ALTERNATIVE TRUSS DESIGN FOR FOOTBRIDGES

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Declaration by the candidate

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Dedication

"To my family and friends"

ABSTRACT

Footbridges are constructions that are needed where a pathway has to be provided for people to cross some physical obstacle like a river or to cross traffic flows. They can be of various types e.g. beam, arch, suspension, truss and cable-stayed. From the various types of bridges, the focus was on steel truss bridges. The research sought to address the problem of over-design in classical trusses which can be solved through, though still far from ideal, varying the cross section area for different truss intervals. Instead of using single elements to connect truss nodes, multiple members of uniform cross sections were used in parallel so that variation in area is achieved through use of a number of members that make up the area. The objective of the research was to make a catalogue of trusses for footbridges of three spans and two widths. The trusses were intended to approach a fully stressed design as much as possible by varying the number of members in parallel from two to five. For the diagonal elements, only two members were used. The members were connected by horizontal beams. Distances between members on either side of the beam were determined such that these members had equal forces and the joint was in equilibrium. The bridge deck was a frame consisting of C-sections with an overall dimension of 2.44 m by 0.61 m and 3 mm tear plate surfacing welded on it. Two deck widths of 1.22 m and 1.83 m were considered. A number of trusses were analysed from which optimal ones were selected. The design for all trusses and connections were done according to the Eurocodes. The result obtained was a catalogue of trusses for different lengths of footbridges, connection details for the truss members, drawings and details. Three dimensional drawings were also produced to show the completely assembled footbridge. To conclude, a truss consisting of members in parallel is advantageous for slender trusses where it can save up to 21.3% and 26.4% by mass of steel for 1.83 m wide and 1.22 m wide footbridges, respectively when compared to the classical design. It can achieve 75% fully stressed design with most members being utilized to greater than 70% capacity. In addition, it is aesthetic and ease of assembly and disassembly is ensured by use of bolted connections. Also, elements can be replaced while the footbridge is still in place. Use of cross sections of different wall thickness is recommended in order to save more weight. Another recommendation is to galvanise the components of the footbridge to protect them against rusting. Finally, further research should be done on the design of a truss consisting of plates with different thicknesses for the elements to achieve as close as possible a fully stressed design. In this case compression elements will have stiffener ribs.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

$A_{ref,x}$	Reference area in the x-direction [mm^2]
A_s	Tensile stress area of the bolt [mm^2]
A_{nt}	Net area subjected to tension [mm^2]
A_{nv}	Net area subjected to shear [mm^2]
C	Wind load factor [-]
d	Bolt diameter [mm]
E	Young's Modulus [N/mm^2]
F	Member force [N]
$F_{t,Rd}$	Tension resistance for ordinary bolts [N]
$F_{v,Rd}$	Shear resistance per shear plane for ordinary bolts [N]
$F_{b,Rd}$	Bearing resistance for ordinary bolts [N]
F_W	Wind force in the x-direction [kN/m]
f_{ub}	Ultimate tensile strength of a bolt [N/mm^2]
f_u	Ultimate tensile strength of connected parts [N/mm^2]
f_y	Yield strength of connected parts [N/mm^2]
$G_{k,j}$	Permanent action(s).
I	Moment of inertia [m^4]
k	Effective length factor of element whose value depends on the end support conditions [-]
k_1, k_2	Reduction factor for bolts [-]
L	Member length [m]
N_{cr}	Euler buckling load [N]
$Q_{k,i}$	Variable action(s).
q_{fk}	Characteristic uniformly distributed traffic load [kN/m^2]
t	Thickness of connected parts [mm]
v_b	Basic wind speed [m/s]
$V_{eff,1,Rd}$	Design block tearing resistance [N]
α_v	Reduction factor depending on the bolt class [-]
α_b	Reduction factor for end bolts [-]
β	Effective buckling length factor [-]
β_w	Correlation factor for welded connections [-]
γ_G	Partial safety factor for dead loads [-]
γ_Q	Partial safety factor for variable loads [-]
γ_{M0}	Partial safety factor for material strength [-]
γ_{M2}	Partial safety factor for resistance of bolts [-]
$\psi_{0,i}$	Load combination factors [-]
$\tau_{\parallel}, \sigma_{\perp}, \tau_{\perp}$	Stresses in the throat of the weld [N/mm^2]

σ_c Combined stresses (Von Misses stress) [N/mm^2]

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CHAPTER ONE

1.0 INTRODUCTION

1.1 An overview of types of bridges

Bridges are part of a country's infrastructure. They provide passage over physical obstacles such as traffic flows, gorges, valleys, rivers and other water bodies. There are various kinds of bridges which differ in their load channelling mechanisms. For example, cable stay bridges rely on tension in the cables to channel loads to the ground. Beam bridges carry load through flexure while truss bridges consist of members resisting axial loads (Ressler, 2011). An overview of various types of bridges is given in this section.

1.1.1 Beam Bridges

A beam is the simplest kind of a bridge. In its basic form, a beam bridge consists of a horizontal beam that is supported at each end by piers, Figure 1.1. The beam needs to be stiff enough, have sufficient moment of inertia and torsional stiffness, to resist bending and twisting under load. As the length of the beam increases, it gets "weaker". In order to span great distances, a series of beam bridges are normally joined together to create a continuous span. The beams can be made from reinforced concrete or steel to form large box girders. Timber bridges may consist of a log across a stream. Two logs or timber beams across the stream may be employed and a deck consisting of timber planks.

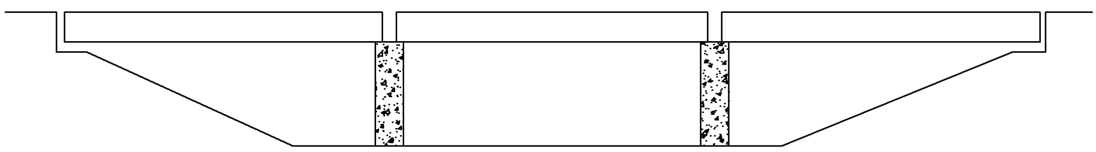

Figure 1.1: Beam Bridge.

1.1.2 Arch bridges

They are among the oldest types of bridge constructions. In the past, they were built from stone or brick. In the present days, they are built from concrete or steel. Arches carry loads entirely in compression. The compressive force is pushed outward along the curve of the arch bridge towards the abutments. Figure 1.2 shows some arch bridge configurations.

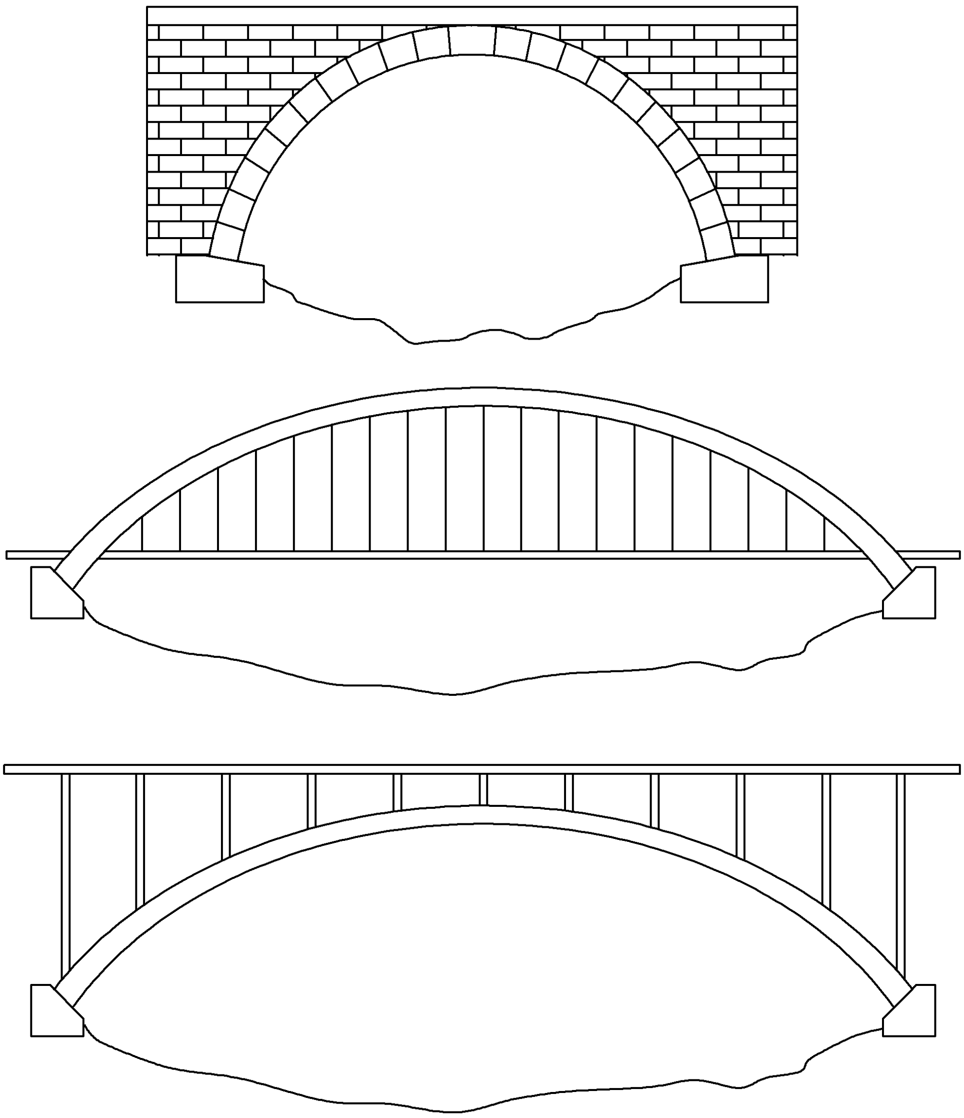

Figure 1.2: Arch bridge configurations.

Another type of arch bridge is the tied arch bridge. In this type of bridge, the horizontal outward forces of the arch are resisted by bottom chords. The bottom chords could either be tie-rods or the deck of the bridge. This kind of construction is advantageous in areas of unstable soil since only vertical loads are transferred to the ground.

1.1.3 Suspension bridges

Suspension bridges, Figure 1.3, consist of a bridge deck that is stiffened by trusses or stiffening girders. The trusses are suspended by vertical cables which transmit loads to two or more main cables. The main cables are the principal load-carrying elements of a suspension bridge and are held aloft by towers. The towers transmit the loads to the foundation. To maintain tension in the main cables, each of them must be anchored to the earth at the end. All suspension bridges are susceptible to vibrations due to wind because they are relatively light and flexible. A solution to the problems is deck stiffening. The 1940 Tacoma Narrows Bridge is a famous example of aerodynamic instability in a suspension bridge.

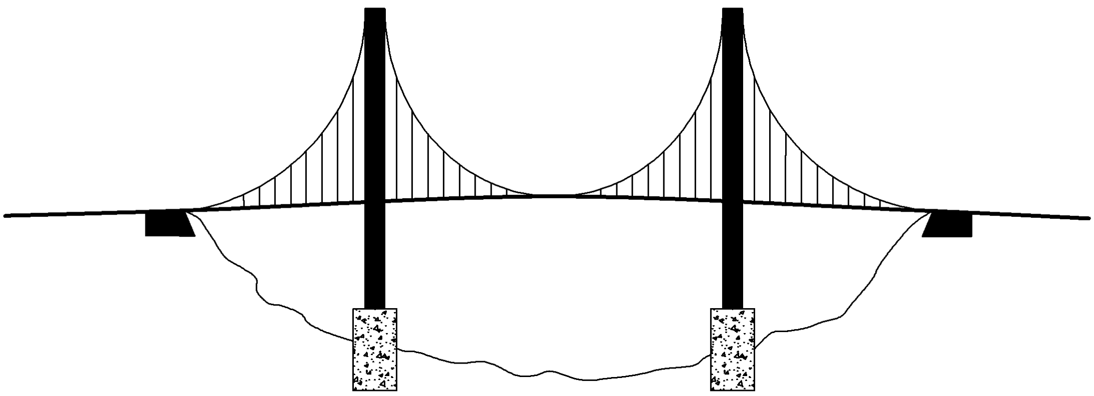

Figure 1.3: Suspension bridge.

1.1.4 Cable-stayed bridges

This kind of bridges have one or more towers from which cables are connected to support the bridge deck. Cable-stayed bridges can be classified as fan or the harp type, see Figure 1.4. The classification is based on how the cables are attached to the towers. The primary load bearing structure in a cable-stayed bridge are the towers. These towers transmit bridge loads to the foundation. Unlike suspension bridges, cable-stayed bridges do not require anchorages to resist the horizontal pull of the cables.

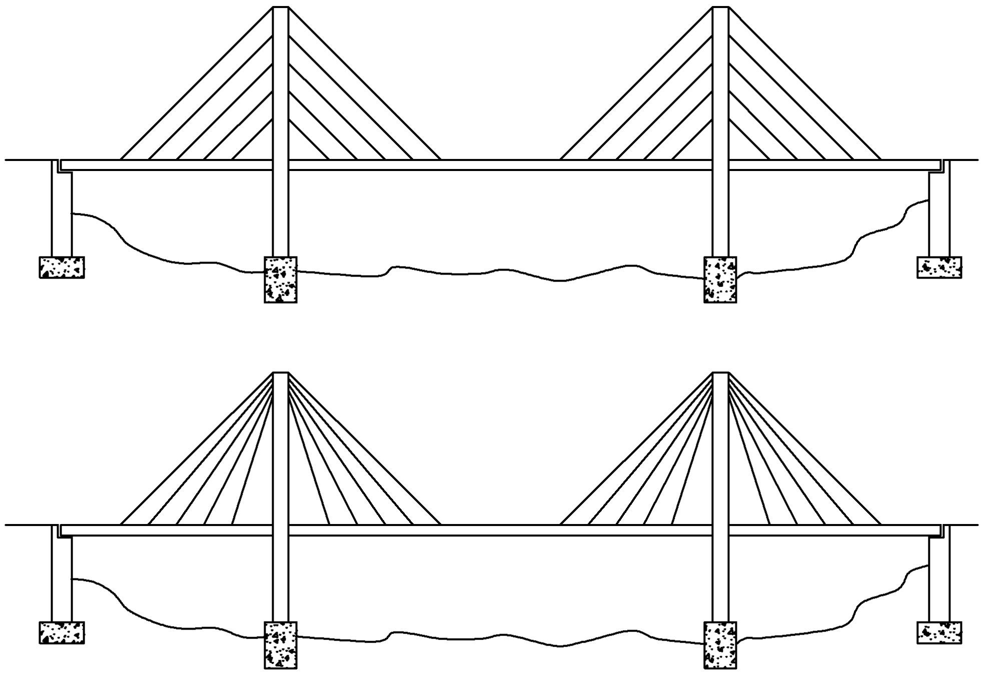

Figure 1.4: Cable stay bridges (fan and harp types).

1.1.5 Truss bridges

The loads in this kind of bridges are resisted by a truss. A truss is a rigid framework composed of members connected at joints and arranged into a network of triangles. The triangle is a stable structural configuration and is the source of a trusses' structural rigidity. The connected elements may be stressed in tension, compression or sometimes both in response to dynamic loads. Trusses are strong, stiff and light. Some trusses are shown in Figure 1.5.

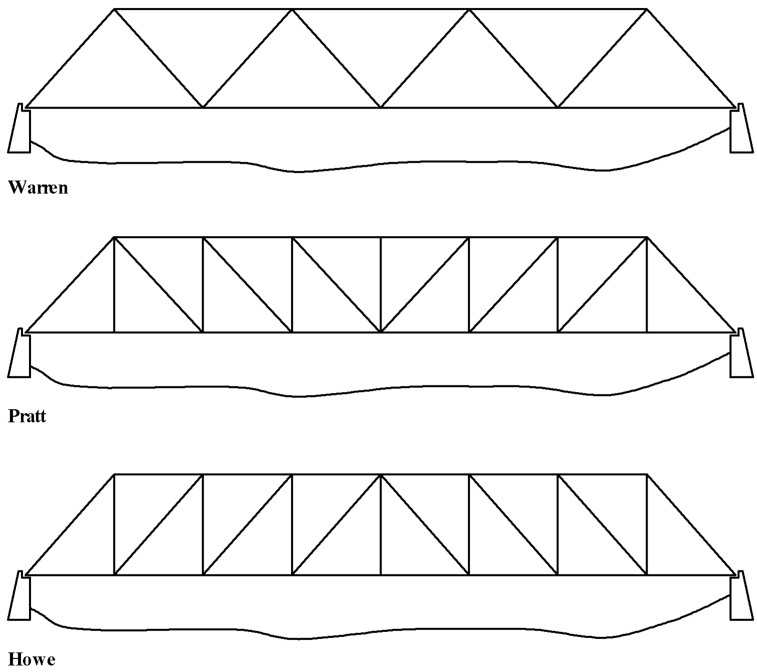

Figure 1.5: Warren, Pratt and Howe trusses.

1.1.6 Cantilever bridges

Cantilevers are structures that project horizontally into space and are supported only on one end. Therefore, cantilever bridges are constructed using cantilevers. Cantilever bridges can consist of trusses built from steel (Figure 1.6) or box girders built from pre-stressed concrete (Figure 1.7). The bridge in Figure 1.7 was cantilevered during the construction phase.

1.1.7 Pre-stressed segmented Arch Bridges

Figure 1.8 shows a pre-stressed segmented arch bridge built by university students using small diameter timber. The small diameter timber has its natural growth-ring structure preserved for strength and predictability. The joints are the bridge sockets where the rounded ends of the small diameter timber slide in Ibrahim, 2008.

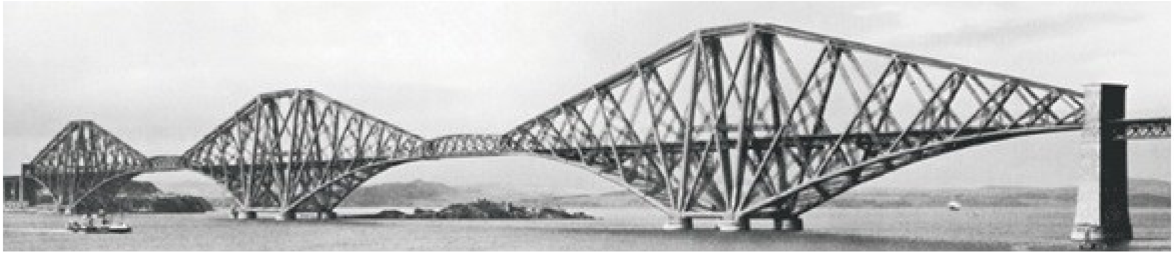

Figure 1.6: Forth Bridge[1].

Figure 1.7: Raftsundet Bridge under construction[2].

Figure 1.8: Light pre-stressed segmented arch bridge.

[1] http://upload.wikimedia.org/wikipedia/commons/9/98/Forth_bridge_histo_2.jpg
[2] http://structurae.net

1.2 Steel truss bridges

1.2.1 An overview of different types of truss bridges.

There are very many different kinds of truss configurations that are used for steel truss bridges. They were invented and used in ancient bridges and are still being applied today. Some of the trusses will be highlighted in this section.

1.2.1.1 Warren, Howe and Pratt truss Bridges

These trusses are shown in Figure 1.5. The warren truss consists of members arranged in such a way that they form alternately inverted equilateral triangles. Since all members can be made of equal length, it is favourable to use in prefabricated modular bridges. The Howe truss on the other hand consists of vertical members and diagonals that slope up towards the centre of the truss. This is the opposite of the Pratt truss. Some examples of footbridges using the Warren, Howe and Pratt trusses are shown in Figure 1.9, Figure 1.10 and Figure 1.11, respectively.

Figure 1.9: Sainte-Tulle Bridge in France [3](Warren Truss).

[3] http://files1.structurae.de/files/photos/1/saintetulle04.jpg

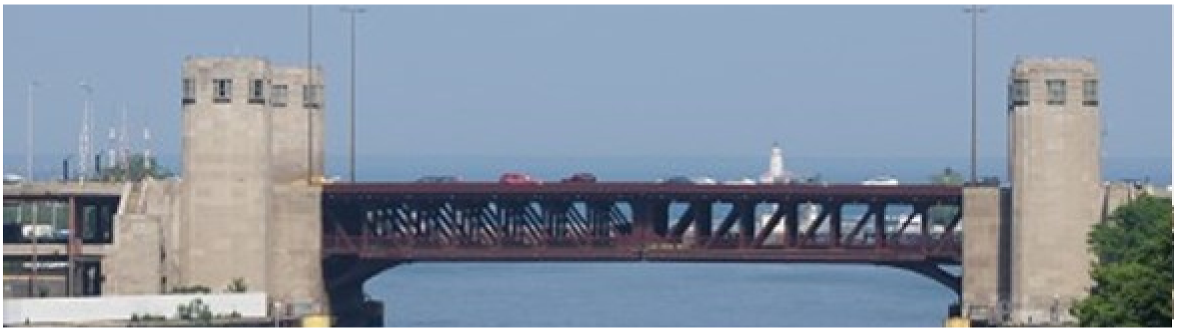

Figure 1.10: Lakeshore drive bridge in Chicago [4](Pratt Truss).

Figure 1.11: Howe Truss Bridge[5].

1.2.1.2 Parker (Camelback) truss

The Parker bridge truss is a modification of the Pratt truss design such that its upper chords form a polygon. A subset of the Parker truss is the 'camelback' truss. The upper chords of the 'camelback' truss consist of five segments. An example is the Woosley Bridge in Arkansas, Figure 1.12.

[4] http://files1.structurae.de/files/photos/1/20100524/dsc03189_shift.jpg
[5] https://www.alamy.com

Figure 1.12: Woosley Bridge in Arkansas[6].

1.2.1.3 Bailey Bridges

They were originally designed for military use. They consisted of prefabricated and standardized truss elements which could be combined in different configurations according to the span and load requirements of a given situation. This bridge required no special tools or heavy equipment to assemble. An example is the single story Bailey bridge over the Meurthe River in France, which has two trusses in parallel at each side, Figure 1.13.

Figure 1.13: Bailey bridge over the Meurthe River in France[7]

[6]http://1.bp.blogspot.com/_em7n1HMqRVA/TOMmRruJMnI/AAAAAAAAAtc/F8enA79KSdM/s1600/woolsey33.jpg

[7] http://upload.wikimedia.org/wikipedia/commons/thumb/b/be/PontBailey.jpg/240px-PontBailey.jpg

1.2.1.4 Bowstring arch truss bridge

The Bowstring arch truss is very similar to a tied arch bridge but has diagonal load bearing members. This truss resembles an archer's bow with the top chords forming the shape of the bow. The bottom chords are stretched in tension like the string in the archer's bow while top chords resist compressive forces. Figure 1.14 shows a bowstring truss bridge in Rochester Minnesota.

Figure 1.14: Bowstring truss bridge in Rochester Minnesota[8]

1.2.1.5 Cantilevered truss bridge

Most trusses exhibit a behaviour similar to that of a simply supported beam. This means that the top chords are normally in compression while the bottom chords are in tension. However, in a cantilevered truss, the situation is reversed over some portions of the span. The "balanced cantilever" is a typical cantilevered truss bridge and a good example is the Forth Bridge (Figure 1.6). Movable bridges are also often of the cantilevered type, for example the Wells Street Bridge (see Figure 1.15).

[8] http://www.wheeler-con.com/wp-content/gallery/rec_steel_bowstring/bn_t10415_5lg.jpg

Figure 1.15: The Wells Street Bridge[9].

1.2.1.6 Lenticular Truss bridges

They consist of trusses whose shape resemble that of a lens. They were very common in Europe and were introduced to America in the 19th century. Most of the initial lenticular truss bridges were constructed using iron. Figure 1.16 shows a lenticular truss bridge constructed using steel.

Figure 1.16: Bardena river footbridge in Italy[10]

[9] http://theinfrastructureshow.com
[10] http://files1.structurae.de/files/photos/1791/mvc_004f.jpg

1.2.1.7 Lattice truss bridge

A lattice truss consist of small closely spaced diagonal elements forming a lattice, Figure 1.17. This kind of bridge can be constructed using a number of relatively light iron or steel members.

Figure 1.17: Nordsternpark Lattice Truss Bridge, Germany[11].

1.2.2 Truss Design and Analysis methods

Truss analysis is done in order to know what loads are carried by each member of the truss. Knowing the loads carried by each member enables the engineer to design the members and joints of the truss. Before the advent of computers analytical methods were used to perform analysis of structures by early engineers and are still taught in undergraduate programs today. The method of joints, method of sections and graphical methods were some of the examples that were used in the past. These methods could easily be applied to analyse statically determinate trusses. However, by making suitable approximations, a statically indeterminate truss can be reduced to a determinate truss in order use these methods.

1.2.2.1 Method of joints

The method of joints was developed by Squire Whipple (Ressler, 2011). In the method of joints, if a truss is in equilibrium, then each of the joints must be in equilibrium. This

[11] http://files1.structurae.de/files/photos/1/20120907/DSC04287.JPG

method involves satisfying the equilibrium equations for forces in both horizontal (x) direction and vertical (y) direction acting on each of the joints, $Eq.$(1.1) (Hibbeler, 2012).

$$\sum F_x = 0 \ and \sum F_y = 0 \tag{1.1}$$

The first step in the method of joints is to determine the support reactions of the truss. Starting from a joint where only two unknown reactions exist, for a two dimensional truss, a free body diagram is drawn. The unknowns are then determined by applying the equilibrium equations or by use of the polygon of forces.

1.2.2.2 Method of sections

This method is used when forces in few members of a truss are to be determined. It involves cutting the truss into sections by an imaginary line. If the entire truss is in equilibrium, then each imaginary section must be in equilibrium (Hibbeler, 2012). This is expressed mathematically by $Eq.$ (1.2)

$$\sum F_x = 0\,; \ \sum F_y = 0\,; \ \sum M_z = 0 \tag{1.2}$$

where x and y are the horizontal and vertical directions in the plane of the truss and z is the horizontal direction perpendicular to the plane of the truss.

1.2.2.3 Graphical method

The Cremona-Maxwell method is a graphical method that is used in statics of trusses to determine internal forces in the members. The method combines all the force polygons necessary to analyse a statically determinate truss. It was originally developed by Prof. James Clerk Maxwell and was later extended by Prof. Cremona.

1.2.2.4 Numerical method

It is very cumbersome to do joint-by-joint free-body analysis of trusses for large and complex structures and especially for statically indeterminate structures which require that displacement compatibility be taken into account in addition to static equilibrium. The Matrix Method of structural analysis, a forerunner of the Finite Element Method, enables quick solution of trusses regardless of the static indeterminacy. The Finite Element Method is a more systematic and generalized method that has become popular in engineering analysis (Chennakesava, 2008; Harry, 1989). This method has been implemented in a number of computer software applications used in industry. Some examples of software applying this method are: Autodesk Robot Structure, SCIA Engineer, Prokon, CSC Orion, DIANA and Ansys.

1.2.3 General remarks on structural optimization

In mechanics, a structure consists of an assembly of members for the purpose of resisting loads. To optimize or optimization means "making things the best". Structural optimization is therefore the subject of making a structure resist loads in the "best" way possible. The "best" may refer to making the structure as light as possible i.e. minimizing the weight, make it as stiff as possible or making it less sensitive to buckling or instability as much as possible (Peter & Anders, 2009; Thomas, et al., 2006). In order to perform such optimizations, constraints are necessary e.g. limitation on the amount of material. Quantities such as stresses, displacements and geometry are normally constrains in structural optimization problems. These constraints may also be used as measures of performance of the structure (Peter & Anders, 2009). A lot of research has also been done on structural optimization of trusses using morphological indicators (Samyn, 1999; Latteur, 2000; De Wilde, 2007; Thomas et al., 2006) and genetic algorithms (Hultman, 2010).

1.3 Footbridges

Footbridges are constructed to provide passage of pedestrians (and loaded donkeys in rural areas) over traffic flows and rivers. The loads carried by footbridges, compared to highway or railway bridges, are quite modest and therefore in most circumstances a fairly light structure is normally required. For long and clear spans, the stiffness of the footbridge is an important consideration (Corus Construction, 2005). Since bridges are often in the view of the public, their appearance is a factor to consider.

1.3.1 General information

Footbridges have the main purpose of shortening a route from one place to another whether crossing rivers, streets or valleys. Footbridges can also be symbolic for example to mark the beginning of a new era like the millennium bridge (Figure 1.18) in London or ornamental, to enchant or delight the eye (Ursula and Mike, 2008).

Figure 1.18: The millennium bridge in London (Ursula et al., 2008).

In the 19th century, advances in transport technology influenced bridge building and high standards were set for rail and road bridges. Footbridges were affected indirectly by the technological changes, therefore their development took a different course. For railway bridges, engineers had to start dealing with faster moving trains. They also had to deal with wider roads for road bridges. In the case of footbridges, the human being remained a constant factor in the equation. Technical progress, imagination and functional variety in the case of footbridges is open to other influences which bring forth an inexhaustible variety of distinctive designs (Ursula and Mike, 2008).

Since people react differently to oscillations and heights, a footbridge construction should be structurally sound, easy to maintain and "cheap". Lighting design also does play an important role for pedestrians to experience night time illumination (Ursula and Mike, 2008).

The engineer has more freedom in designing footbridges than rail and road bridges. There is a lot of freedom in choosing the geometry of the bridge deck for example by making it curved, see Figure 1.20. Spatial experience may be achieved by the suspension of the deck, by a movable bridge (Figure 1.21), or by the intersection of multiple pathways. The gradient of the deck may be relatively freely chosen which opens up new possibilities for emphasizing the spatial geometry of the footbridge. For example walkable arches, stress ribbon bridges are possible alternatives (see Figure 1.19). Gradients greater than 6 % present problems for wheel chair users. The potential energy required to overcome the slope presents a problem. Alternative pathways must be offered for wheelchair users where steep deck gradients or stairways exist (Ursula and Mike, 2008).

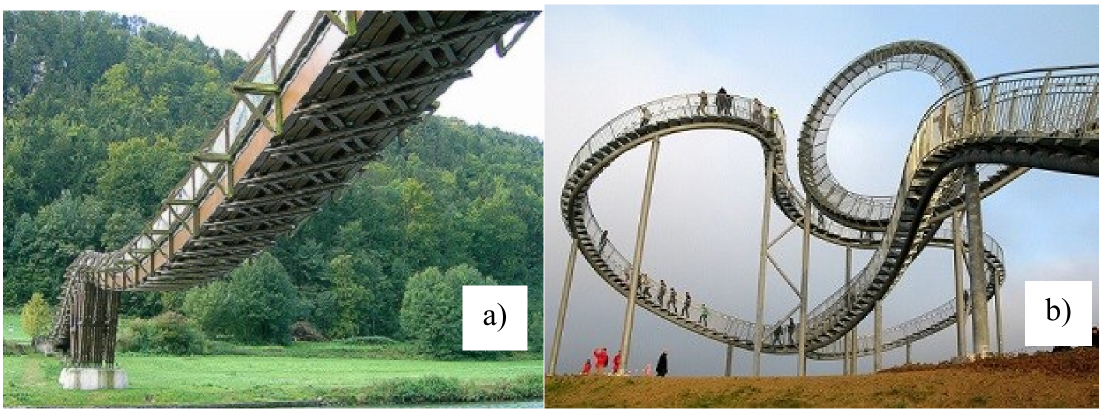

Figure 1.19: a) Stress ribbon pedestrian bridge new Essing[12]and b) Germany's walkable roller coaster[13]

Figure 1.20: West Park Bridge in Bochum, Germany, 2003(Ursula and Mike, 2008).

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12 https://en.wikipedia.org/wiki/Stressed_ribbon_bridge#/media/File:Holzbr%C3%BCcke_bei_Essing_1.jpg

13 whenonearth.net/germanys-walkable-rollercoaster/

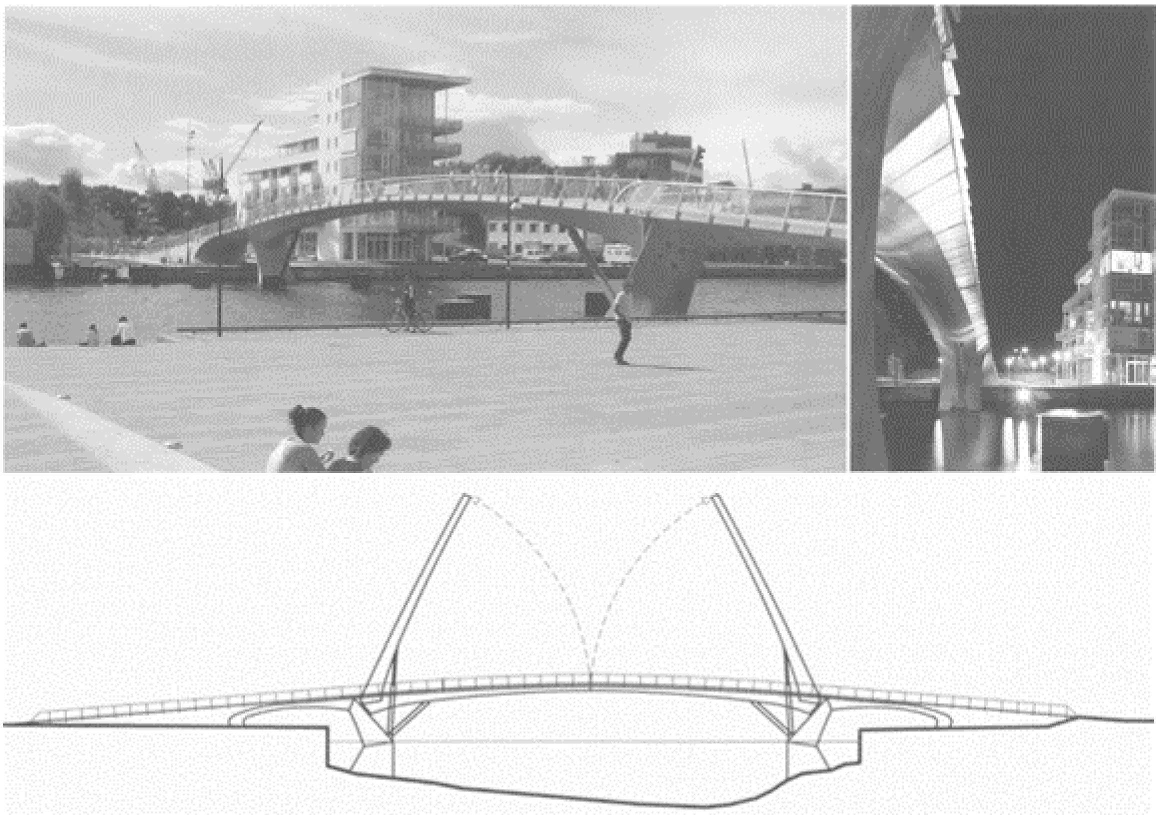

Figure 1.21: Moveable Bridge in Fredikstad, Norway, 2006 (Ursula and Mike, 2008).

Most footbridges have narrow deck widths of between 3 and 4 meters. The rule of thumb is that 30 pedestrians per minute for every meter of deck can cross the bridge without impeding one another. A minimum deck width of 2 m for bridges open to pedestrian and cycle traffic is recommended (NZ Transport Agency, 2013). With the stated pedestrian densities, the Eurocode prescribes a pedestrian live load of 5 kN/m^2 which roughly is equal to the loading of the main road of a roadway bridge. This loading may be reduced, in some countries, for longer bridges as statistics show that such a crowding (equivalent to 6 people per square meter) is improbable on a long bridge deck. Since pedestrians are less sensitive to deflections than road or railway traffic, footbridges may be much more slender and light weight than road or rail bridges. For this reason, footbridges are often lively and dynamic analyses of the structure should be carried out in the early phases of the design (Ursula and Mike, 2008).

1.3.2 Footbridges in Kenya

In Kenya, there is a need for small footbridges to cross small seasonal rivers which change course continuously. The width of the rivers range from 3 m to 15 m. These rivers act as barriers cutting off communities from health facilities, markets, schools and business. Examples of such rivers are the "Maoni" river separating Darajani area and Kambu area in Eastern Province and the "Awach" river in Nyanza separating Kodhoch and Landa areas. For over 20 years there has been efforts to put up footbridges in the rural areas using poles and timber. However, this kind of constructions do not last long (Pyl and Sitters, 2012).

On the other hand there are quite a number of footbridges in Nairobi, which is the capital city of Kenya. These footbridges are mainly for providing passage over fast moving traffic. A number of footbridges were erected along Thika road, Mombasa road, University way and at Uthiru to provide safe crossing for pedestrians (Figure 1.22). The footbridges along Thika road and Mombasa road have steel trusses and concrete decks while the one at Uthiru and University way are made of concrete.

1.3.3 Geometry of the truss considered for the footbridge

There are very many types of trusses that can be used for a footbridge. However, this research is confined to the parallel trusses and particularly of the Warren type. Parallel trusses have the top and bottom chords parallel to each other and could be of the Warren, Howe, or Pratt type. From previous research, it was found that the Warren type truss was the optimal one of the three. *Pyl and Sitters (2012)* used morphological indicators at the preliminary design stage for optimization of trusses for a footbridge. They found that the warren truss was suitable for the design of their footbridge. *Koumar (2012)* also applied morphological indicators and found the Warren truss to be the optimal one for the design of a footbridge.

Figure 1.22: Some Footbridges in Kenya. a) and c) are footbridges across a river in Meru Town. b) and d) are footbridges across Thika road and Mombasa road respectively to provide safe passage over fast moving traffic. e) footbridge at Uthiru and f) at University way

1.3.4 Classical truss and alternative truss

The *Classical truss* refers to the typical truss, which is a structure consisting of members arranged in a triangular shape, Figure 1.23. The members are assumed to be connected to each other by frictionless joints. Real trusses have stiff joints due to welding or bolting of the members together. A frictionless joint model is still accurate even with the stiffness in the joints if the centre of gravity axes of each member meet at the nodes avoiding secondary or parasitic moments. Using this approach only one member is used between the nodes.

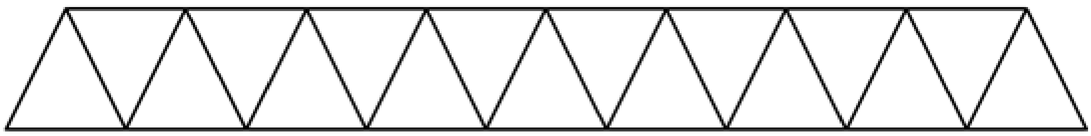

Figure 1.23: Warren truss (classic truss)

The *alternative truss* also has members arranged in a triangular shape. Multiple members connected by joint beams are used between each node making it three dimensional as shown in Figure 1.24.

Figure 1.24: Alternative truss

To design a truss consisting of members in parallel presents a challenge of modelling. In steel, buckling instability is of great concern. The concept of buckling was applied in determination of number of members in parallel. In addition, some computer programming was necessary to generate truss models.

From a large number of trusses, optimal trusses were selected to form a catalogue. Trusses and joints for these trusses were designed according to the Eurocodes. Also, drawings of the connection details were made to guide in fabrication and construction.

1.4 Research statement

In the classical design approach, the member cross-section is commonly determined by the heaviest loaded member. This means that most of the members are overdesigned. Some weight saving can normally be achieved by introducing members of say two different cross-sections. But the situation is still far from ideal.

The best situation is to have a fully stressed design. This means that all members are loaded to the allowable stress of the material either in tension or in compression (and taking buckling into account). The disadvantage of this approach is that many members with different cross-sections have to be fabricated, which is not economical. On the other hand, from an assembly point of view commercially available members of the same cross-sections are preferred.

The challenge of this research is to unite these two conflicting statements and to come up with a design that is (almost) fully stressed and also uses commercially available members with a small variety of cross-sections. This is achieved by a configuration whereby the member is formed by a number of (almost) equally loaded members in parallel.

1.5 Objectives

The main objective of this research was to produce a design catalogue for footbridges of three spans and two deck widths, utilising Warren trusses having parallel elements in which a fully stressed design is approached using commercially available members.

From the catalogue it will be possible to select footbridges for three different spans (or lengths) and a corresponding width (either 1.83 m or 1.22 m wide footbridge).

The specific objectives are to:

1. Do analysis of the loading on the footbridge according to Eurocodes.
2. Design the bridge deck and supporting beams (stringers).
3. Design the trusses including joint beams with members composed of a number of elements in parallel and compare the results with the classical design.
4. Produce a catalogue from which span, width and truss of the footbridge can be selected.

1.6 Research Methodology

1. Analysis of loading on the footbridge according to Eurocodes

Eurocode 1 *(EC1 or EN 1991)* deals with the basis of structural design. The actions on the footbridge are obtained from Eurocode 1 as summarized in Table 1.1.

Table 1.1: Eurocode 1 references for actions on a footbridge.

Action on footbridge	Reference
Self-weight	EN 1991-1-1
Traffic actions	EN 1991-2
Wind actions	EN 1991-1-4

Load combinations and safety factors are obtained from Eurocode 0 *(EC0 or EN 1990)* which deals with the basis of structural design. See Table 1.2.

Table 1.2: Eurocode 0 references for load combinations and safety factors

Parameter	Reference
Load combination factors	EN 1990
Safety factors	EN 1990-A2

These loads and load combinations will be applied in a software *(SCIA Engineer)* for analysis of the footbridge.

2. Design of Bridge Deck and supporting beams (stringers)

The bridge deck will be a frame consisting of C-sections on which a tear plate will be welded, Figure 1.25. I – sections that will be connected at the nodes of the truss will be used as supporting beams (or cross-beams) on which the deck will rest.

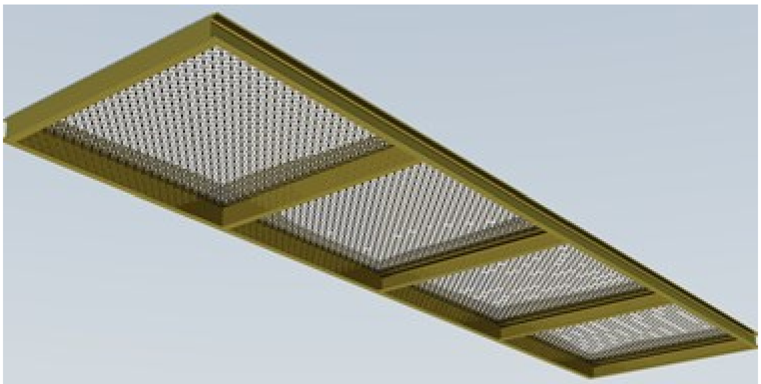

Figure 1.25: The bridge deck

3. Design of trusses with parallel members and comparison with the classic design

The challenge of the research is to replace a single member in a Warren truss with a minimum of two and up to a maximum of five members. At the preliminary design stage the Eurocodes are not used. The theoretical Euler buckling load and the theoretical load at yield in tension are used to simplify the determination of number of elements in parallel. The Euler buckling load will be applied for members in compression to compute the moment of inertia of a section. On the other hand, for members under tension, the tensile stress will be applied to obtain the area of a section. The idea is to obtain the member with the highest load (for both tension and compression) in a classical truss. Using a fifth of this load, a standard member can be obtained to be used in multiples. Having obtained the number of members in parallel, the trusses (both the alternative and classical) will then be modelled and analysed using SCIA Engineer. Using the same software, the design of the members will be carried out according to the Eurocodes. Also, comparisons are made on the basis of the weight of the trusses. In addition to carrying out a comparison by weight, a comparison by fully stressed classical trusses is also done. Finally, a spreadsheet programme will be used to design the joints of the selected trusses according to the Eurocodes.

4. Catalogue of trusses

The catalogue of trusses will contain trusses for the different footbridges that are most economical from weight and therefore cost point of view. Tables, drawings and details of the trusses and joints will be provided from which a truss of a selected span and width for a footbridge can be obtained.

1.7 Justification and scope of the research

Classical trusses have a number of members that are overdesigned and even with the use of different cross sections. An alternative approach based on the use of members in parallel is used to try and achieve as close as possible a fully stressed design and compared to the classical approach. Analysis and design of the trusses in the thesis are done according to the Eurocodes using SCIA Engineer software. The Eurocodes are used because the British Standards, which are used in Kenya, are no longer being updated. This means that in the near future there shall be a shift to the Eurocodes in Kenya.

1.8 Structure of the thesis

After the introduction in chapter 1, a review of literature and some background information on structural optimization of trusses, the properties of steel and its behaviour, the Eurocodes used in design and software applications are presented in chapter 2.

Chapter 3 covers the loading on the footbridge. The loadings are determined according to the Eurocodes. In the Eurocodes they are referred to as design actions. After determining the design actions, the limit state (ULS and SLS) load combinations are also presented. The load combinations were used in the analysis and design of the footbridges.

In chapter 4, the design of the footbridge consisting of members in parallel is presented. The concept of Euler buckling load (members in compression) and Yield stress area (members in tension) are applied to determine the number of elements in parallel. The design of joints is also discussed in this chapter.

The findings of the research are presented and discussed in chapter 5. In addition, drawings and details of connections are also provided. Finally, in chapter 6, the conclusions and recommendations of the research are presented.

CHAPTER TWO

2.0 LITERATURE REVIEW AND BACKGROUND INFORMATION

2.1 Introduction

In this chapter, a review of literature is presented to provide a background on the concepts that are applied in the thesis. Previous work on structural optimization of trusses is presented and the Eurocodes, used for the design of steel trusses, are discussed. Also some special subjects are highlighted such as the behaviour of steel under tension and compression, the AutoCAD DXF file format and the SCIA Engineer software that is used for design and analysis.

2.2 Structural optimization of trusses.

Morphological indicators are dimensionless numbers that represent a property of a structure for example the volume or stiffness. These indicators are a function of the geometrical slenderness L/H of the structure, where L is the length and H is the height of the rectangular window framing the structure (Samyn, 1999; Pyl et al., 2013). These indicators allow for optimization of structures at conceptual design stage. Morphological indicators were introduced by Samyn (1999). He introduced two important indicators:

1. **Volume indicator** (W): concerned with the strength of the structure i.e. the minimum volume of material required for a structural typology achieving a fully stressed design.
2. **Displacement indicator** (Δ)**:** concerned with the stiffness of the structure i.e. maximum displacement of the structure for a fully stressed design.

The two indicators introduced by Samyn (1999) did not take into account the sensitivity of a structure to buckling instability. Latteur (2000) later introduced the **Buckling indicator** (Ψ) to fill this gap.

Using morphological indicators, different structures that have the same function can be compared for example a truss footbridge. For example, a comparison of the Warren, Howe and Pratt truss types using morphological indicators shows that the Warren type is the most economical. Koumar (2012) did the comparison of the trusses, during the conceptual design stage, using morphological indicators and selected the Warren truss for a footbridge. Also, because the Warren truss has the best performance of the three, it was selected by Pyl and Sitters (2012) for their modular footbridge. In addtion, the Warren truss, based on equilateral triangles, has members that are equal in length making it favourable in the construction of a modular footbridge. Therefore, for these reasons, the Warren truss was selected for this research.

The footbridge of Pyl and Sitters (2012) was designed according to the Eurocodes. In order to optimize the truss further, sections of same dimensions but different wall thicknesses were used to save material. Koumar (2012) also designed a pedestrian bridge according to the Eurocodes. However, the author used up to a maximum of two members to replace a single member in the Warren truss.

2.3 Steel as a construction material

The most commonly used materials in construction are reinforced concrete, structural steelwork, timber and masonry (McKenzie, 2004). Steel is superior in terms of strength and ductility compared to all the other materials. It has the highest strength to weight ratio of the building materials meaning that the dead weight of steel structures is relatively small. This property makes steel a very attractive structural material for high-rise buildings, long span bridges and structures located on soft ground. Using steel as a construction material has the advantage of a simplified foundation construction and structures that are easily disassembled for repairs or alterations and relocations. "Steel offers economic and attractive forms of construction which suit all the requirements demanded of a footbridge" (Corus Construction, 2005).

2.3.1 Material properties

Important mechanical properties of steel under a tensile load can be idealized in a stress-strain diagram as shown in Figure 2.1 (the stress in the diagram is based on the original cross-sectional area of a tensile bar, therefore it is basically a force displacement diagram). Initially, steel exhibits a linear elastic behaviour where the slope of the stress-strain diagram is the Young's Modulus of Elasticity (E). The values of E vary in the range $200\,000 - 210\,000$ N/mm^2. In Eurocode 3 *(EC3)*, a value of $210\,000$ N/mm^2 is assumed. The limit of the linear elastic behaviour is approximated by the yield stress $\left(f_y\right)$ (and a corresponding strain ε_y). Beyond the yield stress, mild steel undergoes a considerable plastic deformation without an increase in stress until strain-hardening takes place. This plastic deformation accounts for the ductility of steel. When strain-hardening (ε_{st}) is exceeded, the stress increases above the yield stress (f_y) until the ultimate tensile stress (f_u) is reached. After this, large reductions in the cross-section occur and the load capacity decreases until tensile fracture occurs (Trahair et al., 2008).

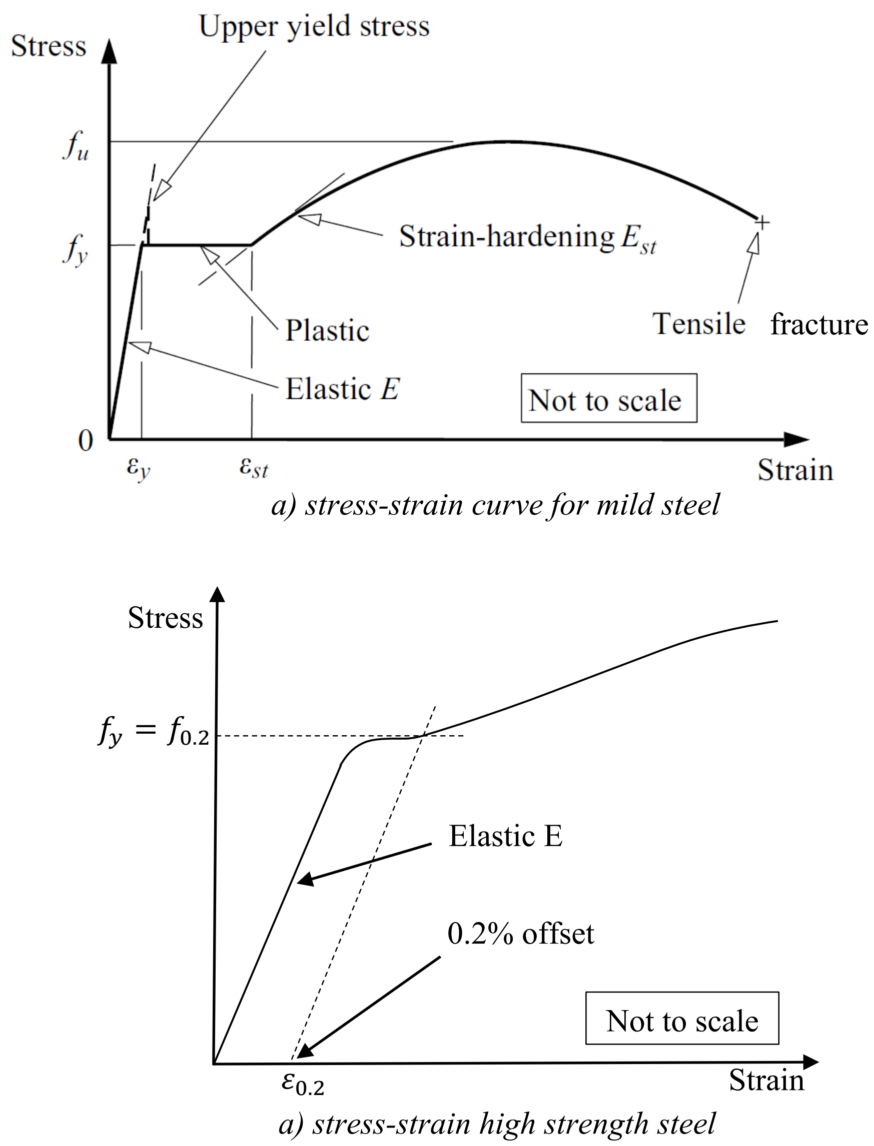

Figure 2.1: Stress-strain relationship for a) mild steel and b) high strength steel

The yield stress of steel varies with its chemical constituents. Carbon and manganese increase the yield stress of steel. Steels of higher quality (high carbon content) do not have a definite yield stress, instead $f_{0.2}$ is used. $f_{0.2}$ is the stress giving a permanent strain of 0.2% (in this case $\varepsilon_y = \varepsilon_{0.2} = \varepsilon_{st}$). The yield stress also varies with the heat treatment used and the amount of working that occurs during the rolling process. More worked thinner plates have higher yield stresses than thicker plates of the same chemical constituents. Cold working also increases the yield stress of steel (Trahair et al., 2008).

In addition to obtaining the values of the Youngs' modulus and the yield strength, the values of the Poisson's ratio can also be determined from the change in length and diameter of a specimen under elastic loading. The value of Poission's ratio is taken to be $\nu = 0.3$ in the elastic stage. Other important design values of material coefficients in Eurocode 3 are the shear modulus ($G \approx 81000\, N/mm^2$) and the coefficient of linear thermal expansion ($\alpha = 12 \times 10^{-6}\, per\ K$ for $T \leq 100^{\circ}C$).

There are various grades of steel to choose from that are specified in Eurocode 3. The choice of steel grade is dependent on:

1. Mechanical material properties
2. Ductility requirements
3. Toughness properties
4. Through thickness properties – Guidance given in EN 1993-1-10

With these requirements, the designation of the steel grade is defined in the product standard for hot-rolled products and structural steel EN 10025. The classification for steel grades is based on the minimum specified yield strength at ambient temperature. For the purpose of this work, a steel grade of S 235 was selected for the purposes of the design of steel elements.

2.3.2 Steel under tension

The load-deformation behaviour of concentrically loaded uniform tension members closely parallels the stress-strain behaviour of structural steel obtained from the results of tensile tests. Thus a member will exhibit a linear-elastic behaviour, even with residual stresses and initial crookedness, until the yield load is approached. In real applications, a tension member, due to its connection, is not always loaded along its neutral axis (eccentric loading) so that bending moments are introduced. It may also have transverse loads acting on it which result in additional bending actions. There are simplified design procedures that neglect the effects of eccentric connections. However, a more special account must be taken of the bending actions in design. The load extension behaviour of an ideal member under tension can be summarized in Figure 2.2, where $N_{pl} = Af_y$ or $N_{pl} = Af_{0.2}$ corresponds to the yield load and $N_u = Af_u$ corresponds to the ultimate load. The yield load N_{pl} is often taken as the load capacity of the member.

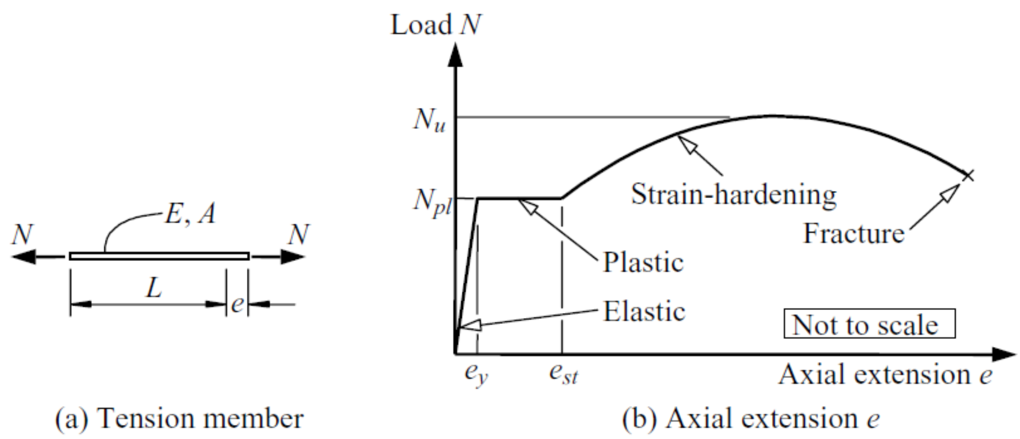

Figure 2.2: Load-extension behaviour of an ideal member in tension.

In the Eurocodes, the design resistance of an axially loaded member with holes present is taken as the lesser of the yielding of the gross section or rupture of the net section i.e. the lesser of $N_{pl.Rd} = Af_y/\gamma_{M0}$ *or* $N_{u.Rd} = 0.9A_{net}f_u/\gamma_{M2}$ (Trahair et al., 2008).

2.3.3 Steel under compression

In addition to members loaded in tension, another type of an axially loaded structural element is a member subjected to compression. Stocky compression members behave in a similar way to tension members until the material starts to flow plastically at the squash load. The resistance of a member in compression decreases with an increase in its length. Therefore, the compressive resistance of a slender member may be much less than its resistance in tension. This reduction in resistance of compression members is due to the phenomenon of buckling.

2.3.3.1 Buckling

This is a sudden failure of a structural element under compressive stress. It occurs at a level of stress that is less than what the material itself can withstand, therefore it is dependent on the geometrical properties of the element. Two types of buckling are treated in the Eurocode for elements with closed cross-sections i.e. flexural buckling and local buckling (Hultman, 2010; Ressler, 2011; Trahair et al., 2008).

2.3.3.2 Local and flexural buckling

Local buckling resistance of cross-sections depend on their width-to-thickness ratios. Local buckling is analogous to the collapse of an empty soda can under axial compression. Flexural buckling on the other hand is analogous to the collapse of a long, raw spaghetti as in Figure 2.3. In order to take into account varying local instability among cross sections, they are divided into four different cross-sectional classes (Hultman, 2010). In this work details of section classification will not be covered as they are all available in Eurocode 3. In addition, structural analysis software such as SCIA Engineer also designs structures according to the Eurocode and takes into consideration all the effects of local buckling, flexural buckling, slenderness of members and member imperfections.

2.3.4 Critical buckling load

Leonhard Euler derived the critical buckling load for bars. A bar can buckle in different shapes, or modes, depending on the support conditions. For trusses, both member end conditions are normally assumed to be hinged, which means than they are free to rotate (McKenzie, 2004).

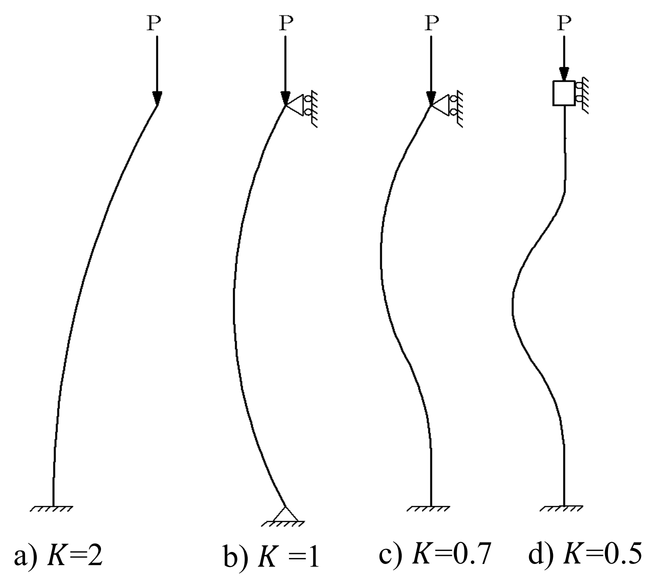

Figure 2.3: Euler's buckling modes

The critical buckling load N_{cr} is given by the formula *Eq.* (2.1):

$$N_{cr} = \frac{\pi^2 EI}{(KL)^2} \tag{2.1}$$

Where:

K	$-$	Effective buckling length factor for the member.
E	N/m^2	Modulus of elasticity.
I	m^4	Moment of inertia for the cross-section about the weak axis.
L	m	Length of the element.

In the alternative truss design, buckling will play an important role in determination of the number of members in parallel for the top members and the diagonal members that are under compression. Square and circular cross sections are normally used in order to reduce the risk of buckling. Unlike solid bars, the material in Square and circular hollow sections are normally distributed far away from the centroid leading to high values of moment of inertia. These cross sections are also less susceptible to lateral torsional buckling when compared to I sections.

2.3.5 Structural joints

In order to form a complete structure, steel elements have to be connected together. The design guidelines for structural steel joints can be found in EN 1993-1-8 (see Figure 2.4). Mainly welded and bolted connections were applied in this research. More specifically, fillet welds were chosen for connector plates. Bolted connections were used for the truss members to be joined to the connector plates. Table 3.1 of EN 1993-1-8 gives the nominal values of the yield strength(f_{yb}) and the ultimate tensile strength(f_{ub}) for bolts. In that table there is bolt class 4.6, 4.8, 5.6, 6.8, 8.8 and 10.9. The bolt classes are in the format $(a.b)$. This means that the ultimate tensile strength is $a \times 100\ \mathrm{N/mm^2}$ and the yield strength is $0.b\ \times (100a)\ \mathrm{N/mm^2}$. Also bolt diameters of 8, 10 and 12 mm were used to check the connections. To avoid having different bolt diameters and classes, one bolt class and diameter was proposed for an entire truss and consequently the footbridge.

2.4 Design Codes

Structural design codes have been formulated to enable engineers to design suitable and safe structures using steel, concrete, timber and masonry (McKenzie, 2004). In Kenya the British standards have been used for a long time. However, the British standards are no longer supported and will soon be phased out.

2.4.1 Eurocodes

The Eurocodes underpin all structural designs irrespective of the construction material. It establishes principles and requirements for safety, serviceability and durability of structures. The Eurocodes use a probabilistic approach to determine realistic values for actions that occur in combination with each other. The Eurocodes are used together with the national annexes. National annexes take into account local characteristics of loads, materials and safety requirements. Since there are no national annexes developed for

Kenya, the Belgian national annex was adopted for this work. There are 10 structural Eurocodes which are summarized in Table 2.1.

Table 2.1: Structural Eurocodes

1	EN 1990 or EC0	Basis of structural design.
2	EN 1991 or EC1	Actions(loads)
3	EN 1992 or EC2	Concrete
4	EN 1993 or EC3	Steelwork
5	EN 1994 or EC4	Composite steel and concrete
6	EN 1995 or EC5	Timber
7	EN 1996 or EC6	Masonry
8	EN 1997 or EC7	Geotechnical design
9	EN 1998 or EC8	Earthquake design
10	EN 1999 or EC9	Aluminium

Figure 2.4 shows how one can navigate the Eurocodes to determine the design actions and design phases for a steel bridge. Design actions refer to the loads which act on a footbridge. They include the self-weight, traffic actions and wind actions. Safety factors and load combinations are also determined which are useful in the limit state design approach. The design of steel elements in covered in Eurocode 3 where checks have been prescribed to take care of buckling instability, imperfections, section capacity and connections.

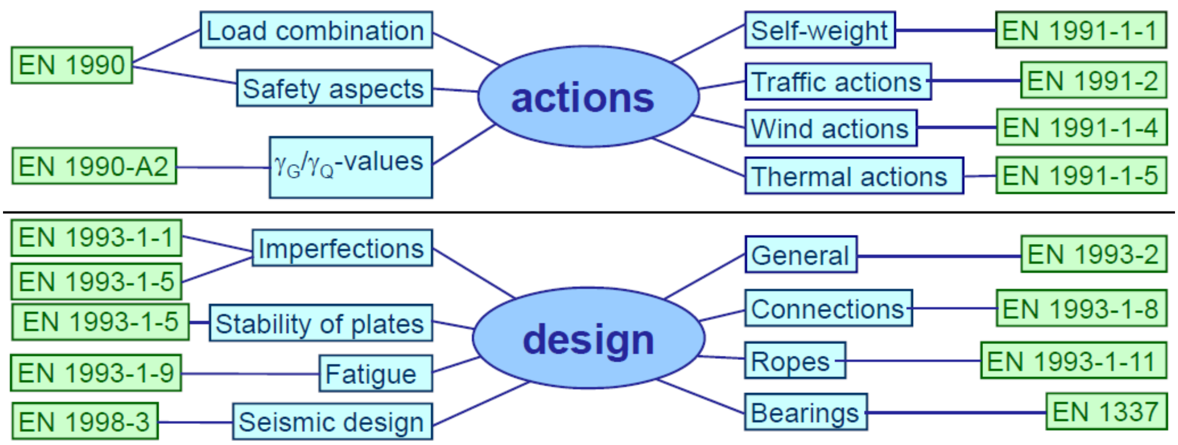

Figure 2.4: Eurocodes navigation for steel bridge design.

2.4.2 Limit state design

In the Eurocodes, the limit state design approach is used i.e. the ultimate limit state (ULS) and serviceability limit state (SLS) *(Martin and Purkiss, 2008).* Calculations for the limit states involves determination of loads, load factors, material factors and material strengths. Stability which is an ultimate limit state is the ability of a structure, or part of a structure to resist overturning and overall failure. Calculations should consider the worst realistic combination of loads. Deflection is a serviceability limit state. Deflections should not impair the efficiency of a structure, or its components nor cause damage to the finishes. Generally the worst realistic combination of unfactored imposed loads is used to calculate elastic deflections. Dynamic effects to be considered at the serviceability limit state are vibrations caused by machines, and oscillations caused by harmonic resonance, for example, wind gusts on structures. The natural frequency of the construction should be different from the exciting force to avoid resonance. The dynamic effects were however not considered in this research.

2.5 Design Software

The Finite Element method has been developed over the years and is well suited for computer applications. Therefore, it is used in computer software packages that perform analysis of structures. Examples of computer software used in the industry include SCIA Engineer, Autodesk Robot Structure, Ansys and many others. An important feature is that these analysis software can share information with AutoCAD through the DXF file format.

2.5.1 SCIA Engineer

SCIA Engineer is a software that is used for static and dynamic analysis of structures. It is also used for the design of structures according to standards (e.g. the Eurocode). SCIA

Engineer applies the displacement-based finite element method to analyse structures. This software applies finite elements indirectly by generating a finite element mesh on structural elements before calculations are performed (Nemetschek, 2012). Using SCIA Engineer, structures consisting of 1D members (linear finite elements), plates and curved slabs (2D finite elements) can be calculated and designed.

2.5.2 AutoCAD DXF file

AutoCAD DXF stands for Drawing Exchange Format. It is a file format that was developed by Autodesk for enabling data interoperability between AutoCAD and other applications. ASCII versions of the file can be read with a text-editor. The attributes of the DXF file are summarized in Table 2.2 (AutoCAD, 2008*)*.

Table 2.2: DXF file attributes.

1	Header section
2	Classes section
3	Tables section
4	Blocks section
5	Entities section
6	Objects section
7	End of file

Since trusses are idealized as lines during structural analysis, it is possible to write a simple computer program that will store a structure (a truss in this thesis) as an AutoCAD DXF file. The approach adopted in this thesis results in a truss with some intricate details (truss consisting members in parallel). Using standard available software, such as SCIA Engineer, there are no catalogue blocks for the automatic generation of this kind of truss. However, standard software such as SCIA Engineer can read AutoCAD DXF files.

CHAPTER THREE

3.0 LOADING ON THE FOOTBRIDGE

3.1 Introduction

A structure is required to support loads and resist forces, and to transfer these loads and forces to the foundations of the structures. Therefore, the loads and forces in a footbridge may arise from its mass (self-weight), or from its use (provision of access to people), or from forces of nature (for example wind loading). In this section, the design values of the loads that are required to analyse and design the footbridge will be determined.

3.2 Methodology

The loads that act on the footbridge were determined as prescribed by Eurocode 1. The self-weight (also called permanent actions) of the elements of the footbridge were obtained from EN 1991-1-1. Traffic actions on the footbridge were determined from EN 1991-2 while the wind loads from EN 1991-1-4. When applying the Eurocode, the aforementioned loads are referred to as *design actions*. In addition to the actions, load combinations and safety factors were determined from EN 1990 and EN 1990-A2 respectively.

3.3 Design actions

3.3.1 Permanent actions

These are loads as a result of the members of the truss, the bridge deck and the cross beam. In order to determine these loads, the material densities are required. Material densities can be obtained from EN 1991-1-1 (EUROCODE 1). The named elements were all modelled in SCIA Engineer which automatically computes the self-weight of the footbridge. The weight of the safety barrier and connections (nuts and bolts) were neglected during the analysis.

3.3.2 Uniformly distributed load

EN 1991-2 concerns traffic actions on footbridges. In clause 5.3.2.1 it is stated that for the design of footbridges, a uniformly distributed load q_{fk} should be defined and applied only in the unfavourable parts of the influence surface, longitudinally and transversely. The recommended value for $q_{fk} = 5.0$ kN/m^2 (EUROCODE 1). This loading is already sufficient to take care of the possible presence of loaded donkeys or cattle.

3.3.3 Concentrated load

According to EN 1991-2, a characteristic value 10 kN acting on a square surface of sides 0.10 m should be taken into account on the footbridge. This is only applied when local verification is to be carried out and should not be combined together with the uniformly distributed load (EUROCODE 1). During the global analysis, local effects were not considered therefore this load was not necessary.

3.3.4 Horizontal load

To simulate the load of people acting on the safety barrier, a load of 1kN/m is applied at a height of 1m. This load may also be applied as a moment of 1kNm/m on the bridge deck (EUROCODE 1).

3.3.5 Wind actions

The wind actions to be considered apply to the deck of the footbridge (EUROCODE 1). Figure 3.1 shows the directions of wind actions that are considered in the Eurocodes. In this work, only the wind load in the *x*-direction was considered to be the most critical.

Since dynamic effects are not considered (for footbridges of length less than 30m), the simplified approach was used.

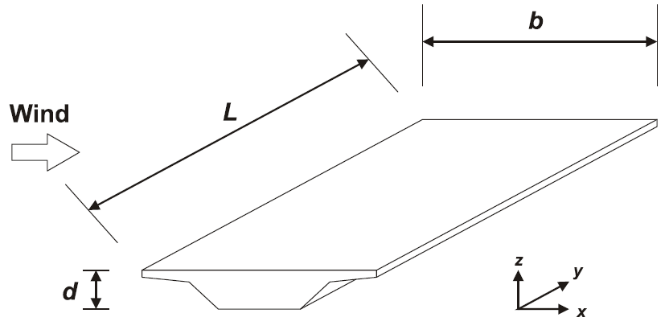

Figure 3.1: Direction of wind actions (EUROCODE 1): EC 1-4 Figure 8.2

3.3.5.1 Force in *x*-direction : simplified method

The force in the *x*-direction may cause significant lateral deformations. It is given by the formula:

$$F_W = \frac{1}{2} \times \rho \times v_b^2 \times C \times A_{ref,x} \tag{3.1}$$

Where:

v_b	$\mathrm{m/s}$	Basic wind speed. Belgian case will be assumed (26.2 m/s)
C	$-$	Wind load factor $C = c_e.c_{f,x}$, where c_e is the exposure factor and $c_{f,x}$ the force coeficient.
$A_{ref,x}$	m	Reference area per meter of deck length in the *x*-direction
ρ	$\mathrm{kg/m^3}$	Air density taken as 1.25

The Belgian wind speed was chosen because there is no Kenyan annex where such nationally determined parameters are defined. In addtion, the Belgian wind speed would give the worst case scenario since it is on the safe side for the Kenyan case.

Since no terrain information was available as to where the footbridge will be constructed, recommended values given in Table 8.2 of EN 1991-1-4 were used for the C factor. Assuming $b/d_{tot} > 4$ and $z_e \leq 20$ m, $C = 3.6$ ($d_{tot} = 0.4$ and assuming $z_e =$

20 since no properties of the terrain where the bridge will be constructed are known). d_{tot} is the depth of the reference are per meter of the deck and b is the breadth of the deck.

$A_{ref,x} = d + 0.3$ m for an open parapet or an open safety barrier in Eurocode 1 where d is the depth of the bridge deck. Assuming a depth of 0.1m (the largest deck was found to have a depth of 60 mm), $A_{ref,x} = 0.4$ m

$$F_W = \frac{1}{2} \times 1.25\ \mathrm{kg/m^3} \times (26.2\ \mathrm{m/s})^2 \times 3.6 \times 0.4\ \mathrm{m}\ = 0.62\ \mathrm{kN/m}$$

3.4 Load combinations

3.4.1 The ultimate limit state (ULS)

The Eurocodes adopt the limit state design principles. Partial safty factors for the ultimate limit state are summarized in Table 3.1.

Table 3.1: ULS partial safety factors (EUROCODE 1).

Load	Unfavourable	Favourable
γ_G (dead load)	1.35	1.0
γ_Q (variable load)	1.35	0.0

The load combination for the Ultimate Limit state design is given by the formula:

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (3.2)$$

Where:

$G_{k,j}$	is the permanent action (own weight of structure).
$Q_{k,1}$	is the dominant/leading variable action (udl traffic action)
$Q_{k,i}$	other variable actions (wind in this case).
$\gamma_{G,j}, \gamma_{Q,1}, \gamma_{Q,i}$	Partial safety factors
$\psi_{0,i}$	Load combination factor to take into account the reduced probability of simultaneous occurrence of the most unfavourable values of several independent actions.

Subscript k	Denotes the characteristic value of the load.
Subscript j	Refers to numbers given to permanent actions (simply a summation of all the relevant permanent actions)
Subscript i	Same as Subscript j but for variable loads or actions

The uniformly distributed load had the greatest influence compared to the wind loading. It is for that reason that it was considered the dominant variable. The values for the partial safety factors are given in Table 3.1. Values of the load combination factors $\psi_{0,i}$ for footbridges can be found in EUROCODE 0, ANNEX 2 as $\psi_0 = 0.3$ for wind loads.

After substituting the values of the partial safety factors and the load combination factors and taking the uniformly distributed load (traffic load) as the leading variable, the final load combination equation becomes:

$$1.35G_k + 1.35Q_{k,1} + 1.35(0.3)Q_{k,(i=1)} = 1.35G_k + 1.35Q_{k,1} + 0.41Q_{k,(i=1)} \quad (3.3)$$

This load combination is the most critical than taking the wind load as the dominant variable load.

3.4.2 Serviceability limit state (SLS)

The check for deflections of the footbridge was carried out in the serviceability limit state. This is also a basis of design that is adopted by the Eurocodes as was discussed in Chapter 2. The load combination equation for this limit state is given by the formula:

$$\sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \quad (3.4)$$

Notice that there are no partial safety factors in the formula. This is known as the characteristic combination. The dominant variable action is still the uniformly distributed load and is taken as the value of $Q_{k,1}$. The value of $\psi_{0,i} = 0.3$ as determined in the case of the ultimate limit state. After pluging in the value of the load combination factor, the formula for the serviceability limit state simplifies to:

$$G_{k,j} + Q_{k,1} + 0.3Q_{k,(i=1)} \tag{3.5}$$

It can be seen that the effect of the load combination factor $\psi_{0,i}$ is to reduce the value of the other variable actions (wind in this case).

3.5 Summary

A summary of the design actions and load combinations are given in Table 3.2 and Table 3.3 respectively.

Table 3.2: Summary of design actions

Permanent actions	
Self-weight of trusses	$G_{k,t}$ = automatically computed
Self-weight of cross beams	$G_{k,c}$ = automatically computed
Weight of deck	$G_{k,d}$ = automatically computed
Weight of safety barrier and connections	Neglected
Variable Actions	
Uniformly distributed load (Traffic load)	$Q_{k,u} = 5kN/m^2$
Wind Actions	
x-direction	$Q_{k,Wx}$ $0.62kN/m$

Table 3.3: Summary of load combinations

ULS	$1.35G_k + 1.35Q_{k,1} + 0.45Q_{k,(i=1)}$
SLS	$G_{k,j} + Q_{k,1} + 0.3Q_{k,(i=1)}$

CHAPTER FOUR

4.0 DESIGN OF THE BRIDGE CONSISTING OF TRUSSES WITH MEMBERS IN PARALLEL

4.1 Introduction

In this chapter, the approach adopted in the design of a footbridge consisting of trusses with members in parallel is discussed. But first, basic dimensions of 2.44 m by 0.61 m were adopted for the bridge deck. These dimensions were chosen because standard chequered plates, which will form the surface of the bridge deck, are supplied in lengths of 2.44 m by 1.22 m in Kenya. Therefore, half of each standard chequered plate can be used for each bridge deck unit. The bridge deck unit was divided into four sections giving rise to the dimensions $0.61\ \text{m} \times 0.61\ \text{m}$. These dimensions were chosen so that each deck unit could be held by hand and bridge deck widths of 1.22 m and 1.83 m could be obtained.

Steel was selected as the material to be used in the construction of the footbridge. For the purposes of the design of the elements of the footbridge, steel of strength class S235 was chosen. It is possible to have higher steel strength classes e.g. S355, however, S235 was selected to give the worst case scenario.

In this work, the Warren type truss was selected for the footbridge because it consists of members which are of equal length. Also, from previous research it was found that the Warren truss was the optimal one when compared to the Howe and Pratt trusses using morphological indicators (Pyl et al., 2012; Aushim, 2012).

4.2 Methodology

The bridge deck was conceived to be a frame consisting of C – sections arranged in the form of a 1 by 4 grid on top of which a 3 mm tear plate will be welded. In order to support the bridge deck, cross beams were applied to transfer the loads to the trusses. Cross beams consisted of I – sections that were connected at the truss nodes. Using element cross sections in parallel presented some challenges which were solved by writing a program in Java. The program used the Euler buckling load and the Yield stress area of the elements to determine the number of cross sections to be used in parallel for members in compression and tension respectively. Also, the program modelled the entire footbridge as a DXF file for later export to SCIA Engineer. Static analysis and design of elements to the Eurocodes was done in SCIA engineer from which the mass of the trusses was obtained for later comparison. After comparison, the joints of the selected trusses were then designed and detailing carried out.

4.3 Design of bridge deck and supporting beams

4.3.1 Design of bridge deck

Each bridge deck unit is a frame consisting of C-sections of 2.44 m and 0.61 m as illustrated in Figure 4.1. On top of each deck, a 3mm chequered plate forms part of the surface over which people will walk. The bridge deck units rest on cross-beams (which are I-sections) that are connected to the bottom joint beams of each truss. During analysis, the uniformly distributed load that is used to simulate the traffic on the footbridge is applied on the bridge deck. This is because the whole foobridge was modelled in the software and the load would therefore be transferred to all the components of the footbridge. After analysis, the design of the bridge deck and cross-beams was done using SCIA Engineer steel design modules according to the Eurocodes and adequate sections were then adopted. The deck was designed such that the weight was limited to enable it to be handled by hand without hoisting equipment. For that reason, the weight was expected to be around 600N. From the design of the bridge decks, it was found that the heaviest deck weighed 672N.

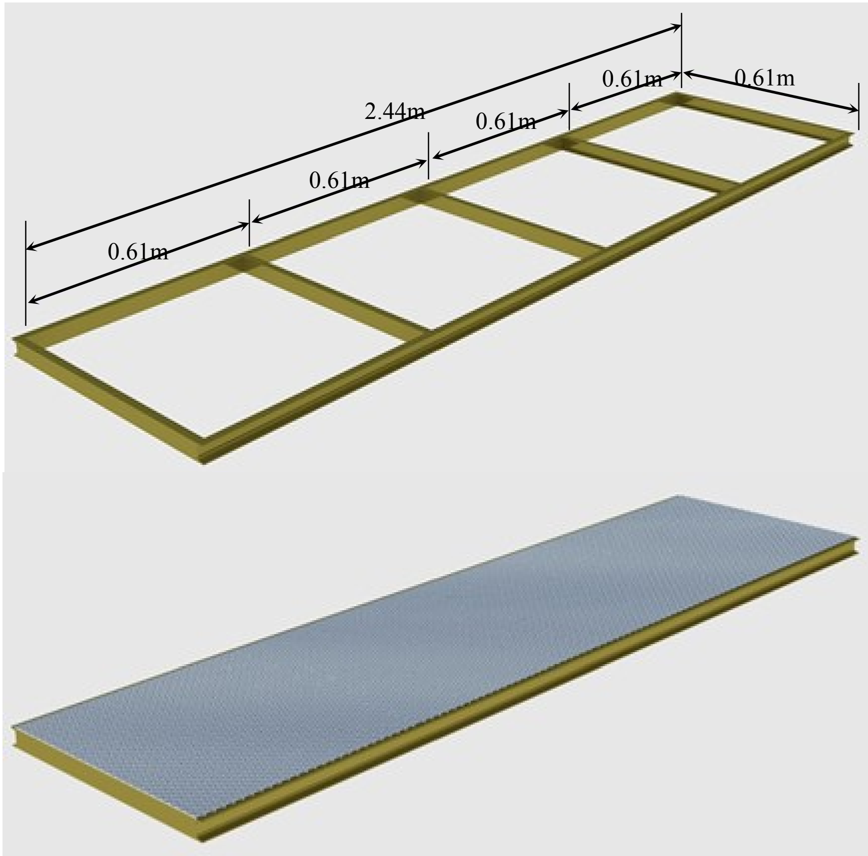

Figure 4.1: The bridge deck.

4.3.2 Design of Supporting beams (cross beams)

The supporting beams or cross beams were modelled as beams and I – sections were chosen to resist the loads that were transferred from the bridge deck (see Figure 4.5 and Figure 4.7). They were part of the entire footbridge which was modelled, analysed and designed in SCIA Engineer. Cross beams were connected at the centre of the bottom joint beams of the trusses. The loading on the cross beams was not applied directly but was automatically transferred from the bridge deck by SCIA Engineer. During design, section checks (compression, torsion, shear, bending moment and combined bending, axial and shear force checks) and stability checks (flexural buckling, lateral torsional buckling, compression and bending, and shear buckling checks) were carried out by SCIA Engineer.

4.4 Fully stressed design and modelling of the truss

4.4.1 Fully stressed design

A fully stressed design can never be completely achieved. However, the intention was to approach a fully stressed design as close as possible by varying the number of parallel members as well as varying the wall thickness of the members. Parallel connections up to a maximum of five members were used instead of one or two different cross-sections. Top and bottom members were varied from two to five members as shown in Figure 4.3. This arrangement was chosen because of the variation of the forces in the top and bottom truss members as shown in Figure 4.2. The truss member at the centre is subjected to the highest force. Members far away from the centre are subjected to lower forces. Therefore, a higher number of members is needed in parallel to resist a greater force while a lower number of members is needed to resist a lesser force. The number of diagonal members is maintained at two since the variation of forces in these members is much less than that of the top and bottom members.

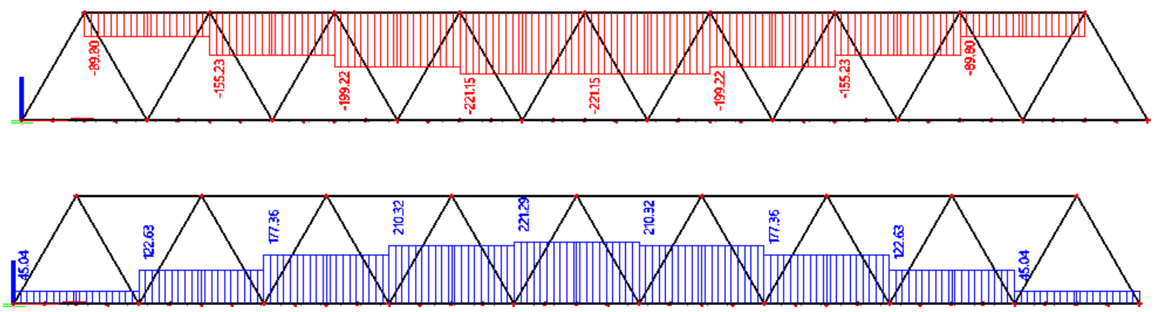

Figure 4.2: Force variation in the top and bottom members of a truss loaded with a uniformly distributed load of 5 kN/m^2 and dead loads (of truss, deck and cross beams).

The truss in Figure 4.2 is part of a three dimensional footbridge modelled in SCIA Engineer which is supported at both ends. At one side of the bridge the support is pinned while a sliding support is set at the other end. The members are modelled as fixed at the nodes of the truss. Some moments and shear forces were developed in the members of the truss but they were neglected since they were small in magnitude (Figure 4.2 is used to show the variation of axial forces in a classical warren truss and was not used to determine the number of elements in parallel). However, shear forces and bending moments were considered in SCIA Engineer when carrying out design of the truss elements.

A two dimensional analysis, where the truss was pinned at one end and sliding at the other end and the elements were assumed hinged at both ends, was only carried out to determine the number of elements in parallel at the preliminary design stage for the alternative truss.

In SCIA Engineer, whole footbridges were modelled for both the classical and alternative trusses. For the footbridges, a three dimensional analysis was carried out to determine truss element sections for both the classical and the alternative designs. The truss elements for all the footbridges were modelled as fixed at both ends. All the moments and shear forces in addition to the axial loads were taken into account during design in the software. Footbridges whose trusses were selected to form the catalogue, were further checked for lateral stability.

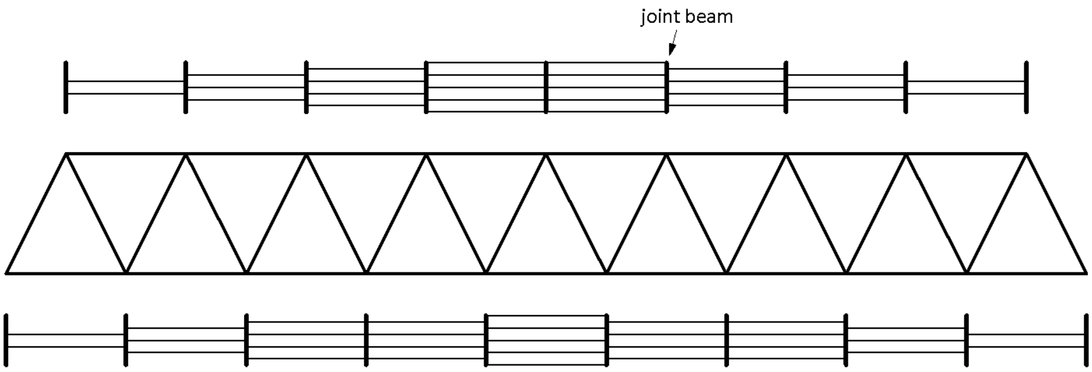

Figure 4.3: Varying number of members in the truss.

A three dimensional truss consisting of members in parallel is shown in Figure 4.4. This truss consists of circular hollow section members, the ends of which are flattened.

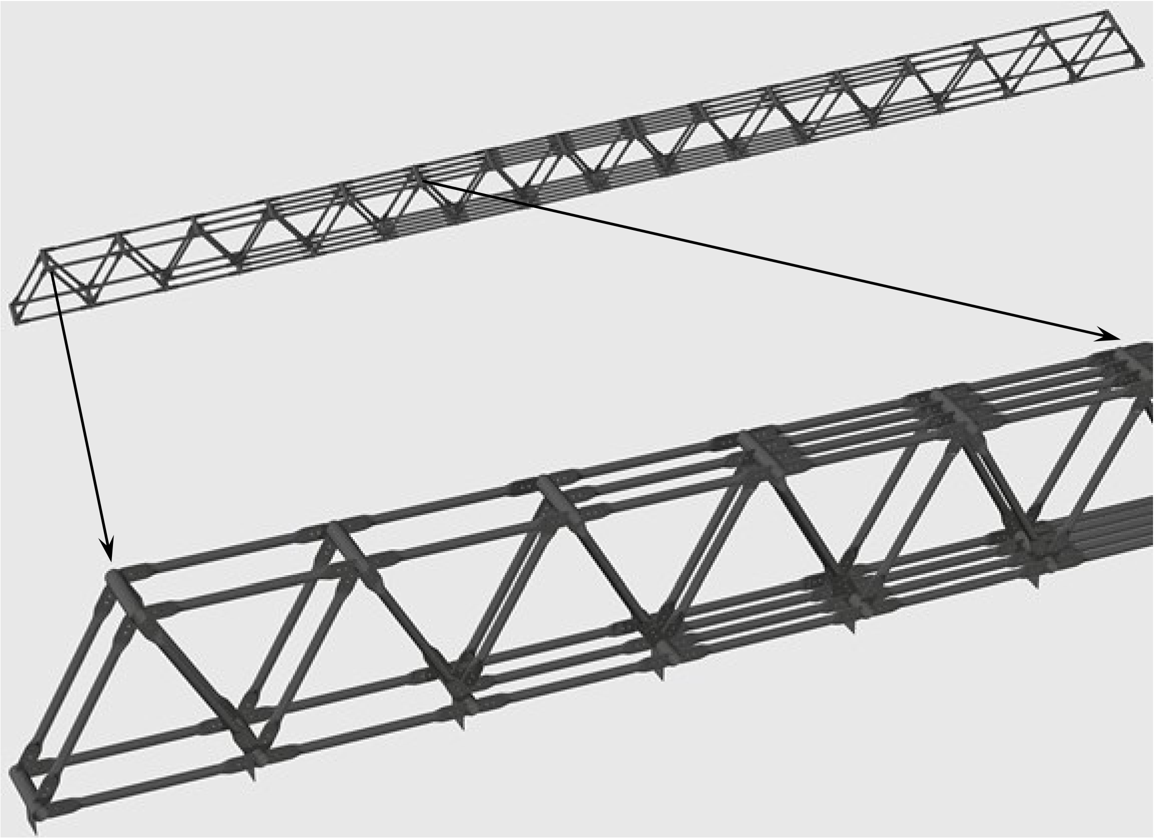

Figure 4.4: Three dimensional truss consisting of members in parallel.

A footbridge can be constructed by using two trusses consisting of members in parallel. To visualize such a footbridge, a three dimensional drawing was made using AutoCAD as shown in Figure 4.5.

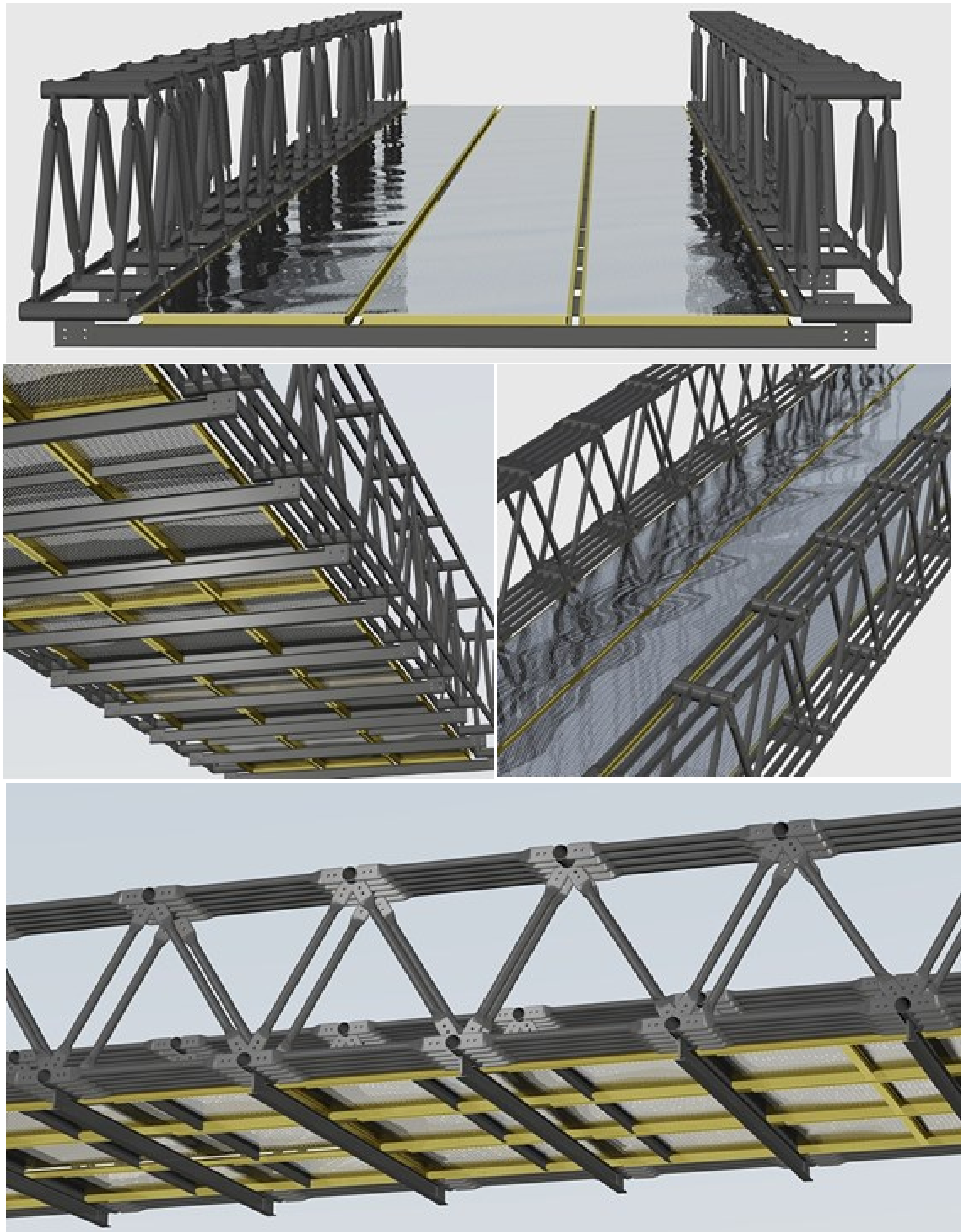

Figure 4.5: Three dimensional footbridge consisting of members in parallel.

4.4.1.1 Selection and determination of number of members in parallel

Two criteria were used in selection and determination of the number of members in parallel and are summarized in Figure 4.6. The criteria were:

1. The Euler Buckling Load criterion.

2. The Yield Stress Area criterion.

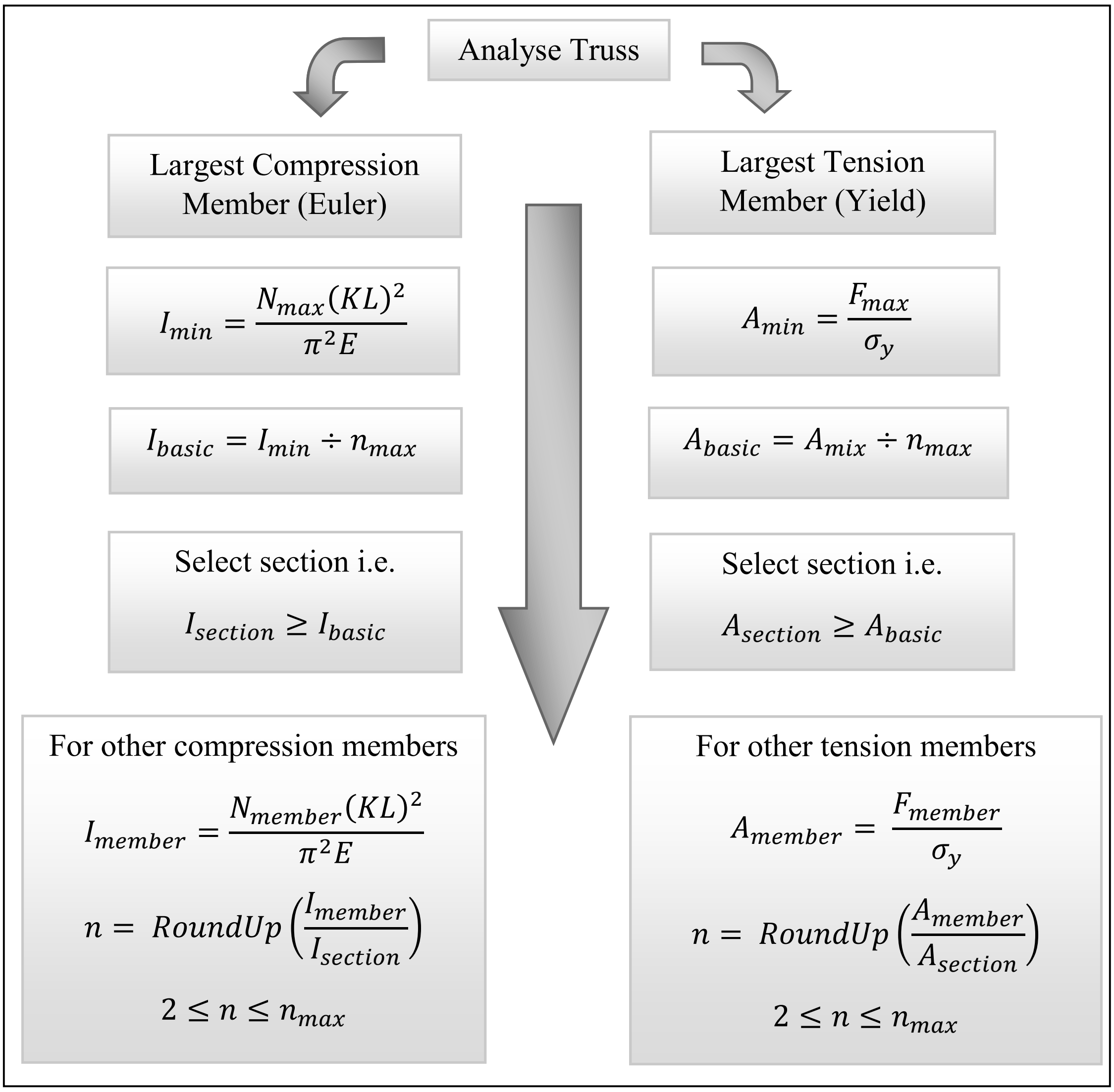

Figure 4.6 : Summary of section selection and determination of number of members in parallel.

It should be noted that the above criteria were used in the preliminary design stage to determine the number of members in parallel and to model the truss. A two dimensional analysis of a truss with elements hinged at both ends ($K = 1$) was carried out to determine the number of elements in parallel. The trusses were then modelled in SCIA Engineer where they were analysed and finally designed according to the Eurocode.

4.4.1.2 Euler Buckling Load Criterion: Compression Members

Euler buckling load defines the axial compression that will cause a strut to fail in elastic flexural buckling. For a given buckling load, the area moment of inertia can be obtained by rearranging the Euler buckling formula given by *Eq.* (2.1) as follows:

$$I = \frac{N_{cr}(KL)^2}{\pi^2 E} \qquad (4.1)$$

Where:

E	$\mathrm{N/m^2}$	Young's Modulus
I	$\mathrm{m^4}$	Area moment of inertia
L	m	Unsupported length of member in compression
N_{cr}	N	Critical buckling force (compression)
K	−	Effective length factor of element whose value depends on the end support conditions. K=1 since hinges were assumed

The formula was applied as the basis for selecting the sections to be used for the members in parallel. If for example five members were to be used in instead of one member, the area moment of inertia was divided by five and a basic section selected.

The area moment of inertia(I_{min}) of the member with the largest compression force was computed using Euler's formula. It was assumed that the largest compression force was the Euler buckling load. A basic area moment of inertia for the compression members was computed as follows:

$$I_{basic} = I_{min} \div n_{max} \tag{4.2}$$

Where n_{max} was the maximum number of compression members that were to be placed in parallel. Using I_{basic}, a section was selected for members in compression (which had a moment of inertia ($I_{section}$). For the remaining compression members, the respective moments of inertia were calculated (I_{member}) with the assumption that the compressive forces in the members were the Euler buckling loads. The number of members needed to replace a single member were obtained by the expression:

$$n = RoundUp\left(\frac{I_{member}}{I_{section}}\right) \; ; \; 2 \leq n \leq n_{max} \tag{4.3}$$

It should be noted that the Euler buckling formula is theoretical. It is derived from an idealized case overlooking important aspects that will be present in practical compression members such as (Martin and Purkiss, 2008; Trahair et al., 2008):

1. Initial lack of straightness of members.
2. Material that is not perfectly linear elastic.
3. Stocky members in compression.

Design of compression members according to the Eurocode 3 takes into account the above effects. The Perry-Robertson formula is the basis of design of compression members according to the Eurocode 3 (Martin *et al.*, 2008). It is derived from the expression for the maximum stress in an axially loaded initially curved column. Including all the above effects in the selection criteria would be very complicated. Since SCIA Engineer includes the functionality to design steel elements according to the Eurocodes, it takes care of the above effects which were neglected in section selection and determination of number of members in parallel. This approach made it simpler to generate the model of the truss consisting of members in parallel.

4.4.1.3 Yield Stress Area Criterion: Tension Members

The yield stress in a tie is theoretically computed by the formula:

$$\sigma_y = \frac{F}{A} \tag{4.4}$$

Where:

A	m^2	Area of cross section of the member
σ_y	$\mathrm{N/m}^2$	Yield stress of the material
F	N	Tensile force being sustained by the member

The area is computed by rearranging the formula as follows:

$$A = \frac{F}{\sigma_y} \tag{4.5}$$

A similar approach to the one used for members in compression was applied. The only difference being the use of the yield stress area rather than the area moment of inertia. The yield stress area (A_{min}) of the member with the largest tensile force was computed using the yield stress formula (the net area as a result of presence of bolt holes was neglected in this calculation). It was assumed that the member will yield at the maximum tensile force (F_{max}). A basic area for the members in tension was computed by the formula:

$$A_{basic} = A_{min} \div n_{max} \tag{4.6}$$

Where n_{max} is the maximum number of tension members that are to be placed in parallel. Using A_{basic} a section was selected for the members in tension (which had an area

$A_{section}$). For the remaining members in tension, the respective yield stress areas (A_{member}) were computed with the assumption that the tensile forces in the members cause them to yield. The number of members that were needed to replace a single member was computed by the formula:

$$n = RoundUp\left(\frac{A_{member}}{A_{section}}\right);\ 2 \leq n \leq n_{max} \tag{4.7}$$

4.4.2 Modelling of the truss

A truss consisting of members in parallel poses a challenge during modeling. In SCIA Engineer there is a catalogue block for generating trusses of various kinds and sizes. However, there is no catalogue block for generating a truss consisting of members in parallel. SCIA Engineer provides basic tools for modeling a structure element by element from scratch. Using the basic tools provided by SCIA Engineer to manually model a truss consisting of members in parallel is quite tiresome and prone to mistakes. Moreover, when there are corrections to be made in the model it becomes difficult to edit and rearrange the number of bars.

In order to solve the problem of modeling the truss consisting of members in parallel, a small computer program was written in Java (see APPENDIX 5) and used to generate the trusses. The program saved the model as an AutoCAD DXF file which can be loaded by any structural analysis software capable of reading AutoCAD DXF files. SCIA Engineer reads AutoCAD DXF files as input data for structural models. For the computer program, the entities section was of most importance. This section contains drawing entities for example lines, poly-lines and circles. There are various group codes that define the data stored in the file. Truss members were idealized as lines and were represented by the line entity in the DXF file.

4.4.3 Modelling of the footbridge

The program to model the truss consisting of members in parallel was extended to model the entire footbridge and store it in a DXF file. Entities contained in the DXF file are summarized in Table 4.1.

Table 4.1: DXF file entities used to represent the three dimensional footbridge.

Truss Property	**DXF file Entity**
Truss member	Line
Joint beam	Line
Truss cross beam	Line
Deck beams	Line
Deck plate	Polyline

A three dimensional model imported in SCIA Engineer and the corresponding support conditions is shown in Figure 4.7. For analysis of the footbridges in the research, SCIA Engineer was set to analyse structures of type "General XYZ". The trusses of the footbridges were modelled as three dimensional frames. Top, bottom and diagonal members were therefore fixed at the points where they were connected to the joint beams. The cross beams were also modelled as beams and were therefore expected to resist bending moments and shear forces. To prevent development of a torque, the cross beams were connected at the centre of the truss for the alternative design. The elements of the bridge deck were also modelled as beams. Bridge deck elements had hinges set such that they could be allowed to slide on the cross beam (along the length of the bridge) at the point of connection to prevent development of compression forces at the bottom members of the trusses. The wind bracing elements were modelled to take axial loads only and the deck plates were modelled as isotropic plate elements.

The traffic load of 5 $\mathrm{kN/m^2}$ was applied on the surface of the bridge deck plate and was automatically transferred from the deck (deck plate then deck beams) to the cross beams and from the cross beams to the truss by the software during analysis.

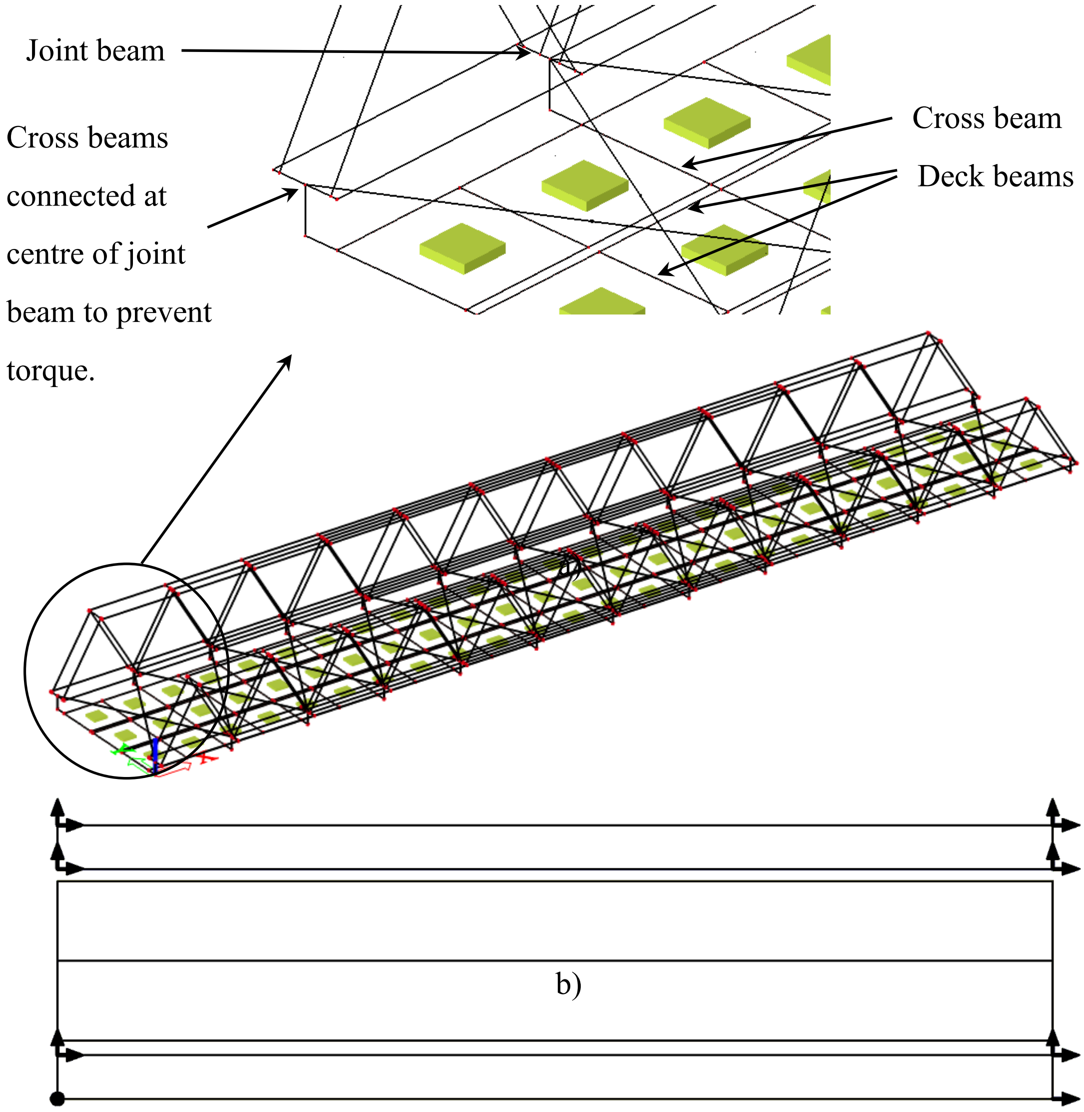

Figure 4.7: a) Footbridge model imported in SCIA Engineer from a DXF file. The green boxes are graphical representations of the meshing of the deck plate. b) Boundary conditions. The arrows show the unconstrained displacements.

All the internal axial, bending and shear forces were available for all the members of the truss (including the joint beams, cross beams and deck beams) for the design to be carried out for the respective elements. During the global analysis of a larger number of trusses only vertical loads were considered and after the trusses that constitute the catalogue were selected the wind load was taken into account to determine the bracing for the footbridge. The wind load was converted to a point load and then applied to the bottom nodes of the truss at the location where the cross beams are connected. In addition, the load to simulate the weight of people acting on the safety barrier was converted to a point load and applied to the top joint beams of the trusses. It was assumed that slender trusses also acted as the safety barrier for the load to be applied as stated.

4.4.3.1 Design of the footbridge

SCIA engineer was also used for the design of the members that constitute the footbridge. The truss members that were under compression (as much as they experienced some bending moments, shear and torsional forces) were assumed to be restrained at the ends. Therefore the buckling length was taken to be equal to the length of the truss member under compression. For elements under compression, a section check and stability check was carried out by the software. Section checks for compression members included a compression check, shear check, bending moment check and combined bending, axial force and shear checks. Checks for stability included flexural buckling check, lateral torsional buckling check and compression and bending checks. The same checks were also done for elements that were under tension since they were also expected to have some parasitic moments, shears and torques.

Section checks that were performed on the cross beams, joint beams and deck beams include compression check, torsion check, shear check, bending moment check and combined bending, axial and shear force checks. The stability checks for this case were flexural buckling check, check for lateral torsional buckling, compression and bending check and shear buckling check.

4.5 Global analysis of a large number of trusses and comparison with classical trusses

A number of trusses were analysed using SCIA Engineer software. From these trusses, the optimal ones were selected. The comparison with the "classical" trusses was based on the weight of the trusses. It should be noted that during the analysis of these trusses, only the dead and traffic loads were considered. This was done to avoid anti symmetric loading conditions during determination of number of members in parallel. The influence of the wind load was used for sizing the wind bracing elements after narrowing down to specific trusses.

Table 4.2: Trusses that were considered.

	3/8 U	**4/8 U**	**5/8 U**	**6/8 U**	**7/8 U**	**8/8 U**
TRUSS	A	B	C	D	E	F
n	16	12	10	8	7	6
L ±14.7	14.7	14.7	15.3	14.7	15.0	14.7
TRUSS	G	H	I	J	K	L
n	24	18	14	12	10	9
L ± 22.1	22.1	22.0	21.5	22.1	21.4	22.1
TRUSS	M	N	O	P	Q	R
n	32	24	19	16	14	12
L ± 29.4	29.5	29.3	29.1	29.5	30.0	29.4

The dimensions of the trusses are related to the dimension of a bridge deck unit, see Figure 4.8. The bridge deck unit has a length $\mathbf{U} = 2.44$ m. This length was considered as the basic dimension from which the truss field widths were determined. Taking truss A in Table 4.2 for example, the field length was obtained as $(3/8) \times 2.44$ m $= 0.915$ m. Since $n = 16$ the length of the truss was 16×0.915 m $= 14.7$ m. This was done to determine the dimensions for all the trusses summarised in Table 4.2 (The trusses are numbered A to R, **n** is the number of fields in the truss and **L** the length of the footbridge)

. In addition, two footbridge widths were considered i.e 1.22 m and 1.83 m wide footbridges.

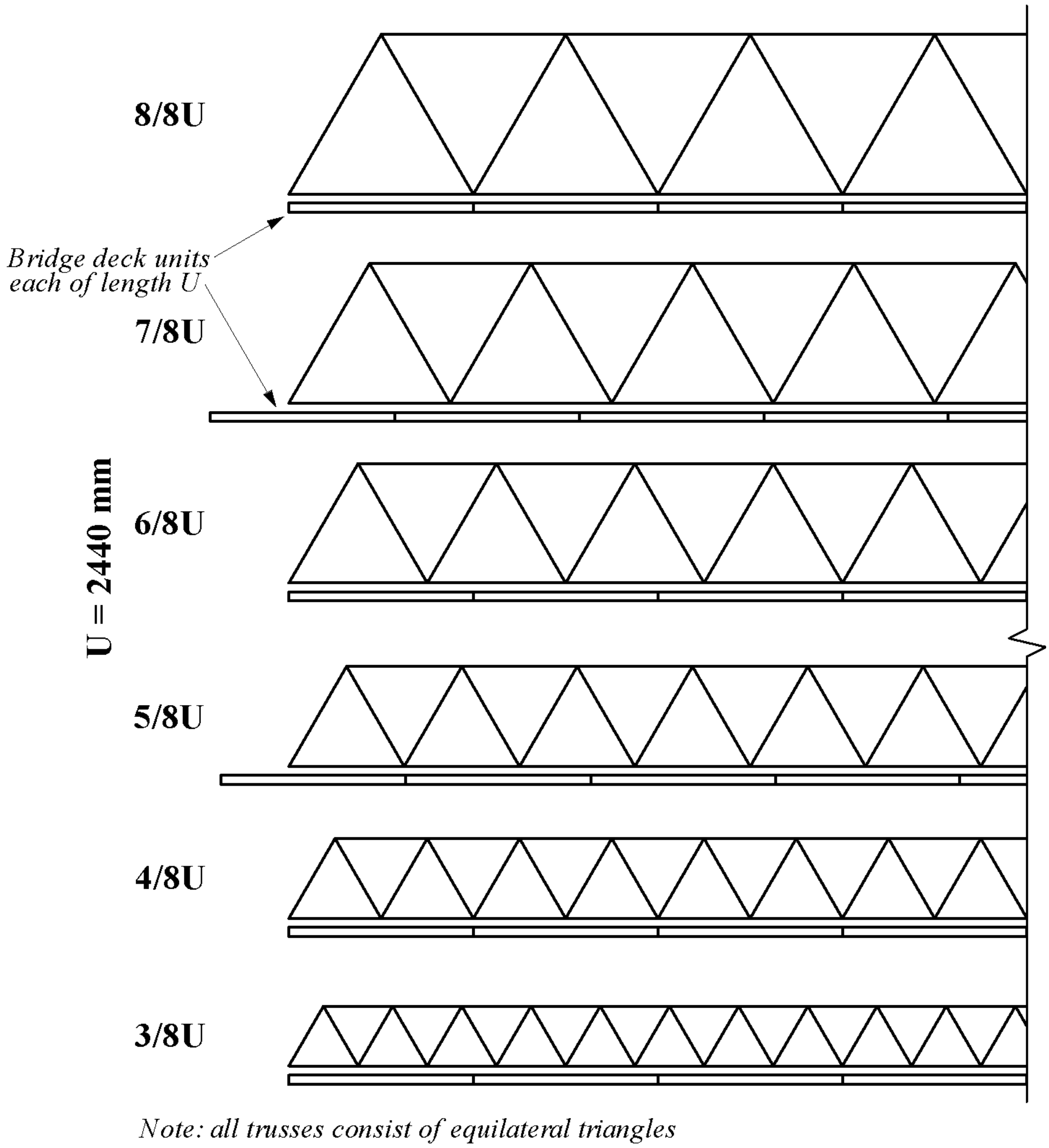

Figure 4.8: Relationship between bridge deck and truss dimensions.

4.6 Modelling and design of joints

4.6.1 Joint modelling

Joints of the trusses were modelled as horizontal beams which are perpendicular to the direction of the truss. Forces acting on the joint beam were idealized as shown in the example of Figure 4.9.

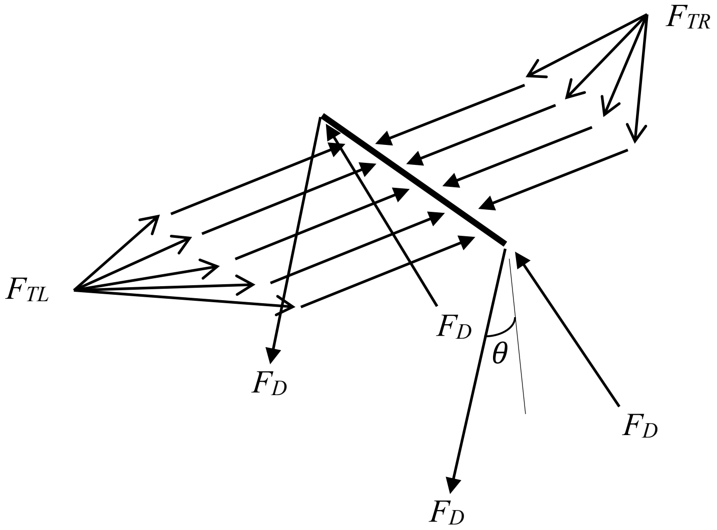

Figure 4.9: Joint beam forces.

Forces acting on the diagonal members were resolved into horizontal components and placed at the end of the members as shown in Figure 4.10 (The distances a, b, c and d are the spacing's between the members in parallel).

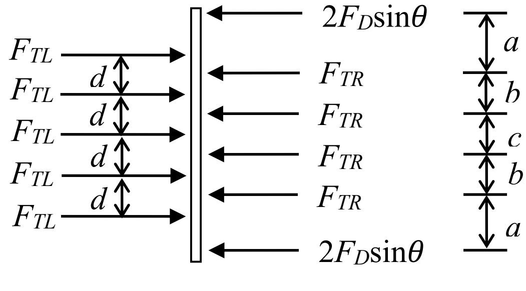

Figure 4.10: Force distribution on the joint beam.

For the horizontal force distribution on the joint beam shown in Figure 4.10 the following conditions should hold:

1. All the forces F_{TR} in the top members on the right side of the joint beam should be equal.
2. All the forces F_{TL} in the top members on the left side of the joint beam should be equal.
3. Equilibrium: $4F_{TR} + 4F_{D}sin\theta = 5F_{TL}$.

Starting with the forces on the left side F_{TL} being equal and equidistantly spaced it is possible to theoretically compute the distances a, b, c and d on the right side such that all the forces F_{TR} are the same. However, numerical experiments were carried out with horizontal forces (both F_{TR} and F_{TL}) being equidistantly spaced and only varying the distance a for the diagonal forces ($F_{D}sin\theta$). Two obervations were made regarding the spacing of the diagonals which gave a distribution of forces that approached the above listed conditions. These observations were:

1. Spacing the diagonals at 30% of the length of the joint beam where the number of members on either side of the joint beam differ (Figure 4.11 b).
2. Spacing the diagonals at 80% of the length of the joint beam where the number of members on both sides of the joint beam were the same (Figure 4.11 a).

It should be noted that these observations were made with respect to the top members. Therefore, the spacing and the position of the diagonal members were determined by the number of top members on either side of the joint beam.

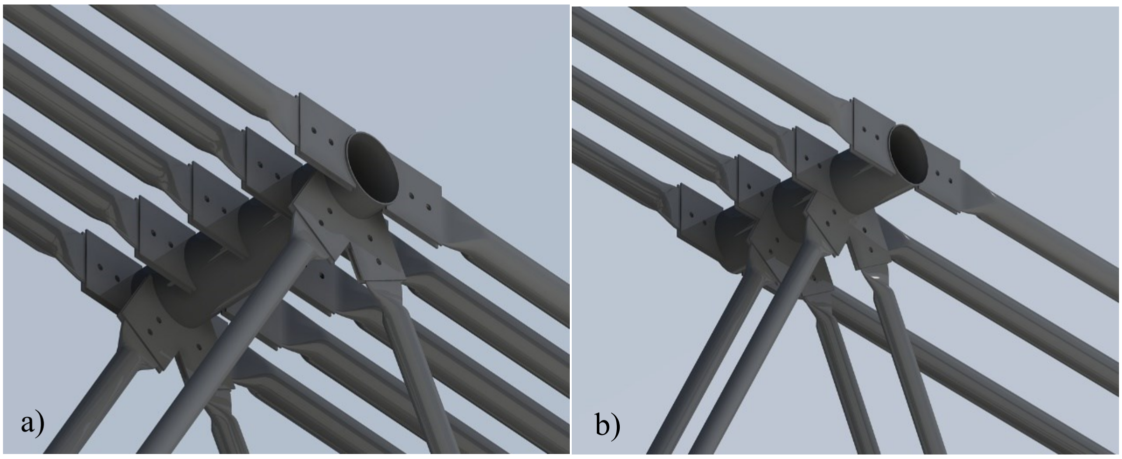

Figure 4.11: Diagonal member spacing. a) Same number of members on either side of the joint beam. b) Different number of members on either side of the joint beam.

4.6.2 Joint conceptualization

The truss members are circular hollow sections. These members were to be flattened at the end and two bolt holes drilled. These members are expected to fit in between two connector plates welded on the joint beam as shown in Figure 4.12.

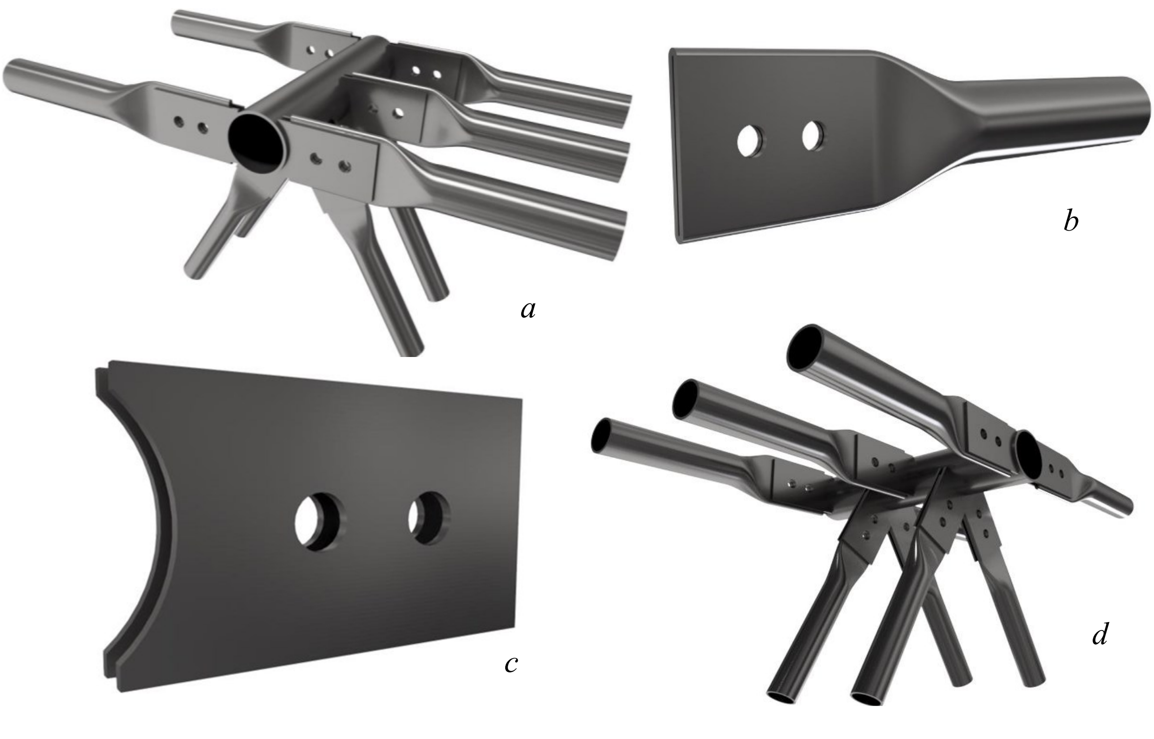

Figure 4.12: Three dimensional joint drawing. a) and d) are the assembled joint. b) Flattened end of a circular hollow section. c) Connector plates that b) fits into.

Using bolts and nuts, the members will be fastened to the plates welded to the joint beams and therefore forming a connection. The connection for the cross beams of the truss also consist of two plates. The flange of the cross beam (which is an I-section) is cut so that the web can fit in between the two plates as shown in Figure 4.13.

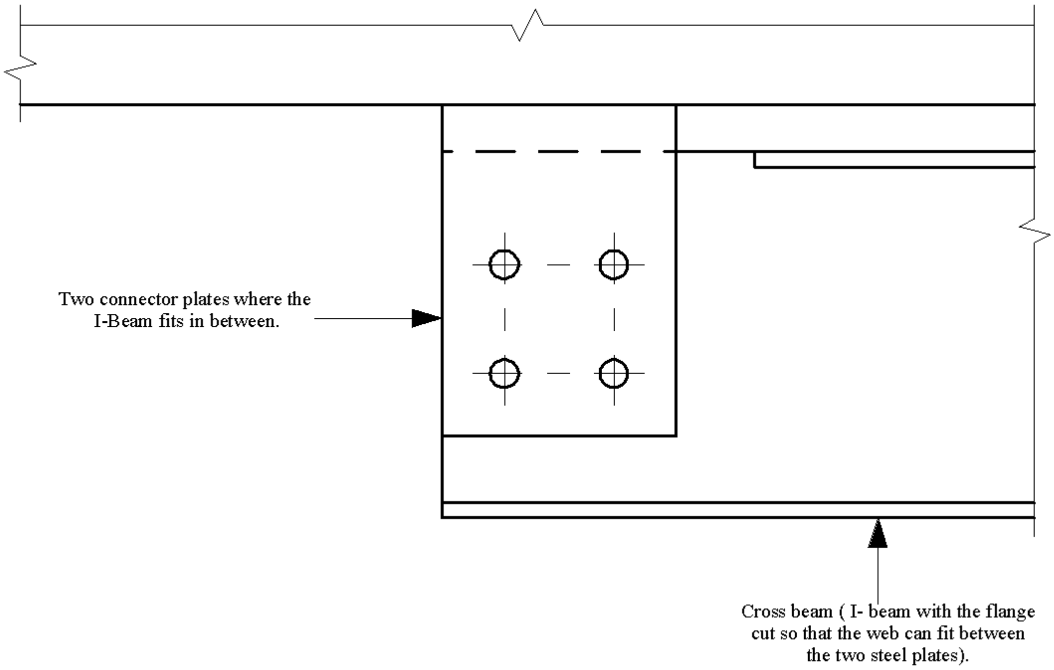

Figure 4.13 Cross beam connection

4.6.3 Joint design

Design of joints was carried out according to the EN 1993-1-8 (EUROCODE 3). Possible failure modes for bolted connections are:

1. Tension resistance.
2. Shear resistance per shear plane.
3. Bearing resistance.
4. Block tearing

The design of the joints was split into two as follows:

1. Bolt design.
2. Plate design.

4.6.3.1 Bolt design

The bolts used to fasten the connections in the footbridge were all loaded in shear. ***The Shear resistance*** per shear plane for ordinary bolts where the shear plane passes through the threaded portion of the bolt is expressed by *Eq.* (4.8).

$$F_{v,Rd} = \alpha_v f_{ub} \frac{A_s}{\gamma_{M2}} \tag{4.8}$$

Where:

$F_{v,Rd}$	N	Shear resistance per shear plane for ordinary bolts.
A_s	mm^2	Tensile stress area of the bolt.
γ_{M2}	$-$	Partial safety factor (for resistance of bolts $\gamma_{M2} = 1.25$).
f_{ub}	$\mathrm{N/mm^2}$	Ultimate tensile strength of the bolt.
α_v	$-$	Reduction factor depending on the bolt class.

In addition, where the shear plane passes through the unthreaded portion of the bolt the value of $\alpha_v = 0.6$ and A_s is replaced by A which is the gross-section area of the bolt. Since it is not known a priori where the shear plane will pass, it is assumed that it passes through the threaded portion of the bolt.

4.6.3.2 Plate design

For connection plates, checks need to be carried out for the following possible failure modes:

1. Bearing resistance.
2. Block tearing.
3. Tension resistance.

4.6.3.2.1 Design for bearing resistance

The ***bearing resistance*** of a connector plate for ordinary bolts is expressed by *Eq. (*4.9*)*.

$$F_{b,Rd} = k_1 \alpha_b \frac{dt}{\gamma_{M2}} \tag{4.9}$$

Where:

$F_{b,Rd}$	N	Bearing resistance for ordinary bolts
d	mm	Bolt diameter
t	mm	Thickness of connected parts
γ_{M2}	$-$	Partial safety factor connections ($\gamma_{M2} = 1.25$).
f_u	$\mathrm{N/mm^2}$	Ultimate tensile strength of connected parts.
α_b	$-$	$\alpha_b = min\left(\frac{e_1}{3d_0}; \frac{f_{ub}}{f_u}; 1.0\right)$ $\alpha_b = min\left(\frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1.0\right)$ $for\ inner\ bolts$
k_1	$-$	$k_1 = min\left(2.8\frac{e_2}{d_0} - 1.75; 2.5\right)$ $for\ edge\ bolts$ $k_1 = min\left(1.4\frac{p_2}{d_0} - 1.75; 2.5\right)$ $for\ inner\ bolts$

Distances e_1 and e_2 are the end and edge distances while the distances p_1 and p_2 are the spacing of fasteners. These distances are illustrated in Figure 4.14.

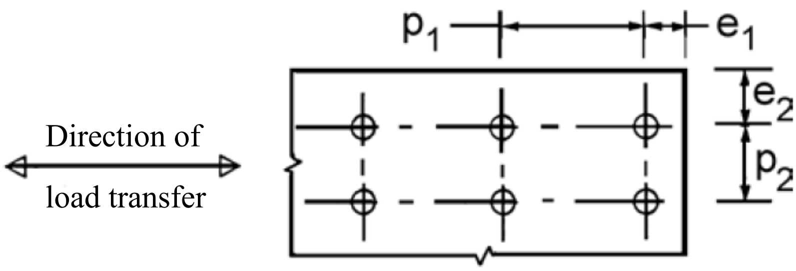

Figure 4.14: End, edge distances and fastener spacing: Fig 3.1, EN 1993-1-8.

4.6.3.2.2 Design for block tearing

According to Eurocode 3, block tearing consists of failure in shear at the row of bolts along the shear face of the hole group accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group. Block tearing is illustrated in Figure 4.15.

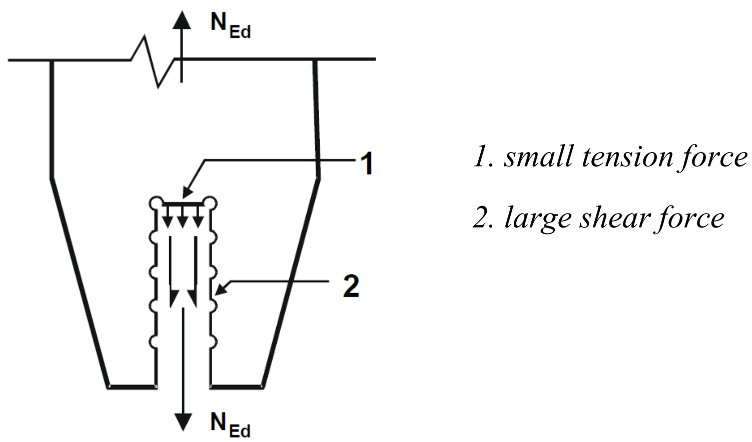

Figure 4.15: Block tearing (fig. 3.8 EN 1993-1-8)

The formula used to check for ***block tearing***, *Eq.* (4.10), is:

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \frac{f_y A_{nv}}{\sqrt{3}\,\gamma_{M0}} \tag{4.10}$$

Where:

$V_{eff,1,Rd}$	N	Design block tearing resistance.
f_u	$\mathrm{N/mm^2}$	Ultimate tensile strength of connected parts.
f_y	$\mathrm{N/mm^2}$	Yield strength of connected parts
A_{nt}	$\mathrm{mm^2}$	Net area subjected to tension.
A_{nv}	$\mathrm{mm^2}$	Net area subjected to shear.
γ_{M2}, γ_{M0}	$-$	Partial safety factors.($\gamma_{M2} = 1.25, \gamma_{M0} = 1.0$)

4.6.3.2.3 Design for tension resistance

The connector plates were designed as members loaded in tension. The ***design tension resistance*** is given by *Eq.* (4.11).

$$the\ lesser\ of\ N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}}\ or\ N_{u,Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}} \tag{4.11}$$

Where:

$N_{pl,Rd}$	N	Design plastic resistance of the plate.
$N_{u,Rd}$	N	Design ultimate resistance of the plate.
A	mm^2	Gross area of the cross-section.
A_{net}	mm^2	Net are of the cross-section (gross area less area of bolts).
γ_{M2}	$-$	Partial safety factor (for resistance of bolts $\gamma_{M2} = 1.25$).
f_y	N/mm^2	Yield strength of the plate.
f_u	N/mm^2	Ultimate tensile strength of the plate.

These formulae were programmed in an excel worksheet to simplify the design of the bolted connections. Truss P was one of the trusses that was selected to constitute the catalogue. The calculations for bolted connections for members of truss P are presented in APPENDIX 1 because it had the largest internal force in its members for the 1.83 m wide footbridge. Also presented in APPENDIX 1 is the design for bolted connections for the cross beams of truss P for the 1.83 m wide footbridge using the same design guidelines.

4.6.3.3 Checks for the ends of the truss members

The ends of the members of the truss that were under tension were checked using *Eq.* (4.11). In addition, the ends of members that were under compression were also checked for buckling resistance using *Eq.* (4.12).

$$N_{cr} = \frac{\pi^2 EI}{KL} \; ; \bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \; ; N_{bRd} = \frac{\chi A f_y}{\gamma_{M1}} \tag{4.12}$$

Where:

N_{cr}	N	Euler critical buckling load.
N_{bRd}	N	Design buckling resistance.
A	mm^2	Gross area of the cross-section.
E	$\mathrm{N/mm}^2$	Young's Modulus.
γ_{M1}	$-$	Partial safety factor.
f_y	$\mathrm{N/mm}^2$	Yield strength.
I	mm^4	Moment of inertia.
χ	$-$	Reduction factor for buckling. (from figure 6.4 EN1993-1-1)
$\bar{\lambda}$	$-$	Non-dimensional slenderness.

The details for the calculations are presented in APPENDINX 1 for truss P for the 1.83 m wide footbridge. This truss had the greatest internal forces.

4.6.3.4 Welded joints

The design of welded joints was done according to EN 1993-1-8 (EUROCODE 3). Stresses in the conventional section were determined then transformed to stresses in the plane of the throat. Figure 4.16 shows the stresses acting on a weld.

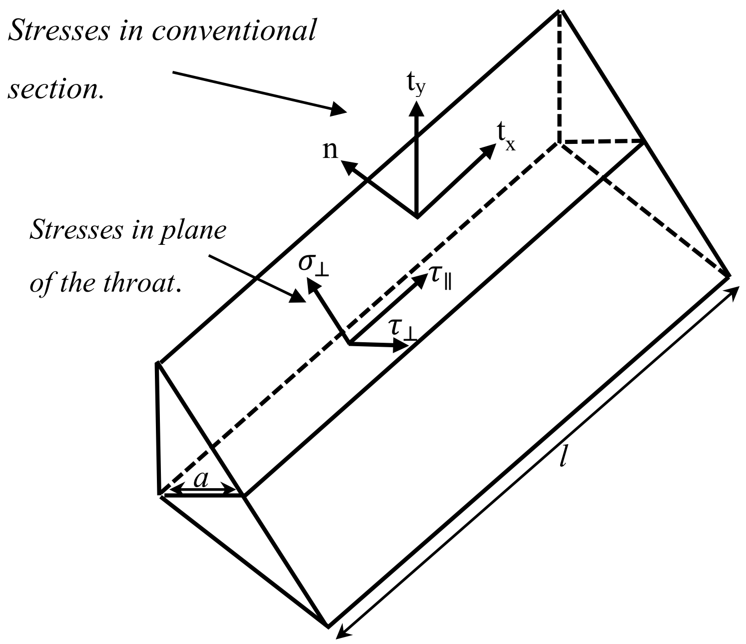

Figure 4.16: Stresses acting on a weld.

Stresses in the throat of the weld was obtained by *Eq.* (4.13).

$$\tau_\parallel = t_x \ ; \ \sigma_\perp = \frac{n + t_y}{\sqrt{2}} ; \ \tau_\perp = \frac{-n + t_y}{\sqrt{2}} \tag{4.13}$$

Where:

$\sigma_\perp$	$\mathrm{N/mm^2}$	Normal stress perpendicular to the throat.
$\tau_\perp$	$\mathrm{N/mm^2}$	Shear stress (in the plane of the throat) perpendicular to the axis of the weld.
$\tau_\parallel = t_x$	$\mathrm{N/mm^2}$	Shear stress (in the plane of the throat) parallel to the axis of the weld.
n	$\mathrm{N/mm^2}$	Normal stress perpendicular to the conventional section.
t_y	$\mathrm{N/mm^2}$	Shear stress (in the plane of conventional section) perpendicular to the axis of the weld.

The combined stress was given by *Eq.* (4.14).

$$\sigma_c = \sqrt{\sigma_\perp{}^2 + 3(\tau_\perp{}^2 + \tau_\parallel{}^2)} \tag{4.14}$$

The design resistance of a fillet weld was checked by *Eq.* (4.15).

$$\sigma_c \leq \frac{fu}{\beta_w \gamma_{M2}} \quad and \ \sigma_\perp \leq 0.9 \frac{fu}{\gamma_{M2}} \tag{4.15}$$

Where:

fu	$\mathrm{N/mm^2}$	Nominal ultimate tensile strength of the weaker part joined.
β_w	$-$	Correlation factor (0.8 for S235)
γ_{M2}	$-$	Partial safety factor for welded connections (1.25).

The formulae presented above for the design of welded connections according to Eurocode 3 part 1-8 were programmed in an Excel worksheet to simplify the design process. Details of calculations for the welded connections for truss P are summarized in APPENDIX 2. Truss P for the 1.83 m wide footbridge is selected because it had the highest internal force in its members.

CHAPTER FIVE

5.0 RESULTS AND DISCUSSIONS

5.1 Introduction

In this chapter, the findings of the research are presented. The weight of the trusses from the global analysis and selected optimal trusses that form the content of the catalogue are presented. Also presented are the sections that were selected for members of these trusses and the number of elements in parallel for each of the fields of the trusses. Drawings and details are also provided to show the various dimensions of truss elements and connection details.

5.1.1 Analysis of the loading on the footbridge according to Eurocodes.

A global analysis of a large number of footbridges was carried out. Modelling of the footbridges in SCIA Engineer (i.e. boundary conditions and the points of application of the loads) was done as explained in 4.4.3. Trusses that constitute the footbridges were modelled as three dimension frames. Cross beams, joint beams and deck beams were modelled as explained in 4.4.3 such that shear forces, moments and torques can be developed in them. The results obtained from the analysis of the loading on the footbridges were shear forces, bending moments and axial loads in the members which were used for the design of the structural elements that constitute the footbridge (i.e. the truss members, cross beams, joint beams, and deck beams). Checks done during the design by SCIA Engineer were section and stability checks as explained in 4.4.3.1.The alternative truss design (the one consisting of members in parallel) consists of very many elements (see Figure 5.1 together with Table 5.3 and Table 5.4). In addition, the entire footbridge was modelled for two cases i.e. the classical approach and the alternative approach. A total of 72 foot bridges were analysed (Trusses numbered A to R are 18 and taking into account the two deck widths gives a total of 36. Considering the classical and the alternative designs results in 72 trusses, Table 4.2). The results of these trusses were too much information to be presented in this document. Therefore, the SCIA Engineer files for these trusses were written on a disc which is attached in the document.

5.1.2 Design of the bridge deck and supporting cross beams

The bridge deck and supporting beams for each of the footbridges were designed using SCIA Engineer according to the Eurocodes. The member sections shown in Table 5.1 and Table 5.2 (refer to section 4.5) were obtained for the deck frame and supporting beams (or cross beams, see Figure 4.7), respectively. Since the entire footbridge was modelled and analysed as a whole, the bridge deck was designed using the loads determined in the Ultimate Limit State (bending moments and shear forces in the beams computed in SCIA since the whole footbridge was modelled). Section checks that were performed for the deck beam include shear, bending moment and combined bending, axial and shear force checks were done by SCIA Engineer. Stability checks included lateral torsional buckling and shear buckling checks.

Table 5.1: Bridge deck frame sections for 1.22 m wide and 1.83 m wide footbridges

	Field Width					
	3/8 U	**4/8 U**	**5/8 U**	**6/8 U**	**7/8 U**	**8/8 U**
Truss	**A**	**B**	**C**	**D**	**E**	**F**
1.22 m	U40x20x4	U40x20x4	U40x20x4	U40x20x5	U50x25x5	U50x25x5
1.83 m	U40x20x4	U40x20x4	U40x20x4	U40x20x5	U50x25x5	U50x25x5
Truss	**G**	**H**	**I**	**J**	**K**	**L**
1.22 m	U40x20x4	U40x20x4	U40x20x4	U40x20x5	U50x25x5	U50x25x5
1.83 m	U40x20x4	U40x20x4	U40x20x4	U40x20x5	U50x25x5	U50x25x5
Truss	**M**	**N**	**O**	**P**	**Q**	**R**
1.22 m	U40x20x4	U40x20x4	U40x20x4	U40x20x5	U50x25x5	U50x25x5
1.83 m	-	U40x20x4	U40x20x4	U40x20x5	U50x25x5	U50x25x5

Section checks performed for the supporting beams (or cross beams) were shear, bending moment and combined bending, axial and shear force checks. Stability checks performed for the cross beams were the lateral torsional buckling and shear buckling checks.

Table 5.2: Supporting beam sections (cross beams) for 1.22 m wide and 1.83 m wide footbridges.

	Field Width					
	3/8 U	**4/8 U**	**5/8 U**	**6/8 U**	**7/8 U**	**8/8 U**
Truss	**A**	**B**	**C**	**D**	**E**	**F**
1.22 m	IPE80AA	IPE80AA	IPE80AA	IPE80AA	IPE80	IPE100AA
1.83 m	IPE80A	IPE80	IPE100AA	IPE100	IPE120AA	IPE120
Truss	**G**	**H**	**I**	**J**	**K**	**L**
1.22 m	IPE80AA	IPE80AA	IPE80AA	IPE80	IPE80	IPE100AA
1.83 m	IPE80	IPE100AA	IPE100AA	IPE100	IPE120AA	IPE120
Truss	**M**	**N**	**O**	**P**	**Q**	**R**
1.22 m	IPE80AA	IPE80AA	IPE80AA	IPE80	IPE100AA	IPE100AA
1.83 m	-	IPE100	IPE100	IPE120AA	IPE120	IPE120

In Table 5.1 and Table 5.2, the field width corresponds to the spacing of the cross beams of the truss (or supporting beams), see Figure 4.8. It is also the dimension of the equilateral triangles that make up the trusses. From Table 5.1, it can be observed that for every field width (i.e. **3/8U**, **4/8U**, **5/8U** and so on) trusses have the same member sections for the bridge deck frame. This is because the bridge deck is supported at the same positions for a given field width. Since the loading on the deck is the same (i.e. 5 kN/m^2) for all the footbridges, the resultant moments and forces are the same leading to same cross sections. For larger field widths (i.e. **6/8U**, **7/8U** and **8/8U**) cross sections with higher moment of inertia were required because of the larger spans over which the deck frame is supported. The heaviest deck unit was found to be 672 N.

A similar trend can also be observed in Table 5.2 for the supporting beams (also called stringers or cross beams). The load on the supporting beam (or cross beam) is proportional to the width of the footbridge. Therefore, a 1.83 m wide footbridge will be subjected to a greater load than a 1.22 m wide footbridge of the same length. As a result the supporting beams (or cross beams) for a 1.83 m wide footbridge will be of greater moment of inertia than those of a 1.22 m wide footbridge of the same length. I-sections were used for the supporting beams because they have a high moment of inertia which makes them suitable for resistance of flexural forces. They are however susceptible to lateral torsional buckling (which was taken care of during the design in SCIA Engineer).

Finally, for a given footbridge, the sections for the deck frame and supporting beams can be selected from Table 5.1 and Table 5.2 respectively and purchased from the local hardware stores for fabrication purposes. Equivalent sections can also be used if the tabulated ones cannot be obtained. Any equivalent sections should be verified according to the Eurocode design specifications before application. In addition, the sections IPE 80AA, IPE 100AA and IPE 120AA are special cross sections that are available on request to the manufacturer. These sections are present in the database of ArcelorMittal products in SCIA Engineer which automatically selected them as the optimal ones. It is therefore recommended that IPE 80, IPE 100 and IPE 120 (which are heavier, deeper and have higher moment of inertia but more readily available) to be used respectively in such cases.

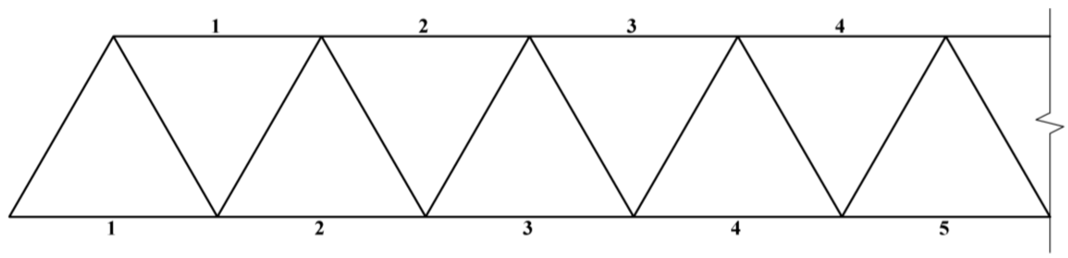

Figure 5.1: Field numbering for half a truss.

5.1.3 Design of the trusses and comparison with classical design

The trusses of the footbridges were also designed according to the Eurocodes using SCIA Engineer. This was done for both the classical truss and the alternative truss (consisting of members in parallel and including its joint beams). When designing the classical trusses, the most critically loaded member was used to determine the cross sections for the top members. Similarly, sections were determined for the diagonal and the bottom members. The geometrical configuration of the trusses consisting of members in parallel are shown in Table 5.3 and Table 5.4 for the 1.83 m and 1.22 m wide footbridges, respectively. Table 5.3 and Table 5.4 show the number of members in parallel in each field for half of each truss since they are symmetrical (the fields are numbered as shown in Figure 5.1). Using member sections adopted during the design, the mass of the trusses were computed (in SCIA Engineer) and tabulated for the purpose of comparison (see Table 5.5 and Table 5.6).

Table 5.3: Number of elements in parallel for the 1.83 m wide footbridge trusses.

Truss	n	Loc.	1	2	3	4	5	6	7	8	9	10	11	12
A	16	top	2	2	3	4	4	4	5					
		bot	2	2	3	4	4	5	5	5				
B	12	top	2	3	4	5	5							
		bot	2	3	4	4	5	5						
C	10	top	2	3	4	4								
		bot	2	3	4	5	5							
D	8	top	2	3	4									
		bot	2	3	4	5								
E	7	top	3	4	5									
		bot	2	3	4	4								
F	6	top	3	4	5									
		bot	2	3	4									
G	24	top	2	2	2	3	3	4	4	4	5	5	5	
		bot	2	2	2	3	3	4	4	5	5	5	5	5
H	18	top	2	2	3	3	4	4	5	5				
		bot	2	2	3	3	4	4	5	5	5			
I	14	top	2	3	3	4	5	5						
		bot	2	2	3	4	5	5	5					
J	12	top	2	3	4	4	5							
		bot	2	3	4	4	5	5						
K	10	top	2	3	4	5								
		bot	2	3	4	5	5							
L	9	top	2	4	5	5								
		bot	2	3	4	5	5							
M	32	top	Too many members											
		bot	Too many members											
N	24	top	2	2	2	3	3	4	4	4	5	5	5	
		bot	2	2	2	3	3	3	4	4	4	5	5	5
O	19	top	2	2	3	3	4	4	5	5	5			
		bot	2	2	3	3	4	4	5	5	5	5		
P	16	top	2	2	3	4	4	5	5	5				
		bot	2	2	3	4	4	5	5	5				
Q	14	top	2	2	3	4	4	4	4					
		bot	2	2	3	4	5	5	5					
R	12	top	2	2	3	4	4	5						
		bot	2	2	3	4	4	5						

Table 5.4: Number of elements in parallel for the 1.22 m wide footbridge trusses.

Truss	n	Loc.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
A	16	**top**	2	3	3	4	5	5	5	5								
		bot	2	2	3	4	4	5	5	5								
B	12	**top**	2	3	3	4	4	4										
		bot	2	2	3	4	5	5										
C	10	**top**	2	3	4	4	5											
		bot	2	2	3	4	4											
D	8	**top**	3	4	5	5												
		bot	2	2	3	3												
E	7	**top**	2	3	4													
		bot	2	2	3	3												
F	6	**top**	2	4	4													
		bot	2	2	3													
G	24	**top**	2	2	2	3	3	4	4	5	5	5	5	5				
		bot	2	2	2	3	3	4	4	4	5	5	5	5				
H	18	**top**	2	2	3	3	4	4	4	4	4							
		bot	2	2	3	4	4	5	5	5	5							
I	14	**top**	2	3	4	4	5	5	5									
		bot	2	2	3	4	5	5	5									
J	12	**top**	2	3	4	4	5	5										
		bot	2	3	4	4	5	5										
K	10	**top**	2	3	4	4	5											
		bot	2	3	4	5	5											
L	9	**top**	2	4	4	5												
		bot	2	3	4	5	5											
M	32	**top**	2	2	2	2	3	3	4	4	4	4	4	5	5	5	5	5
		bot	2	2	2	2	3	3	3	4	4	4	4	5	5	5	5	5
N	24	**top**	2	2	2	3	3	4	4	4	5	5	5	5				
		bot	2	2	2	3	4	4	4	5	5	5	5	5				
O	19	**top**	2	2	3	4	4	5	5	5	5	5						
		bot	2	2	3	3	4	4	5	5	5	5						
P	16	**top**	2	2	3	4	4	5	5	5								
		bot	2	2	3	4	4	5	5	5								
Q	14	**top**	2	3	3	4	4	5	5									
		bot	2	2	3	4	4	5	5									
R	12	**top**	2	3	4	5	5	5										
		bot	2	3	3	4	4	5										

Table 5.5 Comparison of mass of the classical and alternative trusses of 1.83 m wide footbridges.

Truss	***n***	***H* (m)**	***L* (m)**	***Mass of classical truss* (Kg)**	***Mass of classic truss + joints* (Kg)**	***Mass of parallel member truss* (Kg)**	***% Saved by parallel member truss***
A	16	0.7924	14.64	752.7	827.97	682.7	21.3
B	12	1.0566	14.64	707.2	777.92	640	21.6
C	10	1.3207	15.25	645.1	709.61	661.4	7.3
D	8	1.5848	14.64	630.8	693.88	616.5	12.6
E	7	1.8490	14.945	550.4	605.44	638.3	-5.1
F	6	2.1131	14.64	447.4	492.14	686.8	-28.3
G	24	0.7924	21.96	2357.4	2593.14	2220.4	16.8
H	18	1.0566	21.96	1898.2	2088.02	1845.5	13.1
I	14	1.3207	21.35	1511.6	1662.76	1440	15.5
J	12	1.5848	21.96	1441.8	1585.98	1491.1	6.4
K	10	1.8490	21.35	1187.9	1306.69	1334.8	-2.1
L	9	2.1131	21.96	1115.5	1227.05	1506.5	-18.5
M	32	0.7924	29.28	5561.3	6117.43	Too many members	
N	24	1.0566	29.28	4105	4515.5	5063.7	-10.8
O	19	1.3207	28.975	3501.2	3851.32	3206.8	20.1
P	16	1.5848	29.28	3044.4	3348.84	2999.3	11.7
Q	14	1.8490	29.89	3149.5	3464.45	2883	20.2
R	12	2.1131	29.28	2443.6	2687.96	2554.9	5.2

Table 5.5 and Table 5.6 summarize the percentage of material saved by the alternative truss when compared to the classical truss for the 1.83 m wide footbridges and the 1.22 m wide footbridges, respectively. However, there are cases where the classical truss saves material when compared to the alternative truss i.e. the cases where the percentage saved by use of parallel members are negative values in Table 5.5 and Table 5.6.

From the results tabulated in Table 5.5 and Table 5.6, it can be observed that the alternative truss saves 5.2% to 21.6% of material for the 1.83 m wide footbridges (see Table 5.5). It also saves 6.2% to 26.4% of material for 1.22 m wide footbridges (see Table 5.6).

Table 5.6: Comparison of mass of the classical and alternative trusses of 1.22 m wide footbridges.

Truss	***n***	***H* (m)**	***L* (m)**	***Mass of classical truss* (Kg)**	***Mass of classic truss + joints* (Kg)**	***Mass of parallel member truss* (Kg)**	***% Saved by parallel member truss***
A	16	0.7924	14.64	526.2	578.82	500.2	15.7
B	12	1.0566	14.64	506	556.6	500.2	11.3
C	10	1.3207	15.25	475	522.5	565.8	-7.7
D	8	1.5848	14.64	479.2	527.12	544.3	-3.2
E	7	1.8490	14.945	462.9	509.19	557	-8.6
F	6	2.1131	14.64	424	466.4	579.9	-19.6
G	24	0.7924	21.96	1661.3	1827.43	1587.1	15.1
H	18	1.0566	21.96	1241	1365.1	1119.7	21.9
I	14	1.3207	21.35	1058.6	1164.46	1035.4	12.5
J	12	1.5848	21.96	1119.2	1231.12	1152.8	6.8
K	10	1.8490	21.35	881.7	969.87	1173.5	-17.4
L	9	2.1131	21.96	862.4	948.64	1507.8	-37.1
M	32	0.7924	29.28	3732.6	4105.86	3864.3	6.3
N	24	1.0566	29.28	2999.3	3299.23	2623.8	25.7
O	19	1.3207	28.975	2717	2988.7	2365	26.4
P	16	1.5848	29.28	2130.3	2343.33	2207	6.2
Q	14	1.8490	29.89	2372.2	2609.42	2169	20.3
R	12	2.1131	29.28	1899.5	2089.45	2760	-24.3

From the results, it is evident that most of the slender trusses of the alternative design (the ones with large number of fields (n)) showed the most savings in weight except for truss N for the 1.83 m wide footbridge and truss M for the 1.22 m wide footbridge. For example, the first group of trusses (A, B, C, D, E, and F) have reducing number of fields (i.e. 16, 12, 10, 8, 7, and 6, respectively). Since they have similar lengths, truss A consists of equilateral triangles of 0.915 m sides and truss F consists of equilateral triangles of 2.44 m sides. In other words, truss A is slender and truss F is stocky. The members in truss A develop higher internal forces in its members than those of truss F for the same loading and length. Members in compression for truss F are more susceptible to buckling even when they have small internal forces developed than those members in compression for truss A. Truss F, therefore, required a cross section with a high moment of inertia which resulted in heavier cross sections than those of truss A for elements under compression.

As much as truss N for the 1.83 m wide footbridge is slender it doesn't save weight. The high internal forces developed in this truss coupled with the short length of its elements results in the compression members being susceptible to crushing and therefore requiring cross sections with larger areas and hence increased weight. The same explanation applies to truss M for the 1.22 m wide footbridge. Also the elements under tension experience great forces which require heavier cross sections that will provide sufficient area.

A general observation is that as the number of fields increase for a given length (or group of trusses) the weight of the alternative trusses start decreasing and then they increase i.e. the weight is greater for the extreme ends of the group where the number of fields (n) is largest and least. This observation is important for the 29 m long trusses (M, N, O, P, Q and R) which show some exceptions. When the number of fields (n) are large for the 29 m long trusses, crushing loads take effect and minimal weight savings are registered. On the other hand when the number of fields (n) are small, buckling comes into play and minimal weight savings are observed. This is more pronounced for the 29 m long trusses because they are relatively long compared to the other lengths of trusses i.e. 14 m and 21 m.

Truss R for the 1.83 m wide footbridge is the only case where the stocky truss saved 5% by weight. Truss R may be considered stocky in the group of 29 m long trusses, but it also still has a large number of fields (n=12) compared to the stocky trusses for the other two groups. For the classical truss design, most savings were registered for the stocky trusses. The alternative design fails in the case of the stocky trusses since it is not possible to divide the already low internal forces developed in the truss members so that a number of sections in parallel are used.

Using the results obtained from Table 5.5 and Table 5.6, six trusses were selected based on the weight of the truss, constructability considerations and slenderness to form a catalogue of trusses.

5.1.3.1 Comparison with a fully stressed design

Another comparison was also done where the volume of the truss consisting of members in parallel was compared with the volume of a truss that is "fully stressed". The "fully stressed" truss was modelled and analysed in SCIA Engineer as a frame whereby all forces developed in the members were considered for design. The boundary conditions were set to be fixed at one end and sliding at the opposite end. Each of the members of the "fully stressed" truss were designed according to the Eurocodes. During design, a cross section was selected for each of the fields of the truss from the database of cross sections available in SCIA Engineer (Cold formed circular hollow sections were used). Therefore, each member of the truss had different cross sections (with symmetry about the centre of the truss). The analysis and design of the truss that is "fully stressed" was done for truss A for the 1.83 m wide footbridge where the mass was found to be 512 Kg. The mass of truss A for the alternative design was earlier found to be 682.7 Kg (Table 5.5). The mass of the "fully stressed" truss was taken to be M and therefore the mass of the alternative truss was 1.3334M. It was then found that the alternative design can achieve 75 % fully stressed design ($100\% \times 1/1.3334$). The comparison was only done for truss A to represent the slender trusses which were found to be more advantageous when using the alternative design. Computing a fully stressed design in SCIA for the remaining 35 trusses, which have different dimensions and number of fields, is not necessary since some of them were already eliminated.

When computing the "fully stressed" design in SCIA Engineer, different cross sections were used for each of the top and bottom fields. The same was also done for each of the diagonal elements. This approach was adopted to achieve as close as possible to 100 percent fully stressed design. It is difficult to obtain a theoretically 100% fully stressed design since steel manufactures cannot produce every possible dimension of cross section that would be required (as evident in the database of SCIA Engineer). In addition, SCIA Engineer takes into account a number of factors which are prescribed in the Eurocodes i.e. member imperfections, material imperfections and slenderness of the elements which are expected to be present in a practical element under compression. The "fully stressed" design for truss A is found to be lighter because different cross sections have been used between all the fields, thereby ignoring constructability and resistance to overall lateral torsional buckling of the truss (this is to try and achieve as close as possible to theoretically 100% fully stressed design).

5.1.4 Catalogue of trusses

5.1.4.1 Truss Selection

During global analysis, it was realized that trusses C, I, O, E, K and Q pose a challenge in the placement of the bridge deck frames and were eliminated. The main reason was that the length of these trusses divided by the length of a bridge deck frame results in a fraction (instead of an integer). For example, the length of truss C is 15.25 m and each bridge deck frame is 2.44 m. Therefore, truss C, required 6.25 bridge deck frames on a row along its length. For the purpose of analysis, though not ideal for construction purposes, the number of bridge deck units was rounded up to the next integer and then offset relative to the footbridge. Truss A, on the contrary, has a length of 14.64 m and required exactly 6 bridge deck frames on one row along its length. Selected trusses were the ones with the most savings in weight with the exception of truss B (for 1.83m wide footbridge) which was selected because it had more fields (slender) instead of truss D. Also truss J for the 1.83 m wide footbridge was selected because it weighed the least in the group after the eliminations were done and truss P for the 1.22 m wide footbridge was selected because it weighed the least compared to truss N in the group. For the 1.22m wide footbridge trusses A, H and P were selected and for the 1.83m wide footbridges trusses B, J and P were selected to form the catalogue of trusses.

5.1.4.2 The catalogue

The selected trusses were then designed according to the Eurocodes and the sections for truss members were obtained using the forces obtained in the ultimate limit state. Section checks performed for elements under tension included the normal force, shear, bending moment and combined bending, axial force and shear force checks. Lateral torsional buckling check was done for elements, in addition, as a stability check (though not necessary for circular hollow sections). For the joint beams and elements under compression, section checks included the compression, shear, bending moment and combined bending, axial force and shear force checks. Stability checks for the joint beams and elements in compression included flexural buckling, lateral torsional buckling and combined compression and bending checks. Sections that were obtained for the truss members are given in Table 5.7. The circular hollow sections that are tabulated have the diameter and the thickness of the cross section specified, respectively.

The designation CHSCF stands for cold formed circular hollow sections. The sections were selected from the database of British Standards BS EN 10219-2:1997/Part 2 that is found in SCIA Engineer. Indicated in the table are also the lengths and widths over which the trusses can be applied. These trusses constitute the catalogue of trusses.

Table 5.7: The catalogue of trusses.

1.83m wide footbridge							
TRUSS	***TOP (CHS)***	***DIAGONAL (CHS)***	***BOTTOM (CHS)***	***JOINT BEAM (CHS)***	***BRACING***	***CROSS BEAM***	***LENGTH (m)***
B	CHSCF 42.4/2.0	CHSCF 33.7/2.5	CHSCF 26.9/3	CHSCF 60.3/3	L20x20x3	IPE80	14.7
J	CHSCF 60.3/2.5	CHSCF 60.3/2.0	CHSCF 42.4/2.5	CHSCF 88.9/3	L25x25x3	IPE100	22.0
P	CHSCF 76.1/3.0	CHSCF 76.1/2.0	CHSCF 60.3/3.0	CHSCF 101.6/4.	L25x25x3	IPE120AA	29.3
1.22m wide footbridge							
TRUSS	***TOP (CHS)***	***DIAGONAL (CHS)***	***BOTTOM (CHS)***	***JOINT BEAM (CHS)***	***BRACING***	***CROSS BEAM***	***LENGTH (m)***
A	CHSCF 33.7/2.5	CHSCF 26.9/3.0	CHSCF 26.9/2.5	CHSCF 60.3/3	L20x20x3	IPE80AA	14.7
H	CHSCF 48.3/2.5	CHSCF 42.4/2.0	CHSCF 42.4/2.5	CHSCF 76.1/3	L25x25x3	IPE80AA	22.0
P	CHSCF 76.1/2.0	CHSCF 60.3/2.0	CHSCF 48.3/2.5	CHSCF 76.1/4	L25x25x3	IPE80	29.3

From the large number of trusses, only six trusses were selected. The trusses can be used for bridges of length 14.7 m, 22 m and 29.3 m. For each of those lengths, footbridge widths of 1.83 m and 1.22 m can be constructed. Using the length and width of the footbridge, the section properties can be obtained from Table 5.7. The number of fields in each of the trusses and number of elements per field are detailed in Figure 5.6

In a case where the sections prescribed in the catalogue are not available, alternative equivalent sections can be used. Any alternative sections should be verified according to the Eurocode design specifications before application.

The trusses were also checked to verify that they do not exceed deflection limits, L/250, (EUROCODE 3). The results are tabulated in Table 5.8. From the table it is clear that the displacement of the trusses (δ_v and δ_l) are within the limits. δ_v are the displacements the vertical direction and δ_l are the displacements in the lateral direction. The displacements, δ_l were obtained by considering the load applied to simulate the weight of people acting on the safety barrier. It was assumed that the truss (which is slender) acts as the safety barrier.

From Table 5.8 it can be noted that the trusses met the displacement limits set in the Eurocodes. This means that the trusses have sufficient stiffness under the design loads. Stiffness is an important consideration under the serviceability limit state. Excessive deflections would damage finishes or make pedestrians fear using the footbridge (for example a wooden hand rails, paint or lighting fixtures and electrical conduits for illuminated bridges).

Table 5.8: SLS checks for selected trusses.

1.22 m wide footbridge					***1.83 m wide footbridge***				
TRUSS	***L (mm)***	***L/250 (mm)***	***δ_v (mm)***	***δ_l (mm)***	***TRUSS***	***L (mm)***	***L/250 (mm)***	***δ_v (mm)***	***δ_l (mm)***
A	14640	73.2	37.4	62.1	B	14640	73.2	29.6	36.1
H	21960	109.8	70.2	41.6	J	21960	109.8	45.9	40.4
P	29280	146.4	87.0	58.3	P	29280	146.4	87.7	32.7

5.1.5 Bolts for members and cross beams

The connections for the truss members were designed according to the Eurocodes (see APPENDIX 2). The required bolts for the fastening of the truss members that were obtained from calculations are summarized in Table 5.9. It can be observed that the bolts class and diameter differ for members and cross beams. In practice, it is discouraged to use bolts of different classes. Also, a minimum of 12 mm bolts are used in practice. Therefore it is recommended that 12 mm bolts of class 8.8 to be used to fasten the members of the truss and the cross beams for all the trusses. It is also possible to use 10 mm bolts of class 8.8 for all the other trusses except truss P for the 1.83 m wide footbridge since that bolt size is used in Kenya. The recommended bolts will already satisfy the requirements since they are of greater specifications than the ones obtained from calculations. Finally, it is also recommended that 12 mm bolts of class 8.8 to be used for connecting the bridge deck to the cross beams.

Table 5.9: Bolts for fastening the truss members.

1.22 m wide footbridge		
Truss	***4 bolts per member***	***8 bolts per cross beam***
A	10mm bolts class 5.6	8mm bolts class 5.8
H	10mm bolts class 8.8	8mm bolts class 5.8
P	10mm bolts class 8.8	8mm bolts class 5.8
1.83 m wide footbridge		
Truss	***4 bolts per member***	***8 bolts per cross beam***
B	10mm bolts class 5.6	8mm bolts class 5.8
J	10mm bolts class 8.8	8mm bolts class 5.8
P	12mm bolts class 8.8	8mm bolts class 5.8

5.1.6 Use of members in parallel

Truss members were placed at equal distances and diagonal member spacing was varied to have equal forces in the members. The forces in truss J for the 1.83m wide footbridge is shown in Figure 5.2. Having modelled the truss as a frame, moments, shear and some values of torque are expected in the elements but they have been neglected (but were taken into account during design) since they are low in order of magnitude compared to the axial loads and would be too much information to be displayed. The internal forces in the members were the result of the Ultimate Limit State (ULS) loads applied to the truss. It is clear from the Figure 5.2 that the internal forces in the members are nearly equal in each field of the truss (note that in this case the wind loading was taken into account).Therefore, the conditions that were discussed in chapter 3 regarding the joint modelling were closely approached. A similar distribution of forces was observed in all the other trusses (see APPENDIX 3 for forces in the in trusses in the catalogue). In addition, more information on the forces on all the trusses can be found on the SCIA Engineer files written on the attached disc since there were many trusses that were analysed.

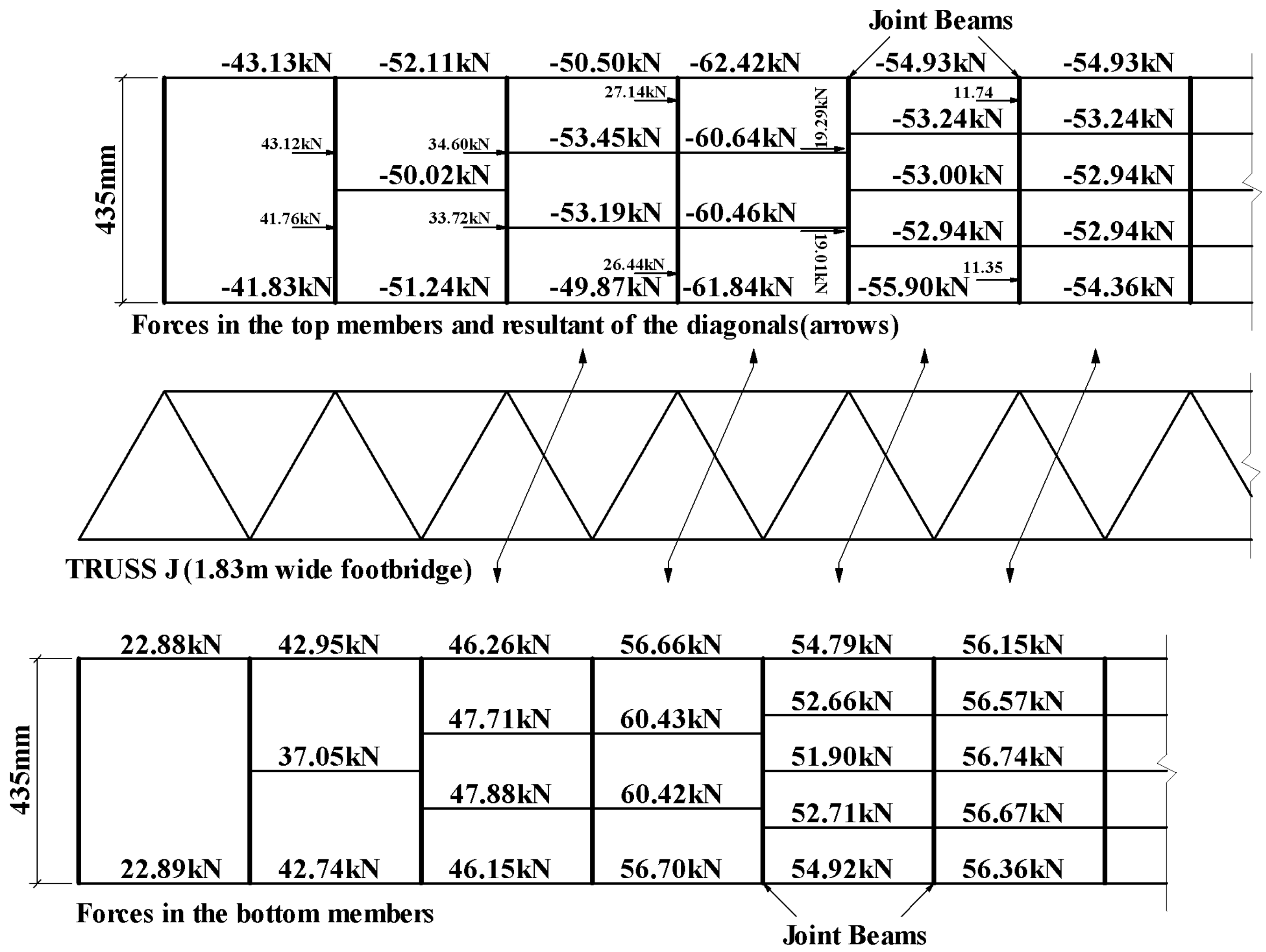

Figure 5.2: Forces in top and bottom and diagonal members of truss J for 1.83m wide footbridge (small values of moments and shear neglected)

5.1.7 Spacing of diagonals

The spacing of diagonals played a very important role in balancing the forces on either side of the truss joint beam, Figure 5.2, and ensuring that members between joint beams had same forces as much as possible. Members considered during determination of diagonal spacing were the top members because they were susceptible to buckling failure. Diagonal members were spaced in two ways (Figure 5.3):

1. Spaced at 80 percent the length of the joint beam where there were equal number of members on either side of the joint beam.
2. Spaced at 30 percent the length of the joint beam where there were different number of members on either side of the joint beam.

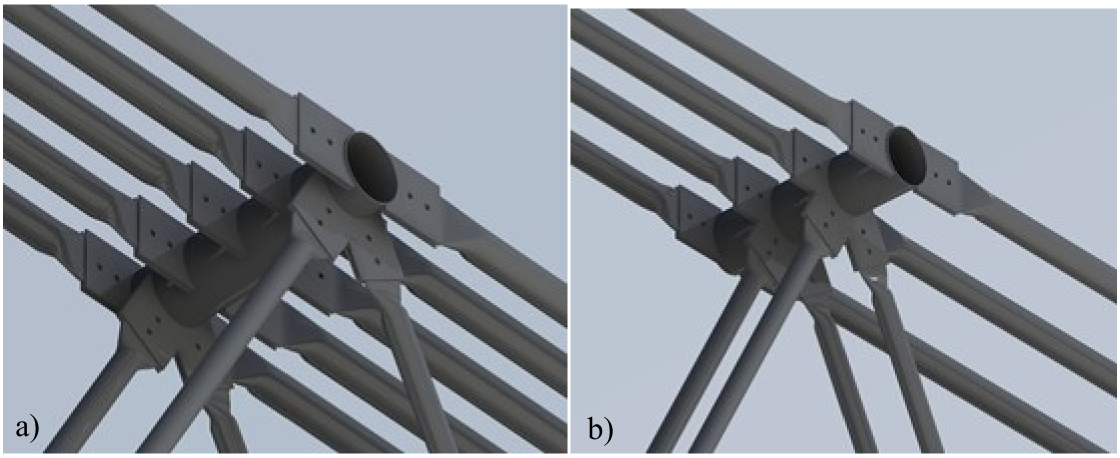

Figure 5.3: Diagonal spacing. a) 80 percent joint beam length. b) 30 percent joint beam length.

5.1.8 Fully Stressed Design

It was intended that a fully stressed design of the truss be achieved as close as possible by use of members in parallel. At the beginning of the study, emphasis was not put on the spacing of the diagonals. However, during the research it was found that the spacing of the diagonals played a great role. Stresses developed in top and bottom members of truss J for 1.8 m wide footbridge are shown in Figure 5.5. The maximum stress for the top members is 146.8 MPa while the least stress is 118.5 MPa giving a difference of 28.3 MPa. In addition, stresses in members between each of the joint beams are very close or in some cases the same. Top members are critical because they are susceptible to buckling failure.

Bottom members do have similar results with the exception of the first two members at the end of the truss. Difference in stresses between the most and least stressed member is 104.6 MPa considering all the members. However, neglecting the first two members the stress difference between the most and least stressed member is 62.5 MPa. The difference in this case is quite large compared to the top members. Figure 5.4 shows results of the unity checks for truss J (for 1.83m wide footbridge). A value of 0.8 means the member is utilized to 80% its capacity. Top members are all utilized to more than 70% of their capacity. Members that are furthest from the centre are utilized to a lower percentage because of the reduction of forces with increase in distance from the centre of the truss. The bottom members are utilized to much lower capacities compared to top members. Most bottom members, with the exception of those members in the first two fields at the supports are utilized to more than 63% their capacities.

From the above results it can be observed that the top members are utilized to greater capacities than the bottom members. The reason is that the bottom members will have two bolt holes. These bolt holes reduce the area of members in tension and may result in tearing. To take care of the effects of tearing at the bolt holes, there was a need to use thicker cross section. In addition, using one cross section for all the bottom members has the effect that some elements will be utilized to lesser capacities (especially for the ones near the supports for a warren truss). Using several different cross sections will pose a challenge in the spacing of the connectors. Therefore, constructability and uniformity is achieved with a penalty of increased cross-sectional area (hence weight) in bottom members. Another contributing factor is that the number of elements in one field was chosen to be the same for both the top and the bottom (see Figure 5.4) for uniformity.

Also, the same explanation (with the exception of tearing at the bolt holes) would apply for the difference in the largest and least utilized elements for the top members. Different cross sections would be selected that would further optimize the truss but this would present challenges in the connections by, for example, having different spacing of connector plates for different members throughout the truss.

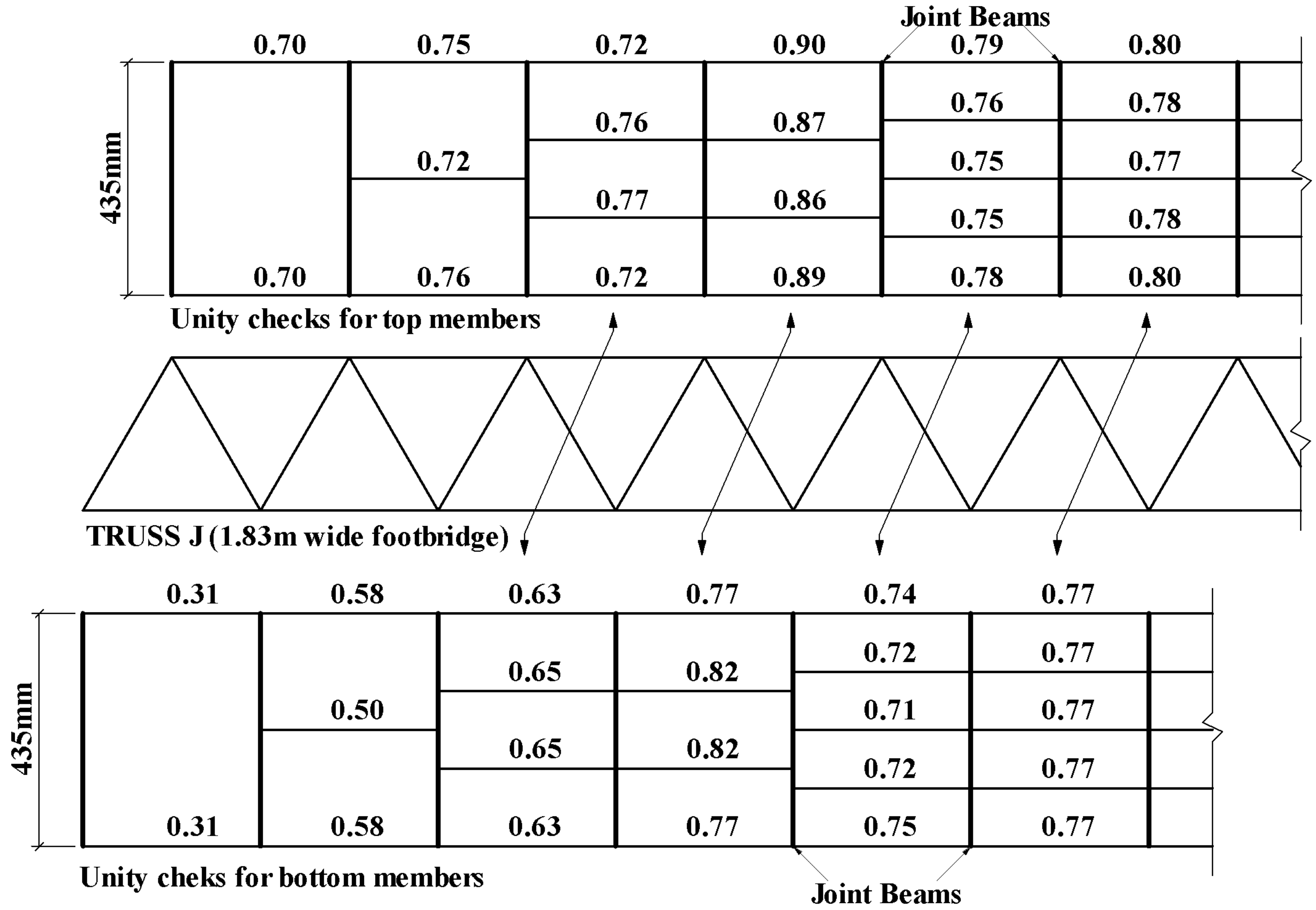

Figure 5.4: Unity checks for truss J for 1.83m wide footbridge (half the footbridge).

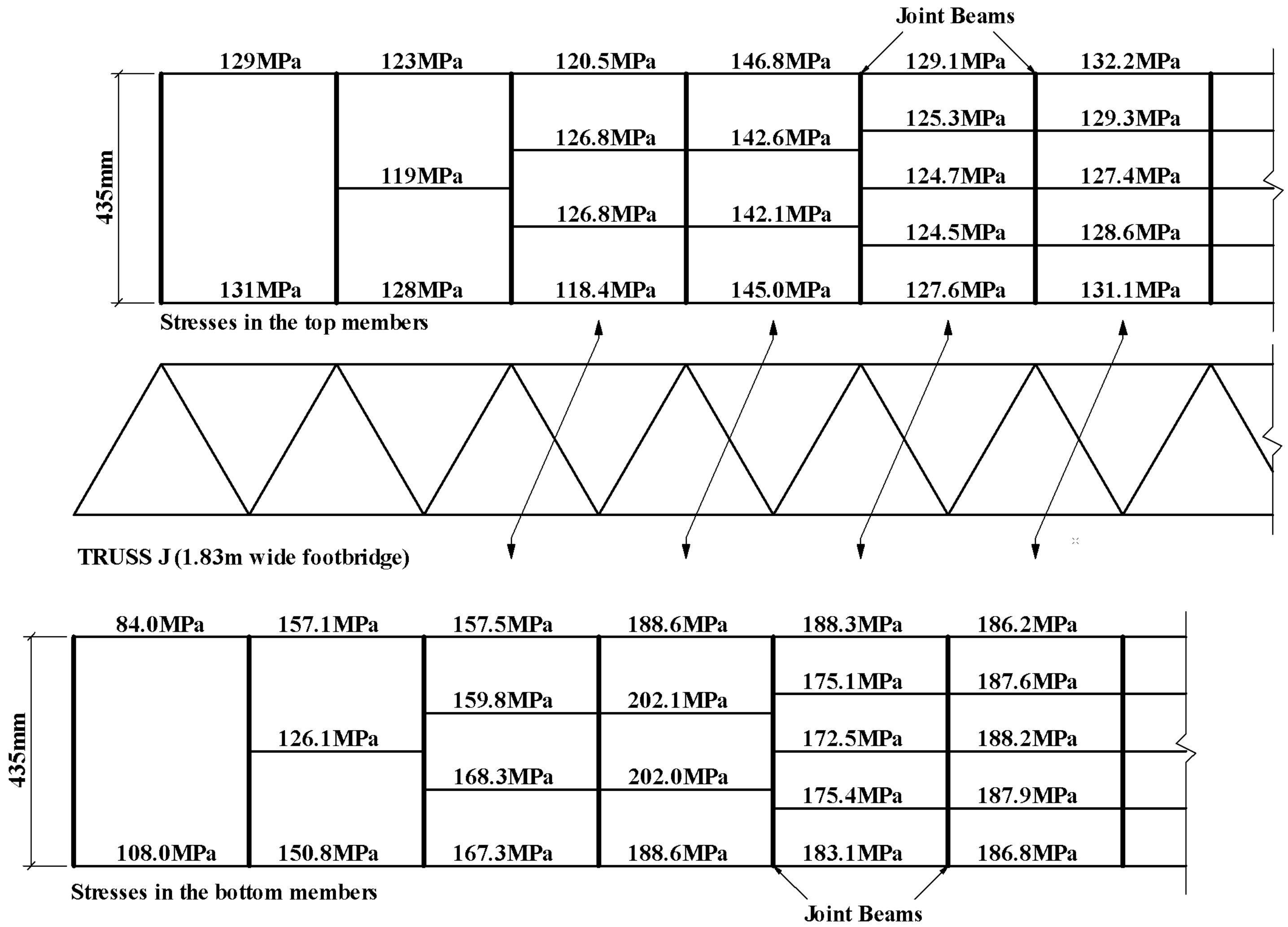

Figure 5.5: Stresses in top and bottom members of truss j for 1.83m wide footbridge.

5.2 Drawings and details

Details of the connections/joints are important for fabrication and assembly of a truss and of course a footbridge. The number of members in various positions in the trusses vary. Figure 5.6 shows the number of members for the trusses which were selected for the catalogue of trusses. The length and sections selected for members of the catalogue of trusses are given in Figure 5.7 and Figure 5.8 for 1.83m wide and 1.22m wide footbridges respectively. The connection details of truss J for the 1.83m wide footbridge are shown in Figure 5.9 and Figure 5.10. Connections details for all the other trusses are similar to that of truss J (see APPENDIX 4). Figure 5.12 shows how the members of truss J are arranged. Since all the members for the individual trusses are of equal length, they form equilateral triangles and are therefore at an angle of 60 degrees with respect to each other.

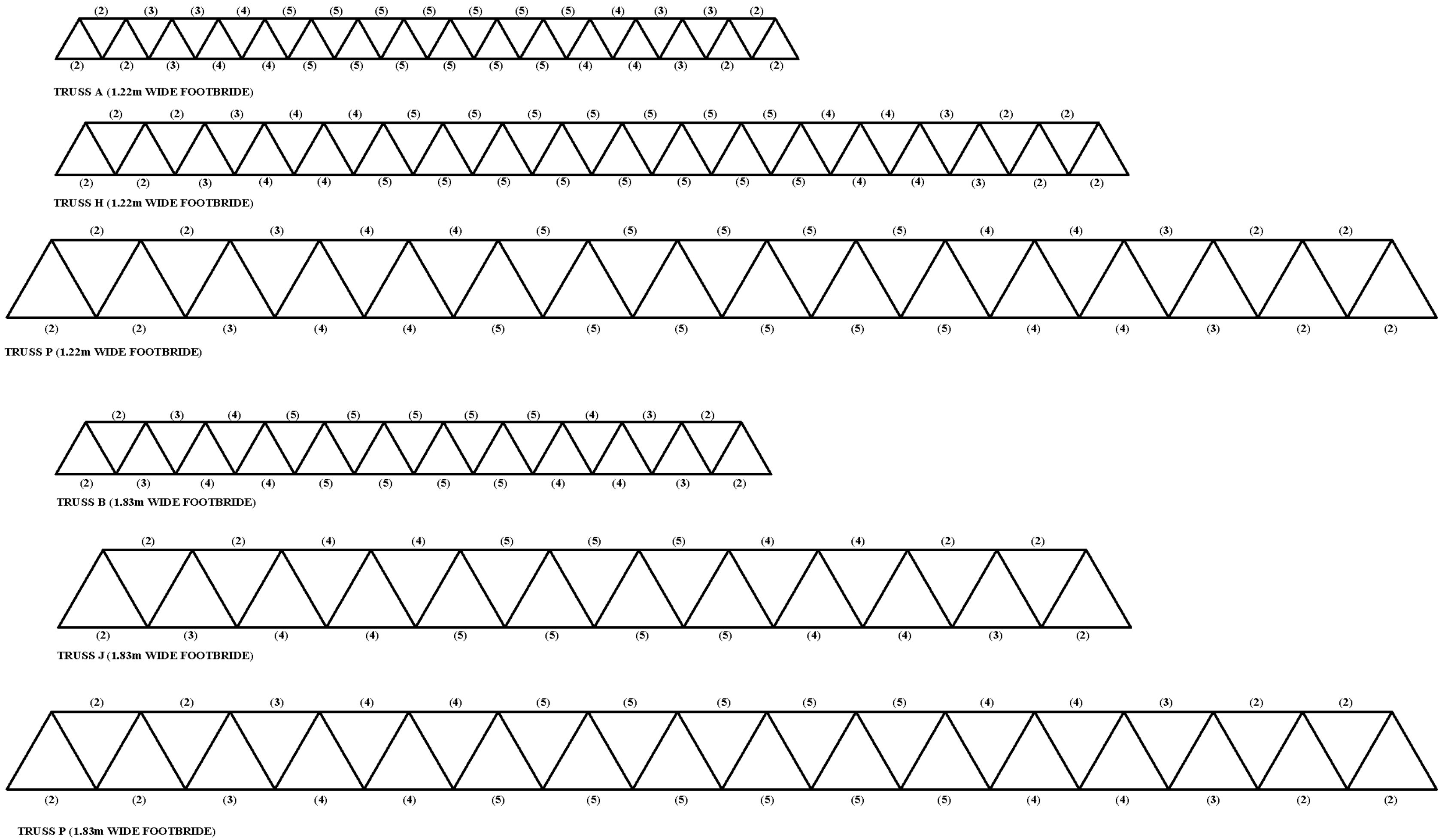

Figure 5.6: Number of members at various positions in the trusses. Only 2 members are used for all the diagonals.

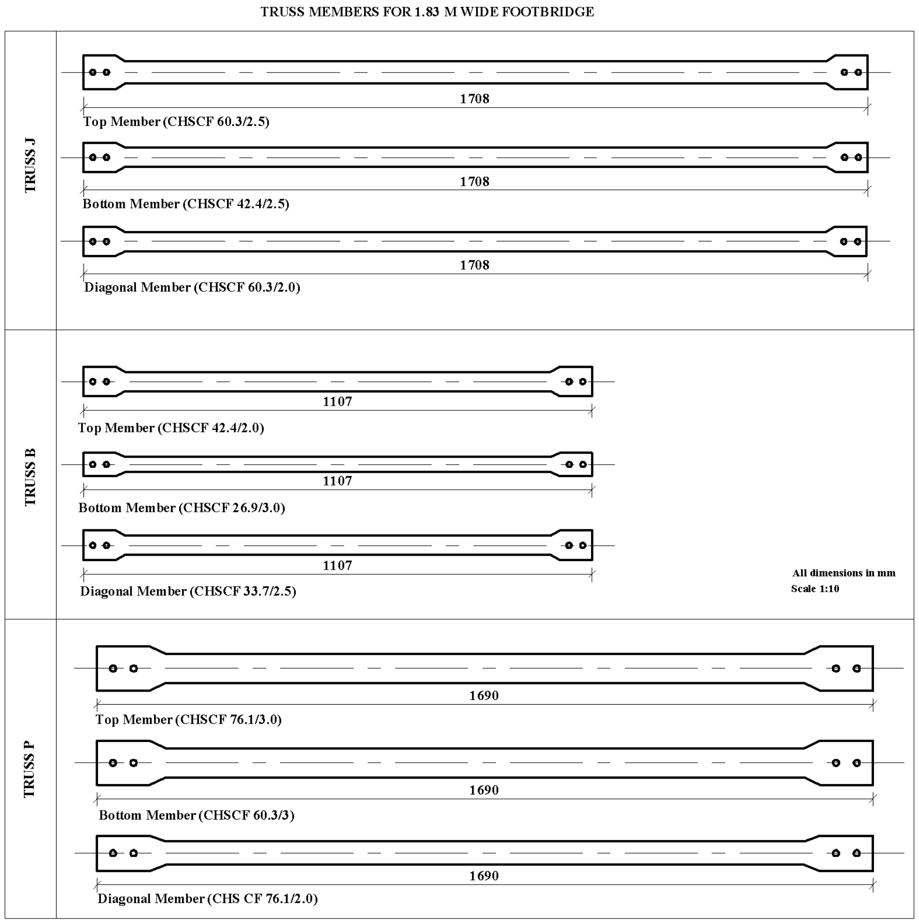

Figure 5.7: Truss members for 1.83m wide footbridges

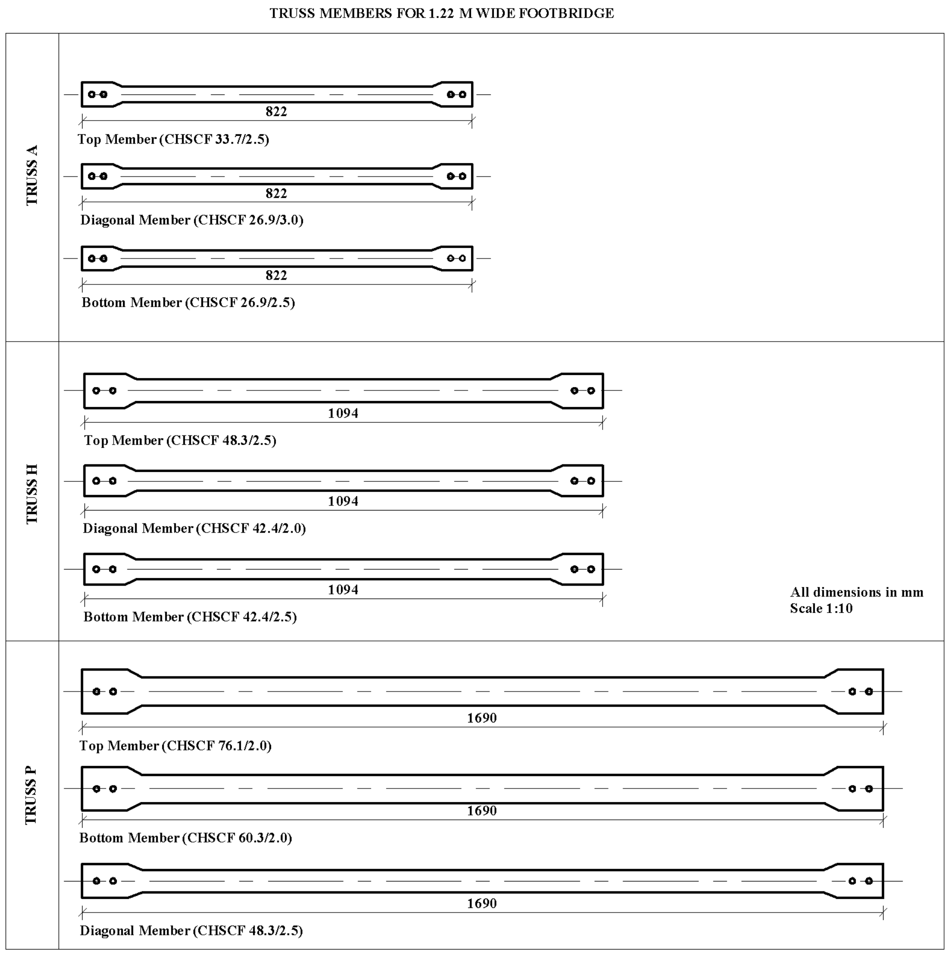

Figure 5.8: Truss members for 1.22m wide footbridges

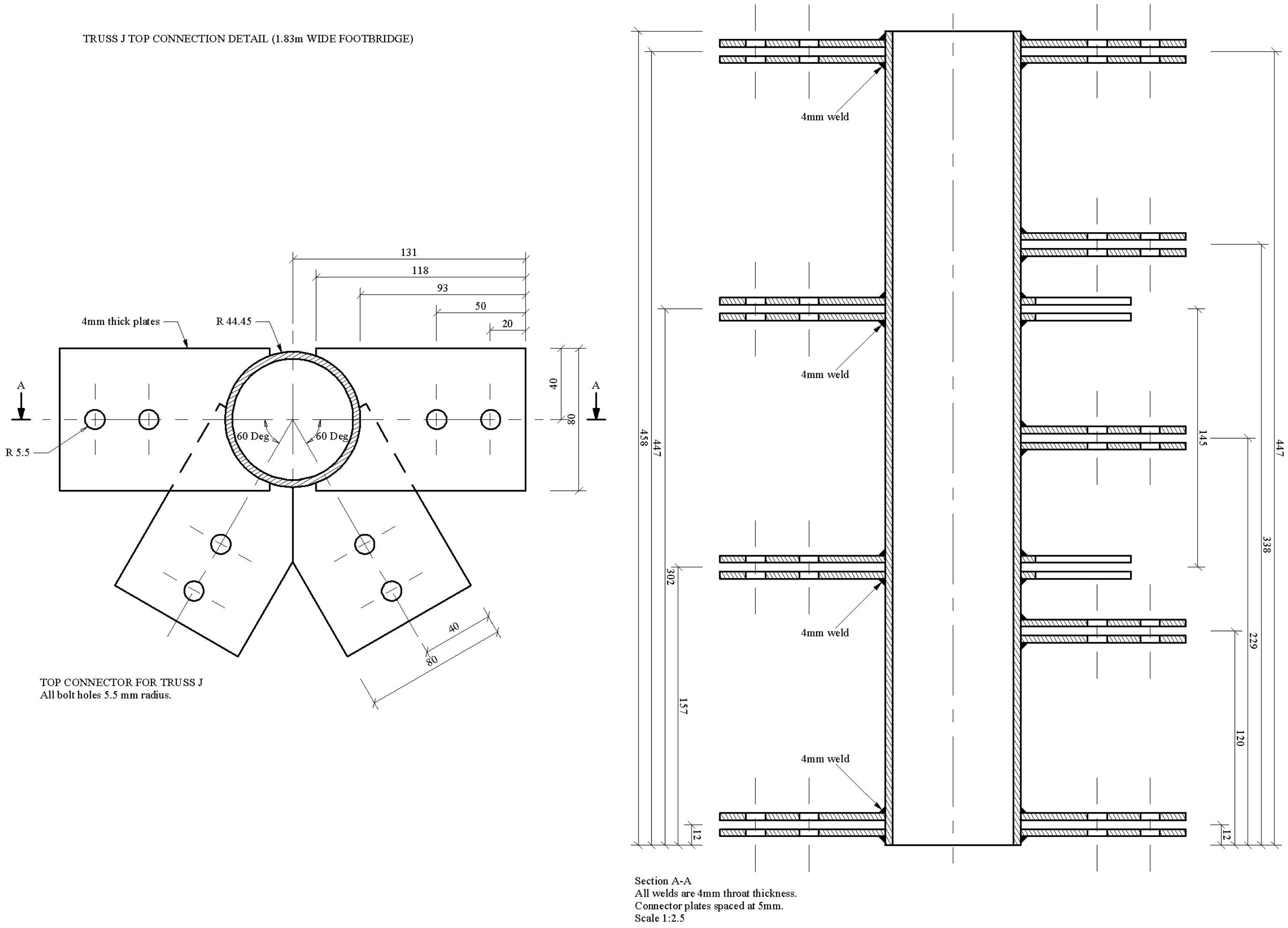

Figure 5.9: Top connection details for truss J (for 1.83m wide footbridge).

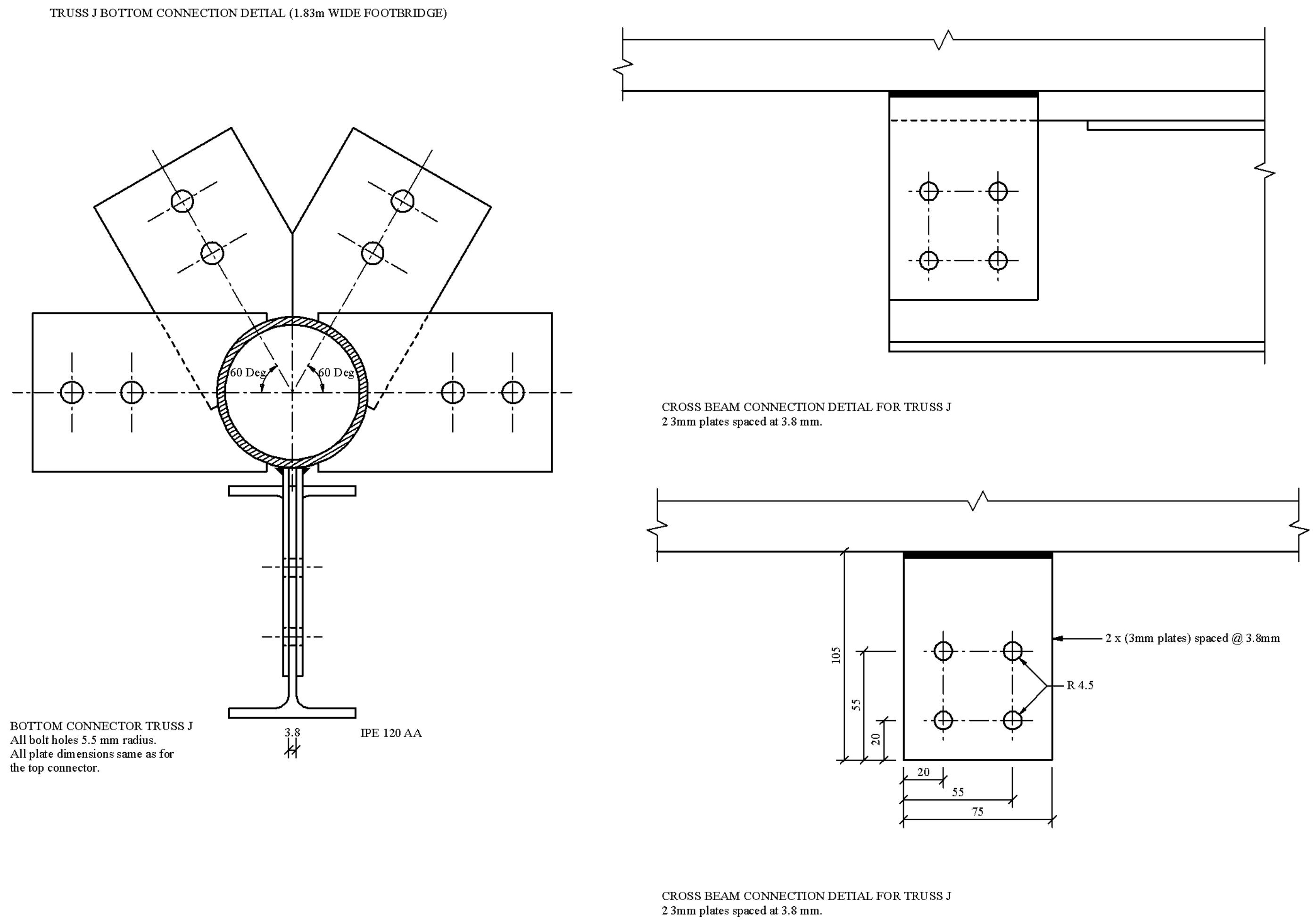

Figure 5.10: Bottom connection detail for truss J (for 1.83m wide footbridge).

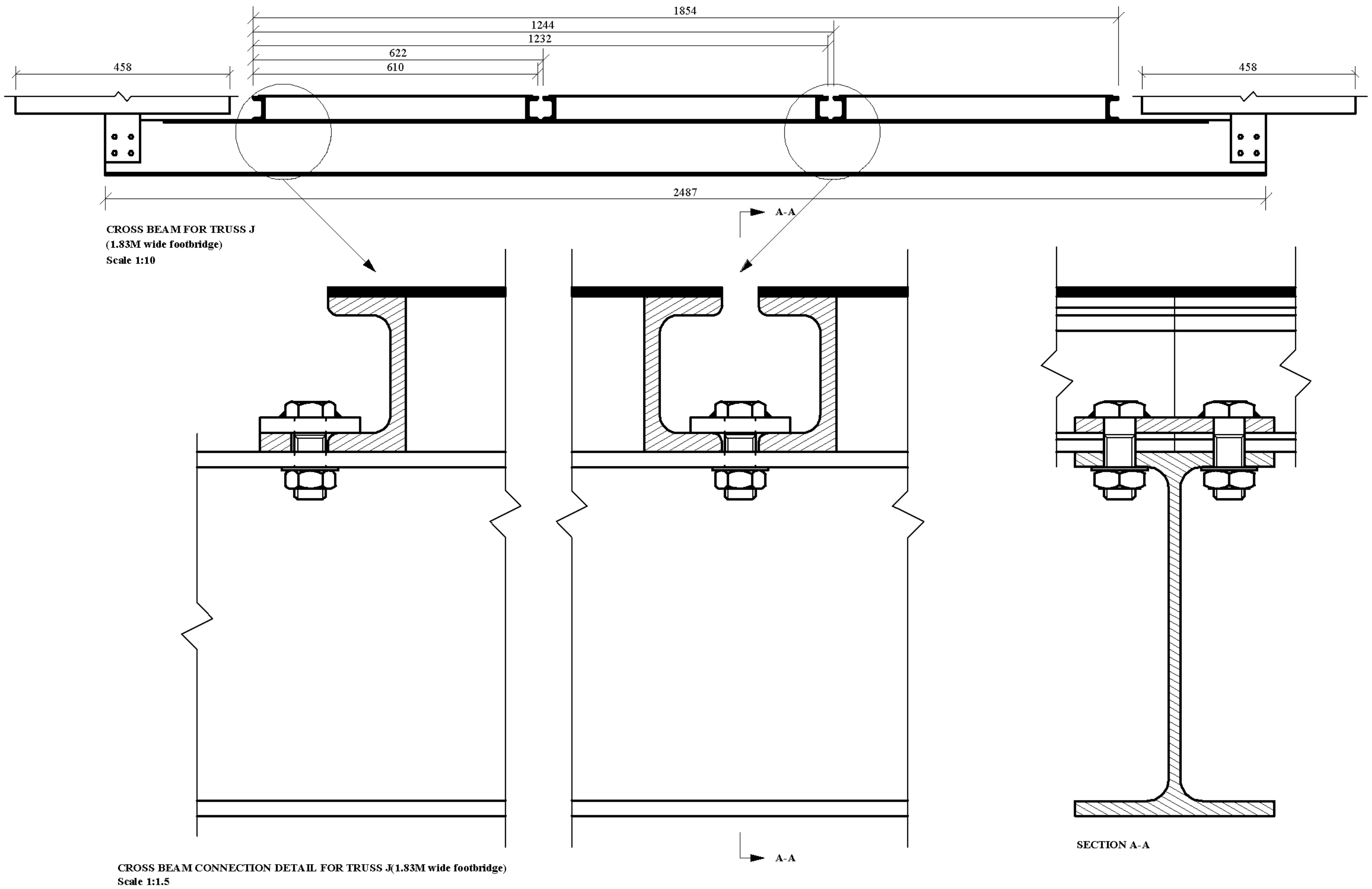

Figure 5.11: Cross beam connection detail for truss J (1.83m wide footbridge). All dimensions in mm.

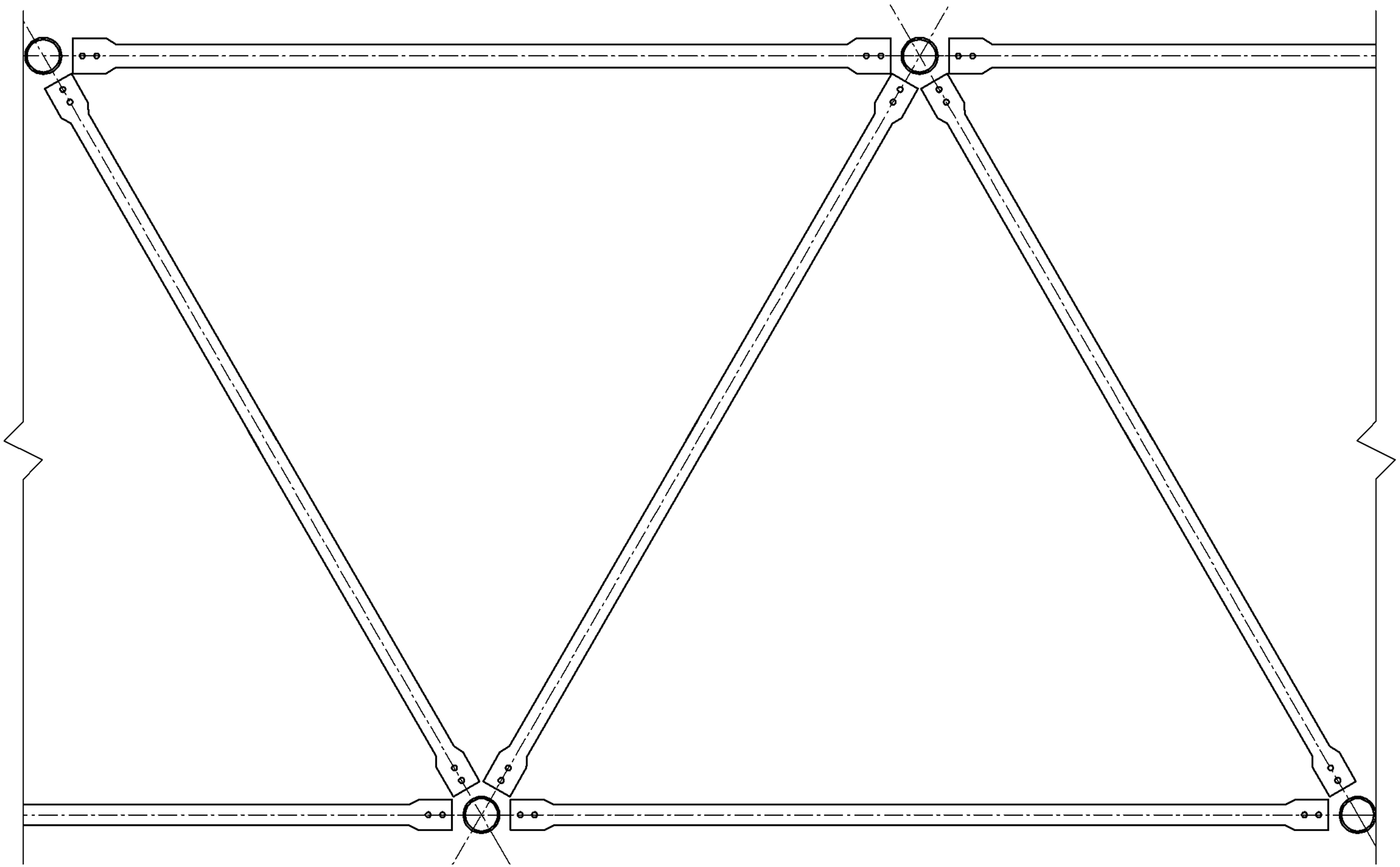

Figure 5.12: Member arrangement for truss J (for 1.83m wide footbridge).

In order to visualize how the components are assembled together, three dimensional drawings were made using AutoCAD.

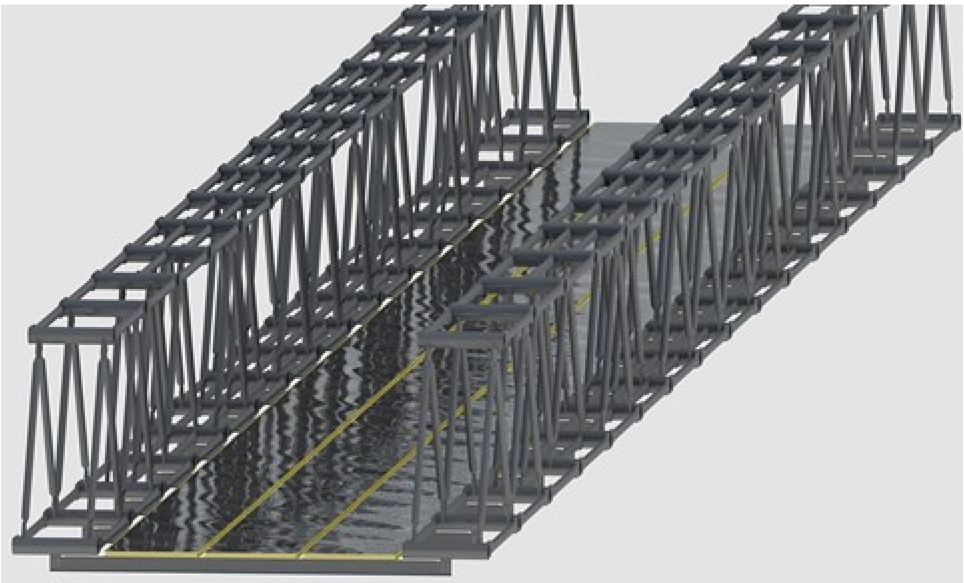

Figure 5.13: Three dimensional drawing a 1.83m wide footbridge (using truss J).

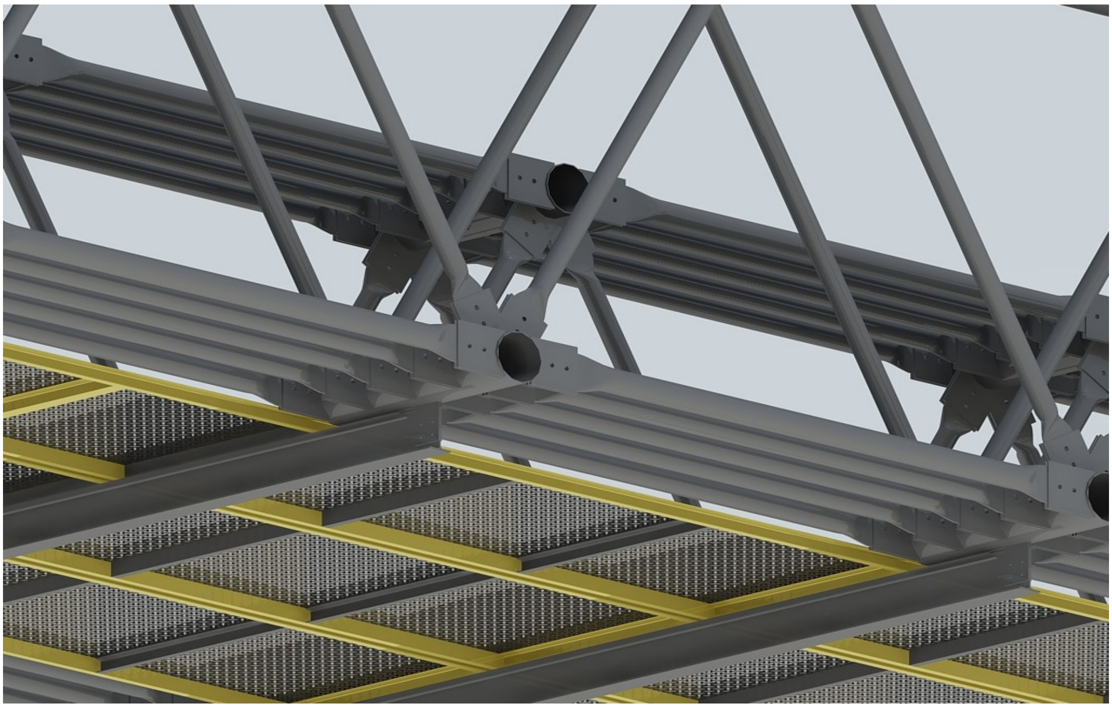

Figure 5.14: Three dimensional view a footbridge (using truss J).

Figure 5.13 and Figure 5.14 show a three dimensional drawing of a footbridge constructed using truss J for 1.83m wide footbridge.

CHAPTER SIX

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This research begins by introducing bridges which are important constructions because they provide passage over physical barriers (e.g. traffic flows, rivers and large water bodies). Various kinds of bridges are then highlighted, from which the research narrowed down to steel truss bridges of the warren type. The research sought to address the problem of over-design in classical trusses by using a design whereby trusses consisted of members in parallel. The main objective of this research was to produce a design catalogue for parallel steel truss footbridges of three spans and two widths, in which a fully stressed design was approached. Through a literature review and background information, the work that was done by previous researchers (e.g. morphological indicators) and concepts that were useful in this research (e.g. Euler buckling and Design According to the Eurocodes) were presented. Analysis of loading on the footbridge, design of the bridge consisting of members in parallel (in chapter 3 and chapter 4) and the design of the joints were discussed. Finally, the results of this research were presented.

From the research, it can be concluded that there is a need to develop national annexes for Kenya. During load analysis using Eurocodes, the Belgian annexes were applied. Some parameters for example the wind speed was chosen using the Belgian case. Though representing a worst case scenario, there is need to determine the parameters that fully represent the local situation.

It can also be concluded that the bridge deck units that were designed can be carried by two people. This is because the weight of the largest bridge deck was found to be 672 N.

Another conclusion is that a truss consisting of members in parallel is more advantageous for shallow (or slender) trusses because it can compared to classical designs save up to 21.3% for 1.83 m wide footbridges and up to 26.4% for 1.22 m wide footbridges by mass. The weight saving was the highest for shallow (or slender) trusses. The reason being that more number of bars can be used per field compared to the deep (or stocky) trusses. Even though material is not saved in the some of the deep (or stocky) trusses, it could be selected on the basis of aesthetics and modular construction. By using members in parallel, 75% fully stressed design (for slender trusses) can be achieved with most of the members utilized to a greater capacity (most members were utilized to greater than 70%). In addition, a footbridge made of trusses consisting of members in parallel is aesthetic. One can easily understand the way loads are resisted by this truss. The larger the force, the more number of members are used in parallel up to a maximum of five. The spacing of diagonals also play a role in ensuring forces in members connecting at joint beams are close to the same value as much as possible. Using the 30% and 80% spacing of diagonals for different and same number of members on either side of the joint beam respectively, the position of the diagonals changed as the number of elements in parallel varied.

Apart from length and height, the trusses consisting of members in parallel have a width. The view from one end of the truss, Figure 6.1, shows a closed form which is particularly useful in resisting torsion. A classical truss, for a slender bridge, will require lateral restraint or very large cross sections to resist torsional buckling. Also, for that reason, slender trusses consisting members in parallel are more advantageous than slender classical trusses.

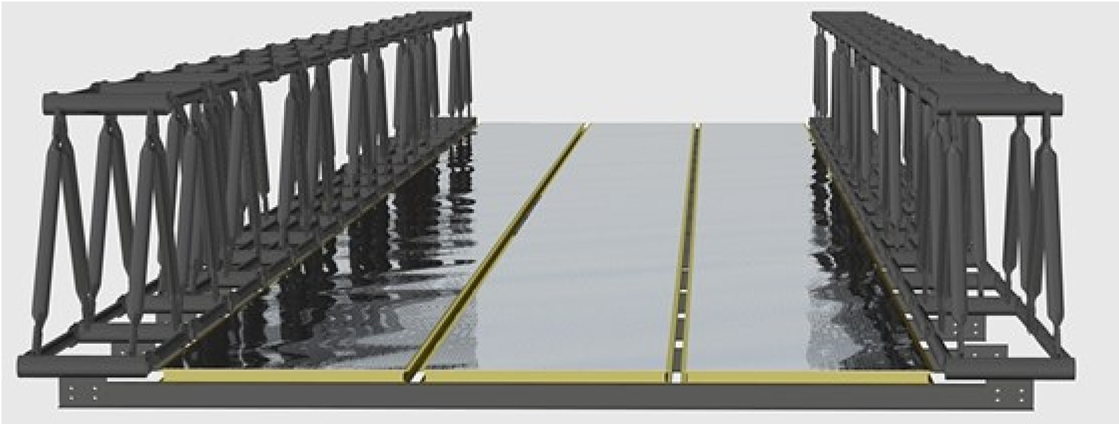

Figure 6.1: End view of footbridge with truss consisting members in parallel.

Using bolted connections allows for ease of assembly and disassembly of the truss. This means that the footbridge can be disassembled and reassembled in another site. It is possible to carry out maintenance of the truss in an interesting way. Consider a situation where a member needs to be replaced, the bridge can be closed and one person can replace the member while it is still in place. The member being replaced could have been eaten away for example by rust.

Finally, it can be concluded that it is only possible to select footbridges of 14.7 m, 22.0 m and 29.3 m from the catalogue. The width of the bridges are limited to 1.22 m or 1.83 m. These dimensions of the trusses were determined by the dimensions of a standard tear plate in Kenya (2.44 m by 1.22 m).

6.2 Recommendations

The use of less bottom members is recommended. Referring to Figure 5.4, in the first field for the bottom members, one member is sufficient but for the structure to be stable, two members are required. For the second field there are three members (i.e. 0.58 + 0.50 + 0.58 = 1.66 < 2) while two members are sufficient. For the third field it is also possible to have three elements instead of the provided four (0.63 + 0.65 + 0.65 + 0.63 = 2.56 < 3). It is also possible to save an element in field five resulting in a saving of 6 elements

(3x2) for the whole bridge. This corresponds to an additional saving of 6.5 % since the truss has 92 elements.

The use of circular hollow sections of different wall thicknesses is recommended to save additional weight. For this case, spacer plates may be required in some cases e.g. when the new selected cross sections are thinner than the provided one. This would not only reduce the weight of the truss but also ensure that they are utilized to greater than 70% of their capacity. When steel comes into contact with moisture or water it rusts. Rusting can cause structural failure in a bridge leading to its collapse. Therefore it is also recommended that the steel is galvanised as treatment to prevent rust. Nuts and bolts used to fasten the joints can be spot welded to prevent the theft of components of the footbridge.

To use the catalogue, it is recommended to select the truss with a length that is sufficient for the desired span from Table 5.7. Only two bridge width dimensions are possible, either 1.22 m or 1.83m. Depending on the selection, Table 5.7 contains the member cross sections for the various components of the bridge. From Figure 5.7 and Figure 5.8, the length of the truss members can be obtained while the number of elements in each field are summarised in Figure 5.6. The bolts required to fasten the members of the bridge are shown in Table 5.9 and the details of the connections are provided in APPENDIX 4.

Further research can be done on the design of a truss consisting of plates with different thicknesses for the elements to achieve as close as possible a fully stressed design. Plates are subject to effects of local buckling which will have to be taken into account. Elements can consist of rectangular plates that are stiffened shown in Figure 6.2.

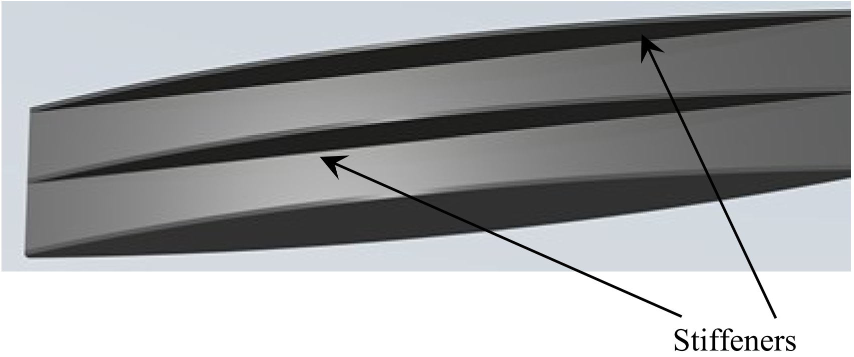

Figure 6.2: Stiffened plates a) triangular plates b) rectangular plates.

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APPENDIX 1

CALCULATIONS FOR BOLTED CONNECTIONS

Joint design calculations

Before presenting the calculations for the design of bolts and connection plates for truss P, two important tables are provided. Table 1 gives the strength of bolts and Table 2 gives the reduction factor α_v for bolted connections. The end and edge distances for bolted connections are shown in Figure 1. The calculations for connections were done for the trusses that constitute the catalogue using Microsoft Excel. For both the 1.83 m wide and 1.22 m wide footbridges in the catalogue of trusses, truss P had the greatest force in its members as obtained from analysis in SCIA Engineer. Therefore calculations for this truss are presented in this section.

Table 1: Strength of bolts

Bolt classes	4.6	5.6	8.8	10.9
f_{yb}	240	300	640	900
f_{ub}	400	500	800	1000

Table 2: reduction factor α_v

Bolt classes	4.6	5.8	5.6	5.8	6.8	8.8	10.9
α_v	0.6	0.5	0.6	0.5	0.5	0.6	0.5

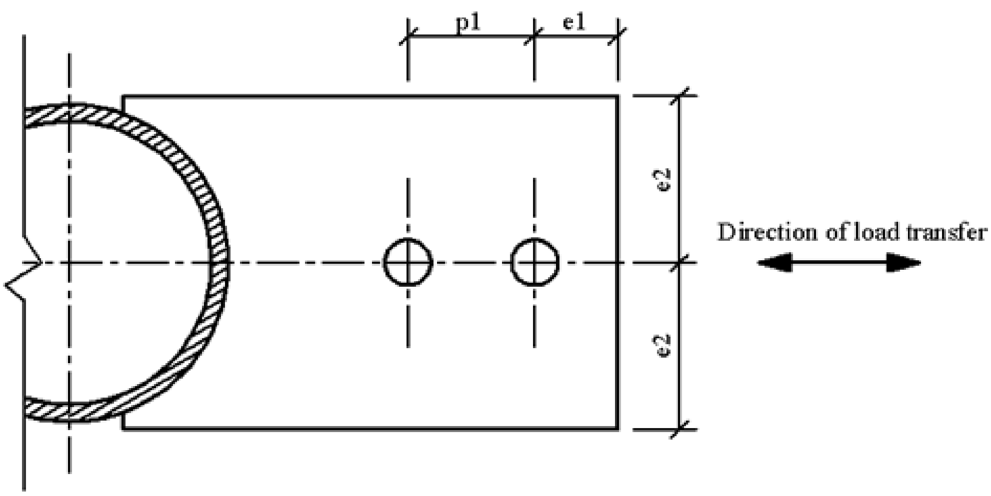


Figure 1: End and inner distances of connector plates.

CONNECTION DESIGN FOR MEMBERS OF TRUSS P FOR THE 1.83 M WIDE FOOTBRIDGE

Design for bolts in shear

Using two 12 mm bolts of class 8.8 for a force of 112.2 kN and four shear planes.

$\alpha_v = 0.6; f_{ub} = 800\ \mathrm{N/mm^2}$; $\gamma_{M2} = 1.25$ and $A_s = 84.3\ \mathrm{mm^2}$

The shear resistance per shear plane $(F_{v,Rd})$ is:

$$F_{v,Rd} = \alpha_v f_{ub} \frac{A_s}{\gamma_{M2}} = 0.6 \times 800\ \mathrm{N/mm^2} \frac{84.3\ \mathrm{mm^2}}{1.25} = 32370\ \mathrm{N} = 32.37\ \mathrm{kN}$$

The shear resistance for four shear planes= 32.37 kN $\times 4 = 129.48$ kN. Therefore, the bolts are sufficient in shear.

Design for plate bearing resistance

Design parameters:

Applied force	F	121.62kN
No. bolts	n	2
Bolt diameter	d	12mm
Hole diameter	d_o	13 mm
Plate thickness	t	4 mm
Plate yield stress	f_y	$235\ \mathrm{N/mm^2}$
Ultimate tensile strength of connected parts	f_u	$360\ \mathrm{N/mm^2}$
Bolt class	$-$	8.8
Ultimate tensile strength of bolt	f_{ub}	$800\ \mathrm{N/mm^2}$
Safety factor	γ_{M2}	1.25

End and edge distances:

		min	max
e_1	35 mm	$1.2d_o = 15.6$ mm	$4t + 40$ mm $= 56$ mm
e_2	50 mm	$1.2d_o = 15.6$ mm	$4t + 40$ mm $= 56$ mm
p_1	45 mm	$2.2d_o = 28.6$ mm	$\min(4t, 200\ mm) = 56$ mm
p_2	00 mm	0 mm	$\min(4t, 200\ mm) = 56$ mm

For end bolts:

$$\alpha_b = min\left(\frac{e_1}{3d_0}; \frac{f_{ub}}{f_u}; 1.0\right), \frac{e_1}{3d_0} = \frac{35}{3(13)} = \frac{35}{39}\ and\ \frac{f_{ub}}{f_u} = \frac{800}{360} = \frac{20}{9}$$

$\therefore \alpha_b = 35/39$

For inner bolts:

$$\alpha_b = min\left(\frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1.0\right), \frac{p_1}{3d_0} = \frac{45}{3(13)} - \frac{1}{4} = \frac{47}{52}\ and\ \frac{f_{ub}}{f_u} = \frac{800}{360} = \frac{20}{9}$$

$\therefore \alpha_b = 47/52$

In the direction perpendicular to the direction of load transfer (for edge bolts):

$$k_1 = min\left(2.8\frac{e_2}{d_0} - 1.75; 2.5\right), 2.8\frac{50}{13} - 1.75 = 9.019$$

$\therefore k_1 = 2.5$

$$F_{b,Rd} = k_1 \alpha_b f_u \frac{dt}{\gamma_{M2}}$$

$F_{b,Rd}$ for end bolts:

$$F_{b,Rd} = 2.5 \times \frac{35}{39} \times 360 \frac{12 \times 4}{1.25} = 31.02 \text{ kN}$$

$F_{b,Rd}$ for inner bolts:

$$F_{b,Rd} = 2.5 \times \frac{47}{52} \times 360 \frac{12 \times 4}{1.25} = 31.24 \text{ kN}$$

$\therefore F_{b,Rd} = 31.02$ kN for each bolt.

For two plates per bolt and two bolts in this case, $F_{b,Rd} = 4 \times 31.02 \text{ kN} = 124.08 \text{ kN}$ and therefore sufficient to sustain the applied load of 112.2 kN.

Design for plate resistance in tension

l	115 mm	
w	100 mm	To take care of weld length and joint beam diameter of 101.6 mm
t	4 mm	

A	$w \times t = 100 \times 4$	400 mm^2
A_{net}	$A - td_o = 400 - 4(13)$	348 mm^2
$N_{pl,Rd}$	$Af_y/\gamma_{M0} = 400 \times 235/1$	94 kN
$N_{u,Rd}$	$0.9A_{net}f_u/\gamma_{M2} = 0.9 \times 348 \times 360/1.25$	90.20 kN

Therefore, since there are two plates, the total plate resistance in tension is $2 \times 90.20 \text{ kN} = 180.4$ kN which is adequate to resist the applied load of 112.2 kN.

Design for block tearing of the plate

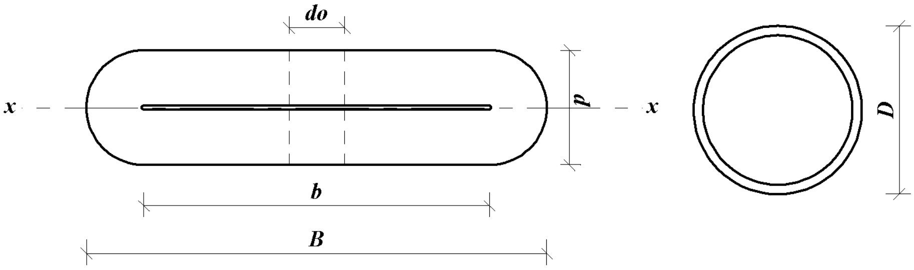
l_v	$p_1 + e_1 - d_o - (d_o/2)$	$45 + 35 - 13 - (13/2) = 60.5$ mm
t_v	t	4 mm
$A_{v,net}$	$t_v \times l_v$	$4 \times 60.5 = 242$ mm^2
$No.planes\ (n)$	2	2
$V_{eff,1,Rd}$	$2 \times f_y A_{nv}/\sqrt{3}\,\gamma_{M0}$	$2 \times 235 \times 242/(\sqrt{3} \times 1) = 65.67$ kN

Since there are two plates, the total design block resistance is:

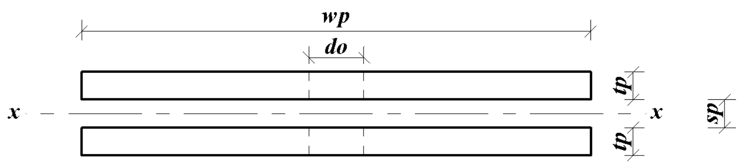
$V_{eff,1,Rd} = 65.67 \times 2 = \ 131.34$ kN

Therefore the connection plates have sufficient strength to resist block tearing when a load of 112.2 kN is applied.

CHECK FOR THE BOLTED TRUSS CONNECTION IN COMPRESSION FOR TRUSS P FOR 1.83 M WIDE FOOTBRIDGE



Flattened portion of circular hollow section



Connector plates for the truss members

Formulae used to compute the moment of inertia are summarized below.

Moment of inertia for the flattened circular hollow section	
$I_{xx} = \dfrac{(B - d_0)d^3}{12}$	$b = \pi(D - 2t)/2$ $B = b + 2t;$ $d = 2t$
Moment of inertia for the two connector plates	
$I_{xx} = 2\left[\left(\dfrac{s_p + t_p}{2}\right)^2 (w_p - d_0)t_p\right]$	

moment of inertia of the circular hollow section			
Diameter	D	76.1	mm
Thickness	t	2	mm
Depth	d	4	mm
Inside breadth	b	113.2544	mm
Outside breadth	B	117. 2544	mm
Bold diameter	d_o	13	mm
Area	A_{pipe}	465.584	mm
M. Inertia	$I_{xx\ pipe}$	556.023	mm^4
Moment of inertia of the plates			
Length	l_p	115	mm
Width	w_p	100	mm
Thickness	t_p	4	mm
Spacing	s_p	4	mm
Area	A_{plates}	800	mm
M. Inertia	$I_{xx\ plates}$	11136	mm^4
Total M. of inertia	I_{xx}	11696.02	mm^4
Total Area	A	1265.584	mm^4

$$N_{cr} = \frac{\pi^2 E I_{xx}}{KL} \; ; \; \overline{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \; ; N_{bRd} = \frac{\chi A f_y}{\gamma_{M1}}$$

χ is obtained from curve c of Figure 6.4 in EN 1993-1-1 and $L = l_p$. A value of $K = 2$ is chosen assuming the connector plate is fixed at one end of the joint beam and free at the other end.

E	210000	$\mathrm{N/mm^2}$
K	2	$-$
f_y	235	$\mathrm{N/mm^2}$
N_{cr}	458.092	kN
$\overline{\lambda}$	0.80575	$-$
χ	0.67	$-$
N_{bRd}	199.266	kN

The section can therefore sustain an applied load of 112.2 kN which was obtained as the largest compressive force for the top members.

CHECK FOR TOP MEMBER IN TENSION FOR TRUSS P FOR THE 1.83 M WIDE FOOTBRIDGE

check for the bar resistance			
Bar thickness	t	3	mm
Bar Diameter	D	60.3	mm
Bar area	A_b	540.0398	mm^2
Bar Net Area (less bolt holes)	A_b, net	501.0398	mm^2
Bar Yield Stress	f_y	235	N/mm^2
Ultimate Stress	f_u	360	
Af_y/γ_{M0}	$N_{pl,Rd}$	126.9	kN
$0.9A_{net}f_u/\gamma_{M2}$	$N_{u,Rd}$	129.8	kN

The selected cross section can therefore resist the maximum internal force of 107.3 kN which was obtained as the maximum force for the members under tension for truss P.

CONNECTION DESIGN FOR CROSS BEAMS FOR TRUSS P FOR THE 1.83 M WIDE FOOTBRIDGE.

Figure 2 shows the connection detail for the cross beam. The cross beam connection is positioned at the centre of the lower joint beam of the truss. It was assumed that no moments were transferred to the truss and therefore only shear from the cross beam was considered (F).

Design for bolts in shear

Using 8 mm bolts of class 5.8 for an applied force 13.58 kN and eight shear planes.

$\alpha_v = 0.5; f_{ub} = 500\ \mathrm{N/mm^2}$; $\gamma_{M2} = 1.25$ and $A_s = 36.6\ \mathrm{mm}^2$

The shear resistance per shear plane $(F_{v,Rd})$ is:

$$F_{v,Rd} = \alpha_v f_{ub} \frac{A_s}{\gamma_{M2}} = 0.5 \times 500\ \text{N/mm}^2 \frac{36.6\ \text{mm}^2}{1.25} = 7320\text{N} = 7.32\ \text{kN}$$

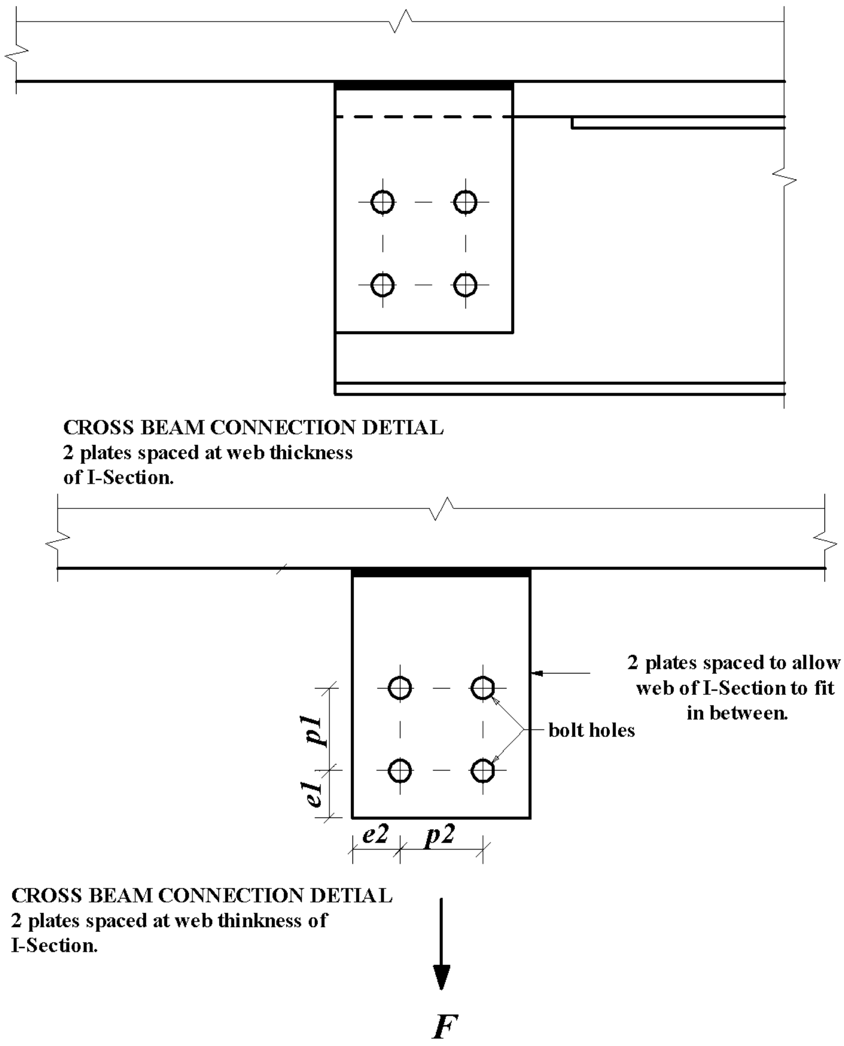


Figure 2: Connection detail of the cross beam.

The shear resistance for four shear planes is $7.32\ \text{kN} \times 8 = 58.56\ \text{kN}$. Therefore, the bolts are sufficient in shear.

Design for plate bearing resistance

Applied force	F	14.84 kN
No. bolts	n	4
Bolt diameter	d	8 mm
Hole diameter	d_o	9 mm
Plate thickness	t	3 mm
Plate yield stress	f_y	$235\ \mathrm{N/mm^2}$
Ultimate tensile strength of connected parts	f_u	$360\ \mathrm{N/mm^2}$
Bolt class	$-$	5.8
Ultimate tensile strength of bolt	f_{ub}	$500\ \mathrm{N/mm^2}$
Safety factor	γ_{M2}	1.25

End and edge distances:

		min	**max**
e_1	20 mm	$1.2d_o = 10.8$ mm	$4t + 40$ mm $= 52$ mm
e_2	20 mm	$1.2d_o = 10.8$ mm	$4t + 40$ mm $= 52$ mm
p_1	35 mm	$2.2d_o = 19.8$ mm	$\min(4t, 200\ mm) = 42$ mm
p_2	35 mm	$2.4d_o = 21.6$ mm	$\min(4t, 200\ mm) = 42$ mm

For end bolts:

$$\alpha_b = min\left(\frac{e_1}{3d_0}; \frac{f_{ub}}{f_u}; 1.0\right), \frac{e_1}{3d_0} = \frac{20}{3(9)} = \frac{20}{27}\ and\ \frac{f_{ub}}{f_u} = \frac{500}{360} = \frac{25}{18}$$

$\therefore \alpha_b = 20/27$

For inner bolts:

$$\alpha_b = min\left(\frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1.0\right), \frac{p_1}{3d_0} = \frac{35}{3(9)} - \frac{1}{4} = \frac{113}{108}\ and\ \frac{f_{ub}}{f_u} = \frac{500}{360} = \frac{25}{18}$$

$\therefore \alpha_b = 113/108$

In the direction perpendicular to the direction of load transfer (for edge bolts):

$$k_1 = min\left(2.8\frac{e_2}{d_0} - 1.75; 2.5\right), 2.8\frac{20}{9} - 1.75 = 4.4722$$

$$\therefore k_1 = 2.5$$

$$F_{b,Rd} = k_1 \alpha_b f_u \frac{dt}{\gamma_{M2}}$$

$F_{b,Rd}$ for end bolts:

$$F_{b,Rd} = 2.5 \times \frac{20}{27} \times 360 \frac{8 \times 3}{1.25} = 12.8 \text{ kN}$$

$F_{b,Rd}$ for inner bolts:

$$F_{b,Rd} = 2.5 \times \frac{113}{108} \times 360 \frac{8 \times 3}{1.25} = 18.08 \text{ kN}$$

$\therefore F_{b,Rd} = 12.8$ kN for each bolt.

For two plates per bolt and 4 bolts in this case, $F_{b,Rd} = 8 \times 12.8 \text{ kN} = 102.4 \text{ kN}$ and therefore sufficient to sustain the applied load of 13.58 kN.

Design for plate resistance in tension

l	75 mm	$e_1 + P_1 + e_1$
w	75 mm	$e_2 + P_2 + e_2$
t	3 mm	

A	$w \times t = 75 \times 3$	225 mm^2
A_{net}	$A - 2td_o = 225 - 2(3)(9)$	171 mm^2
$N_{pl,Rd}$	$Af_y/\gamma_{M0} = 225 \times 235/1$	52.875 kN
$N_{u,Rd}$	$0.9A_{net}f_u/\gamma_{M2} = 0.9 \times 171 \times 360/1.25$	44.32 kN

Therefore, since there are two plates, the total plate resistance in tension is $2 \times 44.32\text{ kN} = 88.64\text{ kN}$ which is adequate to resist the applied load of 14.84 kN.

Design for block tearing of the plate

l_v	$p_1 + e_1$	$35 + 20 = 55$ mm
t_v	t	3 mm
$A_{v,net}$	$t_v \times l_v$	$3 \times 55 = 165\text{ mm}^2$
$No.planes\ (n)$	2	2
$V_{eff,1,Rd}$	$2 \times f_y A_{nv}/\sqrt{3}\,\gamma_{M0}$	$2 \times 235 \times 165/(\sqrt{3} \times 1) = 44.77$ kN

Since there are two plates, the total design block resistance is:

$V_{eff,1,Rd} = 44.77 \times 2 = 89.54$ kN

Therefore the connection plates have sufficient strength to resist block tearing when a load of 13.58 kN is applied.

APPENDIX 2

CALCULATIONS FOR WELDED CONNECTIONS

The design for the welded connections for all the trusses in the catalogue are presented in this section. Figure 1 shows the parameters of the welded connection at the joint beam. The internal forces that are applied were obtained from the analysis results in SCIA Engineer in the ultimate limit state.

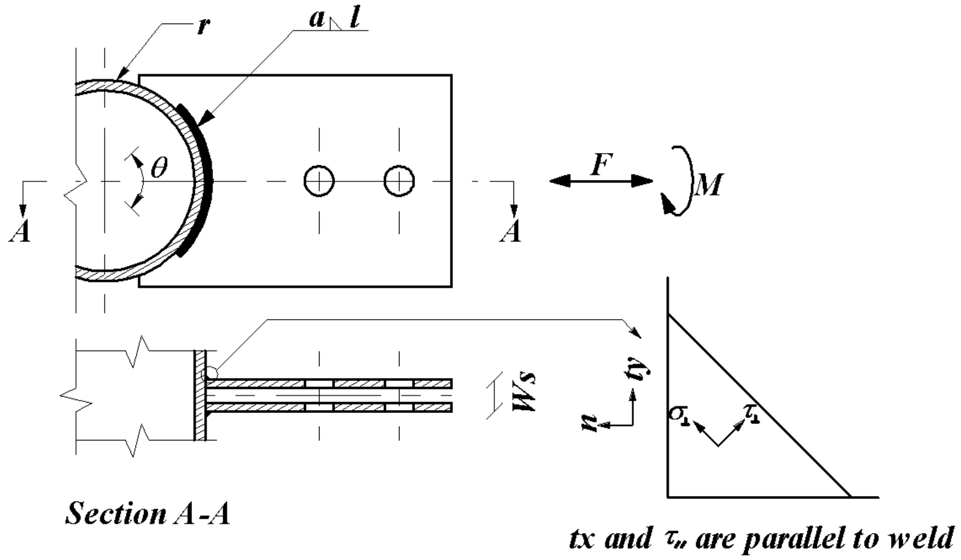


Figure 1: Parameters of the welded connections.

WELDED CONNECTIONS

Weld design for truss P (1.83 m wide footbridge)

description	***symbol***	***value***
applied force	F	112.2 kN
throat thickness	a	4 mm
weld length	l	135 mm
stresses in the conventional section.	n	$0\ \mathrm{N/mm^2}$
	t_x	$32.15\ \mathrm{N/mm^2}$
	t_y	$207.59\ \mathrm{N/mm^2}$
stresses in the plane of the throat.	$\sigma_\perp$	$146.79\ \mathrm{N/mm^2}$
	$\tau_\parallel$	$32.15\ \mathrm{N/mm^2}$
	$\tau_\perp$	$146.79\ \mathrm{N/mm^2}$
combined stresses	σ_c	$298.15\ \mathrm{N/mm^2}$
ultimate tensile strength	f_u	$360\ \mathrm{N/mm^2}$
correlation factor	β_w	0.8
partial safety factor	γ_{M2}	1.25

description	**symbol**	***value***
moment	M	0.25 kNm
plate spacing	w_s	14.4 mm
force parallel to weld	F_x	17.36 kN
bar radius	r	50.8 mm
angle covered by weld	θ	152.26^o

$$\frac{f_u}{\beta_w \gamma_{M2}} = 360\ \mathrm{N/mm^2}\ and\ 0.9\frac{f_u}{\gamma_{M2}} = 259.2\ \mathrm{N/mm^2}$$

joint check: $\sigma_c < \frac{f_u}{\beta_w \gamma_{M2}}\ and\ \sigma_\perp < 0.9\frac{f_u}{\gamma_{M2}}\ \therefore checks\ O.K.$

APPENDIX 3

FORCES IN SELECTED PARALLEL MEMBER TRUSSES

ALL FORCES IN kN

These trusses are for the 1.83 m wide footbridges

TRUSS B

TOP (COMPRESSION)

27.65	33.75	33.65	31.24	35.05	36.17
	32.37	33.61 33.57	32.03 32.72 32.00	34.78 34.56 34.76	35.74 35.40 35.70
27.28	33.50	33.53	31.22	35.00	36.13

BOTTOM (TENSION)

14.26	27.48	30.24	35.46	33.98	36.27
	23.51	29.51 29.46	39.21 39.17	33.90 34.06 33.86	35.85 35.55 35.82
14.23	27.33	30.10	35.34	33.93	36.21

TRUSS P

TOP (COMPRESSION)

60.24	111.55	109.39	96.75	111.90	97.58	101.97	103.49
		92.08	94.48 94.24	107.52 107.24	94.65 94.64 94.20	100.16 99.00 99.69	101.86 100.68 101.38
59.17	110.46	108.12	96.03	110.91	96.70	101.08	102.55

BOTTOM (TENSION)

31.58	86.97	97.64	90.44	102.17	96.52	99.42	103.55
		77.69	86.03 86.07	107.10 107.16	91.98 90.46 92.18	100.22 100.66 100.50	103.22 102.95 103.51
31.35	86.31	97.41	90.62	102.33	97.05	100.03	104.15

These trusses are for the 1.22 m wide footbridges

ALL FORCES IN kN

TRUSS A

TOP (COMPRESSION)

19.25	23.39	33.64	31.08	27.92	30.72	32.32	32.84
	24.51	31.99	29.99	27.97	30.51	32.04	32.54
			29.91	20.24	30.31	31.82	32.32
				27.86	30.39	31.90	32.40
18.92	23.15	33.33	30.87	27.72	30.48	32.05	32.55

BOTTOM (TENSION)

10.21	27.96	30.16	29.97	32.83	30.29	32.28	33.31
		27.67	26.80	34.55	29.90	32.11	33.16
			26.88	34.56	29.89	32.02	33.08
					29.96	32.18	33.32
10.16	27.78	30.11	30.00	32.87	30.43	32.42	33.46

TRUSS H

TOP (COMPRESSION)

29.82	55.77	55.05	49.15	57.85	50.73	54.24	56.35	57.06
		46.63	48.33	55.37	49.75	53.50	55.64	56.34
			48.12	55.15	49.98	53.00	55.09	55.79
					49.43	53.16	55.26	55.96
29.18	54.97	54.19	48.58	55.14	50.11	53.56	55.62	56.30

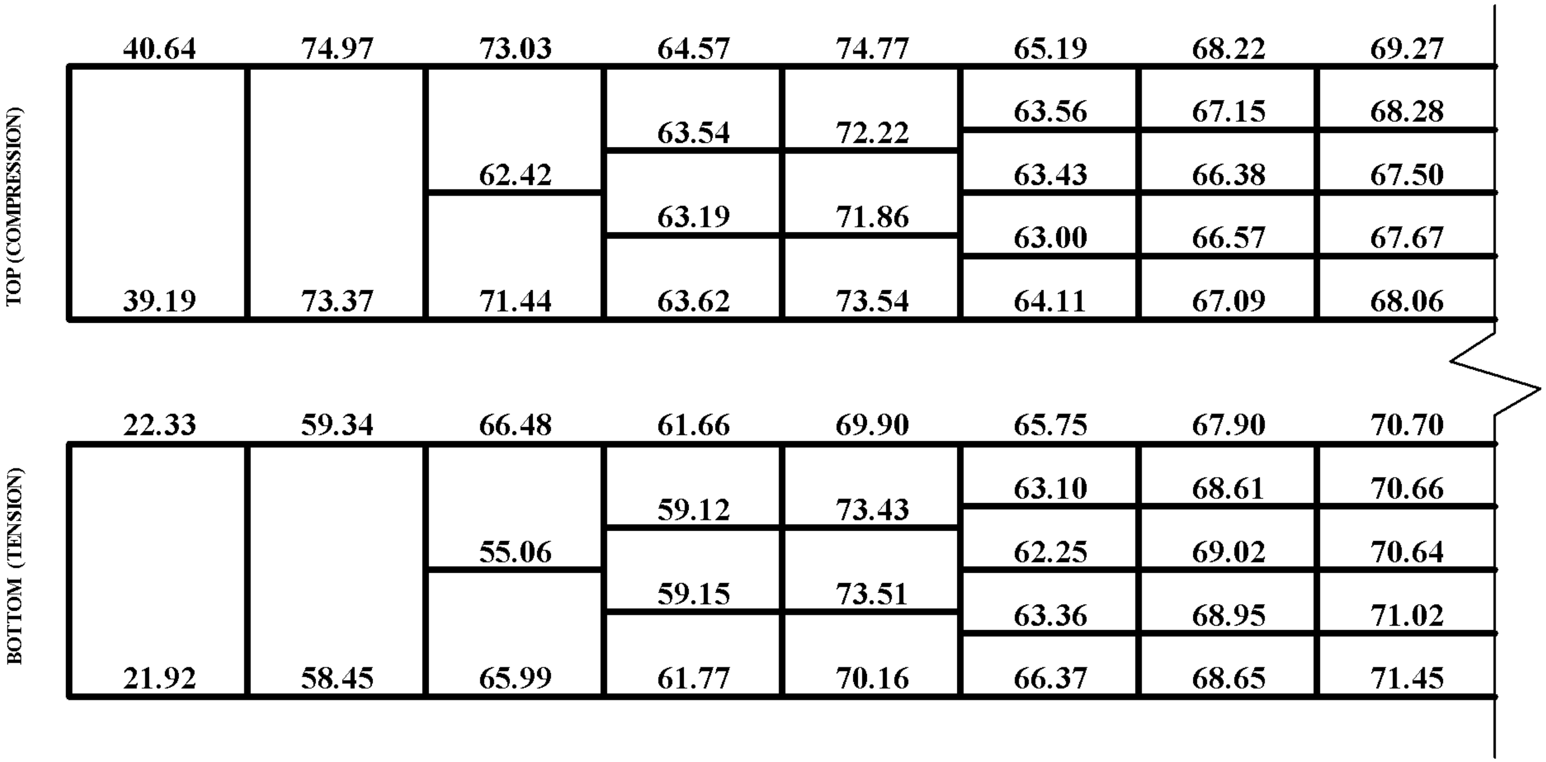
BOTTOM (TENSION)

15.95	43.61	49.57	46.24	52.96	50.57	52.81	56.27	57.85
		39.59	44.06	55.48	48.31	53.59	56.26	57.64
			44.09	55.55	47.61	54.06	56.24	57.52
					48.49	53.83	56.52	57.90
15.86	43.25	49.47	46.40	53.15	51.02	53.32	56.80	58.40

ALL FORCES IN kN

These trusses are for the 1.22 m wide footbridges

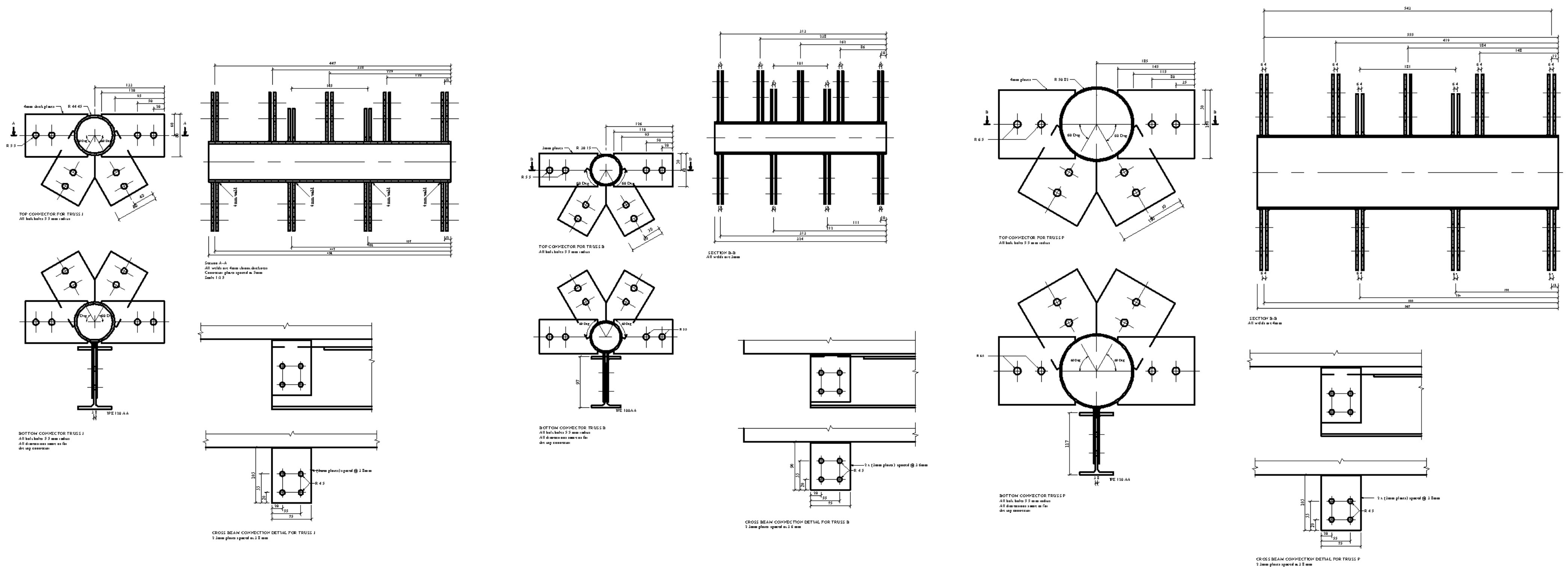
TRUSS P



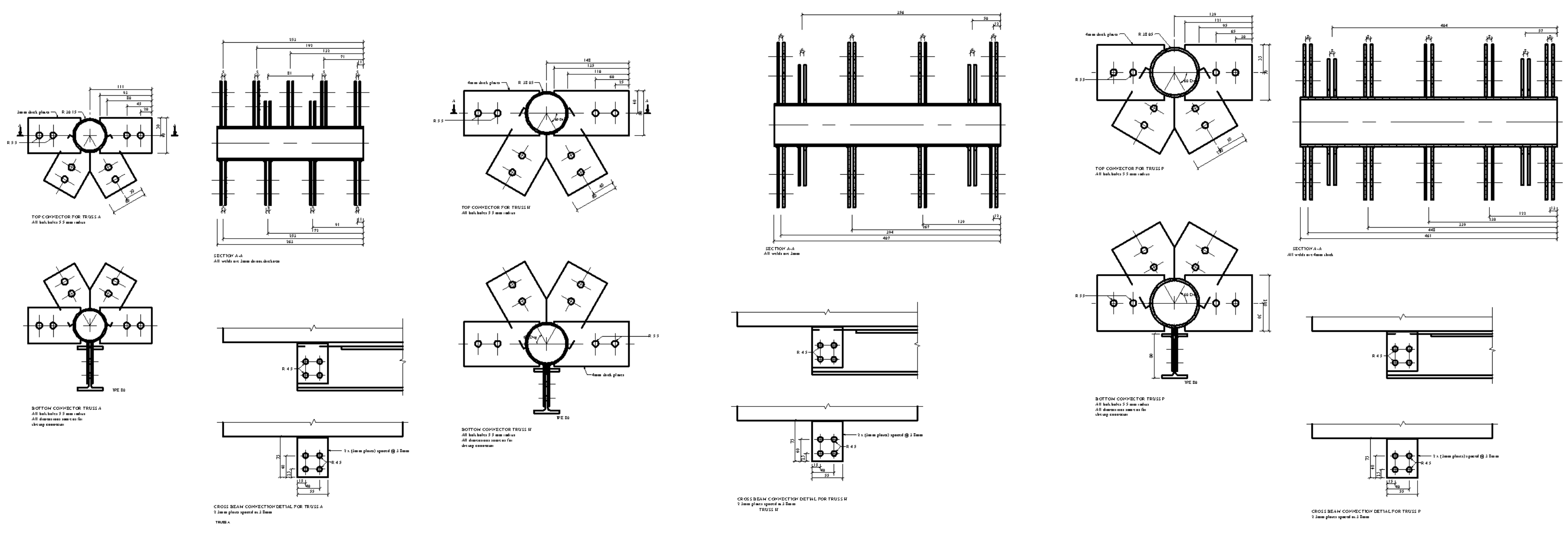
APPENDIX 4

DRAWINGS AND DTAILS FOR TRUSS CONNECTORS

1.83 m wide footbridges



1.22 m wide footbridges



APPENDIX 5

IMPORTANT METHODS OF THE JAVA PROGRAM

```
private void prepareTrusses() {

        Vector trussParameters = parametersGui.getTrussParameters();

        FemPlaneTruss trussInstance = (FemPlaneTruss) cboTopology.getSelectedItem();

       for (int k = 0; k < trussParameters.size(); k++) {

            //for (int k = 0; k < 1; k++) {

            RowData row = (RowData) trussParameters.get(k);

            String name = (String) row.getValueAt(0);

            double L = (Double) row.getValueAt(2);

            int n = (Integer) row.getValueAt(3);

            //double H = (Double) row.getValueAt(1);

            double H = L / (n * 1d);

            if (chkEquilateral.isSelected()) {H = H * Math.sin((Math.toRadians(60d))); }

            FemPlaneTruss planeTruss = trussInstance.getInstance(n, L, H);

            Section section = (Section) this.cboSection.getSelectedItem();

            Integer yieldStress = (Integer) this.cboYieldStress.getSelectedItem();

            double W = Double.parseDouble(this.txtTrussWidth.getText());

            double E = Double.parseDouble(this.txtModulus.getText());

            double q = Double.parseDouble(this.txtLiveLoad.getText());

            double gammaQ = Double.parseDouble(this.txtLoadFactor.getText());

            double gammaG = 1.35;

            section.E = E;

            section.sigmaYield = yieldStress;

            planeTruss.setSection(section);

            //get weigt of truss

            double trussWeight = planeTruss.computeWeight() * gammaG * 10.0d / 1000.0d /
n; //weight in kN

            double liveLoad = q * L * W * gammaQ / 2 / n; //half the live load in kN

            double deckLoad = gammaG * ((W / 0.61) / 2) * ((L / 2.44) * 77.0d / n) *
10.0d / 1000d; // weight of deck in kN

            double trussLoad = liveLoad + trussWeight + deckLoad;

            System.out.println("Truss Nodal Load : " + trussLoad);

            planeTruss.setUserDefinedLoad(trussLoad);

            planeTruss.setNodalLoads();

            planeTruss.setSectionProperties();

            BarElement2D[] elements = planeTruss.getElements();

            FemNode[] nodes = planeTruss.getNodes();
```

```
            SimpleFreedoms simpleFreedoms = new SimpleFreedoms(new NodalFreedom2D(2),
nodes, elements);

            simpleFreedoms.computeNodalFreedoms();

            simpleFreedoms.computeElementFreedoms();

            FemEngine engine = new FemEngine();

            engine.setEcho(false);

            engine.setNodes(nodes);

            engine.setElements(elements);

            engine.startEngine();

            engine.solveProblem();

            this.printElementForces(elements);

            BarElement2D[] topMembers = planeTruss.getTopMembers();

            BarElement2D[] bottomMembers = planeTruss.getBottomMembers();

            // get largest top bar force

            BarElement2D largestForceTopBar =
this.searchLargestForceBar(planeTruss.getTopMembers());

            Section strutSection = getStrutSection(largestForceTopBar);

            //get largest bottom bar force

            BarElement2D largestForceBottomBar =
this.searchLargestForceBar(planeTruss.getBottomMembers());

            Section tieSection = getTieSection(largestForceBottomBar);

            //get number of bars for the top bars

            int numBars = this.getInt(txtMaxNumStruts.getText());

            computeNumberOfStrutBars(getStrutParallelBarSection(strutSection, numBars),
topMembers);

            //get the number of bars for the bottom bars

            numBars = this.getInt(txtMaxNumTies.getText());

            computeNumberOfTieBars(getTieParallelBarSection(tieSection, numBars),
bottomMembers);

             //write the truss to the dxf file

            writeToDXF(topMembers, bottomMembers, planeTruss, name, W,
chkFullBridge.isSelected());

        }

    }

private void writeToDXF(BarElement2D topMembers[], BarElement2D
bottomMembers[],FemPlaneTruss planeTruss, String trussName,

            double bridgeWidth, boolean isFullBridge) {

        double fieldWidth = planeTruss.getW();

        double fieldHeight = planeTruss.getH();

        double jointBeamLen = 0.6;

        System.out.println("Field Width " + fieldWidth + " Field Height " +
fieldHeight);

        double jointBeamOffsetSame = this.jbOffsetSame.getValue();
```

```
        double jointBeamOffsetDiff = this.jbOffsetDiff.getValue();

        int topSequence[] = new int[topMembers.length];

        int bottomSequence[] = new int[bottomMembers.length];

        for (int i = 0; i < topSequence.length; i++) {topSequence[i] =
topMembers[i].getNumBars();}

        for (int i = 0; i < bottomSequence.length; i++) {bottomSequence[i] =
bottomMembers[i].getNumBars();}

        PMTWarren warren = null;

        if (isFullBridge) {

            warren = new PMTWarren(planeTruss.getN(),

                    jointBeamLen, fieldHeight, fieldWidth, bridgeWidth, isFullBridge);

            double deckOffset = Double.parseDouble(this.txtDeckOffset.getText());

            double deckWidth = Double.parseDouble(this.txtDeckWidth.getText());

            double deckLength = Double.parseDouble(this.txtDeckLength.getText());

            warren.setDeckLength(deckLength);

            warren.setDeckOffset(deckOffset);

            warren.setDeckWidth(deckWidth);

        } else {warren = new PMTWarren(planeTruss.getN(),jointBeamLen, fieldHeight,
fieldWidth); }

        warren.setIsOffsetEndDiagonals(this.chkOffsetEndDiagonals.isSelected());

        warren.setSameBarOffset(this.getSameBarsOffset());

        warren.setDifferentBarOffset(this.getDiffBarsOffset());

        //THE JOINT BEAM LENGTH

        if (chkComputeJointBeamLength.isSelected()) {

            warren.setJointBeamLen(strutJointBeamLength > tieJointBeamLength ?
strutJointBeamLength : tieJointBeamLength);

        } else
{warren.setJointBeamLen(Double.parseDouble(txtJointBeamLength.getText()));}

        warren.setTopSequence(topSequence);

        warren.setBotSequence(bottomSequence);

        warren.generateMembers();

        ArrayList<TrussMember> members = warren.getMembers();

        //prep parallel member bridge

        String loc = "E:\\KNOWLEDGE\\MASTERS NOTES\\THESIS\\DXF FILES\\";

        writeMembersToDXF(warren.getMembers(), warren.getDeckPlates(), trussName, loc);

        //prep the classical bridge

        prepBridge(trussName, fieldWidth, fieldHeight, planeTruss.getN()); }
```