Steel Plate Girder Bridge Design Using Indian Standard Method And British Standard Method

by

Nur Afifah binti Azemi

Dissertation submitted in partial fulfilment of

the requirements for the Bachelor of Engineering (Hons)

(Civil Engineering)

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CERTIFICATION OF APPROVAL

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Approved by,

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(Dr. Narayanan Sambu Potty)

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

January 2014

CERTIFICATION OF ORIGANILITY

This is to certify that I am responsible for the work submitted in this project that the original work is my own except as specified in the references and acknowledgements, and that the original work herein have not been undertaken or done by unspecified source or person.

NUR AFIFAH BINTI AZEMI

ABSTRACT

This paper compares the Indian Standard Method and British Standard Method in designing a steel plate girder bridges. From the comparison, the author comes out with a design example for both design method. By using excel spreadsheet, the author compares the weight of the plate girder bridge designed using both codes as the span increases with a fixed yield strength used. The design codes used for this study is BS 5400, IS 800:1984, Railway Bridge Rules, and Steel Construction Institute (SCI) Publication.

ACKNOWLEDGEMENT

First of all, thanks to Allah Almighty for ease my way of graduating in Bachelor of Civil Engineering program in Universiti Teknologi Petronas (UTP). Thank you Allah for granting me an opportunity, health and ability to complete this Final Year Project. This project is a partial requirement for the Bachelor Of Engineering (Hons) in Civil Engineering in UTP.

Special gratitude to my Final Year Projet (FYP) Supervisor, Assocs. Prof. Dr. Narayanan Sambu Potty for all your help, guidance, and endless support for me to complete this project. Without the help from you, I am sure that this project could not be completed in time.

Millions thanks I bit to my mother, my late father, family members and friends for the support given to me in completing this project.

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CHAPTER 1

INTRODUCTION

This chapter describes the background of this study, problem statement, and objectives also scope of study, relevancy of study and the feasibility of the study.

1.1 BACKGROUND OF STUDY

Girder bridges can be constructed using several materials such as steel, concrete and even wood. The steel girder bridges is a structure in which a floor system and roadway, concrete or timber is supported by girders, usually rolled section beams which are incased in concrete. It began to be built around 1850 where metal truss being form was evolving into variations. By the end of nineteenth century, the girder bridge was established in all its forms like plate girders, I-beams and concrete encased I-beams. In one technical paper entitled Steel Girder Bridges, they mentioned that in 1900, the girder bridges were used for spans less than 100 feet long but in 1930; the spans were built up to 150 feet long. Plate girder bridges will be described in more detail in the next chapter.

Bridges history in the world noted that the first iron bridge built in 1779 at Coalbrookdale, Telford by Abraham Darby (the third). It was the first large structure been constructed from iron at that time. It was reported by V.Ryan in the year 2009 in Technology Student website.

Figure 1: The first iron bridge in the world

Figure 2: The first iron bridge in the world

Tata Steel Europe in their website reported that this iron bridge is still in use today to carry occasional light transport and pedestrians. Around 1800s, the cast iron being replaced by wrought iron and many of these bridges were built of riveted wrought iron construction. Steel began to replace this wrought iron in the late 1800s. Since then, steel become one of the top materials to build different structures around the world especially bridge. It has many advantages in terms of construction strength and ductility. This material contains high level of strength and tension as compared to concrete.

The chronology of some of the bridges been built in the early ages are as follows:

Table 1 Chronology of bridges built in early ages

Year	Bridge	Descriptions
1857	Weichsel Bridge 	The first large wrought iron girder railway bridge to be built in Germany
1863	Menangle Viaduct 	The oldest existing railway bridge in Australia. Having wrought iron riveted box girders and three equal spans of 49.4m. Now, the span has been halved by adding the intermediate piers to allow it to carry heavier loading.
1870	Kymijoki Railway Bridge, Finland 	The first three span bridge built in Finland. At first, this bridge being design as a railway bridge but been converted to carry road traffic in 1923 and still being used until today as footbridge.
1883	Brooklyn Bridge, USA	The first steel wire and steel

		bridge built in the world.
1884	Garabit Viaduct, France 	One of the first wrought iron truss arch bridges build in the world.
1888	Tenryu Gawa Bridge, Japan 	First railway bridge built in Japan using steel.
1890	Forth Bridge, Edinburgh, Scotland.	The world longest spanning bridge at the time of its construction. Having two main spans of 518m. Still being used until today on the main Edinburgh to Aberdeen

		line.
1931	Golden Gate Suspension Bridge, USA 	Construction started in 1933. Designed by Chief Engineer Joseph Strauss. It is hybrid cantilever and suspension bridge. Been opened to public on May 28, 1938.
1932	Sydney Harbour Bridge 	Designed by Dorman Long and Co. Ltd and open to public in 1932. Built at Sydney Harbour and used by vehicles, bicycles, and other pedestrian and rail traffic. It connects Sydney Central business District and the North Shore. Known as steel through arch bridge which provides a dramatic view in Sydney harbour. Being called the coat hanger due to its arch shaped design.

		Awarded as the world's long span bridge and the tallest steel arch bridge at 134 meters.

1.2 PROBLEM STATEMENT

Plate girder had been used since the late 1800s where they use in constructions of railroad bridges. As the technology evolved, there are different methods been initialized by the professional in designing a plate girder bridges where each method has their own priorities. Hence, there will be a slight differences and similarities in each of them. This paper is aimed to compare the design method for steel plate girder bridges for railway.

1.3 OBJECTIVES OF THE STUDY

The objective of this study to compare the design method in designing Railway Bridge using Indian Standard Method and British Standard Method. At the end of this study, the author will compare the provision of respective design standard and the difference in weight of the structure designed when the span is varied with the same yield strength used.

1.3 SCOPE OF STUDY

This study focuses on the designing steel plate girder bridge using Indian Standard Method and British Standard Method. The reference tools that is used in this study are IS 800-1984, Indian Bridge Rules (Railway Specification for loads), BS 5400-1, BS 5400-2, and BS 5400-3.

This study will comprise the differences of the main and important provisions highlighted in different codes of practice in designing plate girder bridges. By the end of this study, the author will come out with the design example and the spreadsheet to ease the calculation of designing welded plate girder bridges for all codes being studied.

1.4 RELEVANCY OF THE PROJECT

This study is relevant to clearly see the different between the Indian Standard Method and British Standard Method in designing the plate girder railway bridge as the Indian Method is actually adopted from the British Standard in the first place. However, Indian Method is then been modified to match with their country condition.

1.5 FEASIBILITY OF THE PROJECT

This study is very feasible to be completed in 28 weeks. Gantt chart has been prepared for the author to ensure that everything is on track and meet the objectives of the study.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This chapter will cover the introduction to plate girder bridges components, factors being considered in designing plate girder bridges and the previous comparison being made on the codes provisions for bridges.

2.2 PLATE GIRDER BRIDGES

Plate Girder Bridge is a bridge supported by two or more plate girder. The plate girder is typically I-beams made up from separate structural steel plates (rather than rolled as a single cross section), which are welded, bolted or riveted together to form a vertical web and horizontal flanges of the beam. The first tubular wrought iron plate girder bridge was built in 1846 by James Millholland for Baltimore and Ohio Railroad. These kinds of bridges are suitable for short and medium spans and may support railroads, highways or other traffic. It is usually prefabricated and the length limit is set by the mode of transportation used to move the girder from the fabricator to the construction site.

Figure 3: Example of plate girder railway bridge

The main component of plate girder is the vertical middle section called web, and the upper and lower horizontal member called upper and lower flanges. The intermittent vertical pieces perpendicular on the plate girder bridges is called stiffeners which functioned to prevent the web from buckling or twisting.

The depth or height of plate girder is not less than 1/15 of the total span and for the given load bearing capacity, the depth around 1/12 of the span minimizes the weight of the girder. The top and the bottom of the flanges plates are normally reinforced in the middle of the span as the stresses exerted near the center of the span are greater than near the end of the span. The vertical stiffeners help to prevent the web plate from buckling under shear stresses.

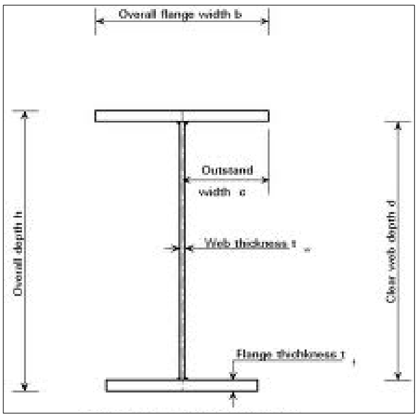


Figure 4 : Plate Girder proportion

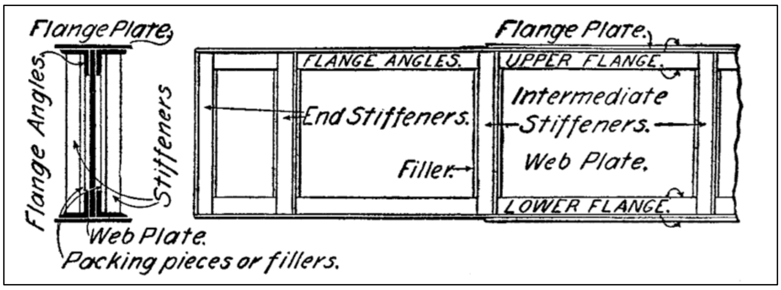


Figure 5: Anatomy of the plate girder

There are several types of plate girder bridges as follows:

Table 2: Different Type of Plate Girder Bridge

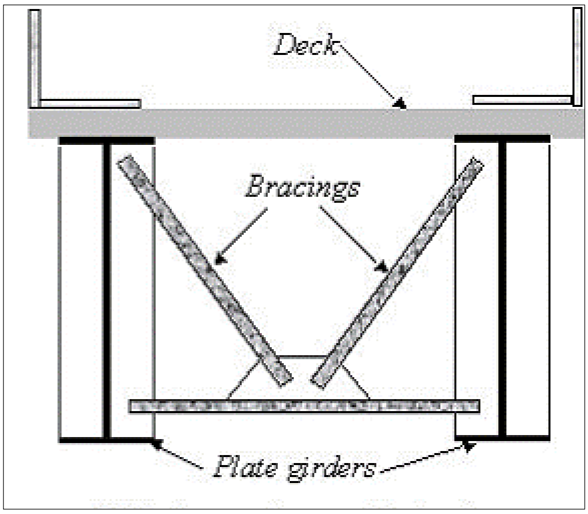
<table>
<tr><th>Types</th><th>Characteristics</th></tr>
<tr><td>1) Deck type plate girder bridge</td><td>Wood, steel or reinforced concrete bridge deck is supported on top of two or more plate girder and act compositely. For the railroad bridge, the railroad will be fixed onto the girder to form the bridge deck and the deck will support ballast on which the track is placed.


Figure 6: Deck Type girder bridge
Bracing is added to the structure to prevent the girders from buckle.</td></tr>
</table>

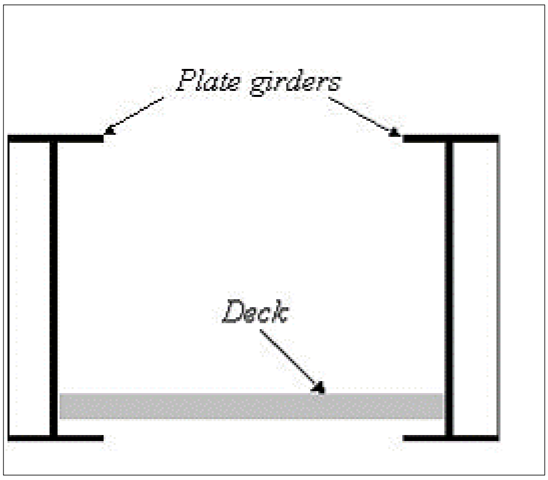
2) Half through plate girder bridge	Also called ponny truss. The deck is supported between two plate girders, usually on top of the bottom flange. The vertical stiffeners are used to prevent the girder from buckle instead of cross bracing. Usually used on railroads and the construction depth (distance between the underside of the vehicle, and the underside of the bridge) is less. This is to allows obstacles to be cleared with less change in height.  Figure 7: Half through plate girder bridge
3) Multi-span plate girder bridge	Piers act as the intermediate abutments between the end abutments of bridge. Separate plate girder bridge span between each pair of abutments in order to allow for the expansion joints between the spans. Concrete will be used for low piers and steel trestle work will be used for the high bridge.

Figure 8: Multi span plate girder bridge

According to Prof. S. R. Satish Kumar and Prof. A. R. Santha Kumar in their writing Design of Steel Structure, the plate girders became popular in the late 1800's and are used in the construction of railroad bridges. The plates were joined together using angles and rivets to obtain the desired size. By 1950s, the riveted plated girder and bolted plate girders were replaced by welded plate girder due to their better quality, aesthetics and economy.

The main girders require web stiffening (either transverse or both transverse and longitudinal) to increase efficiency. The stiffeners are used to prevent buckling at the main girder. From the economical design point of view, variation of flanges sizes and capacity are needed since the bending moment happened in the main girders are vary. For example, a thicker flange can be used where the bending moment is high while for a very long continuous span (span > 50) variable in flanges depth can be considered.

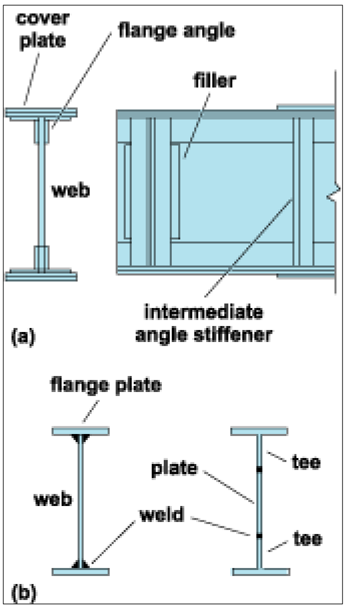


Figure 6: The flange in the main girder

Practically, the initial design of the main plate girder is based on the experiences of the designer and the normal indicative range values are as follows:

Table 3: Rule of thumb for main plate girder design

Overall Depth, D	I/18 ≤ D ≤ I/12 (Highway bridges) I/10 ≤ D ≤ I/7 (Railway bridges)
Flange width, b	D/4 ≤ 2b ≤ D/3
Flanges Thickness, T	b/12 ≤ T ≤ b/5
Web Thickness, t	t ≈ D/125

- I is the length between points of zero moment.

For the detailed design of main girder plate, the load effects shall be determined using un-factored load cases. BS5400: Part 3 prohibits the redistribution of forces due to plastic mechanism as bridges is subjected to cyclic loading and exposed to fatigue.

2.3 FACTORS CONSIDERATIONS

There are several factors being considered in designing the main plate girder based on the Limit State of Collapse as follows:

a) Shape limitation based on the local buckling

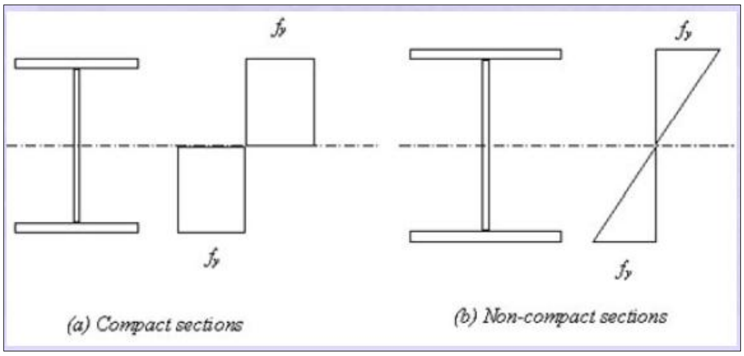


Figure 7: Design Stress

Based on the figure 7(a), a compact section can develop full plastic moment. The section should keep minimum thickness of elements on the compression zones so that they do not buckle locally before the entire compression zone yields in compression. The minimum thickness of elements for a typical compact section is shown in Figure 8.

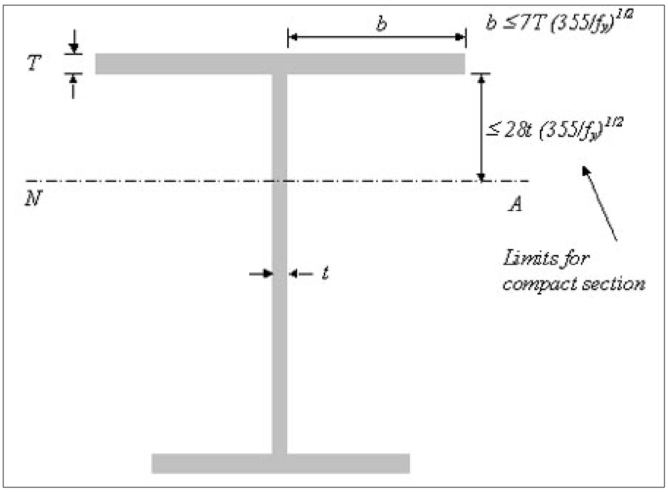


Figure 8 : Shape Limitations for plate girder

The non-compact section may buckle locally before full section plastic capacity is reached. Hence, the design of non-compact section is based on the triangular stress block as shown in Figure 7(b) where yielding at the extreme fibre limit the design moment.

Theoretically, the design capacity of the compact and non-compact cross section will be can be analyzed by the following formula:

$M_u = Z_p f_y / \gamma_m$ (for compact section)

$M_u = Z f_y / \gamma_m$ (for non-compact section)

$Z_p = plastic\ modulus$

$f_y = yield\ stress$

$Z = elastic\ modulus$

$\gamma_m - partial\ safety\ factor\ for\ material\ strength$ (1.15)

b) Lateral torsional buckling

The typical bridge girder which its compression flange is laterally unrestrained is expected to experience lateral torsional buckling. The displacement at the mid span where the beam is laterally restrained will only be vertical. Part of the beam between restraints can translate downwards and sideways and rotate about shear center. Failure will be controlled by lateral torsional buckling and it depends on the understrained length of compression flange, the geometry cross section, moment of gradient and etc.

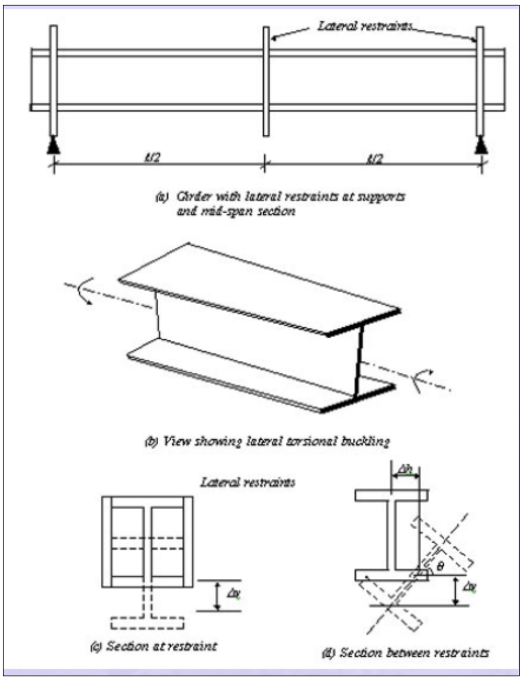


Figure 9: Distrosion caused by lateral torsional buckling

c) Web buckling

Plate girder resists the shear in three modes:

- Pure shear
- Tension field action
- Formation of collapse mechanism

The elastic local buckling of the web in shear does not lead to collapse Limit State due to the stable post buckling behavior. In tension field action mode, the tension field develops in the panel after shear buckling. The maximum shear capacity is reached when the pure shear stress mode and membrane stress cause yielding of the panel and plastic hinges in the flanges. This will lead to the formation of the collapse mechanism.

d) Interaction of bending and shear

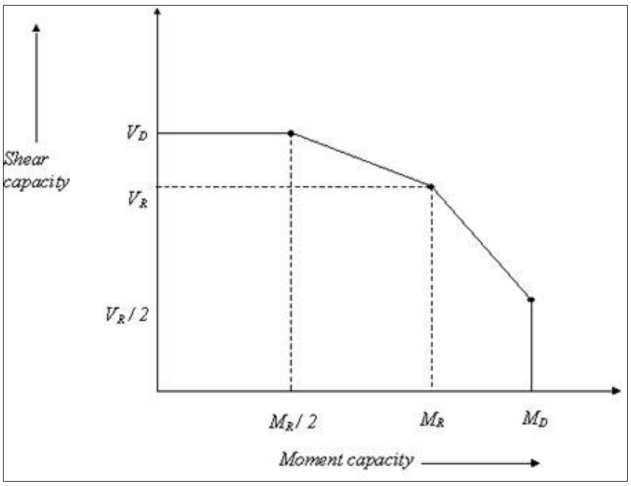


Figure 10: Shear moment capacity diagram

M_D and M_R are the bending capacities of the whole section with and without considering the contribution of the web, respectively.

V_D and V_R are the shear capacities with tension field theory, considering the flanges and ignoring the flanges, respectively.

e) Fatigue effect

Flaws in the tension zone under cyclic load will lead to the increasingly crack and finally failure even though the stress exerted on the bridge is within the design limit. IS:1024 provides the guideline for evaluating fatigue strength of the welded details which can help in evaluating the fatigue strength. Stress concentration can cause the premature cracking the bracing stiffener and shear connector welds. To increase the design life of plate girder, a proper detailing of connections may be needed.

f) Lateral bracing for plate girder

Plate girder is very likely to experience a lateral torsional instability when the bend about major axis. This is due to the very low torsional stiffness and a very high ration of major axis to minor axis moment of inertia. Practically in the completed structure, the flange is stabilized by the deck. Modes of instability of plate girder are illustrated in Figure 11 below.

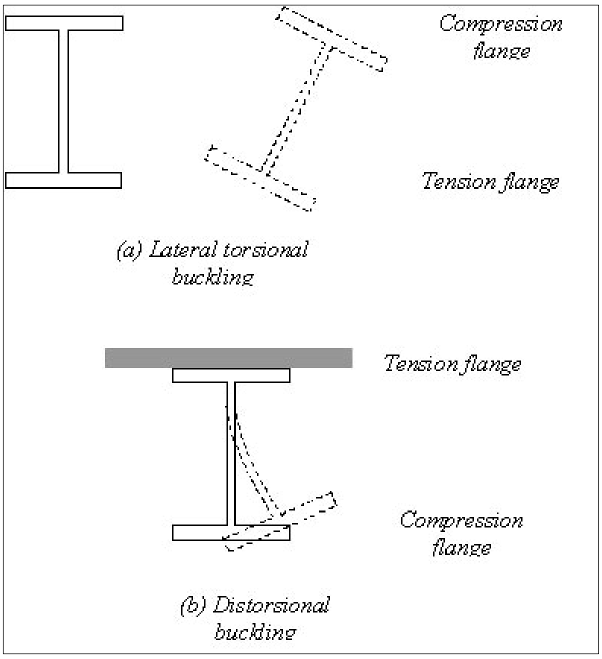


Figure 11: Modes of instability of plate girder

Distorsional buckling may happen if the undestrained flange is in compression. Hence, a bracing system of cross frames and bracings can be located in the horizontal plane at the compression flange of the girder to increase lateral stability.

Wind load can also cause the lateral bending due to the lateral transverse load that acts on the plate girder. The higher the depth of the plate girder, the larger the surface area over which the wind load can act. This lateral load may cause the instability of the compression flanges of the girder. So lateral bracing may be needed to counter this problem. In normal practice, triangulated bracing is provided for the deck to increase lateral stability of the compression flange. But this kind of bracing is not suitable for half through and through girder bridges as it will affect the function of the bridge itself. Hence, the deck is designed as a horizontal beam providing restraint against translation and flange which is far from the deck is stabilized by using U-frame. The effective length of a compression flange is normally calculated just like the theory of the beams on elastic foundation, the elastic support being the U-frame.

2.4 PREVIOUS CODES COMPARISONS

Comparison study between design codes is not new in the industry. SAM which is one of the well-known software used to test loading, do analysis and design a small to medium span bridges. In their study, they did a simple comparison of design of a pre-tensioned bridge to Eurocodes and British Standard. They designed a simple concrete bridge deck using BS 5400 and then using UK National Annex . the deck was a combination between two 20m spans with 25° skew, made continuous over its central support carrying single carriagewat and was constructed with UK standard Y3 beams at 1m centers. The BS 5400 beam was designed for a live load sagging moment of 384 kNm and hogging moment of 328 kNm. However the Eurocodes beam was designed for a variable load characteristic sagging moment of 511kNm (383kNm frequent) and characteristic hogging moment of 387kNm (289kNm frequent).

From the study, they found out that the tension limit for the designed bridge using BS 5400 is controlled by stress and Eurocodes is controlled by either decompression or crack width. For BS 5400, 19 tendons was required and Eurocodes design, 17 tendons was require. Each tendons contributes approximately 0.65MPa to the average concrete stress in this example. The difference in the number of tendons arises from the increased jacking force allowed by the Eurocodes, and from the differences in default values for creep and shrinkage suggested by BS 5400.

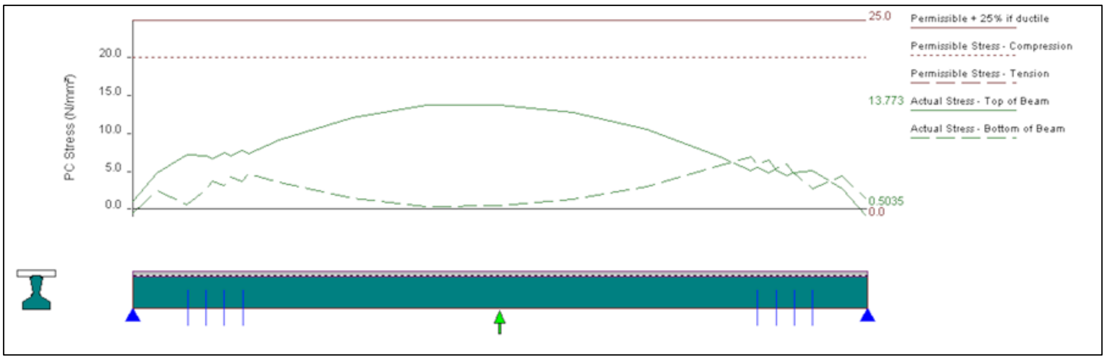


Figure 12: SLS Stress Results for BS 5400 Design for Load Combination 1

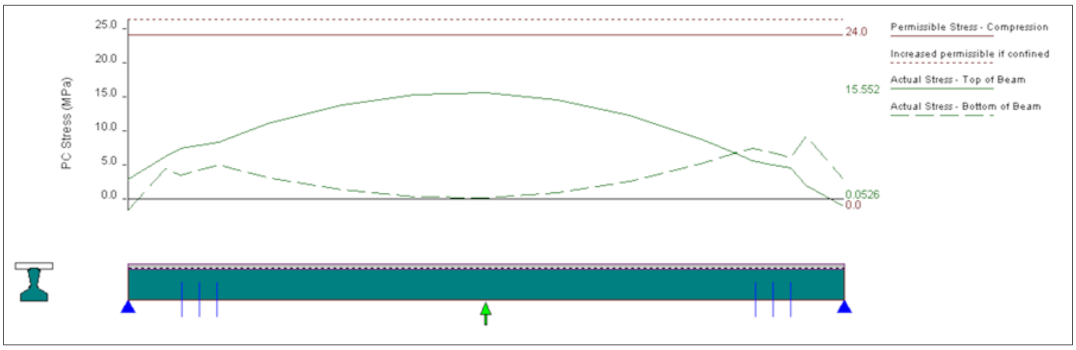


Figure 13: SLS Stress Results for Eurocode Design with Frequent combination of actions

There are some paper written mainly to compare these codes to find out the weakness and the strong points of some popular codes which are commonly used in engineering design. For example in September 2002, in the Buletin of the New Zealand Society of Earthquake Engineering, Richard Fenwick, David Lau and Barry Davidson had come out with a technical paper purposely to compare the seismic design requirements in New Zealand loading standard with major design codes in the world. After doing some analysis on for the building located in the low and high seismic region, they came out with a conclusion that the strength and the stiffness requirement for both New Zealand and Draft Standard is low as compared with the other design codes in high seismic zone.

In Bangladesh, M. A. Noor, M. A. Ansary and S. M. Seraj did the critical evaluation and comparison of different seismic code provisions in the year of 1997. Different parameters used in the evaluation which includes zone factor, importance factor, structural system factor, site geology and soil characteristic, and period etc. the codes chosen to be compared are Uniform Building Code (UBC), 1994 edition, The Criteria for Earthquake Resistant Design Standard Institute (IS), 1984 editions, the National Building Code of Canada (NBC) 1995 edition and the Building Standard Law of Japan (BSLJ), 1987 edition. From the analysis made, they found out that almost all code of provision implement similar definitions for the numerical coefficient of the base shear formula in calculation base shear in stationary methods. These codes had improved through a very detailed process and the concern countries experienced seismic codes regularly. The basic principal of these codes is that yield is allowed to accommodate the seismic loading as long as the yielding does not weaken the vertical load capacity of the structure.

Not only buildings, in 2009 Aguiade Drak El Sebai did a study and compare the seismic codes for bridges. He did a comparison between ASSHTO-2004 (American Association of State Highway and Transportation Official-2004), BSI-EN1998-2:2005, NBCC-2005, C-2005 and the 2007 proposed AASHTO LRFD seismic design provisions with the 2006 CSA 56 Canadian Highway Bridge Design Code (CHBDC). He used 2 span of 90m long bridge to apply the seismic design loads taken from the codes studied. There are three different seismic regions being studied which are Montreal, Toronto and Vancouver. He compared the effects of the seismic design spectra and over strength factors in generating the design moments, shears and displacement ductility demand of the bridge.

While in Pakistan, Muhammad Tariq Amin Chaudhary claims that the Pakistan code of Practice for Highway bridges adopted in 1967 has serious shortages and need to be approved. In Taiwan, a guy named Ching-Chuan Huang investigate the seismic displacements of two highway bridges abutments based on the input ground accelerations suggested by both new and old seismic design codes. He used a pseudo-static-based multi-wedges method in collaboration with Newmark’s sliding block theory. He reported that the design peak ground acceleration used in the new codes is greater than then in old one for some near-fault area in that country. There also some studies being done on the pile foundation on bridge. For example, studies done by Baydaa Maula in 2011 where he present the current of existing vast gap liquefiable and liquefaction-induced lateral spreading ground between Chinese and Japanese seismic design specification. It seems that in Chinese specification is too general and less systematics and maneuverability than Japanese Specification.

In 2005, Edoardo, Marino, Masayshi and Khalid come out with a paper which compares Eurocode 8 (EC8) and the Japanese seismic design code (BCJ) for steel moment frames and braced frames. They compare the features of the codes which include soil classification, magnitude and shape of unreduced elastic response spectra, member ductility demand and etc. after completed the study, they claims that both codes are slightly similar except for the seismic force specified for the serviceability limit state where EC8 recommended 2.5 larger forces for this limit state. This leads to the greater net strength than BCJ for steel moment frames. But for the braced frames, BCJ have large lateral story strength except for chevron braced frames with slender braces.

Comparison of codes provisions for design of steel bridges enables us to know which country spends more money to meet their design standard and which country imposed maximum safety standards (Midhun B Sankar, Priya A Jacob , 2013). Midhun and Priya did a study to compare the Indian and Europeans standard for railway bridge which concentrated more on the total deflection and weight of the steel girder by manipulating the grade of the steel, the panel aspect ratio, and web slenderness ratio. From the study, they concluded that for a constant span and depth of bridge, the total deflection of the girder increases as the grade of steel increase but the total weight decreases based on both Indian Standard and the Eurocodes. The stiffener spacing has much impact on the deflection of plate girder. The maximum deflection as per Indian Standard is more as compared to European Standard and they found out that the Indian Standard spend more money to meet the requirement as compared to the European Standard.

CONCLUSION

From the previous studies that have been done on the seismic design codes, it shows that seismic design codes is being modified based on the technologies and earthquake history of that country. Design codes are an important tool for that country to maintain the safety of all structures built. The differences and the similarities of design parameters show the different standard being highlighted. Regardless of the similarities and the differences, every code is aimed to provide a safe design structure for the benefit of the country.

From the literature review, we can see that there is less comparison being made on the codes of seismic design of bridges. So this paper is aimed to focus on the comparison between the provision of codes using Indian Standard (IS 1893:1962) and Eurocode 8 – Part 2: Design of Bridges and Retaining Wall to see the differences and the similarities of those codes.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 INTRODUCTION

This chapter describes the methodology used to complete the study.

3.2 PROJECT ACTIVITIES

i. Literature Review

During this activity, the author did research study on the existing studies been done by the professionals that are related to the topic discussed. This is to get the ideas and information and also to get familiar with the terms used in discussing the topic.

i. Comparison of design method

At this stage, the author will study the design method and do some comparison between those methods. There are some aspects that will be compared which are the design procedure, the loading calculation and estimation and the size limitation of the plate girder used in the design. The author also designed a plate girder for Railway Bridge using both Indian and British Standard Method to clearly see the difference between these two methods.

ii. Data analysis

From the design calculation of these two codes, the author did some analysis to see the pattern of weight changes when the span changes. For this analysis, the parameter which is fixed is the yield strength used for the design which is 340 N/mm^2. To ease the data analysis, excel spreadsheet is designed to calculate the size needed when the span changes and also the weight difference between both design example.

3.3 KEY MILESTONE

Submission o f Extended Proposal	• Extended proposal need to be submitted to the supervisor which contains all the preliminary research of this research study
Submission of Draft Interim report	• Draft Interim report need to be submitted to the supervisor as the progress report before the final Interim report
Submission of the Interim Report	• Interim report is the final report for preliminary research for this research study
cSubmission of Progress Report	• Progress report is documented to recorded all the progress for this study.
Submission of Dissertation (soft bound)	• Soft bound dissertation is the draft of the final dissertation submitted to ensure that the project is on track
Submission technical Paper	• Technical paper is a compulsory to be submitted to complete the subject requirement
Oral presentation	• This project is presented orally to the examiners
Submission of the Project Deissertatin (Hard Bound)	• Submission of the final dissertation is compusory as a record that this project is completed and will be assessed by the examiners

3.4 GANTT CHART

To ensure the study being run smoothly and on track, the author has prepared a Gantt chart which lists all activities that need to be completed in a specific time frame.

Detail/Week	1	2	3	4	5	6	7		8	9	10	11	12	13	14
Selection of Project Type															
								M							
Preliminary Research Work								I							
								D							
Submission of Extended Proposal								S							
								E							
Proposal Defense								M							
Project work continues								B							
								R							
Submission of Interim Draft Report								E							
								A							
Submission of Interim Report								K							

Detail/Week	15	16	17	18	19	20	21		22	23	24	25	26	27	28
Data Collection & Review – Eurocode 8															
								M							
Data Collection & Review – Indian Standard 1893								I							
								D							
Analysis of the data obtained – Differences and Similarities								S							
								E							
Design Example using both codes								M							
FYP 2 Presentation															
Report Preparation															

PROCESS

SUGGESTED MILESTONE

3.5 TOOLS REQUIRED

i. Microsoft Office

Microsoft Office is used to record the data extracted from the codes reviewed and studied. Besides that, this software will be used by the author to write report that need to be submitted to complete the study.

ii. Microsoft Excel

Microsoft Excel is used to create the excel spreadsheet to ease the calculation of the size of the plate girder needed and to analyses the difference in terms of the weight of the railway bride when the span is varied.

iii. Adobe Reader

Adobe Reader software is used to view the codes in soft copy format to ease the data collection.

iv. Codes that will be studied:
   - Indian Method:
     1) Bridge Rule (Railway)
     2) Steel Bridge Code
   - British Standard Method
     1) BS 5400 – 1
     2) BS 5400 – 2 (Loads)
     3) BS 5400 – 3

CHAPTER 4

RESULTS

Part 1:

Comparison between Indian Standard Method and British Standard Method

1) Codes used

Table 4: Codes and standard used

Indian Standard Method:	**British Standard Method**
- BridgeRules (for loading) - Steel bridge code	- BS 5400-1 - BS 5400-2 - BS 5400-3 - SCI Page 318

2) Dead Load

The dead load of railway bridge structure includes the weight of sleepers, the rails, the floor system and supporting structure.

Indian Standard Method

In design, the weight of structure is assumed. This method is only applicable for a simple structure bridge. Here, an approximate self-weight of a structure is assumed and checked after structure is designed. Design should be repeated if there is a large difference between the assumed value and the calculated value. Hence it is important to assume the dead weight with sufficient accuracy so that the repetition is not necessary. However, it is difficult to formulate the expressions predicting the self-weight of the bridge accurately because the amount of steel in a bridge of given span and for given service depends on the number of panels, the depth of girder or truss, the specifications under it is designed, the individuality of the designer and other factors. It should be good to assume the dead weight of the structure by comparing it with the similar types of structures which are in uses.

i) For Truss Bridges (Hudson's Formula)

Hudson's formula gives the dead weight of bridge as a function of bottom chord area. In metric unit, Hudson's formula gives the following rules:

$$w = 7.85A$$

Where, w = weight of two trusses and their bracing in kg per meter of bridge.

A = net area of the largest tension chord in sq.cm

In calculating the maximum stress in tension chord, it is necessary to assume in advance the weight of the trusses and bracing. The above formula was derived pn the basis that the average weight per meter of truss could be represented as proportional to the net area of the largest tension chord as follows:

Bottom chord	= 1.00 A
Top chord	= 1.25 A
Web System	= 1.25 A
Details	= 1.00 A
Bracing	= 0.50 A
Hence, total for one truss	= 5.00 A

If weight of the steel is taken as 0.875 kg per meter length of one sq.cm of area, the weight in kg per meter of both trusses and bracings, w will be as follows:

$$w = 2 \text{ X } 5A \text{ X } 0.785$$

$$= 7.85A$$

The above formula does not assume any loading and allowable stresses and can be used with any specifications.

ii) Plate girder bridges (Waddell's Extensive Data)

Weight of steel plate Girder Bridge carrying single tract railway loading can be expressed as follows:

$$w = kL\sqrt{W}$$

Where,

w = weight of the two girders together with bracing in kg per m length of bridge

k = a constant, equal to about 16.5 for deck bridges

L = effective span of bridge, m

W= heaviest axle load of engine, t

Therefore, using axle load for main line loading as 229t and branch line loading as 17.3 t from Figure 16-2 in appendix 1, we get the weight in kg per m of both girders and bracings

w = 79.0L (Main Line) and w = 68.5L (Branch Line)

<u>British Standard Method</u>

Just like the Indian Standard Method, the dead load for whole structure shall be accurately assumed before calculating the actual weight. The factor, Y_{fL} should be applied to all parts of the dead load. The factors are as follows:

Table 5: Dead Load Factors

	For Ultimate Limit State	For Serviceability Limit State
Steel	1.05	1.0
Concrete	1.15	1.0

The value of Y_{fL} superimposed dead load is different and should be taken as follows:

Table 6: Superimposed Dead Load

For Ultimate Limit State	For Serviceability Limit State
1.75	1.2

However, if the value of Y_{fL} specified above causes a less severe total effect than using the value of 1.0, the values of 1.0 should be considered.

Superimposed dead load:

The factor Y_{fL} should be applied to all parts of superimposed dead load, irrespective of whether these parts have an adverse or relieving effect, shall be taken for all five load combinations.

For the ultimate Limit State	For the serviceability limit state
1.75	1.20

*this value may be reduced not less than 1.2 for ultimate limit state and 1.0 for serviceability limit state.

3) Live Loads

<u>Indian Standard Methods</u>

Live loads due to train loadings have been specified in 'Bridge Rules' for various types of tract. Some of these loadings are given below:

Broad Gauge

i) Standard Main Line (M.L) loading of 22.9 tonnes axle loads and train of 7.67 tonnes per meter behind the engines is specified in Figure 16.2 (a).

ii) Standard branch Line (B.L) loading for 17.3 tonnes axle laods and a train of 5 tonnes per meter behind the engines is specified in Figure 16.2 (b).

It is complicated to calculate the maximum force in all truss members due to the moving train with concentrated wheel loads. For simplicity, Bridge Rules have given equivalent uniform distributed loads for computing the maximum bending moment and shear forces. The equivalent uniformly distributed loads for various type of loading have been given in Appendix 2.

British Standard Method

According to BS 5400-2 clause 8.1, the standard railway bridge consists of two types as follows:

RU Loading

This loading allows all combination of vehicles currently running or planned to run on railways and to be used for design of bridge carrying the main line railways of 1.4m gauge and above. This nominal load consists of four 250kN concentrated loads preceded and uniformly distributed load of 80kN as shown in the figure below:

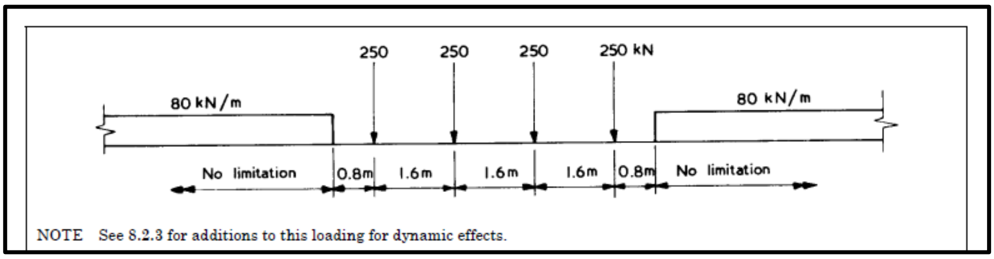


Figure14: RU Loading

RL Loading

This is a reduced loading for use only passenger rapid transit railway systems on line where main line locomotives and rolling stock do not operate. The nominal load consists of a single concentrated load coupled with a uniformly distributed load of 50kN/m for loaded lengths up to 100m. for excess length of 100m, the distributed nominal load shall be 50kN for the first 100m and shall be reduced to 25kN/m for lengths in excess.

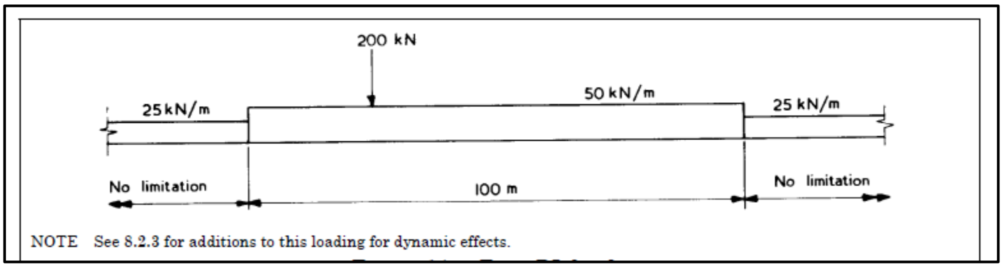


Figure 15: RL Loading

4) Impact Load

Indian Standard Methods

The impact factors depends on many aspects such as the type of loading, speed, type of structure, material of structure, loaded length and etc. design codes generally gives the different expressions for impact factor for railway bridges, highway bridges, combined road-rail bridges, foot bridges, steel bridges, pipe culvert or arch bridges etc. for a particular type of loading and bridge, an impact factor can be specified involving one parameter such as loaded length. All other parameters are taken care of by the constant in the expression for impact factor.

For broad and meter gauge railway bridges of steel carrying a single track, the impact factor is given by the following expression.

$Impact\ Factor = \frac{20}{14+L}$, L = loaded length of span in meters.

For design of chord members, the whole span should be loaded but for maximum stress in web members, only one part of the span is to be loaded. For floor beams, the loaded length will be equal to the two panel lengths in the case of intermediate floor beams and one panel length in the case of end floor beams. For stringers, the loaded length should be one panel length. On sleepers, the whole load comes suddenly and the maximum impact i.e. 1.0 should be used.

British Standard Method

In British standard, the dynamic factor for RU loading and RL loading is given separately.

Table 7: Dynamic factor for RU loading

Dimension L	**Dynamic factor for evaluating**	
	Bending moment	**Shear**
Up to 3.6m	2.00	1.67
From 3.6 to 67	0.73 + (2.16/$\sqrt{(L-0.2)}$	0.82 + (1.44/$\sqrt{(L-0.2)}$
Over 67	1.00	1.00

Dynamic factor for RL loading:

The dynamic factor should be taken as 1.2 when evaluating the moments and shears except for unballasted tracks where for rail bearers and single-track cross gorders, the dynamic factor shall be increased to 1.40. However, the dynamic factor applied to temporary works may be reduced to unity when rail traffic speeds are limited to not more than 25km/h.

5) Load due to curvature of the track

Indian Standard Methods (Bridge Rule Clause 2.5)

Where a railway bridge is situated on a curve, all portions of the structure are affected by centrifugal force of the moving vehicles. The centrifugal force can be calculated as follows:

$$\boxed{centrifugal\ force, C = \frac{WV^2}{127R}\ (MKS\ unit)\ \text{ or }\ C = \frac{WV^2}{12.95R}\ \text{SI unit}}$$

Where, C = centrifugal force in tonne/kN per meter of span

W = Equivalent Distributed live Load in tonne/kN per meter run

V = Maximum speed in km per hour

R = Radius of curvature in metres

For railway bridges, the following loads must be considered

- The extra load on one girder due to the additional reaction on one rail and tu the lateral displacement of the track calculated under the following load condition
  - i) Live load running at the maximum speed
  - ii) Live load standing with half normal dynamic arrangement
- The horizontal load due to centrifugal force for which may be assumed to act at a height of 1830mm for “25t loading 2008“ for BG, 3000mm for “DFC loading (32.5 axle load)” for NG and 1450mm for MG (ablove rail level)

Absolute minimum radii in Indian Railway laid down in SOD

- BG – 175m
- MG – 109m
- NG – 44m

Any speed higher than 120kmph is considered as high speed.

From Indian Policy circular No.7:

- BG – up to 110 kmph

- MG – 75 kmph

The maximum speed of train in Indian Railway is 160 kmph.

British Standard Method

The nominal centrifugal force, F_c, in kN, per track acting radially at height 1.8m above rail is calculated using the following formula:

$$F_c = \frac{P(V_t+10)^2}{127r} f$$

Where,

P = static equivalent uniformly distributed load for bending moment when designing for RU loading; for RL loading, a distributed load of 40kN/m multiplied by L is deemed sufficient.

r = radius of curvature (in m)

v_t = greatest speen envisaged on the curve in question (in km/h)

$f = 1 - \left[\frac{v_t - 120}{1000}\right] \times \left[\frac{814}{v_t} + 1.75\right] \times [1 - \sqrt{\frac{2.88}{L}}]$, for L greater than 2.88m and v_t less than 120km/h.

f = unity for L less than 2.88m or v_t less than 120km/h

L = loaded length of the element being considered

British Standard Method

Unlike the Indian Standard Method of calculating wind speed, BS Methods is more detail in calculating the wind speed. The wind loads given in BS 5400 have been derived from general wind tunnel tests and conservative. Nominal transverse wind load P_t (in N) is taken at the centroids of the appropriate areas and horizontally unless local conditions change the direction of the wind and is calculated as follows:

$$P_t = qA_1C_D$$

Where, q = dynamic pressure head (= $0.613V_{c,}$ in N/m^2, V_c in m/s)

A_1= solid area in mm^2

C_D = drag coefficient

Value of $C_{D:}$

Single plate girder = 2.2

Two or more plate girder = 2.2 each girder without any allowance for shielding

Combined girders = C_D = 2(1+c/20d), but not more than 4. Where c is the distance center to center of adjacent girders and d is the depth of the windward girder.

6) Racking Forces

Indian Standard Methods (Bridge Rules)

Due to small lateral movement of trains even when moving on straight track, lateral forces are applied by the train to the track. This horizontal lateral load is taken equal to 600kg/m and treated as moving load. This load is considered only in the lateral braces. Its effect is not considered in design of chord members. For bridges with effective span less than 20m, lateral bracing may be designed for a combined lateral moving load of 900kg/m due to wind and racking forces treated as moving load in addition to centrifugal force if any.

British Standard Method

In BS 5400-2, racking force is described as **nosing** in clause 8.2.8 where a lateral loads applied by the trains to the track should be taken as a single nominal load of 100kN. It acts horizontally in either direction at right angles to the track at rail level and a point in the span to produce maximum effect in the element which is under consideration. Also, the vertical effects of this load in secondary elements such as rail bearers should be considered.

7) Longitudinal Loads

Indian Standard Methods

The longitudinal load act in the direction of the span and are caused due to the following reasons:

i) The tractive effort of the driving wheels of the locomotives.
ii) The braking effect resulting from the application of the brakes to all braked wheels.
iii) The resistance offered by bearings to the movement at the roller end.

The frictional load due to the frictional resistance at the roller bearing will be equal to the vertical reaction at bearing multiplied by the coefficient of friction. The coefficient of friction for different type of bearing is given in the table below

Table 8: Coefficient of friction for Indian Standard Method

Types of bearing	Coefficient of friction
Roller bearing	0.03
Sliding bearings of steel on hard copper alloy bearings	0.15
Sliding bearings of steel on cast iron or steel bearings	0.25
Sliding bearings of steel on ferrobestos	0.20

British Standard Method

For bridge supporting ballasting track, up to one third of the longitudinal load may be assumed to be transmitted by the track to resistance outside the bridge structure, provided that no expansion switches or similar rail discontinuities are located on, or within, 18m of either end of the bridge.

Structure or element carries single tracks shall be designed to carry the larger of the two loads produced by the traction and braking in either direction parallel to the track. Where a structure or element carries two tracks, both tracks shall be considered as being occupied simultaneously. Where the tracks carry traffic in opposite directions, the load due to braking shall be applied to one track and the load due to traction to other. Structures and elements carrying two tracks in the same direction shall be subjected to braking or traction on both tracks, whichever gives the greater effect.

Table 9: Longitudinal Loading for RU and RL loading

Standard Loading Type	Load arising from	Loaded length (m)	Longitudinal load kN
RU	Traction (30% of the load on driving wheels)	up to 3	150
		3-5	225
		5-7	300
		7-25	24(L-7)+300
		Over 25	750
	Braking (25% of the load on braked wheels)	up to 3	125
		3-5	187
		5-7	250
		Over 7	20(L-7)+250

RL	Traction (30% of the load on driving wheels)	Up to 8	80
		8-30	10 k N/m
		30-60	300
		60-100	5 kN/m
		Over 100	500
	Braking (25% of the load on braked wheels)	Up to 8	64
		8-100	8 kN/m
		Over 100	800

8) Lateral Bracing

<u>Indian Standard Method</u>

When the flanges are reduced in thickness or breadth between the points of effective lateral restraint, the compressive stress of maximum section is calculated using a reduction factor k1 which is calculated using the table below:

Table 10: Reduction factor for Indian Standard Method

'	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
k1	1.0	1.0	1.0	0.9	0.8	0.7	0.6	05	0.4	0.3	0.2

¥ = coefficient to allow for reduction in thickness or breadth of flanges between points of effective lateral restraint

k1= a ratio of the total area of both flanges at the point of least bending moment to the corresponding area at the point of the greatest bending moment between such point of restraint.

- The flanges should not be reduced in breadth to give a value of ¥ lower than 0.25

In case of bridge or crane girder where dynamic effect of live loads are important, it may be necessary to restrict plate thickness to 20mm, if steel of IS226 is used, from welding consideration. In such cases, more than one plate may be required. The change in flange plate size is accomplished by using various length plates of different thickness.

If the reduction of thickness of the thicker plate is impracticable or the joint is not subject to dynamic load, the weld mild should be built up at the junction to dimension of 25% greater than those of the thinner part.

Similarly, k_2 is a coefficient which depends on the ratio w which is defined as ratio of the moment of inertia of the compression flanges to the sum of the moment of inertia of both flanges. It is calculated about its own axis, parallel to y-axis of the girder.

Table 11: K2 coefficient

'w	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K2	0.5	0.4	0.3	0.2	0.1	0	-0.2	-0.4	-0.6	-0.8	-1.0

- Note that when beam is symmetrical, w will become 0.5 giving k_2=0. Thus the additive factor k_2 vanish for a uniform symmetrical section.
- The maximum permissible compressive stress σ_{bc} for laterally unsupported beam with unequal flanges mat be obtained by using the Merchans Rankine formula:

$$\sigma_{bc} = 0.66 \frac{f_{cb} \times f_y}{[(f_{cb})^n + (f_y)^n]^{1/n}}$$

F_y = yield stress of steel

n = a factor = 1.4

f_{cb} = elastic critical stress in bending

$$fcb = k1(X + k2Y)\frac{C_2}{C_1}$$

Where X=

$$X = Y\sqrt{1 + \frac{1}{20}[\frac{L \times T}{\gamma y D}]}$$

$$Y = \frac{26.5 \times 10^5}{(\frac{L}{\gamma y})^2}$$

Based on Bridge Code clause 5.13, all span shall be provided with end cross frame and a lateral bracing system extending from the end to end of sufficient strength to transmit the bearing from wind or seismic, racking and centrifugal forces if any as specified in the Bridge Rule. Deck type span of over 20m effective span should be provided with end cross frame and a lateral bracing system between the top flange, of sufficient strength to transmit to the bearing the total lateral load due to wind or seismic, racking and centrifugal force and with a lateral bracing system between the bottom flanges of sufficient strength to transmit ¼ of the total lateral loads.

Wind Loading

Wind loads are the lateral loads which are caused due to the obstruction in the flow of wind by the bridge structure and the moving load on it. The intensity of wind pressure depends on the wind velocity which in return depends upon the height of the structure above the mean retarding surface.

For broad gauge railway bridges, the bridges shall be assumed not to carry any live load when the wind pressure exceeds 150 kg/m^2 . The **wind load is calculated by multiplying the wind pressure and the exposed area.** The exposed area consists of the area of the moving load, the horizontally projected area of the span (on windward side) not covered by moving load on leeward side. The area of the moving load will be taken from 600mm above rail level to the top of the highest stock for which the bridge is designed.

In plate girder bridges, the wind pressure on leeward girder depends on the spacing of the girder. If spacing is less than half the depth, no area of leeward girder is considered. If spacing is between full depth and one and a half, 50% area is considered. And for spacing between one and half and twice the depth, full area of leeward girder is taken in calculating wind load.

The lateral bracing between compressive flanges of all span shall in addition be designed to resist a transverse shear at any section equal to 2(1/2) percent of the total compressive force carried by both flanges at the section under consideration. where, however, the transverse sleepers rest directly on compressive flanges and offer against buckling of their flanges. this additional transverse shear may be ignored.

Existing plate girder with transverse sleepers need not be condemned on account of the absence of lateral bracings, provided they show no sign of distance or under internal oscillation

Seismic Force (IS 1893-1984)

The seismic coefficient method shall be used for computing the seismic force. Response spectrum is not needed for this design. The basic **horizontal seismic** coefficient ($\propto_0$) is given in Table 2 (IS 1893-1984) and clause 2.12.3.3 in Bridge Rules.

$$\boxed{\alpha = (\beta)(\mathrm{I})(\propto_0)}$$

Where, β = a coefficient depending upon the foundation system

I = a coefficient depending upon the importance of the structure

The design of vertical seismic coefficient can be takn as half of the design horizontal seismic coefficient. for horizontal acceleration, the stress can be calculated as the effect of force applied horizontally at the centre of main elemet of the bridge units which it is conveniently divided for the purpose of the design. the force shall be assumed to come from any horizontal direction.

For design of super and sub structure of the bridges in different zones, the seismic force may be considered as below:

Zone I-III – seismic force shall be considered in case of bride of overall length more than 60m or span more than 15m

Zone IV & V – for all span.

Horizontal seismic load force due to the live load on the bridge shall be ignored acting in the direction of the traffic but when acting perpendicular to the traffic, this is to be considered for 50% of the design live load without impact.

From clause 2.12.7:

$$\boxed{Seismic\ force, F = W_m \times \alpha_n}$$

British Standard Method

Wind Load calculation

Wind Gust Speed

$$\boxed{V_c = vK_1S_1S_2}$$

Where, v = hourly mean speed (5.3.2.1.1)

K = wind coefficient (5.3.2.1.2) (taken as 1 for highway, railway and foot/cycle track)

S1 = funneling factor (5.3.2.1.3)

S2 = gust factor (5.3.2.1.4/5)

Wind Load, Pt

$$Pt = qAC_D$$

Where, q = dynamic pressure head (0.613 Vc^2) N/m^2

A = solid area (in m2)

Cd = drag coefficient (5.3.3.2)

9) Bearing stiffener

Indian Standard Method:

End bearing stiffener

Clause 5.10.1.1 of steel bridge code states that stiffeners over points of support and load bearing stiffeners should have sufficient area to carry the entire reaction without exceeding the specified intensity of working stress for struts having a length equal to three-quarters of the depth of the girder. The section of the stiffener may be assumed to include a length of the web plate equal to the overall width of the stiffener.

Whereas clause 6.7.5.3 of IS 800:1984 allows the consideration of length of girder 20t on both sides of stiffener to act with the stiffener. The end bearing stiffener can be considered to have an effective length of 0.7 times the length.

Intermediate Stiffener (CL 6.7.4.1 IS 800:1984)

When the thickness of the web is less than limit specified in CL 6.7.3.1, it has to be rechecked and intermediate stiffener is needed. Clause 6.7.3.1 states that the thickness of web plate shall not be less than the following:

a) For unstiffened webs, the thickness should be greater than

$$\frac{d_1\sqrt{\tau_{va,cal}}}{816} \; and \; \frac{d_1\sqrt{fy}}{1344} \; but \; should \; not \; be \; less \; than \; \frac{d_1}{85}$$

Where $\tau_{va,cal}$ = calculated average stress in the web due to shear force

d = height of the web

b) The code stipulates the requirement of web thickness when the intermediate vertical stiffeners are provided as the greater of

i) 1/180

ii) $\frac{d_2\sqrt{fy}}{3200}$

iii) But not less than $d_2/200$

The vertical stiffener shall be designed so that Ixx is greater than 1.5 x d^3 x t^3/c^2 where c is the spacing.

Note to clause 6.7.3.1 that in no case shall the greater clean dimension of a web panel exceed 270t; nor the lesser clean dimension of the same panel exceed 180t, where t is the thickness of the web plate.

Panel Dimension Requirement

Clause 6.7.4.1 stated that in no case shall the greater unsupported clean dimension of a web panel exceed 170t nor the kisser unsupported clean dimension f the panel exceed 180t, provide a vertical stiffener at spacing of 170t or 180t whichever is used.

The connection between Intermediate Stiffener and plate girder web (CL 6.7.4.6)

intermediate stiffener (Vertical and Horizontal) not subjected to external loads shall be connected to web by rivets or welded, so as to withstand a shearing force, between each component of the stiffener and the web of not less than

$$125t^2/h,$$

Where t is the thickness and h = width of the stiffener$\sqrt{355}/\sigma_y$

British Standard Method

Web stiffener (CL 9.3.3.2)

The opening in the web may be unstiffened provided that

a) The overall greatest internal dimension does not exceed 1/10 depth of the web, nor for the longitudinal stiffened web, 1/3 depth of the panel containing the opening.
b) The longitudinal distance between the boundaries of the adjacent opening is at least three times the maximum internal dimension.
c) Not more than one of the opening is provided at any cross section.

Flanges stiffener (CL 9.3.2.1)

For unstiffened flanges in compression, the ratio b_{f0}/t_{f0} should not exceed

$$12\sqrt{355/\sigma y}$$

Part 2:

Weight comparison of both design methods when the span is varied with fixed yield strength of 340N/mm^2

Fy= 340, span =20m

Indian Standard Method

Table 12: Weight for 20m span using Indian Standard Method

	Section	Size	Area (m2)	Density kN/m3	Weight per m (N/m)	No	Length (m)	Weight (kN)
Main girder			0.0396	77		2	20	121.968
top bracing	diagonal	100 x 65 x 8			99	20	2.828427125	5.600285707
	strut	90x90x8			108	9	2	1.944
	end strut	90x90x8			108	2	2	0.432
end cross frame	diagonal	100 x 65 x 8			99	4	2.828427125	1.120057141
	bottom strut	70x70x8			63	2	2	0.252
intermediate	bottom strut	70x70x9			63	9	2	1.134
	diagonal	100 x 65 x 8			99	20	2.828427125	5.600285707
Stiffener	end stiffener	90 x 21			1570	4	0.09	565.2
	intermediate stiffener	80 x 10			785	12	0.08	753.6
track					3000	1	20	60
							Total	1516.850629

British Standard Method

Table 13: Weight for 20m span bridge using British Standard Method

	Section	Size (mm)	Area (m2)	Density kN/m3	Weight per m (N/m)	No	length (m)	Weight (kN)
Main girder			0.06104	77		2	20	188.0032
top bracing	diagonal	80 x 80 x 10			119	20	2.82842712	6.731656557
	strut	80x80x6			73.4	9	2	1.3212
	end strut	80x80x6			73.4	2	2	0.2936
end cross frame	diagonal	80 x 80 x 10			119	4	2.82842712	1.346331311
	bottom strut	60 x 60 x10			86.9	2	2	0.3476
intermediate	bottom Strut	60 x 60 x10			86.9	9	2	1.5642
	diagonal	80 x 80 x 10			119	20	2.82842712	6.731656557
Stiffener	end stiffener	200x20			1570	4	0.2	1256
	intermediate stiffener	80x10			785	4	0.08	251.2
Track					3000	1	20	60
							Total	1773.539444

<u>Fy= 340, span =40m</u>

Indian Standard Method

Table 14: Weight for 40m span bridge using Indian Standard Method

	Section	Size	Area (m2)	Density kN/m3	Weight per m (N/m)	No	Length (m)	Weight (kN)
Main girder			0.09511663	77		2	40	585.91844
top bracing	diagonal	100 x 65 x 8			99	20	2.828427125	5.600285707
	strut	90x90x8			108	9	2	1.944
	end strut	90x90x8			108	2	2	0.432
end cross frame	diagonal	100 x 65 x 8			99	4	2.828427125	1.120057141
	bottom strut	70x70x8			63	2	2	0.252
intermediate	bottom strut	70x70x9			63	9	2	1.134
	diagonal	100 x 65 x 8			99	20	2.828427125	5.600285707
Stiffener	end stiffener	90 x 36			2750	4	0.09	990
	intermediate stiffener	80 x 15			1178	8	0.08	753.92
track					3000	1	20	60
							Total	2405.921069

British Standard Method

Table 15: Weight for 40m span using British Standard Method

	Section	Size (mm)	Area (m2)	Density kN/m3	Weight per m (N/m)	No	length (m)	Weight (kN)
Main girder			0.209312	77		2	40	1289.36192
top bracing	diagonal	80 x 80 x 10			119	20	2.82842712	6.731656557
	strut	80x80x6			73.4	9	2	1.3212
	end strut	80x80x6			73.4	2	2	0.2936
end cross frame	diagonal	80 x 80 x 10			119	4	2.82842712	1.346331311
	bottom strut	60 x 60 x10			86.9	2	2	0.3476
intermediate	bottom Strut	60 x 60 x10			86.9	9	2	1.5642
	diagonal	80 x 80 x 10			119	20	2.82842712	6.731656557
Stiffener	end stiffener	90x20			1570	4	0.2	1256
	intermediate stiffener	80x10			785	7	0.08	439.6
Track					3000	1	20	60
							Total	3063.298164

Fy = 340, span = 80

Indian Standard Method

Table 16: Weight for 80m span using Indian Standard Method

	Section	Size	Area (m2)	Density kN/m3	Weight per m (N/m)	No	Length (m)	Weight (kN)
Main girder			0.3139	77		2	80	3867.248
top bracing	diagonal	100 x 65 x 8			99	20	2.828427125	5.600285707
	strut	90x90x8			108	9	2	1.944
	end strut	90x90x8			108	2	2	0.432
end cross frame	diagonal	100 x 65 x 8			99	4	2.828427125	1.120057141
	bottom strut	70x70x8			63	2	2	0.252
intermediate	bottom strut	70x70x9			63	9	2	1.134
	diagonal	100 x 65 x 8			99	20	2.828427125	5.600285707
Stiffener	end stiffener	150 x 44			3530	4	0.09	1270.8
	intermediate stiffener	80 x 15			1178	14	0.08	1319.36
track					3000	1	20	60
							Total	6533.490629

British Standard Method

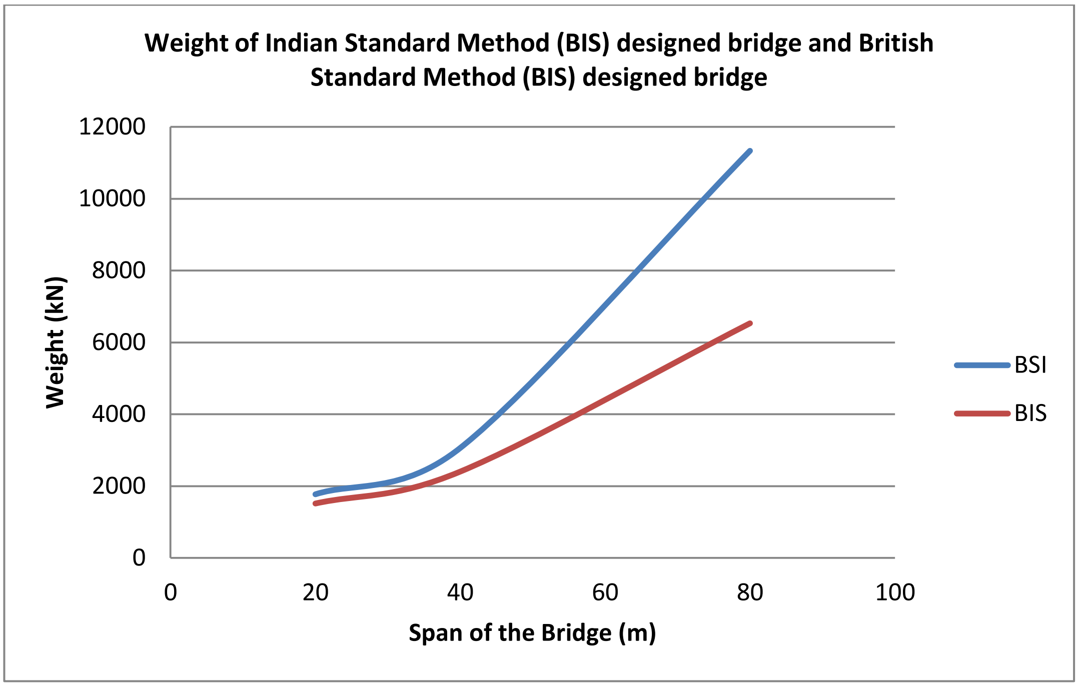
Table 17: Weight for 80m span using British Standard Method

	Section	Size (mm)	Area (m2)	Density kN/m3	Weight per m (N/m)	No	length (m)	Weight (kN)
Main girder			0.84112	77		2	80	10362.5984
top bracing	diagonal	80 x 80 x 10			119	20	2.82842712	6.731656557
	strut	80x80x6			73.4	9	2	1.3212
	end strut	80x80x6			73.4	2	2	0.2936
end cross frame	diagonal	80 x 80 x 10			119	4	2.82842712	1.346331311
	bottom strut	60 x 60 x10			86.9	2	2	0.3476
intermediate	bottom Strut	60 x 60 x10			86.9	9	2	1.5642
	diagonal	80 x 80 x 10			119	20	2.82842712	6.731656557
Stiffener	end stiffener	90x20			1570	4	0.09	565.2
	intermediate stiffener	80x10			785	7	0.06	329.7
Track					3000	1	20	60
							Total	11335.83464

CHAPTER 5

DISCUSSIONS

The graph below shows the differences in weight between the Indian Standard Method and British Standard Method



From the graph above, we can see that the total weight of bridge designed using British Standard Method is much higher than Indian Standard Method. Indian Standard Method is actually adopted from the British Standard Method. So, from the calculation and the code provisions, there is not much difference between these two methods. The only difference is that the loading calculation for both design. The EUDL for both designs is stated in their design code respectively depending on the span of the bridge itself. Other than that, the factors of the loading also different which contribute to the differences in the design bending moment and also design shear force.

Besides that, British Standard Method highlighted the ranges of the plate girder depth, the flanges thickness and also the ranges of the web based on the total span of the bridge. However, for Indian Standard Method, the geometry of the plate girder is calculated or determined by the total designed bending moment and shear forces. This answers the reason why there is big difference between the graphs shown.

There are several factors that are not considered in this design such as seismic loading and the centrifugal force. The lateral force that is taken into account is only wind loading. However, the wind calculation for both designs is different. The design method for lateral bracing for both design are the same. The only difference is that the size of the bracing used. All in all it will still depend on the design load for both cases.

Other factors that contribute to the differences are the number of stiffeners and size of stiffener used. The number of stiffener used in this design depends on the loading that need to be catered by the plate girder.

In terms of economical design, based on the graph shown above, it can be concluded that Indian Standard Method is much more economical as compared to the British Standard Method. This is roughly based on the size of the plate girder needed and the number of stiffener needed when the material yield strength used is the same. However, for this design, there are many others factors that are not considered. If the parameter for this design example is added, the changes in the graph may be different.

Table below shows the summarization of the plate girder size calculation and the design check extracted from the comparison made in the first part of the results.

Table 18: Comparison of design dimension calculation between Indian Standard Method and British Standard Method

Indian Standard Method	British Standard Method

a) Depth of the girder $$d = 1.1\sqrt{\frac{M}{\sigma_b \times t}}$$ M = design moment including all dead load and live load σ_b = allowable bending stress t = web thickness Depth of the girder = 1/8 to 1/12 of the total span.	b) Depth of girder $$\frac{L}{10} \leq D \leq \frac{L}{7}$$ L = Length of the bridge span
c) Minimum web thickness $$t = \frac{total\ shear\ force}{\tau \times web\ height}$$ τ= 0.4 (fy)	d) Minimum web thickness $$t = \frac{D}{125}$$
e) Flanges Area of flanges: $$A_f = \frac{M}{\sigma_b \times d_i}$$ M = design moment including all dead load and live load σ_b= 0.65(fy) d = height of the web width of flange plate: $$width = \frac{Area}{Thickness}$$	f) Flanges width $$\frac{D}{4} \leq 2b \leq \frac{D}{3}$$ Flanges thickness $$\frac{b}{12} \leq T \leq \frac{b}{5}$$

Flanges outstand: $$= \frac{width\ flange\ plate - web\ thickness}{2}$$ Limiting value of outstand: $$limiting\ value = 12t$$	

From the above table, it can be concluded that the depth of the plate girder for Indian Standard Method is calculated based on the design moment and the yield strength of the material used while the thickness of the plate girder is estimated using rules of thumbs and experiences. However for the British Standard Method, the depth of the plate girder is fixed at some ranges which are in between 1/10 to 1/7 of the total span of the bridge.

The minimum web thickness of the plate girder of Indian Standard Method is determined by from the total shear strength and the yield strength of the material used while for British Standard Method is determined based on the ratio of the depth of the plate girder to 125. This contributes to the differences in the size needed for both design method when the span increases.

In terms of the design check, both design method adopted the same design check formula. The design check for these both methods is summarized in the table below.

Table 19: Design check comparison of Indian Standard Method and British Standard Method

Indian Standard Method	British Standard Method

a) Check for moment capacity $$Design\ Moment,$$ $$Md = Ze \times \frac{f_y}{\gamma_{mo}},$$ Ze = Section Modulus Fy = Yield Strength γ_{mo} = material factor	a) Check for moment capacity $$Mc = P_{yf} \times A_f \times hs$$ $$P_{yf} = yield\ strength$$ $$A_f = Area\ of\ the\ flanges$$ hs = centre to centre distance between flanges
b) Check for Shear buckling Simple post critical method in Clause 8.4.2.21 (IS 800:2007) $$\frac{c}{d} \geq 1$$ c = spacing of the stiffener, Simple post critical method: Vn = Vcr $V_{cr} = A_v \times \tau_b$ τ_b = shear stress corresponding to web bucklng, determined as follows: • When λ ≤ 0.8, $$\tau_b = \frac{f_{yw}}{\sqrt{3}}$$ • When λ < 0.8 < 1.2, $\tau_b = [1- 0.8(λ-0.8]$ $\frac{f_{yw}}{\sqrt{3}}$ • When λ> 1.2 $\tau_b = f_{yw}/\sqrt{3}$ λ^2 Where, λ = non dimensional web	a) Check for shear buckling if d/t > 66.2 e $$\varepsilon = \sqrt{250/fy}$$ Check for shear buckling resistance, Fv ≤ Vb Where Vb = Vw = d x t x qw d = depth of the web t = thickness of the web qw = shear strength of the web

slenderness ratio for shear buckling stress	

For this design example, the total load calculated for both design is different. For Indian Standard Method, the dead load is calculated based on the the formula $w = kL\sqrt{W}$ and for live load, the EUDL is taken from the Table specified in the Indian Code. The difference in the EUDL specified for both codes also affect the total design loading. This Equivalent Uniform Distributed Load is fixed according to the span of the bridge. For British Standard Method, this designed EUDL is taken from the BS 5400:2. For British Standard Method, the designed dead load is calculated based on the estimation weight of the plate girder size and the assumed weight of the track.

For British Standard Method, the entire calculated designed load is controlled by the Limit State Coefficient which is the Ultimate Limit State and Serviceability limit State coefficient. This coefficient is different as compared to the impact factor coefficient used in the Indian State Method. British Standard Method has higher coefficient value as compared to the Indian Standard Method.

CHAPTER 6

CONCLUSIONS

As the conclusions, the only different between Indian Standard Method and the British Standard Method are the loading calculation and the shape limitation of both design method. In terms of the safety, in my opinion, the British Standard Method considers a higher safety as the limit state coefficient used in loading calculation is higher than the Indian Standard Method.

Indian Standard Method is actually adopted from the British Standard Method. Hence, there are not much different in terms of the design check calculation. However, the difference in terms of the loading coefficient and the size limitation or ranges gives a bigger range of weight difference between these both methods.

The objectives of this study is to compare the design method between the Indian Standard Method and the British Standard Method in terms of the provisions in related codes and documents which is summarized in the part 1 of the result. Besides that, this study is to compare the changes in the total weight of the designed bridge when the span is varied with the same yield strength used.

From the result obtained, it can be concluded that Indian Standard Method is more economical as compared to British Standard Method. This is because, the weight of the bridge designed using Indian Standard Method is lighter than the British Standard Method with the same yield strength of the material used. Logically speaking, the cost of the bridge will increase when the span of the bridge increases. Theoretically, there are span limitation standardized at certain country in order to cater the cost and the safety of the users of the bridge.

RECOMMENDATION

For this study, the author did not consider many aspects and only focused on the different in the design calculation formula and also the total weight of the bridge after the span is varied. To ensure the more accurate result obtained, for further study, the deflection of the designed bridge can be should be determined to ensure that the bridge is safe to be used.

Furthermore, the lateral loading calculation considered in this design only focused on the wind loading. In the real situation, seismic loading need to be considered as Indian is one of the most-frequent earthquake attacked due to seismic strike.

This study can be used as reference for the future engineer to ensure determine which codes is relevant for their design in order to design for an economical bridge.

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APPENDIX

Part 1: Steel Plate Girder Bridge Design using Indian Standard Method

Span: 20m

Yield strength: 250 Mpa

Loading Calculations:

Dead Load

The weight of the plate girder bridge carrying single track railway loading is expressed in the form of

$$\boxed{w = kL\sqrt{W}}$$

k = 52.177 (constant for deck bridge)

L = effective span of bridge

W = heaviest axle load of engine (245.2kN)

$w = 52.177 \times 20\sqrt{245.2}$

$w = 16340.65021$ N/m

Dead Load for one girder = $\frac{16340.65021}{2} = 8170.325\ N/m$

It is stated that the values are conservative and gives weight about 10 to 15% greater than the actual ones. Alternatively can use the formula

DL of each girder = 220L+600 = 220(20) + 600 = 5000 N/m

Weight of the track with sleepers = 8000 N/m

Dead load from track per girder = 8000/2 = 4000 N/m

Total Dead Load = 5000+4000 = 9000 N/m

Total Dead Load = 9000 (20) **= 180kN**

Live Load

Referring to Table Appendix XXIII (for 20m span bridge)

Live Load for bending moment = 2065.50kN

Live load for bending moment per girder = 2065.50/2 = 1032.75

Live load for shear force = 2272.42 kN

Live Load for shear force per girder = 2272.42/2 = 1136.21 kN

Impact factor (CDA) = 0.458

Maximum bending moment due to dead load = 180 x 20 /8 = 450kN

Maximum bending moment due to live load = 1032.75 x 20 /8 = 2581.875

Bending moment due to impact factor = 0.458 x 2581.875 = 1182.5 kNm

Design bending moment = 450 + 2581.875 + 1182.5 = 4214.4 kNm

Maximum shear force due to dead load = 180/2 = 90kN

Maximum shear force due to live load = 1136.21/2 = 568.1 kN

Shear force due to impact factor = 0.458 x 568.1 = 260.2 kN

Design Shear force = 90 + 568.1 + 260.2 = 918.3 kN

Geometry of plate girder is finalized based on

Span = 20m

Bending Moment = 4214.4 kNm

Shear Force = 918.3 kN

Web

Depth of the girder is varies between 1/8 to 1/12 of the total span

Depth of the web can be calculated based on the following formula

$$\boxed{d = 1.1 \times \sqrt{\frac{M}{\sigma_b \times t}}}$$

M = design moment including all dead load and live load

σ_b = allowable bending stress

t = web thickness (10mm)

Depth of girder = $d = 1.1 \times \sqrt{\frac{4214.2}{250 \times 10}}$

d = 1757.828 mm.

A small variation in d in the form of economic depth will not increase the weight of girder coincidently. For example a 10% change in depth d will increase the weight about 1 % only. Including economy of the fabrication also, it is found that d should be taken about 10% less than that gives by above equation. Finally depth of the web plate as rolled should be adopted so that cutting in the longitudinal direction is avoided. Such width are 800, 900, 1000, 1100, 1250, 1400, 1600, 1800, 2000, 2200 and 2500mm.

Hence, the depth of the girder is taken as **1800mm.**

Hence, minimum web thickness for shear consideration can be calculated as follows:

$$\boxed{t = (total\ shear\ force)/(\tau \times web\ height)}$$

where $\tau = 0.4$ (fy)

t = (918.3 x 1000N) / (0.4 x 250)(1800mm)

t = 5.1 mm < 10mm ok!

The minimum web thickness calculated is less than thickness provided for the web so ok

Flanges

Flanges area required can be calculated using formula below:

$$Af = \frac{M}{di \times \sigma}$$

Area of flanges:

$$A_f = \frac{M}{\sigma_b \times d_i}$$

M = design moment including all dead load and live load

σ_b= 0.65(fy)

d = height of the web

Area of the flanges required = (4214.4 x 10^6)/(0.65)(250Mpa)(1800mm)

= 14190 mm^2

Provide 30mm flange thickness, the width of the flange = 14190mm^2/30mm =473mm.

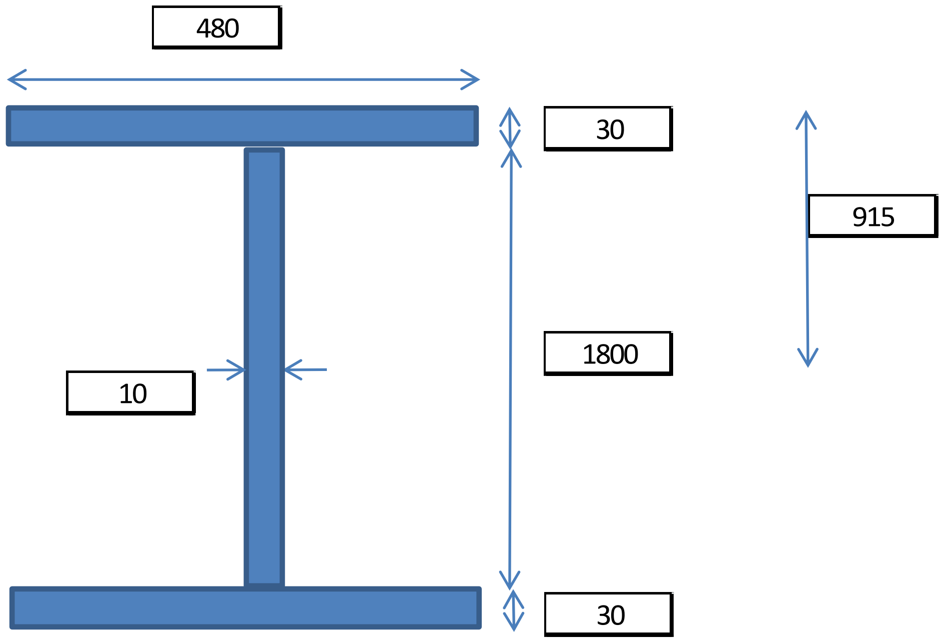
So the total width of flange plate = 480mm

Flange outstand = 480-10/2 = 235mm

The limiting value = 12 x flange thickness = 360mm > 235mm, ok.

Hence, the flange area provided = 480 x 30 = 1440mm^2 > 14190 mm^2 ok

Moment of inertia of plate girder, Ixx



$$Ixx = \frac{10 \times 1800^3}{12} + 2[\frac{480 \times 30^3}{12} + 480 \times 30 \times \left(\frac{1800}{2} + \frac{30}{2}\right)^2$$

$$= 28974240000 \text{ mm4}$$

Maximum bending tensile stress can be calculated as follows:

$$\boxed{\text{Maximum bending tensile stress} = M/I \times y}$$

Maximum bending tensile stress = (4214.4 x 10^6/ (28974240000) x 930

= 135.3 N/mm2 < 165 N/mm2 ok

Maximum bending tensile stress is less than allowable bending stress, so ok.

Curtailment

Flanges outstand = 235mm

If the flanges is curtailed to 20mm, the permissible maximum outstand will be 12(t) = 240mm > 235mm so ok. More curtailment cannot be done since it will rotate the permissible outstand correlation.

Moment of inertia of plate girder after curtailed is as follows:

$$Ixx = \frac{10 \times 1800^3}{12} + 2[\frac{480 \times 20^3}{12} + 480 \times 20 \times \left(\frac{1800}{2} + \frac{20}{2}\right)^2$$
$$= 20760160000 \text{ mm4}$$

$M_R = \sigma_b \times \frac{Ixx}{y/2}$

Maximum bending moment = 0.66(250) x 20760160000/920 = 3723.3 kNm

Design of end bearing stiffener

The maximum shear force calculated is 918.3 kN

The maximum area required = (918.3 x 1000) / (Permissible shear stress)

= 918.3 x 1000/ (0.75 x 250) = 4897.6mm^2

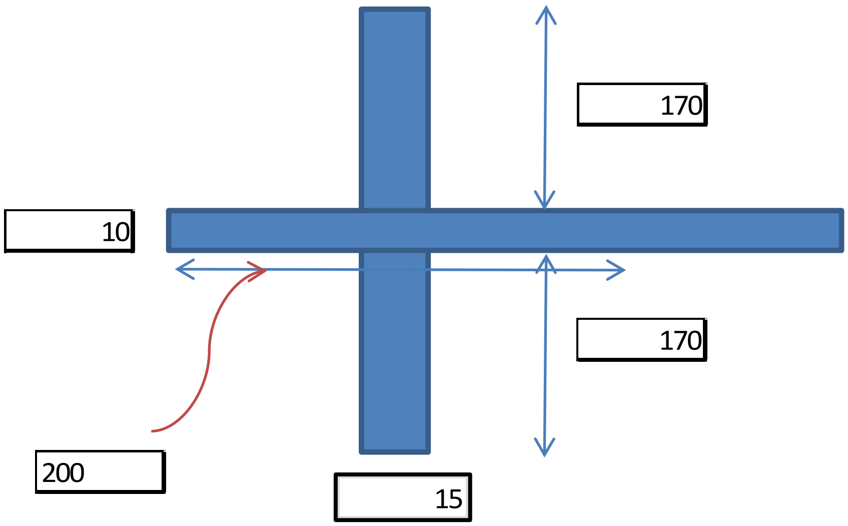
The limiting width of the stiffener = 480-10/2 = 3235mm

Trying 2 plates of 170mm width of stiffener, the thickness of the stiffener plate is

= 4897.6/(2 x 170) = 15mm

Outstand (170mm) should not be more than 12t = 12(15) = 180mm. ok.

Bearing area provided = 2 x 170 x 15 = 5100mm^2 > 4897.6mm^2 ok



So area of the stiffener = 2 x (170 x 15) + (2 x 200 x 10) = 9100mm^2

$$Ixx = \left[2\left(\frac{15 \times 170^3}{12}\right) + 15 \times 170 \times \left(\frac{170}{2} + \frac{10}{2}\right)^2\right] + \frac{400 \times 10^3}{12}$$
$$= 53625833.33\text{mm4}$$

$$\gamma = \sqrt{\frac{I}{A}} = \sqrt{\frac{53625833.33}{9100}} = 76.76554382$$

The end bearing stiffener can be considered to have an effective length of 0.7 times the length (IS800:1984).

So the effective length of end bearing stiffener = 0.7 x 1800mm = 1260mm

Shear stress of the stiffener is provided in Table 5.1 in IS 800-1984

$$\lambda = \frac{l}{\gamma} = \frac{1260}{76.76554382} = 16.4$$

For λ = 16.4 and fy = 250 N/mm^2, So the safe load = 148.73 x 9100 = 1353352N > 918.3 kN ok!

So, provide 170 x 170 x 15 plates of stiffener.

Intermediate stiffener

The intermediate stiffener is needed if the thickness of web plate is greater than $\frac{d_1\sqrt{\tau_{va,cal}}}{816}$ and $\frac{d_1\sqrt{fy}}{1344}$ but should not less than $\frac{d_1}{85}$.

$\tau_{va,cal}$ = calculated average stress in the web due to shear force, and d is the web height.

Minimum web thickness = $\frac{d_1\sqrt{\tau_{va,cal}}}{816} = \frac{1800\sqrt{\frac{918.3\times10^3}{1800\times10}}}{816} = 15.76mm$

Minimum web thickness = $\frac{d_1\sqrt{fy}}{1344} = \frac{1800\sqrt{250}}{1344} = 21.18mm$

Minimum web thickness = $\frac{d_1}{85} = \frac{1800}{85} = 21.18mm$

The web thickness provided is 10mm which is less than 21.18mm. So, intermediate stiffener is needed.

The requirement of vertical stiffener to be provided is that the web thickness provided should be greater than or equal to

i) 1/180 of the clean panel dimension

ii) $\frac{d_2\sqrt{fy}}{3200}$

iii) But not less than $d_2/200$

Clean panel dimension is assumed to be 180t = 1800mm. The web thickness of 10mm provided fulfills the requirement of the vertical intermediate stiffener to be provided.

Vertical stiffener is provided at spacing 180t = 1800mm

Number of stiffener needed is 20000/1800 = 12

Actual clean panel dimension = 20000/12 = 1666.66mm, so take 1660mm.

Spacing of the stiffener = (1660/1800) d = 0.922d.

d/t = 1800/10 = 180

From table 6.6A of IS 800 – 1984, for d/t = 180 and c=0.922d, the τ_{va} is 83.56 N/mm^2 > $\tau_{va,cal}$ **ok**

Outstand of vertical stiffener = 12t = 12x10 = 120mm

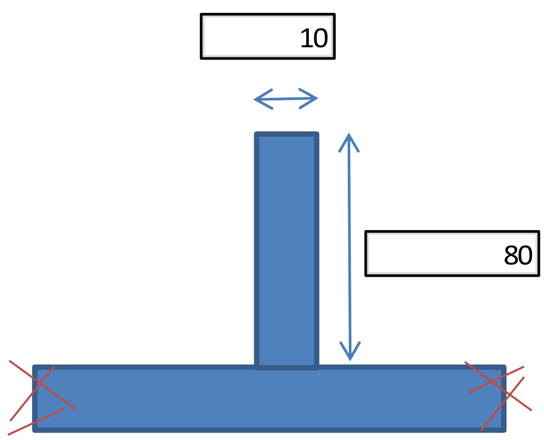
Minimum required thickness from shear consideration,

t = 918.3 x 10^3/ (1800 x 83.56) = 6.1mm

Try flat section **80mm x 10mm**

The vertical stiffener should be designed so that I is not less than 1.5 x d^3 x t^3 / c^2 where c is the spacing.

1.5 x d^3 x t^3 / c^2 = 1.5 x 1800^3 x 10^3 / 1660^2 =720579.8 mm4



Moment of inertia about face of web

$$Ixx = \left[\left(\frac{10 \times 80^3}{12}\right) + 10 \times 80 \times 40^2\right] = 1706666.7\text{mm4} > 720579.8 \text{ mm4 ok!}$$

Design of Lateral Bracing (Based on IS 875-3-1987)

Design of lateral bracing

Spacing of the girder = 2000mm

Wind load for this example is calculated based on Indian Code

Wind pressure calculated = 1500N/m^2

Depth of the train = 3.5 m

Depth of girder and track = 2.5 m

Wind pressure on train = 1500 N/m^2 x 3.5 = 5250 N/m

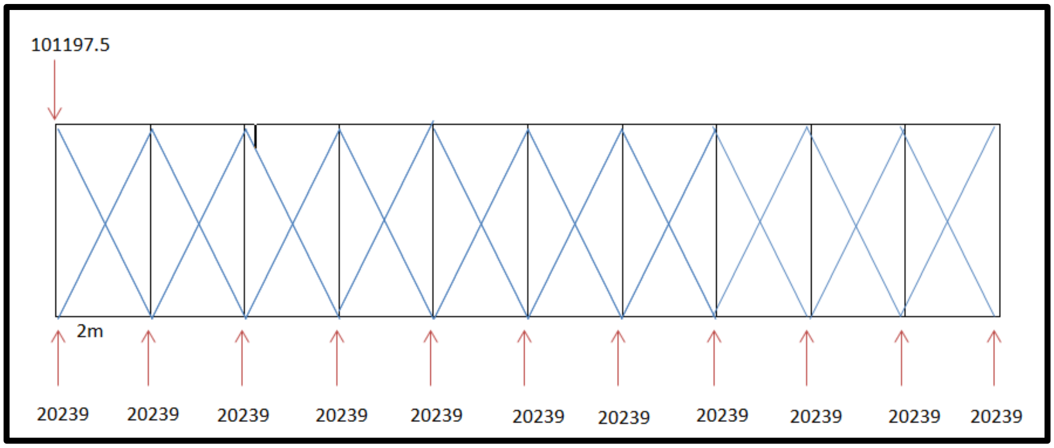
Wind pressure on windward girder = 1500 N/m^2x 2.5m = 3750 N/m

Wind pressure on leeward girder = 1500 N/m^2 x 2.5 x 0.25 = 937.5 N/m

Total wind force = 9937.5 N/m

Lateral load at each node = 9937.5 x 2m = 19875 N

At end reaction = 9937.5 x 20m/2 = 99375 N



Length of the lateral = $\sqrt{2^2 + 2^2}$ = 2.828 m

Force in end lateral = (99375-19875) x (2.828/2) = 112413 N

Try an equal angle of 90x90x8

A = 1379 mm^2 ; r = 17.5

l/r = 2828 / 17.5 = 161.6

From figure 37,

Ultimate compressive stress σ_c = 134 N/mm^2

Resistance capacity of the angle = 134 N/mm^2 x 1379 mm^2 = 184786 N > 114474.612N ok

Bottom lateral bracing

All force will be 25% of force in top lateral

Force in end lateral = ¼ (112413 N) = 28103.25 N

Use equal angle of 60 x 60 x 5

A = 582 mm^2 ; r = 18.2

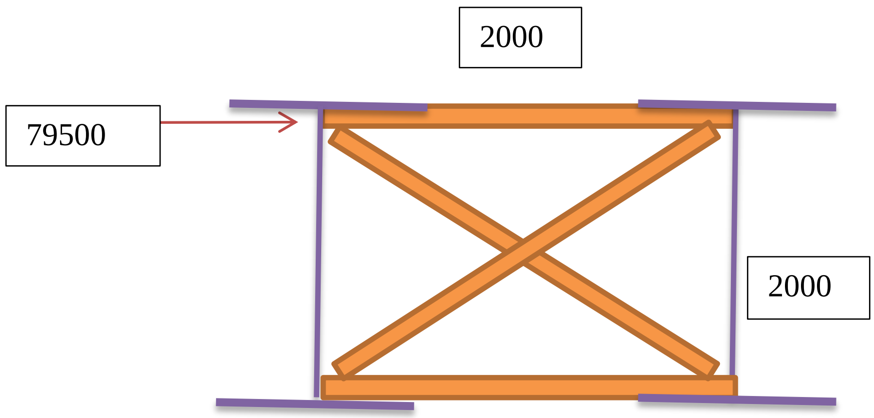
l/r = 2828/18.2 = 155

From figure 37,

Ultimate compressive stress σ_c = 77.05 N/mm^2

Resistance capacity of the angle = 77.05 N/mm^2 x 582 mm^2 = 44843 N > 28103.25 N ok

End cross frame



Top strut

Effective length of the top strut = 0.7(2000) = 1400mm

Using equal angle of 80 x 80 x 6

A = 935 mm^2 ; r = 24.4

l/r = 1400 / 24.4 = 57.37

From figure 37,

Ultimate compressive stress σ_c = 284.75 N/mm^2

Resistance capacity of the angle = 284.75 N/mm^2 x 935 mm^2 = 266241.25 N > 79500 ok

Bottom strut

Same as top strut

<u>Vertical diagonal of end cross frame</u>

Force in each diagonal = 79500 N x $\sqrt{2}$ / 2 = 56214.99 N

Length of diagonal = 2828 mm

Try 90 x 90 x 6

A = 1060 mm^2 ; r = 27.6

l/r = 102.464

From figure 37,

Ultimate compressive stress σ_c = 99.53 N/mm^2

Resistance capacity of the angle = 99.53 N/mm^2 x 1060 mm^2 = 105508 N > 56214.99 ok

Part 2: Steel Plate Girder Bridge Design using British Standard Method

Span: 20m

Yield strength: 340 Mpa

Effective span of the bridge = 20m

Overall depth of plate girder

$$\boxed{\frac{L}{10} \leq D \leq \frac{L}{7}}$$

$$2 \leq D \leq 2.86$$

So, choose D = 2000mm

Flange width, b

$$\boxed{500 \leq 2b \leq 666.67}$$

Hence b = 250mm

Flange Thickness, T

$$\boxed{\frac{b}{12} \leq T \leq \frac{b}{5}}$$

$$20.83 \leq T \leq 50$$

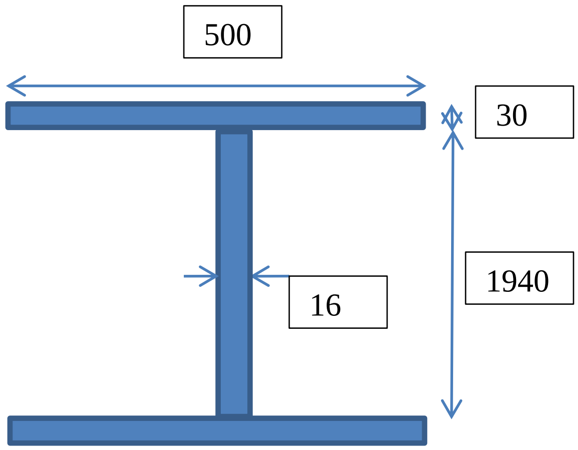
So, choose T = 30mm

Web thickness, t

$$\boxed{t = \frac{D}{125}}$$

$$t = 16$$

So, web thickness = 16mm



LOADING CALCULATION

Live Load

The nominal live load is taken to be EUDL and nosing loading with respect to the Ultimate Limit State and Service Limit State. EUDL is taken from Table 22 BS 5400:2

EUDL = 1 track x 3003 kN/20 m x 1 = 75.075

Nosing = 100kN x 1/20 x 2250/2000 = 5.625kN/m

Nominal Load (kN/m)		**ULS**		**SLS**	
		Factor	**Load**	**Factor**	**Load**
EUDL	75.075	1.4	105.105	1.1	82.5825
Nosing	5.625	1.4	7.875	1.1	6.1875

Total of factored ULS = 113 kN/m

Total of factored SLS loading = 88.77 kN/m

Total live load = 201.77 kN/m

Assumed Dead Load

Cross section area of main girder = 2 (0.5)(0.03) + (1.94)(0.016) x 2

= 0.12208 mm^2

				ULS		SLS (kN/m)	
	Area	Density (kN/m3)	Load	Factor	Load	Factor	Load
Main girder	0.12208	77	10.34018	1.1	11.37419	1.1	11.37419
Track		2	2	3.89	7.78	1	2
Services		0.8	0.8	1.2	0.96	1	0.8
				Total	20.11419		14.17419

Total factored ULS dead load = 20.114 kN/m

Total factored SLS dead load = 14.17419kN/m

Total dead load = 34.288 kN/m

Design Shear Force

Design Shear force due to dead load = w/2 = 17.14 kN/m

Design shear force due to live load = w/2 = 100.875 kN/m

Total shear force = 118.015 kN/m

Total shear force = 118.015 (20) = **2360.383872**

Fatigue Load = 1 track x 3003 kN/m x 10m = 75.075 kN/m

ULS factor for fatigue loading = 1.0, hence factored fatigue loading = 75.075 kN/m

Bending moment = $wl^2/8$

Ultimate Limit State (ULS)

Bending moment due to ULS (dead load) = (20.11419 kN/m x 20^2)/8 = 1005.70968 kN.m

Bending moment due to ULS (live load) = 112.98 kN/m x 20^2/8 = 5649 kN.m

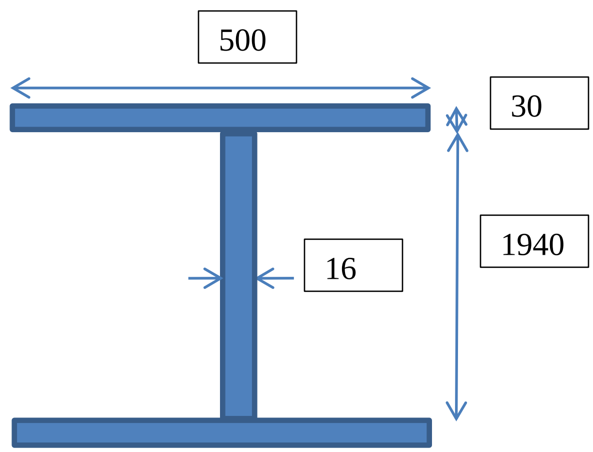
Bending moment due to ULS (fatigue) = 75.075 kN/m x 20^2/8 = 3753.75 kN.m

Serviceability Limit State

Bending moment due to SLS (dead load) = 14.17 kN/m x 20^2/8 = 708.70968 kN.m

Bending moment due to SLS (live load) = 88.77 kN/m x 20^2/8 = 4438.5 kN.m

Total design bending moment = **11802 kN.m**



Moment of inertia of plate girder, Ixx

Ixx = (16 x 1940^2)/12 + 2 [(500 x 30^3)/12 + (500)(300)$(970+15)^2$] = 3.8844 x 10^{10} mm^4

Since the criteria for the unstiffened web is not satisfied as in the clause 9.3.3.2, the intermediate stiffener is needed for this 20m span bridge. For flanges, the criteria for an unstiffened flanges in compression satisfied the ratio given in clause 9.3.2.1, hence, the flanges outstand does not need any stiffener.

Design of Stiffener

Assuming spacing = 18t = 2880 mm

Number of stiffener needed = 20000/2880 = 6.944 ≈ 7

Actual spacing = 20000/7 = 2857 ≈2860 mm

<u>End bearing stiffener</u>

Limiting width of stiffener = 242 mm

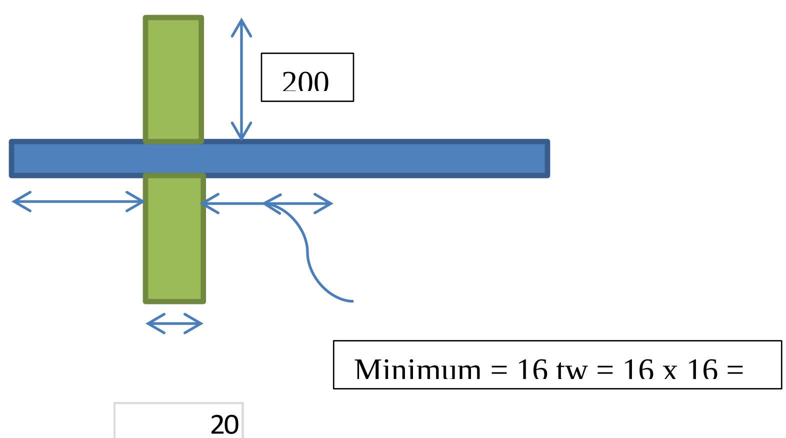
Maximum shear force = 2360.383872 kN

Permissible shear force = 0.75 (σ_y) = 255 N/mm^2

Bearing area required = 2360.383872 x 10^3/ 255 = 9256.407341 mm^2

Trying flat section of **200 mm** width stiffener on both side of the web

Thickness of the stiffener = 9256.407341 mm^2/ 200 (2) mm = 23.14101835 mm ≈ 20 mm



Area of the stiffener = 2 (200) (20) + 2 (256) (16) = 16192 mm^2

Ixx = 2 [(20 x 200^3) / 12 + 20 x 200 x (200 / 2 + 16 / 2)^2] = 119978667 mm^4

Check $\frac{hs}{ts}\left(\sqrt{\frac{\sigma ys}{355}}\right)$ < 10, ok. (cl 9.3.4.1.2)

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2.15952 \text{ x } 109}{26192}}$ = 86.07990283 mm

Effective length = 0.7(200) = 140 mm

From figure 37,

The ultimate compressive stress σ_c = 340 N/mm^2

Resistance capacity of the stiffener = 340 (16192) = 5505280 N > 2360.383872 kN

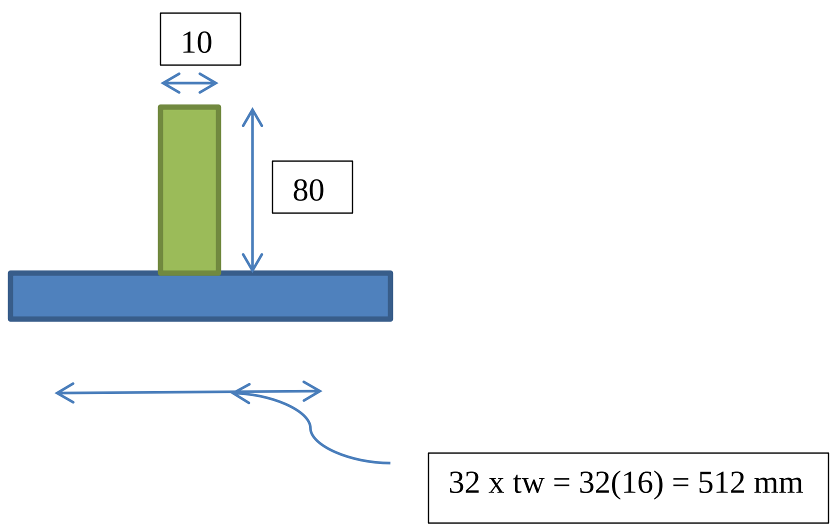
ok

Intermediate stiffener

Axial stress in the web = 2360.383872 x 10^3 N / 1940 x 16 mm^2 = 143 N/mm^2

Trying 80 x 10 mm flat section stiffener

Check: $\frac{hs}{ts}\left(\sqrt{\frac{\sigma ys}{355}}\right)$ < 10 ok



Ixx = (10 x 80^3/12) x (10)(80) x 40^2 = 1285333.33 mm^4

Area = 8992 mm^2

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1285333.33}{8992}} = 13.776$

Effective length of stiffener = 0.7 (80) =56

From Figure 37,

σ_c = 340 N/mm^2 > 76.05329 N/mm^2 ok

The resistance capacity of the stiffener = 3057280 N

Design of lateral bracing

Spacing of the girder = 2000 mm

Wind load for this example is calculated based on the clause 5.3 in BS5400:2

Wind pressure calculated = 1527.6 N/m^2

Depth of the train = 3.5 m

Depth of girder and track = 2.5 m

Wind pressure on train = 1527.6 N/m^2 x 3.5 = 5346 N/m

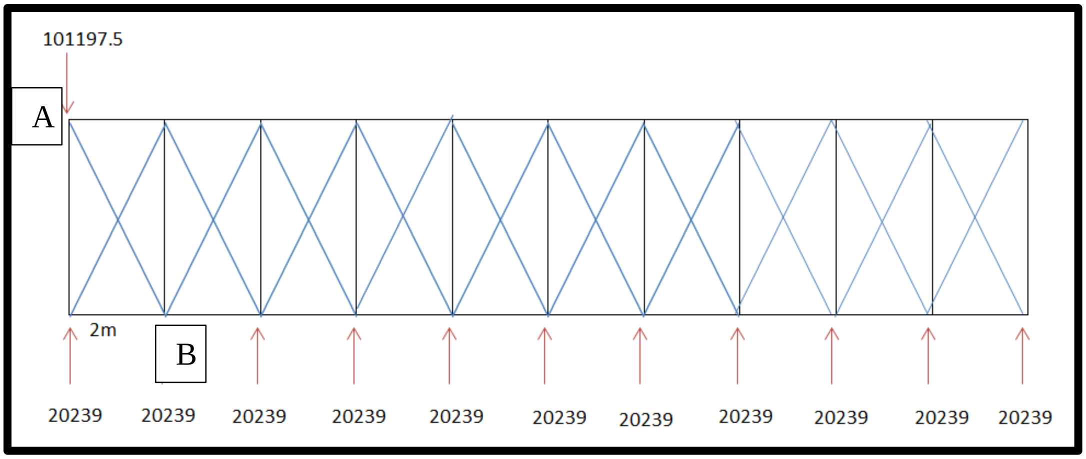
Wind pressure on windward girder = 1537.6 N/m^{2}x 2.5 = 3819 N/m

Wind pressure on leeward girder = 1537.6 N/m^2 x 2.5 x 0.25 = 954.75 N/m

Total wind force = 10119.75 N/m

Lateral load at each node = 10119.75 x 2m = 20239.5 N

At end reaction = 10119.75 x 20m/2 = 101197.5 N



<u>Top Lateral Bracing</u>

Length of the lateral = $\sqrt{2^2 + 2^2}$ = 2.828 m

Force in end lateral = (101197.5 – 20239) x (2.828/2) = 114474.612 N

AB will be designed as a tension member

Permissible stress in tension = 0.6 x 340 = 204 N/mm^2

Try an equal angle of 80x80x10

A = 1511 mm^2 ; r = 24.4

l/r = 2828 / 24.4 = 115.9

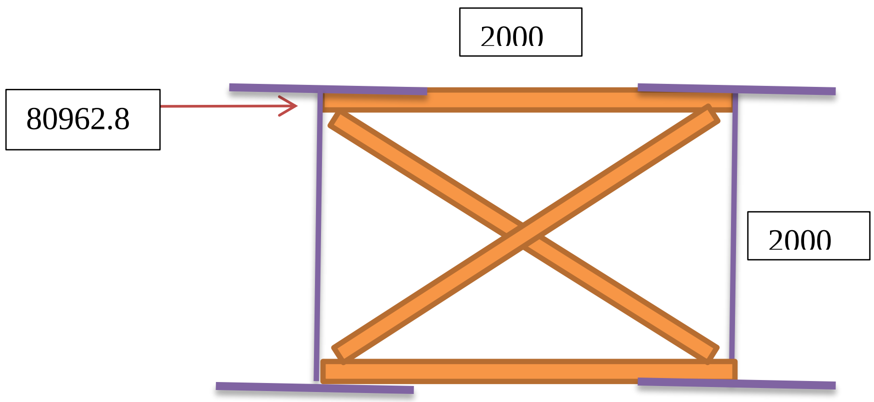
$$Ae = k1k2 \text{ x } At$$

k1	1
k2	1
At	1500
Ae	1500

Load Carrying Capacity = 204 N/mm^2 x 1500 = 306000 N ok

In this case, only the diagonal in tension will be affected. The force in the diagonal lateral bracing at AB will be maximum and also equal, the section required is designed and provided for other diagonal

End cross frame



Top strut

Force, F = 80962.79593

Effective length of the top strut = 0.7(2000) = 1400mm

Using equal angle of 80 x 80 x 6

A = 935 mm^2 ; r = 24.4

l/r = 1400 / 24.4 = 57.37

From figure 37,

Ultimate compressive stress σ_c = 289 N/mm^2

Resistance capacity of the angle = 289 N/mm^2 x 935 mm^2 = 270215 N > 80963 Nok

Bottom strut

Force = 20240.69898 N

Using 60 x 60 x 10

Area = 1111 mm^2

r = 8

l/r = 175

$$\boxed{\frac{l}{r}\sqrt{\frac{\sigma y}{355}} = 171}$$

From figure 37,

Ultimate compressive stress sc = 64.6 N/mm2

Resistance capacity of the angle = 71770.6 N ok