Structural Evaluation of Tied-Arch and Truss Bridges Subjected to Wind and Traffic Loading

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ABSTRACT

It has been years that bridge designers and engineers are not only concerned about stability of bridge structures but being concerned about their efficiency and aesthetic as well. Nowadays, as the need is greater than ever, tied-arch bridges and truss bridges have proven they have been of interest to bridge designers when span range of 40 to 550 m are required. As of today most bridges in this range of span uses in countries like United States, Japan, China and Australia are tied-arch and truss bridges.

The aim of the thesis is to investigate the structural behavior of these bridges when they subjected to wind and traffic loading and their efficiency comparing to each other. To do so, two tied-arch bridges and two truss bridges with long and medium span has been designed according to AASHTO LRFD specifications and then analyzed by MIDAS Civil software according to the load defined in specification in order to evaluate stability, aesthetic and economy of each bridge.

The steel weight needed for the long and medium span bridges is assessed from the final design to compare and evaluate tied-arch bridge and truss bridge efficiency. These results are compared together in order to identify the most optimal bridge.

Keywords: tied-arch Bridge, truss bridge, stability, aesthetic, efficiency, optimal.

ÖZ

Uzun yıllardır köprü tasarımcıları ve mühendisleri köprülerin sadece sağlamlıkları ile değil ayrıca etkin kullanımları ve estetikleri ile de ilgilendiler. Günümüzde 40-550m açıklıkları olan köprü kullanım ihtiyacının artması, kemerli ve makaslı köprülere karşı köprü tasarımcılarının ilgisini de artırmıştır. Bugün itibarı ile Amerika, Japonya, Çin ve Avusturalya gibi ülkelerde yukarıda belirtilen uzunluklarda ihtiyaç olan köprülerde kemerli ve makaslı köprüler kullanılmaktadır.

Bu tezin amacı rüzgar ve trafik yüklerine maruz kalan köprülerin yapısal davranışlarını incelemek ve köprülerin birbirleri ile karşılaştırıldığında etkinliklerini araştırmaktır. Bunu yapabilmek için, AASHTO LRFD standardlarına göre uzun ve orta açıklıklı 2 kemerli köprü ve 2 makaslı köprü tasarlanmış ve sonrasında yukarıda belirtilen standardlarda tanımlanan yüklere göre her köprünün sağlamlığını, estetiğini ve ekonomisini değerlendirmek için MIDAS Civil yazılımı kullanılarak analiz edilmişlerdir.

Uzun ve orta açıklıklı kemerli ve makas köprülerin son tasarımlarının sonuçlarından elde edilen çelik ağırlığı adı geçen köprülerin verimliliğini karşılaştırmak amacı ile kullanılmıştır. Optimal köprü tipini belirlemek amacı tasarımlarda elde edilen sonuçlar karşılaştırılmıştır.

Anahtar Kelimeler: Kemerli köprü, makaslı köprü, sağlamlık, estetik, verimlilik, optimal.

For my Family

“No one has ever been given more loving and support than I have been given by you”

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INTRODUCTION

1.1 Brief History of Bridge Engineering

Human always searched for a way to transport their products from one place to another over the years. Previous means of transportations with horses and camels were hard and took tremendously long time to cross over valleys, rivers or other obstructions; this was done by moving along the valleys and rivers to find suitable crossing points which were time consuming. After many years passed, population growth resulted in higher demand of products, such as, agricultural products, and also usage of more advanced and heavier vehicles, such as, cart. All this made transportation process even harder. This resulted in the idea of creating a passage over rivers and valleys to have a much quicker access in order to fulfill the requirements of increasing population. Today these passages are known as “bridges”.

Bridge Engineering began with the use of stone and wood for structures as early as the first century B.C. This industry has undergone a dramatic evolution in terms of analysis and use of materials. Kuzmanovic (1977) describes stone and wood as the first bridge building materials. Iron was introduced during the transitional period from wood to steel [1]. First iron bridge with 42 m span was built in England, Coalbrookdale 1779 [2]. Records show that, concrete was used in France as early as 1840 for a bridge 12 m long to span across the Garoyne Canal, on that time reinforced concrete was not used until the beginning of the century[1].

Researches show that, arch bridges were constructed in Rome, Ancient Greece and other European cities in Middle Ages [1]. According to Wikipedia, “the oldest existing arch bridge is the Mycenaean Arkadiko bridge in Greece from about 1300 BC” [3]. These arches were half-circular, with the flat arches beginning to dominate bridge work during the Renaissance period. Design of arches had improved by Perrronet at the end of the 18^{th} century which was structurally adequate to accommodate the upcoming railroad loads [1]. Stone bridges have not changed in terms of analysis and use of materials. The first theoretical treatment used in the practical designs in the early 1770s, developed by Lahire (1695) by introducing the pressure line concept [1].

The first wooden truss bridges were in the 16^{th} century, when Andrea Palladio (1570) invented triangular trusses to construct bridges with spans up to 30.5 m [1]. Several timber bridges were constructed in Western Europe beginning in the 1750s with span up to 61 m [2]. However, during 19^{th} century significant number of timber and iron bridges was constructed in the United States. Fairmount truss bridge in Pennsylvania with span of 102 m could be a great example. This was an iron arched-truss bridge which was later destroyed by fire in 1838 [2].

Truss wooden bridges provided the ideal solution in terms of economic considerations including the initial low cost and fast construction [1].

The wooden lattice bridge developed by Town in 1820 became the prototype of the early non-lattice bridges [4]. In 1840 Howe introduced a patented truss system that became the standard for many early railroad bridges [1].

A further development in wooden trusses was the arch type with or without ties. A detailed account of American bridges was provided by Culmann (1851, 1852). On the theoretical side, Culmann proposed new methods for stress calculation, and these included statically redundant trusses [1]. One of the most outstanding wooden trusses was developed by Long (1839). Cascade Bridge (1837) of the Erie Railroad spanning a valley of 533 m deep and 91.4 m wide is a notable arch bridge of these periods.

1.2 Bridge Structure

1.2.1 General

A bridge is a structure that crosses over a river, road, railway or other obstructions, which permits a smooth and safe passage of vehicles, trains and pedestrians [5]. A bridge structure can be divided into two main parts. First the upper part called superstructure, which consists of the deck, the floor system such as stringers and floor beams and the main trusses or girders, second the lower part called the substructure, which are columns, piers, towers, footings, piles and abutments [5].

The superstructure provides horizontal spans such as deck and girders and carries the traffic loads and other permanent loads directly. The function of substructure is to support the superstructure of the bridge.

1.2.2 Bridge Classification

Bridges can be classified in several ways depending on the objective of classification. Few of these Classifications are listed below [5, 6].

1. Classification by materials:
    - Steel Bridges: Wide variety of structural steel components and systems, such as, decks, arches, trusses, stayed and suspension cables are used.
    - Wooden bridges: Bridges using wood and having relatively short spans.

- Concrete bridges: Bridges using reinforced and prestressed concrete.
- Stone bridges: In ancient times stone was the most common materials used to construct magnificent arch bridges.

2. Classification by function:
   - Highway bridges: Bridge carrying vehicle traffic.
   - Railway bridges: Bridges carrying trains.
   - Combined bridges: Bridges carrying both trains and vehicles.
   - Pedestrian bridges: Bridges carrying pedestrians.

3. Classification by relative position of floor:

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

   - Deck Bridge: the deck is supported at the top of supporting structure.
   - Semi-through bridge: The deck is supported at the intermediate level of the supporting structures.

     Through bridge: The deck is supported at the bottom.

4. Classification by structural system:
   - I-Girder or Beam Bridges: The main girder consists of either plate girders or rolled I-shapes.
   - Box-girder Bridges: The main girder consists of a single or mostly multiple box beams fabricated from steel plates.
   - T-beam Bridges: Multiple reinforced concrete T-beams are placed side by side to support live loads.
   - Orthotropic deck Bridges: Bridge deck consists of a steel deck plate and rib stiffeners.

- Truss Bridges: Truss Members resists axial forces, either in compression or tension. These members are arranged in a continuous pattern based on structural rigidity of triangles.
- Arch Bridges: The structure is vertically curved and resists loads mainly in axial compression. Curved arch transfers compression loads in to abutments.
- Cable-stayed Bridges: Main girders are supported by high strength cables directly from one or more towers. These types of bridges are suited for long span distances.
- Suspension Bridges: Vertical hangers support the main girders, which are supported by main suspension cable extending over tower anchorage to anchorage. Design is suitable for large span and long bridges.

5. Classification by support condition:
   - Simply supported bridges: The main girders or trusses are simply supported by a movable hinge at one end and fix hinge at the other end. They can be analyzed using conditions of equilibrium.
   - Continuously supported bridges: Girders or trusses are continuously supported, resulting in a structurally indeterminate system. These tend to be more economical since fewer expansion joints will have less service and maintenance problem. Settlements at supported in this system is neglected.
   - Cantilever bridge: a continuous bridge is made determinate by placing intermediate hinges between the supports.
   - Rigid frame bridges: The girders are rigidly connected to the substructure.

6. Classification depending on the life of the bridge:
   - Temporary bridge: A bridge that is used for short time and is then demolished and used in other areas whenever the need arises as in military bridges.
   - Permanent bridges: Bridge that is used throughout its lifetime. Life time of bridges depends on their design, sometimes it is as long as 200 years.
7. Classification depending on span length:
   - Short span bridges: bridges with span length less than 50 meters.
   - Medium span bridges: bridges with span length between 50 and 200 meters.
   - Long span bridges: bridges with span length more than 200 meters.

Based on the above classification, the study of this thesis will focus on simply supported, through type, steel truss and arch bridges.

1.2.3 Selection of Bridge Type

The selection of bridge type is complex task to achieve the owner's objectives [5]. This requires the collection of extensive data from which possible options are chosen.

1.2.3.1 Factors Affecting the Selection of Bridge Type

The following factors govern the selection of type of bridges: [7]

1. Volume and nature of the traffic
2. The nature of the river and its bed soil
3. The availability of materials and fund
4. Time available for construction of the bridge.
5. Physical feature of the country.
6. Whether the river is used for navigation purposes or not

7. Availability of skilled and unskilled workers
8. Facilities available for erection of bridge and maintenance.
9. Economic length of the span.
10. Level of high flood level and the clearance required
11. Climatic condition
12. Strategic condition
13. Hydraulic data
14. Length and width of the bridge
15. Foundation conditions for piers and abutment
16. Live load on the bridge
17. Appearance of bridge from aesthetic point of view

1.3 Aim and Scope

Over the years bridges have become important elements of infrastructure. Many designs have been evolved to suit the different requirements of span length, materials, environmental conditions, economics and aesthetics.

In recent years, construction of steel arched and truss bridges became more common when span range of 40 to 550 meters is required. Countries like China, Australia and United states are the leaders of these bridges in the longest bridge span ranking [3]. A brief survey indicates that, most of the bridges built in United States are arch bridge and truss bridges in the above mentioned range of span [3].

States like, Alabama, Alaska, New York, Ohio, Pennsylvania and California are using steel truss bridges for span range of 45 to 260 meters and steel arched bridges for span range of 98 to 380 m. The longest bridge spans in United States are 518 meters and 366 meters for arch and truss bridge respectively [3].

Despite all these developments in bridge construction industry, bridge failure under variety of circumstances is one of the major worry of bridge designers and engineers. A bridge failure could be a disaster; lives of hundreds of people who pass through these bridges every day could be at risk.

In order to design a bridge three important factors should be considered: 1. Stability, which provides a safe passage for passengers, 2. Economy: which represent the efficiency of a bridge and finally, the aesthetic appeal of bridge structure. Once all these three factors overlapped together on would have optimum design.

The purpose of this thesis is to evaluate tied-arch and truss bridges with long and medium spans while they are subjected to wind and traffic loading. Therefore two tied-arch bridges with spans of 225 m and 126 m and two truss bridges with the same spans are designed. Later the bridges are compared according to the most important analysis outcome such as, support reaction, deflection and economy in order to identify the optimum bridge. Therefore, this thesis can be divided in to two main parts, bridge design (chapter 4) and bridge analysis and assessment (Chapter 5).

This research is aimed at describing the procedures for bridge design and specifying the appropriate solution through factors affecting the design of these types of bridges. Analysis part is expected to reveal the most critical points and sections, the reactions at bridge supports, bridge deflections and influence lines when bridges are subjected to wind and traffic loading.

1.4 Thesis Outline

This thesis comprises of six chapters including the introductory chapter. Appendices are also included to provide supporting data for bridge design, analysis results and model creation.

The introductory chapter summarizes the main features of bridge engineering, bridge structures, bridge classification and also gives a brief history of bridge, and discusses the aim and scope of this thesis.

In Chapter 2, properties and principal components of tied-arch and truss bridges have been discussed. Definition of bridge failures, progressive collapse of bridges and causes of bridge failures in the past has been discussed. Bridge design specification and limitation as specified in AASHTO LRFD have been described.

Chapter 3 comprises of, thesis methodology and methods of bridge analysis. Preliminary design of tied-arch and truss bridges with different spans and their geometry aspects specified by codes are discussed. Loads applied on bridges, load factors and combinations specified by AASHTO LRDF have also described. Truss and tied-arch Bridge’s geometry’s and factors affecting the design of these bridges have mentioned. Different loading on bridge structure specified in AASHTO LRDF have been discussed.

In Chapter 4, architectural design and plans of four bridges have provided. The preliminary analysis and design of the bridges were carried out and bridge sections designed accordingly. A summary of the materials and sections used for the bridges have been provided.

In Chapter 5, the bridges designed in chapter 4 were modeled and analysed by MIDAS software. The results of each bridge evaluated individually based on,

1. Forces
2. Deflections
3. Support reactions
4. Average dead load
5. Weight of the structure.

Finally, results were compared to reveal which bridge type is better in terms of stability, durability and economics.

In Chapter 6, summary of bridge engineering history, advantages and disadvantages of tied-arch bridges and truss bridges discussed. Brief requirements of American Association of State Highway and Transportation (AASHTO) specification for bridge design and analysis are mentioned. Comparison of long and medium span bridges have been gathered and summarized. Designed bridges were compared with other bridge types. Ways of preventing bridge failure were investigated. Finally, recommendations for future work on bridge study were provided.

Chapter 2

LITERATURE REVIEW

2.1 Arch Bridge

Arch can define as a curved structural member spanning an opening and serving as a support for the loads above the opening [8]. This definition omits a description of what type of structural element, a moment and axial force element, makes up the arch [8]. Figure 2.1 describes the arch bridges.

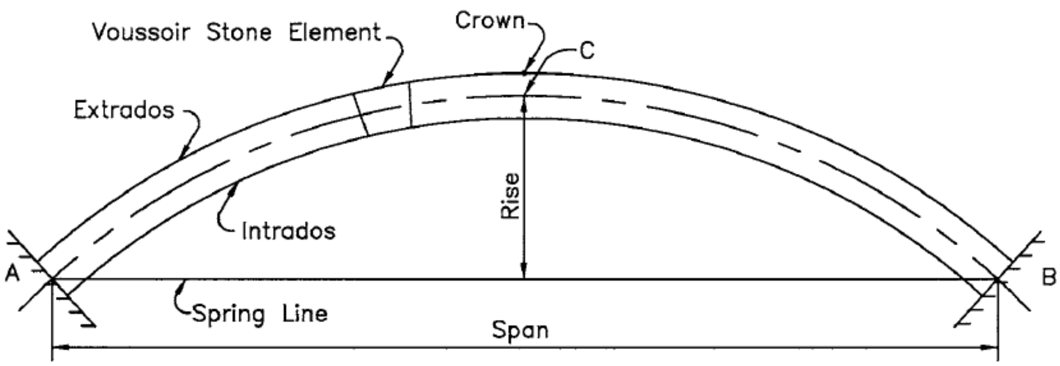


Figure 2.1: Arch Nomenclature [8]

Chen and Duan (2000) reviewed the arch bridge loading. They noted that arch bridges are usually subject to multiple loadings (dead load, live load, moving load, etc.) which produces bending moment stresses in the arch rib that are generally small compared with the axial compressive stress [8].

2.1.1 Tied-arch Bridge

In a tied-arch bridge, the thrust is carried by the arch solid rib, but for variable loading conditions the moment is divided between arch and tie, somewhat in proportion to the respective stiffness's of these two members [9]. In this type of arch bridge horizontal forces acting on the arch ribs are supplied by a tension tie at deck level of a through or half-through arch. The tension tie is usually a steel plate girder or a steel box girder and, depending on its stiffness, it is capable of carrying a portion of the live loads [8]. Since box section has high bending and torsional stiffness, they are usually preferred to the other sections especially with solid ribs and long span steel arch bridge [9, 10]. Tied-arch bridge components are shown in Figure 2.2.

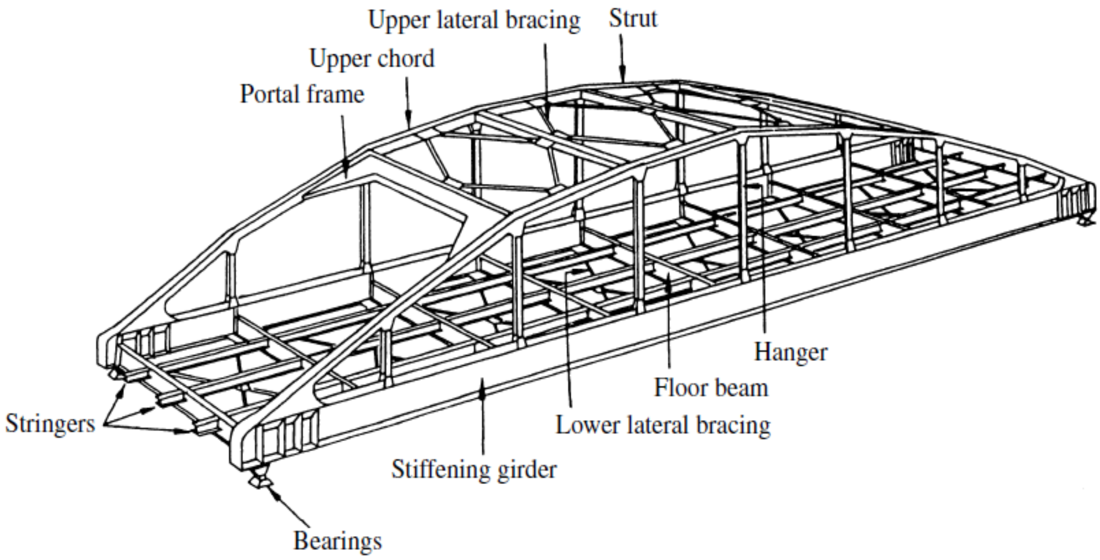


Figure 2.2:. Steel arch bridge components [5]

Merritt (2006) described the effect of arch rib and ties depth on each other.

> If a deep girder is used for the arch and a very shallow member for the tie, most of the moment for variable loading is carried by the arch rib. The tie acts primarily as a tension member. But if a relatively deep member is used for the tie, either in the form of a girder or a truss, it carries a high proportion of the moment, and a relatively shallow member may be used for the arch rib. (p.723)

According to Chen and Duan (2000),

> A weak tie girder requires a deep arch rib and a thin arch rib requires a stiff deep tie girder. Since they are dependent on each other, it is possible to optimize the size of each according to the goal established for aesthetics and/or cost. (p. 430)

Merritt and Brockenbrough (2006) did a study on effect of form of tied solid-ribbed arches on Economy of Construction. They checked two alternate designs of 228.5 m span arch bridges, one with a 1.5 m constant-depth rib and 3.8 m deep tie and the other with 3.1 m deep rib and 1.2 m deep tie. The results showed that the latter arrangement, with shallow tie and deep rib, required 10 % more material than the former alternative with deep tie. They calculated that the construction cost increased by 5 %, since the constant cost for fabrication and erection would not be affected by the variation in weight of material [9].

Merritt and Brockenbrough (2006) stated that, hangers must be designed with sufficient rigidity to prevent vibration due to aerodynamic forces or very slender members must be used. A number of long-span structures incorporate the latter device [9].

I-sections (welded or rolled), circular hollow sections or cables may be used for the hangers. Opinions differ about the optimum choice of section. Cables must be made of high tensile steel, due to the high stress and the effects of creep which cause the elongation [11]. In order to prevent vibrations, slender members such as wire rope or bridge strand must be use [9].

2.2 Truss Bridges

Merritt and Brockenbrough (2000) defines truss as a structure that acts like a beam but with many components or members, subjected primarily to axial stresses, and arranged in triangular patterns [9].

The ideal design of trusses is the one wher the end of each member at joint is free to rotate independent of the other members at the same joint. Otherwise, the member will be subjected to secondary stresses. On the other hand, if a truss subjected to loads other than joint or panel loads, then bending stresses would produce in that particular member [8, 9].

Early U.S engineers constructed pin connected trusses, in order to eliminate secondary stresses due to rigid joints. European's primarily used rigid joints. The rigid trusses gave satisfactory service and eliminated the possibility of frozen pins, which induce stresses not usually considered in design [9]. Experience indicates that rigid and pin-connected trusses are nearly equal in cost, except for long span [9]. Therefore modern design prefers rigid joints.

2.2.1 Truss Bridge Components

Principal parts of a highway through-truss bridge are illustrated in Fig. 2.3.

Chords are top and bottom members that act like the flanges of a beam. They resist the tensile and compressive forces induced by bending. In a constant-depth truss, chords are essentially parallel [9].

Web members consist of diagonal and vertical members, where the chords are essentially parallel, diagonals provide the required shear capacity, and verticals carry shear, provide additional panel points for introduction of loads and reduce the span of the chords under dead-load bending. Usually, deck loads are transmitted to the trusses through end connections of floorbeams to the verticals [8, 9].

End posts are compression members at supports of simple-span trusses. For practical reasons, trusses should have inclined end posts [9].

Sway frame or sway bracings should be placed between truss verticals to provide lateral resistance in vertical planes. Where the deck is located near the bottom chords, such bracing, placed between truss tops, must be kept shallow enough to provide adequate clearance for the passage of traffic below it [9].

Portal bracing is sway frame placed in the plane of end posts. In addition to serving the normal function of sway bracing, portal bracing also transmits loads from the top lateral bracing to the end posts [9].

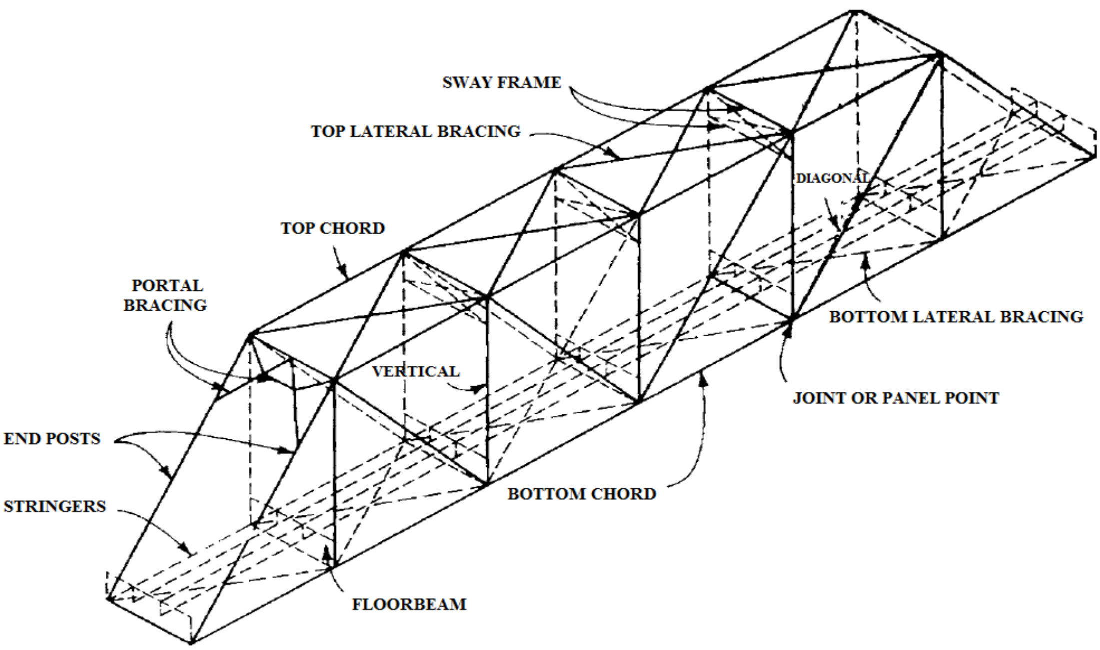


Figure 2.3: Components of truss bridge [9]

2.2.2 Warren Truss Bridge

Warren trusses (Figure 2.4), with parallel chords and diagonals, are generally, but not always, constructed with verticals in order to reduce panel size. Warren trusses are favored because of their web efficiency system when rigid joints are used. Most modern bridges are of Warren configuration [9].

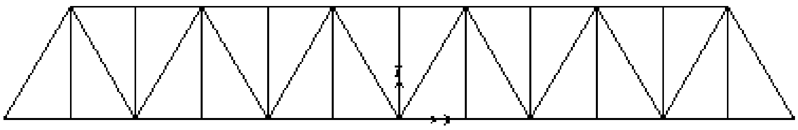


Figure 2.4: Warren truss with vertical members

Tension (mostly bottom chords) members should be arranged so that there will be no bending in the members due to eccentricity of the connections. If this is applicable, then the total stress can be considered uniform across the entire net area of the member [9].

Compression members should be arranged as such to avoid bending in the member due to eccentricity of connections. They should be designed in a way that the main elements of the section are connected directly to gusset plates, pins, or other members [9].

Posts and hangers are the vertical members in truss bridges which are designed as compression and tension members respectively [8]. A post in a Warren deck truss delivers the load from the floorbeam to the lower chord, and hanger in a Warren through truss delivers the floorbeam load to the upper chord. Posts are designed as compression member and hangers are designed as tension members [9].

2.2.2.1 Warren Truss Elements

There is a large variety of sections suitable for warren truss's tension and compression members. Basically choice will be influenced by the proposed type of fabrication and range of areas required for members [9]. Built-up Members such as, I-sections, channels and plates are used in the case of long span bridge trusses [12].

2.3 Bridge Failures

2.3.1 Overview

The collapse of the Tacoma Narrows Bridge is perhaps the best recorded and documented bridge failure in the bridge engineering history. The spectacular and prolonged failure process was captured on extensive live footage, giving a unique document for the investigation committee as well as for the engineering society at large [14]. The footage has since then been used in civil engineering classes all around the world for educational purposes. Consequences of neglecting dynamic forces in the construction of suspension bridges can be clearly observed [14].

Figure 2.5: Tacoma Narrows Bridge collapse [15]

The flexibility of the bridge decks (i.e. their lack of stiffness) can cause not only problems with vibration and swaying during wind loading, but also, when marching troops are passing. Through the combined effect of heavy wind and the steps interlocking with the Eigen frequency of the bridge, a large troop of marching soldiers in 1850 set the suspension bridge over the river Maine at Angers in France in violent vibrations. The bridge collapsed and 226 soldiers lost their lives [14].

2.3.2 Causes of Bridge Failures

In practice, failures occur in different forms in a material and are likely to be different for steel, concrete, and timber bridges. Common types of failure that occur in steel bridges are yielding (crushing, tearing or formation of ductile or brittle plastic hinges), buckling, fracture and fatigue (reduced material resistance, reversal of stress in welds and connections, vibrations), shearing and corrosion. Large deformations due to impact, sway, violent shaking during seismic events, erosion of soil in floods or settlement due to expansive soils may induce failure in both steel and concrete bridges [13].

The most common causes of bridge failure include: overstress of structural elements due to section loss, design defects and deficiencies, long- term fatigue and fracture, failures during construction, accidental impacts from ships, trains and aberrant vehicles, fire damage, earthquakes, lack of inspection and unforeseen events [13].

Any one of the above causes may contribute to bridge failure or may trigger a collapse, but failures actually occur due to a critical combination of loads [13].

Lessons from these failures should be treated as learning experiences, because when a bridge collapses it has certainly been pushed to the limit in some way. Therefore bridge collapses, have a significant effect on the development of the knowledge of structural action and material behavior and have spurred research into particular fields [13].

Causes of failures should be identified in any case to find ways to fix the problem and to avoid them in the future.

2.3.2.1 Causes of arch bridge failure

Researches show that 60 percent of bridge failures are because of scour which is the most frequent causes of bridge failure in the U.S.A [14]. Floods and collisions are a good example of this type of failure.

Bridge overload and lateral impact forces from trucks, barges/ships, and trains constituted 20 percent of the total bridge failures [13]. In the U.S.A. alone, over 36,000 bridges are either scour critical or scour susceptible [14].

Tables 2.1 to 2.3 are classified according to the causes of arch bridge failures and the details of the bridges involved are provided in their tables [15]. All causes of damage have been considered with the exception of acts of war, chemical action and natural catastrophes such as volcanic eruptions and landslides.

The Following tables are valid for all arch bridge types, but may not necessarily be applicable to tied-arch bridges alone.

2.3.2.1.1 Failure of arch bridge during construction

The failed arch bridges during construction or reconstruction are given in Table 2.1 [15]. It is interesting to note that two of the bridges listed (No.6 & No.10) failed during demolition.

Table 2.1: Failed arch bridges during construction, demolition or reconstruction [15]

Case No	Year	Bridge				Failure and Injuries
		Type	Country	For	Span (m)	
1	1892	Semi-parabolic truss arch	Serbia	Road	62	Chain collapse of arches shortly before completion. Probably caused by insufficient bearing capacity of lower sections of piers due to use of broken stone Masonry with rubble filling instead of cut stone. No Dead or injuries recoded
2	1894	R.C arch bridge	Germany	Road	54	The foundation with short piers on ground softened by floods was too weak for assumed restraint, causing overload of the arch crown cross section. No Dead or injuries recoded
3	1905	Tied-Arches.	Germany	Road	71	Failure of erection bridge, L=30m due to lateral displacement of upper chords while a 14m high portal crane was moving over the bridge. No Dead or injuries recoded
4	1908	Tied-Arch.	Germany	Rail	165	Truss auxiliary bridge of 65m, for construction of main span collapsed. Cause unknown. 8 people died and 111 people injured.
5	1910	Stone Arch bridge.	Germany	Road	-	Collapsed during dismantling immediately after removal of keystone. 1 person injured.
6	1926	R.C Arch Bidge.	Germany	Road	58	Underwater concrete in lower part of a pier of insufficient strength. Collapse of pier and two arches. 3 people died.
7	1959	Arch bridge	Sweden	Road	278	Transverse oscillations of slender tube columns, no collapse. No Dead or injuries recoded

8	1997	3-span concrete arch bridge with elevat-ed road deck.	China	Road	160	In each span 2 of 10 closely arranged arches, each 2.2 m deep and 1.56m wide box, were concreted. Arches of middle and one side span collapsed in one night, and about 12 weeks later also arches of other side span. The inquiry showed that arches were narrow in relation to span length and thus aerodynamically unstable. Arches collapsed due to overload caused by wind vibrations. No Dead or injuries recoded
9	2007	Stone arch bridge with 4 spans.				13, 2007. Cause: lack of standard design and construction. The bridge totally collapsed. 64 people died.

2.3.2.1.2 The Failure of Arch Bridges in Service

Arch bridges failed in service are listed in Table 2.2. Effects of accidental actions, seismic actions and explosions are not included. Three of these bridges have failed due to brittle fracture [13].

The 75 m span Vierendeel truss Hasselt tied-arch bridge (Case No.2) was one of the 52 welded arch type bridges built in Belgium in 1930's. It collapsed within a year of its opening to traffic [15]. The collapse occurred at the ambient temperature of -20ºC with clear indication of brittle fracture [13].

The damage found in the tied-arch bridge (Case N.4) was entirely due to the brittle fracture tendency of structural steel, which contrary to the specifications contained an excess of carbon, manganese and sulphur [15]. In 1982 a brittle fracture tore off the 70mm plates of the upper flange of tied-arch bridge (Case No.5) which was discovered only 15 months after the bridge opening. The failure was again attributed to lower than required steel toughness [13].

Table 2.2: Failure of arch bridges in service [13]

Case No	Year	Bridge				Failure and Injuries
		Type	Country	For	Span (m)	
1	1926	3-span R.C arch bridge Single track with piers for later double track.	Romania	Rail	30	A pier was suddenly displaced by approx. 1.2 m. Cause: old masonry pier used for foundation was too weak. Also high water 1.2 m above assumed highest water level. No Dead or injuries recoded.
2	1938	arch bridge	Belgium	Road	75	Brittle fracture of bow-shaped main girders. No Dead or injuries recoded.
3	1967	Masonry arch	Italy	Valley	312	The two upper middle arches of the 114-year old, three-level masonry arch bridge collapsed. Cause unknown. 2 People died.
4	1979	Suspended deck arch	U.S.A	Road	141	Crack in box stiffening girder led to closure of bridge. Cause: an excess of carbon, manganese and sulfur makes steel susceptible to brittle fracture. No Dead or injuries recoded
5	1982	Corrugated steel arched culvert	U.S.A	Road	-	10-year old culvert, 4.5m high collapsed. Cause: unsuitable. Filling material, also design errors, structure was too flexible. It was at that time the largest culvert structure in the USA. 5 people died and 4 injured.
6	1982	Suspended deck arch	U.S.A	Road	130	Brittle fracture in 800 x 70 upper chord plates that had been put in to replace a plate rejected due to surface defects. Independent testing showed lack of toughness in contradiction to testing of manufacturer. No Dead or injuries recoded
7	1999	CFST tied arch.	China	Road	140	Cause for the bridge collapse was poor construction quality, including bad welding quality of the arch rib and the insufficient strength of the in-filled concrete, which was less than 1/3 of the designed strength. Additionally, serious corrosion appeared on the hanger and anchorage. The other cause was unreasonable bid procedure. Design and construction contracts were illegal due to bribe. No Dead or injuries recoded

8	2001	Half through RC arch bridge	China	Road	224	The deck near the arch springing collapsed due to the break of the short hanger. Investigation found 4 pairs of symmetrical hangers broken. No Dead or injuries recoded
9	2003	Bridge steel suspended deck arch bridge.	Canada	Road	110	3 hangers of 40-year old bridge broke. Causes: possible bending of hangers due to poor design, steel did not satisfy toughness specifications, hanger sections were not accessible for inspection and fatigue. No Dead or injuries recoded
10	2006	R.C arch highway bridge	Venezuela	Road	300	Viaduct 1 on Caracas-La Guairá Highway collapsed after landslides along its entire length. Failure could not be prevented because the south mountainside was pressing on the viaduct. The earth movement increased in recent years and stability of all foundations was impaired. No Dead or injuries recoded.

Viaduct 1 concrete arch bridge (Case No.10), was the fifth largest concrete arch bridge in the world with 300m length, 21 m width and 61 m height [13]. It collapsed due to landslides along its entire length and although the problem was noticed on time, it could not be saved, because the earth movement just kept on increasing and pushing against the bridge foundations, particularly endangering the arch abutment (Figure 2.6).

Figure 2.6: Collapse of Viaduct 1 concrete arch bridge [13]
Left: east arch rupture (January 5, 2006); Right: Bridge collapse (March 19, 2006)

2.3.2.1.3 Failure Due to Flood Water and Scour

Failures of arch bridges due to flood water, scour and ice packs are numerous [13]. Many stone arch bridges comprising many arches of relatively small spans collapsed due to flood and scour [13]. In Table 2.3 some of them are listed.

Table 2.3: Failure due to flood water, scour and ice packs

Case No	Year	Bridge				Failure and Injuries
		Type	Country	For	Span (m)	
1	1926	4-span R.C arch bridge	Germany	Road	30	Collapse of a 25m span and severe damage to other bridge parts due to flood water scour. No Dead or injuries recoded.
2	1938	Truss arch bridge	U.S.A	Road	256	Pressure of ice on arch springing caused bridge to collapse. No Dead or injuries recoded.
3	1964	Old stone bridge 23 arches.	U.S.A	Rail	24	Scouring of two piers by extremely high water. Piers sank by up to 36 cm. Collapse did not occur No Dead or injuries recoded.
4	1978	13 stone arches.	France	Road	141	During flood water a pier sank and a span collapsed. Cause: wooden piles had rotten during low water periods in previous years. The next day the backwater build-up destroyed further piers and arches. No Dead or injuries recoded
5	1982	Stone arch bridge	Italy	Rail	70	2 piers scoured. 3 arches with total length of approximately 70 m destroyed. No Dead or injuries recoded
6	1987	Häderslis Bridge	Switzerland	Road	-	The masonry arch bridge built in 1969 was swept away in floods. No Dead or injuries recoded
7	1993	Stone arch bridge	Keyna	Rail	-	Flood water destroyed an arch of the 95 year old bridge just before a sleeper train crossed. 144 people died.
8	2003	Arch bridge	France	Road	-	Sudden swelling of Rhöne River and its tributaries damaged many bridges, some severely. In Givors the road deck of an arch bridge collapsed under a truck. No Dead or injuries recoded
9	2007	Stone arch bridge	Spain	Road	-	Heavy rain caused river to swell. The stone arch bridge was swept away No Dead or injuries recoded.

10	2010	Masonry arch bridge	China	Road	233.7	The bridge collapsed due to a heavy flood. 50 people dieed.

The most striking example of ice pack collision is the collapse of the Falls View arch bridge in 1938 (Case No.4) [15]. The ice jam piled up to the height of 15m above normal river level, or 3m above pins supporting the arch. The ice pack moved downstream like a glacier covering at least 9m of the upstream truss, causing the failure of bracing members and finally the buckled section of lower chord broke and the bridge collapsed [13].

2.3.2.1.4 Failure Due to Ship Collision

The number of failures of bridges due to impact of ship collision have dramatically increased over the years [13]. A notable example is the total collapse of a very famous arch bridge in Sweden, the Almö Bridge across the Askerö Sound near Göteborg (Fig. 2.7).

Figure 2.7: Arch bridge collapsed due to ship collision [15]

The bridge was across a busy navigation channel, where passage of ships up to 230,000t was allowed, it was opened to traffic in 1960 and hit by a 27,000t ship, not fully loaded in 1980, 35m from the west abutment. Seven cars were on the bridge at that time and 8 people died [13].

2.3.3 Progressive Collapse of Tied-Arch Bridge

Memorial Bridge (Case No.9) could be a great example of progressive collapse of tied-arch Bridges. As the function of the hangers was just to transfer the vertical loads to the arch, the inability of the pin joints to adjust to the rolling load on the bridge deck, led to back-and-forth bending deformations of the hangers [15]. Therefore over the long run a fatigue crack was initiated in one of the hangers. It was the hanger closest to the abutment of the northwest corner of the bridge that failed first, being the shortest thus experiencing more dynamic effects than the longer and softer hangers. After several years and due to cyclic loading, the hanger suddenly lost its load-carrying capacity, and it fell down, but was stopped after 75 millimetres [15].

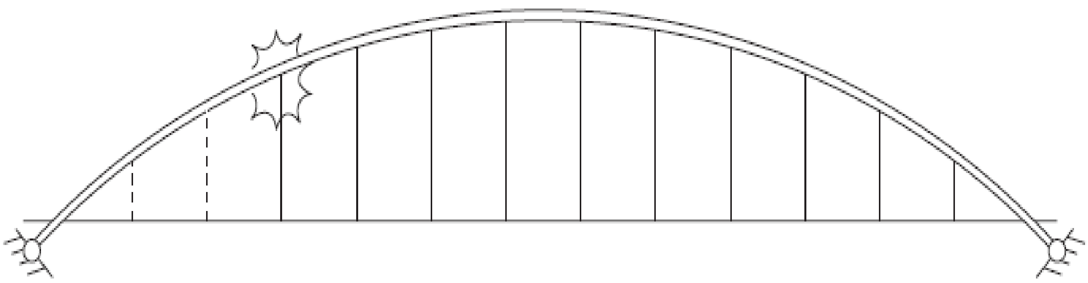


Figure 2.8: Broken of hanger 3 during the passage of a tractor-trailer [15]

As the load-carrying capacity of Hanger 1 became zero, the weight of the deck and the load from the traffic had to be transferred to the adjacent hanger (Hanger 2). Soon a fatigue crack was initiated in Hanger 2 and after a while,this hanger was also not able to carry loads. Hence Hanger 3 had become heavily strained (Figure 2.8) [15].

Finally during the passage of a southbound tractor-trailer on 14 January 2003, at around 3 in the afternoon, Hanger 3 finally broke. An extremely low temperature at the time of the trailer passage (−25◦C) contributed to the brittle fracture of Hanger 3. When Hanger 3 fractured the deck collapsed completely and fell down about two meters. (Figure 2.9) [15].

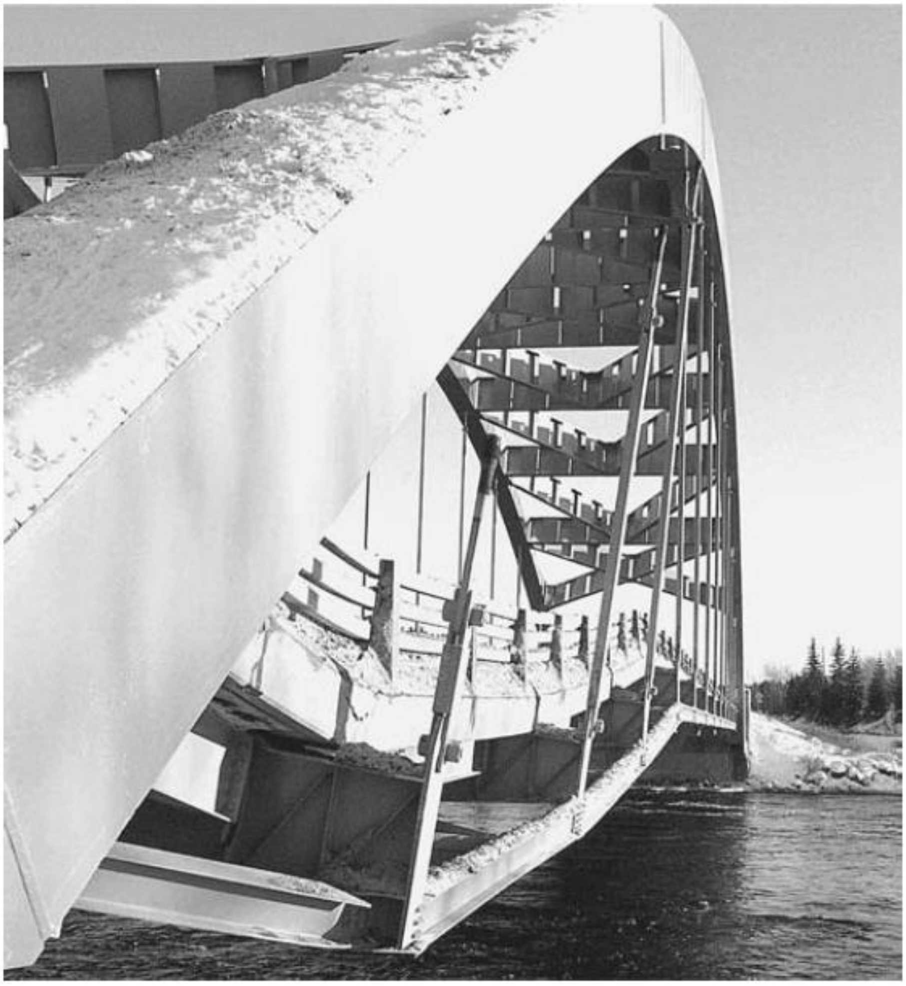


Figure 2.9: The (partial) collapse of tied-arch Bridge [15]

2.3.4 Failed Truss Bridges

Truss bridges failed over time are listed in Table 2.4 [3, 15]. The table prepared only includ the major failures which were fatal and/or total collapse. Many failures are neglected.

Table 2.4: Failures of steel truss bridge [3]

Case No	Year	Bridge				Failure and Injuries
		Type	Country	For	Span (m)	
1	1876	Truss Bridge	U.S.A	Rail	47	Bridge failure because of heavy snow and fatigue failure of iron elements. 92 killed and 64 injured.
2	1891	Iron Truss bridge	Switzerland	Rail	-	Train falls through the centre of bridge. Fatigue of iron and combined dead load and live load is the possible failure. 71 killed and 171 injured.
3	1907	Cantilever truss bridge	Canada	Road	549	The collapsed 152.5m long south anchor arm of the Quebec Bridge occurs because of distress I anchor's. No Dead or injuries recoded.
4	1945	Truss Bridge	Germany	Rail	-	Collapse due to previous battle damage. 28 soldiers killed.
5	1958	Through Truss	Canada	Road/ Rail	142	It was found that the lower transverse beam at the bottom of the falsework truss had failed. The purpose of this beam was to distribute the concentrated load. 19 killed 72 injured.
6	1967	Suspended Truss bridge	U.S.A	Road	445	The suspenders were not able to carry the loads after 40 years, and one afternoon the bridge fell down. 46 people killed.
7	1978	Truss	Scotland	Road	75	When the storm had calmed down, the extent of the tragedy became evidently clear. The entire high-girder section had collapsed into the river; close to one kilometre of the bridge gone. 75 people killed.
8	1989	Truss Bridge	U.S.S	Road	-	15 m of the upper section collapsed. 1 person killed.
9	2007	Arch Truss	U.S.A	Road	139	The bridge's design specified steel gusset plates that were undersized and inadequate to support the intended load of the bridge a load which had increased over time. 13 people killed 139 injured.

2.3.5 Progressive Collapse of Truss Bridges

If a single primary member or gusset plate connection of the main trusses fails then the steel deck truss bridges being determinate systems and not having redundancy and can progressively collapse over the entire span [16].

On 2007 the 40 years old I-35W steel deck truss bridge over the Mississippi River in Minneapolis, suddenly and without almost any noticeable warning collapsed entirely into the river, causing the deaths of 13 people and injury to more than 100 others who were crossing the bridge in their vehicles at the time of the collapse (Case No.9) [16]. The failed bridge can be seen in Figure 2.10.

Figure 2.10: View of the collapsed I-35W bridge [3]

The connections in the main trusses were double gusset plate shop-riveted and field-bolted. Researchers and engineers believed that it was the gusset plates that fractured on August 1, 2007 through their net section and initiated the progressive collapse of the entire I-35W bridge [16].

Analysing the pictures of the bridge taken 4 years before the collapse, one could say that the gusset plates had already developed edge buckling failure mode due to the addition of dead load of the 5 cm wearing surface and curbs during or prior to 2007 [16].

Researchers indicate that due to corrosion, some gusset plates and even some members may have thinned over the years and did not have the originally designed thicknesses at the time of collapse [16]. Some engineers and investigators in Berkley University believe that the gusset plate thickness were much less than what would be needed by design according to the governing specification, AASHTO Specification 1961 [16].

The addition of considerably heavy loads due to vehicles, construction material and equipment the gusset plates got over-stressed and reach to the limit of their net section capacity and fracture through the net section. After fracture of the net section of gusset plate, the progressive collapse of the main trusses occurred quite rapidly and in a brittle manner due to lack of redundancy in the trusses and presence of net sections in the perforated members and finally the bridge totally collapsed [16].

2.4 AASHTO LRFD Specification and Limitations

The bridge design standards prescribed by the American Association of State Highway and Transportation Officials (AASHTO) have followed a design philosophy called Allowable Stress Design (ASD), in 1931 [17].

In the 1950s, as extensive data on failure mechanisms of structures began to accumulate in laboratories, researchers recognized some weaknesses in the concepts of the ASD code [17]. Allowable stress codes do not permit design directly against the actual failure limit states; unless those limit states occur within the elastic range

[8]. This limitation applies to all materials where inelastic behavior occurs at the onset of failure [17].

The first generation of AASHTO code to use a limit state method for design of steel structures is called Load Factor Design (LFD). It was introduced in the 1970s as an alternative to the ASD specifications [17]. Researchers began developing the new design specifications by using the probabilistic concepts that have been the subject of intensive research since around 1969. In 1986, AASHTO started to look into ways of incorporating Load Resistance Factor Design (LRFD) philosophies into the standard specifications [8, 17].

David Simons (2007) conducted a series of bridge design to compare the results of design and analysis difference of LRFD and ASD. Several bridges of different spans covering the range most commonly encountered in practice were selected for design. In this study, he realized that, the girders designed by using the LRFD specifications typically required less steel than the girders designed using the ASD code. Material savings between 20 – 30% was observed over the entire bridge when using the LRFD specification. The bridges designed by using the LRFD code became more efficient as fewer girders were used.

2.4.1 Arch Bridge

2.4.1.1 Rise-Span Ratio

The most suitable ratios of rise to span cover a range of about 1:5 to 1:6 [9, 2]. This might be variable depending on the span length [9]. There are bridges with rise-span ratio range from a maximum of 1:4.5 to minimum of 1:7.6 (For example: Arkansas River Bridge and Old State Route 8 Bridge, U.S.A respectively) [2, 9]

2.4.1.2 Panel Length

For solid-ribbed arches fabricated with segmental chords, the panel length should not exceed 1/15 of the span. This is recommended for esthetic reasons, to avoid large angular breaks at panel points. Also, for continuously curved axes, bending stresses in solid-ribbed arches become fairly severe if long panels are used [9].

2.4.1.3 Depth-Span Ratio

For tied-arches having solid ribs with constant depth and deep ties, rib depth may be small, because the ties carry substantial moments, thus reducing the moments in ribs [9]. Ratio of 1:100 to 1:120 would be suitable for such structures [2]. In some cases this ratio goes as low as 1:187 (Glen Field Bridge, U.S.A) [9].

2.4.1.4 Allowable Deflection

The Standard Specifications impose deflection limitations. Highway bridges consisting of simple or continuous spans should be designed so that deflection due to live load plus impact does not exceed 1/800th of the span. For bridges available to pedestrians in urban areas, this deflection should be limited to 1/1000th of the span [9, 18].

2.4.2 Truss Bridge

2.4.2.1 Span-Depth Ratio

A span-to-depth ratio between 6 and 8 for railway bridges and between 10 and 12 for road bridges offer the most economical design [2, 19]. In general terms the proportions should be such that the chords and web members have approximately an equal weight [19].

2.4.2.2 Truss division length

The bay widths should be proportioned so that the diagonal members are inclined at approximately 50° or slightly steeper. For large-span trusses subdivision of the bays is necessary to avoid having excessively long web members [19]. The spacing of bridge trusses depends on the width of the carriageway for road bridges and the required number of tracks for railway bridges [19]. In general the spacing should be limited to between 1/18 and 1/20 of the span, with a minimum of 4m to 5m for through trusses and approximately 1/15 of the span [18, 19].

2.4.2.3 Allowable Deflection

Deflection of steel bridges has always been important in design. If a bridge is too flexible, the public often complains about bridge vibrations, especially if sidewalks are present that provide access to the public [9]. Bridges should be designed to avoid undesirable structural or psychological effects from their deflection and vibrations [9]. According to F.S Merrit (2005) "While no specific deflection, depth, or frequency limitations are specified herein, any large deviation from past successful practice regarding slenderness and deflections should be cause for review of the design to determine that it will perform adequately".

The Standard Specifications impose deflection limitations. Highway bridges consisting of simple or continuous spans should be designed so that deflection due to live load plus impact does not exceed 1/800th of the span. For bridges available to pedestrians in urban areas, this deflection should be limited to 1/1000th of the span for safety and comfort of the passengers [9, 18].

According to recommended Specification, deflections due to steel, concrete and future surfacing weight should be reported separately. There are no limitations for deflection due to dead load, but vertical cambers should be specified to compensate for computed dead load deflections [9, 18].

Chapter 3

METHODOLOGY, MODELING AND LOADING

3.1 Methodology

A series of design studies were conducted using AASHTO LRFD specification. All bridges considered as Roadway Bridge with multilane and orthotropic deck.

Tied-arch Bridge and Truss Bridge of different spans covering the range most commonly encountered in practice were selected for design. Each bridge was designed with a span of 225 m and 126 m which is considered as large and medium span. For more accurate comparison between the two sets of design, the same deck plan design and similar width was used in each case, only varying with the thickness and the number of girders. Orthotropic deck has been considered for bridges, which is the most common deck system. All bridges considered are Roadway Bridge and they are simply supported. Width of 33 m with six lane and 20 m with four lanes were used for those with large and medium span respectively. For each bridge sidewalk has also been considered. The designs were not fully detailed and some aspects of bridge design, such as fatigue, non-linear analysis and substructure design were not taking into consideration. It should be noted that these aspects do not significantly influence the results of interest in the current study. However, sufficient information was obtained about the design of the superstructure to achieve a reasonable result.

After the design was completed, bridges modeled and analyzed under dead load, vehicular live load, dynamic allowance (impact factor) and wind load by Midas Civil software.

Once bridge analyses were done, numerous comparisons were made between the Tied-arch and Truss bridges with different spans. The maximum live load deflection under service loads, maximum wind load displacement, support reactions and the amount of steel required for each bridge were selected as appropriate points of comparison between the two sets of design.

3.2 Geometry of Bridges

3.2.1 Long Span Tied-Arch Bridge

Since it has been decided to use of long and medium span bridges, span of 225 m satisfy the long span requirement and provides a clear length and number of panels. (Figure 3.2)

3.2.1.1 Arch Rise

The rise of the bridge is an important factor for both the structural behaviour and the aesthetic [9]. AASHTO specification advises a rise range between 1:5 and 1:6, although this is interchangeable due to the requirement of the design [18].

In order to satisfy both structural and aesthetic requirements, rise of 1:6 has been chosen for this study (See Appendix A for details).

3.2.1.2 Panel Arrangement

Because of esthetic reasons panels should not exceed 1/15 of the bridge span [9]. Therefore 15 panels provided (See Appendix A for details).

3.2.1.3 Arrangement of Hangers

Hangers provided every 15 metres. Their height can be obtained from the arc equation in x-y plane. (Figure 3.1)

$$y = f\left[1 - \left(\frac{2x}{L} - 1\right)^2\right]$$

Where f is the crown of the arc and L is the span of the bridge.

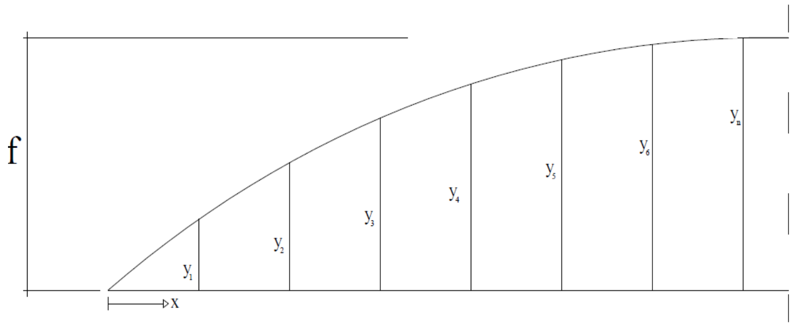


Figure 3.1: Arc equation parameters

3.2.1.4 Arch Bridge Elevation

After considering all the mentioned factors by AASHTO LRFD Specifications, the favorable tied-arch bridge has been modeled and illustrated in Figure 3.2.

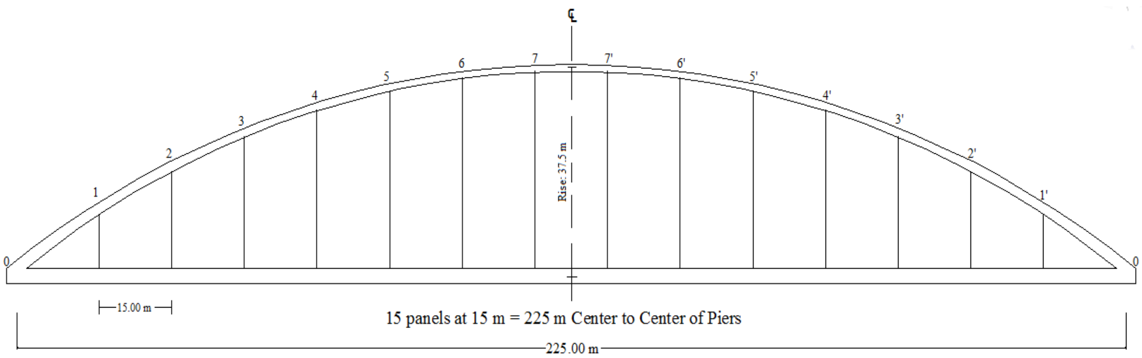


Figure 3.2: Tied-arch bridge elevation

The plan layout of bridge's lateral bracings, roadway and elevation are provided in Appendix A.

3.2.2 Medium Span Tied-Arch Bridge

As stated previously, bridge spans ranging from 50 m to 200 m is considered as medium span bridges [6]. A bridge with 126 m of span provided as medium span tied- arch bridge.

All those factors for long arch bridge, such as arch rise is applicable in medium span as well. Because of the bridge esthetic rise to span ratio of 1:6.3 has been used. Elevation of tied-arch bridge with 126 m span is shown in Figure 3.3.

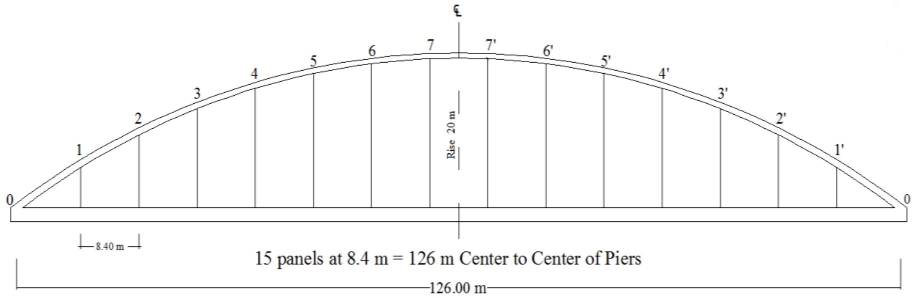


Figure 3.3: Tied-arch bridge elevation (126m)

3.2.3 Long Span Truss Bridge

In order to obtain a more accurate result and better comparison, same span truss bridge 225 m has been chosen.

3.2.3.1 Truss Depth

For roadway truss bridges span to depth ratio between 10 and 12 gives the most economical design [9]. For this purpose ratio of 12 was used. (See Appendix A)

3.2.3.2 Truss Division

Since the truss divisions (Panels) are limited to $1/18^{th}$ or $1/20^{th}$ of the span [9], 18 panels were provided for long span truss bridge. (See Appendix A)

3.2.3.3 Inclination of Diagonals

Panel's bays provided in such a way that the diagonal's angle are θ = 56.3 ° , which satisfy the codes requirement. In Figure 3.4 diagonals configuration has been shown.

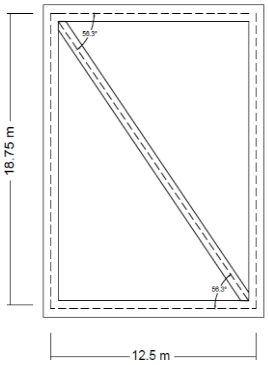


Figure 3.4: Inclination of Diagonals

3.2.3.4 Truss Bridge Elevation

By consideration of mentioned specification, Truss bridge of 225 m modeled according to AASHTO LRFD specifications and the elevation is illustrated in Figure 3.5.

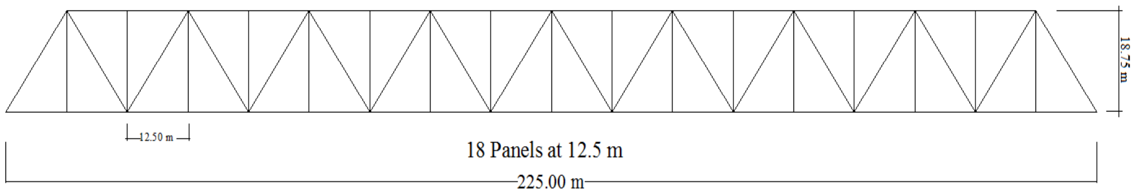


Figure 3.5: Warren truss elevation

3.2.4 Medium Span Truss Bridge

A bridge with 126 m of span provided as medium span truss bridge. 14 panels each are being 9 m were provided, so that the diagonals inclination angles become close to the code requirements (θ =54.3 °) (See Figure 3.6).

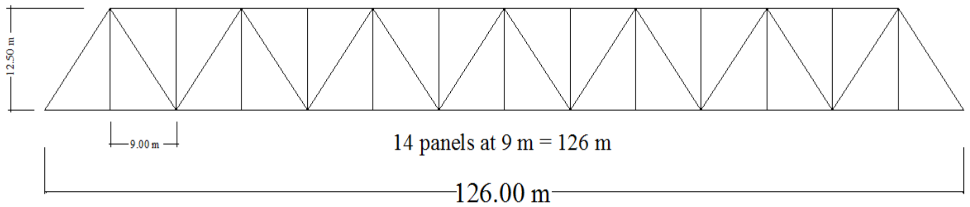


Figure 3.6: Medium span truss bridge elevation

3.3 Bridge Loading

Various types of loading which need to be considered for analysis and design are classified as permanent or transient (variable). Permanent loads are those due to the weight of the structure itself and permanently attached to the structure. They act on the bridge throughout its life. Transient loads are those loads that vary in position and magnitude and act on the bridge for short periods of time such as live loads, wind loads and seismic loads etc.

The following permanent and transient loads and forces specified by AASHTO LRFD were considered for this study [18].

1. Permanent Loads
   - Dead load of structural components (*DC*)
   - Dead load of wearing surfaces (*DW*)
2. Transient Loads
   - Vehicular dynamic allowance (*IM*)
   - Vehicular live load (*LL*)
   - Pedestrian live load (*PL*)
   - Wind on live load (*WL*)
   - Wind load on structure (*WS*)

3.3.1 Dead Load

The dead load on superstructure is the weight of all structural and non-structural parts of the bridge components above the bearing. This would include the girders, floor beams, stringers, the deck, sidewalk, bracings, earth covering, utilities, parapets and road surfacing [18].

3.3.2 Live Load

3.3.2.1 Vehicular Loads

The live load for bridges consists of the weight of the applied moving load of vehicles and pedestrians. The traffic over a highway bridge consists of different types of vehicles. To form a consistent basis for design, standard loading conditions are applied to the design model of structure. These loadings are specified AASHTO LRFD. These loads divided as follows:

1. Design Truck

   The weight and the spacing of the axle and wheel for design truck shall be as specified in Figure 3.7.

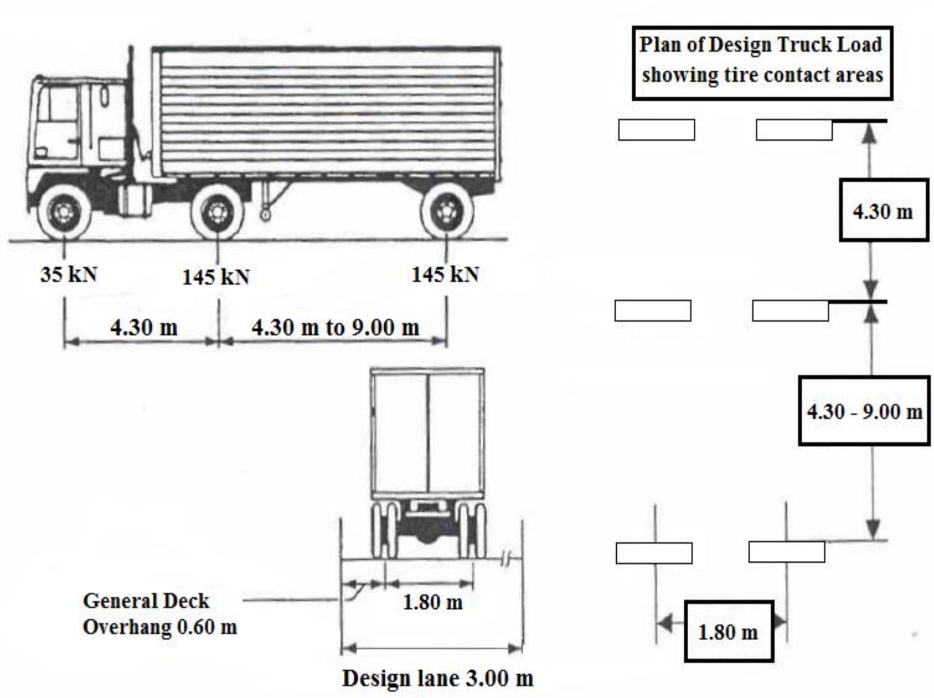


Figure 3.7: Characteristics of the Design Truck [18]

2. Design Tandem

   The design tandem used for strategic bridge shall consist of a pair of 110kN axles spaced at 1.2m apart. The transverse spacing of wheels shall be taken as 1.8m. The design tandem load is shown in the Figure 3.8

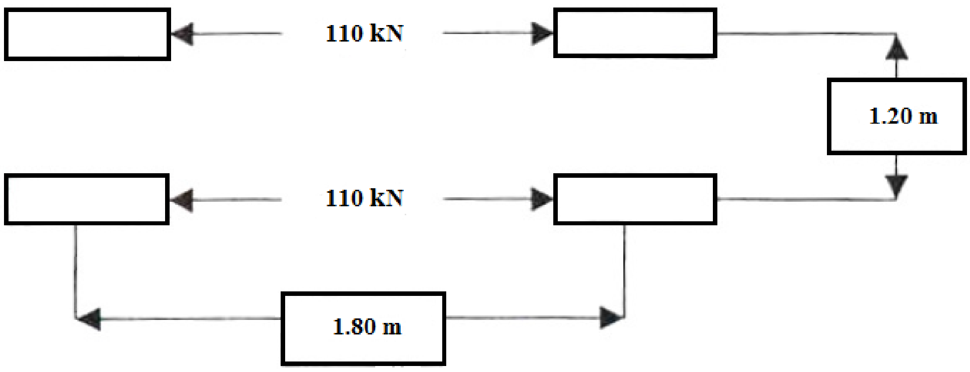


Figure 3.8: Characteristics of the Design Tandem [18]

In both tandem and truck design a dynamic load allowance shall be considered [18].

3.3.2.2 Design Lane Load

The design lane load shall consist of a load of 9.3kN/m, uniformly distributed in longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over 3m width. A dynamic load allowance shall not be considered [18].

Where the traffic lanes are less than 3.60 m wide, the number of design lane shall be equal to the number of traffic lane and the width of the design lane shall be taken as the width the traffic lane [18].

3.3.2.3 Pedestrian Loads

A pedestrian load of 3.6 kN/m^2 shall be applied to all sidewalks wider than 60 cm and considered simultaneously with the vehicular design live load in vehicle lane.

3.3.3 Dynamic Load Allowance

The truck loads on bridges are applied not gradually but rather rough, causing increase in stress. Therefore, additional loads called impact loads must be considered [2]. The static effect of the design truck or tandem shall be increased by the percentage specified by AASHTO LRFD article 3.6.1.7 [18].

The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load. The factor to be applied to the static load shall be taken as:

$$\left(1+\frac{IM}{100}\right)$$

Where *IM* is given in Table 3.1 from AASHTO LRDF Specifications [18].

Table 3.1: Dynamic Load Allowance, *IM* [18]

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

3.3.4 Wind Load

Wind load shall be assumed to be uniformly distributed on the area exposed to the wind [18]. The exposed area shall be the sum of areas of all components including railing.

It has been assumed that the superstructure of bridges is 20 m above the low ground and that it is located in open country.

3.3.4.1 Wind Load on Structure

The direction of the design wind shall be assumed to be horizontal. Design wind pressure, P_D, used to compute the wind load on the structure, W_S, which is determined as specified in article 3.8.1.2 [18].

In the absence of more precise data, design wind pressure, in kPa, can be determined as follows [18]:

$$P_D = P_B \frac{V_{DZ}^{\ 2}}{240}$$

Where: P_B = base wind pressure (ksf) specified in article 3.8.1.2.1 (see Table 3.2)

V_{DZ} = design wind velocity (mph) at design elevation, Z

Table 3.2: Base Pressure P_B in ksf [18]

Superstructure Components	Windward Load, ksf	Leeward Load, ksf
Trusses, Columns And Arches	0.050	0.025
Beams	0.050	NA
Large Flat Surfaces	0.040	NA

For bridges or parts of bridges more than 10 m above low ground or water level, the design velocity, V_{DZ},, in mph should be adjusted according to:

$$V_{DZ} = 2.5\ V_0 \left(\frac{V_{30}}{V_B}\right) \ln\left(\frac{Z}{Z_0}\right)$$

Where: V_{30} = wind velocity at 10 m above design water level (mph)

V_B = base wind velocity of 100 mph

Z = height of structure at which wind load are being calculated,> 30 ft

V_0 = friction velocity (mph) specified in article 3.8.1.1 (See Table 3.3)

Z_0 = friction length (feet) specified in article 3.8.1.1 (See Table 3.3)

Table 3.3: Values of V_0 and Z_0 for various surface conditions m [18]

Condition	Open Country	Suburban	City
V_0 (mph)	8.20	10.90	12.00
Z_0 (ft)	0.23	3.28	8.20

3.3.4.2 Wind Pressure on Vehicles

When vehicles are present, the design wind pressure shall be applied to both structures and vehicles. Wind pressure shall be represented by an interruptible, moving force of 1.45 kN/m acting normal to, and 1.80 m above, the roadway shall be transmitted to the structure.

3.3.5 Earthquake Load

Seismic analysis is not required for superstructure of single-span Bridge, regardless of the seismic Zone 1 [6].

In Appendix B calculation of actions and applied loads on bridges such as dead load, traffic loading and wind load have provided.

3.4 Load Factors and Combinations

Applied loads such as dead load, live load and wind load are factored and combined to produce extreme adverse effect on the members being designed. All components of the bridges shall be designed under the applicable combinations of factored extreme force effect [18]. These factors and combinations have specified by AASHTO LRFD in Section 3.4.1 (See Table 3.4)

In this study for bridge design, Strength I for dead load and live load combination, Strength III for dead load and wind load combination and Service I for dead load, live load and wind load combination have been used in order to find the most critical load combinations.

Table 3.4: Load combination and factor [18]

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use one of these at a time			
										EQ	*IC*	*CT*	*CV*
Strength I	γ_p	1.75	1.00	-	-	1.00	0.5/ 1.2	γ_{TG}	γ_{SE}	-	-	-	-
Strength II	γ_p	1.35	1.00	-	-	1.00	0.5/ 1.2	γ_{TG}	γ_{SE}	-	-	-	-
Strength III	γ_p	-	1.00	1.40	-	1.00	0.5/ 1.2	γ_{TG}	γ_{SE}	-	-	-	-
Strength IV	γ_p	-	1.00	-	-	1.00	0.5/ 1.2	-	-	-	-	-	-
Strength V	γ_p	1.35	1.00	0.40	1.00	1.00	0.5/ 1.2	γ_{TG}	γ_{SE}	-	-	-	-
Extreme Event I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.0	-	-	-
Extreme Event II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.0	1.0	1.0
Service I	1.00	1.00	1.00	0.30	1.00	1.00	1.0/ 1.2	γ_{TG}	γ_{SE}	-	-	-	-
Service II	1.00	1.30	1.00	-	-	1.00	1.0/ 1.2	-	-	-	-	-	-
Service III	1.00	0.80	1.00	-	-	1.00	1.0/ 1.2	γ_{TG}	γ_{SE}	-	-	-	-
Service IV	1.00	-	1.00	0.7	-	1.00	1.0/ 1.2	-	1.0	-	-	-	-
Fatigue I-*LL, IM & CE Only*	-	1.50	-	-	-	-	-	-	-	-	-	-	-
Fatigue I II – *LL, IM & CE*	-	0.75	-	-	-	-	-	-	-	-	-	-	-

Where:

γ_p = Load factor for permanent loading

DC = Dead load of structural components

DW = Dead load of wearing surface

IM = Vehicular dynamic allowance

LL = Vehicular live load

BR = Vehicular braking

LP= Pedestrian live load

WL = Wind on live load

WS = Wind load on structure

EQ = Earthquake load

Chapter 4

2-D ANALYSIS AND DESIGN OF BRIDGES

4.1 Overview

In this chapter, design of tied-arch bridge and truss bridge in different spans has been discussed. Bridges have been designed in accordance to AASHTO LRFD specifications and design procedure of bridges with long span has been discussed. Same procedure has been applied for medium span bridge and sections designed for final analysis.

Application of loads and actions on bridges are provided in Appendix B and the results have been used for design.

4.2 Tied-Arch Bridge Design

In order to calculate dead load with more accuracy the design of tied-arch bridge components has started with slab of the deck (orthotropic), since slab dead load transfers to stringers, then to floor beams and finally ties.

Computers greatly facilitate preliminary to final design of all structures. Designs of the bridge components were done by hand calculation and for accurate analysis 2-D models of tied- arch, stringers, floor beams have been simulated and analyzed by SAP2000 software. Design of the tied-arch bridge is discussed in the next sections.

4.2.1 Long Span Bridge (225m)

4.2.1.1 Deck Concrete Slab design

Assumed cross section of the roadway slab is shown in Figure 4.1. Since the design of deck slabs using the traditional method (based on flexure) are still permitted by the LRFD Specifications [9], then the concrete slab was designed using this method.

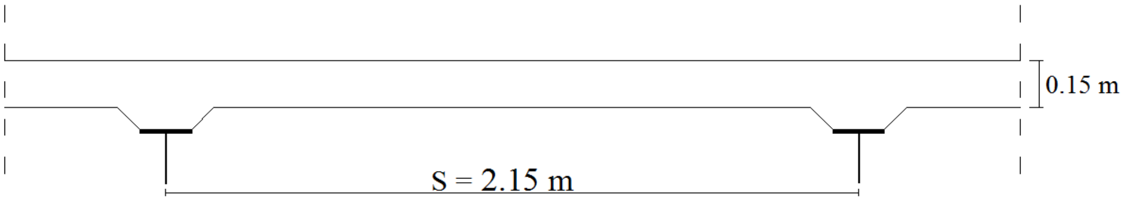


Figure 4.1: Concrete Slab cross section

Moment due to dead load:

$$M_d = \pm\frac{qL^2}{10} = \pm\frac{4.7\times2.15^2}{10} = 2.2 \text{ kNm}$$

Moment due to live load:

$$M_l = \pm 0.8 \times \frac{(1.64\times S)+1}{16} \times P = \pm 0.8 \times \times 80 = \pm 18.1 \text{ kNm}$$

Dynamic allowance (Impact factor):

From Table 4.1 => IM = 33% , $M_l = 18.1 \times 1.33 = 24.073$ kNm

Total Moment:

$M_T = M_d + M_l = 26.3$ kNm

Required area of reinforcing bars:

$f_s = 2000 \frac{kg}{cm^2}$ $\quad$ $f_c = 0.4\, f_c^{'} = 0.4(250) = 100\frac{kg}{cm^2}$ $\quad$ $\frac{f_s}{f_c} = 20$

$n = 9$ $\quad$ $k = \frac{9}{9+2} = 0.325$ $\quad$ $J = 1 - \frac{k}{3} = 0.875$

$$A_s = \frac{M}{f_s Jd} = \frac{26.3\times10^4}{2000\times0.875\times11} = 13.67 \text{ cm}^2$$

Reinforcing bars for slab are listed in Table 4.1.

Table 4.1: Distribution of slab reinforcement

Elements	Minimum Reinforcement	Minimum Area	Corresponding Area
Longitudinal Bars	Ø20/20 cm	13.35 cm^2	15.70 cm^2
Distributed Bars	Ø16/20 cm	8.94 cm^2	10.05 cm^2

4.2.1.2 Design of Stringers

Stringers are placed on bridge width at every 2.15 m and floor beams are placed at bridge length every 15 m to support stringers.

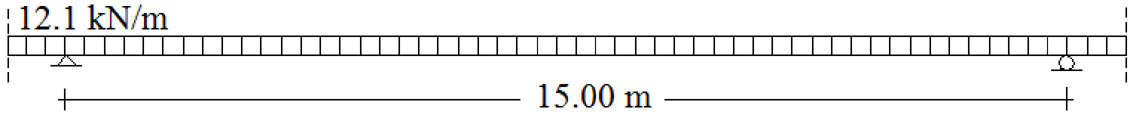


Figure 4.2.:Dead load application on stringers

Specification live loading used in this study is HS20-44 Truck loading. The application of this loading has shown in Figure 4.3

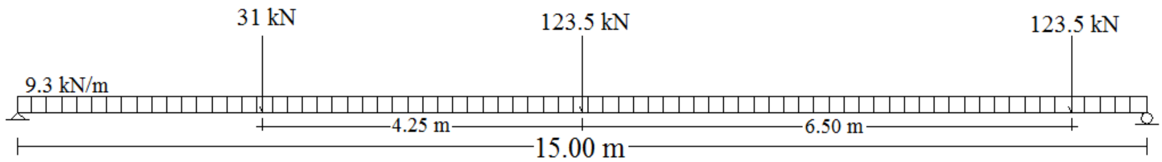


Figure 4.3: Application of design truck on stringers

Maximum moments and shear forces due to dead load, live load and dynamic allowance have been obtained by SAP2000 software and listed in Table 4.2. In order to indicate the most critical value, Strength I from load combination limit state of AASHTO LRFD specification which is given in chapter 3, is chosen.

*Strength I = 1.25 DL + 1.25 DW + 1.75 (LL+I)

Table 4.2: Design moments and reactions for stringer

Load Type	Maximum Bending Moment	Maximum Shear Force
Dead Load (DC)	216.57 kNm	57.5 kN
Dead Load (DW)	67.5 kNm	18 kN
LL + I	816.55 kNm	246.3 kN
Factored *	1783.08 kNm	525.33 kN

Design:

Web: $\frac{1}{20} \times L = \frac{1}{20} \times 1500 \text{ cm} = 75 \text{ cm}$, Thickness = t = 15 mm

Check: $\frac{h}{t} < 170 = \frac{75}{1.5} = 50 < 170$ O.K

$F_b = 0.55\ f_y = 0.55 \times 270000 = 148500$ kPa

$$A_f = \frac{M}{f_s d} - \frac{A_w}{6} = \frac{1783.08}{148500 \times 0.78} - \frac{0.75 \times 0.015}{6} = 0.0135 \text{ m}^2 = 135 \text{ cm}^2$$

Flange design:

$A_f = 135 \text{ cm}^2$ Selected: 500 × 30 mm H = 80.4 cm

$$f_b = \frac{1783.08}{0.012463} = 121878.4 < F_b \quad \text{O.K}$$

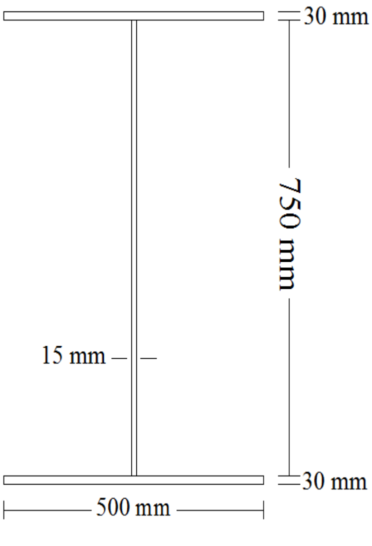


Figure 4.4: Stringers cross-section

4.2.1.3 Design of Floorbeams

Determination of dead load:

Floor beams placed every 15 m on length of the bridge. A bracing is placed at center of every floor beam deck (Appendix A). In Figure 4.5 application of dead load on floor beams has shown. The additional load from deck bracing is also added.

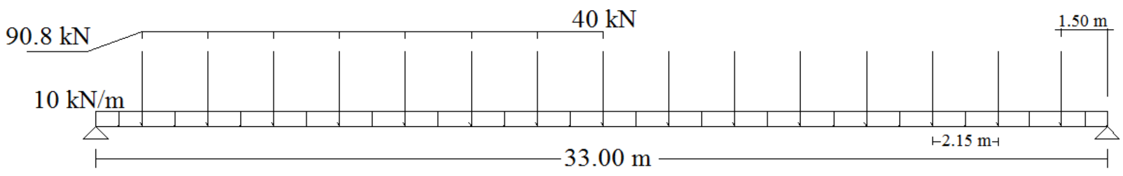


Figure 4.5: Stringers and bracing reaction on floor beam

Determination of support reaction, R:

In order to find the maximum forces from vehicles on floor beams, all possible truck arrangement on a series of stringers (2 stringers) were checked. The most critical arrangement that causes the extreme reaction is shown in Figure 4.6.

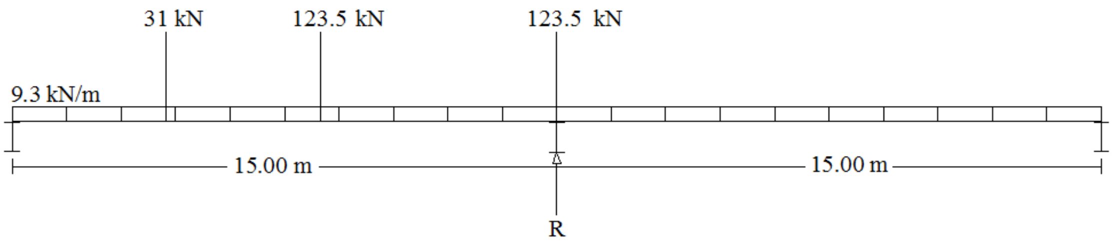


Figure 4.6: Application of truck loading on stringers

R obtained from SAP2000 Software: R = 411.13 kN

Determination of maximum moment at mid span:

In every cross-section of the deck three vehicles will be placed next to each other, there will be 6 lanes on the deck. Live load for sidewalk shall apply since its width is greater than 0.7m [18].

In Figure 4.7 equivalent wheel load reaction arrangement on floor beams is shown.

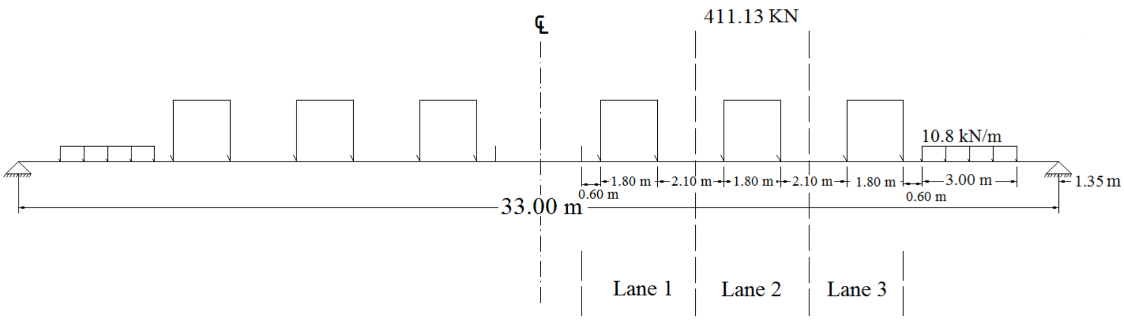


Figure 4.7: Equivalent wheel load reaction

Maximum moment and shear force were obtained by SAP2000, As AASHTO LRFD specifies in Article 3.6.1.1.2, multiple presence factors shall be used when effect of three or more lanes loaded are investigated [18].

M_{max} = 24766.87 kN $\qquad$ V_{max} = 2808.3 kN

Since the number of loaded lanes is greater than three, the presence factor shall be taken as 0.65 [18].

Maximum moment, shear force and load combination (Strength I) are given in Table 4.3.

Table 4.3: Design moments and reactions for floor beam

Load Type	Maximum Bending Moment	Maximum Shear Force
Dead Load $(DC+DW)_{stringer}$	5489.7 kNm	665.42 kN
Dead Load $(DC+DW)_{Floor\ beam}$	1735.94 kNm	190.42 kN
LL + I	16098.47 kNm	1825.38 kN
Factored *	37204.37 kNm	4264.22 kN

Design:

H_w = 3.00 m $\qquad$ t_w = 1.4 cm

$$A_f = \frac{M}{f_s d} - \frac{A_w}{6} = \frac{37204.4}{148500\times3.1} - \frac{3\times0.014}{6} = 0.0728\ m^2 = 728\ cm^2$$

Flange Design:

$A_f = 728\ cm^2$ Selected: 130×6.5 cm

$f_b = \frac{37204.4}{0.2611} = 142490\ kPa < F_b$ O.K

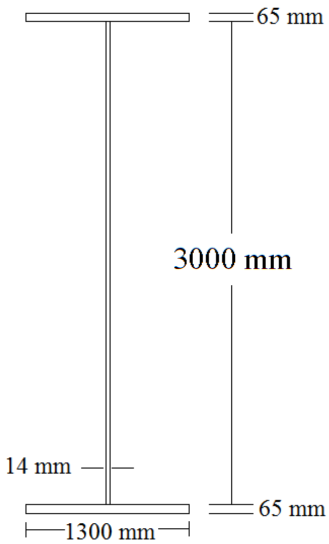


Figure 4.8 Floor beam cross-section

Longitudinal Stiffeners:

NOT OK $\frac{h_w}{t_w} = \frac{300}{1.4} = 214 > 170$ Therefore, longitudinal stiffeners are required. For stiffeners 200×12 mm steel plates have been provided. Stiffeners placed h/5 from bottom flange, which is 300/5 = 60 cm. Minimum moment of inertia of Stiffeners I_s can be obtain from equation below [2]:

$I_s = ht_w[2.4\left(\frac{a}{h}\right)^2 - 0.13]$

a=h (assumed)

$I_s = 300 \times 1.4\ [2.4\left(\frac{300}{300}\right)^2 - 0.13] = 817\ cm^4$ (Minimum)

$I_{s\ available} = 1.2 \times \frac{20^3}{3} = 3200 > 817$ O.K

Transverse Stiffeners:

The floorbeam transverse stiffeners must be designed in accordance with section 6.10.11.1 of AASHTO LRFD specification. The moment of inertia of a transverse stiffener is dependent on a factor, *J*, computed below.

For transverse Stiffeners 2 plates of 200 ×15 mm on both side of web are provided. (Figure 4.9)

$I_{min} = \frac{a\, t_w^2}{10.92} J$

$J = 25 \left(\frac{h}{a}\right)^2 - 20 \geq 5$

$J = 25 \left(\frac{300}{150}\right)^2 - 20 = 80$

$I_{min} = \frac{150 \times 1.4^2}{10.92} \times 80 = 3015\ cm^4$

$I_{Available} = 1.5 \times \frac{41.4^3}{12} = 8870 > 3015\ cm^4$ OK

$\frac{b}{t} = \frac{20}{1.5} = 13.33$ OK

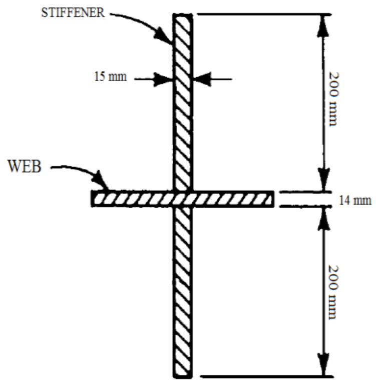


Figure 4.9: Transverse stiffeners

4.2.1.4 Design of Arch Ribs

The design of arch ribs and ties have to be done by combined axial compression/tension and flexure, governed in article 6.9.2.2 of the LRFD Specifications. For design of arch components such as arch ribs, ties and hangers 2-D model of the system has been provided and analyzed by SAP2000. The modeled structure is illustrated in Figure 4.10. The highlighted members are the ones which have subjected to the most critical moments and axial forces.

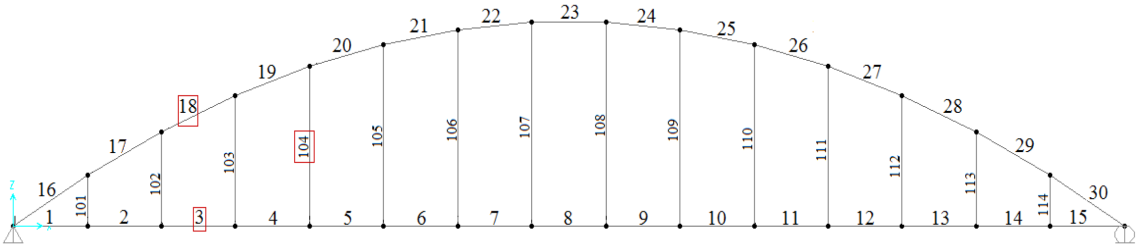


Figure 4.10: 2-D model of arch system and labels

Application of dead load on arch and ties:

Arch rib was designed for most critical axial force and moments obtained from dead and live loads. The dead load applied on arch system and analyzed by SAP2000. Application of dead load on arch system is shown in Figure 4.11. Calculation of dead loads on arch components can be found in Appendix B.

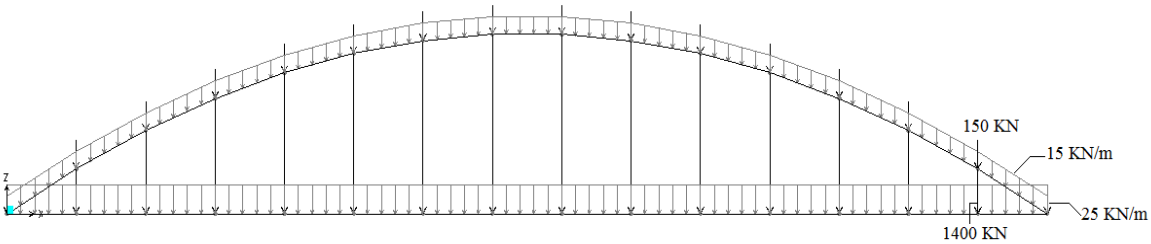


Figure 4.11: Application of dead load on arch system

Determination of Axial Force:

The arch system analyzed by SAP2000 for axial force is shown in Figure 4.12.

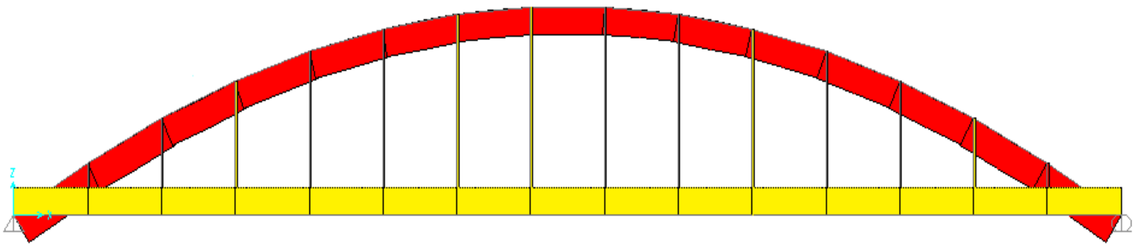


Figure 4.12: Axial force diagram due to dead load

Member 18 (Figure 4.10) has the most critical axial force due to dead load.

$$P_{max} = -26933 \text{ kN (C)}$$

Determination of Moment:

Moment diagram due to dead load is illustrated in Figure 4.13.

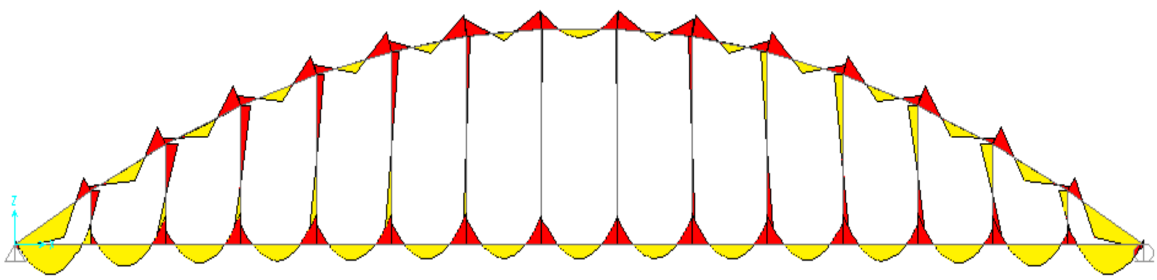


Figure 4.13: Moment diagram due to dead load

Member 18 (Figure 4.10) has the most critical Moment due to dead load.

$$M_{max} = 386 \text{ kNm}$$

Determination of Shear Force:

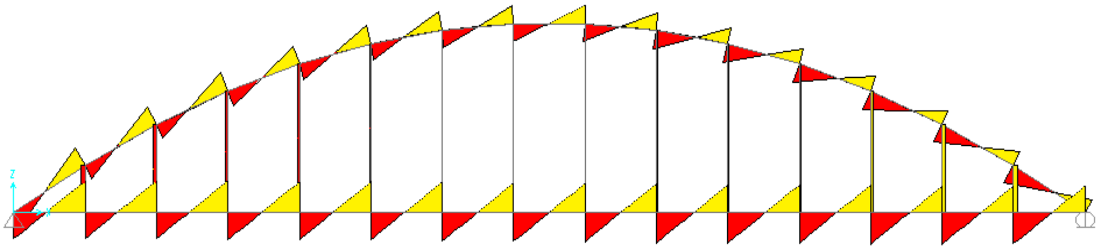


Figure 4.14: Shear force diagram due to dead load

Member 18 has the most critical shear force due to dead load.

$$V_{max} = 131 \text{ kN}$$

Determination of Live Load:

Live load on arch ribs and ties are considered as a moving load without an impact (*1M*=1) factor since span is 225 m and greater than 150 m [2].

Equivalent live load was calculated, considering that all 6 lanes are fully loaded:

$$q_L = 6 \times 9.3 \times \frac{1}{2} = 27.9 \text{ kN/m}$$

Determination of Influence lines:

Influence line for arch ribs due to moment, axial and shear force is provided in order to find the most critical live load. Influence line has been drawn by MIDAS/Civil software for bridge design and analysis.

All members influence line was controlled to find the most critical value. Member 18 found to be the most critical member for moving loads. In Figure 4.14 – 4.16 influence line due to moment, shear force and axial force are shown.

Node number 19, Member 18:

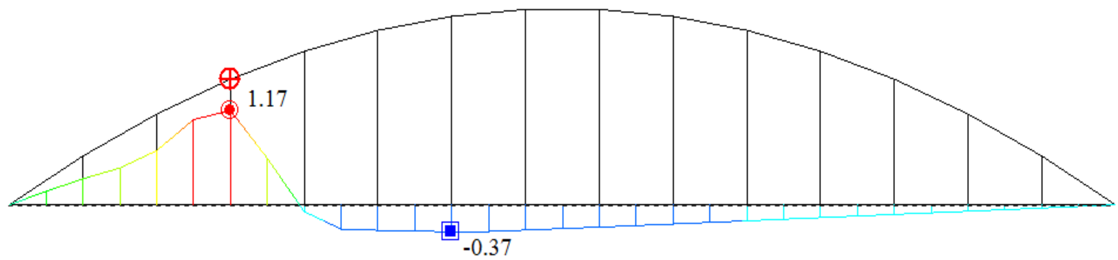


Figure 4.15: Influence line due to moment for arch rib

$$+ \text{Maximum} = \frac{1.17 \times 45}{2} + \frac{1.17 \times 13}{2} = 33.93 \text{ m}^2$$

$$-\text{Maximum} = \frac{0.37 \times 32}{2} + \frac{0.37 \times 135}{2} = 30.90 \text{ m}^2$$

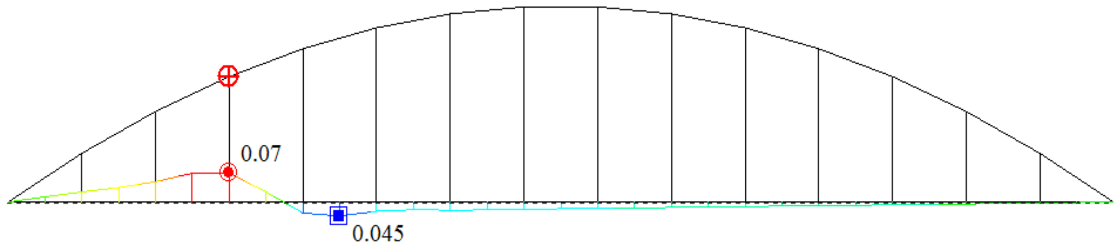


Figure 4.16: Influence line due to shear force for arch rib

+ Maximum = 1.84 m^2

−Maximum = 1.68 m^2

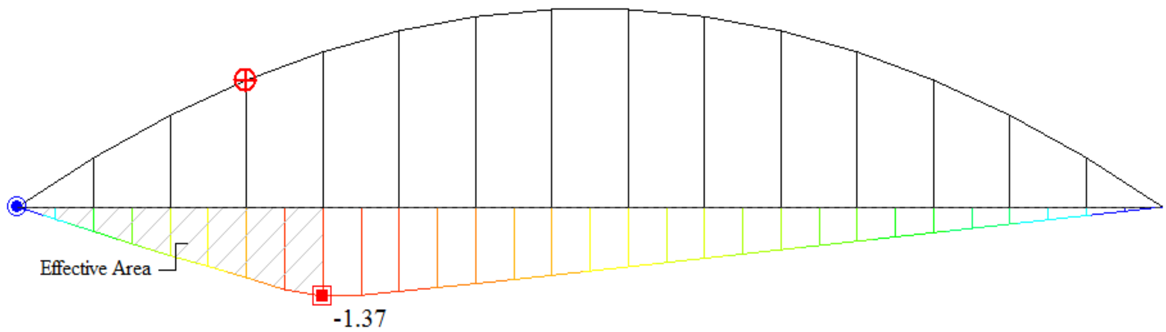


Figure 4.17: Influence line due to axial force

−Maximum = 154.13 m^2

Effective area: 41.10m^2

Design Live Load:

$M_{max} = 27.9\ kN/m \times 33.93\ m^2 = 946.65$ kNm

$P_{max} = = 27.9 \times 41.1 = 1146.7$ kN

Maximum moment and axial forces due to dead load and live load with load combination (Strength I) are given in Table 4.4. Shear force has been neglected because of its small value.

Table 4.4: Loads on the arch rib section

Load Type	Thrust	Maximum Bending Moment
Dead Load	- 26933 Kn	386 kNm
Live Load	- 1146.7 kN	1537.16 kNm
Factored *	- 35673 kN	3172.57 kNm

Design of Arch Ribs:

For arch rib design, assumption of preliminary cross section is required. The assumed cross section, is A514 steel, and shown in Figure 4.17.

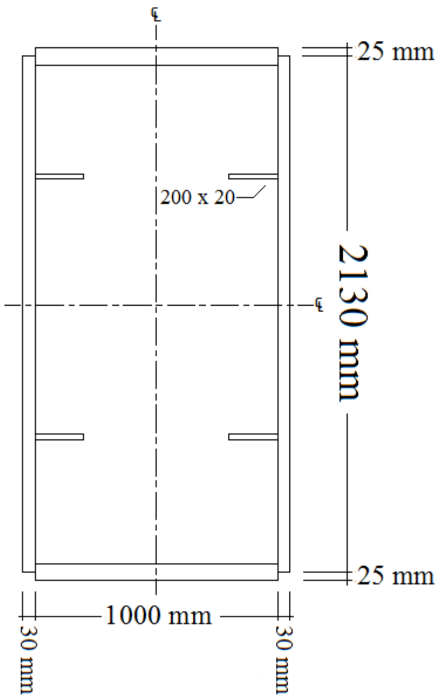


Figure 4.18: Arch rib cross-section

The section properties given in Table 4.5 are required for design and combined axial flexure check.

Table 4.5: Properties of arch rib

Section	Area (cm^2)	I_x (cm^4)	I_y (cm^4)	r_x (cm)	r_y (cm)	L/r_x	L/r_y	S_x (cm^3)
Arch Rib	2218	15×10^6	3.7×10^6	82.24	40.84	20.07	40.4	140852.2

The design of members for combined axial compression and flexure is governed by article 6.9.2.2 of the LRFD Specifications. The first step is to checking the adequacy of the rib section and determines the nominal compressive resistance of the rib section according to article 6.9.4.1.

$$\lambda = \left(\frac{Kl}{r_y\pi}\right)^2 \left(\frac{F_y}{E}\right) = \left(\frac{1\times40.4}{\pi}\right)^2 \left(\frac{689500}{2.2\times10^8}\right) = 0.52$$

Since $\lambda \leq 2.25$:

$$P_n = 0.66^{\lambda} F_y\, A_g = (0.66)^{0.52} \times (689500) \times (0.2218) = 123213.6 \text{ kN}$$

$$P_r = \varphi_c\, P_n = (0.90)(123213.6) = 110892.25 \text{ kN}$$

$$\frac{P_u}{P_r} = \frac{35673 \text{ kN}}{110892.25 \text{ kN}} = 0.32 > 0.2$$

Since $\frac{P_u}{P_r} > 0.2$, then the applicable axial-moment interaction equation applicable is:

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

Since $M_{uy} = 0$, then equation simplifies to:

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) \leq 1.0$$

$$M_{rx} = \varphi_f M_{nx} = (1.00)\, M_{nx} = M_{nx} \text{ (To be determined by Section 6.12.2.2.2)}$$

$$M_{nx} = F_y S_x \left\{1 - \left[\frac{0.064 F_y S_x l}{AE}\right] \left(\frac{\Sigma^b/_t}{I_y}\right)^{1/2}\right\}$$

$$M_{nx} = 689500 \times 0.1409 \left\{1 - \left[\frac{0.064 \times 689500 \times 0.1409 \times 16.5}{1.96 \times 2.2 \times 10^8}\right] \left(\frac{182}{0.037}\right)^{\frac{1}{2}}\right\}$$

$$M_{nx} = 96987.2 \text{ kNm} \Rightarrow M_{rx} = 96987.2 \text{ kNm}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) = \frac{35673 \text{ kN}}{110892.2 \text{ kN}} + \frac{8}{9}\left(\frac{3172.57 \text{ kN.m}}{96987.2 \text{ kN.m}}\right) \leq 1.0$$

$$0.322 + 0.029 = 0.351 \leq 1.0 \quad \text{OK}$$

Where:

S = section modulus about the flexural axis (m^3)

A = area enclosed within the centerlines of the plates comprising the box (m^2)

ℓ = unbraced length (m)

I_y = moment of inertia about an axis perpendicular to the axis of bending (m^4)

b = clear distance between plates (m)

t = thickness of plates (m)

Plate Buckling in Arch Rib Flanges:

Compression plates are checked to ensure that width–thickness ratios meet the requirements of LRFD Specifications (articles 6.14.4.3 and 6.14.4.2 for flanges and webs respectively) [9].

$$b/_t \leq 1.06 \left[E/_{(f_a + f_b)}\right]^{1/2}$$

The total stress due to axial load f_a and concurrent bending moment f_b :

$$f_b = \frac{P_u}{A_s} + \frac{M_{ux}}{S_x} = \frac{35673}{0.2218} + \frac{3172.57}{0.1409} = 183351.50 \text{ kN/m}^2$$

$$b/_t \leq 1.06 \left[\frac{2.2 \times 10^8}{183351.5}\right]^{1/2} = 36.72$$

$$\text{Flanges: } b/_t = \frac{1}{0.05} = 20 < 36.72 \quad \text{O.K}$$

Plate Buckling in Arch Rib Webs:

For webs, $D/_{\mathrm{t_w}} \leq \mathrm{k} \left(\mathrm{E}/_{\mathrm{f_a}}\right)^{0.5}$, for one longitudinal stiffener, k = 1.88.

$$D/_{\mathrm{t_w}} \leq 1.88 \left(2.2\times10^8/_{160834.1}\right)^{0.5} = 69.54$$

$$D/_{\mathrm{t_w}} = \frac{2.13}{0.03} = 71 > 69.54$$

The difference found for plate buckling of arch rib is so small that can be negligible. To be on the safe side web thickness can increase to 31 mm.

4.2.1.5 Design of Ties

In section 4.2.1.4 arch system was analyzed by SAP2000 and maximum moments, shear forces and axial forces were obtained (Figures 4.11 – 4.14).

Design procedure for the tie will be illustrated for member 3 which was found as the most critical member. The tie is subject to combined axial tension and bending. In this case, the axial stress is so high that no compression occurs on the section due to bending [9]. Maximum moments, shear forces and axial force due to dead load and live load are given in Table 4.6.

Determination of Live Load:

Equivalent live load when 6 lanes are fully loaded:

$$q_L = 6 \times 9.3 \times \frac{1}{2} = 27.9 \text{ kN/m}$$

Determination of Influence lines:

Influence lines for ties due to moment, axial and shear force was provided in order to find the most critical live load value. Influence line has obtained by MIDAS/Civil software.

Influence lines for all members were checked to find the most critical value. Member 3 found to be the most critical member for moving load. In Figure 4.18 – 4.20 Influence line due to moment, shear force and axial force are shown.

Node number 4, Member 3:

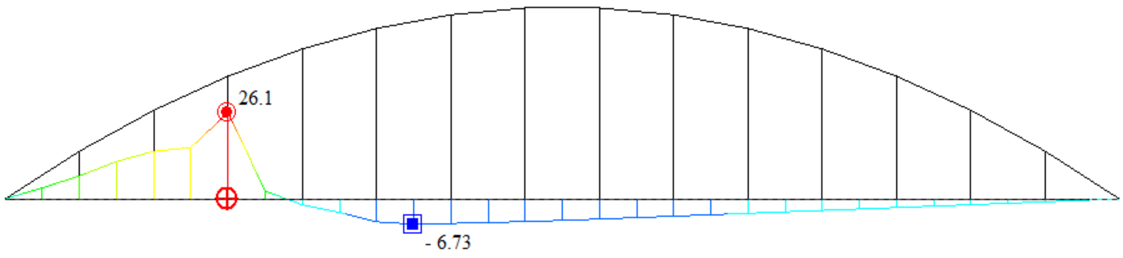


Figure 4.19: Influence line due to moment for ties

+ Maximum == 730.80 m^2

$-$Maximum = 555.23 m^2

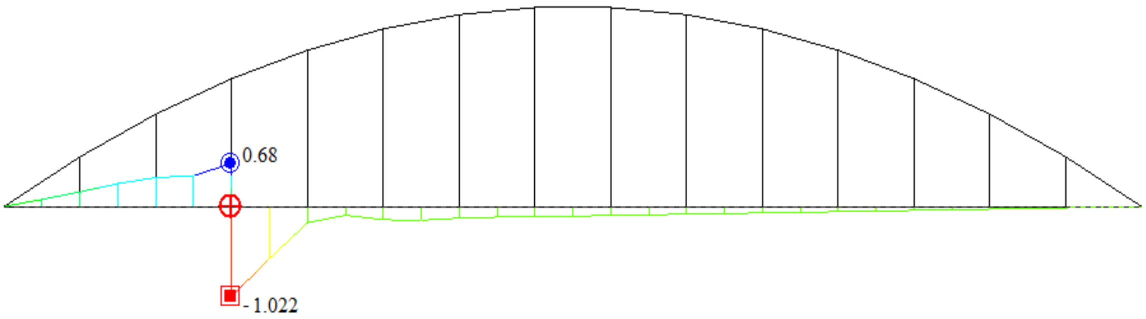


Figure 4.20: Influence line due to shear force for ties

+ Maximum =15.30 m^2

$-$Maximum = 91.98 m^2

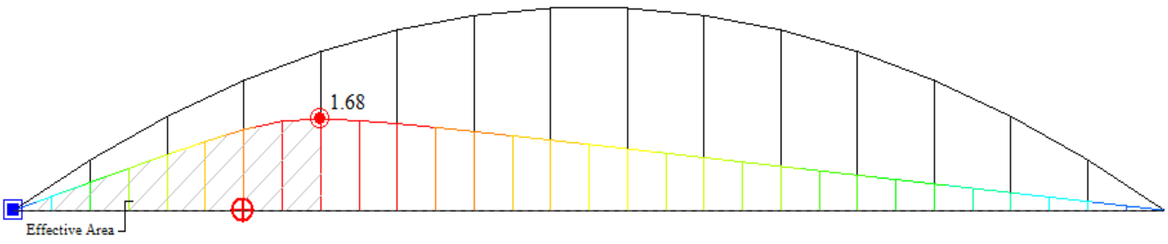


Figure 4.21: Influence line due to axial force for ties

+ Maximum = 189.0 m^2

Effective area: 50.4 m^2

Design Live Load:

M_{max} = 27.9 kN/m × 730.8 m^2 = 20389.32 kN.m

P_{max} = = 27.9 × 50.4= 1406.16 kN

Maximum moment and axial tension due to dead load and live load with load combination (Strength I) are given in Table 4.6. Shear force has been neglected because of its small value.

Table 4.6: Loads on the Tie Section

Load Type	Tensile	Maximum Bending Moment
Dead Load	24569.65 kN	538.35 kNm
Live Load	1406.16 kN	20389.32 kNm
Factored *	33172.85 kN	36354.25 kNm

Design of Ties:

The preliminary assumed cross section, of A588 steel, is shown in Figure 5.21.

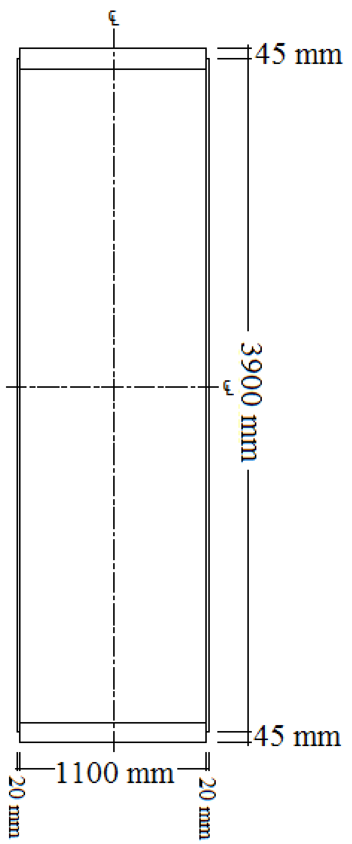


Figure 4.22: Tie cross-section

Properties of the tie section are given in Table 4.7.

Table 4.7: Properties of tie

Section	Area (cm^2)	I_x (cm^4)	I_y (cm^4)	r_x (cm)	r_y (cm)	L/r_x	L/r_y	S_x (cm^3)
Tie	3576	9.65×10^5	7×10^6	163.30	44.30	10.11	37.25	483592

The design of members for combined tension and flexure is governed by article 6.8.2.3 of the LRFD Specifications. The first step in checking the adequacy of the tie section is to determine the factored tensile resistance of the section, *Pr* from section 6.8.2.1.

$$P_r = \varphi_y\, P_{ny} = \varphi_y\, F_y\, A_g = (0.95) \times (340000)\times(0.3576) = 115504.8 \text{ kN}$$

$$P_r = \varphi_u\, P_{nu} = \varphi_u\, F_u\, A_n U = (0.8) \times (480000) \times(0.242)\times(1) = 92928 \text{ kN}$$

Where*:* A_n = product of the thickness of the element and its smallest net width (m^2)

A_g = gross cross-sectional area of the member (m^2)

The lesser value is used[9]:

$$\frac{P_u}{P_r} = \frac{33172.85 \text{ kN}}{92928 \text{ kN}} = 0.36 > 0.2$$

Therefore, the applicable axial-moment interaction equation is:

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

Since $M_{uy} = 0$, equation is simplifies to:

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) \leq 1.0$$

$$M_{rx} = \varphi_f M_{nx} = (1.00)\, M_{nx} = M_{nx} \text{ (to be determined by article 6.12.2.2.2)}$$

$$M_{nx} = F_y S_x \left\{1 - \left[\frac{0.064 F_y S_x l}{AE}\right] \left(\frac{\Sigma^{b}/_{t}}{I_y}\right)^{1/2}\right\}$$

$$M_{nx} = 340000 \times 0.4836 \left\{1 - \left[\frac{0.064 \times 340000 \times 0.4683 \times 15}{4.29 \times 2.2 \times 10^{8}}\right] \left(\frac{415}{0.07}\right)^{\frac{1}{2}}\right\}$$

$$M_{nx} = 162373.62 \text{ kNm} \Rightarrow M_{rx} = 162373.62 \text{ kNm}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) = \frac{33172.85 \text{ kN}}{92928 \text{ kN}} + \frac{8}{9}\left(\frac{36354.25 \text{ kNm}}{162373.62 \text{ kNm}}\right) \leq 1.0$$

$$0.36 + 0.2 = 0.56 \leq 1.0 \quad \text{OK}$$

Since the tie is a tension member, no stiffeners due to web and plate buckling are required [9].

4.2.1.6 Design of Hangers

There is no explicit procedure for the design of such hangers in the LRFD Specification [9]. From the computer analysis of the arch-tie system, the most highly stressed hanger found as Hanger 104 (Figure 4.10). This member carry a 1798.23 kN dead load and a 442.22 kN live load plus dynamic allowance of 33% and according to Table 3.4 a total of:

Factored load (Service II) = 1.00 (DW+DC) + 1.30 (LL+I) = 2373.2 kN

Factored load (Strength I) = 1.25 (DW+DC) + 1.75 (LL+I) = 3021.35 kN

Hangers designed for load combination Strength I since it cause the highest tensile stress. 4 hangers together could carry 3021.35 kN.

Design:

$$f_t = 0.33 \times f_u$$

$$f_t = 0.33 \times 400 \text{ MPa} = 132 \text{ MPa} = 132000 \text{ kPa}$$

Minimum are of steel:

$$\frac{P_u}{f_t} = \frac{3021.35}{132000} = 0.02289 \text{ m}^2 = 228.9 \text{ cm}^2$$

4 hangers were considered:

$$4\ \phi\ 80 \text{ mm} = 4 \times \frac{\pi \times 8^2}{4} = 201.06 \text{ cm}^2 < 228.9 \text{ cm}^2 \quad \text{Not Good}$$

$$4\ \phi\ 90 \text{ mm} = 4 \times \frac{\pi \times 9^2}{4} = 254.34 \text{ cm}^2 > 228.9 \text{ cm}^2 \quad \text{Good}$$

4.2.1.7 Bottom Lateral Bracing

The plan of the bracing used in the plane of the tie is shown in Appendix A. Bracings are subjected to both axial loads from lateral wind load on the structure and bending due to its own weight. Therefore, combined axial flexure h procedure has applied for design of bracings.

Wind Load on Structure:

Design wind velocity given in article 3.3.4.1 required in order to find the wind pressure on the structure.

$$V_{DZ} = 2.5\ V_0 \left(\frac{V_{30}}{V_B}\right) \ln\left(\frac{Z}{Z_0}\right)$$

$$V_{DZ} = 2.5 \times 8.20 \left(\frac{100}{100}\right) \ln\left(\frac{72}{0.23}\right) = 117.8 \text{ mph}$$

Structure assumed to be in open country. Form Table 3.3 =>$V_0 = 8.20$ and Z_0 =0.23

Design Wind Pressure:

$$P_D = P_B \frac{{V_{DZ}}^2}{240}$$

$$P_D = 0.05 \times \frac{117.8^{\ 2}}{240} = 2.89 \text{ kN/m}^2$$

$P_B = 0.05$, (Table 3.2)

Design Wind Load:

Area exposed: Tie height + Rails = 3.9 + 1 = 4.9 m

Wind load: 2.89 × 4.9 = 14.16 kN/m

Joint load at one span: $14.16 \times \frac{225}{2} = 1593.2$ kN

Wind load on each bracing: $\frac{1}{2} \times 1593.2 \times \frac{22.25}{16.5} = 1074.2$ kN

Factored load (Strength III from Table 3.4): 1.4 W_S = 1.4 × 1074.2 = P_u = 1503.9 kN

Moment Due to Dead Load:

Maximum dead load bending moment:

$$M_u = \frac{WL^2}{8} = \frac{2.4 \times (22.25)^2}{8} = 148.52 \text{ kN.m}$$

Factored load (Strength III from Table 3.4): 1.25 DL = 1.25 × 148.52 = 185.65 kN.m

Bottom Lateral Bracing Design:

The preliminary assumed cross section, of A536 steel, is shown in Figure 4.22.

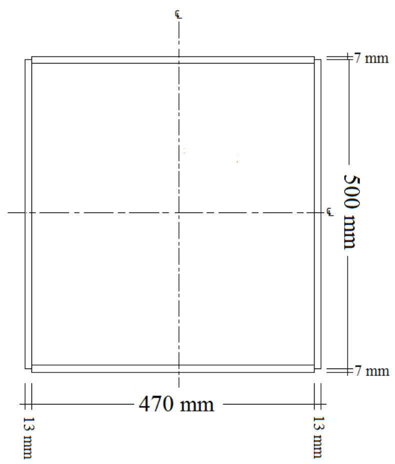


Figure 4.23: Cross-section of bottom bracing

Properties of the tie section are given in Table 4.8.

Table 4.8: Properties of Bottom Bracing

Section	Area (cm^2)	I_x (cm^4)	I_y (cm^4)	r_x (cm)	r_y (cm)	L/r_x	L/r_y	S_x (cm^3)
Bottom Bracing	255	$1.\times10^5$	88318	19.9	18.7	111.8	119	4020.5

The design of members for combined axial compression and flexure is governed by article 6.9.2.2 of the LRFD Specifications. The first step is to checking the adequacy of the rib section and determine the nominal compressive resistance of the section according to article 6.9.4.1. K= 0.75 for truss member.

$$\lambda = \left(\frac{Kl}{r_y \pi}\right)^2 \left(\frac{F_y}{E}\right) = \left(\frac{89.25}{\pi}\right)^2 \left(\frac{250000}{2.2\times10^8}\right) = 0.92$$

Since $\lambda \leq 2.25$:

$$P_n = 0.66^{\lambda} F_y \, A_g = (0.66)^{0.92} \times (250000) \times (0.0254) = 4332.66 \text{ kN}$$

$$P_r = \varphi_c \, P_n = (0.90)(4332.66) = 3889.4 \text{ kN}$$

$$\frac{P_u}{P_r} = \frac{1503.9 \text{ kN}}{3889.4 \text{ kN}} = 0.387 > 0.2$$

Since $\frac{P_u}{P_r} > 0.2$, then the applicable axial-moment interaction equation applicable is:

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

Since $M_{uy} = 0$, then equation simplifies to: $\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) \leq 1.0$

$M_{rx} = \varphi_f \, M_{nx} = (1.00)\, M_{nx} = M_{nx}$ (to be determined by Section 6.12.2.2.2)

$$M_{nx} = F_y\, S_x \left\{1 - \left[\frac{0.064 F_y S_x l}{AE}\right] \left(\frac{\Sigma b/t}{I_y}\right)^{1/2}\right\}$$

$$M_{nx} = 25\times10^{4}\times4.021\times10^{-3} \left\{1 - \left[\frac{0.064\times250000\times0.004021\times22.25}{0.229\times2.2\times10^{8}}\right] \left(\frac{144.07}{8.832\times10^{-4}}\right)^{\frac{1}{2}}\right\}$$

$$M_{nx} = 993.72 \text{ kN.m} \Rightarrow M_{rx} = 993.72 \text{ kN.m}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}}\right) = \frac{1503.9 \text{ kN}}{4964.4 \text{ kN}} + \frac{8}{9}\left(\frac{148.52 \text{ kN.m}}{993.72 \text{ kN.m}}\right) \le 1.0$$

$$0.387 + 0.133 = 0.52 \le 1.0 \quad \text{O.K}$$

Plate Buckling in Lateral Bracing:

Compression plates have been checked to ensure that width–thickness ratios, b/t, meet the requirements of LRFD Specifications (article 6.9.4.2). The requirement is as follows:

$$b/t \le \text{k} \left(\frac{E}{f_y}\right)^{0.5}$$

Where k is the plate-buckling coefficient equal to 1.40 (LRFD Table 6.9.4.2-1). However, for members designed using the equations of article 6.9.2.2, F_y may be replaced with the maximum calculated compressive stress due to the factored axial load and concurrent bending moment.

$$\frac{P_u}{A_s} + \frac{M_{ux}}{S_x} = \frac{1503.9}{0.0225} + \frac{148.52}{0.004021} = 103776.1 \text{ kN/m}^2$$

$$b/t \le 1.40\left(\frac{2.2\times10^{8}}{103776.1}\right)^{0.5} = 64.46$$

Flange: $b/t = 33.57 < 64.46$ OK Web: $b/t = 38.46 < 64.46$ OK

4.2.1.8 Design of Rib Bracings

The plan of the A36 steel bracing used for the arch rib is shown in Appendix A. Rib bracing is designed to carry its own weight, wind on ribs and rib bracing and buckling shear from an assumed compression of the ribs. Therefore, combined axial flexure procedure was applied for the design of the bracings.

Wind Load on Structure:

Design wind velocity given in Section 3.3.4.1 required in order to find the wind pressure on the structure.

$$V_{DZ} = 2.5\ V_0 \left(\frac{V_{30}}{V_B}\right) \ln\left(\frac{Z}{Z_0}\right)$$

$$V_{DZ} = 2.5 \times 8.20 \left(\frac{100}{100}\right) \ln\left(\frac{188.65}{0.23}\right) = 137.55 \text{ mph}$$

Structure assumed to be in open country. Form Table 3.3 =>$V_0 = 8.20$ and Z_0 =0.23

Design Wind Pressure:

$$P_D = P_B \frac{{V_{DZ}}^2}{240}$$

$$P_D = 0.05 \times \frac{137.55^{\ 2}}{240} = 3.94 \text{ kN/m}^2$$

$P_B = 0.05$, (Table 3.2)

Design Wind Load:

Area exposed: Height of the arch ribs = 2.130 m

Wind load: 3.94 × 2.130= 8.4 kN/m

Arch Length: where a = 37.5m and b = 225 m

$$\frac{1}{2}\sqrt{b^2+16a^2}+\frac{b^2}{8a}\ \text{Ln}\left(\frac{4a+\sqrt{b^2+16a^2}}{b}\right) = 240.7 \text{ m}$$

Joint load at one span: 8.4 × $\frac{240.7}{2}$ = 1011 kN

Wind load on each bracing: $\frac{1}{2} \times 1011 \times \frac{36.25}{15} = 1145.28$ kN

Factored load (Strength III, Table 3.4): 1.4 W_S = 1.4 × 1145.28 = P_u = 1603.4 kN

Moment Due to Dead Load:

Maximum dead load bending moment:

$$M_u = \frac{WL^2}{8} = \frac{5.5 \times (36.25)^2}{8} = 918.2 \text{ kNm}$$

In Table 4.9 loads on the first panel brace are given and design was corsedant using these loads. Load combination Strength III been used since it gives the highest stresses.

$$\text{Strength III} = 1.25\text{ DL} + 1.4\, W_s$$

Table 4.9: Loads on brace between the arches

Load Type	Axial	M_x	M_y
Dead Load	-	918.2 kNm	98.53 kNm
Wind Load	1145.28 kN	-	90.85 kNm
Factored	P_u = 1603.4 kN	M_{ux} = 1147.75 kNm	M_{uy} = 250.36 kNm

Rib Bracing Design:

The preliminary assumed cross section of A536 steel is shown in Figure 4.23.

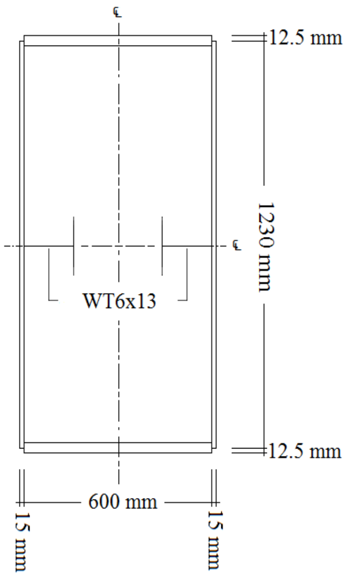


Figure 4.24: Rib bracing cross-section

Properties of the rib bracing are given in Table 4.10.

Table 4.10: Properties of Rib Bracing

Section	Area (cm^2)	I_x (cm^4)	I_y (cm^4)	r_x (cm)	r_y (cm)	L/r_x	L/r_y	S_x (cm^3)	S_y (cm^3)
Rib Bracing	654	$15.\times10^5$	$4.\times10^5$	48	25	37.8	72.5	24390	13098

The design of members for combined axial compression and flexure is governed by article 6.9.2.2 of the LRFD Specifications. The first step is to checking the adequacy of the rib section and determines the nominal compressive resistance of the section according to Section 6.9.4.1. K= 0.75 for truss member and unsupported length of the brace is L/2 = 18.125 m.

$$\lambda = \left(\frac{Kl}{r_y \pi}\right)^2 \left(\frac{F_y}{E}\right) = \left(\frac{54.375}{\pi}\right)^2 \left(\frac{250000}{2.2\times10^8}\right) = 0.34$$

Since $\lambda \leq 2.25$:

$$P_n = 0.66^{\lambda} F_y A_g = (0.66)^{0.34} \times (250000) \times (0.0654) = 14195.9 \text{ kN}$$

$$P_r = \varphi_c P_n = (0.90)(14195.9) = 12776.3 \text{ kN}$$

$$\frac{P_u}{P_r} = \frac{1603.4 \text{ kN}}{12776.3 \text{ kN}} = 0.125 < 0.2$$

Since $\frac{P_u}{P_r} < 0.2$, then the applicable axial-moment interaction equation is:

$$\frac{P_u}{(2.0\,P_r)} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

$$M_{rx} = \varphi_f M_{nx} = (1.00)\, M_{nx} = M_{nx}$$

$$M_{ry} = \varphi_f M_{ny} = (1.00)\, M_{ny} = M_{ny} \text{ (to be determined by article 6.12.2.2.2)}$$

$$M_{nx} = F_y\, S_x \left\{1 - \left[\frac{0.064 F_y S_x l}{AE}\right] \left(\frac{\Sigma b/t}{I_y}\right)^{1/2}\right\}$$

$$M_{nx} = 4\times10^{5}\times0.0244 \left\{1 - \left[\frac{0.064\times400000\times0.0244\times18.13}{0.723\times2.2\times10^{8}}\right] \left(\frac{212}{4\times10^{-3}}\right)^{\frac{1}{2}}\right\}$$

$$M_{nx} = 9600.1 \text{ kNm} \Rightarrow M_{rx} = 9600.1 \text{ kNm}$$

$$M_{ny} = F_y\, S_y \left\{1 - \left[\frac{0.064 F_y S_y l}{AE}\right] \left(\frac{\Sigma b/t}{I_x}\right)^{1/2}\right\}$$

$$M_{nx} = 4\times10^{5}\times0.0131 \left\{1 - \left[\frac{0.064\times400000\times0.0131\times18.13}{0.723\times2.2\times10^{8}}\right] \left(\frac{212}{0.015}\right)^{\frac{1}{2}}\right\}$$

$$M_{nx} = 5216.2 \text{ kNm} \Rightarrow M_{rx} = 5216.2 \text{ kNm}$$

$$\frac{P_u}{(2.0\, P_r)} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) = 0.125 + 0.12 + 0.048 = 0.3 \leq 1.0 \quad \text{OK}$$

<u>Plate Buckling in Lateral Bracing:</u>

Compression plates have checked to ensure that width–thickness ratios, *b*/*t*, meet the requirements of LRFD Specifications (article 6.9.4.2). The requirement is as follow:

$$b/t \leq \text{k} \left(\frac{E}{f_y}\right)^{0.5}$$

Where *k* is the plate-buckling coefficient equal to 1.40 (LRFD Table 6.9.4.2-1). However, for members designed using the equations of Section 6.9.2.2, F_y may be replaced with the maximum calculated compressive stress due to the factored axial load and concurrent bending moment.

$$\frac{P_u}{A_s} + \frac{M_{ux}}{S_x} = \frac{1603.4}{0.0654} + \frac{1145.75}{0.0244} = 71473.8 \text{ KN/m}^2$$

$$b/_t \leq 1.40\left(\frac{2.2\times10^{8}}{71473.8}\right)^{0.5} = 77.68$$

Flange: $b/_t$ = 24 < 77.68 OK Web: $b/_t$ = 82 > 77.68 Not OK

Longitudinal stiffeners were attached at the middle of webs (Figure 4.23). It has been assumed that a node will occur at the location of these stiffeners. Thus, the *b*/*t* ratio will be rechecked based on a clear distance between the stiffener and flange.

$$\text{Web: } b/_t = \frac{615}{15} = 41 < 77.68 \quad \text{Good}$$

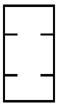
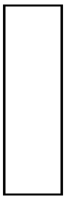
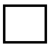
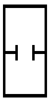
4.2.2 Medium Span Bridge

A 126 m span bridge designed as a medium span Tied-Arch Bridge. The plans of this bridge is provided in Appendix B. Same factors and specification specified in Chapter 2 for the geometry design of this bridge (medium span) has been considered.

Design procedures governed by AASHTO LRFD Specifications which is applied for long span tied-arch bridge components; is also applied for the medium span bridge. Members dealing with both axial load and flexure, such as arch rib, were designed by procedure governed by Article 6.9.2.2 of the LRFD Specifications. Those with combined tension and flexure, such as ties, were designed by article 6.8.2.3 of the LRFD Specifications.

The tied-arch bridge designed and their sections and properties arehave listed in Table 4.11 for final analysis and assessment.

Table 4.11: Medium span tied-arch bridge section properties

Section		Area (cm^2)	I_x (cm^4)	I_y (cm^4)	r_x (cm)	r_y (cm)	Weight per Meter (kg/m)
Floor Beam		404	12.7×10^5	4.2×10^4	56	10.2	311.1
Stringer		130	8.8×10^4	6.8×10^3	26	7.3	101
Arch Rib		523.32	6.1×10^5	4.5×10^5	34.2	29.3	403.5
Tie		1132	7.2×10^6	8×10^5	80	26.6	873
Deck Bracing		138.3	2×10^4	2×10^4	11.8	11.8	107
Rib Bracing		234	1.4×10^5	6×10^4	23.9	16	180

The maximum tension at hangers due to load combination, Strength I, is equal 432.95 kN. Minimum area of hangers should be 32.8 cm^2. Therefore, four wire ropes with diameter of 50 mm have been provided (4 ϕ 50 mm).

4.3 Truss Bridge Design

Span length same as those of tied-arch bridge have been considered in order to obtain accurate comparison between bridges with medium and large spans.

Design of the bridge components were done by hand calculation. Same procedures were used for slab, stringers and floor beams. 2-D models have been provided by SAP2000 and MIDAS/Civil for final analysis. In the following sections design of the truss bridge has been discussed in detail.

4.3.1 Long Span Truss Bridge

4.3.1.1 Design of Concrete Deck Slab

Slab which is same cross section as that of the roadway slab has provided for truss bridge. Since the design of deck slabs using the traditional method are still permitted by the LRFD Specifications [9], concrete slab was designed using this method.

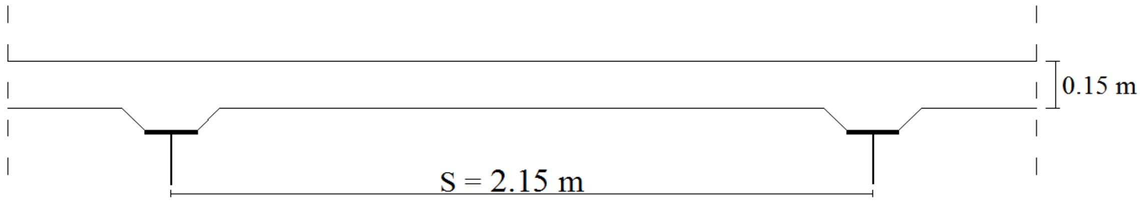


Figure 4.25: Concrete Slab cross section

Moment due to dead load:

$$M_d = \pm\frac{qL^2}{10} = \pm\frac{4.7\times2.15^2}{10} = 2.2 \text{ kNm}$$

Moment due to live load:

$$M_l = \pm 0.8 \times \frac{1.64 \times s+1}{16} \times P = \pm 0.8 \times \frac{(1.64 \times 2.15)+1}{16} \times 80 = \pm 18.1 \text{ kNm}$$

Dynamic allowance (Impact factor):

From Table 4.1 => IM = 33% , $M_l = 18.1 \times 1.33 = 24.073$ kNm

Total Moment:

$M_T = M_d + M_l = 26.3$ kNm

Required area of steel:

$$f_s = 2000\ \frac{kg}{cm^2}, \qquad f_c = 0.4\ f_c' = 0.4(250) = 100\ \frac{kg}{cm^2}, \qquad \frac{f_s}{f_c} = 20$$

$$n = 9, \qquad k = \frac{9}{9+2} = 0.325, \qquad J = 1 - \frac{k}{3} = 0.875$$

$$A_s = \frac{M}{f_s J d} = \frac{25.7 \times 10^4}{2000 \times 0.875 \times 11} = 13.35\ cm^2$$

Table 4.12: Distribution of slab reinforcement

Elements	Minimum Reinforcement	Minimum Area	Corresponding Area
Longitudinal Bars	Ø20/20cm	13.35 cm^2	15.70 cm^2
Distributed Bars	Ø16/20cm	8.94 cm^2	10.05 cm^2

4.3.1.2 Design of Stringers

Stringers are placed on bridge width at every 2.15 m and Floor beams have placed at bridge length every 12.5 m to support stringers.

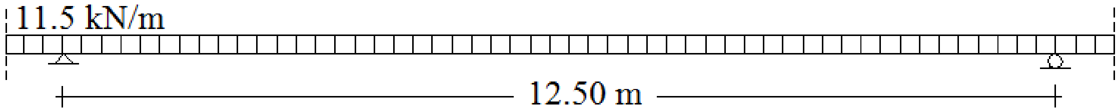


Figure 4.26: Dead load application on stringers

Live load, HS20-44 Truck loading, which is suggested by the specification is used in this study. The application of this loading is shown in Figure 4.3

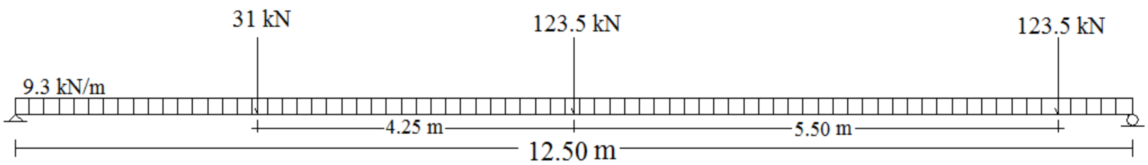


Figure 4.27: Application of design vehicle on stringers

Maximum moments and shear forces due to dead load, live load and dynamic allowance have been obtained by using SAP2000 and they are listed in Table 4.2.

Table 4.13: Design moments and reactions for stringer

Load Type	Maximum Bending Moment	Maximum Shear Force
Dead Load (DC+DW)	236.33 kNm	72 kN
LL + I	626.96 kNm	233.11 kN
Factored *	1392.6 kNkm	497.8 kN

*Strength I = 1.25 DL + 1.25 DW + 1.75 (LL+I)

Design:

Web: $\frac{1}{20} \times l = \frac{1}{20} \times 1250 \text{ cm} = 62.5 \text{ cm} = 70 \text{ cm}$ Thickness = t = 15 mm

Check: $\frac{h}{t} < 170 = \frac{70}{1.5} = 46.7 < 170$ OK

$F_b = 0.55 f_y = 0.55 \times 270000 = 148500 \text{ kPa}$

$$A_f = \frac{M}{f_s d} - \frac{A_w}{6} = \frac{1392.6}{148500 \times 0.68} - \frac{0.7 \times 0.015}{6} = 0.0121 \text{ m}^2 = 121 \text{ cm}^2$$

Flange design:

$A_f = 121 \text{ cm}^2$ Selected: 525 × 25 mm

$$f_b = \frac{1392.6}{0.00953} = 146129 < F_b \quad \text{OK}$$

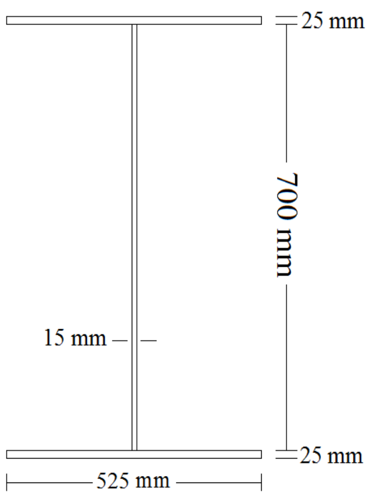


Figure 4.28: Stringers cross-section

4.3.1.3 Design of Floor Beams

<u>Determination of dead load:</u>

Floor beams placed every 12.5 m on length of the bridge. Plan bracing placed at the center of every floor beam deck (Appendix A). In Figure 4.5 application of dead load on floor beams are shown. The additional load from deck bracing has been added.

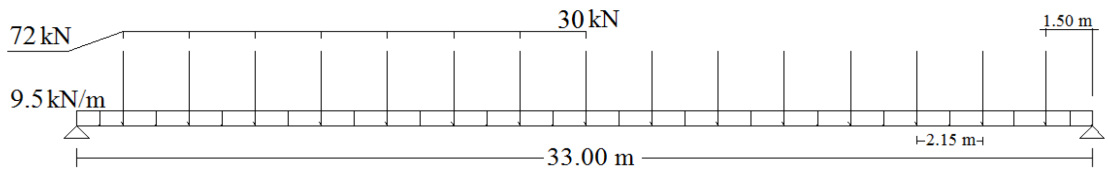


Figure 4.29: Reaction from stringers and bracing on floor beam

<u>Determination of support reaction:</u>

In order to find the maximum forces from vehicles on floor beams, all possible truck location on a series of stringers (2 stringers) were checked. The most critical location that causes the extreme force has shown in Figure 4.6.

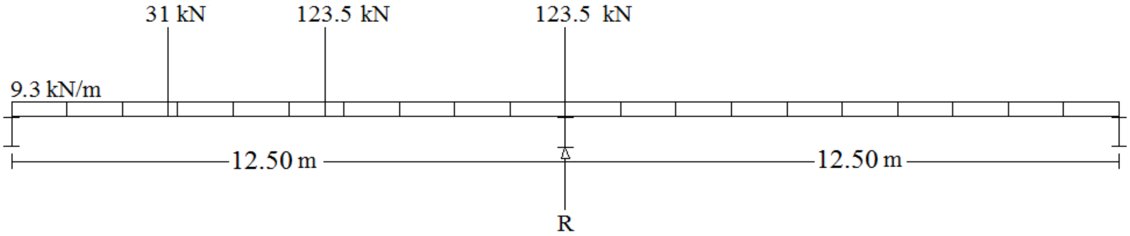


Figure 4.30: Application of truck loading on stringers

R was obtained from SAP2000 Software: R = 366.14 kN

Determination of maximum moment at mid span:

In every cross-section of the deck three vehicles will be placed next to each other, there will be 6 lanes on the deck. Live load for sidewalk shall apply since its width is greater than 0.7m [18].

In Figure 4.7 equivalent wheel load reaction arrangement on floor beams are shown.

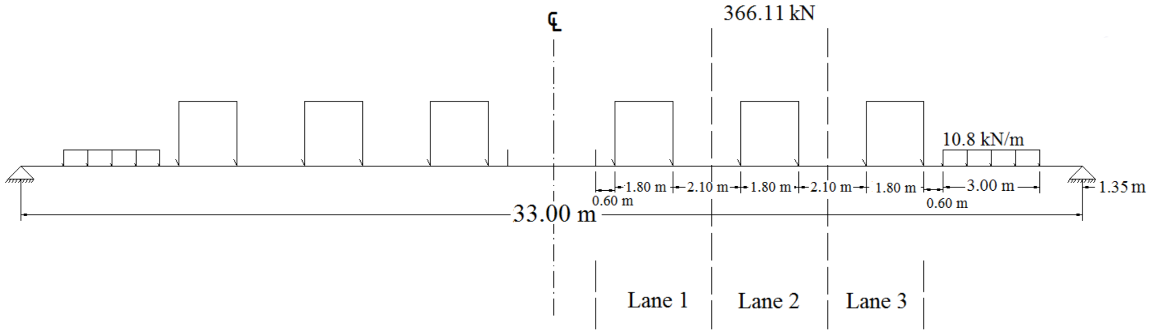


Figure 4.31: Equivalent wheel load reactions

Maximum Moment and shear force have been obtained by using SAP2000 and as AASHTO LRFD specified in article 3.6.1.1.2 multiple presence factors were used when investigating the effect of three or more lanes being loaded [18].

M_{max} = 22020.23 kN $\quad$ V_{max} = 2501.1 kN

Since the number of loaded lanes is greater than three, the presence factor were taken as 0.65 [18].

Maximum moment and shear force and load combination (Strength I) are given in table below (Table.4.14).

Table 4.14: Design moments and reactions for floor beam

Load Type	Maximum Bending Moment	Maximum Shear Force
Dead Load Stringer + Floor beam	6039.4 kNm	681.8 kN
LL + I	14313.2 kNm	1625.7 kN
Factored *	32597.3 kNm	3697. kN

Design:

H_w = 3.00 m $\quad$ t_w = 1.4 cm

$$A_f = \frac{M}{f_s d} - \frac{A_w}{6} = \frac{32597.3}{148500\times3.1} - \frac{3\times0.014}{6} = 0.0638\ m^2 = 638\ cm^2$$

Flange Design:

$A_f = 638\ cm^2$ Selected: 120×6 cm

$f_b = \frac{32597.3}{0.2261} = 144172$ kPa $< F_b$ O.K

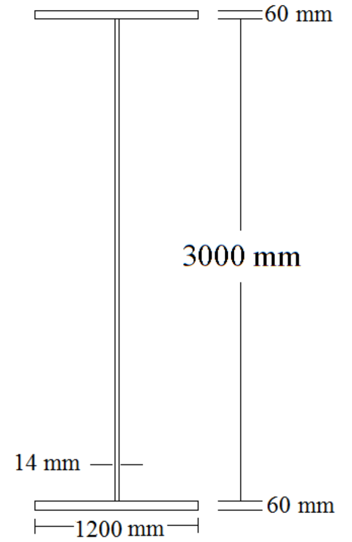


Figure 4.32: Floor beam cross-section

Longitudinal Stiffeners:

$\frac{h_w}{t_w} = \frac{300}{1.4} = 214 > 170$ NOT O.K Therefore longitudinal stiffeners are required. For stiffeners 200×12 mm steel plates have been provided. Stiffeners placed h/5 from bottom flange, which is 300/5 = 60 cm. Minimum moment of inertia of Stiffeners I_s can be obtained from equation below [2]:

$I_s = ht_w[2.4\left(\frac{a}{h}\right)^2 - 0.13]$

a=h assumed

$I_s = 300 \times 1.4\ [2.4\left(\frac{300}{300}\right)^2 - 0.13] = 817\ cm^4$ (minimum)

$I_{s\ Available} = 1.2 \times \frac{20^3}{3} = 3200 > 817$ OK

Transverse Stiffeners:

The floor beam transverse stiffeners must be designed in accordance with Section 6.10.11.1 of AASHTO LRFD specification. The moment of inertia of a transverse stiffener is dependent on a factor, *J*, computed below.

For Transverse Stiffeners 2 plates of 200 ×15 mm on both sides of the web are provided. (Figure 4.9)

$$I_{min} = \frac{a\, t_w^2}{10.92} J$$

$$J = 25\left(\frac{h}{a}\right)^2 - 20 \geq 5$$

$$J = 25\left(\frac{300}{150}\right)^2 - 20 = 80$$

$$I_{min} = \frac{150 \times 1.4^2}{10.92} \times 80 = 3015\ cm^4$$

$$I_{available} = 1.5 \times \frac{41.4^3}{12} = 8870 > 3015\ cm^4 \quad OK$$

$$\frac{b}{t} = \frac{20}{1.5} = 13.33 \quad O.K$$

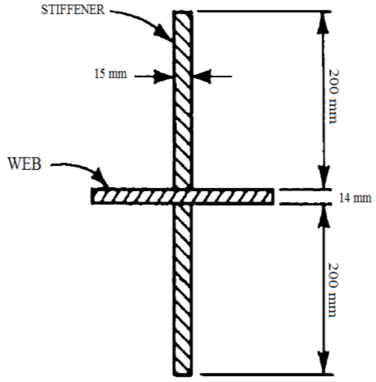


Figure 4.33: Transverse stiffeners

4.3.1.4 Truss Design

Warren truss has been modeled in 2-D by using SAP2000 Version 14 to obtain internal forces such as tension and compression. Article 6.6.8 and 6.6.9 of AASHTO LRFD Specifications have been studied for truss member design. The modeled structure is illustrated in Figure 4.33.

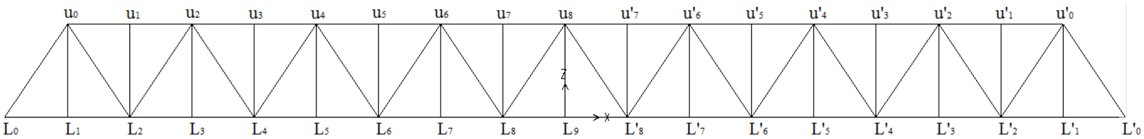


Figure 4.34: 2-D model of truss and labels

Truss Dead Load:

In order to calculate internal forces of the truss, truss self-weight was estimated. The dead load of a truss bridge consists of the weight of the floor system, truss, and bracing. Floor systems weight gets transferred to truss chord.

When using hand calculation, the weight of the truss can be estimated by increasing the other dead loads by some percentage or by using some approximate formula [9]. Charles W. Hudson studied trusses and indicates the following formula for truss weight calculation:

$$W = \frac{39.25\ S\ L}{1000\ s}$$

Where:

W: is the total weight of the bridge truss including its bracing (kN)

S: is the maximum total tensile stress in the most stressed chord member (kN)

L: is the length of the truss (m)

s: is the allowable tensile stress (kN/cm^2)

Member $L_8 L'_8$ found as the one that subject to the highest tensile stress from the live load analysis and influence lines (Figure 4.37). Reaction of floor beam on truss calculation is given in section 4.2.1.3, Figure 4.28, which is 681.76 kN. In order to find loads on truss joint (Panel loads) and to indicate the self-weight of the truss, the above mentioned load has been increased by 25 percent.

Tensile force in Member $L_8 L'_8$ found as follows:

$$L_8 L'_8 = \frac{(3 \times 681.76 \times 25) - (852.2 \times 12.5)}{12.5} = 3238.4 \text{ kN}$$

Live load plus dynamic allowance should be added to find total load:

$$\text{DL+LL+IM} = 3238.4 + 1625.72 = 4864.12 \text{ kN}$$

Total weight of the truss calculated as:

$$W = \frac{39.25 \times 4864.12 \times 225}{1000 \times 18.975} = 2263.84 \text{ kN}$$

Total weight divided by 18 joints to find panel loads at each joint:

$$2263.84/18 = 125.78 \text{ kN}$$

Total dead load contains floor system such as stringer and floor beams, therefore:

$$\text{Total dead load of structure: } 125.8 + 681.76 = 807.56 \text{ kN}$$

The dead load obtained has considered as panel load at each joint of bottom chord for structural analysis. The application of truss dead load is shown in Figure 4.34. During the final analysis the exact weight of the structure, sway frames and bracing were considered.

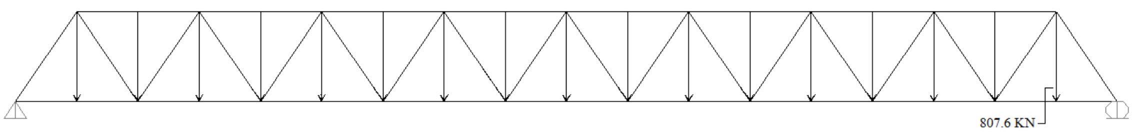


Figure 4.35: Application of dead load on truss bridge

Determination of Truss Internal Forces:

Dead Load:

Truss was analyzed by using SAP2000 which delivered the tensile and compressive forces due to dead load as shown in Figure 4.35 - 4.36.

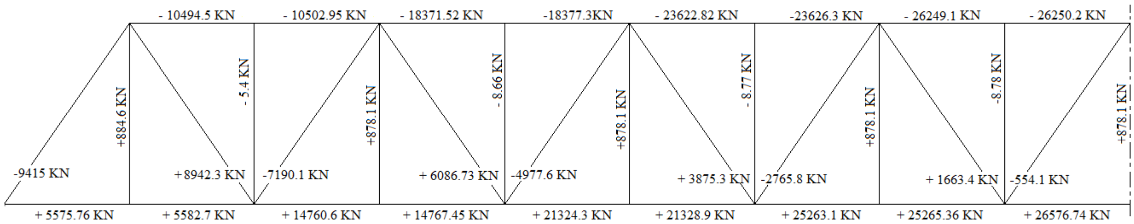


Figure 4.36: Truss internal forces

Live Load:

Equivalent live load considering that 6 lanes are fully loaded:

$$q_L = 6 \times 9.3 \times \frac{1}{2} = 27.9 \text{ kN/m}$$

Dynamic allowance (Impact factor)

IM = 33 %   form table 3.1.

$$1 + \frac{IM}{100} = 1 + \frac{33}{100} = 1.33$$

Total equivalent live load: $27.9 \times 1.33 = 37.1$ kN/m

Influence Line:

In order to obtain the internal forces of truss members due to live load, Influence line was obtained by using MIDAS/Civil software.

The influence lines of all members were controlled to find the most critical value. The area of influence line covered by the load has been written on each diagram. In Figure 4.36 Influence line of truss member has shown.

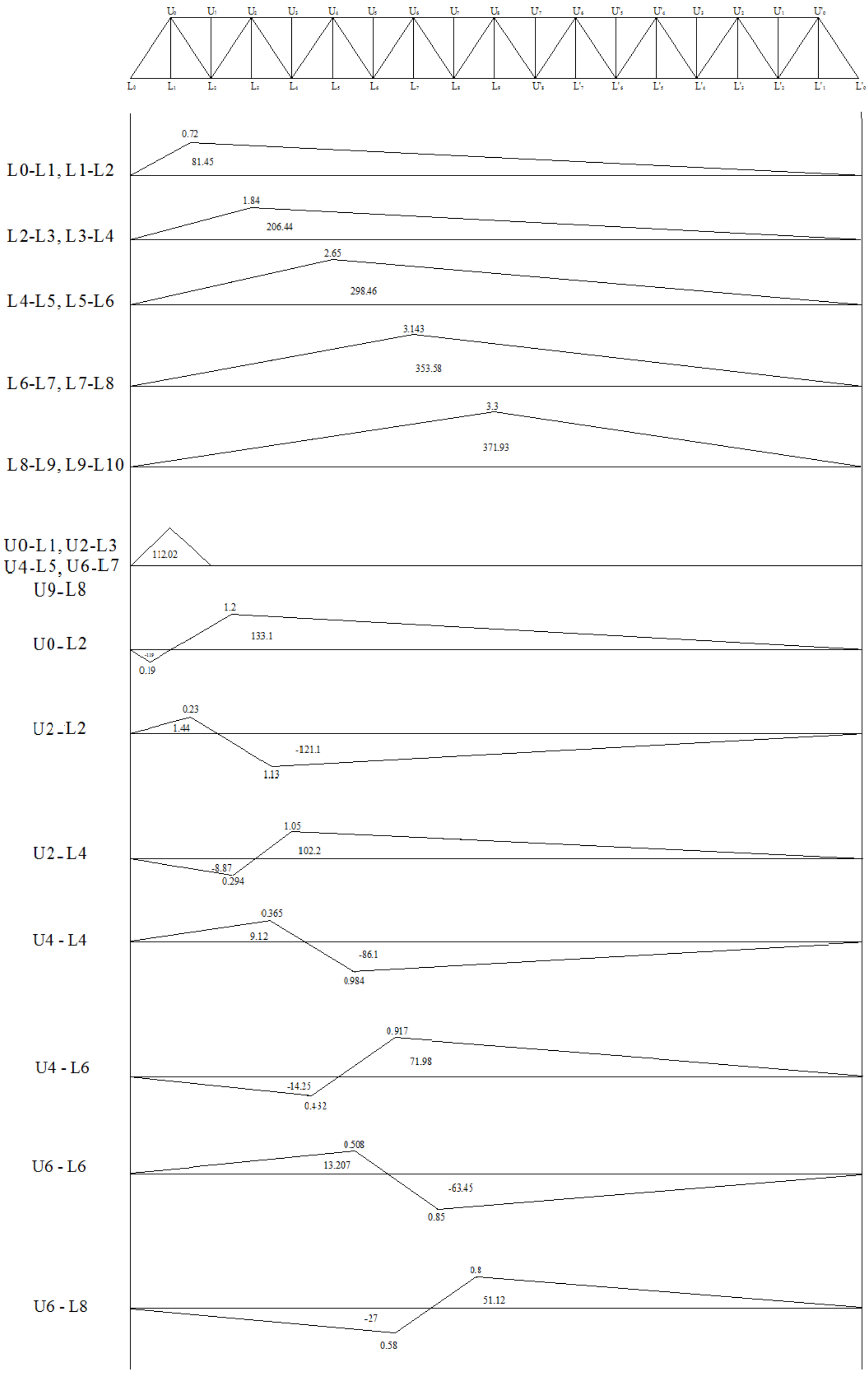


Figure 4.37: Influence line for truss members

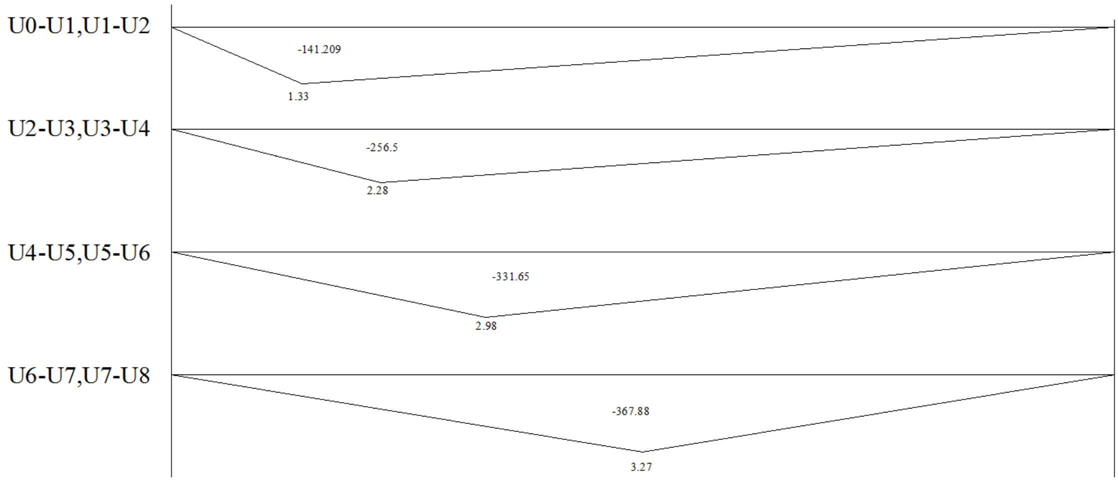


Figure 4.38: Influence lines for top chord

Determination of Internal Forces Due to Equivalent Live Load:

The total equivalent live load has been computed by intensity of the load multiplied by the area of influence line diagram covered by load.

Bottom chords:

Member L_0 - L_1, L_1 - L_2 F (LL + I) :

$$81.45 \times 37.1 = 3111.4 \text{ kN}$$

Member $L_2 - L_3$, $L_3 - L_4$ F (LL + I):

$$206.44 \times 37.1 = 7886.1 \text{ kN}$$

Member $L_4 - L_5$, $L_5 - L_6$ F (LL + I) :

$$298.46 \times 37.1 = 11401 \text{ kN}$$

Member $L_6 - L_7$, $L_7 - L_8$ F (LL + I):

$$349.01 \times 37.1 = 13506.76 \text{ kN}$$

Member $L_8 - L_9$, $L_9 - L_{10}$ F (LL + I):

$$37.192 \times 37.1 = 14207.6 \text{ kN}$$

Verticals and Diagonals:

Member $U_0 - L_0$ F (LL + I) = -5726.95 kN

Member $U_0 - L_1$ F (LL + I) = 4278.1 kN

Member $U_0 - L_2$ F (LL + I) = 5083.66 kN

Member $U_2 - L_2$ F (LL + I) = -4622.2 kN

Member $U_2 - L_3$ F (LL + I) = 476 kN

Member $U_2 - L_4$ F (LL + I) = 3903.66 kN

Member $U_2 - L_4$ (-) F (LL + I) = -338.46 kN

Member $U_4 - L_4$ F (LL + I) = 348.4 kN

Member $U_4 - L_4$ (-) F (LL + I) = - 3289.02 kN

Member $U_4 - L_5$ F (LL + I) = 474.44 kN

Member $U_4 - L_6$ F (LL + I) = 2749.64 kN

Member $U_6 - L_7$ F (LL + I) = 474.44 kN

Member $U_6 - L_8$ F (LL + I) = 1952.706 kN

Member $U_8 - L_8$ F (LL + I) = -1606 kN

Top chords:

Member $U_0 - U_1$, $U_1 - U_2$ F (LL + I) = -5394.19 kN

Member $U_2 - U_3$, $U_3 - U_4$ F (LL + I) = -9798.3 kN

Member $U_4 - U_5$, $U_5 - U_6$ F (LL + I) = -5394.19 kN

Member $U_6 - U_7$, $U_7 - U_8$ F (LL + I) = -14053.1 kN

Dead load, live load plus impact shall be combined in accordance to AASHTO LRFD Specification to find the highest stress in the desired member for truss design.

Design of the Truss Members:

In truss member design limiting slenderness ratio specified by AASHTO LRFD article 6.8.4 and 6.8.5 should be considered. Slenderness ratio for compression members should not be greater than 120 (KL/r < 120) and for tension members not be less than 200 (L/r < 200).

Where:

L: is unbraced length (cm)

r: is radius of gyration (cm)

K: is effective length factor, taken 1 for single angles regardless of end connection

Allowable stress may be increased by 25 percent for combination dead load, live load, and impact. Thus, if the sum of DL + LL + I is less than 1.25 times the allowable force in the member at nominal allowable stress, the member is acceptable.

Allowable stress for tension:

$$F_a = 0.55\, f_y$$

For steel A 514 Grade 50:

$$F_a = 0.55\ 689 = 378.95 \text{ MPa}$$

Allowable stress for compression:

$$C_c = \sqrt{\frac{2\pi^2 E}{f_y}} < KL/r \quad => \quad F_a = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$$C_c = \sqrt{\frac{2\pi^2 E}{f_y}} > KL/r \quad => \quad F_a = \left[1 - \frac{1}{2}\left(\frac{\frac{KL}{r}}{C_c}\right)^2\right] \times \frac{f_y}{F.S}$$

Where:

$$F.S = \frac{3}{5} + \frac{3}{8} \times \frac{\frac{KL}{r}}{C_c} - \frac{1}{8}\left(\frac{\frac{KL}{r}}{C_c}\right)^3$$

Table 14.15: Truss member design

Member	Stress in kN			L (cm)	r_{min} (cm)	L/r (Selected)	Allowable Stress	Area Required (cm^2)	Section (cm^2)
	DL	LL + I	Total						
U_0-U_2	−10502.95	−5394.19	−15897.14	1250	10.42	83.4	311852.97	509.76	W33 × 291(552)
U_2-U_4	−18377.3	−9798.3	−28175.6	1250	10.42	80.65	333821.1	844.03	W33 × 468 (884)
U_4-U_6	−23626.3	−12669.1	−36295.4	1250	10.42	78.125	418263.53	867.77	W33 × 468 (884)
U_6-U_8	−26250.3	−14053.1	−40303.4	1250	10.42	75.75	441723.57	912.42	W33 × 515(974)
L_0-L_2	+5582.7	+3111.4	+8694.1	1250	6.25	36.76	378950	229.43	W33 × 141(268)
L_2-L_4	+14767.45	+7886.1	+22653.55	1250	6.25	33.78	378950	597.8	W33 × 318(603)
L_4-L_6	+21328.9	+11401	+32729.9	1250	6.25	33.2	378950	863.7	W33 × 468 (884)
L_6-L_8	+25265.3	+13506.76	+38772.06	1250	6.25	32.65	378950	1023.15	W33 × 567(1071)
L_8-L'_8	+26576.74	+14207.6	+40784	1250	6.25	33.78	378950	1076.3	[illegible]
U_0-L_0	−9415	−5726.95	−15142	2253.47	18.78	112.68	170839	886.3	W33 × 468 (884)
U_0-L_2	+8942.3	+5083.66	+9526	2253.47	11.27	36.76	378950	251.4	W33 × 141(268)
U_2-L_2	−7190.1	−4622.2	−11812.3	2253.47	18.78	115.57	162423.04	727.3	W33 × 387 (729)
U_2-L_4	+6086.73	+3903.66	+4590.4	2253.47	11.27	37.74	378950	121.14	W33 × 118 (224)
U_4-L_4	−4977.6	−3289.02	−8266.62	2253.47	18.78	117.06	158284.98	522.3	W33 × 291(552)
U_4-L_6	+3875.3	+2749.64	+6624.94	2253.47	11.27	37.74	378950	72.56	W33 × 118 (224)
U_6-L_6	−2765.8	−2432.79	−5198.6	2253.47	18.78	118.61	154184.02	157.8	W33 × 118 (224)
U_6-L_8	+1663.4	+1952.71	+3616.11	2253.47	11.27	37.74	378950	95.43	W33 × 118 (224)
U_8-L_8	−554.1	−1605	−2159.1	2253.47	18.78	120	150632.78	143.33	W33 × 118 (224)
L_1-U_0	+884.6	+4278.1	+5162.7	1875	9.34	37.74	378950	136.24	W33 × 118 (224)
L_4-U_3	−8.66	−202.11	−210.77	1875	15.63	110.3	178310..65	11.78	-

4.3.1.5 Design of Lateral Bracings

AASHTO Specification requires that bridge trusses should be designed for a wind pressure of 3.6 kPa. Wind load on the live load should be taken as 4.8 kPa. The specifications further require total forces for not being of less than 4380 N/m and 2190 N/m in the plane of the loaded chord and in the plane of the unloaded chord respectively [12]. The wind has assumed to act on whatever area will result in the maximum force in particular member.

Area Exposed:

Railing plus parapets and floor:	$0.75 \times 12.5 = 9.375\ m^2$
Stringers:	$0.75 \times 12.5 = 9.375\ m^2$
Top chords:	$0.42 \times 12.5 = 5.25\ m^2$
Bottom chords:	$0.65 \times 12.5 = 8.125\ m^2$
Verticals:	$0.26 \times 18.75 = 4.875\ m^2$
Diagonals:	$0.4 \times 22.54 = 9.02\ m^2$
Total:	$46.02\ m^2$

Wind Load on Structure:

Wind on top chords:	$46.02 \times 3.6 = 165.672$ kN
Wind on bottom chords:	$8.125 \times 3.6 = 29.25$ kN
Wind on vehicles:	$0.3 \times 4.8 = 1.45$ kN/m

Chords:

Only 30 percent of the wind force on the structure need be taken in combination with wind and live load [12].

Load at on panel: 165.672/12.5 = 13.26 kN/m

$$M = \frac{1}{8} \times (225)^2 \times (0.3 \times 12.85 + 1.45) = 34337.22 \text{ kNm}$$

$$U_6\text{-}U_8 = 34337.22 / 33\text{m} = 1040.22 \text{ kN}$$

Checking the adequacy of member:

$$DL + LL + I + Ws < 1.25 \times F_a \times A_g$$

$$40303.4 + 1040.22 < 1.25 \times 441723.57 \times 0.0974$$

$$41343.62 \text{ kN} < 53779.85 \text{ kN} \quad \text{OK}$$

Adequacy of member for DL + Ws should be checked:

$$M = \frac{1}{8} \times (225)^2 \times 13.26 = 83910.94 \text{ kNkm}$$

$$U_6\text{-}U_8 = 83910.94 / 33\text{m} = 2542.76 \text{ kN}$$

$$DL + Ws < 1.25 \times F_a \times A_g$$

$$40303.4 + 2542.76 < 1.25 \times 441723.57 \times 0.0974$$

$$42846.16 \text{ kN} < 53779.85 \text{ kN} \quad \text{OK}$$

Top and Bottom Lateral Bracing Design:

Panel Shear at D_1 (Figure 4.38): 165.672 × 2 = 331.345 kN

Diagonal stress: $331.345 \times \frac{20.7}{16.5} = 415.687$ kN

The allowable compressive stress is taken to be that for *L/r* = 120:

$$F_a = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \Rightarrow F_a = 150785.6228 \text{ kN/m}^2$$

Required area: $\frac{415.687}{150785.63} = 0.002757 \text{ m}^2 = 27.57 \text{ cm}^2$ <u>Use W6 × 15</u>

For strut: r = 33 / 140 = 0.235 m = 23.5 cm <u>Use W21 ×122</u>

Since top lateral bracing carrying higher wind load, design has done in accordance of this bracing. Same section <u>W6 × 15</u> has used for bottom lateral bracing.

4.3.1.6 Portal and Sway Frame Design

AASHTO LRFD Specifications specifies that “the need for vertical cross-frames used as sway bracing in trusses shall be investigated” [18]. All through-truss bridges should have portal bracing, made as deep as clearance permits [9].

End panels of simply supported, through-truss bridges have compression chords that slope to meet the bottom chords [9]. Bracing between corresponding sloping chords of a pair of main trusses is called portal bracing [9] (Figures 2.3 and 4.38). Bracing between corresponding vertical posts of a pair of main trusses is called sway bracing [9] (Figures 2.3 and 4.38).

Through trusses should have sway bracing at least 1.5 m deep in highway bridges at each intermediate panel point [9] (Figure 4.38).

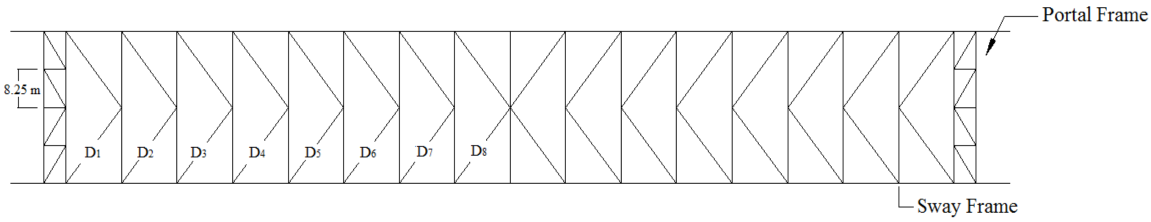


Figure 4.39: Top lateral bracing and location of sway frames

4.3.1.6.1 Portal Frame Design

Portal bracing should be designed to carry the full end reaction of the top chord lateral system [9, 18]. Therefore:

$$155.92 \times 17 = 2650.76 \text{ kN}$$

Reaction at each portal frame: $\frac{2650.8}{2} = 1325.4$ kN

In Figure 4.39 provided portal frame has shown. This frame was placed at end post of the truss where there is a slope.

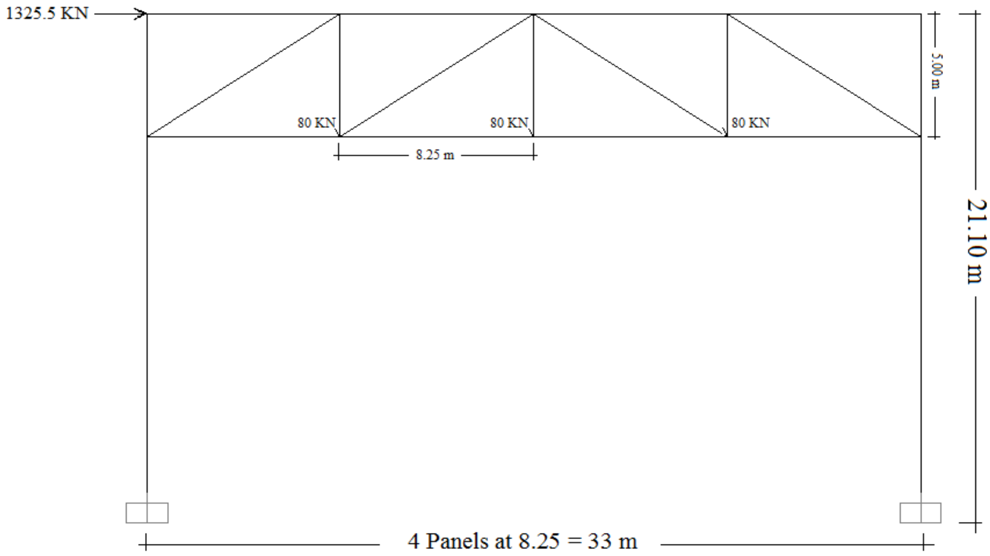


Figure 4.40: Portal frame loading

The provided portal frame considered as a fixed support frame. For design, load combination Strength III (1.25 DL + 1.4 Ws) has been used since the highest internal forces are at the frame. The structure has been analyzed by using SAP2000 Version 14 and the following results have been considered for design. The allowable stress is taken as the one for compression *L/r* = 120.

Top chords: -2527.93 kN

$$F_a = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \Rightarrow \frac{\pi^2 \times 2.2 \times 10^8}{(120)^2} = 150632.78 \text{ kN/m}^2$$

Required steel area: $\frac{2527.93}{150632.78} = 0.01356 \text{ m}^2 = 135.6 \text{ cm}^2$ <u>Use W33 × 118</u>

Bottom chord: 1608.3 kN

$$\text{Steel A572: } F_a = 0.55 f_y \Rightarrow 0.55 \times 345000 = 189750 \text{ kN/m}^2$$

Required steel area: $\frac{1608.3}{189750} = 0.008475 \text{ m}^2 = 84.75 \text{ cm}^2$ <u>Use W33 × 118</u>

Diagonals: -1547.5 kN

Required steel area: $\frac{1547.5}{150632.78}$ = 0.01027 m^2 = 102.73 cm^2 <u>Use W24 × 62</u>

Verticals: -530 kN

Required steel area: $\frac{530}{150632.78}$ = 35.18 cm^2 Due to aesthetic reasons: <u>Use W24 × 62</u>

4.3.1.6.2 Sway Frame Design

Sway bracing has been provided at each intermediate panel point (Figure 4.38). They should carry wind load and their own weight. In Figure 4.40 designed sway frame is shown. In order to design the frame for highest internal forces load combination Strength III from Table 3.4 has been used.

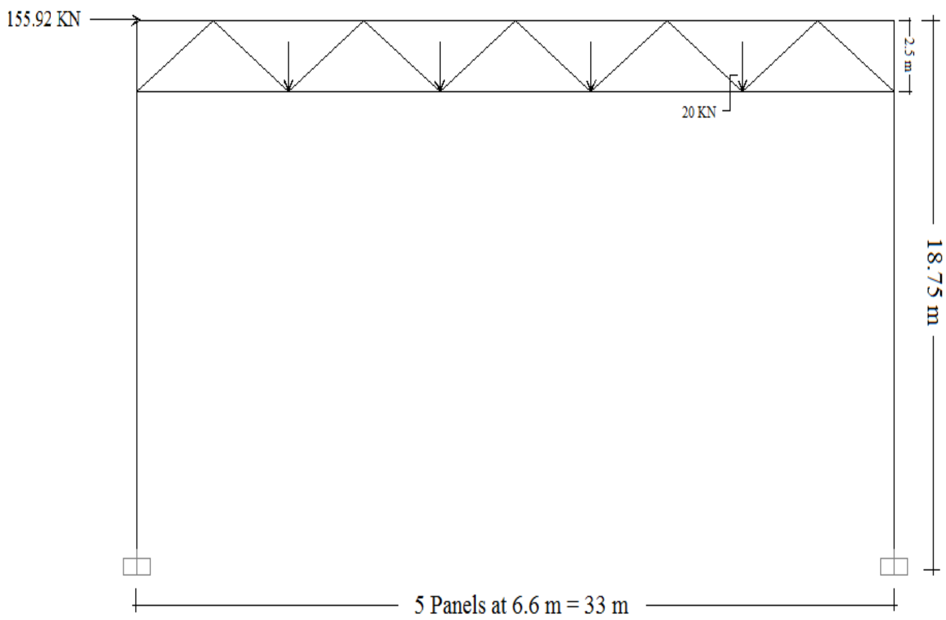


Figure 4.41: Sway frame loading

The frame has been analyzed by using SAP2000 version 14. Allowable stress for compression taken as *L/r* = 120.

The results for tension and compression members are as follows:

Top chords: -582.5 kN

$$F_a = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} => \frac{\pi^2 \times 2.2 \times 10^8}{(120)^2} = 150632.78 \text{ kN/m}^2$$

Required steel area: $\frac{582.5}{150632.78} = 0.00387 \text{ m}^2 = 38.7 \text{ cm}^2$ <u>Use W14 × 30</u>

Bottom chord: 407.3 kN

Steel A36: $F_a = 0.55\, f_y => 0.55 \times 240000 = 132000 \text{ kN/m}^2$

Required steel area: $\frac{470.3}{132000} = 0.00356 \text{ m}^2 = 35.63 \text{ cm}^2$ <u>Use W14 × 30</u>

Diagonals: -220.4 kN

Required steel area: $\frac{220.4}{150632.78} = 0.00146 \text{ m}^2 = 14.64 \text{ cm}^2$ <u>Use W12 × 14</u>

4.3.2 Medium Span Bridge Design

Truss bridge with medium span (126 m) has been designed according to articles 6.6.8 and 6.6.9 of AASHTO LRFD Specifications. Minimum slenderness ratio of L/r = 200 and L/r = 120 took into consideration for tension and compression members respectively. It should be mentioned that same design of portal frame and sway frames as long span (225 m) truss bridge have been provided. The plans and details of the bridge have been given in Appendix A.

The designed section and their properties have given in Table 4.16. These sections have been considered for final model and analysis by MIDAS/Civil in Chapter 5.

Table 4.16: Medium span truss bridge section properties

Section		Area (cm^2)	r_x (cm)	L/r_x	S_x (cm^3)	Weight per Meter (kg/m)
Floor Beam	H= 140 cm W= 50 cm W_t= 1.5 cm F_t= 2 cm	404	56	35.71	18096	311.1
Stringer	H= 75 cm W= 20 cm W_t= 1 cm F_t= 2 cm	133	27	33.34	2200	102.5
Top Chord	W21*275	521.3	24.5	36.73	10350	401.4
Bottom Chord	W21*223	422	24.3	37	8352	325
Verticals	W21*68	130	22	56.82	2296	101
Diagonal	W21*57	107	21.3	72.3	1822	82.5
Bottom Bracing	W6*15	28.6	6.6	203	160	22.1
Top Bracing	W6*15	28.6	6.6	203	160	87.3
Portal Frame (Top and Bottom Chord)	W21*83	157	22.1	18.1	2798.7	120.9
Portal Frame (Vertical and diagonal)	W21*62	118	21.7	18.5	2076.7	90.86
Sway Frame (Top and Bottom Chord)	W21*55	95	20.8	16.8	1549	73.15
Sway Frame (Vertical and diagonal)	W21*44	84	20.5	19.66	1337.8	64.7

Chapter 5

3-D ANALYSIS AND ASSESSMENT OF BRIDGES

5.1 Overview

In this chapter four bridges were analyzed and assessed for final comparison according to their designed components in the previous chapter. In order to have realistic and accurate enough results on all the bridges, computer analysis must be carried out. Real load cases can be simulated with all the actions considered in this project, such as dead load, traffic loads and wind load to compare the structural behavior of these bridge types.

The 3-D models have been created by MIDAS/Civil, which is a powerful software for all kinds of bridge analysis including tied-arch and truss bridge.

5.2 Establishment of the Models

Given that MIDAS/Civil is an extensive program, the choice of using different parameters to create the model is essential. These parameters usually have direct influence on the final analysis results.

5.2.1 Geometry

The geometry of tied-arch and truss bridge could easily be done by MIDAS/Civil. Arches and trusses are already defined in the software and only dimensions and number of segments or divisions has to be defined in layout tab.

5.2.2 Properties

5.2.2.1 Elements

In property module the characteristic of elements should be defined. Elements, such as ties and arch ribs, which carry moments were defined as beam elements, and those which only carry tension and compression like top chords, bottom chords and ties (For arch bridge) were defined as truss element.

Modeling of built up sections could easily be done by MIDAS Civil. Since most of the sections used in bridges (especially in Arch bridge) were built up sections, then using this software is an advantage.

5.2.2.2 Materials

The material properties are same as those in Tables 4.11 and 4.16 for arch and tied-arch bridge, respectively. ASTM Standard materials have already been defined in MIDAS Civil, therefore section materials could be easily be selected.

5.2.2.3 Loads and Boundary Conditions

For final analysis different loads and boundary conditions have been selected according to AASHTO LRFD Bridge Design Specifications. Real dead loads, traffic loads (Truck HS20-44) and wind loads were calculated and applied to structures as static load in MIDAS/civil. Non-linear analysis has neglected in this study.

Boundary conditions are simply supported on the right end and pinned at the left end for all bridges. Beam end release for truss members were considered as pinned-pinned condition due to truss rigidity, and for arch bridge both ends of hangers considered as pin joint condition about the Z-axis. Both ends of bracings and stringers considered as pinned-pinned and both ends of cross beams considered as fixed-pinned condition for all bridges.

5.3 Long Span Bridge Analysis and Comparison

In this section long span bridges (225m span) were modeled by MIDAS/Civil in order to compare the results of the behaviour of bridges final analysis of bridges behaviour under the specified loads. Bridges were analyzed, and the results gathered and separately in different parts for more convenient and accurate comparison. The results of analysis are compared in section 5.5 “Comparison of Results”.

5.3.1 Tied-Arch Bridge

The 225 m span tied-arch bridge modeled by using MIDAS/Civil is shown in Figure 5.1. Bridges were checked and compared according to the most important analysis outcomes, such as: support reactions, deformation, deflection and finally steel weight assessment.

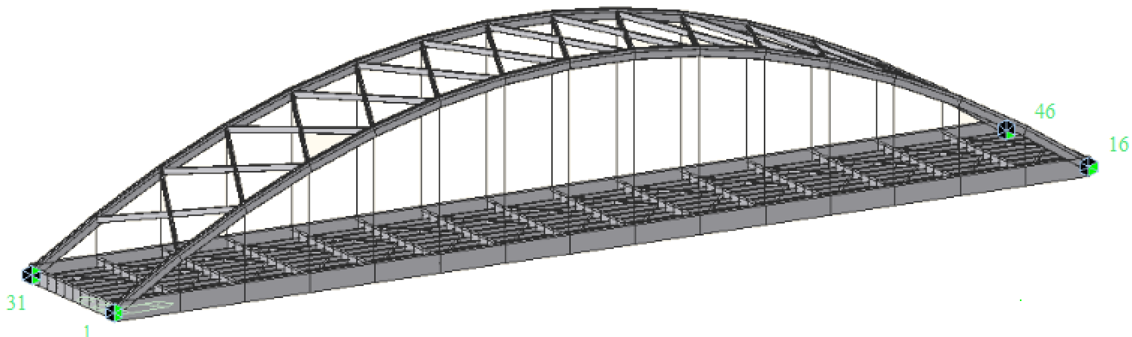


Figure 5.1: 3-D Model of Tied-Arch Bridge by MIDAS/Civil.

Real self-weight of the structure due to designed components was calculated for dead load application. Traffic loading defined as moving load in MIDAS/Civil. Vehicle class of HS20-44, which is the most common vehicle class in bridge design, was defined as moving load case. Dynamic load allowance of 33% is considered for moving load. Wind load was applied as a horizontal force in y-direction along the bridge’s length.

5.3.1.1 Supports Reaction

Supports reactions in each direction were investigated by MIDAS/Civil software. The values of reactions produced by dead load, live load, wind load and seismic load are needed for the design of abutments. Therefore as the forces at supports increase then the pier sections need to be made higher and stronger.

Supports reaction obtained by MIDAS/Civil in 3 directions (x,y,z) for different loading are listed in Table 5.1. It should be noted that in this study seismic forces are neglected.

Table 5.1: Support Reaction

Support (Node)	Load Type	Fx (kN)	Fy (kN)	Fz (kN)
1	Strength I	-462.8	68.0	18306.3
16	Strength I	0	-63.0	18310.5
31	Strength I	-427.0	0	18307.5
46	Strength I	0	0	18309.3
1	*WS*	-732.4	-698.0	-196.1
16	*WS*	0	0	-143.1
31	*WS*	253.4	0	196.1
46	*WS*	0	0	0

The highlighted rows are the most critical reactions which have appeared at support 1 (See Figure 5.1). It should be noted that, Fx and Fy are forces due to wind load on structure (*WS*) and Fz due to dead load (*DL*) and live load (Traffic loading). These reactions are illustrated in Appendix C.

5.3.1.2 Displacement and Deflections

According to AASHTO LRFD Bridge Design Specification, the deflections of the bridge should not exceed certain limits. Deflection due to live load plus impact should not exceed 1/800th of the span. This range has limited to 1/1000 of the span length for pedestrians safety and comfort.

The maximum vertical (Z-Direction) deflections at mid span due to different loading are presented in the following table. Maximum displacement shall be determined and reported from combination of dead load plus live load plus dynamic allowance.

Table 5.2: Tied-arch bridge vertical deflections and their limits

	Deflection (cm)	Limit (cm)
Dead Load	-16.3	-
Traffic Load	- 4.6	22.5
Wind Load	+ 0.788, - 0.76	-
DL + (*LL* + *IM*)	- 22.2	-

In order to find out the adequacy of bridge, deflection due to live load (Traffic load) should be checked. The deflection due to service live load is under the limit values. There is no problem concerning the deflections. The positive and negative value of wind load deflection in z-direction (Vertical) is due to lateral torsional buckling, which cause uplift (+) of the bridge span where wind pressure applied, and sag of the span on opposite side.

The maximum horizontal deflection (Y-Direction) due to wind loading calculated as 7.3 cm by MIDAS/Civil. The deflected structure due to wind load has been illustrated in in Appendix C.

5.3.1.3 Steel Weight Assessment

In Table 5.3 tonnage of steel used in tied-arch bridge is calculated. Area of each element is multiplied by weight per unit volume of steel and its own length. Weight per unit volume of steel is taken as 7.85×10^{-6} ton/cm^3.

Table 5.3: Tied-arch bridge steel weight assessment

Element	Steel Area (cm^2)	Length (cm)	Number of Element	Steel Weight (tonne)
Arch Ribs	2380	24070	2	899.4
Ties	3470	22500	2	1225.8
Rib Bracing	690	3625	26	510.5
Bottom Bracing	255	2122	30	127.5
Floor Beams	2300	3300	16	953.3
Stringers	403.5	1500	210	997.8
Hangers	245.5	Variable	28	1.5
Total Sum				4715.8

5.3.1.4 Total Weight of the Bridge

In previous section total weight of steel used in tied-arch bridge was calculated. In order to find the total weight of the structure, deck slab, roadway surfacing and railing were also considered. Total weight of the structure calculated in table 5.4.

Table 5.4: Tied-arch bridge total weight

	Area (m^2)	Weight per Volume (Ton/m^3)	Length (m)	Total Weight (tonne)
Deck Slab	4.95	2.4	225	2673
Roadway Surfacing	1.3	2.2	225	643.5
Railing and Parapets	0.2	7.9	225	706.5
Steel Weight	-	-	-	4715.8
Total Sum				8738.8

5.3.2 Warren Truss Bridge

For final analysis and comparison truss bridge was modeled by using MIDAS/Civil. The modeled truss is illustrated in Figure 5.2.

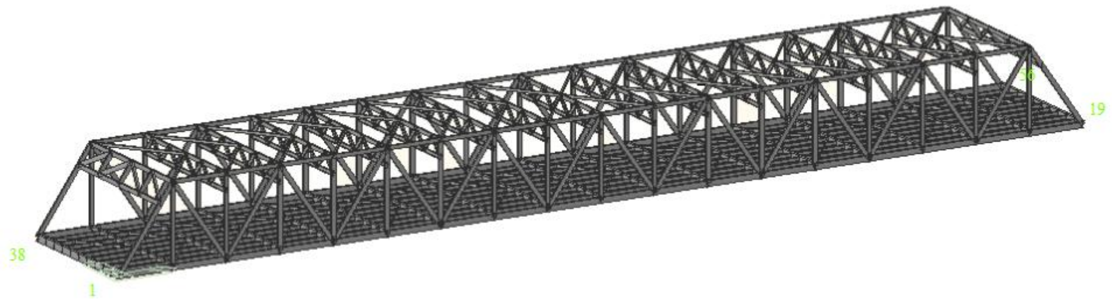


Figure 5.2: 3-D long span truss bridge model by using MIDAS/Civil

Traffic loading defined as moving load in MIDAS/Civil. Vehicle class of HS20-44, which is the most common vehicle class in bridge design, is defined as moving load case. Dynamic load allowance of 33% was considered for moving load

5.3.2.1 Supports Reaction

The support reactions obtained as a result of computer analysis for load combination Strength I produced the most critical forces which are listed in Table 5.5

Table 5.5: Supports Reaction for Truss Bridge

Support (Node)	Load Type	Fx (kN)	Fy (kN)	Fz (kN)
1	Strength I	-479.5	-80	3723.5
19	Strength I	0	80	3741.6
38	Strength I	483.4	0	3676.8
56	Strength I	0	0	3658.6
1	*WS*	-489	-209.5	-1.6
19	*WS*	0	150.6	1.9
38	*WS*	0	-182	-0.5
56	*WS*	0	0	0

5.3.2.2 Displacement and Deflections

The maximum vertical (Z-Direction) deflections at mid span due to different load cases are presented in the Table 5.6. Maximum displacement was determined from combination of dead load plus live load plus dynamic allowance. Dynamic allowance of 33% was considered from Table 3.1.

Table 5.6: Truss bridge vertical deflections and their limits

	Deflection (cm)	Limit (cm)
Dead Load	- 22.5	-
Traffic Load	- 6	22.5
Wind Load	+ 2.52, - 2.55	-
DL + (*LL* + *IM*)	- 28.5	-

Deflection due to live load is not exceeding the limit. There is no problem concerning the deflections. The positive and negative values of wind load deflections in z-direction (Vertical) are due to lateral torsional buckling, which caused uplift (+) of the bridge span where wind pressure applied and sagging of the span on the opposite side.

The maximum horizontal deflection (Y-Direction) due to wind loading calculated as 4.57 cm for truss bridge with sway frame. This value rises up to 97.12 cm for truss without sway frame. The deflected structure due to wind load is illustrated in Figure 5.3.

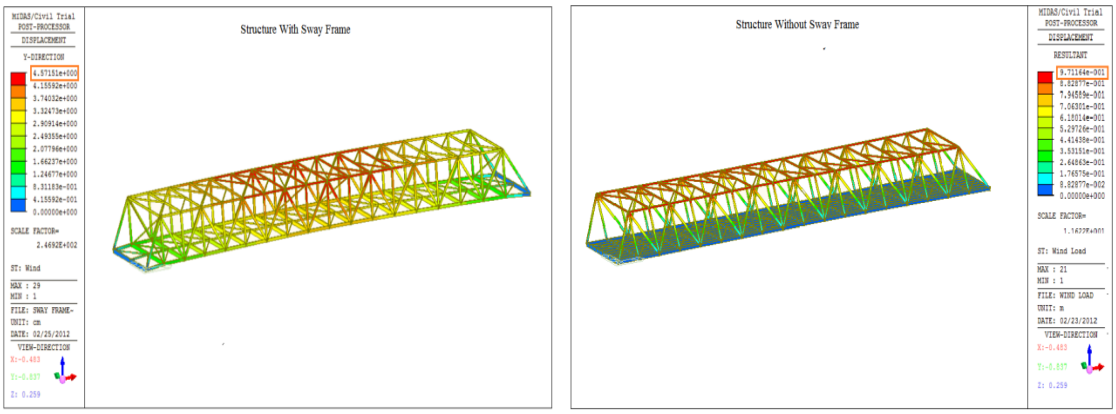


Figure.5.3: Horizontal deflections with and without sway frame

5.3.2.3 Steel Weight Assessment

In Table 5.7 tonnage of steel used in truss bridge is calculated. Weight per unit volume of steel is taken as 7.85×10^{-6} ton/cm^3.

Table 5.7: Truss bridge steel weight assessment

Element	Steel Area (cm^2)	Length (cm)	Number of Element	Steel Weight (Ton)
Top Chords	914	20000	2	287
Bottom Chords	1168	22500	2	412.6
Diagonals	884	2253.5	36	563
Verticals	224	1875	34	112.1
Floor Beams	1980	3300	19	974.55
Stringers	360	1250	252	890.2
Bottom Bracing	28.5	2070	34	15.75
Top Bracings	28.5	2070	30	13.9
Sway Frames $_{Tot}$	Variable	Variable	15	57.38
Portal Frames $_{Tot}$	Variable	Variable	2	33.07
Total Sum				3359.55

5.3.2.4 Total Weight of the Bridge

Total weight of the structure calculated in table below.

Table 5.8: Truss bridge total weight

	Area (m^2)	Weight per Volume (Ton/m^3)	Length (m)	Total Weight (Ton)
Deck Slab	4.95	2.4	225	2673
Roadway Surfacing	1.3	2.2	225	643.5
Railing and Parapets	0.2	7.85	225	706.5
Steel Weight	-	-	-	3359.55
Total Sum				7382.55

5.4 Medium Span Bridges Analysis and Comparison

In this section medium span bridges (126m span) are analyzed and compared together. The modeled bridges are illustrated in Figures 5.6 and 5.7. Support numbers are given for analysis comparison in next section. Analysis was carried out by MIDAS/Civil software and results are given in the following section. The same analysis parameters as those of long span bridges are considered for the medium span bridges.

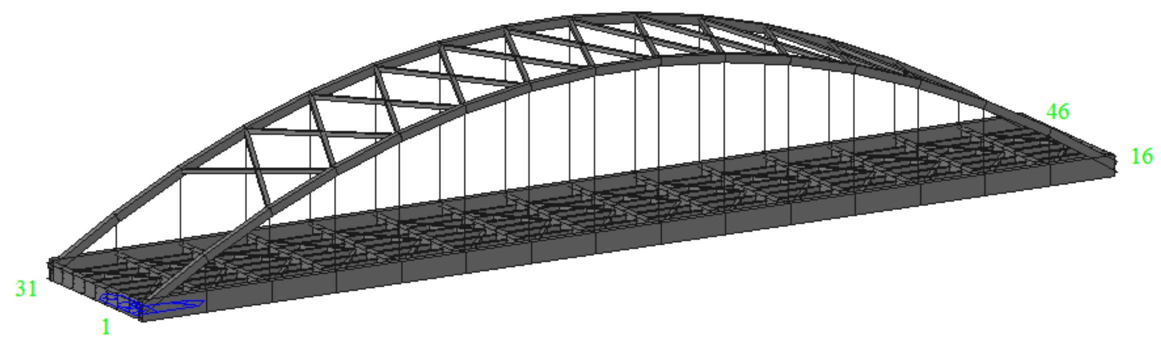


Figure 5.4: Medium span tied-arch Bridge

Truss bridge modeled by MIDAS/Civil is shown in Figure below. The plans of both bridges have illustrated in Appendix A. For more detail refer to the mentioned section.

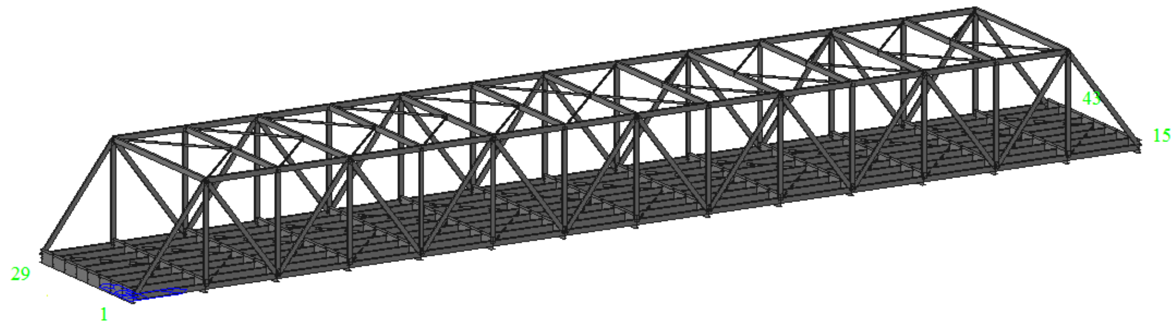


Figure 5.5: Medium span truss Bridge

5.4.1 Supports reaction

Supports reactions of both truss and tied-arch bridge obtained by MIDAS/Civil in 3 directions (x,y,z) are listed in Table 5.9. Reactions are produced by load combination Strength I which gives the most critical values.

Table 5.9: Support reaction of medium span bridges

Bridge Type	Support	Fx (kN)	Fy (kN)	Fz (kN)
Tied-Arch Bridge	1	-396	-63	6964.5
	16	0	63	6967.2
	31	396	0	6968.1
	46	0	0	6963.6
	1	-866.2	-747.5	-170
	16	0	-472.5	-128.8
	31	-864.2	0	169.3
	46	0	0	130
Truss Bridge	1	149.2	-22.7	1613.8
	15	0	23.7	1493.8
	29	149.2	0	1613.8
	43	0	0	1861.45
	1	-206.3	-84.3	-23.5
	15	0	-80.7	23.4
	29	206.3	0	23.5
	43	0	0	-23.5

In Table 5.9 support reactions of both tied-arch bridge and truss bridge are listed as a result of the final analysis. The highlighted rows are the most critical reactions which may be considered for piers and abutment design. These reactions appeared at support 1, due to load combination for both structures (Figures 5.6 and 5.7) and support 31 and 29 due to wind load for tied-arch and truss bridge respectively.

Reactions in X and Z-direction (Fz and Fx) of tied-arch bridge found to be more than 4 times of those of the truss bridge. In Y-direction also tied-arch bridge has higher reaction. This difference goes up to 665 kN which is considerably high.

5.4.2 Deflections

Vertical deflections (Deflections in Z-direction) of tied-arch bridge and truss bridge with span of 126 m due to different loading with their limits are listed in Table 5.10. It should be noted that there are no unique limitation for displacement of bridge subjected to wind load and dead load. Usually this matter is handled by using engineering judgement and/or previous experiences.

Table 5.10: Medium span bridges deflections

Bridge Type	Load Type	Deflection (cm)	Limit (cm)
Tied-Arch Bridge	Dead Load	-14	-
	Traffic load	- 5.1	12.6
	Wind Load	+ 0.345, - 0.341	-
	DL + (*LL* + *IM*)	-17	-
Truss Bridge	Dead Load	- 9.5	-
	Traffic load	-7	12.6
	Wind Load	+1.2, -1.2	-
	DL + (*LL* + *IM*)	-16.2	-

According to Table 5.10 deflection due to live load for both bridges are under limitation, therefore they are adequate and safe. Deflection due to wind load has cause the lateral torsional buckling for both bridges (row 3 for tied bridge and row 7 for truss bridge) which this deflection for tied-arch bridge is one third of the truss bridge.

Deflections in Y-direction due to wind load in both bridges are similar in value. For tied-arch bridge this deflection is 3.51 cm due to wind load, and for truss bridge is 3.77 cm. In Appendix C part C.3 deflected structures are illustrated.

5.4.3 Steel Weight Assessment of Medium Span Bridges

In Table 5.11 the tonne of steel used in tied-arch bridge and truss bridge are calculated. Weight per unit volume of steel is taken as 7.85×10^{-6} ton/cm^3.

Table 5.11: Steel weight assessment of medium span bridges

Bridge Type	Element	Steel Area (cm^2)	Length (cm)	Number of Elements	Steel Weight (tonne)
Tied-Arch Bridge	Arch Ribs	523.32	13401.5	2	110.2
	Ties	1132	12600	2	224
	Rib Bracing	234	2170	26	103.7
	Bottom Bracing	136.3	1300	30	41.8
	Floor Beams	404	2000	16	101.5
	Stringers	133	840	135	118.4
	Hangers	78.54	Variable	28	0.7
Total Sum					700.3
Truss Bridge	Top Chords	521.3	10800	2	88.4
	Bottom Chords	422	12600	2	83.5
	Diagonals	137	1540.3	28	46.4
	Verticals	130	1250	24	30.6
	Floor Beams	404	2000	15	94.3
	Stringers	133	900	117	110
	Bottom Bracing	28.5	1345.4	28	8.5
	Top Bracings	28.5	1345.4	24	7.3
	Sway Frames $_{Tot}$	Variable	Variable	13	38.3
	Portal Frames $_{Tot}$	Variable	Variable	2	19
Total Sum					462.8

In Table 5.11 steel weight used in medium span bridges with span of 126 m is given. The total amount of steel used in tied-arch bridge turns out to be 1.5 times higher than truss bridge. This amount makes a difference of 237.5 Ton of steel between these two bridges.

In order to find out the total weight of the structures, other factor such as deck slab, road way surfacing, parapets and railing took in to consideration. In Table 5.8 total weight of the structure is assessed.

Table 5.12: Total weight of medium span bridges

Bridge Type	Element	Area (m^2)	Weight per Volume (Ton/m^3)	Length (m)	Total Weight (Ton)
Tied-Arch Bridge	Deck Slab	2.6	2.4	126	786.3
Tied-Arch Bridge	Roadway Surfacing	0.75	2.2	126	207.9
Tied-Arch Bridge	Railing and Parapets	0.2	7.85	126	198
Tied-Arch Bridge	Steel Weight	-	-	-	700.3
Total Sum					1892.3
Truss Bridge	Deck Slab	2.6	2.4	126	786.3
Truss Bridge	Roadway Surfacing	0.75	2.2	126	207.9
Truss Bridge	Railing and Parapets	0.2	7.85	126	198
Truss Bridge	Steel Weight	-	-	-	462.76
Total Sum					1654.8

As it was expected the total weight of tied-arch bridge structure became higher with average dead load of 97.5 kN/m than truss bridge. The average dead load of truss bridge calculated as 78.5 kN/m.

5.5 Comparison of Results

In this chapter tied-arch bridge and truss bridge have analyzed and compare in different spans of 225 m and 126 m. This analysis and comparison has done according to the designed components in previous chapter. The most important aspects of analysis such as support reaction, for required abutment, piers, and foundation; bridge deflection, for bridge stability, safety and comfort of the passengers; and weight of the structure for bridge economy have been investigated for each bridge.

5.5.1 Support Reaction

Loads applied on the bridge deck, transfers to the earth through piers and foundation. For design of piers, support reaction plus pier's own weight, and lateral loads (wind load, seismic load, impact force, braking force, soil pressure and etc.) should be calculated. After calculation, the most unfavorable forces shall be considered for the design. It is obvious that as the forces at supports become greater the piers section would be bigger. Therefore, support reactions were investigated in Table 5.13 to identify which bridge type needs bigger substructure (Piers, abutment and foundation).

Table 5.13: Support reaction of long and medium span bridges

Bridge Type	Span (m)	Fx_{max} (kN)	Fy_{max} (kN)	Fz_{max} (kN)	Difference (kN)
Tied-arch	225	-733	-698	18311	Fx = 244 Fy = 488 Fz = 14570
Truss	225	-489	-210	3742	
Tied-arch	126	-866	-747	6968	Fx = 660 Fy = 662 Fz = 5354
Truss	126	-207	-85	1614	

Basically piers and footings design for stresses which calculated by axial forces at supports (Forces in Z-direction) and lateral forces (X and Y-directions) which produce the moment in piers.

The axial force has direct effect on steel area required for footing, since stress in footing is $\sigma = \frac{P}{A}$ and moment produced at footing equals to $M = \frac{\sigma \times L^2}{2}$, steel required for footing can calculated from $A_s = \frac{M}{fJd}$.

Maximum supports reaction for long span bridges (225 m) appeared in tied-arch bridge is nearly 5 times higher than the support reaction of truss bridge in same direction. This difference is considerably high when it comes to footing design.

The minimum area of steel requires in footing for tied-arch bridge would be higher than truss bridge. On the other hand bridge piers shall design and check for lateral forces which cause the moment. In this respect tied-arch Bridge has also higher support reaction.

Maximum support reaction in Z-direction of tied-arch bridge with span of 126 m (Medium span) is nearly 4 times higher than truss bridge with the same span. Reactions due to lateral load (wind load on structure) are also higher in tied-arch bridge. These results show that tied-arch bridge requires a stronger piers and footings in both ranges of span.

5.5.2 Deflections

Since the structures are considered as simply supported, the deflection is expected to occur at mid-span (Appendix C). Deflections due to, traffic load (live load) plus dynamic allowance, wind load and dead load plus traffic load plus dynamic allowance for both medium (126 m) and long span (225 m) bridges are listed in Table 5.14.

Table 5.14: Deflections of medium and long span bridges

Bridge Type	Span (m)	*LL+IM* (mm)	*WS* (mm)	*DL+LL+IM* (mm)	Difference (mm)
Tied-arch	225	-46.0 < 225	+7.9,-7.6	-222.2	*LL+IM* = 14.0 *WS* = 17.9 *DL+LL+IM* = 63.0
Truss	225	-60.0 < 225	+25.2,-25.5	-285.0	
Tied-arch	126	-51.0 < 126	+3.5, -3.4	-170.0	*LL+IM* = 19.0 *WS* = 8.5 *DL+LL+IM* = 8.0
Truss	126	-70.0 < 126	+12.0,-12.0	-162.0	

As previously mentioned deflection due to live load plus dynamic load allowance should not exceed 1/1000th of span length. These deflections are below the limits (third column) and all bridges are adequate in this respect.

Vertical deflection of truss bridges (for both medium and long span) due to wind load, which caused the lateral torsional buckling, is considerably higher than tied-arch bridges; despite the fact that the wind pressure is higher in tied-arch bridges because of their exposed area.

5.5.3 Required Steel Weight

Steel weight assessment of four bridges (medium and large span) is shown in figure 5.8.

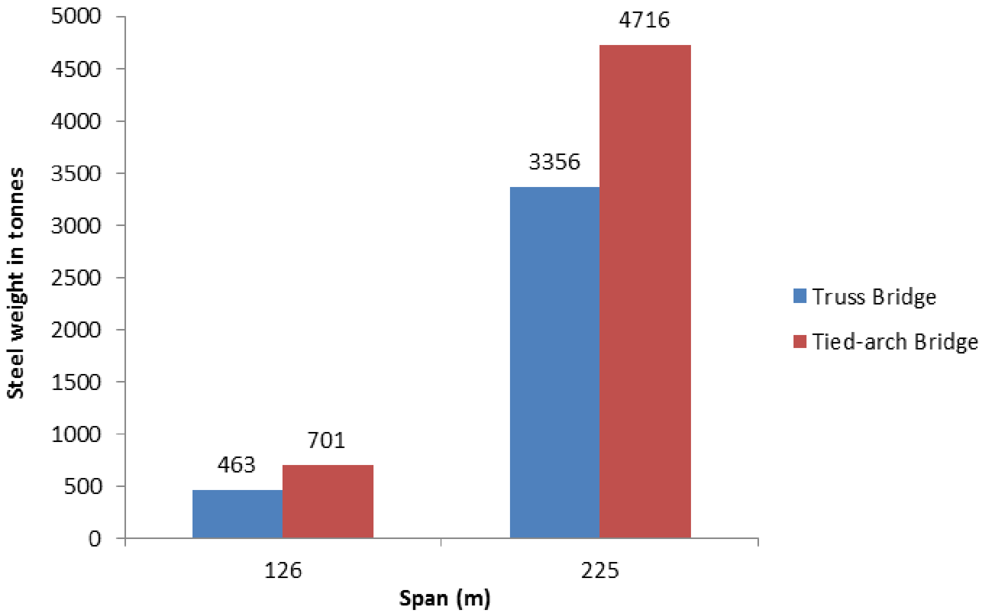


Figure 5.6: Steel weight assessment of medium and long span bridges

Form figure 5.8 one can observe that total weight of steel needed for tied-arch bridge is 40 percent higher than the truss bridge with same span of 225 m. This is mostly due to steel sections used for the arch ties which carry substantial moments and tensile stresses. However, it should be noted that in this study most of the assumed steel sections have ratios of one half or one third of the limitation. Therefore, more efficient, steel section can be selected. In medium span bridges the total steel required of tied-arch bridge was 49 percent higher than truss bridge.

Chapter 6

SUMMARY AND CONCLUSION

6.1 Summary

Current research is derived from the importance of bridge engineering in recent years. This study indicates that most of the bridges built in United States are arch bridge and truss bridges where span range of 40 m to 380 m is required. The tied-arch and truss bridges are very competitive when spans up to 280 meters are considered. This was the reason for the objective of the design and evaluation of these bridges under certain loading in different ranges of span. Therefore, spans lengths of 126 m and 225 m considered for design and evaluation of these bridge types.

Tied-arch and truss bridges are usually built out of steel since it has lighter weight and more flexibility than concrete. Steel superstructures are rarely governed by earthquake criteria, because they are generally lighter in weight than a concrete superstructure, lower seismic forces are transmitted to the substructure elements.

Bridges should be designed in accordance to the most critical situation of vehicles on bridge. The design of steel bridges should be carried out due to wind load and later on to be checked for substructure stability due to seismic forces. Therefore, this study focused on superstructure of the bridges, the design and analysis were done due to traffic load, wind load and structures' self-weight.

In order to prevent bridge failure it is important to learn from the past experiences. In this study bridges failed due to variety of causes with the exception of seismic action for both truss and arch bridges were identify and listed in chapter 2. This research is based on the recorded bridge failures from 1876 to 2010 over internet and published papers.

During these years 52 bridge failures were recorded for both truss and arch bridge type (concrete and steel). Steel truss bridge (18%), Steel arch bridge (21%) and concrete arch bridge (27%) account for a total of 66 percent of all the bridge failures. Arch bridge with masonry, stone and/or brick has 34 percent failure rate which was the highest percentage of failure. These materials were generally used in the past and they might have experienced more actions, such as, war damages, earthquake and so on. Therefore, it is logical for this type of bridge to have the highest percentages.

In arch bridges, 10 failures occurred during construction 5 of which were concrete, 3 were steel structures and 2 were bridge built with other materials. Most of these failures for concrete arch bridges were resulted from inadequate scaffolding. On the other hand, 10 of these failures occurred during the bridge was in service. Five of these failures were steel structure and only 3 recorded for concrete structure. Mostly these failures were due to fatigue and corrosion in steel and fracture in concrete structures.

In truss bridges, 56% of the failures were due to fatigue and corrosion, especially in gusset plates and connections. Other failures were due to storms, ice pack collision and during construction.

The weakest components of the bridge should be inspected regularly giving them greater attention. For example, in truss bridges gussets and connections are in danger of corrosion and fatigue, this matter could result in progressive collapse of bridge while subjecting to traffic load in long term. I-W35 Bridge is a great example of truss bridge progressive collapse. In tied-arch bridges the most effective components are hangers. Since hangers are always in tension, vibration due to vehicles could easily loose its strength and rigidity and cause the progressive collapse of tied-arch bridge. Hangers must be designed with sufficient rigidity to prevent adverse vibration due to aerodynamic forces. Corrosion resistance and provision for future replacement are other concerns which must be addressed in design of wire hangers.

Box shapes generally offer greater resistance to vibration due to wind and buckling in compression, and torsion, but require greater care in the selection of welding details.

Several design approximation were developed in the process of tied-arch bridge and truss bridge design. Both designs conducted by AASHTO LRFD bridge design specification. First deck components such as concrete slab, stringers and floor beams were designed, and then 2-D analysis and design of bridges structure conducted. This approach provided a useful starting point in the design.

The calculation and design procedure evolved in tied-arch bridge were time consuming and considerably more complicated than truss bridge since tied-arch bridge components deal with different load conditions and more parameters are required for design procedure.

6.2 Major Findings

This study indicated that, the designed components of both tied-arch bridge and truss bridges are suitable and both bridges performed well with minor difference in deflection due to traffic load. Tied-arch bridge for span range of 225 m has lower deflection due to dead load and traffic loading. These deflections might be problematic during construction (dead load) and in the long term (traffic load).

Lateral torsional buckling due to wind load is considerably higher in both truss bridges (medium and long span) which could cause the movement of the bridge deck. In this respects tied-arch bridge structures' shows better performance**.** Therefore, it is better to use box section or H shaped sections for trusses in this ranges of span, since box section resisted better against the torsion in tied-arch bridge when subjected to wind pressure twice that of truss bridge. Furthermore Box sections or H shaped sections can also reduce the dead load deflection of long truss bridge.

From the final analysis one can conclude that, tied-arch bridges of 126 m and 225 m span (medium and long span) requires stronger and/or bigger section of piers, footings and abutments than truss bridge. Of course this depends on the type of the supports, location of the bridge (over river, road and so on), type of the foundations, soil conditions and etc. But it can be concluded that since the reactions at the supports of tied-arch bridge is higher, then it required relatively stronger supports.

For span ranges of 126 m and 225 the truss bridge requires less steel than the tied-arch bridge for the same ranges of span. This shows the efficiency of truss bridges, which can span up to 225 with a lower amount of steel. The other advantage of truss bridge is being an ideal bridge for places where large parts or sections cannot be shipped or where large cranes and heavy equipment cannot be used during erection. Overall it can be said that truss bridges are easier to construct. The disadvantage of the truss bridge is its lack of aesthetic appeal because of its structural system.

One of the most important advantages of the tied-arch bridge is the capability of its ties to withstand the horizontal thrust forces which would normally be exerted on the abutments of an arch bridge (without ties). Thus there is no need of relying on the foundation to restrain the horizontal forces.

The tied-arch bridge has high cost and it is difficulty to construct. Usually, the construction requires more material (for temporary structures) and advanced equipment, such as, cranes.

In conclusion, selecting on of these bridge types in this range of span depends on some factors such as, economy, aesthetic, availability of equipment, location of the structure, structural application and etc. However, nowadays aesthetic of bridge is as important as its economy.

Therefore, if the cost of the arch bridge is same or slightly higher for than the truss bridge, or the budget is not a problem, then due to aesthetic and stability considerations the arch bridge would be selected instead of the truss bridge, especially in places where public usage is required.

6.3 Recommendations for Future Studies

This thesis is expected to introduce new opportunities for graduate students at EMU to investigate further into bridge engineering. Although this study considered many aspects of bridge design and analysis and answered many questions regarding the evaluation of tied-arch and truss bridges with span ranges of 126 m and 225 m and additional topics remain unexplored by the current research. Such as:

- For more accurate result more advanced software, such as, LUSAS Bridge should be used for the design and analysis of the bridges.
- Seismic performance of structures should be determined for substructure of bridges. If needed non-linear analysis should be carried out
- Dynamic analysis could be carried out for tied-arch bridges, since there are many problems concerning the vibration of hangers.
- Construction methods and maintenance over the life time of tied-arch bridge and truss bridge should be studied to have a more idea about the advantages of these bridges over each other.
- Similar bridges with reinforced concrete can be designed and analyzed to find out the advantages and disadvantages of steel bridges over reinforced concrete ones.
- Further comparison of bridges with longer spans such as, cable-stayed and suspension bridges can be carried out.

REFERENCES

[1] Xanthakos, P.P. 1994. Theory and Design of Bridges, John Wiley & Sons, Inc. New York, NY.

[2] Tahooni, Sh. 2008. Bridge Design, 2nd edition. Tehran University Press. Tehran, Iran.

[3] http://en.wikipedia.org/wiki/List_of_longest_bridges_in_the_world

[4] ASCE, 1976 "American Wood Bridges", Committee on History and Heritage, ASCE, Historical Publication No.4.

[5] Chen, W.F. and E.M. Lui., Ed., 2005. Handbook of Structural Engineering, Second Edition, CRC Press.

[6] Tihitina Siyoum, Thesis, 2007. Analysis and Design of Assembled Steel Truss Bridge. Addis Ababa University.

[7] V.N. Vazirani and S.P. Chandola. (2003), Railway, Bridges and Tunnels, 4th edition, Delhi, KHANNA Publishers.

[8] Chen, W.F. and Duan, L., Ed., 2000. Bridge Engineering Handbook, CRC Press, Boca Raton, FL.

[9] Roger L. Brokenbrough & F.S. Merritt, 2006. Structural Steel Designer's Handbook. The McGraw-Hill Companies, Inc

[10] Lianying Zheng & Zhongchu Tian, 2008. Construction and Construction Monitoring of Steel Box Arch Bridges. Journal of Changsha Communication University -14 July 2008.

[11] O' Connor & Colin 1971. Design of Bridge Super Structures, Wiley Press. New York, NY.

[12] Edwin H. Gaylord, Charles N. Gaylord & James E. Stall Meyer, 1992. Design od Steel Structures. The McGraw-Hill International Editions, Inc.

[13] Zlatko Šavor, Marta Šavor, Jing Gao & Marin Franetović, 2011. Failure of Arch Bridges – Causes, Lessons Learned and Prevention. University of Zagreb.

[14] Åkesson, B. 2008. *Understanding Bridge Collapses*. London: Taylor & Francis.

[15] Scheer, J. 2010. *Failed Bridges – Case Studies, Causes and Consequences.* Berlin: Ernst & Sohn

[16] Astaneh-Asl, A., (2008). "Progressive Collapse of Steel Truss Bridges, the Case of I-35W Collapse", Invited Keynote paper, *Proceedings* , 7th International Conference on Steel Bridges, Guimarães, Portugal, 4-6 June, 2008.

[17] David C.Simons, 2007. The Impact of the AASHTO LRFD Design Code on Bridge Stiffeners and Strength.

[18] AASHTO. 2010. AASHTO LRFD Bridge Design Specifications, 5th Edition, American Association of State Highway and Transportation Officials, Washington, DC.

[19] Buick Davison & Graham W. Owens, 2003. Steel Designers' Manual, 6th Edition. Blackwell Press.

[20] SAP2000, Version 14.0.0, Computers and Structures, Inc., Berkeley.

[21] MIDAS CIVIL, Version 2011, Midas Information and Technology Co. Ltd.

APPENDICES

Appendix A: Plans and Properties of Bridges

A.1 Long Span Arch Bridge

Type: Tied, Through, Solid Ribbed arch, 15 panels at 15m

Span: 225 m   Rise: 37.5 m   Rise/Span: 1:6

No. of Traffic Lanes: 6 lanes of 3.6 m   Width: 33 m

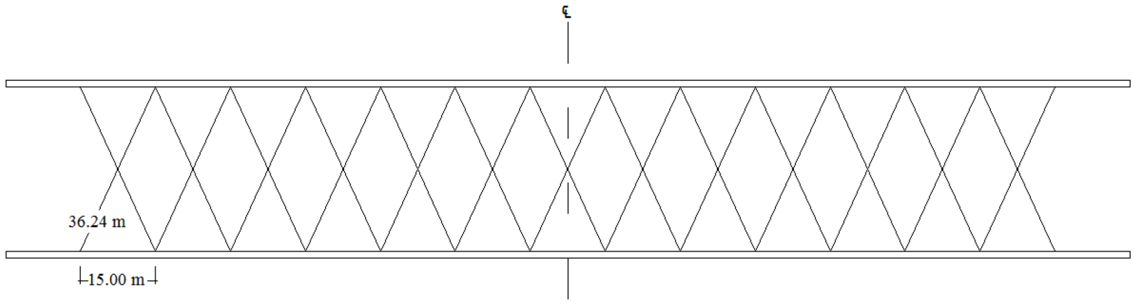
Structure's Total Weight and Average Dead Load: tonnes

- Deck slab and roadway surface 3316.5
- Railing and parapets 706.5
- Arch ties 1225.8
- Arch rib 899.4
- Floor bracing 127.5
- Rib bracing 510.5
- Floor beams 953.3
- Stringer 997.8
- Hangers 1.5
- Total Weight .... 8738.8 Ton
- Average Dead Load .... 381.5 kN/m

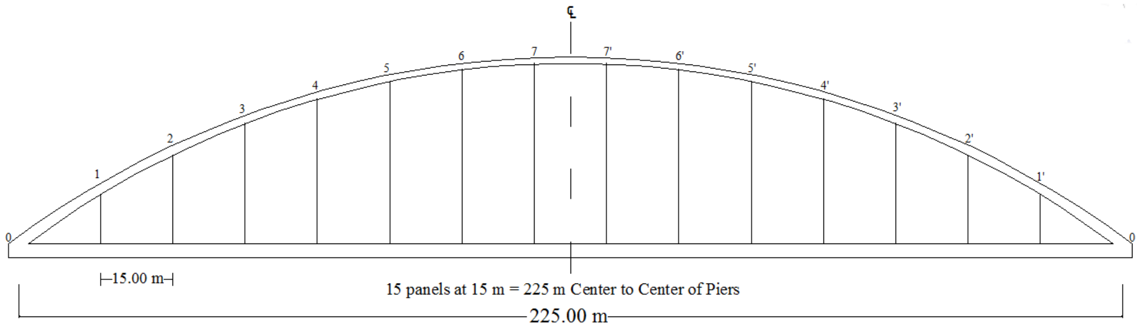
Specification Live Loading: HS20-44

Types of Steel in Structure:

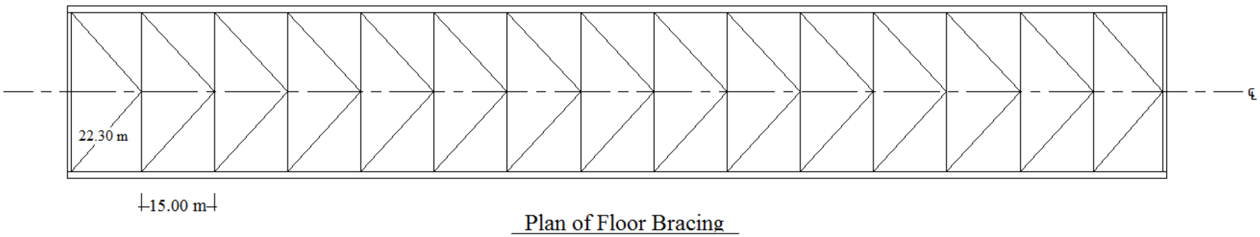
- Arch ribs A514
- Ties A588
- Bracings A36
- Floor beams and Stringers A36
- Hangers .... wire rope



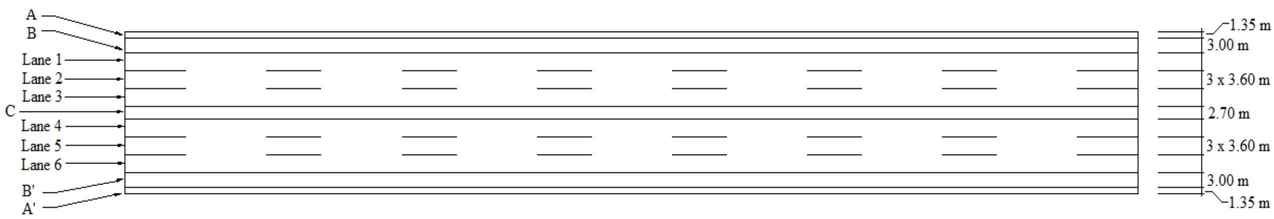
Arch Rib Bracing Plan



Arch Elevation



Plan of Floor Bracing



Roadway Plan

A, A' - Railing and Parapet

B, B' - Pedestrian Sidewalk

C - Road Seprator

A.2 Long Span Truss Bridge

Type: Warren, Through, with vertical members, 18 panels at 12.5m

Span: 225 m   Rise: 18.75 m   Rise/Span: 1:12

No. of Traffic Lanes: 6 lanes of 3.6 m   Width: 33 m

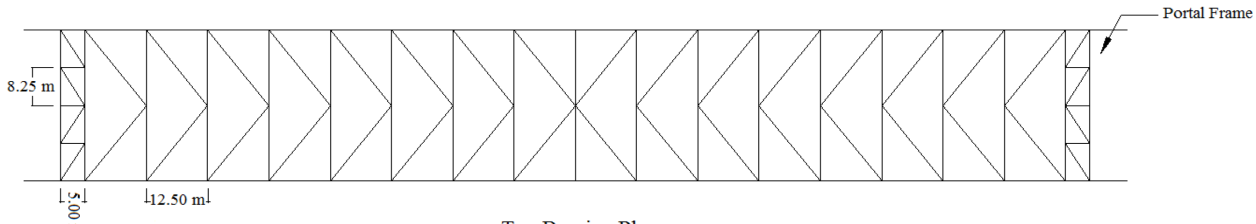
Structure's Total Weight and Average Dead Load:

	tonnes
Deck slab and roadway surface	2673
Railing and parapets	643.5
Top chord	287
Bottom chord	412.6
Verticals	112.1
Diagonals	563
Bottom bracing	15.75
Top bracing	13.9
Sway frame	57.4
Portal frame	33.07
Floor beams	974.6
Stringers	890.2
Total Weight	7382.5
Average Dead Load	277.5 kN/m

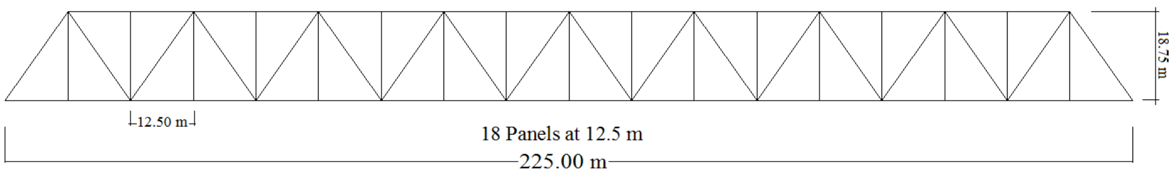
Specification Live Loading: HS20-44

Types of Steel in Structure:

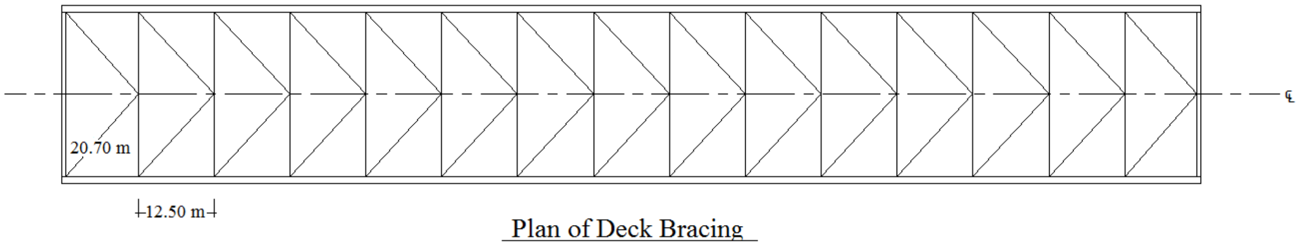
Top chords and bottom chords	A570
Verticals and diagonals	A514
Bracings	A36
Floor beams and Stringers	A36
Sway frames	A36



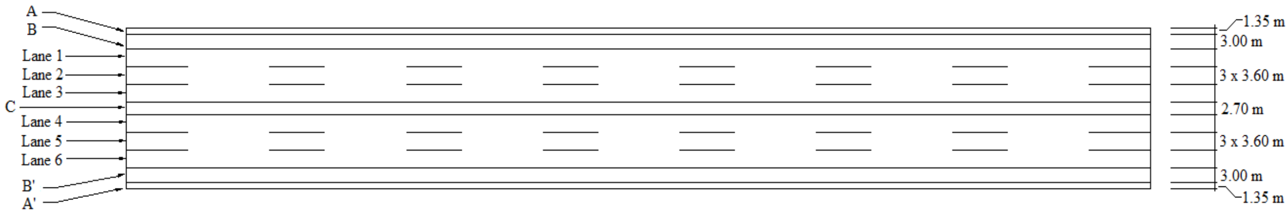
Top Bracing Plan



Truss Elevation



Plan of Deck Bracing



Roadway Plan

A, A' - Railing and Parapet

B, B' - Pedestrian Sidewalk

C - Road Seprator

A.3 Medium Span Arch Bridge

Type: Tied, Through, Solid Ribbed arch, 15 panels at 8.4 m

Span: 126 m   Rise: 20 m   Rise/Span: 1:6.3

No. of Traffic Lanes: 4 lanes of 3.6 m   Width: 20 m

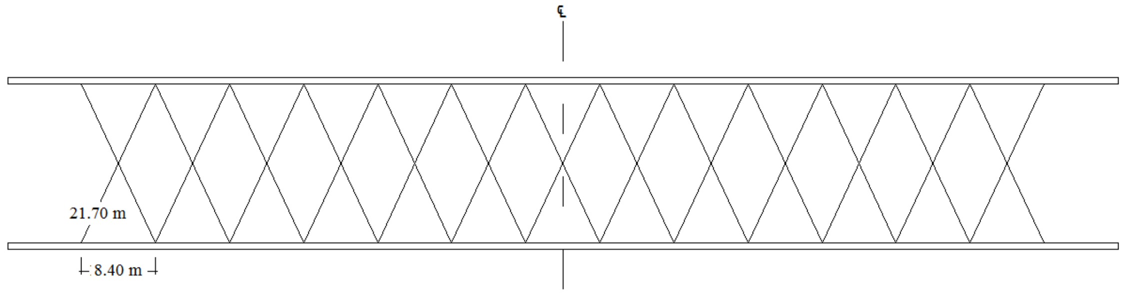
Structure's Total Weight and Average Dead Load   tonnes

Deck slab and roadway surface	994.2
Railing and parapets	198
Arch ties	224
Arch rib	110.2
Floor bracing	41.8
Rib bracing	103.7
Floor beams	101.5
Stringer	118.4
Hangers	0.7
Total Weight	1892.4
Average Dead Load	97.5 kN/m

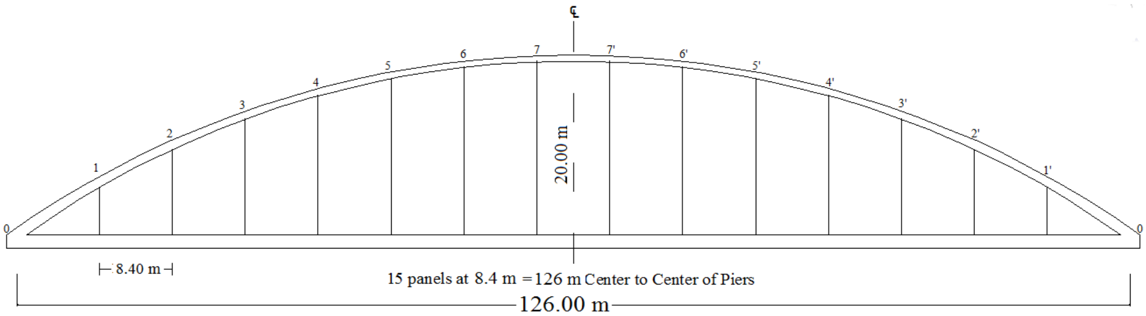
Specification Live Loading: HS20-44

Types of Steel in Structure:

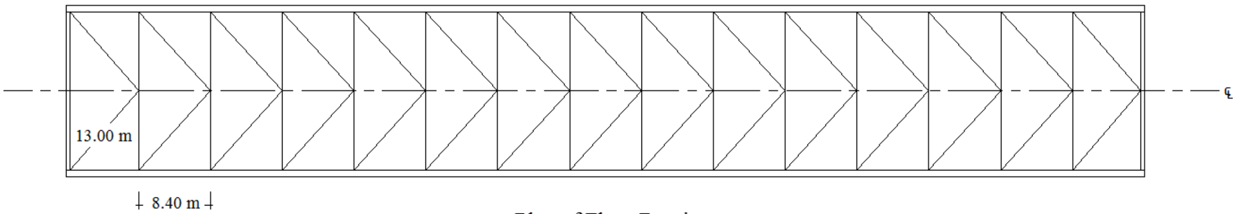
Arch ribs	A 514
Ties	A 588
Bracings	A 36
Floor beams and Stringers	A 36
Hangers'	wire rope



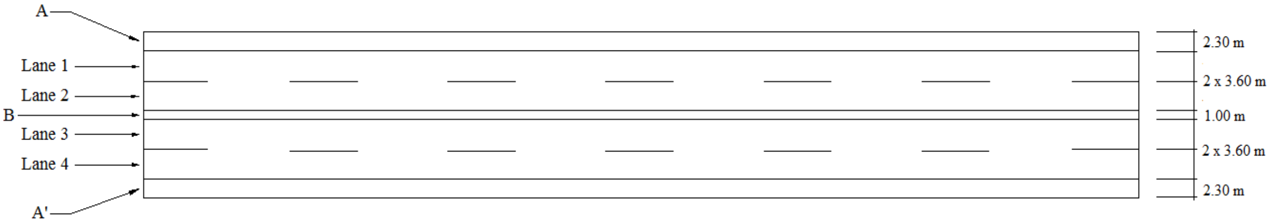
Arch Rib Bracing Plan



Arch Elevation



Plan of Floor Bracing



Roadway Plan

A, A' - Sidewalk and Railing

B - Road Separator

A.4 Medium Span Truss Bridge

Type: Warren, Through, with vertical members, 14 panels at 9 m

Span: 126 m   Rise: 12.5 m   Rise/Span: 1:10

No. of Traffic Lanes: 4 lanes of 3.6m,   Width: 20 m

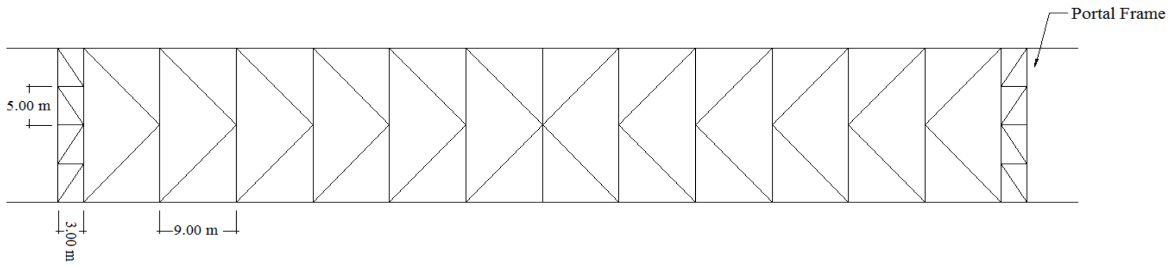
Structure's Total Weight and Average Dead Load: tonnes

Deck slab and roadway surface	786.3
Railing and parapets	207.9
Top chord	88.4
Bottom chord	83.5
Verticals	30.6
Diagonals	46.4
Bottom bracing	8.5
Top bracing	7.3
Sway frame	38.3
Portal frame	19
Floor beams	94.3
Stringers	110
Total Weight	1654.8
Average Dead Load	78.5 kN/m

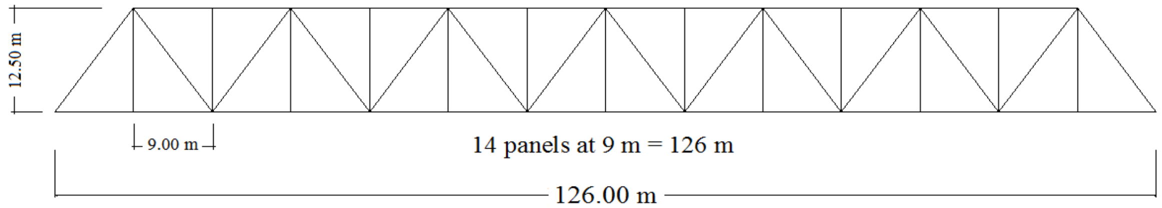
Specification Live Loading: HS20-44

Types of Steel in Structure:

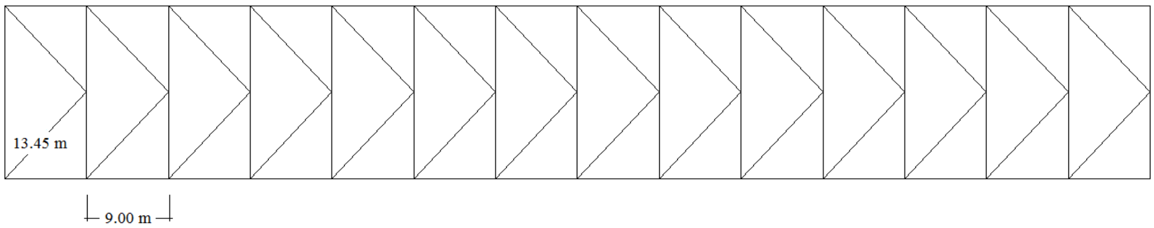
Top chords and bottom chords	A570
Verticals and diagonals	A514
Bracings	A36
Floor beams and Stringers	A36
Sway frames	A36



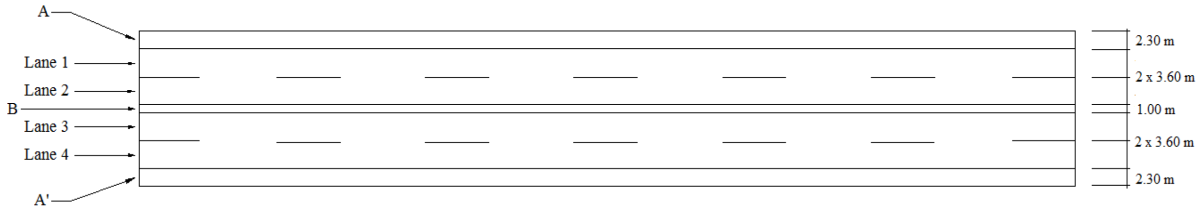
Top Bracing Plan



Truss Elevation



Deck Bracing Plan



Roadway Plan

A, A' - Sidewalk and Railing

B - Road Separator

Appendix B: Calculation of Loads and Actions on Bridges

B.1 Tied-Arch Bridge

- Deck Concrete Slab Dead Load:

Asphalt: $0.05\text{m} \times 22\ \text{kN/m}^3 = 1.1\ \text{kN/m}^2$

Concrete Slab: $0.15\ \text{m} \times 24\ \text{kN/m}^3 = 3.6\ \text{kN/m}^2$

Total Dead Load For Slab: $4.7\ \text{kN/m}^2$

- Stringers Dead Load and Live Load:

Dead load:

Assumed Stringer Self Weight: 2 kN/m

Slab Weight = DC: $2.15 \times 0.15 \times 24 =$ 7.74 kN/m

Asphalt = DW: $2.15 \times 0.05 \times 22 =$ 2.4 kN/m

Total Dead Load= 1 2.1 kN/m

Live Load:

Live load on stringer: $\frac{s}{1.68} = \frac{2.15}{1.68} = 1.28$

Impact Factor IM = 33% => $\frac{33}{100} + 1 = 1.33$

HS20-44 Vehicles:

Front Wheel: $18.125 \times 1.33 \times 1.28 = 30.1$ kN

Rear Wheel: $72.5 \times 1.33 \times 1.28 = 123.5$ kN

9.3 kN/m shall be taken as distributed lane load.

- Floor beams Dead Load and Live Load:

Dead Load:

DC and DW from Stringers: 75.5 kN

Bracing Dead Load at centre: 40 kN

Floor beam Self weight: 10 kN/m

Live Load:

Live load on stringer: $\frac{s}{1.68} = \frac{2.15}{1.68} = 1.28$

Impact Factor IM = 33% => $\frac{33}{100} + 1 = 1.33$

HS20-44 Vehicles:

Front Wheel: 18.125×1.33×1.28 = 30.1 kN

Rear Wheel: 72.5×1.33×1.28 = 123.5 kN

Sidewalk: 3.6 kN/m^2 × 3 m = 10.80 kN/m

9.3 kN/m shall be taken as distributed lane load.

- Arch Ribs:

Dead Load:

Arch rib weight: 15 kN/m

Arch bracing weight (Panel Load): 150 kN

Live Load:

Equivalent live load for 6 lanes:

6×9.3×1/2 = 27.9 kN/m

- Ties:

Dead Load:

Tie beam weight: 25 kN/m

Floor beams (Panel Load): 1400 kN

Live Load:

Equivalent live load for 6 lanes:

6×9.3×1/2 = 27.9 kN/m

B.2 Truss Bridge

- Deck Concrete Slab Dead Load:

Asphalt: $0.05\text{m} \times 22\ \text{kN/m}^3 = 1.1\ \text{kN/m}^2$

Concrete Slab: $0.15\ \text{m} \times 24\ \text{kN/m}^3 = 3.6\ \text{kN/m}^2$

Total Dead Load For Slab: $4.7\ \text{kN/m}^2$

- Stringers Dead Load and Live Load:

Dead load:

Assumed Stringer Self Weight: 1.4 kN/m

Slab Weight = DC: $2.15 \times 0.15 \times 24 =$ 7.74 kN/m

Asphalt = DW: $2.15 \times 0.05 \times 22 =$ 2.4 kN/m

Total Dead Load= 11.5 kN/m

Live Load:

Live load on stringer: $\frac{s}{1.68} = \frac{2.15}{1.68} = 1.28$

Impact Factor IM = 33% => $\frac{33}{100} + 1 = 1.33$

HS20-44 Vehicles:

Front Wheel: $18.125 \times 1.33 \times 1.28 = 30.1$ kN

Rear Wheel: $72.5 \times 1.33 \times 1.28 = 123.5$ kN

9.3 kN/m shall be taken as distributed lane load.

- Floor beams Dead Load and Live Load:

Dead Load:

DC and DW from Stringers: 72 kN

Bracing Dead Load at centre: 30 kN

Floor beam Self weight: 8 kN/m

Live Load:

Live load on stringer: $\frac{S}{1.68} = \frac{2.15}{1.68} = 1.28$

Impact Factor IM = 33% => $\frac{33}{100} + 1 = 1.33$

HS20-44 Vehicles:

Front Wheel: 18.125×1.33×1.28 = 30.1 kN

Rear Wheel: 72.5×1.33×1.28 = 123.5 kN

Sidewalk: 3.6 kN/m^2 × 3 m = 10.80 kN/m

9.3 kN/m shall be taken as distributed lane load.

- Portal Frame (At end posts):

Dead Load:

Assumed self-weight: 80 kN

- Sway Frame (At panels):

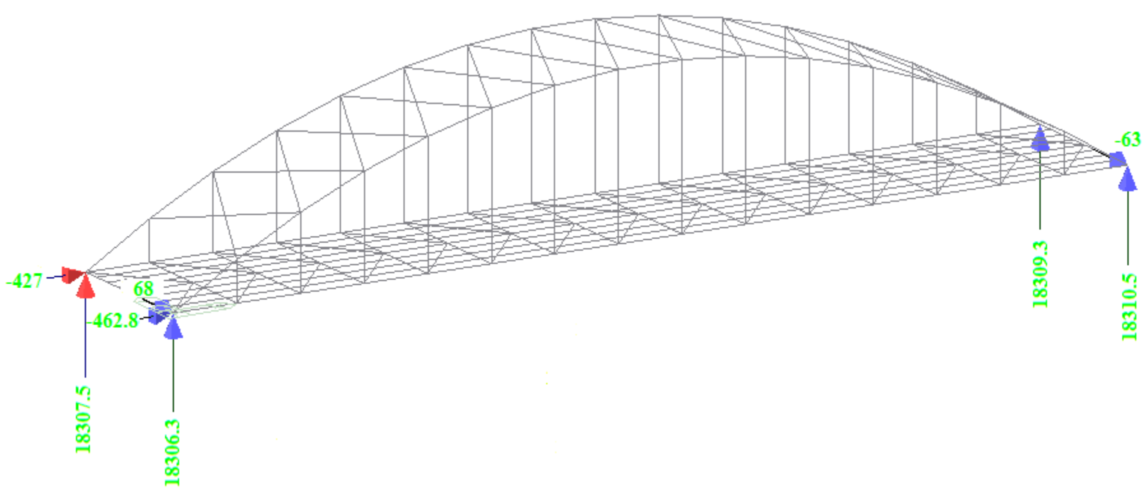
Dead Load:

Assumed self-weight: 20 kN

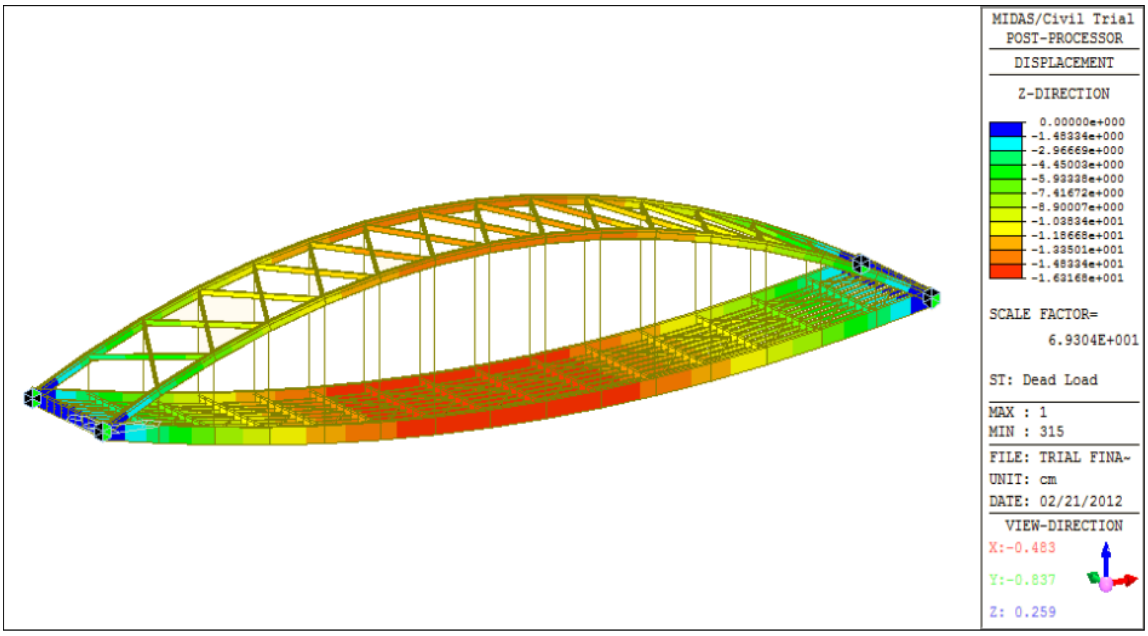
Appendix C: Bridges Analysis Results by MIDAS/Civil

C.1 Long Span Tied-Arch Bridge

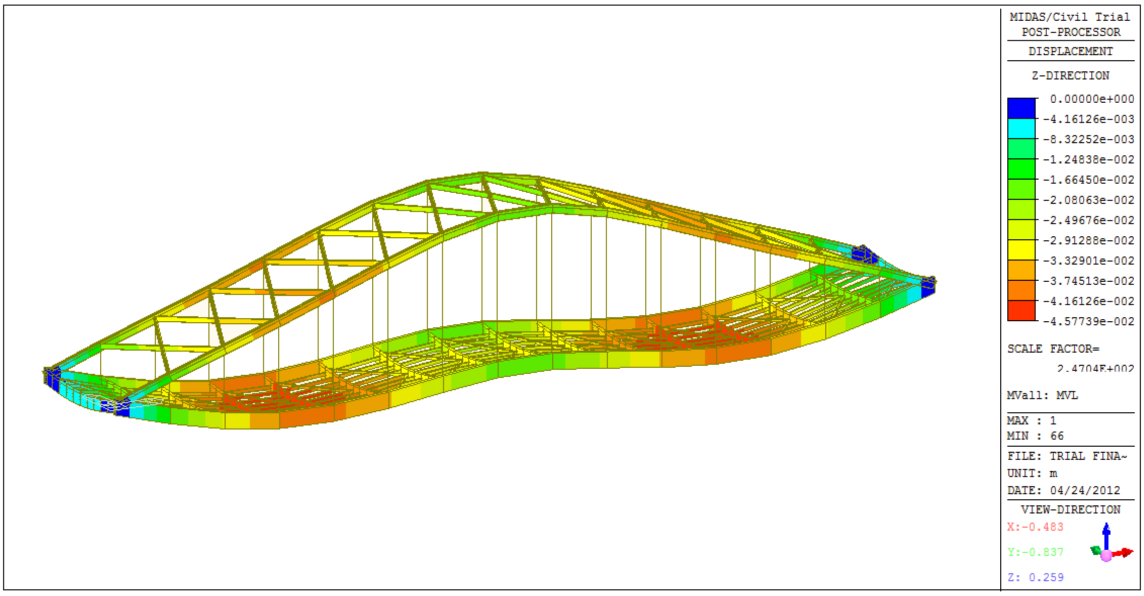
- Support Reaction in X,Y and Z-direction due to dead load, traffic load (Live load) and wind load combination Strength I.



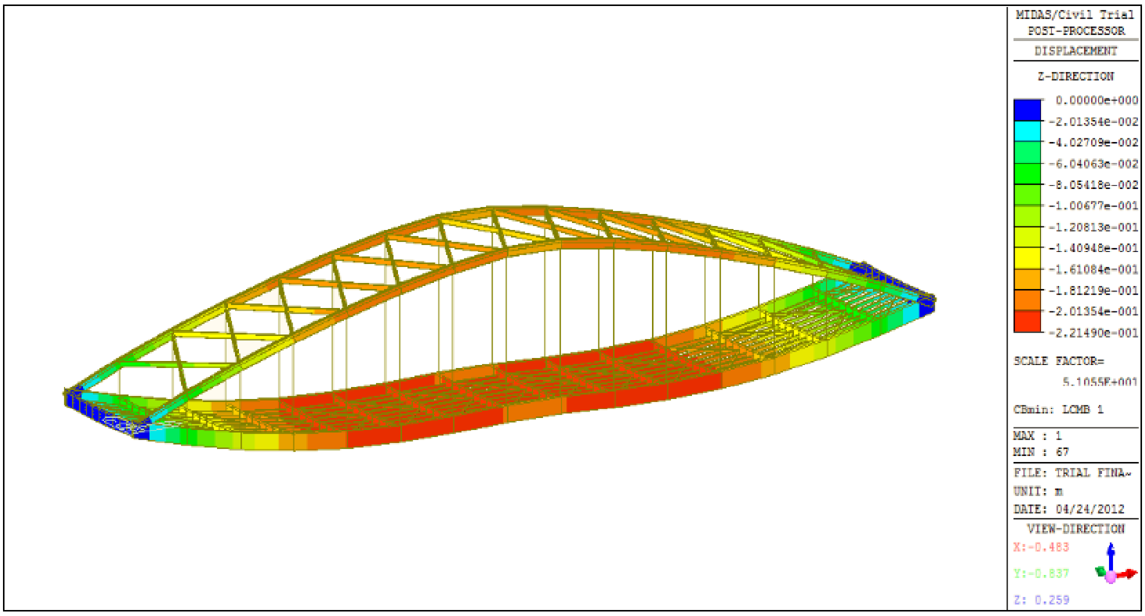
- Vertical deflection due to Dead Load:



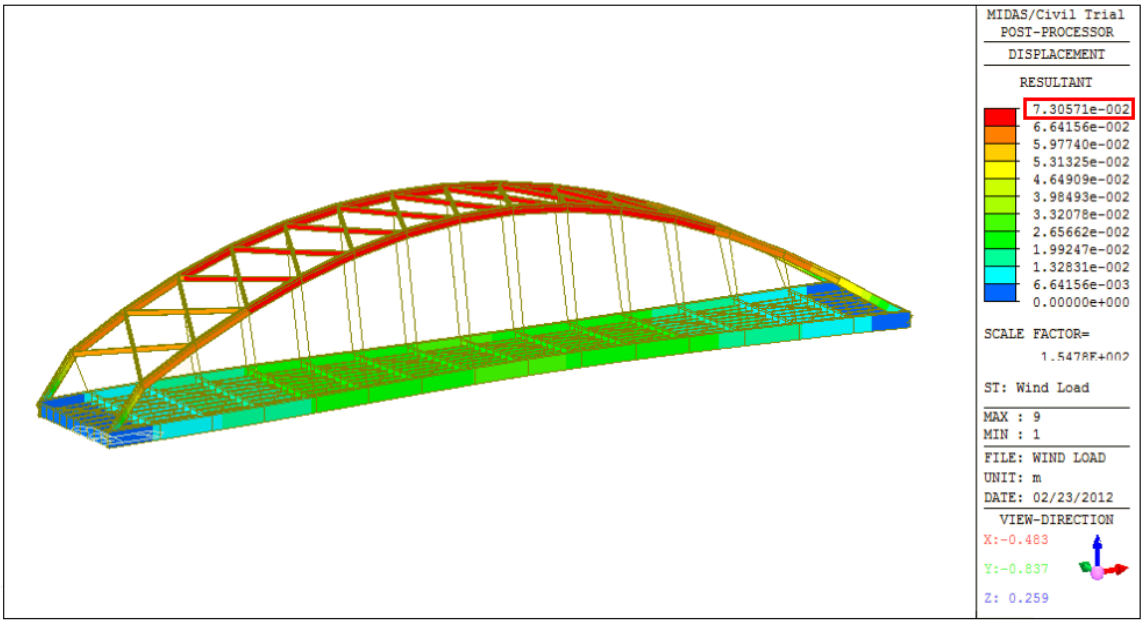
- Vertical deflection due to Traffic Loading.



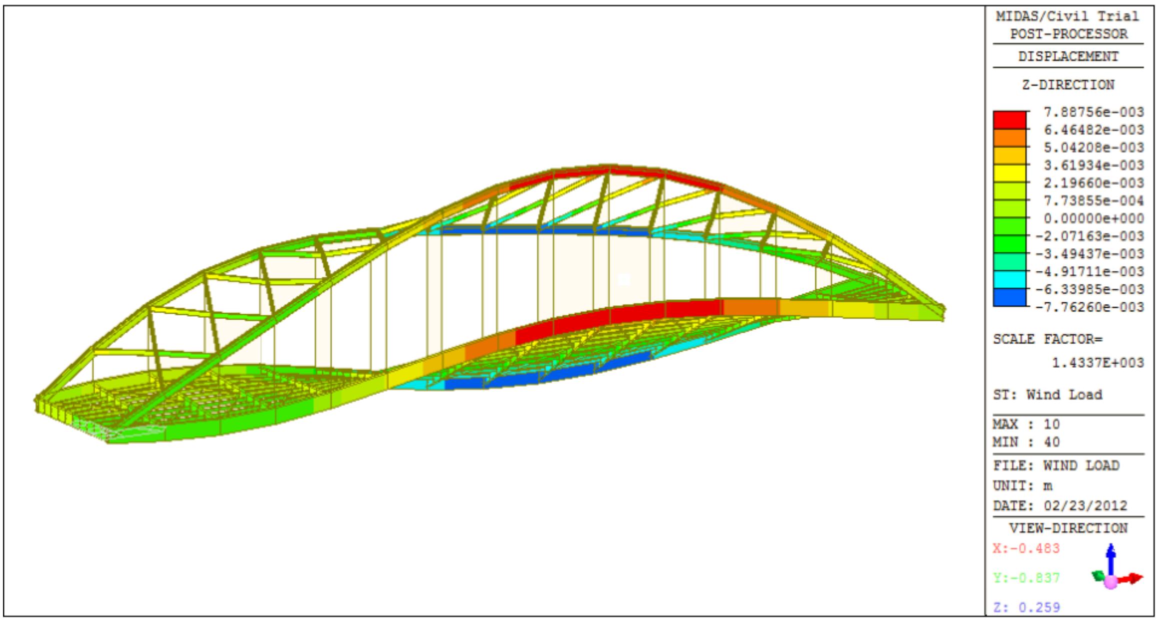
- Vertical deflection due to *DL+LL+IM*



- Horizontal deflection due to wind load.

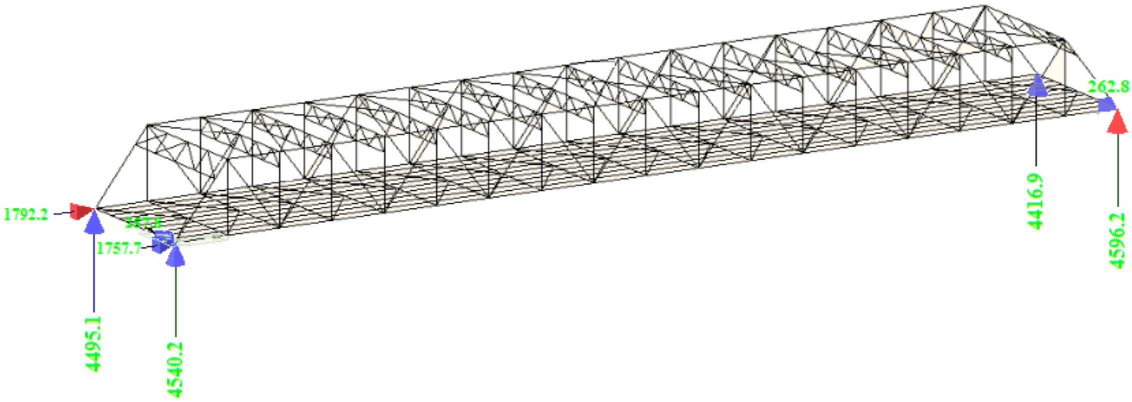


- Vertical deflection due to wind load.

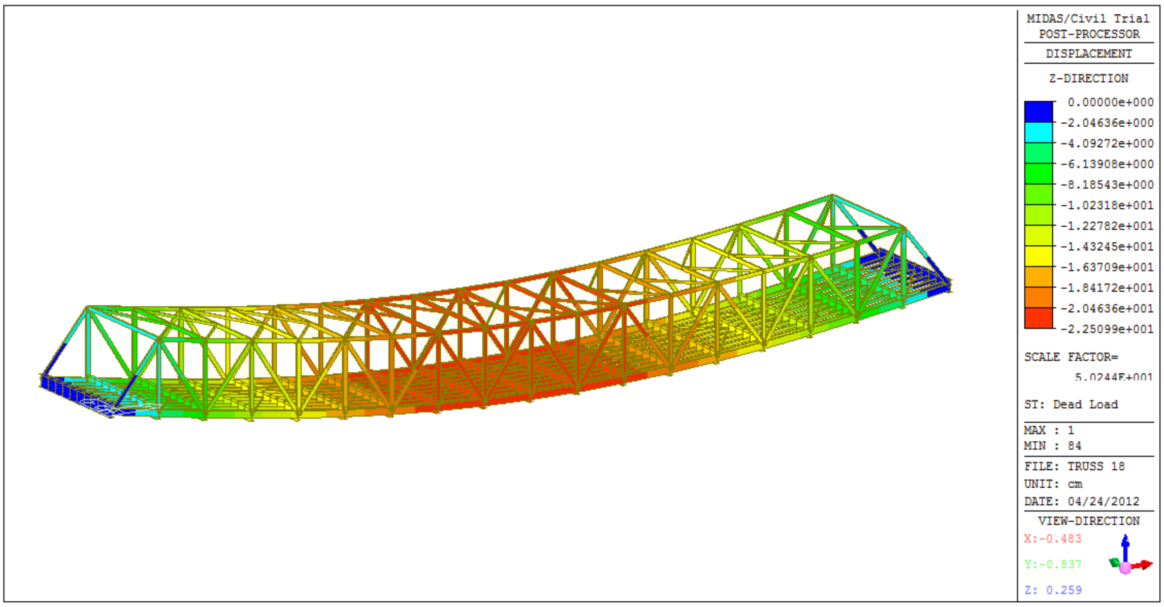


. C.2 Long Span Truss Bridge

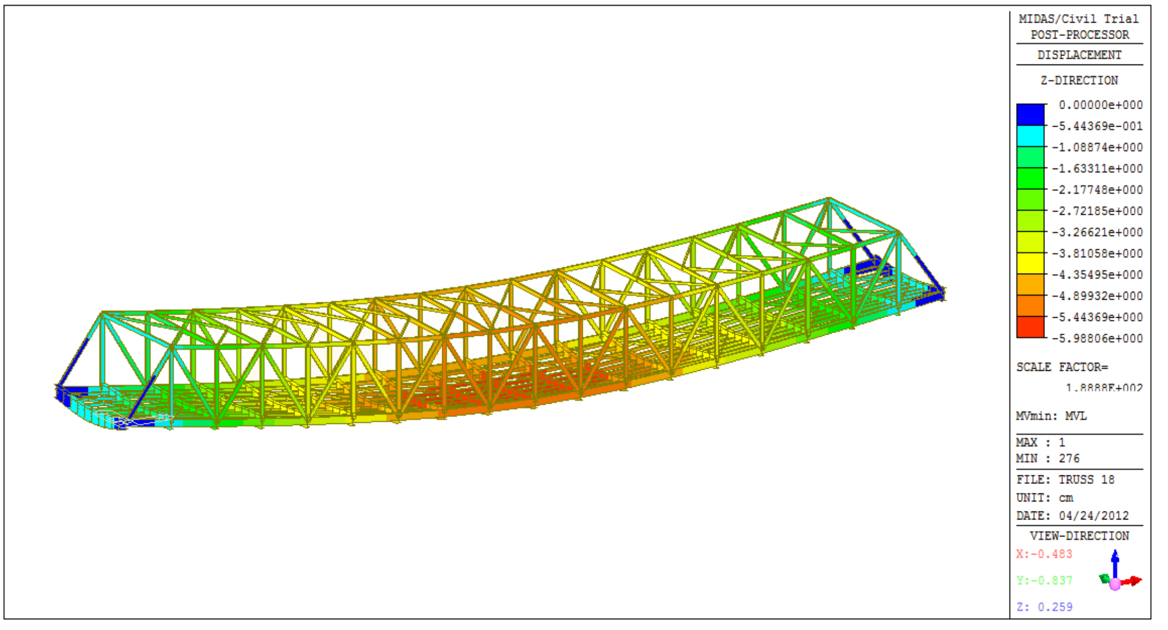
- Support Reaction in X,Y and Z-direction due to dead load, traffic load (Live load) and wind load combination Strength I.



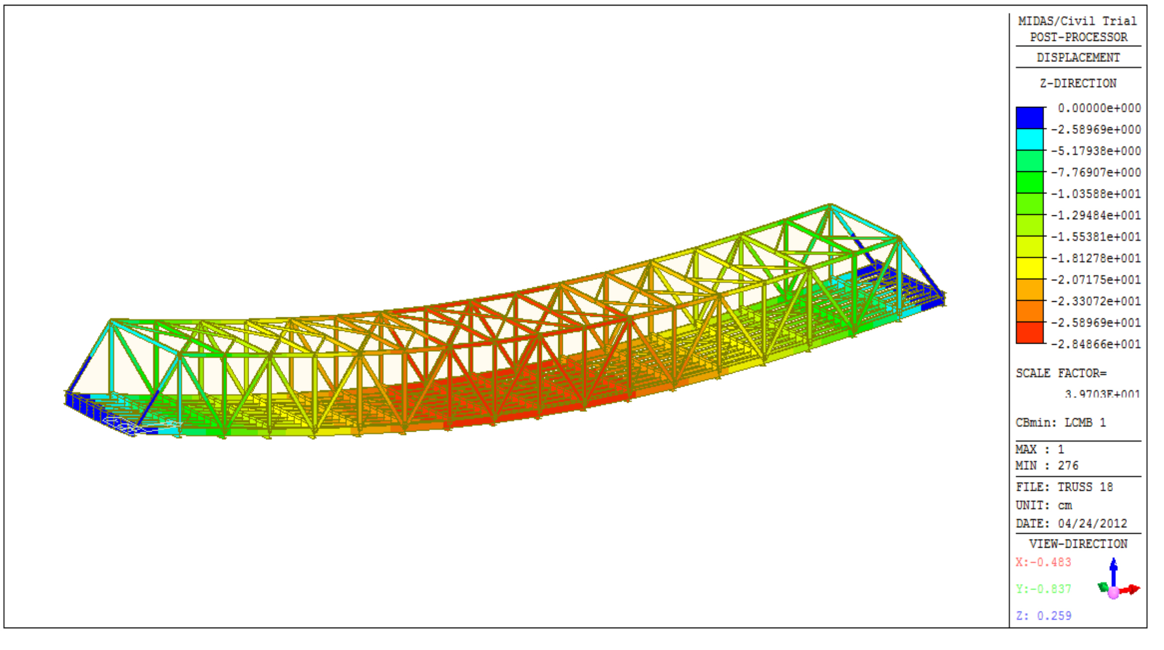
- Vertical deflection due to Dead Load:



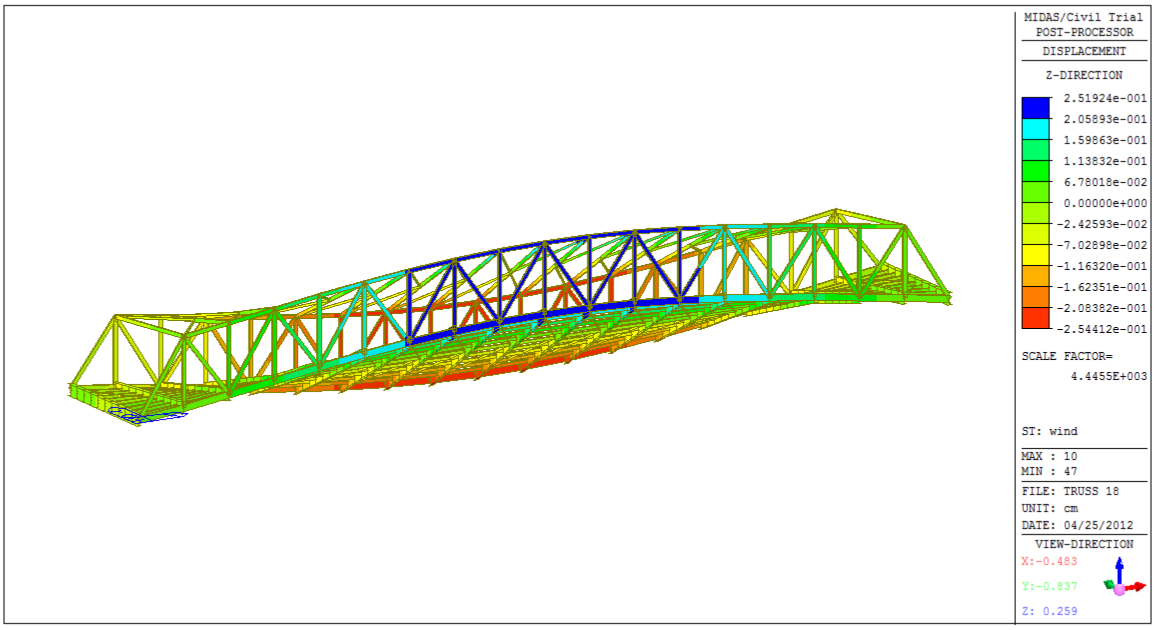
- Vertical deflection due to Traffic Loading.



- Vertical deflection due to *DL+LL+IM*



- Vertical deflection due to wind load.



- Horizontal deflection due to wind load.