Continue

Share copy and redistribute the material in any medium or format for any purpose, even commercially. Adapt remix, transform, and build upon the material for any purpose, even commercially. The licensor cannot revoke these freedoms as long as you follow the license terms. Attribution You must give appropriate credit , provide a link to the license, and indicate if changes were made . You may do so in any reasonable manner, but not in any way that suggests the licensor endorses you or your use. ShareAlike If you remix, transform, or build upon the material, you must distribute your contributions under the same license as the original. No additional restrictions You may not apply legal terms or technological measures that legally restrict others from doing anything the license permits. You do not have to comply with the license for elements of the material in the public domain or where your use is permitted by an applicable exception or limitation . No warranties are given. The license may not give you all of the permissions necessary for your intended use. For example, other rights such as publicity, privacy, or moral rights may limit how you use the material. The United States remains one of the world's fastest growing industrialized nations. Its population is projected to increase to 392 million by 2050 more than a 50-percent increase compared with 1990, according to the U.S. Census Bureau. This growth is expected to create increased demand for pedestrian steel truss bridges for many purposes including, but not limited to, recreational and fitness activities and tourist attractions. The most common use of prefabricated pedestrian steel truss bridges is for trail applications in park and community environments, which is the main focus of this article.The history of prefabricated steel truss bridges dates to the 1930s when modular systems were used to meet the needs of the British military in remote locations. In the 1950s, deck/girder bridges were developed as a replacement for deteriorating timber bridges. Today, truss designs are longer, wider, stronger, and more durable. These designs, combined with technology improvements and manufacturing efficiencies, will continue to support the ever-growing and ever-changing needs of society as they relate to pedestrian, bicycle, and equestrian traffic.This article provides an overall approach to siting and design considerations for prefabricated steel truss pedestrian bridges, but is not intended to be an all-inclusive, step-by-step tutorial on bridge design. A basic understanding of bridge components and layout, including design codes, bridge styles, and loading, is explained in general detail.Bridge SitingThe layout and design of a prefabricated steel truss bridge begins with a site reconnaissance to determine the bridge width and length (also known as span) dimensions. A simple site investigation paying close attention to abutment constraints, flood marks, the size of upstream logs, streambed strata, and drift locations is advised. A geotechnical investigation is recommended. Reference the CE News article titled "Articulated Concrete Block Design," dated August 2010 for more information about scour countermeasures in those suspect locations.Width The most common method to specify the width of a pedestrian truss bridge is the clear width measurement between railing system elements. In the event that this dimension is "fixed," it should be highlighted on the bid plans and also in the specification documents. To control lateral deflections and "sway," the horizontal center- to -center of truss dimension should preferably be no less than 1/20th of the bridge span, but should not except in extreme cases be less than 1/25th of the bridge span.Length The length of the bridge is a function of channel geometry (elevation differences), stream geomorphology, and horizontal and vertical alignments. Typically, the most cost-effective structures are those with horizontal alignments constructed perpendicular to the stream because the result is the shortest bridge length. Bridges with a slight grade are good for surface water runoff. The Americans with Disability Act (ADA) allows for a maximum of 8.33 percent grade (1/12 slope) and 5.0 percent grade (1/20 slope) with and with out a pipe handrail, respectively. Environmental considerations such as wetlands, endangered species, and floating debris may also play a factor in determining the length.The most common method to specify the length of a pedestrian truss bridge is the distance between abutment backwalls. Note that the bridge bearing is typically 3 inches to 9 inches from the abutment backwall and thermal expansion of the bridge is accounted for with slide bearings and a gap derived from the formula below, which assumes 100 degree temperature differential. Base plates with slotted holes on one end of the bridge allow for this thermal movement in the field.Thermal Expansion (inches)=Bridge length (in inches)x0.00065Bridge FunctionalitySeveral attributes and accessories are associated with the overall functionality, aesthetics, and service of the bridge including deck type, steel finish, safety railing systems, camber, toe plates, and fencing.Deck type The deck type selected is a function of the bridge use and has a direct impact on the dead load of the structure. Industry standard deck options include cast-in-place, reinforced concrete, precast planks, open grating, and composite or wood decking.Finish Three main finish options are typically offered for prefabricated steel truss bridges: weathering steel, paint, and galvanized. Each finish has its purpose based on the bridge application and environmental exposure.Safety rail system Either horizontal rails or vertical pickets are the industry standard safety systems. Horizontal rails are used most widely for economical reasons while a more costly vertical picket system is used for aesthetic reasons or, at times, safety concerns as the horizontal safety rails may act as a ladder for children to climb, creating a potential hazard. The specifier needs to consider the setting and usage of the bridge, as well as economics, when determining the appropriate safety system for the project.Camber Camber is put into bridges for two primary reasons: visual effect and/or to increase clearances beneath the structure.The use of camber as a structural "arch" is virtually never considered for prefabricated bridges and should not enter into the decision on how much to camber a bridge. Camber is typically specified in one of four ways or by some combination of the following:A percentage of the dead load deflection. Bridges deemed to be "flat" are usually cambered to offset 150 percent of the full dead load deflection to ensure that there is no sag due to fabrication tolerances, et cetera. Once the bridge is in place, a slight "crown" is hardly noticeable but a small amount of "sag" in the bridge may be visually unappealing.A percentage of the full bridge length (typically 1 to 2-1/2 percent).A specified vertical dimension at bridge midspan.To produce a desired deck slope (typically in conformance with ADA where required).Camber is typically based on a circular curve. There is minimal difference between a circle and a parabola at the typical cambers used in pedestrian structures. Also, vehicle speeds, if any, are low; therefore, the increased cost to detail and fabricate a parabolic arch is unwarranted.Additional accessories and their general use(s) are as follows:Toe plate required on all H-section bridges (Figure 1b) or whenever there is no continuous structural member at or near deck level. Used to keep people from inadvertently stepping off the bridge and to prevent objects from rolling off the bridge.Fencing typically used on overpass structures to ensure pedestrian safety and prevent falling objects.Rubrail typically used for bridge structures on bicycle paths. Mechanically attached at handlebar height of 42 inches from top of deck per AASHTO "Guide for the Development of Bicycle Facilities."Bridge DesignThe design of prefabricated steel truss pedestrian bridges is based on the siting and functionality factors previously discussed combined with the loading conditions wind, dead, live, fatigue, snow, seismic, and stream force required for the bridge. Seismic and stream load forces are key determinations that should be addressed by the specifying engineer during the specification phase and are discussed briefly in this article.Truss type Often, the aesthetics of a particular truss type is the driving factor for the selection process. As previously mentioned, prefabricated truss bridges went into use in the 1930s, but the design and use of truss bridges dates to 1820 when the Town's Lattice Truss was patented, which is one of only 27 truss types in use globally. Today, one of the most common styles is the Pratt truss, patented in 1844, which has vertical members and diagonals sloping to the center, resulting in a statically determinate structure that is ideal for long spans. This section focuses on Pratt truss design considerations that include four basic cross section details as shown in Figure 1. The following is a brief discussion of typical scenarios where each of the common bridge types could be utilized most efficiently. These scenarios consider shipping and fabrication costs as well as efficient structural use of materials, recognizing situations where geometry is a concern.A Pratt truss with an underhung floor beam(shown in Figure 1a) is typically most cost-effective on relatively short-span bridges (up to about 50 feet in length) or on some longer spans where belowdeck clearance is an issue. However, their economical use depends on the bridge loading and deflection requirements. Heavy loads such as large vehicles can affect the decision on when to use a Pratt Truss with underhung floor beams, which may not be the most efficient bridge type to use with heavy concentrated floor loads.H-Sections(shown in Figure 1b) are typically most efficient on medium- to long-span structures (50 feet to 240 feet) under one or more of the following situations:below-deck clearance is not an issue;on heavily loaded structures;bridges with large vehicle loads or stringent deflection or vibration limitations;where a maximum handrail height is a limitation and a deep truss is required structurally;where no overhead structural elements are a desired condition.Through or box trussesare used on relatively long spans (100 feet to 250 feet) where below-deck clearance is an issue, where enclosing the structure with glazing or fencing is desired, or where the structure requires roofing.The economical determination of the truss depth (center-line to center-line of chord dimension) depends on a number of factors: guardrail height, chord stress levels, shipping considerations, and lateral support of the top chord. As a general rule, the truss depth should be no less than 1/20th of the bridge span but except in extreme cases shall not be less than 1/25th of the bridge span. These ratios, through usage, have proven economical in maintaining reasonable truss deflection limits (1/400 of the span length) and vibrations; however, they may not be adequate if more stringent deflection requirements or if vibration or frequency limits are a concern.Design code Two main design codes are used to govern prefabricated steel truss pedestrian bridge types:"AASHTO Guide Specification for the Design of Pedestrian Bridges," published by the American Association of State Highway and Transportation Officials (AASHTO);"International Building Code" (IBC) for design loads in conjunction with the specification for structural steel buildings published by the American Institute of Steel Construction (AISC) for member and connection design.In general, AASHTO Guide Specifications for the Design of Pedestrian Bridges is referenced most commonly on projects where state and/or federal funds are allocated to the bridge construction.Dead load Unless otherwise specified, the dead load used in the design combinations shall only be the dead weight of the superstructure and the weight of the original decking material used with no future overlays considered. These weights shall be determined by the bridge manufacturer; however, in the case of deck materials by others (usually asphalt or concrete), the specifier must indicate the weight of the decking material (typically the unit weight of normal or lightweight concrete) and the approximate deck depth. Bear in mind that as the bridge manufacturer is usually responsible for the deck design and that the deck thickness is a function of final deck support member spacing, the bridge manufacturer must be allowed to modify the depth of concrete decks and therefore the dead load to fit individual bridge scenarios.Live loads Two live loads are applicable for truss design: pedestrian and light vehicular.Pedestrian live load The typical pedestrian traffic live load is 85 pounds per square foot (psf). This represents an average person occupying 2 square feet of bridge deck area and is considered a reasonable, conservative service live load that is difficult to exceed with pedestrian traffic. In applications where pedestrian traffic is light (trail bridges, walkways, et cetera), live loads as low as 60 psf (see IBC Table 1607.1 (40)) have been used with approval of the owner. This reduced live loading is not acceptable for entrance or exit ways or in locations where a large number of people are expected to congregate.Reduction of live loads for deck influence areas exceeding 400 square feet is consistent with the provisions of ASCE 7, "Minimum Design Loads for Buildings and Other Structures," and is intended to account for the reduced probability of large influence areas being simultaneously loaded to the maximum design value. Table 1: Minimum recommended design vehicle loadsBridge Width (w)Two-wheel vehicle (lbs)*Four-wheel vehicle (lbs)*w < 4'-0"1,200NA4'-0" = w = 6'-5"1,2004,0006'-6" = w = 8'-5"1,2006,0008'-6" = w = 10'-5"1,2008,000w = 10'-6"1,20010,000* Weights are minimum recommendations only; owner-specified or site-specific vehicles may be larger. Vehicular live load The proposed vehicle loads listed in Table 1 are intended as minimum default values in cases where no other vehicle load is specified. Table 1 provides a recommendation of design vehicles that could commonly access a pedestrian or bicycle structure (golf carts, garden tractors, riding lawnmowers, utility vehicles, ATVs, snowmobiles, cars, pick-up trucks, vans, sport utility vehicles, and motorcycles).Because smaller vehicles have tighter axle and wheel spacing possibly causing their lighter wheel loads to be located nearer the midspan of a decking member all vehicles up to and including the largest vehicle selected for design must be used in determining the critical decking member stresses. Loaded skid-steer loaders, emergency vehicles, construction equipment, and heavy trucks are all special load cases and shall be considered when there is any reason or likelihood that they could obtain access to the bridge structure. The specifier or "operating agency" shall indicate the maximum vehicle load requirements of the structure.On all structures, including those where vehicle access is restricted from the bridge either by stairs or bollards at each end of the structure, the bridge and floor system typically are designed for a minimum concentrated load of 1,000 pounds placed on any area 2-1/2 feet by 2-1/2 feet square. Recognizing the fact that vehicle loadings on these structure types are usually infrequent and travel speeds are low (normally less than 10 mph), an impact allowance to the vehicle load is usually deemed unnecessary.Seismic loads The calculation of seismic forces in areas subject to earthquakes should be as set forth in the applicable design code or specification (typically IBC/ASCE 7 or AASHTO). For states with high seismic activity, such as California, it is common to be required to meet the seismic requirements of local or state agencies (i.e., CALTRANS or the California Building Code). A geotechnical investigation to determine relevant site conditions is recommended for all bridges that may encounter high seismic forces so that an appropriate seismic evaluation can be accomplished.Stream force loads If the bridge is partially or completely submerged during future rain events, the specifier shall state the high water elevation, the average water velocity, and the bridge elevation in the contract documents. The stream force load is consistent with the "AASHTO Standard Specifications for Highway Bridges, Section 3.18.1.1" as it pertains to superstructure loads. When applicable, the specifier shall determine both the maximum stream flow velocity at flood stage and the height of flood waters. Once determined, the bridge can be designed for stream forces as described below or the length of the structure can be increased by raising the elevation of the bridge such that it is above the high water flood elevation.The bridge designer will design the bridge accordingly, based on one of the following four methods:design for no stream load on the superstructure;design for uniform stream force load on the superstructure, which will typically keep the bridge in place during a flood, but not necessarily usable;design for uniform streamforce load and debris impact loads;design the bridge anchorage as a "break-away" system, which is typically done to prevent the bridge from acting as a dam.ConclusionThe prefabricated steel truss pedestrian bridge industry is mature with thousands of successful structures nationwide. Proper siting and design of a prefabricated steel truss pedestrian bridge requires a comprehensive understanding of many variables as explained. This article identified a number of relevant factors; however, designers may contact a bridge manufacturer to assist with the specification process to ensure proper design for the project site in question.REFERENCESAASHTO Guide Specifications for Design of Pedestrian Bridges, American Association of State Highway and Transportation Officials.Guide for the Development of Bicycle Facilities, American Association of State Highway and Transportation Officials.Standard Specifications for Highway Bridges, 16th edition, American Association of State Highway and Transportation Officials.ASCE Standard 7-95, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers.2006 International Building Code.Prefabricated Bridge Elements and Systems, U.S. Department of Transportation, Federal Highway Administration, www.fhwa.dot.gov/bridge/prefab/psbsreport03.cfm.Truss BridgeThis kind of bridge is called the Truss Bridge and is commonly used when iron and other metals were scarce, and wood was abundant. This kind of bridge is suitable for long spans. This bridge is considered one of the best designs for railroad bridges.Most Truss Bridges use the strength of triangles and spreads the force out across the bridge.All forces are spread out through the triangle formation due to equal side lengths and angles, The forces of compression and tension along the sides of the triangles balance out equally in order to carry the load.Although this bridge is generally considered the best, most of the Truss Bridge types use some kind of metal for more support.Unfortunately we do not have access to metal material, which may pose to be a problem when we try to build Truss Bridges. Furthermore, more modern kinds of bridges using different types of materials put this bridge into the shadow. Simulator for K Truss Design:Using a bridge simulator, our team created the K Truss Design and tested it against 100N. The result is a bridge that survives the test and able to withstand more weight.The numbers represented on each member is the percentage of the load that will be forced onto the bridge. In this bridge, the highest percentage will be 75% near the center of the load. This means that the section is distributed good and will not take all the stress by itself. Home Products & Services Bridges & Steel Structures Truss bridges Truss bridges, which do not have the redundant web plate portion that girder bridges do, are an extremely rational, efficient bridge type adopting a structure made up of a combination of thin members that convey only axial force. Some types of truss bridges are named after their patent holder, such as the Howe truss, the Pratt truss, the Fink truss, and the Bollman truss. Today, the Warren truss is most widely adopted. The applied span of truss bridges is generally 50 to 110 meters, and the Quebec Bridge (Canada; 1917) is the longest truss bridge in the world, with a span length of 549 meters. JFE Engineering has great experience in manufacturing and erecting truss bridges. Name Myoko Bridge Year completed 2021 Bridge type Truss bridge Client Hokuriku Regional Development Bureau, Ministry of Land, Infrastructure and Transport Bridge length 203m Features The bridge was constructed using a combination of crawler crane pile venting and traveler crane overhang methods in an environment where the bridge crosses a steep 50m deep valley at the intersection with the Otagiri River. Name Sazanamigawa Bridge Year completed 2011 Bridge type Truss bridge Client Ministry of Land, Infrastructure, Transport and Tourism Hokuriku Regional Development Bureau Bridge length 159m Features 2-span continuous deck type truss girder of non-uniform section structure Name Tsukuba Express Tonegawa Bridge Year completed 2003 Bridge type Through type 3-span continuous double-track truss bridge, etc. Client Japan Railway Construction Public Corporation Bridge length 897m Features Railway bridge Name Sekihoku Bridge Year completed 2001 Bridge type Deck type 3-span continuous truss bridge Client Hokkaido Regional Development Bureau Bridge length 212m Features Weather-resistant bridge Name Kyono Bridge Year completed 1994 Bridge type Triple through type 2-span continuous truss bridge Client Tochigi Prefecture Bridge length 388m Name Sky Gate Bridge (connected to Kansai International Airport) Year completed 1994 Bridge type Sextuple 3-span continuous truss bridge, etc. Client Kansai International Airport Land Company, Ltd. Bridge length 3,750m Features Double-decked truss bridge (road-railway bridge) Name Sogaura Bridge Year completed 1990 Bridge type Deck type simple truss bridge Client Shizuoka Prefecture Bridge length 225m Name Yoshima Bridge Year completed 1988 Bridge type 3-span continuous truss bridge Client Honshu-Shikoku Bridge Authority Bridge length 585m Features 780 N/mm2 grade high tensile steel used Double-decked truss bridge (road-railway bridge) Name Oshima Bridge Year completed 1975 Bridge type Through type 3-span continuous truss bridge, etc. Client Japan Highway Public Corporation Bridge length 1,020m We are independent & ad-supported. We may earn a commission for purchases made through our links. Advertiser Disclosure Our website is an independent, advertising-supported platform. We provide our content free of charge to our readers, and to keep it that way, we rely on revenue generated through advertisements and affiliate partnerships. This means that when you click on certain links on our site and make a purchase, we may earn a commission. Learn more. How We Make Money We sustain our operations through affiliate commissions and advertising. If you click on an affiliate link and make a purchase, we may receive a commission from the merchant at no additional cost to you. We also display advertisements on our website, which help generate revenue to support our work and keep our content free for readers. Our editorial team operates independently of our advertising and affiliate partnerships to ensure that our content remains unbiased and focused on providing you with the best information and recommendations based on thorough research and honest evaluations. To remain transparent, weve provided a list of our current affiliate partners here. All The Science, in your inbox Our latest articles, guides, and more, delivered daily. By Mary Elizabeth Updated Jun05,2024 By Michael Pollick Updated May21,2024 By C.H. Seman Updated May21,2024 Explore fresh perspectives Popular picks Genetic engineering is manipulation of cell DNA. Though many people question the ethics, genetic engineering is commonly used for... Solar power is energy derived from the sun and converted into heat or electricity. Versatile and renewable, solar power can... All The Science, in your inbox Our latest articles, guides, and more, delivered daily. Privacy Policy Terms and Conditions All The Science, in your inbox Share copy and redistribute the material in any medium or format for any purpose, even commercially. Adapt remix, transform, and build upon the material for any purpose, even commercially. The licensor cannot revoke these freedoms as long as you follow the license terms. Attribution You must give appropriate credit , provide a link to the license, and indicate if changes were made . You may do so in any reasonable manner, but not in any way that suggests the licensor endorses you or your use. ShareAlike If you remix, transform, or build upon the material, you must distribute your contributions under the same license as the original. No additional restrictions You may not apply legal terms or technological measures that legally restrict others from doing anything the license permits. You do not have to comply with the license for elements of the material in the public domain or where your use is permitted by an applicable exception or limitation . No warranties are given. The license may not give you all of the permissions necessary for your intended use. For example, other rights such as publicity, privacy, or moral rights may limit how you use the material. Previous | Next | Contents ESDEP WG 15B STRUCTURAL SYSTEMS: BRIDGES OBJECTIVE/SCOPE This lecture gives information about the design and detailing of truss bridges. It is intended for engineers with some experience. PREREQUISITES Lecture 15B.2:Actions on Bridges RELATED LECTURES Lecture 15B.1:Conceptual Choice Lecture 15B.3:Bridge Decks SUMMARY The history of truss bridges is reviewed and different configurations are described. Design principles are presented, e.g. span ranges, span-to-depth ratios and arrangement of diagonals. Different sections for chords and diagonals are shown and discussed together with the connections between the members. Analysis of trusses is treated in a general way and recommendations are given on what has to be considered and what can be neglected. 1.INTRODUCTION Typical configurations of truss beam bridges are shown in Figure 1. Underslung trusses are rarely used in modern construction. Through and semi-through truss bridges are used when the depth of deck construction is very limited, for instance, when a highway or a railway crosses a canal. It is unusual for through-trusses to be economic for highway bridges, except for very long spans. With less severe restrictions on the gradients on the approach embankments it is, in addition, much easier for a road to gain the height required for a deck bridge than for a railway to do so. Semi-through trusses tend therefore to be used for highways while through and semi-through trusses are still used for railways. The principle of a truss is simple. The structure is composed of top and bottom chords triangulated with diagonals and/or verticals in the webs so that each member carries purely axial load. Additional effects do exist but in a well designed truss they will be of a secondary nature and may be neglected. Global moment on a truss is carried as compression and tension in the chords as shown in Figure 2a. Global shear is carried as tension or compression in the diagonal and vertical members. In the simplified case, where the joints are considered as pinned, and the loads are applied at the nodes, the loading creates no bending moment, shear, or torsion in any member. Loads applied in such a way as to cause bending, shear, or torsion usually result in inefficient use of material. The saving of material compared to a plate girder is clear when the webs are considered. In a truss the webs are mainly 'fresh air' - hence less weight and less wind pressure. A truss can be assembled from small easily handled and transported pieces, and the site connections can all be bolted. Trusses can have a particular advantage in countries where access to the site is difficult or supply of skilled labour is limited. Undamaged parts of a truss bridge can easily be re-used after an accident or the effects of war. 2.DIFFERENT TYPES OF TRUSS 2.1Historical Background The truss as a structural form dates back to Roman times. A bronze truss was used in the Pantheon. In the nineteenth century, the United States can claim to have created the greatest number of different types of truss. Their use of timber and their enthusiastic pioneering spirit created some unlikely looking structures, but nevertheless firmly established the truss as the ideal form of bridge at that time for medium spans. Eiffel built lattice trusses in France (Figure 2f). Fowler and Baker, however, introduced a major innovation by adopting steel tubular sections as the main compression members for the Forth Bridge which is well known throughout the world for its grandeur. Modern truss bridges also use box sections for the compression members. The Hungarian architect Virgil Nagy built the very aesthetic Ferenc Jozsef truss girder bridge in Budapest over the Danube in 1892. The bridge is supported by Pratt-truss girders of variable height (Figure 2e). The central span is 175m long with an isostatic central part of 47m. For most modern bridgework the Warren truss (with its modifications) is perhaps the most commonly used type because of its simplicity. Modern labour costs dictate a minimum of members and connections. 2.2Highway Truss Bridges The Warren configuration shown in Figure 2 is usually chosen. When the length of the gap to be crossed makes the use of a multiple span bridge unavoidable, it is cheaper and usually possible to raise the road line and build another type of bridge requiring a greater depth under the deck. For this reason highway truss bridges usually have only one span (Figure 3). Their appearance is well adapted aesthetically to cross canals in flat landscapes. The spans are usually between 60 and 120m which is the normal economic range. The longest span was the old Neuwied bridge over the Rhine (212m) which was replaced by a cable stayed bridge. The span-to-depth ratio is normally about 15. 2.3Choice of Truss Configuration For Railway Bridges Three basic truss bridge configurations are shown in Figure 1. The most economic truss bridge configuration, especially for railway bridges, is the underslung truss where the live load runs at the level of the top chord. The top chord then serves the dual function of support for the live load (as the sleepers sit directly on the chord) and the main compression member. There is, however, the disadvantage that clearance under the bridge is reduced. It is thus common for the approach spans over a flood plain, or over unnavigable parts of the river, to be underslung while the navigation channels are crossed with through trusses. Where the spans are short, and underslung trusses are not possible, it may be economic to have the top chord below the loading gauge level by using semi-through trusses. Bracing between the top chords is not possible and restraint to the compression members has to be provided by U-frames. However for spans where semi-through trusses have been used in the past, plate girder bridges are now very competitive, and now semi-through trusses are seldom used for railway bridges. Where the spans of railway bridges are long the economic depth is usually great enough to allow bracing to be provided above the loading gauge level. Such trusses are termed 'through trusses'. The use of material in bracing rather than U-frames is considerably more efficient. For shorter spans the choice is between the Warren and the Pratt configuration. In the simple Warren truss, the diagonals work alternatively in compression and tension, whereas in the Pratt truss, all the diagonals are in tension and the shorter posts take compression. To cater for the heavy loading on railway bridges, the cross girders should be fairly close together. This requirement leads to the hangers of the Modified Warren truss which sub-divide the bottom chord. Economic design of the top compression chord leads to sub-division with a post. The majority of truss bridges are simple spans, but there are many examples of continuous trusses. The immediate benefit on member forces when a continuous structure is employed is offset to some extent by increased fatigue effects. In a simple truss it is common for only some of the diagonals to be influenced by fatigue. These diagonals are usually those at mid-span where the smallest available section has to be used in any case. In contrast, most of the diagonals in a continuous truss, and some of the chord members may well have to be checked for fatigue, particularly when welded construction is used. Even where continuous trusses show savings in the use of steel, they may not be economic. On a 1700m long bridge in India, the alternative continuous truss design was about 5% lighter that the simple spans which were considered more economic on account of standardization of fabrication detail and erection procedure. It should be noted here that the design loading has a considerable effect on the truss configuration. For example, with combined highway and rail loading, trusses with two decks can be very economic. 2.4Particular Applications As dead load is a dominating factor for movable bridges, bascule spans are often built using steel truss girders. Figure 4a shows an example of a rear part made for a movable truss bridge. Most connections are butt-welded and governed by fatigue considerations. This kind of bridge will not be further discussed here. For more information, see Lecture 1B.6.2. Temporary bridges for emergency purposes are almost always truss bridges because of their adaptability to various spans and support conditions, e.g. Eiffel, Bailey, Arromanches, Callender-Hamilton, see Figure 4b. 3.GENERAL DESIGN PRINCIPLES 3.1Span Range For spans from 60m to 120m for highways and from 30m to 150m for railways, simple spans can prove economic when favourable conditions exist. Large spans using cantilever trusses have reached a main span of 550m. Trusses have to compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for longer spans. 3.2Ratio of Span to Depth The optimum value for this ratio depends on the magnitude of the live load that has to be carried. It should be in the region of 10, being greater for road traffic than for rail traffic. For twin track rail loading the ratio may fall to about 7,5. A check should always be made on the economic depth for a given bridge. 3.3Geometry For short and medium spans, it will generally be found economic to use parallel chords to keep fabrication and erection costs down. However, for long continuous spans, a greater depth is often required at the piers, Figure 2e. Skew truss bridges should be avoided as far as possible. An even number of bays should be chosen to suit the configuration of diagonals in Pratt and modified Warren trusses. If an odd number is chosen there will be a central bay with crossed diagonals. This arrangement is not usually desirable except perhaps at the centre of a swing bridge. The diagonals should be at an angle between 50 and 60 to the horizontal. Secondary stresses should be avoided as far as possible by ensuring that the neutral axes of all intersecting members meet at a single point, in both vertical and horizontal planes. This will not always be possible, e.g. cross girders will be deeper than the bottom chord and bracing members may be attached to only one flange of the chords. 3.4Grade of Steel Grade S355 steel should be used for the main members with Grade S275 or S235 used only for members carrying insignificant load, unless the truss has to be fabricated in a country where there is no ready supply of higher grade steel. For a truss designed using Grade S355 steel, the amount of Grade S275 or S235 steel used would normally be about 7%. For very long spans higher grades will be economical, e.g. quenched and tempered steel or thermo-mechanically processed steel with yield strength 500 - 600 MPa, provided that fatigue is not governing. 3.5Compression Chord Members These members should be kept as short as possible and consideration given to additional bracing if economical. The effective length for buckling in the plane of the truss is normally not the same as that for buckling out of the plane of the truss. This effect can be further complicated in through trusses where horizontal bracing may be provided at mid panel points as well as at the main nodes. When making up the section for the compression chord, the ideal disposition of material will be one that produces a section with radii of gyration such that the ratio of effective length to radius of gyration is the same in both planes. In other words, the member is just as likely to buckle horizontally as vertically. Eurocode 3: Part 1.1 [1] permits the effective length factors for truss members to be determined by analysis. Otherwise very conservative values are given of 1,0 and 0,9. However, as Eurocode 3: Part 1 applies to buildings, which have relatively small span trusses where absolute economy in steel weight is not vital, it is assumed that the clause is not appropriate to bridges. It is anticipated that the effective length of bridge truss members will be covered in Part 2 of Eurocode 3 [2]. As an example of current practice see Table II of BS5400 Part 3 [3]. In the case of semi-through bridges, the top chord is supported laterally by the diagonals and behaves as a strut supported on springs. The method of determination of its effective length is given in the appropriate bridge codes. The depth of the member needs to be chosen so that plate dimensions are sensible. If they are too thick, the radius of gyration will be smaller than it would be if the same area of steel was used to form a larger member using thinner plates. The plates should be as thin as possible without losing too much area when the effective section is derived. Trusses with spans up to about 100m often have open section chords, usually of "top-hat" section, see Figure 5. Here it is often desirable to arrange for the vertical posts and struts to enter inside the top chord member, thereby providing a natural diaphragm and also, usually, avoiding the need for gussets at alternate nodes, although packs will be needed. For trusses with spans greater than about 100m, the chords will usually be box shaped so allowing the ideal disposition of material to be made from both economic and maintenance viewpoints. For shorter spans rolled sections or rolled hollow sections may occasionally be used. Advantages and disadvantages and comments on fabrication of the five alternative configurations shown in Figure 5 are: a. Top Hat (i) Welding distortion may be a problem although the situation at the bottom flange may be improved by adding a sealing fillet. This provision is recommended to avoid corrosion. Welds need to be ground flush at gusset positions. Battens or lacing are required as local bracings. b. Top Hat (ii) More suitable for automatic welding than Top Hat (i). Requires machining at nodes to allow entry of verticals and gussets. For fatigue reasons keep toes of fillets at least 10mm from edges of plates. Battens or lacing required. Keep outstand of bottom flange large enough to permit direct attachment of top lateral system. c. Box Provides optimum buckling strength. Provides clean profile and easy maintenance. No lacing required. Access for installation of internal diaphragms is difficult. Additional gussets required for attachment of top laterals. d. Rolled Sections Bad for trapping dirt and debris. Packing required at joints as nominal section depths vary slightly. e. Rolled Hollow Sections Crevices are formed at gussets unless special precautions are taken. 3.6Tension Chord Members Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions. The width out of the plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing. It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices. This means that fracture at the net section will not govern for common steel grades. As with compression members, box sections would be preferable for ease of maintenance but open sections may well prove cheaper. Four alternative configurations are shown in Figure 6. Their advantages and disadvantages are: a. Open Box Welding distortion may be a problem but could be improved by adding sealing fillets at the corners. Welds need to be ground flush at gusset positions. Battens or lacing required. b. Box Provides clean profile and easy maintenance. No bottom plates required. Access for installation of internal diaphragms is difficult. c. Rolled Section Bad for trapping dirt and debris. Packing required at joints. d. Rolled Hollow Section Crevices formed at gussets unless special precautions are taken. 3.7Vertical and Diagonal Members These members should be all the same width normal to the plane of the truss to permit them to fit flush with or to be slotted inside the top chord (where the top-hat section is used) and to fit flush with the bottom chord. However, the width of the diagonals in the plane of the truss should be reduced away from the supports by about 75mm per panel. This reduction may mean that some members are understressed. It is often possible to use rolled sections, particularly for the lightly loaded members, but packs will probably be required to take up the rolling margins. This fact can make welded members more economic, particularly on the longer trusses where the packing operation might add a significant amount to the erection cost. Aesthetically, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel. This arrangement prevents the truss looking over-complex when viewed from an angle. In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred. Five alternative configurations are shown in Figure 7. Their advantages and disadvantages are: a. Open Box (i) The partial penetration welds are expensive and alternative (ii) might be cheaper. Battens or lacing required. b. Open Box (ii) Continuous or intermittent fillet welds can be laid simultaneously. Intermittent welds should only be used in bridges when corrosion is not a significant problem. Battens or lacing required. c. Made-up I Not recommended for end diagonals as it acts as a water chute down to the bearings. d. Rolled Sections Packing required at joints as nominal section depths vary slightly. e. Rolled Hollow Sections Crevices form at gussets unless special precautions are taken. 3.8Maintenance As with any structural design, the problems that may confront the maintenance team should be fully appreciated. The problems can be numerous, but a good design will avoid most of the common difficulties. For example: Water Try to keep water out but always assume it is going to get in and provide a way for it to escape when it does. "Sealed" sections should be provided with a drainage hole at the lowest point. Dirt and Debris Try to keep dirt and debris out, remembering that wind and rain will bring them in. Painting Remember that, if access is difficult, the bridge will not be painted or at least only badly, and probably not inspected. Box sections make painting easier, but rolled hollow sections leave nasty crevices at gusset positions, unless the joints are welded. Birds Birds will nest and roost in the most unlikely places! 4.LATERAL BRACING Unless an orthotropic or concrete deck is provided, stringer bracing, braking girders and chord lateral bracing are needed to transmit the longitudinal live loads and the wind and/or earthquake loads to the bearings and also to prevent the compression chords from buckling. When a solid deck is used, the interaction between deck and trusses has to be considered. For the lateral bracing of the chords, where a "Saint Andrew's Cross" type system as shown in Figure 8a is adopted, the nodes of the lateral system will coincide with the nodes of the main trusses. Interaction will take place which must be taken into account. As a result of the interaction, the lateral system may carry as much as 6% of the total axial load in the chords. Figure 8b shows the lateral system in its original form and in its distorted form after axial compressive loads are applied in the chords. Owing to the shortening of the chord members ac and bd, the rectangular panel deforms as indicated by the dotted lines, causing compressive stresses in the diagonals and tensile stresses in the transverse members. The transverse bracing members are indispensable for the correct performance of a St Andrew's cross bracing system. The interaction can be significantly reduced by using a "Diamond" system of lateral bracing where the nodes of the lateral system occur midway between the nodes of the main trusses, Figure 8c. With this arrangement, "scissors-action" occurs when the chords are stressed, and the chords deflect slightly laterally at the nodes of the lateral system. In the principal buckling mode of a "Diamond" lateral bracing system, one half of the diamonds have all their members in tension (see Figure 9). For the top laterals a diamond system with kickers at the panel points halves the transverse effective length of the compression chord as shown in Figure 10. For railway bridges, Figure 9 illustrates an economic lateral system at deck level which consists of a simple single member which also functions as part of the braking girder. The additional girders help to resist the braking forces arising from trains. In addition the lateral is supported by the stringers so the effective length is only about a third of the panel length. Wind loading on diagonals and verticals can be split equally between top and bottom lateral systems. The end portals (either diagonals or verticals) then have to carry the load applied to the top chord down to the bottom chord. Clearly, where only one lateral system exists (as in semi-through or underslung trusses), then this single system must carry all the wind load. In addition to resisting externally applied transverse loads due to wind, etc. lateral bracing stabilizes the compression chord. The lateral bracing ensures that reasonably small effective lengths are obtained for the truss members. Local lateral bracing is also required at all 'kinks' in the chords where compressive loads are induced into the web members irrespective of whether the chord is in tension or compression because of the angular direction change of the chord. 5.ANALYSIS 5.1Global Load Effects Generally trusses have stiff joints. The secondary stresses due to joint stiffness and truss deformation can be ignored in the ultimate limit state check. They have to be considered where the serviceability limit state check is required, and for fatigue. However, these secondary effects are generally insignificant. The serviceability check is not required for tension members or for some slender compression members. Where it is not required, the traditional manual method of truss analysis assuming pin joints is adequate for global analysis. Computer analysis can take joint stiffness into account and secondary moments are determined automatically. The effects of the primary axial loads and the secondary moments are combined by the use of suitable interaction formulae. In a statically indeterminate truss, temperature effects have to be considered. They are usually not significant. 5.2Local Load Effects i.Loads not applied at truss joints Two types of local load effects have to be considered: a.Those due to loads applied in the plane of the truss away from a joint. A typical example of this type of loading is on the upper chord of an underslung railway bridge where the sleepers rest directly on the top flange of the chord. b.Eccentric loads not in the plane of the truss, such as loads from cross girders. ii.Eccentricities at joints Flexural stresses due to any eccentricity at joints have to be taken into account by sharing the moments due to eccentricity between the members meeting at joints in proportion to their rotational stiffness. For the main trusses the centroidal axes of all members should meet at a point wherever possible. The only case where a small degree of eccentricity is unavoidable is when asymmetrical "top-hat" sections are used and it is not possible for the centroidal axes of adjacent members of different sizes to be co-linear. Where possible the axes of the lateral systems should be in the same planes as those of the truss chords. However sometimes the upper laterals of a through truss have to be connected to the top flange of the upper chord and eccentricity is unavoidable. Since the loads in upper lateral systems are generally small, the additional resulting stresses are insignificant. Similarly on some through bridges, the bottom laterals have to be connected to the bottom flange of the lower chord to avoid the cross girders and stringers. 6.CONNECTIONS 6.1General The major connections in bridge trusses occur at the truss nodes where the web members are connected to the chord members. This connection usually incorporates a splice in the chord member and sometimes also in one or both of the minor truss connections joining the cross girder and the lateral system to the truss. Site connections can be made by high strength friction grip bolts for reasons of economy and speed of erection. Good site welds are difficult to achieve where access is difficult and fatigue life of welded joints is lower than that of bolted joints. However, in several countries, the connections are now usually butt-welded on site. Figure 11 shows different gusset geometries which are used to obtain durability in view of the fatigue-governing effects. When a concrete slab is cast in place to support the highway or the railway, the horizontal forces caused by the shrinkage of the concrete should be taken into account in the design of the lower chord connection joints. 6.2Truss Joints At the nodes of a truss where the web members are connected to the chords, there is a change in load in the chord which necessitates a change in its cross-section area. The node is, therefore, the point at which there is a joint in the chord as well as being the connection point of the web members. The web members are connected to the chords by vertical gusset plates. They are usually bolted to the chord webs and the web members fit between them (Figure 12a). The chord joint is effected by providing cover plates. They should be so disposed, with respect to the cross-section of the member, as to transfer the load in proportion to the respective parts of the section (Figure 12b). The gusset plates form the external web cover plates. Since they work in the dual capacity of cover plate and web connector, their thickness takes this into account. The joint is designed to carry the coexistent load in the lesser loaded chord plus the horizontal component of the load in the adjacent diagonal. The load from the other diagonal is transferred to the more heavily loaded chord through the gussets alone. In compression chords which have fitting abutting ends in contact, design codes allow up to 75% of the compressive load to be carried through the abutting ends. Sometimes the gusset is formed by shop-welding a thicker shaped plate to the chord in place of the chord web. The web members are then all narrower than the chords and the chord splice is offset from the node. An advantage occurs in erection as the web connections can be made before the next chord is erected. At the connections of all tension members and elements, care has to be taken in the arrangement of bolt holes to ensure that the critical net section area of the section is not so small that fracture will govern. If necessary staggering the lines of bolts helps to increase the effective nett area. Remember that the critical net section is usually at the ends of the section or the centre of the cover plates, and that elsewhere some of the load has been transferred to the other parts of the joint and more bolt holes can be tolerated. Connections of web members to gussets are quite straightforward and special treatment such as the use of lug angles is rarely required. In connecting rectangular hollow sections the method shown in Figure 12d is preferable to that of Figure 12c. Unsupported edges of gussets should be such that the distance between connections does not exceed about 50 times the gusset plate thickness (Figure 12a). If this is unavoidable, the edge should be stiffened. 6.3Cross Girder Connections They are quite straightforward. The 2 or 4 rows of bolts in the cross girder end plate are made to correspond with the equivalent central rows of bolts in the gusset. Packing plates may be required to accommodate the difference in height of gussets and cross girders (Figure 12e). 6.4Lateral Bracing Connections As recommended in 5.2(ii) the axes of the lateral systems should be in the same planes as those of the truss chords. This requirement is met in 2 of the 3 types of lateral members and connections described below: i.For long and medium spans, the lateral members are frequently made from two rolled channel sections connected by lacing to give an overall depth the same as the chords. They are connected to the chords by gussets bolted to the chord flanges exactly as the main web members are connected to the main joint gussets. ii.For medium spans, laterals consisting of two rolled angles arranged toe to toe in "star" formation and with intermediate battens are often ideal. They are connected to the chords by gussets positioned at the chord axis (Figure 12f). Note, angles "back-to-back", but separated by a small gap should never be used because of maintenance problems. iii.On short spans single laterals often suffice. They can be connected by a gusset to the upper or lower chord flange, as the moments due to eccentricity are small. 7.CONCLUDING SUMMARY Trusses can be assembled from small pieces and are particularly advantageous where site access is difficult. Keep the configuration simple, using a minimum of members and connections. For long bridges continuous trusses may be the economic solution, but remember that least weight of steel does not necessarily mean least cost. Fatigue effects have to be considered, particularly in continuous trusses. Trusses can be economic for spans of 30m to 200m. Avoid eccentricity of loading and connections to reduce secondary stresses. Configuration of members and careful arrangement of bolts at splices are particularly important. In the case of welded connections, only use butt-welded connections in order to avoid fatigue effects. The weld penetration should be complete. Avoid potential corrosion areas. Remember birds will nest and roost in the most unlikely places! Select a non-participating lateral system. 8.REFERENCES [1]Eurocode 3: "Design of Steel Structures": ENV 1993-1-1: Part 1.1. General Principles and Rules for Buildings, CEN, 1992. [2]Eurocode 3: "Design of Steel Structures": Part 2: Bridges and Plated Structures (in preparation). [3]BS5400: "Steel, Concrete and Composite Bridges", Part 3: 1982: Code of Practice for Design of Steel Bridges, British Standards Institute, London. 9.ADDITIONAL READING Roberts, G., Kerensky, O.A., "Auckland Harbour Bridge, New Zealand Design", Paper 6528, ICE Proc., Vol 18, April 1961, pp. 423-458. Turley, S., Savarkar, S.G., Williams, J., Tweed, R.J.C., "Design, Fabrication and Erection of Ganga Bridge, Mokameh, India", Paper 6425, Proc. ICE, Vol. 15, March 1960, pp. 231-254. Layfield, P., Taylor, G., McIlroy, P., King, C., Casebourne, M., "Tyne and Wear Metro: Bridge N106 over the River Tyne". Paper 8205, ICE Proc, Vol. 66, Part 1, May 1979, pp. 169-189. Previous | Next | ContentsLattice truss structural systems have been employed in constructing railway and highway bridges with great success for so many years. The design of truss bridges involves the analysis of the structure to obtain the internal forces due to moving traffic and permanent loads (self-weight), selection of adequate steel members, design of the connections, and check for fatigue. The availability of numerous commercial design software has made the analysis and design of 3D truss bridges easier than it was in the past.The Warren truss, the Modified Warren truss, and the Pratt truss are the three major truss configurations in use today, and they can all be employed as an underslung truss, a semi-through truss, or a through truss bridge.In an underslung truss, the live loading caused by the passage of automobiles or trains is carried directly by the top chord. In situations where the depth of construction or clearance under the bridge is not critical, underslung trusses can be conveniently used.In semi-through trusses, vehicles travel on the bottom chord of the truss, but the transient live load projects above the top chord members due to the height of the vehicles relative to the top chord of the truss. As a result, the top chords in semi-through trusses cannot be braced laterally, and these chord elements must rely on U-frame action for lateral stability. However in a through truss bridge, vehicles travel through the centre of the bridge on the bottom chord, and the space between the live load and the top chords is sufficient that the top chord members can be braced laterally. Through truss bridge appears to be the most common type of truss bridge.Figure 1: Major types of truss bridges (Parke and Harding, 2008)The members of a truss bridge will predominantly carry axial tension or axial compression stresses if the structure is designed and detailed so that live loading is effectively applied at the nodes. The global bending moment acting on the bridge may be resolved into a couple made up of the compression forces in the top chord and axial tension forces in the bottom chord. Similarly, the diagonal web elements carry the global shear force exerted on the truss bridge, either in axial tension or compression, depending on the configuration of the truss.As an example, the diagonal web elements of a Warren truss alternately carry compression and tension over the bridge. The internal diagonal web members of a Pratt truss, on the other hand, are all loaded in tension, while the shorter vertical web members are loaded in compression.Members of Truss BridgesThe chords and web members of truss bridges can be made out of a variety of steel sections. For the tension and compression chords as well as the web members of short-span (3050 m) highway trusses, rolled H sections and square hollow sections are suitable. Larger fabricated sections, such as a top hat section or box section, will be needed for the chords of longer highway truss bridges or trusses bearing railway loads.Built-up through truss bridgeTruss bridges transmit imposed loads to the foundations through the axial tension and compression forces in the members. Therefore, these structures can be analyzed as pin-jointed members, either as a two-dimensional truss or, more preferably, as a three-dimensional space truss.This form of analysis assumes that member connections are pinned, which means that none of the truss members may attract moment or torsion. By hand, a two-dimensional plane truss analysis can be solved by utilizing equilibrium equations to resolve the forces at each joint in turn, or by employing the method of sections to free-body segments of the bridge truss, again using equilibrium equations to derive member forces.The stiffness method can also be used to calculate node displacements first, and then member forces. Nowadays, truss bridges do not have pinned joints; instead, the connections are welded or bolted; yet, analyzing the structure as a two- or three-dimensional pin-jointed assembly allows for an accurate assessment of member axial stresses but overpredicts truss node displacements.However, since the joints are not pinned in real-life construction, it is necessary to analyze the truss as a three-dimensional space frame with six degrees of freedom at each node in order to obtain a more realistic prediction of node displacements as well as an assessment of the secondary bending and torsion moments, which will be small but still present.Secondary moments and torsions acting on the structure can affect the bridges fatigue life, particularly if the truss is continuous and spans multiple supports. By guaranteeing that the neutral axes of all members meeting at a node intersect at a single location in three-dimensional space, secondary forces and hence stresses can be reduced.A 9.0m wide through-truss bridge is to be designed to carry normal traffic across a river. The total height of the bridge is 5m, and I-sections are to be utilised in the top and bottom chord members of the truss, while square hollow sections will be utilised for the web members. The vertical members of the web are spaced at 2.5m each and the total length of the bridge is 25m.Modified Warren Truss BridgeThe deck of the bridge is composed of primary and secondary steel beam members. The floor beams consist of UB 457x191x161 members supported by the UB 610x305x179 bottom chord rail of the trusses and spaced at 2.5m intervals. The stringers are the secondary UB 305x102x28 members running parallel to the bottom chord and spaced at 1.5m. A 200mm thick reinforced concrete deck is expected to sit on the beams.The truss bridge has been modelled on Staad Pro software as shown below. The top of the truss bridge (top chord) will be braced using UB 254x146x37 members in a K-truss arrangement (see below) to restrain the top chord from sway under wind action.Structural Model of a Truss BridgeIn this article, the truss bridge will be analysed for the self-weight (alldead and superimposed loads) and traffic load. All other environmental loads and indirect actions will not be considered.Dead Load(1) Self-weight of steel members (to be calculated automatically by Staad Pro)(2) Self-weight of 200mm thick reinforced concrete deck = 0.2 25 = 5 kN/m2(3) Self-weight of 75mm thick asphalt wearing course = 0.075 23.5 = 1.8 kN/m2Total pressure dead load = 6.8 kN/m2Live LoadAccording to the requirements of Load Model 1 (LM 1), the carriageway width of 9m can be divided into three notional lanes as shown below;In essence, the traffic on the bridge will be represented by the UDL as specified above and the tandem system. The worst effect of the wheel load on the bridge deck will be considered. However, it is important that the influence line analysis of the bridge be carried out, in order to determine the wheel load location that will produce the worst effects on the structure.The result below depicts the internal stresses induced in the bottom chord at the load combination 1.35gk + 1.5qk (where qk represents the UDL component of the traffic load only).The result below depicts the internal stresses induced on the bottom chord under the unfactored moving tandem wheel load only.A little consideration will show that the following results are applicable for the bottom chord;Design Axial compression (member 1): 462.892 + 1.5(300.842) = 914.155 kNDesign axial tension (members 5 and 6) = 457.486 + 1.5(321.591) = 939.8725 kNDesign major bending moment (member 3): 231.737 + 1.5(231.617) = 579.1625 kNmDesign minor axis bending moment = 2.541 + 1.5(1.590) = 4.926 kNmDesign Shear (major axis) = 101.619 + 1.5(158.288) = 339.051 kNDesign Shear (minor axis) = 1.853 + 1.5(1.17) = 3.609 kNIt is obvious that these maximum forces do not interact on the same point in the section. However, for the sake of simplicity, let us assume they interact at the same point in the structure. The design verifications are as follows;Section s1 results summaryUnitCapacityMaximumUtilisationResultShear resistance (y-y)kN1441.9339.10.235PASSShear resistance (z-z)kN2047.73.60.002PASSBending resistance (y-y)kNm1470.0579.20.394PASSBending resistance (z-z)kNm303.14.90.016PASSCompression resistancekN5598.0914.20.163PASSComb. bending and axial force0.571PASSSection detailsSection type;UB 610x305x179 (BS4-1)Steel grade EN 10025-2:2004; S275Nominal thickness of element; tnom = max(tf, tw) = 23.6 mmNominal yield strength; fy = 265 N/mm2Nominal ultimate tensile strength; fu = 410 N/mm2Modulus of elasticity; E = 210000 N/mm2Classification of cross sections Section 5.5 = [235 N/mm2 / fy] = 0.94Internal compression parts subject to bending and compression Table 5.2 (sheet 1 of 3)Width of section;c = d = 540 mm = min([h / 2 + NEd / (2 tw fy) (tf + r)] / c, 1) = 0.727c / tw = 38.3 = 40.7 396 / (13 1); Class 1Outstand flanges Table 5.2 (sheet 2 of 3)Width of section; c = (b tw 2r) / 2 = 130 mmc / tf = 5.5 = 5.8 9;Class 1Section is class 1Check compression Section 6.2.4Design compression force;NEd = 914.2 kNDesign resistance of section eq 6.10;Nc,Rd = Npl,Rd = Afy / M0 = 6044.2 kNNEd / Nc,Rd = 0.151PASS Design compression resistance exceeds design compressionSlenderness ratio for y-y axis flexural buckling Section 6.3.1.3Critical buckling length;Lcr,y = Ly_s1 = 2500 mmCritical buckling force; Ncr,y = 2EIy / Lcr,y2 = 507456.5 kNSlenderness ratio for buckling eq 6.50; y = (Afy / Ncr,y) = 0.109Check y-y axis flexural buckling resistance Section 6.3.1.1Buckling curve Table 6.2; aImperfection factor Table 6.1;y = 0.21Buckling reduction determination factor;y = 0.5[1 + y(y 0.2) + y2] = 0.496Buckling reduction factor eq 6.49; y = min[1 / (y + (y2 y2)), 1] = 1Design buckling resistance eq 6.47;Nb,y,Rd = yAfy / M1 = 6044.2 kNNEd / Nb,y,Rd = 0.151PASS Design buckling resistance exceeds design compressionSlenderness ratio for z-z axis flexural buckling Section 6.3.1.3Critical buckling length;Lcr,z = Lz_s1 = 2500 mmCritical buckling force;Ncr,z = 2EIz / Lcr,z2 = 37832.1 kNSlenderness ratio for buckling eq 6.50; z = (Afy / Ncr,z) = 0.4Check z-z axis flexural buckling resistance Section 6.3.1.1Buckling curve Table 6.2;bImperfection factor Table 6.1;z = 0.34Buckling reduction determination factor;z = 0.5(1 + z(z 0.2) + z2) = 0.614Buckling reduction factor eq 6.49;z = min(1 / (z + (z2 z2)), 1) = 0.926Design buckling resistance eq 6.47;Nb,z,Rd = zAfy / M1 = 5598 kNNEd / Nb,z,Rd = 0.163PASS Design buckling resistance exceeds design compressionCheck of torsional and torsional-flexural buckling showed that the section is okay. Section 6.3.1.4Check for shear Section 6.2.6Height of web;hw = h 2tf = 573 mm; = 1.000hw / tw = 40.6 = 43.2/ < 72/Shear buckling resistance can be ignoredDesign shear force;Vy,Ed = 339.1 kNShear area cl 6.2.6(3);Av = max(A 2btf + (tw + 2r)tf, hwtw) = 9425 mm2Design shear resistance cl 6.2.6(2);Vc,y,Rd = Vpl,y,Rd = Av(fy

/3) / M0 = 1441.9 kNVy,Ed / Vc,y,Rd = 0.235PASS Design shear resistance exceeds design shear forceCheck bending moment Section 6.2.5Design bending moment;My,Ed = 579.2 kNmDesign bending resistance moment eq 6.13;Mc,y,Rd = Mpl,y,Rd = Wpl.yfy / M0 = 1470 kNmMy,Ed / Mc,y,Rd = 0.394PASS Design bending resistance moment exceeds design bending momentCheck bending and axial force Section 6.2.9Bending and axial force check eq.6.33 & eq.6.34;Ny,lim = min(0.25Npl,Rd, 0.5hwtwfy / M0) = 1070.5 kNNEd / Ny,lim = 0.854Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axisBending and axial force check eq.6.35;Nz,lim = hwtwfy / M0 = 2141.0 kNNEd / Nz,lim = 0.427Allowance need not be made for the effect of the axial force on the plastic resistance moment about the z-z axisN = 2N = max(5n, 1) = 1For bi-axial bending eq.6.41;[My,Ed / Mpl,y,Rd]N + [Mz,Ed / Mpl,z,Rd]N = 0.171PASS Biaxial bending utilisation is acceptableCheck combined bending and compression Section 6.3.3Equivalent uniform moment factors Table B.3;Cmy = 1.000Cmz = 1.000CmLT = 1.000Interaction factors kij for members susceptible to torsional deformations Table B.2Characteristic moment resistance;My,Rk = Wpl.yfy = 1470 kNmCharacteristic moment resistance;Mz,Rk = Wpl.zfy = 303.1 kNmCharacteristic resistance to normal force;NRk = Afy = 6044.2 kNInteraction factors;kyy = Cmy(1 + min(y 0.2, 0.8) NEd / (yNRk / M1)) = 0.986kzy = min(0.6 + z, 1 0.1zNEd / ((CmLT 0.25) zNRk / M1)) = 0.991kzz = Cmz(1 + min(2z 0.6, 1.4) NEd / (zNRk / M1)) = 1.033kyz = 0.6kzz = 0.620Interaction formulae eq 6.61 & eq 6.62;NEd / (yNRk / M1) + kyyMy,Ed / (cLTMy,Rk / M1) + kyzMz,Ed / (Mz,Rk / M1) = 0.55NEd / (zNRk / M1) + kzyMy,Ed / (cLTMy,Rk / M1) + kzzMz,Ed / (Mz,Rk / M1) = 0.571PASS Combined bending and compression checks are satisfied.The result below depicts the internal stresses induced in the bottom chord at the load combination 1.35gk + 1.5qk (where qk represents the UDL component of the traffic load only).The result below depicts the internal stresses induced on the bottom chord under the unfactored moving tandem wheel load only.Design Axial compression : 1181.948 + 1.5(761.358) = 2324 kNDesign axial tension = 928.940 + 1.5(623.655) = 1864 kNDesign major bending moment: (ignored for brevity)Design minor axis bending moment = (ignored for brevity)Design Shear (major axis) = (ignored for brevity)Design Shear (minor axis) = (ignored for brevity)Classification of cross sections Section 5.5 = [235 N/mm2 / fy] = 0.92Internal compression parts subject to compression Table 5.2 (sheet 1 of 3)Width of section;c = b 3t = 212.5 mmc / t = 17 = 18.4

- http://leader66.com/upload/files/86856595562.pdf
- cutawixile
- http://togul.org/sites/default/files/file/11965044599.pdf
- sibe
- rolehu
- http://sgrdpandher.com/userfiles/file/xovurojawujolus.pdf
- informal roles of the president