the two diagonals with the vertical at panel ends using welds as shown in Fig. 94 an Fig. 4.95 below.

B. Plate #2

Plate #2 is a 44 x 46 cm, 5 mm thick plate with f_y = 400 MPa that connects the four diagonals with the vertical at mid panel using welds as shown in Fig. 94 and Fig. 4.96 below.

C. Plate #3

Plate #3 is a 56 x 5 cm, 5 mm thick plate with f_y = 400 MPa that connects the two diagonals with the chord using welds as shown in Fig. 94 below.

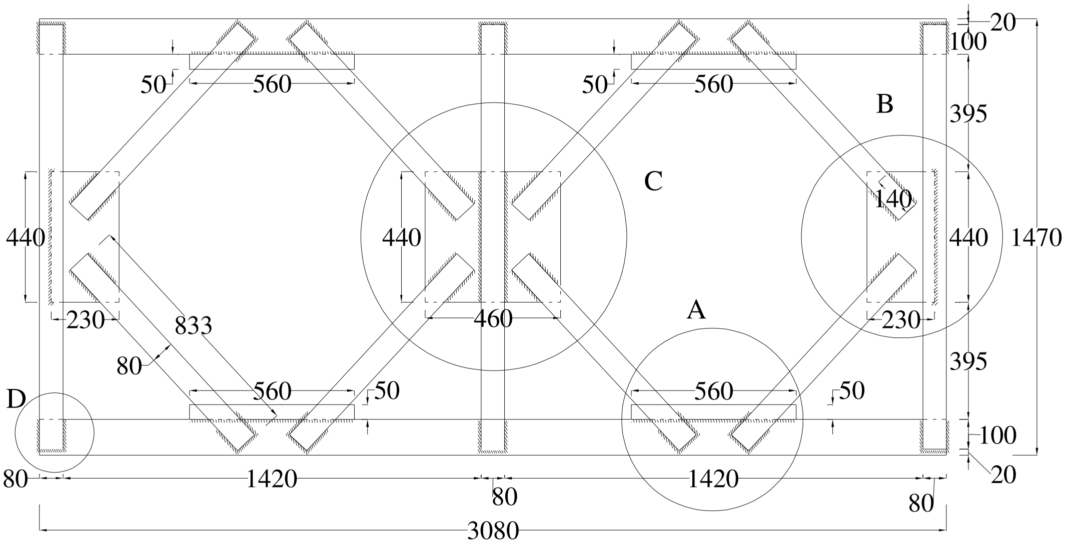


Figure 4.94 Weld connections on panel

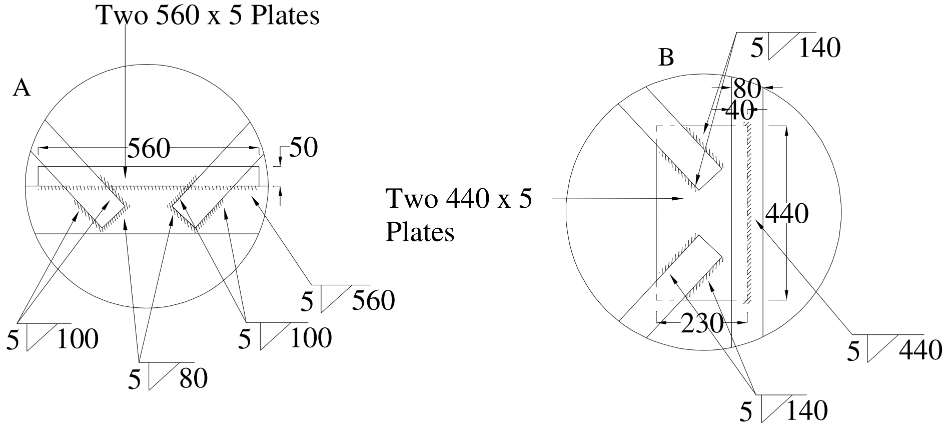


Figure 4.95 Detals A and B of weld connections on panel

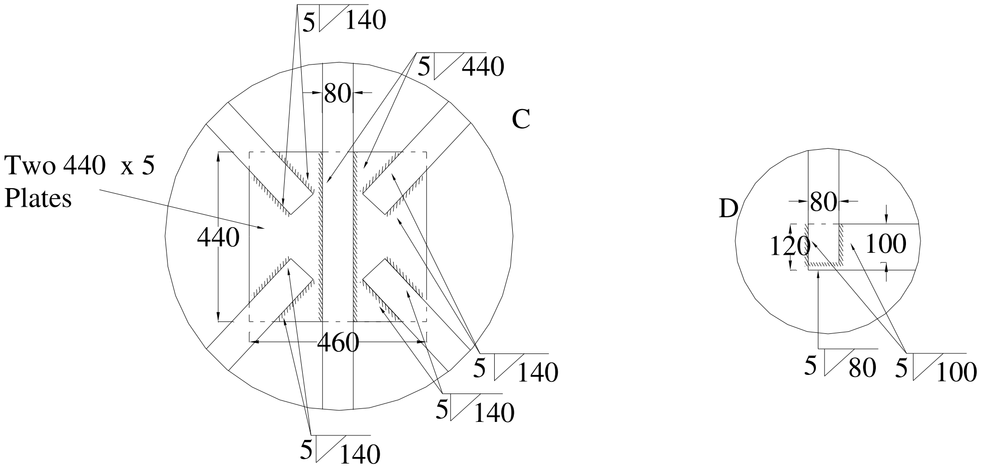


Figure 4.96 Detals C and D of weld connections on panel

4.2.2.2.9.2 Design of pin and pin plates for panel connection

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 [9].

The shear capacity $F_{V,RD}$ of a pin shall be taken as:

$$F_{V,RD} = 0.6\, A\, f_{up} / \gamma_{MP}$$

where, $F_{V,RD}$ is the specified minimum ultimate strength of pin

A is the cross sectional area of pin

The bearing capacity $F_{b,RD}$ of a pin shall be taken as:

$$F_{b,RD} = 1.50\, d\, t\, f_y / \gamma_{MP}$$

where, d is the diameter of pin

t is the thickness of the connected part

f_y is the lower of the nominal yield strength of pin and the connected part.

The moment capacity of the pin M_{RD} shall be taken as:

$$M_{RD} = 0.8\ W f_{yp} / \gamma_{MP}$$

where, f_{yp} is the nominal yield strength of pin

W is the section modulus

Design of pin connection

Check for shear capacity

Assumed pin material is Fe510 $\longrightarrow$ f_{up} = 490 MPa, f_y = 335 MPa

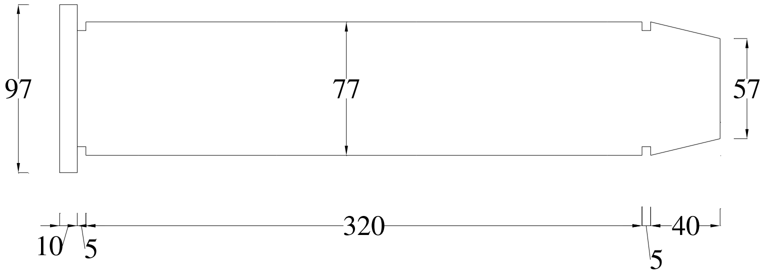


Figure 4.97 Pin dimension in mm

Assumed diameter of pin = 7.7 cm (see Fig. 4.97)

γ_{MP} = 1.25

A = (77) (77) (3.14) / 4 = 4656 mm^2

$F_{V,RD}$ = (0.6) (4656) (490) / 1.25 = 1095.09 kN

The full member capacity of RT128 chord is:

RT128: $N_{t,Rd}$ = (2) (543.64) = 1087.28 kN < 664.91 kN

Check for bearing capacity

Use Fe510 steel plates for connection of panels with pin welded at the ends of bottom and top chords. On one side of the panel four plates, one on each chord, are connected to the inside of the chords. On the other end another four plates, one on each chord, are connected to the outside of the chords. These plates contain holes for the pin connection.

According to AASHTO article 11.4.9.2, the diameter of pin hole shall not exceed that of the pin by more than 1/50 inch for pins 5 inches = 12.7 cm or less in diameter.

Use pin hole diameter of 78 mm. Also use EBCS3, 1995 article 6.3.2 to dimension the plates (see Fig 4.98 and Fig. 4.99).

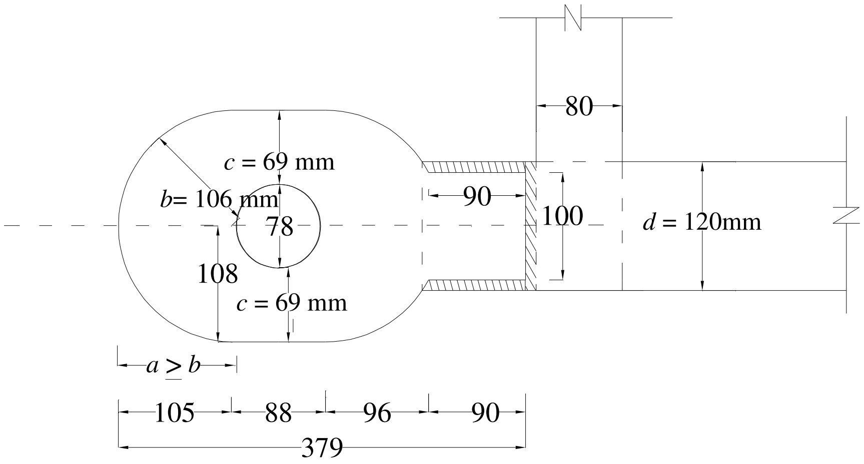


Figure 4.98 Pin connection plate

Assumed thickness of inside plate = 3 cm

Assumed thickness of outside plates = 3 cm

$A = dt = (100)\ (30) = 3000\ \text{mm}^2$

$t \geq 0.25C = (0.25)\ (69) = 17.25$ mm

$bt = (106)\ (30) = 3180\ \text{mm}^2 \geq A = 3000\ \text{mm}^2$

$2ct = (2)\ (69)\ (30) = 4140\ \text{mm}^2 \geq 1.33\ A = 3990\ \text{mm}^2$

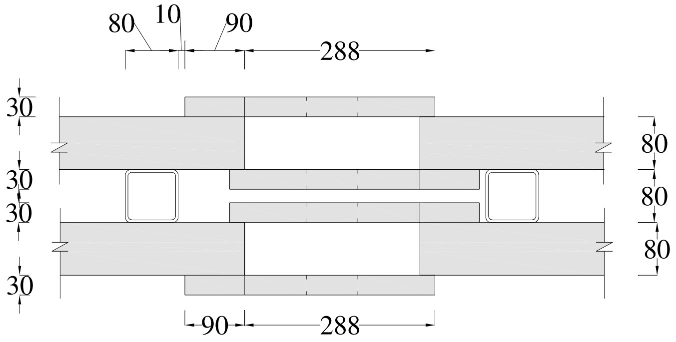


Figure 4.99 Pin plate plan view

$F_{b,RD} = 1.50\ d\ t\ f_y / \gamma_{MP}$

$d = 77$ mm

t = 30 mm

f_y = 355 MPa

$F_{b,RD}$ = (1.5) (77) (30) (355) / 1.25 = 984.06 kN > 543.64kN

Check for bending moment capacity

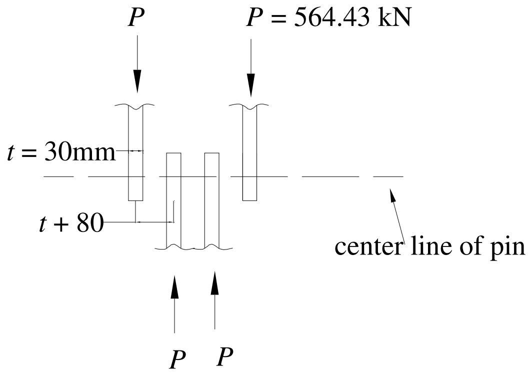


Figure 4.100 Forces on pin

The section modulus of a solid pin $W = \pi d^3/32$

The resisting moment capacity of the pin M_{RD} is

M_{RD} = (0.8) (π) (77^3) (335)/ 1.25 = 307.5 kNm

Moment on pin (see Fig 4.95) = (543.64) (30+80) = 59.80 kNm << 307.5 kNm

The pin and pin plates are okay.

4.3 Cost comparisons

Even though it is possible to design and manufacture assembled steel truss bridges locally, the cost of local design and production as compared with the imported ones is a significant factor in decision making. To get some idea of the cost of local production, an estimate was done by Kality Metal Products Factory for the production of the panel in the design example and compared with the cost of previously imported panels from two different companies in two different countries (see Table 4.10 and appendix B).

Table 4.10 Cost comparison for panel per kg

Year	Country	Company	Material Type	Weight, Kg	Cost (after tax)	Cost / Kg
2001	Austria	Waagner Biro	Panel Standard	335.40	1009.82 (in USD)	3.01 (in USD)
2006	China	Shaanixi Machinery and equipment Import and Export Corporation	Panel Standard	315.00	508.88 (in USD)	1.62 (in USD)
2007	Ethiopia	Kality Metal Products Factory	Panel in example	338.59	9258.54 (in Birr)	27.34 (in Birr)

As shown in table 4.10 the cost of local production is closer to importing from Austria but is definitely more expensive than importing from China. It should be noted that the local cost estimate includes all taxes thus making the total cost higher.

5 Conclusions and Recommendations

5.1 Conclusions

The study has addressed the possibility of analysis and design of steel bridges with locally available steel profiles. Based on the analysis and design made so far, the study has proved that, construction of steel bridge with locally available steel profiles is an option worth considering for remote and inaccessible areas. Even though the cost of local production is closer to importing from Austria, it is still a good choice because it helps in the capacity building of local design, fabrication and construction firms, creates job opportunities for many people and is a saving in foreign currency for our country. For many short span temporary bridges in road construction projects, these local assembled steel truss bridges can be used as temporary bridges.

In addition to the fact that these assembled steel bridges are preferable in inaccessible areas they also take very short time to erect. Furthermore, the construction of steel bridges will reduce the volume of false and form work requirements through which deforestation with in the country can be reduced. If the superstructure of a short span highway bridge fails during the rainy season, the construction of false work and detour may be difficult. In such cases the use of assembled steel truss bridges without false work at the site will solve such problem.

The analysis and design method developed in this study will lead to the design, fabrication and construction of assembled steel truss bridge which is not practiced in Ethiopia at this time by local construction or design companies.

5.2 Recommendations

Being an alternative solution to the method of bridge construction in remote and inaccessible parts of the country, the following recommendations can be made based on the study:

1. Fabrication of standard steel sections with higher strength and large thickness should be encouraged.
2. A counter check on quality needs to be conducted on imported steel bridges to confirm quality standards.
3. The government should take measures that encourage local design and production such as making such productions tax free.
4. The road construction sector needs to be aware of this alternative assembled steel bridge construction method.
5. An emphasis should be made in the utilization of other construction methods of a bridge like assembled steel bridges.
6. Private companies should be encouraged to engage in the design and fabrication steel bridges with locally available steel profiles in designing and building bridges and given supports for the development of the system.
7. The owners of bridges have to be aware of and lead consulting firms to use the developed method in remote areas as an alternative option.
8. The possibility of using steel superstructure for foot bridges should be checked.
9. In the thesis the steel deck was analyzed and designed, so a further study to should be conducted to include the detailing.
10. A study should be conducted to include the design of other parts the bridge so that the whole bridge could be designed and manufactured using local steel profiles.
11. The cost of local production could significantly be reduced if the government introduces a policy of tax reduction for steel intended for assembled steel truss bridges.
12. Fatigue loading was not considered as low traffic was assumed, but has to be accounted for as the traffic gets larger.

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APPENDIX A

STEEL BRIDGES INFORMATION

APPENDIX B

IMPORTED TEMPORARY STEEL BRIDGES COST AND LOCAL COST ESTIMATE

Declaration

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

Name: Tihitina Siyoum

Signature:

Place: Addis Ababa University

Date of Submission: October 2007

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List of Symbols

a= width
A = cross-sectional area
A_{bm} = cross sectional area of the base material
A_{eff} = effective area
A_v = the shear area
A_w = cross sectional area of the weld
b = length
b = overall width
BR = vehicular braking force
C = outstand
CE = vehicular centrifugal force
C_{eff} = effective outstand

CR = creep
C_t = torsion constant
CT = vehicular collision force
C_y = distance from the centroid to the extreme fiber
d = diameter of pin
d = distance between two axes
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
FBC = Flexural boundary conditions.
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
FR = friction
$F_{V,RD}$ = specified minimum ultimate strength of pin
F_w = nominal strength of the weld electrode
f_y = yield strength
h = overall depth
i = radius of gyration about the relevant axis
I = second moment of inertia
I_{comb} = Equivalent stiffness
IM = vehicular dynamic load allowance
I_t = torsion constant
I_{xx} = second moment of inertia about the major axis
KL = effective length of the truss member
KL/r = the slenderness ratio
k_σ = buckling factor corresponding to stress ratio
L = length
L = actual unbraced length of the truss member

L = buckling length of a compression member.
LL = vehicular live load
LS = live load surcharge
M = bending moment due to the applied loads
MBC = Membrane boundary conditions.
MD = design moment
M_{max} = Maximum tensile moment on beam
M_{min} = Maximum compressive moment on beam
M_{RD} = moment capacity of the pin
M_{sd} = the design bending moment
$N_{b,Rd}$ = design buckling resistance of a compression member
$N_{com,Rd}$ = design compression resistance of the cross section
$N_{com,Sd}$ = design value of the compressive force
$N_{pl,Rd}$ = design plastic resistance of the gross section
$N_{t,Rd}$ = design tension resistance capacity of the cross section
$N_{t,sd}$ = design value of the axial tension force at each cross section
$N_{u,Rd}$ = design ultimate resistance of cross section at the bolt hole
p = Truck loading
P = transverse loading
P = Truck loading uniformly distributed over the patch area
P_1 = rear truck axle load
P_2 = front truck axle load
P_3= tandem axle load
PL = pedestrian live load
q = uniformly distributed loading
Q = non-dimensional parameter for uniformly distributed loading
Q_i = force effect
r = radius of gyration of the truss member cross section
R_f = factored resistance
R_n = nominal resistance

S = average stringer spacing
S = maximum total tensile stress in the most stressed chord member
s = allowable tensile stress
S = elastic section modulus
SE = settlement
SH = shrinkage
S_{xx} = section modulus about the major axis
t = plate thickness
t_f = thickness of flange
TG = temperature gradient
T_{max} = Maximum torsion on beam
TU = uniform temperature
t_w = thickness of web.
u = width of the patch area
v = length of the patch area
V_{max} = Maximum shear on beam
$V_{pl, Rd}$ = the design plastic shear resistance
V_{Sd} = the design shear force
w = out of plate deflections

w = clear roadway width between curbs and/or barriers
W = total weight of the bridge truss including its bracing
W = non-dimensional parameter for transverse deflection
WA =water load and stream pressure
W_{el} = elastic section modulus
WL = wind on live load
W_{pl} = the required section design plastic resistance
WS = wind load on structure
Y = ordinate to the centroid of the total area
Y_1 = ordinate to the centroid of the component area
Y_2 = ordinate to the centroid of the component area
β_A = factor
γ = load factor for the limit state under consideration
γ_i = load factor: a statistically based multiplier applied to force effects
γ_{mo} = partial safety factor for class 1 cross-section
γ_{m1} = partial safety factor for class 4 cross-section
γ_p = load factor for permanent loading
γ_{SE} = load factor for settlement
γ_{TG} = load factor for temperature gradient
δ_{all} = allowable deflection
Δ = Maximum deflection
ΔA_f = area that should be ignored from the two flanges
ΔA_w = area that should be ignored from the two webs
ε = factor
η_i = load modifier: a factor relating to ductility, redundancy, and operational importance

η_D = a factor relating to ductility, as specified below
η_R = a factor relating to redundancy as specified below
η_I = a factor relating to operational importance as specified below
λ = the slenderness ratio
$\bar{\lambda}$ = non dimensional slenderness
λ_1 = slenderness
$\bar{\lambda}_p$ = non dimensional slenderness corresponding to appropriate width
ρ = reduction factor
σ_e = equivalent uniaxial state of stress
$\sigma_{emax.}$ = maximum equivalent surface stress
$\bar{\sigma}_{e\,max}$ = non-dimensional parameter related to the actual maximum equivalent stress
σ_{max} = Maximum principal stress on plate element

σ_{min} = minimum principal stress on plate element
σ_x = stress in the x direction
σ_{xy} = shear stress
σ_y = stress in the Y direction
φ = resistance factor: a statistically based multiplier applied to nominal resistance
ϕ = resistance factor
χ = reduction factor for buckling

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Abstract

The level of development of the transportation network is an indication of the economic development of a country. This transportation network consists of urban and rural road networks. Rivers and mountains form physical barriers creating gaps in the network. This necessitates the construction of bridges to form a link between the highways at obstacles.

Ethiopia being a mountainous country has numerous rivers and ravines, thus requiring construction of many bridges for rural communication. Conventional construction of bridges using reinforced concrete will be very expensive and sometimes impractical at remote places. Construction of steel truss bridge made by assembling panels of smaller units of trusses will be a practical solution for such situations.

Moreover, in Ethiopia there are many locations that have bridges in deteriorated conditions and require immediate replacement. Regional Roads Authorities and Ethiopian Roads Authority require replacing failed bridges on the same bridge if a failure of the bridge superstructure only occurs or build a new one. This usually takes time. Therefore, to minimize the disruption of traffic flow detours are constructed and temporary steel bridges are used to cross the obstacle, usually a river, until a replacement or a new bridge is in place. These temporary bridges are of ‘through’ type steel truss bridges with the road way being carried between two main trusses. These units can easily be handled and transported by trucks. They can be assembled and erected within a few days and these enable the quick restoration of traffic. After the bridge is rebuilt the temporary steel bridge on detour will be disassembled. These temporary steel bridges are occasionally being used as permanent superstructure in some regions. The main disadvantage in the use of these temporary steel bridges is that they are imported at high cost.

This problem calls for the design and production of assembled steel bridges with locally available steel profiles. Thus this research is intended to prepare analysis and design procedures that may serve as a reference for designers in the design of assembled steel bridge with locally available steel profiles.

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Appendix A

Steel Bridges Information

Appendix B

Imported Temporary Steel Bridges Cost